Proceedings of the 2010 Coal Operators' Conference

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FOREWORD

The tenth Underground Coal Operators Conference, (Coal 2010), has been organised jointly between the University of Wollongong-Mining Engineering Group, The Illawarra Branch of the Australasian Institute of the Mining and Metallurgy and The mine Managers Association of Australia.

The first Coal Operators’ Conference was held at the University of Wollongong in 1998, and after two years it began once aging in 2001, as a gathering of a select group of people focusing on the Australian Coal Mining Industry operations and achievements. The 2001 colloquium was a one day event, which was held as a tribute to late Dr Alan Hargreaves, for his pioneering role in both gas and geotechnics in the Australian Coal Mining Industry. There were a total of 11 papers with some 75 participants. A year later in February 2002, a two day conference was held at the University of Wollongong, this time as a tribute to Dr Ripu Lama for his expertise in geotechnology and outburst management. There were a total of 22 presentations made over the two day conference which was preceded by a half day seminar on mine gas and outburst management. This event continued in Wollongong in 2003 and 2004. In 2005 the venue of the conference was moved to Brisbane and some 40 papers were presented during three days conference. At that conference the delegates were provided with a CD in addition to the conference proceedings. The CD also contained papers from the past four conferences. In 2006, the conference was held in Wollongong and there was no conference held in 2007. Since 2008 this has become an annual event held at Wollongong. The decision to hold the conference annually with Wollongong as the venue was based on a survey conducted in 2008.

In 2009 all conference proceedings went online via the University of Wollongong online publications http://ro.uow.edu.au/coal, thus demonstrating the conferences main objective to serve the mining fraternity especially in Australia and now worldwide.

The success and sustainability of the conference has been attributed to the increasing numbers of mine operators, engineers, researchers and the like who participate. Thanks should go to the dedicated authors who are so willing to contribute and maintain the conference’s high standard through excellent quality papers.

Many companies and organisations have in the past generously sponsored the coal operators’ conference series, and this year several companies are, once again, providing sponsorship to Coal 2010. The sponsors support is very much welcomed and is a vital factor in keeping the conference registration at an affordable rate.

We would like to express our sincere thanks to:

- The organizing committee members for their diligence and hard work in making this conference a success,
- The authors of the papers, who have taken considerable time and effort in the preparation of their papers.
- The reviewers of the papers, which at times has not been an easy task, but ensured the high standards being maintained,
- The staff of the University of Wollongong Conference and Functions Centre for the management and registration of the conference.
- Leonie McIntyre of the Faculty of Engineering, University of Wollongong for type setting the conference proceedings,
- Bruce Robertson for audio-visual management of the conference venue,
- Staff of The Wollongong University Printery for printing the conference proceedings, and to Tristan Dus, South Coast Graphics, for designing the proceedings covers and Bruce Robertson of the University of Wollongong, audio visual services for assistance.

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

The Australasian Institute of Mining and Metallurgy – Illawarra Branch is extremely pleased to welcomes delegates to this, the 10th Australian Underground Coal Operators Conference. Going through the range of papers that have been previously presented at Coal Operators Conferences and those that are to be presented here, illustrates the value that these events have been to the industry and for the profession over time. There are many examples of coal industry developments that have been initiated, or first developed through the Underground Coal Operators Conferences.

The Illawarra region and the Illawarra AusIMM Branch have a long sustained, continuing history of active involvement and support for the coal industry.

- Conferences and Symposia on coal issues have been organised by the Illawarra Branch since the 1950’s.
- The Australian Mining Hall of Fame includes leaders from the Illawarra Coal industry.
- The well used, industry reference, “Australasian Coal Practices” was initiated in the Illawarra and the latest edition recently revised by members of the Illawarra AusIMM.
- Similarly, the History of Australian Coal Mining, another AusIMM Monograph has been prepared and edited by Illawarra Branch members.
- Recently, a DVD showing the early history of the Illawarra coal industry, “Beneath Black Skies” was completed, with input and support from the AusIMM Illawarra Branch Heritage Sub-Committee, BHP-Billiton- Illawarra Coal and Gujarat NRE.

Congratulations to the authors of papers for the conference. Your willingness to share information is appreciated and significantly contributes to continuous improvement in practices, impacting not only on the industry, but also on national well-being, since the coal industry is currently a major driver of national prosperity.

A big thank you to the sponsors and exhibitors. Without their support, it would be difficult to stage these annual events in the manner that they are held. Please take the time to visit the exhibits and provide feedback about the products and services your enterprise may be looking for in the future.

As the Chair of the Illawarra Branch of the Australasian Institute of Mining and Metallurgy I welcome you to the Illawarra and to Coal 2010.

Ray Tolhurst
Chair, Illawarra Branch
AusIMM
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INVITED PAPER
THE ROLE OF ACARP IN SUPPORTING AUSTRALIAN COAL RESEARCH

Bruce Robertson

ABSTRACT: The Australian coal industry is significant in many ways and is serviced by a number of research activities, but it is unique, globally, in relation to the Australian Coal Industry Research Program (ACARP) – a Research & Development (R&D) investment program funded by a 5c per saleable tonne levy and owned and managed by representatives of the industry.

ACARP has been in operation since 1992 and currently supports research activities into the safety, sustainable production and marketing of coal (but excluding the sustainable use of coal). With an expenditure of about A$15 million per year, the program supports a critical mass of R&D activities covering issues determined to be of interest to coal producers and other key stakeholders. Outcomes of ACARP include new knowledge and technology, and importantly, an informed, representative and cohesive forum for identifying and responding to critical technical issues of importance for the coal industry. This role of ACARP in the Australian industry is described with particular emphasis on the underground sector.

INTRODUCTION

There are a number of technical, sustainability and competition challenges facing the Australian coal industry. A strong and focused research program is a critical element of any significant industry in order to survive and prosper. The coal industry in Australia is served by a number of research providers and initiatives including ACARP (the Australian Coal Industry Research Program), which is somewhat unique in the way that it is funded and managed. This paper examines the way that ACARP operates in supporting R&D that contributes to the sustainability of the coal industry.

Significance of the Industry

The size and significance of the Australian coal industry is well understood by its participants. It is the fourth largest producer of black coal behind China, USA and India, producing about 334 million tonnes of saleable coal in 2008-09 (ABARE, 2009). As the world's largest coal exporter, Australia generated A$55 billion of income for the nation last year. There are about 30,000 people directly employed in coal mining and an estimated additional 100,000 who derive a livelihood from servicing the industry.

Growth outlook

The last few years have seen turbulent economic times, but the outlook for Australian coal remains positive. Global demand for coal is expected to increase, and Australia should be a beneficiary, notwithstanding impacts from climate change policies and increased competition from other significant producers such as Indonesia and China. This is because Australia has abundant reserves of high quality coal, a reliable sovereign risk profile, strong competition between producers and a demonstrated capacity for development and growth. The biggest risks lie in port and rail constraints, increasing license to operate restrictions and possible inability to service projected labour requirements.

The critical trade contribution that the coal industry (and other resource sectors) makes to Australia's economy is well understood by government, as reflected by supportive policies implemented in the past. However, given the relatively small percentage of the Australian population directly engaged in the industry, pressure from groups opposed to coal mining is having increasing impact. This necessitates cohesive, informed responses from industry members to ensure that appropriate decisions can be made regarding coal's future in this country.

1 Anglo American Metallurgical Coal, Brisbane
Role of ACARP

The original agenda for ACARP was biased towards productivity and cost reduction but there has since been a steadfast shift through safety towards license to operate issues. In this regard, ACARP plays an important role, not only in identifying and characterising the issues to be addressed but also in targeting and managing the R&D to generate the required information. Whilst individual producers can, and do establish their own databases to support the sustainability of their business interests, there is an increasing reliance on material developed by ACARP-funded projects because it is independent, representative and available. This is particularly relevant for safety, environment and community issues.

Some examples of ACARP projects delivering important data to aid decision-making are:

- subsidence datasets and methodologies developed over many years in the Illawarra which are used to inform mining consent applications;
- empirical databases of geotechnical design and response cases underpinning nationally accepted design criteria for ground support stability assessment in underground workings.

THE COAL RESEARCH ENVIRONMENT

Research Community

As one would expect for a leading national industry, there is a strong researcher community servicing coal. CSIRO has for many years conducted R&D in the minerals sector, and particularly coal, through a number of divisions, and continues to be the leading research provider. CSIRO works closely with industry members to ensure that its programs meet industry needs as well as serving the national agenda. University-based research is also significant in coal, (including the University of Wollongong hosting this conference) with postgraduate programs and research infrastructure augmenting tertiary minerals education. Several Cooperative Research Centres were established specifically to consolidate and focus research capacity on issues impacting on mining and demonstrated commitment by the Commonwealth to supporting the mining industry. Private technology companies, semi-Government agencies and consultants make up the balance of Australia’s coal mining research community. ACARP works closely with all of these research providers.

Australia has been blessed with some world class engineers and scientists working in coal research. Concerns exist however about the future supply of researchers, especially in some critical fields. Perceptions in the community about the image and outlook for coal do have an influence on career choices for young people.

How ACARP started

ACARP was established in 1992 (commencing operations in July 1993) when industry members successfully lobbied the Federal Government to transfer control from the existing National Energy Research and Development Program (NERDDP). NERDDP had been established in 1977 (The Coal Levy Act) to ensure an ongoing commitment to industry coal research through a levy fund, matched by Federal funds. Project selection and expenditure was recommended by two committees, covering production and coal use, and made up of representatives of government, researchers and producers and a Commonwealth secretariat.

NERDDP was successful in stimulating new research activities but attracted mixed responses as to the quality and relevance of some of the program content. Under an MOU with the Commonwealth, the black coal industry assumed full ownership of the administration and decision making of the research program, pitched at a 5c per tonne levy rate, over a trial three year period. Strategies were developed to address the perceived shortcomings of NERDDP and ACARP was established (under the Coal Research Amendment Act) with the following purpose:

“To provide for the establishment of an industry research arrangement...... designed to provide for collective and integrated research on coal for the purpose of:

- providing strategic leadership to industry R&D and to act as a catalyst to stimulate R&D interest within the coal and associated industries;
It is clear that ACARP was established to be much more than a research funding agency.

The establishment of ACARP resulted in an increase in industry ownership of the research program to meet its needs. Australian Coal Research Limited (ACR) was established to manage the program. Under Board and research committee governance, ACARP has been subjected to ongoing review and discussion to improve its performance and deliver maximum value for its levy payers. This has largely been via a process of five year extensions and associated Business Plans, with the latest extension recently approved from June 2010 to June 2015. The basic administration structure has not changed significantly since it started, but the program content has changed with industry’s needs and there has been a concerted effort to increase awareness and engagement.

One of the more challenging aspects of ACARP is to maintain compliance with a critical clause of the Commonwealth agreement – 100% participation by all black coal producers. From time to time, there are some industry members, who for various reasons have a view that paying a 5c per tonne levy is not an optimal allocation of investment funds for their business. Sometimes this is born out of lack of understanding of the value proposition for ACARP and sometimes it is a consequence of a particular management style. The case has been successfully put to such minority interests that ACARP participation is for the overall benefit of each member and those benefits can be maximised through collaborative participation in the ACARP processes.

How ACARP works

The vision and mission of ACARP are as follows:

**Vision:**
To assist the Australian coal industry develop and adopt world-leading sustainable mining practices and, through collaboration, to ensure a sustainable position for the global use of coal.

**Mission:**
Utilise the collective technical competence and resources of the Australian coal industry to develop and manage a comprehensive research program which, through technological and process innovation assists coal producers achieve their financial, environmental and social objectives for sustainable development.

An underpinning element of the effectiveness of ACARP is the committee structure that draws expertise and representation from all sectors of the industry. Figure 1 shows the committee structure and highlights the division of technical work. Some I task groups support both underground and open cut committees. There are currently over 140 industry experts making voluntary contributions of their time to ACARP. This high level of participation has the added advantage of improving awareness and uptake of research outcomes.
Coordination is provided by the Executive Director of ACR Ltd and administration services are contracted to ACR by Australian Research Administration Pty Ltd (ARA). The key accountabilities of the different committees are illustrated in Figure 2. Recently there have been concerted efforts to strengthen the quality of communication between:

- Task groups and technical committees;
- Research committee and Board; and,
- ACARP and its stakeholders.

**ACARP RESEARCH STRATEGY**

**Research focus**

ACARP research has in the past funded projects addressing both the sustainable production and use of coal. To support a national effort to reduce greenhouse emissions from coal, ACARP has participated in three cooperative research centres - CCSD, CO2CRC and CLET. In order to generate funding for abatement demonstration programs, ACALET (ACA low emissions technologies fund) was formed in 2007 and the responsibility for funding R&D in sustainable coal use was taken over from ACARP. The focus for ACARP research is now on people, productivity and the environment.
ACARP committees endeavour to maintain a balance of funding for ongoing programs and emerging areas of importance. There is a view that the size of typical ACARP grants (A$ 150,000 – 200,000) is too small to attract a critical mass research team or to make a significant impact. In reality, projects usually make up a continuum of work in a program area characterised by an expanding body of knowledge whose direction and application can be more effectively managed by industry. The coal industry is conservative in nature and in a number of areas, ACARP funded research leads industry practice by a number of years.

**Funding model**

The ACARP funding model is an annual round process, although critical projects can be brought forward for consideration at any time (Figure 3). There also exists a “Landmark” concept for funding more significant programs of work; examples include the Longwall Automation and Roadway Development programs and ACARP’s membership of CCSD, CLET and CO2CRC to ensure that levy payers have access to the work of these centres.

Each year, the various technical committees establish new R&D priorities and develop funding requests contingent with their proposed programs of work. The philosophy on funding is to achieve an efficient and equitable investment process with leveraged funds where appropriate. There is an element of competition for some of the funds, again managed by the Research Committee. Whilst there will always be winners and losers, in general there appears to be a healthy quantum of tension between committees for funding, with the best and most important projects receiving funding. Projects are closely managed by industry monitors appointed by the committees to ensure R&D performance expectations are met.

On average about $12 million is committed to R&D annually plus an additional 15% in administration costs. Since its start, ACARP has directed A$180 million of funding to 1,084 projects (ACARP, 2009). In 2009, 230 short proposals were received, which were subsequently short-listed to 86 long proposals, 12 of which were fast tracked. Another 63 projects were finally put forward for funding.
Figure 3 - ACARP funding timetable

Figure 4 shows the distribution of direct ACARP funds, amounting to A$36 million over the last three years. A further A$61 million of external funds and in-kind resources resulted in a leverage factor of 2.7 times investment. Underground projects have attracted some 34% of the pool, which has been expanded by the wind down of funding for the low emissions coal use committee since 2008.

Underground Priorities

The strategy for the ACARP underground program is expressed as:

‘increasing levels of safe reliable underground production through the application of automation technologies and improved management of risk and impacts on people and the environment’

Priority setting is a key aspect of ACARP’s strategy to ensure that R&D matches industry’s needs. Priorities are overhauled each year to take into account industry circumstances, outcomes from previous research programs and input from industry members. Priorities are clustered into strategic areas, which for the underground technical committee for 2009 were:

- Improved Health & Safety;
- Management of Mining Conditions;
• Higher Productivity Mining;
• Equipment and Mining Systems Reliability;
• Sustainability.

Within these categories, guidance is provided for researchers on the issues that technical committee members (and the relevant task groups) agree are most important. Program themes are continued as the body of knowledge is developed with each project. Table 1 summarises the underground program areas supported in recent times (ACARP, 2009).

Table 1 - 2009 Underground Priority programs

<table>
<thead>
<tr>
<th>Program</th>
<th>Strategies</th>
<th>Focus</th>
</tr>
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<tbody>
<tr>
<td>Improved health and safety</td>
<td>Reduced operator exposure to hazards, fires &amp; explosions and health management tools; escape capability</td>
<td>• safety &amp; risk mgt systems;</td>
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<td>• dust; noise exposure;</td>
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<td>• fatigue; vibration;</td>
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<td>• ergonomics; collision avoidance;</td>
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<td>• high pressure fluids safety;</td>
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<td>• diesel emissions;</td>
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<td>• Ventilation, gas &amp; outbursts;</td>
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<td>• spontaneous combustion, fires and explosions;</td>
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<td>• emergency management, escape &amp; rescue</td>
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<td>Management of Mining Conditions</td>
<td>Better exploration methods; Strata characterisation and design tools</td>
<td>• exploration techniques; detecting geological anomalies;</td>
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<td>• resources and reserves estimation;</td>
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<td>• geotechnical characterisation of strata;</td>
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<td>• improve ground support technologies;</td>
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<td>• windblast management</td>
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<tr>
<td>Higher Productivity Mining</td>
<td>Improved underground mining methods &amp; equipment; automation, training</td>
<td>• roadway development;</td>
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<td>• longwall;</td>
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<td>• remote control &amp; automation;</td>
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<td>• improved blasting systems;</td>
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<td></td>
<td></td>
<td>• training systems</td>
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<tr>
<td>Equipment systems and reliability</td>
<td>Root cause analysis, increased asset utilisation</td>
<td>• improve uptime;</td>
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<td></td>
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<td>• conveyors; transport;</td>
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<td>• electrical power systems;</td>
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<td></td>
<td></td>
<td>• intrinsic safety and flameproof protection</td>
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<tr>
<td>Sustainability</td>
<td>Knowledge building, assessment tools, case studies &amp; best practice</td>
<td>• subsidence management;</td>
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<td>• aquifer management;</td>
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<td>• streams; vegetation; biodiversity;</td>
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<td>• fugitive emissions;</td>
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<td>• energy efficiency</td>
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Future focus

On occasions, a “blue sky” project is funded if the opportunity is compelling. The underground technical committee has demonstrated a desire to recommend major investment in “step change” technology areas which are seen as pivotal to the industry’s future, such as longwall automation and roadway development systems. Scoping studies are routinely commissioned to clarify the status of knowledge in
a topic or to define research goals. The L15 study completed in 2007 had the aim of identifying barriers to a single longwall producing 15 Mtpa.

The critical issues identified were:

- High capacity gas drainage;
- Increased roadway development performance;
- Engineering design of system elements;
- Real time maintenance;
- OHS issues – dust, heat, fatigue;
- Prediction of adverse geological & geotechnical conditions;
- Subsidence & aquifer disturbance;
- Water efficiency.

**ACARP ADDING VALUE**

**Measuring value**

An issue of interest to levy payers is the value that their levy generates. For some contributors, 5c per tonne amounts to a considerable sum of money which could otherwise be invested elsewhere. The portfolio of projects funded by ACARP creates different value drivers including productivity and cost savings initiatives, health and safety improvements and environmental safeguards, each with different value measures. Perhaps more significantly, the value generated from research outcomes depends on the quantum and quality of uptake as much as the research product itself. The challenge of articulating a value stream that represents an acceptable return on investment has been dealt with in several ways:

- Selecting a subset of projects where there has been a clear financial benefit developed for the industry (eg longwall automation) and estimating the net present value of the research projects;
- Showcasing a number of significant projects where the value is clearly significant but difficult to quantify (eg EMESRT – the Earth Moving Equipment Safety Round Table);
- Harnessing the collective opinions of the technical leaders of the industry to generate a qualitative view of the gross worth of the ACARP program.

This highlights perhaps the key strength of an industry R&D program such as ACARP in providing a vehicle to conduct research on the industry’s behalf. Important cross sectoral issues such as cumulative impact of mining, subsidence impact or sensitive legislative matters can be more appropriately funded and managed at the industry level rather than at company level. A good example is the current program of work being managed by the Fugitive Emissions Steering Committee to develop draft standards for emissions measurement and reporting. ACARP has been able to respond quickly to the need for funding and industry representation to keep ahead of the Government regulatory agenda.

**Leveraging value**

In most cases, ACARP projects include funding from the researcher or other parties in addition to ACARP money. The leverage rate over the last three years has been 2.7 times ACARP investment, expanding the resources applied to coal research to about A$30 million per year. There is no doubt that ACARP funding has provided an important core of support for several R&D entities in Australia, allowing such organisations to secure essential researchers and infrastructure for specific programs. ACARP projects normally have a maximum limitation of three years and it has been suggested that this should be lengthened to provide even greater security for staff retention. This has not been possible in the past due to the limited forward tenure of the ACARP contract with levy payers. However, now that ACARP is secure into at least its 23rd year in 2015, it has become a stabilising funding influence.
Level of investment

It will come as no surprise that the topic of research levy quantum stimulates debate when raised. Five cents represents somewhere between 0.01% and 0.1% of revenue, depending on coal price. After adding leverage of 2-3 times and bearing in mind significant R&D funding from other sources, the total level of investment still falls well short of typical industry investment rates of, say 3 - 5%. There are of course many suitable research opportunities in coal. A significant increase in ACARP funding would test research capacity in Australia, although the CRC program has demonstrated that resources can be marshalled if required. ACARP grants have largely been directed to upstream research and development, rather than downstream demonstration and commercialisation, which requires significantly more funds and time. To date there has been a reliance on OEMs and individual mining companies to take over the pre-commercial development of ACARP outcomes, which takes time and can slow uptake. For some critical breakthrough technologies, a coordinated effort with ACARP acting as a facilitating agency has been effective.

Facilitating collaboration

Perhaps one of ACARP’s most valuable advantages is the opportunity that the program creates to get capable people together to solve common problems. This is not easy in a competitive business environment with onerous trade practices constraints, so it is useful to acknowledge some of the successful collaboration activities made possible under ACARP:

- regular practitioners workshops to disseminate progress updates on ACARP projects and capture vital intelligence from operators to feed back into R&D efforts (eg, the inseam drilling and outburst workshops coordinated by John Hanes for many years, and the recent series of roadway development workshops coordinated by Gary Gibson);
- research projects requiring participation by a number of different industry members to develop consensus decisions, sufficient funding or multiple site access, and thus providing opportunities for rich interaction and learning; (eg greenhouse measurement standards development, geotechnical and environmental projects and the longwall automation program).

The technical people that sit on ACARP committees represent, in many instances, ideal candidates to represent the industry’s interests on Standards committees. The role of ACARP in the Standards process, as it relates to coal, has come under the spotlight recently. This is in response to significant changes taking place around Standards development and because there are few mechanisms other than ACARP to access collective industry expertise.

Tackling difficult topics

Some of the research programs carried out in ACARP address topics that are more effectively dealt with at an industry rather than enterprise level - typically regulatory issues. An ACARP-supported approach allows engagement between a cross section of stakeholders, increasing the quality of participation and maximising the likelihood of a successful outcome. In many instances, individual companies might have been reluctant to take on the responsibility of leading a reform agenda. Some examples of this process at work include:

- the EMESRT program to develop industry engagement between operators and OEMs;
- research, development and demonstration of new escape and rescue systems (eg development of self rescue vehicle, new fire and ventilation modelling technologies);
- various mine safety research programs (eg stone dust improvements, intrinsic safety testing, coal blasting products, inertisation)

Over the years, a significant body of knowledge has been built up in a number of programs as a result of research strategies managed by technical committees. The process of progressively developing incremental understanding and research outcomes through the stepwise interaction between researchers, project monitors and the sponsoring technical committee has in most cases been very effective. Example programs include management of spontaneous combustion and gases in goafs, longwall geomechanics and dust control.
OEM engagement

The major original equipment manufacturers servicing the underground industry operate in a highly concentrated and competitive environment. This has contributed to a slow evolution of mining equipment in the eyes of some producers, and frustration for researchers looking to develop their new technology. The experiences with commercialising LASC (Longwall Automation Steering Committee) technology and the EMESRT program have demonstrated that a coordinated industry effort (in these cases under ACARP) can make an impact that benefits all parties, if managed effectively. In reality, these initiatives provided market research and communication vehicles that identified what industry really needed.

Fostering research capacity

Bearing in mind the need to maintain a steady supply of new research personnel, ACARP offers post graduate scholarships, targeting industry staff with an interest in developing higher qualifications through research in a particular topic area. The scheme was introduced in 2003 and to date five scholarships have been awarded, all for PhD study. In addition, many of the research projects funded by ACARP include postgraduate or post doctorate researchers.

Technical committees tend to push back on research applications with significant funds directed towards asset building, but inevitably projects contribute to some consolidation of research facilities. Where appropriate, collaboration is encouraged between research organisations to expand the resource capacity available to the project.

Communicating Outcomes and Competency Development

Over the last two years ACARP has introduced a number of communications initiatives to increase the uptake of research outcomes. This was in response to criticism from some quarters that there was a lack of awareness of ACARP activities, results and value. The initiatives included:

- Streamlining the ACARP website and simplifying the means by which industry members can obtain project reports via downloads over the www; the uptake of reports has since skyrocketed;
- Maintaining the frequency and intensity of workshops in critical program areas such as Roadway Development to distribute topical information as well as gather input from industry members about future priorities;
- “ACARP Matters” bulletins to showcase new research outcomes concisely;
- Releasing short summaries of research findings via email to target recipients (including external stakeholders) as alerts;
- Upgrading the quality and content of annual reports;
- More recently, all NERDDP reports have been made available in digital form to complement the ACARP library of reports.

The ACARP website allows easy access to hundreds of ACARP and NERDDP reports. Furthermore, some information portals, developed with ACARP support provide ongoing information databases of interest to the industry (e.g. MIRMgate, undergroundcoal.com.au website). There are numerous opportunities for professional development of industry members who participate in ACARP research projects at mine sites, attend seminars or read research reports. In most instances the knowledge transferred in these events is interesting, topical and relevant to the competency development of individuals and teams. The barriers of access to ACARP knowledge have been purposefully lowered to enhance knowledge transfer.
CONCLUSIONS

Inevitably, the business of mining, processing and marketing coal in Australia will continue to evolve in response to different factors and forces, including:

- challenges from other coal-producing nations and sectors (renewable energy, nuclear, alternative steel making methods) that will put pressure on costs and reliability for Australian coal to remain competitive;
- new research issues, as existing mining districts are depleted and new areas are opened up; and,
- increased pressure on sustainability programs (GHG, environmental).

The role that R&D will play in the management response to these issues will depend on the structure and cultures within the industry and the ability to organise and fund research activities. ACARP has successfully demonstrated that an industry-based research program can be effective in delivering solutions to difficult problems, in a cost-effective way. In providing a reliable ongoing R&D agenda, capacity and activities, somewhat isolated from business cycles, the program provides a forum for discussion and collaboration which is more likely to achieve progress than uncoordinated company actions. In addition to the examples previously cited, the successful bioremediation trial with acid lakes at Collinsville is a good case study of how ACARP projects can deliver solutions to difficult industry problems.

It is expected therefore that ACARP will remain the central mechanism for coordinated coal research for some years into the future. It is also likely that collaboration with international agencies will increase in response to shortage of technical expertise and facilities and the desire to find common solutions to similar problems. Ultimately ACARP’s future effectiveness will depend on the attitude and preparedness of coal producers to embrace innovation, new technology and risk taking to achieve solutions to significant problems.

REFERENCES


GLOBAL TRENDS IN COAL MINE HORIZONTAL STRESS MEASUREMENTS

Christopher Mark¹ and Murali Gadde²

ABSTRACT: Knowledge of in situ stresses is fundamental to many studies in earth sciences, and coal mine ground control is no exception. During the past 20 years, it has become clear that horizontal stress is a critical factor affecting roof stability in underground coal mines. The theory of plate tectonics and the World Stress Map (WSM) project has been extremely helpful in explaining the sources and the orientations of the horizontal stresses observed underground. Recently, WSM geophysicists studying deep-seated stresses have developed a model of how stress magnitudes vary with depth in the crust. They have devoted relatively little attention to near-surface stresses, however. This paper explores the relationships between deep-seated and shallow in situ stresses in several of the world’s coalfields, using a data base of more than 350 stress measurements from underground coal mines. The analysis indicates that distinct regional trends exist, corresponding roughly to the regional stress fields identified by the WSM. The paper presents equations for estimating stress magnitudes that were developed by treating depth and elastic modulus as independent variables in regression analysis. The magnitude of the horizontal stress increases with depth, at rates that range from 0.8 to 2.0 times the vertical stress, just as the WSM “critically stressed crust” model predicts. Overall, it seems that the stress regimes encountered in underground coal mines are closely linked to those that exist deep in the earth’s crust.

INTRODUCTION

As early as the 1940’s, researchers in British coal mines postulated that large horizontal stresses were responsible for much of the roof damage experienced underground. Philips, (1946) observed that “At depths greater than 215 m…lateral compressive forces cause fracturing along the laminations of the roof beds…the lateral compressive forces increase at a greater rate than the vertical compressive force, and ultimately both forces may be equal.” Once rock mechanics researchers began to measure in situ stresses, it became clear that in many cases the horizontal stress actually exceeded the vertical, often by factors of three or more (Dahl and Parsons, 1972; Hoek and Brown, 1978).

A number of theories were proposed to explain the presence of horizontal stress. Two of the earliest were the “Poisson’s effect” and the “lithostatic stress state.” Both of these theories presumed a static earth, in which the horizontal stresses were generated in response to the vertical overburden load. The “Poisson’s effect” model predicted that the horizontal stresses should be about 1/3 of the vertical, while the lithostatic model predicted that the three principal stresses would be approximately equal (McGarr, 1988).

These static earth theories could not explain two key characteristics of horizontal stress:

- Why the horizontal stress often exceeds the vertical ($S_v$) in magnitude, even at depth, and;
- Why horizontal stresses are typically highly anisotropic, with the major one ($S_{Hmax}$) significantly larger than the minor ($S_{Hmin}$).

In addition, the Poisson’s effect model suffers from severe theoretical errors, because it implicitly assumes that soft sediment (or magma) lithifies in the absence of gravity, and then gravity is instantaneously switched on once the rock has reached an elastic state McGarr, 1988; Zoback and

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As Hoek (2007) writes, the Poisson’s effect model was “widely used in the early days of rock mechanics, but proved to be inaccurate and is seldom used today.”

Fortunately, during the 1970’s earth scientists were constructing a revolutionary new theory with the breadth and depth to explain all the major tectonic processes observed on the earth. Plate tectonics describes a dynamic earth, in which the crust of the earth consists of a number of continental plates that are sliding across the softer rock in the mantle below. Where the plates contact each other, their different directions of relative movement create large forces that are transmitted across the plate interiors. The scientists associated with the world stress map (WSM) have used a variety of indicators, including earthquake focal mechanisms, wellbore breakouts, and hydraulic fracturing stress measurements, to identify the lithospheric stresses that result from these plate movements. They found that the state of stress is remarkably uniform over vast regions of plate interiors, and that it is due to present-day forces, and not due to residual stresses from past tectonic activity (Zoback and Zoback, 2007). Today there is complete consensus within the geophysics community about the general validity of the trends identified by the WSM.

The WSM has found that at any given location, the stress direction within the crust is typically consistent from the “upper 2-5 km (1-3 miles), where essentially all of the wellbore breakout and hydraulic fracturing data come from, down through the lower 5-20 km (3-12 miles), where the majority of crustal earthquakes occur” (Zoback and Zoback, 2002). However, the WSM has specifically excluded “near surface” measurements from its database, because they have sometimes found that there are “marked changes in stress orientations and relative magnitudes with depth in the upper few hundred meters, possibly related to effects of nearby topography or a high degree of surface fracturing” (Zoback, 1992). Later, Zoback and Zoback (2002) stated that “only in situ stress measurements made at depths greater than 100 m are indicative of the tectonic stress field.”

Since most underground coal mining takes place within several hundred meters of the surface, it is legitimate to ask how relevant the WSM is to underground coal mining. On the one hand, we do know that topographic features can significantly affect the stresses we observe underground (Molinda et al., 1991; Hasenfus and Su, 2006). On the other hand, since the near-surface is part of the crust, we would certainly expect some relationship to the deep-seated stress patterns. The question is, how closely are the two related? Do we observe the same general trends that the WSM has identified in the deep crust, or are the topographic and other near-surface effects so powerful that they completely mask any relationship?

The first part of the answer to this question was provided in the early 1990’s when researchers compiled a data base of stress measurements from US coal mines. The WSM had identified the eastern portion of North America as a stable mid-plate region with a consistent ENE horizontal stress orientation (Zoback and Zoback, 1989). Sure enough, analysis indicated that 75% of the coal mine stress measurement orientations fell within the NE quadrant (Mark, 1991; see Figure 1). This finding was particularly meaningful because the data base included measurements made all over the eastern U.S., by a variety of researchers using a number of different techniques. The observed trends were highly significant statistically even though no attempt was made to minimize the effects of bad data by applying a “quality ranking” to the individual measurements. Such quality rankings are normally considered essential for discerning underlying trends in stress measurement data (Zoback, 1992; Stacey and Wesseloo, 1998; Lee et al., 2008).

The WSM also defined the stress regime within eastern North America as either strike/slip (where the magnitude of the vertical stress falls between the two horizontal stresses) or reverse faulting (where the vertical stress is smaller than both the principal horizontal stresses). Here, the US stress measurement data was in even better agreement, with the maximum horizontal stress exceeding the vertical 97% of the time.

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1 The most elegant of the static earth theories is the “spherical shell” model proposed by Sheorey (1994), based in part on earlier work by McCutchen (1982). Sheorey's model considers stresses arising from the geothermal gradient in addition to the Poisson's effect, and it predicts that the horizontal-to-vertical stress ratio may be very large near the surface but declining with depth. Like all static earth theories, however, Sheorey’s model can explain neither the near universal anisotropy of the horizontal stress, nor the prevalence of horizontal stresses that are significantly greater than the vertical deep into the crust.
Figure 1 - World Stress Map of the United States (Zoback and Zoback, 1989) compared with stress orientations determined from coalfield stress measurements (Mark and Mucho, 1994). The solid arrows show WSM stress direction and the dotted lines delineate stress.

**THE WSM AND STRESS ORIENTATIONS IN COALFIELDS AROUND THE WORLD**

Stress measurement data bases have now been constructed for a number of the other of the world’s coalfields. It makes sense to ask how well the predictions of the WSM compare in those cases.

**Western US:** According to the WSM, the coalfields of Utah, Colorado, Wyoming, and New Mexico fall within regions of “extension” or normal faulting, where the vertical stress is predicted to be greater than either horizontal stress (see Figure 1). In contrast to the eastern “midplate” region, the western regions are active seismically. Stress directions also vary throughout the region.

Mark (1991) analysed stress measurements from 17 Western US mines, and found no significant regional trends in orientation. The maximum horizontal stress was significantly lower than in the east, and was approximately equal to the vertical stress in most cases.

**Germany and the UK:** The WSM defines the stress regime in western Europe, including the coalfields of Germany and the UK, as a stable mid-plate area subject to uniform NNW maximum horizontal stress. As in eastern North America, the stress is controlled by plate driving forces acting on the plate boundaries (Muller *et al.*, 1992). Western Europe is considered to be a strike slip environment, with the vertical stress as the intermediate principal stress.

A series of 11 hydrofracturing stress measurements were conducted in four German coal mines between 1989 and 1991 (Muller, 1991). These measurements confirmed that the greatest horizontal stress was oriented NNW. The maximum horizontal stress was reported to exceed the vertical stress down to depths of 1200 m.

Cartwright (1997) describes the results from 26 successful overcores at conducted by Rock Mechanics Technology (RMT) at 16 mine sites in “virgin or near virgin conditions.” The depths of cover ranged from 300-1000 m. In every overcore the maximum horizontal stress was located in the NW quadrant, and the vast majority were oriented within a few degrees of NNW (Figure 2). The magnitude of the stress was approximately equal to the vertical stress, but there was considerable spread.
Figure 2 - World Stress Map of northern Europe (after Muller et al. 1992), compared with stress orientations determined from UK coalfield stress measurements (after Cartwright, 1997)

**Bowen Basin, Queensland, Australia:** Australia is considered somewhat unique by the WSM because the stress orientation varies considerably between different regions of the continent, reflecting a variety of plate boundary forces rather than the direction of absolute plate motion (Hillis et al., 1999). In the Bowen Basin coalfields of the central Queensland, the major horizontal stress is consistently oriented NNE, and the vertical stress is either the minor or the intermediate principal stress. The region is not seismically active, and evaluation of the available stress measurements found that few were indicative of faults on the verge of movement.

Nemcik et al. (2005) presented the results of 235 measurements of pre-mining stress made by SCT in Australian underground mines. About a third of these measurements were conducted in the Bowen Basin, all at depths of less than 300 m. Nemcik at al. (2005) reported that “the direction of the major lateral stress was in most cases confined to the N to NE quadrant” (Figure 3). The magnitudes of these stresses almost always exceeded the vertical, sometimes by factors of 3 or more. Enever and Lee (2000) drew similar conclusions from another set of stress measurement data.

**Sydney Basin, NSW, Australia:** The Sydney Basin appears to be the exception that proves the rule. Early studies of horizontal stress in underground mines there found that stress directions could vary widely, even from one section of a mine to another (Gale et al., 1984; Gale, 1986). No consistent regional trend could be observed. As it turns out, the WSM found the same thing (Hillis et al., 1999). The Sydney Basin is one of Australia’s most seismically active, and stress orientations vary widely throughout it. It seems that plate margin effects superimposed on the regional stress direction have resulted in a relatively low anisotropy between the major and minor principal horizontal stresses. As a result, mild perturbations caused by local effects, such as density contrasts, faults, or major geologic structures may cause local stress rotations (Hillis et al., 1999). Moreover, as many as 40% of the stress measurements are indicative of faults at incipient failure. Nemcik et al. (2005) confirmed that even with a very large data base, consisting of approximately 170 stress measurements from mines all over NSW, no consistent trends in stress direction emerged.
Summary: Hillis (1999) concludes that “the apparent consistency between in situ stress measurements and seismicity of the Bowen and Sydney Basins suggests that relatively shallow (300-1000 m) data may be representative of the stress at greater, seismogenic depth.” Our quick tour around other coalfields of the world leads to the same conclusion, at least with regards to stress orientation and relative magnitude. In every case, the WSM model provided a reasonably accurate prediction of (and explanation for!) the typical stress regime that is observed in underground coal mines. The next question is whether we can also predict the magnitude of the stress using the WSM model.

THE CRITICALLY STRESSED CRUST

The “dynamic earth” plate tectonics model implies that lateral forces are constantly being applied to the brittle upper crust. These forces would continue to build unless there was some mechanism for their release. That mechanism is failure of the crust itself, through faulting. Decades of research along a number of lines of evidence have resulted in what is, in the end, a simple but profound model of the magnitude of the stresses that the crust can carry (Zoback and Zoback, 2002).

The model begins with Anderson’s (1951) classification scheme for relative stress magnitudes in the earth:

- Normal faulting regions, where $S_v > S_{H_{max}} > S_{min}$
- Strike slip faulting regions, where $S_{H_{max}} > S_v > S_{min}$
- Reverse faulting regions, where $S_{H_{max}} > S_{min} > S_v$

Research has shown that the strength of faults can be adequately described by the Coulomb criterion:

$$T = C_o + uS_n$$

Where $T$= the shear strength of the fault plane, $C_o$= the fault plane’s cohesion, $u$= the friction coefficient, and $S_n$=the confining stress applied perpendicular to the fault plane.

1 The discussion in this section is based almost entirely on the summary provided by Zoback and Zoback (2002).
Using two-dimensional Mohr-Coulomb analysis, the shear stress at failure of an optimally-oriented fault is a function of the difference between the minor ($S_3$) and the major principal ($S_1$) stresses (Jaeger and Cook, 1979):

$$\frac{(S_1-P_o)}{(S_3-P_o)} = \frac{((u^2 + 1)^{1/2} + u)^2}{2}$$

(5)

Where $P_o$ is the pore pressure, and $(S-P_o)$ is the effective stress.

Studies have shown that, at depth, the cohesion is much smaller than the frictional component of the fault strength, the friction coefficient $u$ is typically 0.6-1.0, and the pore pressure is hydrostatic on active faults (Townend and Zoback, 2000). Assuming $u=0.6$, the following approximate relationships can be derived:

- $S_{H_{\text{max}}} = 2.3 \ S_v$ in reverse faulting regions; (6a)
- $S_{H_{\text{max}}} = 1.6 \ S_v$ in strike-slip faulting regions (assuming $S_v = (S_{H_{\text{max}}} + S_{H_{\text{min}}})/2$) and; (6b)
- $S_{H_{\text{max}}} < S_v$ and $S_{H_{\text{min}}} = 0.6 \ S_v$ in extension faulting regions. (6c)

Stress measurements have now been conducted in several deep boreholes to depths of almost 10 km (6 miles). Figure 4 shows that the measurements confirm the general stress gradients derived above. In particular, in the mid-plate compressive stress regions where these measurements were made, horizontal stresses well in excess of the vertical persist far down into the crust, and the horizontal stress gradient ($k$) is fairly consistent with a value ranging from approximately 1.3 to 2.

Zoback and Zoback (2002) also state that worldwide research has found “no evidence” that “residual stresses” from past tectonic events play any role in today’s stressfields. They speculate that if such stresses exist at all, they can only be important “in the upper few meters or tens of meters of the crust where the tectonic stresses are small.”

If plate tectonics are responsible for virtually all the stresses measured at depth, and if the critically stressed crust model allows us to predict those stresses, then it is reasonable to expect that there is some relationship between the deep crustal stresses and those measured in coal mines. After all, the “near surface” is part of the same crust! In fact, if we don’t find a relationship, then we have a significant problem. For instance, if we conclude that the horizontal stress increases less rapidly than the vertical stress to depths of one or two thousand feet, but we know it exceeds the vertical stress at greater depths, the implication is that there is a major discontinuity in the stressfield somewhere. Let us then see what the actual measurements tell us.
OVERCORING STRESS MEASUREMENTS IN COAL MINES

Overcoring has been the most common technique for measuring stress in underground coal mines. In the US, most measurements have been made using the Bureau of Mines biaxial “borehole deformation gage” (Bickel, 1993). Internationally, the triaxial ANZI or CSIRO HI cells have been by far the most common (Mills, 1997; Nemcik et al., 2005; Cartwright, 1997: Lee et al., 2008). Most recently, downhole wireline stress measurement devices have been developed (Conover et al., 2004).

One important feature of overcoring stress measurements is that interpretation of the data requires the determination of the rock stiffness (elastic modulus, E). The modulus is not required for the other types of stress measurement contained in the WSM data base. Studies in the layered sedimentary geology of coal measure rock have found that the measured stresses in a single hole vary in proportion to the rock stiffness (Aggson and Mouyard, 1988).

Cartwright (1997) pointed out that the relationships between horizontal stress and depth, like those of Hoek and Brown (1980), have typically displayed high scatter, particularly near the surface. Within his data base of UK stress measurements, there was a better correlation between stress and modulus than between stress and depth. He proposed that the two factors might be combined into a single equation:

\[ S_{H} = B_{0} + B_{1} \left[ \frac{v}{(1+v)} \right] \text{(Depth)} + B_{2} \text{(Modulus)} \]  (7)

Where \( B_{0} \) is a constant with units of MPa, \( B_{1} \) is a constant with units MPa/m, \( v \) is Poisson's ratio, and \( B_{2} \) is a dimensionless constant called the “tectonic strain factor” or TSF. Regression analysis provided the following values for the constants, with an r-squared of 0.94:

- \( B_{0} = -4.0 \) MPa
- \( B_{1} = 0.009 \) MPa/m, and
- \( B_{2} = 0.78 \times 10^{-3} \)

Cartwright's analysis indicated that for his data set, the modulus was more important than the depth for predicting the maximum horizontal stress.

Dolinar (2003) studied stress measurements from 37 eastern US underground mines, including several stone mines. His analysis employed a version of equation (7), with \( B_{0} = 0 \) and \( B_{1} \) fixed at 0.025 MPa/m. He found that the remaining regression coefficient, the TSF (\( B_{2} \)), varied between 400 and 900 for the different geographic regions studied, with the highest TSF values in two small areas in central Appalachia.

Nemcik et al. (2005) also calculated the TSF for the large SCT data base of 235 measurements from Queensland and NSW mines. In their analyses, they also used equation (7), setting \( B_{0} = 0 \) and \( B_{1} = 0.025 \) MPa/m. In contrast to both Dolinar and Cartwright, however, Nemcik found that:

- There was a strong correlation between depth and the \( S_{\text{Hmax}} \) in both NSW and Qld,
- The TSF also increased significantly with depth, averaging 0.4 when the depth was less than 100 m deep, but more than 1.3 for mines at depths exceeding 500 m, and;
- At any given depth, a wide range of TSF values were measured.

Nemcik et al.’s work indicates that the TSF can vary significantly within a single region, and that the TSF cannot always explain a large proportion of the variation in \( S_{\text{Hmax}} \).

INTERNATIONAL OVERCORING MEASUREMENTS DATA SET

If the magnitude of the near-surface stresses measured in coal mines were closely related to the deep-seated stresses measured by the WSM, we might expect to find that:

- The depth at least as important as the modulus in predicting the horizontal stress, though both factors together should be better still;
The depth gradient should be somewhere between 1.0-1.6 times the vertical stress for coalfields located in stable, a-seismic mid-plate areas, like those in the eastern US, the UK, Germany, or central Queensland;

The depth gradient should be higher in a seismically active compressive regime like the one found in the Sydney Basin, and it should be lower an active extension regime like the one found in the western US coalfields.

To test these hypotheses, a data set of 565 stress measurements was compiled. The heart of the data set is 373 measurements from underground coal mines. The breakdown of these by region is shown in Figure 5. Approximately two-thirds of the coal data were from Australia, and were provided by SCT. Preliminary statistical analyses indicated that the four eastern U.S. coalfields could be combined into a single “eastern US coal” grouping, and that the UK and German data could be combined into a “northern European coal” grouping.

In addition, about 200 non-coal measurements that were readily available in the literature and at the WSM website (Reinecker et al., 2005) were collected. The purpose of the non-coal data was to provide an independent check on the general regional trends observed within the coal data. The non-coal data set includes stress measurements from the same general regions as the coal data set, though it does not include any measurements from Australia. The non-coal data also provides an opportunity to compare stress trends within bedded coal measure strata to those in other geologic settings. However, it is recognized that while the coal data set is easily the most comprehensive of its kind ever compiled, the non-coal database is quite small compared to how many measurements could be available.

Preliminary analyses were conducted to see where non-coal data sets could be combined. It was determined that the measurements from Ireland, the UK, northern Europe, and Scandinavia could be combined into a single “All Europe” grouping. Similarly, data from the eastern US, Canada, and the western U.S. were combined into a “North America” grouping. It was surprising that while there were distinct differences between the western and eastern US coal data sets, the trends within the US non-coal data did not seem to vary much by region. Figure 6 shows the number of data points within each regional grouping.

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**Figure 5 - Locations of the stress measurements included in the coal data set**

**Figure 6 - Locations of the stress measurements included in the non-coal data set**
Figures 7 and 8 show the range of depth and modulus within each regional group in the coal data set. The greatest depths are encountered in the western European mines (UK and Germany), while the highest modulus rocks are in the eastern US. Queensland has the lowest modulus rock. Note that the small size of the data set from India (n=5) and the relatively small range of depth in the South African data set mean that the results from these two regions would be expected to be less reliable than those from the other regions.

Figure 7 - Range of depths of the coal stress measurements, by region. The middle line represents the median value, and the upper and lower hinges of the box represent the 75th and 25th percentiles of the data, and the dots are outliers

Figure 8 - Range of elastic modulus of the coal stress measurements, by region

The same types of summary data for the non-coal data set are shown in Figures 9 and 10. The depth ranges are similar to coal data, except in South Africa where the non-coal data includes a number of measurements from extremely deep gold mines. In general, the modulus values for the non-coal measurements are about twice as great as those from the coal data.
Data Weighting: In order that the coal data not be overwhelmingly influenced by the Australian data, all the data was weighted by the following formula:

\[
\text{Weight of an individual measurement} = \frac{1}{(n_R)^{0.5}} 
\]

(8)

Where \( n_R \) is the number of measurements from a particular region.

The result is that an individual data point from a region with few measurements will count more heavily than one from a region with a lot of measurements, but overall the heavily populated regions will still have more influence. For example, the data set contains 40 measurements from the eastern US and slightly more than four (4) times as many measurements from NSW. In the weighted analysis, an individual US measurement is given a weight about two times as great as a measurement from NSW, but in all the NSW measurement have twice as much influence over the final equation as do all the eastern U.S. measurements.
As a check, all of the analyses were run using both weighted and unweighted data, and it was found that the results did not differ significantly. The weighted results will be reported here in this paper.

**Depth Gradients and Modulus Effect for the complete data sets:** Multivariate regression was conducted using the statistical package STATA. The first analyses looked at the relationship between depth and stress, with no other variables. The form of the model is thus:

\[ S_{H\text{max}} = B_0 + B_1 (\text{Depth}) \]  

(9)

In this analysis, \(B_1\) is the gradient of the maximum horizontal stress with depth (in MPa/m) and the intercept \(B_0\) can be interpreted as the “excess stress” that is not associated with the depth gradient.

The results are shown in Table 1:

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<th></th>
<th>Excess Stress (B_0) (MPa)</th>
<th>Depth Gradient (B_1) MPa/m</th>
<th>r-squared</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal Data</td>
<td>7.3</td>
<td>0.024</td>
<td>0.33</td>
</tr>
<tr>
<td>Non-Coal Data</td>
<td>9.6</td>
<td>0.028</td>
<td>0.56</td>
</tr>
<tr>
<td>Combined Data Set</td>
<td>8.0</td>
<td>0.026</td>
<td>0.50</td>
</tr>
</tbody>
</table>

In other words, for the combined data set, we can explain half of the variation simply by the depth (and a constant). All the other variability—regional location, modulus, proximity to the entry and other measurement errors (Gadde et al., 2006), is only responsible for the other 50%. Although depth is not as strong a predictor in for the coal data as it is for the non-coal data, the depth gradients are all also approximately equal to the vertical stress gradient of 0.025 MPa/m. The similarity between the coal and non-coal equations is striking—these are two completely independent data sets, drawn from similar parts of the world.

When the analyses were run with just the modulus, instead of depth, the r-squared values were all reduced. In the case of the coal data, the reduction is only from 0.33 to 0.28, but for the non-coal data (and the complete data set) the r-squared is reduced from about 0.5 to approximately 0.20. These results indicate that modulus effect is most pronounced in layered, coal measure geologies, but that even there, depth explains about as much of the variation in the data as the modulus does.

In the next set of analyses, the effects of modulus and depth are explored simultaneously. The regression equation that is used is:

\[ S_{H\text{max}} = B_0 + B_1 (\text{Depth}) + B_2 (\text{Modulus}) \]  

(10)

In this model, the excess stress consists of two components, the intercept \((B_0)\) and plus the modulus term \((B_2*E)\). Note that with this model, the excess stress is independent of the depth. Statistical analyses confirmed that there was minimal interaction between the modulus term and the depth within this data set.

The results in Table 2 show that adding the “modulus factor” improves the r-squared values considerably, from 0.33 to 0.52 for the coal data, and to above 0.60 for the non-coal data set. Note that the values for the depth gradient drop slightly but are still close to 0.025 MPa/m. The biggest change from Table 1, particularly for the non-coal data, is a reduction in the constant. In other words, it appears that the two elements of the excess stress, the intercept and the modulus term, are closely related.

It is also worth pointing out that the relative importance of the depth effect and excess stress change with depth. For example, for a coal mine at 90 m of cover and a modulus of 21 GPa, the total predicted stress is 11.9 MPa. The excess stress accounts for 9.9 MPa, and the depth effect contributes just 1.8 MPa to this total. If the depth increases to 450 m, the predicted stress is now 18.9 MPa, and all the increase is due to the depth effect.
Table 2 - Regression results using equation (10) as the model.

<table>
<thead>
<tr>
<th></th>
<th>Intercept $B_0$ (MPa)</th>
<th>Depth Gradient $B_1$ (MPa/m)</th>
<th>Modulus Factor $B_2 (10^{-3})$</th>
<th>r-squared</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal Data</td>
<td>0.7</td>
<td>0.020</td>
<td>0.44</td>
<td>0.52</td>
</tr>
<tr>
<td>Non-Coal Data</td>
<td>2.0</td>
<td>0.026</td>
<td>0.19</td>
<td>0.62</td>
</tr>
<tr>
<td>Combined Data</td>
<td>2.6</td>
<td>0.025</td>
<td>0.21</td>
<td>0.58</td>
</tr>
</tbody>
</table>

Region-by-region analyses: The next step was to determine predictive equations in the form of equation (10) for each of the individual regions. It is reasonable to expect that different regions might have different relationships between stress, modulus, and depth, owing to different characteristics of the crust, tectonic forces, etc. The results of the individual analyses are shown in Table 3.

Table 3 - Regression results for regional subsets of the data, using equation (10) as the model

<table>
<thead>
<tr>
<th>Region</th>
<th>n</th>
<th>Intercept $B_0$ (MPa)</th>
<th>Depth Gradient $B_1$ (MPa/m)</th>
<th>Modulus Factor $B_2 (10^{-3})$</th>
<th>r-squared</th>
</tr>
</thead>
<tbody>
<tr>
<td>West U.S. Coal</td>
<td>20</td>
<td>-6.4</td>
<td>0.015</td>
<td>0.62</td>
<td>0.71</td>
</tr>
<tr>
<td>UK/Ger Coal</td>
<td>52</td>
<td>-1.7*</td>
<td>0.012</td>
<td>0.51</td>
<td>0.50</td>
</tr>
<tr>
<td>South Africa Coal</td>
<td>22</td>
<td>6.1</td>
<td>0*</td>
<td>-0.01*</td>
<td>0</td>
</tr>
<tr>
<td>India Coal</td>
<td>5</td>
<td>2.6*</td>
<td>0.029</td>
<td>-0.04*</td>
<td>0.79</td>
</tr>
<tr>
<td>NSW Coal</td>
<td>170</td>
<td>-4.4</td>
<td>0.040</td>
<td>0.56</td>
<td>0.71</td>
</tr>
<tr>
<td>Qld Coal</td>
<td>64</td>
<td>-1.5</td>
<td>0.031</td>
<td>0.34</td>
<td>0.51</td>
</tr>
<tr>
<td>U.S./Can Non-Coal</td>
<td>115</td>
<td>-1.9</td>
<td>0.033</td>
<td>0.27</td>
<td>0.70</td>
</tr>
<tr>
<td>N. Europe Non-Coal</td>
<td>47</td>
<td>6.3</td>
<td>0.028</td>
<td>0.02*</td>
<td>0.55</td>
</tr>
<tr>
<td>South Africa Non-Coal</td>
<td>14</td>
<td>2.3*</td>
<td>0.046*</td>
<td>-0.04*</td>
<td>0</td>
</tr>
<tr>
<td>India Non-Coal</td>
<td>16</td>
<td>-5.8</td>
<td>0.049</td>
<td>0.26</td>
<td>0.56</td>
</tr>
</tbody>
</table>

* signifies statistic is not significantly different from 0 at the 95% confidence level.

Several observations can be made on the data in Table 3:

- The r-squared for the individual equations are between about 0.50 and 0.70 for nine of the eleven data sets, which is not significantly better than what was achieved by analysing the entire data set.
- The most consistent coefficients are the depth gradients ($B_1$), and these are also generally the most statistically significant coefficients. The range is from about 0.013-0.040 MPa/m for the larger data sets.
- The modulus factors are also fairly consistent, ranging from about 250 to 600*10^-3. However, in four of the smaller data sets, these coefficients are not significant. Moreover, the modulus effect is more pronounced in the coal measure rocks.
- The excess stress, determined using the mean modulus value for each region, was quite consistent, averaging about 7 MPa for the seven largest data sets. In six of those cases, the average total excess stress (determined using the average modulus for that data set) ranged from 6.0-9.3 MPa.

Overall, while the equations for some regions (western US, NSW) seem reasonably reliable, in many cases the small size of the individual data sets is probably responsible for poor correlations.

Unified analyses, controlling for different regional depth gradients: A statistical technique is available that allows us to combine the power of using the largest possible data set, while
simultaneously allowing for some regional variation. This is accomplished by allowing the coefficients for the depth gradient to vary region by region.

The final equations are in the form of equation (10), but while all of them use the same coefficients for the components of the excess stress, each region has its own depth gradient. The results are shown in Table 4.

**Table 4 - Stress prediction parameters for equation 10 determined for the individual coal regions using the unified analysis regression technique**

<table>
<thead>
<tr>
<th></th>
<th>n</th>
<th>Intercept B₀ (MPa)</th>
<th>Depth Gradient B₁ (MPa/m)</th>
<th>Modulus Factor B₂(10⁻³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>East U.S. Coal</td>
<td>42</td>
<td>-2.1</td>
<td>0.037</td>
<td>0.41</td>
</tr>
<tr>
<td>West U.S. Coal</td>
<td>20</td>
<td>-2.1</td>
<td>0.022</td>
<td>0.41</td>
</tr>
<tr>
<td>UK/Ger Coal</td>
<td>52</td>
<td>-2.1</td>
<td>0.020</td>
<td>0.41</td>
</tr>
<tr>
<td>South Africa Coal</td>
<td>22</td>
<td>-2.1</td>
<td>0.027</td>
<td>0.41</td>
</tr>
<tr>
<td>India Coal</td>
<td>5</td>
<td>-2.1</td>
<td>0.029</td>
<td>0.41</td>
</tr>
<tr>
<td>NSW Coal</td>
<td>170</td>
<td>-2.1</td>
<td>0.041</td>
<td>0.41</td>
</tr>
<tr>
<td>Qld Coal</td>
<td>64</td>
<td>-2.1</td>
<td>0.031</td>
<td>0.41</td>
</tr>
</tbody>
</table>

The regression analysis used to obtain Table 4 achieves an r-squared of 0.69. Accounting for nearly 70% of all the variation in such a large and diverse data set is an impressive accomplishment. The following further observations can be made on these results:

- The greatest depth gradient, approaching twice the vertical stress gradient, was found for the NSW measurements, which corresponds to the prediction based on the WSM;
- The results for the eastern US, Queensland, and the Western US are also in good agreement with WSM predictions, and;
- The depth gradient for Europe is somewhat lower than was expected based on the WSM. This may be attributable in part to extensive past mining in the UK and German coalfields where these measurements were made (Muller, 1991).

The equivalent results for the non-coal data are shown in Table 5. It is significant that all but one of the regional depth gradients determined for the non-coal data are very similar to the gradients found for the coal data. The exception is India where the coal data set is very small.

**Table 5 - Stress prediction parameters for equation 10 determined for the individual non-coal regions using the unified analysis regression technique.**

<table>
<thead>
<tr>
<th></th>
<th>n</th>
<th>Depth Gradient B₁ (MPa/m)</th>
<th>Modulus Factor B₂(10⁻³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>U.S./Can Non-Coal</td>
<td>115</td>
<td>0.034</td>
<td>0.20</td>
</tr>
<tr>
<td>N. Europe Non-Coal</td>
<td>47</td>
<td>0.022</td>
<td>0.20</td>
</tr>
<tr>
<td>South Africa Non-Coal</td>
<td>14</td>
<td>0.021</td>
<td>0.20</td>
</tr>
<tr>
<td>India Non-Coal</td>
<td>16</td>
<td>0.033</td>
<td>0.20</td>
</tr>
</tbody>
</table>

*The constant (B₀) was not statistically significant at the 95% level.

Figure 11 is a type of “residual plot” that helps evaluate the validity of the regression equation. It compares the maximum horizontal coal mine stresses predicted by Table 4 with the measured ones for all regions except Australia. The Australian data appears on Figure 12. Both figures show that the

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1 The technique involves creating interaction terms involving dummy variables (Wooldridge, 2006, pp. 244-252). In this case, a dummy variable is defined based on region, and then that dummy variable is interacted with the variable “depth.”
residuals (discrepancies) are quite evenly distributed for all regions and stress levels, which indicates that the regression results are valid.

Figure 11 - Residual plot for Table 4, non-Australian data

Figure 12 - Residual plot for Table 4, Australian data only

Figure 13 plots the coal stress measurements against the depth of cover. Two ranges of predicted stress derived from Table 4 are shown in the figure. The upper limit for the “normal” grouping is based on the NSW depth gradient, and the lower limit follows the Queensland depth gradient. For the “low gradient” grouping, the upper limit uses the western US gradient, while the lower limit uses the gradient found for the UK and German coalfields. In both cases, the upper limits are based on rock with an elastic modulus of 35 GPa, and the lower limits assume a rock modulus of 7 GPa.
Analysis Using Modulus and Tectonic Strain Factor (TSF): A number of analyses were conducted using model represented in equation (7), and employing the TSF concept as defined by Dolinar (2003) and Nemcik et al. (2005). In these analyses, when only the modulus was used as the dependent variable, the r-squared values were not much different than those obtained when modulus was regressed against $S_{\text{hmax}}$ (less than 0.30 for the coal data, and less than 0.2 for the non-coal data). In an alternative analysis, individual TSF values were determined for each region. This analysis found the highest TSF in NSW, with a value of about 0.85, while the TSF determined for the Queensland, US, and European coalfields was about 0.45. The r-squared for this analysis was only 0.50, however, considerably lower than the 0.69 obtained with Table 4.

Analysis of the minimum principal stress, $S_{\text{hmin}}$: No data was available from Australia for analysis of the minimum principal stress. The results of the analyses for the other regions are shown in Table 6. The model that was employed is shown in equation 11:

$$S_{\text{hmin}} = B_0 + B_1 \text{ (Depth)} + B_2 \text{ (Modulus)} \quad (11)$$

The same “unified” regression technique used to obtain Tables 4 and 5 was used in the analysis. The r-squared for the regression using the coal data was 0.54, and it was 0.67 for the non-coal data. The constant was not statistically significant in either equation.

<table>
<thead>
<tr>
<th>Region</th>
<th>n</th>
<th>Depth Gradient $B_1$ (MPa/m)</th>
<th>Modulus Factor $B_2$ ($10^{-3}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>East U.S. Coal</td>
<td>42</td>
<td>0.030</td>
<td>0.15</td>
</tr>
<tr>
<td>West U.S. Coal</td>
<td>20</td>
<td>0.013</td>
<td>0.15</td>
</tr>
<tr>
<td>UK/Ger Coal</td>
<td>52</td>
<td>0.009</td>
<td>0.15</td>
</tr>
<tr>
<td>South Africa Coal</td>
<td>22</td>
<td>0.005</td>
<td>0.15</td>
</tr>
<tr>
<td>India Coal</td>
<td>5</td>
<td>0.009</td>
<td>0.15</td>
</tr>
<tr>
<td>U.S./Can Non-Coal</td>
<td>115</td>
<td>0.020</td>
<td>0.12</td>
</tr>
<tr>
<td>N. Europe Non-Coal</td>
<td>47</td>
<td>0.011</td>
<td>0.12</td>
</tr>
<tr>
<td>South Africa Non-Coal</td>
<td>14</td>
<td>0.010</td>
<td>0.12</td>
</tr>
<tr>
<td>India Non-Coal</td>
<td>16</td>
<td>0.023</td>
<td>0.12</td>
</tr>
</tbody>
</table>
DISCUSSION

At the beginning of the last section, several predictions were made about the horizontal stress measurements based on the WSM "critically stressed crust" model. In nearly every instance, the prediction was confirmed by the analysis:

- The depth was as important as the modulus in predicting the horizontal stress in the coal mine data set, and it was a much better predictor in the non-coal data set. When both factors were combined, the accuracy of the predictions improved significantly.
- The calculated depth gradient of 1.6 times the vertical stress for the eastern US, and 1.4 times the vertical stress in the Bowen Basin, was within the range of what was predicted for coalfields located in stable, a-seismic, mid-plate areas. The depth gradient that was calculated for the northern European coalfields of the UK and Germany was a little lower than expected, but even there it was still 0.9 times the vertical stress.
- The greatest depth gradient was found to be in the seismically active compressive regime of the Sydney Basin, and one of the lowest depth gradients was in the active extension regime of the western US coalfields.

It is reassuring that the depth gradients that were determined empirically seem to match up well with those that have been identified in the deep crustal measurements made by the WSM. The findings of this study indicate that we neither have to look for nor explain a discontinuity in stress at the transition between the “near surface” and the “deep” crust.

The study also confirmed that the “excess stress” is one significant component of the near-surface stress that does not fit directly into the WSM model. The excess stress typically adds 3.5-10.5 MPa to the horizontal stress that would be predicted by the depth gradient alone. At depths of less than 300 m, the excess stress is usually responsible for at least 50% of the measured maximum horizontal stress.

Some important characteristics of the excess stress that were observed in the analysis are:

- Its magnitude, averaging about 7 MPa, is quite consistent over a broad range of independent data sets;
- Its magnitude does not noticeably increase with depth.
- The value of the modulus factor was about half as large in the non-coal data set as it was in the coal data set. However, since the average modulus in the coal data set is about twice the average non-coal modulus, the overall excess stress is about the same in both geologic regimes.
- The modulus factors determined from the analysis of the minimum principal stresses are consistently about half as large as those calculated for the maximum principal stress. In other words, the modulus effect is just as anisotropic as the vertical stress gradients.

Taken together, these observations indicate that the source of the excess stress is likely the same current tectonic regime that is responsible for the other aspects of today’s stressfields, not “residual stress” or some other cause that would likely vary from locality to locality. One possible explanation is that while the “critically stressed crust” model normally assumes that the cohesion on fault planes is very small relative to the stresses at depth, near the surface that assumption may not be valid (Zoback et al. 2003). The cohesional strength of critically stressed faults may be enough to allow the crust to carry 7 MPa of stress more than would otherwise be the case.

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1 It is also reassuring that other researchers working with mining stress measurement data appear to reaching very similar conclusions. Lee et al. (2008) present results from an analysis of 258 high-quality, three-dimensional stress measurements from Australian metal mines. Stress gradients well in excess of the vertical are apparent to depths of 1.6 km. Lee et al. (2008) also show that the ratio of each principal stress to the first invariant (S1+ S2+S3) is relatively constant, an observation Windsor (2007) confirmed independently working with another hard rock mining data set. Harrison et al. (2007) concluded that this last observation could be explained by a Mohr-Coulomb relationship between S1 and S3, and is “entirely consistent with the hypothesis that the Earth’s crust is in a state of limiting equilibrium.”
One further observation is that the modulus factor was more important to predicting the stress in the coal mine data set than it was in the non-coal data. In fact, for the coal data, the modulus was almost as powerful a predictor of the stress as the depth. It makes sense that in layered, sedimentary coal measure strata there would be a strong tendency for the stiffer rock units to carry more load. On the other hand, the horizontal strain within the rock is certainly not, in general, constant regardless of depth, and it may not even be fully equalized at any given stress measurement location.

While the stress prediction equations that were developed during this study are quite robust statistically, it should be recognized that there are significant unexplained variability in the data. This is most clearly evident near the surface in mountainous terrain, where topographic effects are likely to be substantial. In situ stresses are also much less predictable in seismically active coalfields like NSW and the Western U.S. Site specific stress measurements are still the only technique that can provide assurance of the local stress conditions.

A final comment is that there now seems to be no justification for employing the “Poisson’s effect” or other static earth theories in any aspect of the analysis of in situ stress. The horizontal stresses measured underground are caused by the large plate tectonic forces that are currently being carried by the earth’s crust. Poisson’s effect can play a role when loads are actively applied to the ground, as in a longwall tailgate (Frith and Colwell, 2006), but there is no theoretical or empirical basis for using it to explain any aspect of the in situ stresses that develop over geologic time.

CONCLUSIONS

Using the largest data base of in situ stress measurements from coal mines ever assembled, this study found that the data fits the “critically stressed crust” model based on plate tectonics theory surprisingly well. Past studies had already shown that plate tectonics explained the orientation of the stresses observed underground, now it seems it explains their magnitudes as well. In particular, the maximum horizontal stress was found to increase with depth at rates of approximately 0.8-2.0 times the vertical stress. The study also identified a second component of the measured stress, called the “excess stress,” which adds approximately 3.5-10.5 MPa to the total. The excess stress is apparently independent of the depth.

The study makes a major contribution by providing equations for estimating the maximum horizontal stress in several major coalfields around the world. More importantly, the study provides a framework for understanding the source of the horizontal stresses encountered underground every day. By showing that these “near surface” stresses are tied to those found in the deep crust, it links the worlds of rock engineering and geophysics. Hopefully, future research can build on this foundation to develop better tools for understanding and predicting in situ stress, and for using those predictions to design safer underground structures.

ACKNOWLEDGEMENTS

The authors would like to express their sincere appreciation to Dr. Winton Gale for making the Strata Control Technologies (SCT) stress measurement data base available for this analysis. The SCT data base is undoubtedly the most extensive of its kind in the world, and its inclusion has added immensely to the value of this paper. We would also like to thank Rock Mechanics Technology (RMT) for making a large portion of their very impressive world-wide data base available to other researchers via their website. Thanks are also due to Russel Frith, Hamid Maleki, Dennis Dolinar, and Leo Gilbride for their contributions to the data base.

REFERENCES


3D GEOTECHNICAL MODELS FOR COAL AND CLASTIC ROCKS BASED ON THE GSR

Terry Medhurst 1, Peter Hatherly 2, Binzhong Zhou 3

ABSTRACT: This paper provides the key outcomes and an example of Geophysical Strata Rating (GSR) analysis and modelling as undertaken in recently completed ACARP Project C17009. Various examples from the project demonstrate the use of GSR for overburden characterisation and hazard planning, tailgate stability and longwall face behaviour.

Borehole spacings of 200 m or thereabouts, which are typical of Bowen Basin underground operations, allow geologically meaningful 2D and 3D modelling of GSR and other parameters derived from the geophysical logs. Plots of clay content and porosity can be used, depending on their relevance to the given strata behaviour, and/or as alternative means of investigating geological features in the overburden sequence.

An approach for determining the GSR in coal and carbonaceous has been successfully developed material. In coal, the GSRi is inversely related to brightness as indicated by the ash content evident in density, natural gamma and sonic logs. GSR values are shown to relate to other existing classification schemes, but arguably with more sensitivity to the weaker materials than present in the manual logging methods.

INTRODUCTION

In most coal mining operations, there is a need for repeatable and quantifiable geotechnical assessment procedures that allow appropriate planning and responses to changing conditions. Many underground sites use the Coal Mine Roof Rating (CMRR) or similar classification systems. Others, both open cut and underground, use unconfined compressive strength (UCS) as an index to changing rock conditions. Where UCS is used, sonic logs have become the main data source to estimate UCS. Laboratory measurements are also included, where possible.

At the 2009 Underground Coal Operators Conference a new method for evaluating geotechnical conditions known as the Geophysical Strata Rating (GSR) was presented. A comparison between conventional UCS results at Crinum Mine and the new GSR determinations was provided. The GSR is based on the interpretation of sonic, density and natural gamma logs and is designed to provide a measure of strata properties on a linear scale similar to that used in the Coal Mine Roof Rating. The GSR allows coal bearing strata to be quantitatively assessed in every borehole that is geophysically logged, regardless of height above the working seam.

Through a series of ACARP funded projects C11037 (Hatherly et al., 2003), C15019 (Hatherly et al., 2008) and C17009 (Medhurst et al., 2009) we have made GSR determinations at mine sites throughout the Sydney and Bowen Basins. These have consistently provided useful insights into geotechnical conditions and provided a proven capability of distinguishing changes in strata properties. In the latest project we have had the opportunity to benchmark the GSR against actual mine performance. There was also a requirement to characterise coal and other carbonaceous materials. Through using sonic, density and gamma logs, we have been able to provide a complementary measure of GSR in these materials.

To make full use of the GSR assessment of ground conditions, it is necessary to estimate GSR values between boreholes, in three-dimensions. This requires geostatistical estimation using techniques that have received little attention in geotechnical engineering. The use of 2D and 3D geotechnical models using borehole interpolation from GSR estimates is demonstrated. 3D seismic data, if available, can also be used to constrain the boundaries of the main geological horizons (mainly coal seams) between boreholes.

1 PDR Engineers Pty Ltd
2 Coalbed Geoscience Pty Ltd
3 CSIRO Exploration and Mining
THE GEOPHYSICAL STRATA RATING

The full definition of the GSR is given in Hatherly et al (2008). The components of the GSR are provided through the following scores, with the initial GSR (GSRi) being the sum of the first four scores:

1. strength score (5 to 55) depending on sonic velocity
2. porosity score (0 to -15) depending on porosity
3. moisture score (0 to -10) depending on porosity and clay content
4. cohesion score (10 to 25) depending on sonic velocity and quartz content
5. bed score (0 to 10) depending on the downhole variability of clay content.
6. fracture score depending on the downhole variability of the GSRi.

These terms have geomechanical significance. The first four can be added together to give an estimate of the quality of individual beds – the initial GSR (GSRi). The effects of the variability due to defects and changes in the bedding are captured by the last two terms. Comparison of the various scores indicates that sonic velocity is the main driver of the GSR. To ensure that the GSR is not depth dependent, the velocities are corrected for effective pressure. Compensation to the strength and cohesion scores is then made in consideration of the effects of porosity and clay content on rock strength. The bed score is designed to respond to bedding, as expressed by changes in the clay content. The effect of fracturing is assumed to be expressed within the variability present in the sonic log which, in-turn creates variability in the GSRi.

Significantly, the GSR does not attempt to map individual fractures nor their orientation. It is also unable to identify mineralogical factors such as the presence of water reactive clays. The GSR should be viewed as a scheme which provides bulk rock characterisation, independent of depth and mining considerations.

GSR FOR CARBONACEOUS MATERIAL

For the purposes of the analysis we have taken carbonaceous material to represent rock units containing enough organic components to affect the geophysical log responses but which are not coal. Such material will have low density and low velocity. Unless it is explicitly identified in the GSR calculations, it will be assigned a very high porosity and be scored accordingly. Given that the carbonaceous material that is most commonly encountered is in the form of carbonaceous mudstone and siltstone both of which have low porosities, the score would not be correct.

We have chosen to automatically set the porosity to zero for any carbonaceous material found from a density log which is not identified as coal. The GSR determination then continues in the same manner as for clastic rocks.

GSR FOR COAL

To provide a rating for coal, a different approach has been developed. If we simply extended the approach for clastic and carbonaceous materials into coal, it would be the weakest rock type because it has the lowest sonic velocity. However, we know that coal, by virtue of its organic fabric and the absence of bedding planes, can have an intrinsic strength that is not present in other rock types with low sonic velocities.

Medhurst and Brown (1998) investigated factors affecting variations in coal strength and found that bright coals are weaker than dull coals. This was found to be due to the increased presence of cleats in the brighter coals. Brightness, per se, is not a parameter that can be expected to be associated with a geophysical log response. However, given that coal brightness can be related to coal strength, there is some merit in proposing that the ash profile may be also related to brightness and hence be related to the mechanical properties of the coal. Ash content (mineral matter) is a parameter that influences geophysical log responses. Density, natural gamma and sonic responses all become elevated when

1 The ASTM’s definition for coal is that it is material containing more than 50% by weight and 70% by volume carbonaceous material. The carbonaceous material under discussion here has less than 50% organic material.
the ash content increases. (See for example, Zhou and Esterle (2007) who describe the use of density data to predict ash content.)

From a given log response and estimated values for density, natural gamma and transit times it is possible to estimate a parameter that relates to the overall ash content. A simple model for these associations utilises linear mixing laws to estimate the relative proportions of the organic coal and ash constituents that are present. Logs showing coal volume profiles within four different coal seams are presented in Figure 1. For each of these seams, the three different logs provide similar estimates, thus suggesting that the linear mixing laws are appropriate.

![Figure 1 - Fraction of organic matter in coal seams from a range of locations. Gamma log results in blue, density in plum and sonic in god](image)

The average of these three coal/ash indicators can therefore be calculated to provide an indication of the fraction of coal present and the apparent ash profile in the coal seam. Given that coal brightness may also be related to the ash profile, we propose the following simple relationship to provide an estimate of GSRi:

\[
\text{GSRi} = 5 + 45 \times (1 - \text{CoalFraction})
\]

where the coal fraction it is set to 0.3 if the average apparent ash value is less than 0.3.

In the normal GSR calculations, the defect score (the combined bed and fracture scores) is between 0 and 20. In the case of coal, adding defect scores between 0 and 20 provides too much emphasis to the defects and creates ‘noisy’ GSR estimates. We have therefore decided to halve the values of the defect scores in coal (i.e. the total defect score is between 0 and 10). This is further supported by the observation that horizontal weakness planes are often less well developed in pure coal seams and hence less dominant in their failure behaviour. Cleating normally plays the dominant role. Distinct non-coal weakness planes such as penny bands are picked up via the normal GSR analysis.

\[1 \text{ There is the greatest difference for the Hunter Valley example. In this case the difference may be due to problems in the calibration of the geophysical logs.}\]
Figure 2 shows a comparison between GSRi and average brightness for data from Moranbah North. Although both parameters are based on averages over the given seam interval, the results still show the trend of GSRi increasing with duller coals. Figure 3 investigates the relationship between GSR and CMRR where coal roof, carbonaceous and non-coal materials are present. There is a weak but positive correlation. Note also the relatively small range of CMRR values (40 – 45) compared to GSR (20 – 30). This suggests that the GSR analysis is more sensitive to variations in coal strength.
MODELLING THE GSR

In order to make full use of GSR determinations, it is desirable to extend the results between boreholes to form 2D and 3D models. From these models sections and plans showing vertical and lateral variations can be constructed to enable further geotechnical evaluation.

There are a number of large modelling packages utilised by mining companies. For this project, we did not seek access to any of these packages nor undertake the training required to operate them. Instead, we chose to utilise the low-cost Surfer program to undertake our initial 2D modelling while at the same time developing a 3D modelling capability that would allow us to develop 3D models. The algorithm was to be based on estimation procedures we had already developed in a previous ACARP project for converting seismic reflection data from travel times to depths (Zhou et al., 2004).

Inputs for the 2D and 3D modelling were simple Excel and text files detailing the locations of the boreholes, the GSR data tabulated against down-hole depth and, for our own modelling program, the depths of the control surfaces in each of the boreholes. Our algorithm allows for seam splitting but not faulting in situations where layers replicate themselves (reverse faulting) or are absent (normal faulting).

The outputs from the modelling are grid files showing grid point coordinates and the estimated value of GSR. 2D grid files can be displayed using Surfer. For our 3D modelling, there is the option of generating a series of grid files which Surfer can display, or alternatively organising the results into the same data structures used for 3D seismic data. With the data in this format, we are able to interrogate the model using SeisWin, our interactive 3D seismic interpretation program. A useful feature of Surfer and SeisWin is that they allow dxf files showing mine plans and other map data to be imported and displayed together with the model results.

MORANBAH NORTH EXAMPLE

During extraction of LW105 at Moranbah North, longwall face cavities and uncharacteristic tailgate instability developed at the inbye end of the panel, which required relocation of the face. One aspect of the overburden conditions which was considered to contribute to the instability was the presence of a rider seam/stone interface in the roof.

Detailed discussion of field monitoring and 3D modelling of gateroad behaviour at Moranbah North is provided by Tarrant (2006). As a result of this study, a deformation mechanism termed 'skew roof deformation mechanism' was identified, which related the regional differential horizontal movements to the shear behaviour about gateroads. Tarrant suggests the key factors driving the skew roof mechanism are:

- the magnitude of the vertical and horizontal stresses;
- the shear modulus of the strata pile (shear deformability); and
- the extent of overburden bridging.

The relative location of weak interfaces in the roof and the character of the overburden strata, particularly massive strata, can have a significant effect on roadway support and chain pillar sizing. Interestingly, the influence of massive strata movements and its impact on maingate roof behaviour was previously identified via microseismic monitoring and analysis of surface to seam extensometer data at Southern Colliery (Hatherly et al., 2003). This was attributed to a thickening of an overlying sandstone channel and a weak interface high into the strata, which allowed strata movements towards the goaf. It was therefore considered useful demonstration of the GSR analysis to investigate the 3D variations in overburden strata characteristics in the vicinity of the LW105 fall zone.

The general area investigated showing relevant boreholes is shown in Figure 4. Two-dimensional analyses of borehole data along the gateroads were initially undertaken. Figures 5 and 6 show detailed sections from the 2D analysis along the tailgate and maingate, respectively. The upper and lower white lines show the bounding surfaces. Within the GM seam, the boundaries of the working section are also shown. The plots also show the presence of the rider seam at the lower right edge of the plot and the associated seam split. There is also a split present in the overlying P seam which is associated with the development of an upper sandstone channel.
Figure 4m - Zone used for Moranbah North GSR analysis

Figure 5 - Detailed section along tailgate
Comparison of the two sections indicates a stark contrast in overburden conditions. In particular, the presence of thick sandstones (high GSR values) immediately above the seam and higher up into the strata over the tailgate can be observed. In addition at the location of RDH244, the rider seam is present and is immediately overlain by heavy sandstone strata that also coincide with the edge of the upper sandstone channel. This is the location where the longwall had to be relocated. This clearly shows the benefit of developing detailed overburden sections, as such information could be difficult to determine from analysis of RDH244 in isolation; or from manual analysis of gamma or sonic velocity data.

Detailed 3D analysis was also undertaken using the borehole data. The 3D plots compare well with the 2D sections along the gateroads. Figures 7 and 8 shows plan views of the average GSR over intervals of interest. Figure 7 shows the immediate roof zone. The influence of the rider seam resulting in low GSR values south of RDH243 can be seen. Figure 8 shows a similar plot for the intermediate roof. This highlights the influence of the overlying sandstones and the zone where the heavy roof overlaps the seam split zone.

In summary, the analysis shows the location of heavy roof and how it coincides with the fall area along the LW105 tailgate. Previous analysis had also indicated that the presence of the rider seam under these conditions can lead to particularly challenging roof conditions that require specific ground support requirements. The GSR analysis provides a detailed representation of the ground conditions that would have contributed to the falls and accurately corresponds to the zones in which such roof behaviour would have been triggered.

The analysis provided here was undertaken using data that was available prior to the instability. Whilst roof monitoring alone could not be used to verify the proposed roof failure mechanisms, the geophysics based analysis provides ample evidence to support the proposed roof failure mechanism. With such information available prior to extraction, the mine may have been able to identify hazards, improve monitoring strategies and if necessary undertake additional analysis to incorporate into the strata management plan.
Figure 7 - Average GSR for 5 m interval above GMS working roof

Figure 8 - Average GSR for 4 m to 14 m interval above GMS working roof
DISCUSSION AND CONCLUSIONS

This paper provides the key outcomes and an example of GSR analysis and modelling as undertaken in recently completed ACARP Project C17009. Further examples from GSR modelling from Newlands and Crinum mines are provided in the final report. At our three main investigation sites, our approach for determining the GSR provided a means to identify key strata characteristics that were known to have influenced mining performance. In each case the data used for the analysis was available and could have been used prior to mining.

The examples demonstrated highlight the use of GSR for overburden characterisation and hazard planning, tailgate stability and longwall face behaviour. Previous experience at other operations also shows that a GSR analysis is able to identify subtle changes in strata characteristics that can often be associated with strata control management issues.

The results show that borehole spacings of 200 m or thereabouts, which are typical of Bowen Basin underground operations, allow geologically meaningful 2D and 3D modelling of GSR and other parameters derived from the geophysical logs. Plots of clay content and porosity can be used, depending on their relevance to the given strata behaviour, and/or as alternative means of investigating geological features in the overburden sequence.

An approach for determining the GSR in coal and carbonaceous material has been successfully developed. In coal, the GSRi is inversely related to brightness as indicated by the ash content evident in density, natural gamma and sonic logs. GSR values were also shown to relate to other existing classification schemes, but arguably with more sensitivity to the weaker materials than present in the manual logging methods.

It is accepted that all strata characterisation schemes will have their strengths and weaknesses with regard to capturing key detail. However GSR analysis has several advantages over manual methods, which include

- Utilises large datasets of borehole geophysics data that exist at some sites
- Quantitative, repeatable analysis
- Provides a continuous record of strata conditions over the entire borehole column
- Complimentary assessment of both geological and geotechnical strata characteristics
- Information available in digital format
- Data suited to 2D and 3D modelling of overburden conditions
- Can be used to re-analysis old datasets
- Easy to update as new information becomes available
- Potential to enhance existing seismic datasets
- Compliments traditional borehole, sampling and testing programs

In general, the ability to capture the spatial variability in strata conditions is the key feature of GSR analysis. Such an approach is necessary to support current trends in resource development from initial scoping and project definition through to mine planning and production management. Different implementation strategies may therefore be required to incorporate GSR analyses depending on project requirements. In this regard factors such as data quality, borehole density and modelling technique have to be considered.

ACKNOWLEDGEMENTS

Financial support for this work has been provided by ACARP and CRC Mining. The collaboration of the various mines that provided data, in particular BMA, Anglo Coal Australia and Xstrata Coal is greatly appreciated.
REFERENCES


ROADWAY ROOF SUPPORT DESIGN IN CRITICAL AREAS AT ANGLO AMERICAN METALLURGICAL COAL’S UNDERGROUND OPERATIONS

Ismet Canbulat

ABSTRACT: In order to ensure the stability of roadways Anglo American Metallurgical Coal (AAMC) has developed an advanced roof support design methodology that integrates analytical, numerical and empirical modelling. This methodology currently is based on a deterministic approach (a single factor of safety against failure is calculated). However, an improved methodology, based on stochastic modelling technique, has also been developed and currently being evaluated. The main advantage of this methodology is that as the design is based on probability distributions of input parameters, the outcome is based on a distribution of factors of safety rather than a single factor of safety. Evaluation of factors of safety may also be used to determine the likelihood of failure which in turn may be utilised to determine and evaluate the associated risks quantitatively in decision making process. This methodology has been evaluated at Grasstree and Moranbah North Coal Mines in the designs of roof support in various critical areas and has been proven to be successful and a better way of determining the roof support requirements. A demonstration of application of this methodology from Moranbah North Mine, where the “world’s highest rated longwall” has recently been installed, is presented in this paper.

INTRODUCTION

Anglo American Metallurgical Coal Australia (AAMC) operates three longwall (LW) mines located in Central Queensland. There is an increasing emphasis on the reliability at these operations as the longwalls are getting deeper and facing more geologically challenging conditions. In addition, the Anglo American Vision is to achieve “Zero Harm” through the effective management of safety at all businesses and operations. In order to accomplish this vision, AAMC has developed a pro-active ground control management system for a safe and efficient production of underground reserves.

Pro-active ground control management involves an understanding of the impacts of the geotechnical environment on likely ground behaviour and consists of approximately 15 major elements. One of the most important elements of this pro-active ground control management is to utilise a roof support design methodology that considers different failure mechanisms and also takes into account all important elements. The design methodology becomes even more important when the area in question is a critical area, which is defined as a high risk roadway where any failure may cause increased levels of safety and financial risks to underground workforce and operations.

One of the recent critical area support designs was at Moranbah North Coal Mine (MNC), where the 1 750 t longwall face was recently commissioned. Because of the size of this longwall, one of the widest longwall installation roads in Australia was developed and widened. Following a stand-up time of approximately two months, the longwall was successfully installed and no excessive roof deformations were encountered.

AAMC ROOF SUPPORT DESIGN METHODOLOGY

The AAMC design methodology is defined by the Geotechnical Systems and Standards of the Operations Management System, which provides a set of minimum standards for the primary and secondary roof and rib support assessment, analysis, design and presentation process at AAMC underground operations.

The aim of this support design methodology is to ensure the stability of roadways at AAMC underground operations.
There is no single universally accepted roof and rib support design methodology. In general, the design methodologies for bolt selection and design can be classified into the following six categories:

- Analytical models,
- Empirical models (i.e., statistical analysis of previous data/experience),
- Numerical modelling,
- Field testing and monitoring,
- Geotechnical classification (mainly CMRR etc), and
- Physical modelling (laboratory testing).

These methodologies require extensive input parameters and assumptions with regard to support and rock properties.

AAMC’s strategy is to use a “combined support design methodology” for critical areas. This methodology considers all methods listed above, with the exception of physical modelling to ensure that the design is sound and acceptable.

In AAMC’s roof support design methodology, four analytical models, namely buckling, shear, tendon in suspension and bond in suspension failures, are considered and, where possible, empirical modelling and geotechnical classification techniques are also used to back analyse the designs and/or to derive the input parameters (e.g., horizontal stress magnitudes).

As the details of these analytical models can be found in numerous previously published publications (Frith, 2000; Colwell, Frith and Guy, 2008; Canbulat and van der Merwe, 2009), it is not intended to present the fundamentals of these models in this paper; therefore, only a short summary of these mechanism are given below for the completeness of the paper:

**Suspension Failure**

The suspension mechanism is the most easily understood roof bolting mechanism. The design of roof bolt systems based on the suspension principle has to satisfy the following requirements (Canbulat and van der Merwe, 2009):

- The strengths of the roof bolts and/or long tendons have to be greater than the relative weight of the loose roof layer that has to be carried.
- The anchorage forces of the roof bolts have to be greater than the weight of the loose roof layer.

**Shear Failure**

This mechanism assumes that the coal mine roof contains a series of bedding planes/laminations and the shear strength of these bedding planes can be calculated using the well-known Coulomb theory with the inclusion of pre-tension and the shear strength of the roof bolts and/or long tendons.

In calculation of the factors of safety, the important consideration in this mechanism is the so-called height of softening to determine the shear stress generated by the surcharge load within the bedding planes. It is assumed in this mechanism that as the height of softening increases parabolically (even above the bolted horizon), the resultant shear stress generated within the bedding planes also increases (Canbulat and van der Merwe, 2009).

**Buckling Failure**

This mechanism has been extensively discussed by Frith (2000) and Colwell, Frith and Reed (2008). A factor of safety concept is also utilised in this mechanism, this being a measure of the load applied to that structure in comparison to its ability to accommodate that load without undergoing yield or failure.

In this mechanism it is assumed that the applied load acts horizontally across the roof and is a product of the in situ horizontal stress and concentration thereof as a result of the mining process. The load
bearing ability of the roof strata and the load bearing ability of roof support are calculated using the buckling theory and mechanical advantage concept (Frith, 2000; Colwell, Frith and Reed, 2008).

To this end, Figure 1 provides a diagrammatical representation of AAMC’s combined support design methodology.

**Figure 1 – AAMC’s roof support design methodology**

This methodology follows the below steps:

- Collection and assessment of geological, mining, and monitoring data, including:
  - Development of probability distributions.
Evaluation of the stress environment using local knowledge, measurements and numerical modelling.

- Prepare assessment form, following site inspections (where possible) and/or available information.
- Design the support system. In the design consider the geotechnical environment, a suitable support system and the four failure mechanisms.
- Check the design against the design criteria.
- Determine the financial viability of the design. Consider that the same factors of safety can be achieved using a combination of different support systems.
- Sign it off with the Principal Geotechnical Engineer and/or external experts.
- Implement the design and communicate it to mining personnel.
- Inspection and monitoring of the installation and performance of the installed support during mining operations for a comparison against the initial design performance. This will require:

**UNCERTAINTIES IN ROOF SUPPORT DESIGN**

Many investigations in Australia and elsewhere confirm that rock mass properties exhibit a high degree of uncertainty. The performance of a support system is affected by these uncertainties and ideally they should be taken into account. In traditional deterministic (calculation of a single safety factor) roof bolt design methodologies, the input parameters are represented using single values. These values are described typically either as “best guess” or “worse case” values. Although in many deterministic roof support design cases, a series of sensitivity analyses are also conducted over one or more parameters, these analyses provide only some insight into the underlying mechanisms and usually cannot take into account the variations that exist in almost all input parameters in support design.

The uncertainties in roof support design can be divided into three distinct areas as shown in Figure 2. These areas are:

- **Given** conditions, which contain all geological and stress related design parameters, such as rock strength, unit thicknesses, friction, stress magnitude and direction. These conditions cannot be controlled or changed.
- **Responses** to the given conditions, which contain the support selection and mining selection, such as length, strength and effectiveness of the support, mining method, sequence and dimensions.
- **The resultant** conditions, which contain the height of softening, displacement and stress magnitudes that are resulted due to one or combination of given and/or response conditions.

As the given conditions cannot be changed, the aim of a roof support design should therefore be to improve the responses so that the resultant conditions are controlled and the roof design is successful in preventing the unexpected falls of ground.

It has been widely accepted and reported in coal mines that all the above parameters vary within a short distance in a panel. The roof stability is strongly dependent on these varying properties of the roof support system. This variation can be taken into account using either stochastic modelling or sensitivity analyses in deterministic models. However, as there are many parameters that are considered in a design, sensitivity analyses in deterministic models usually fail to demonstrate the impact of this variation. In stochastic modelling, which is a technique of presenting data or predicting outcomes that takes into account a certain degree of randomness, or unpredictability, these variations of the input parameters are included. It is therefore possible to quantitatively represent the uncertainties. This method is usually used in probabilistic design approaches in which it is acknowledged that realistically there is always a finite chance of failure, although it can be very small. This approach is considered to be a step forward in the design of coal mine roof support systems and therefore is utilised in AAMC’s roof support design methodology to take into account the uncertainties that exist.
Stochastic modelling allows the input parameters to be taken as probability distributions rather than single values using the well known Monte Carlo simulation method. In this method, the distribution functions of each stochastic variable must be estimated. From each distribution, a parameter value is sampled randomly and the value of the performance function calculated for each set of random samples. If this is repeated a large number of times, a distribution of the performance function is obtained. Monte Carlo simulation is thus a procedure in which a deterministic problem is solved a large number of times to build a statistical distribution. It is simple and can be applied to almost any problem and there is practically no restriction to the type of distribution for the input variables.

A fundamental aspect of the Monte Carlo method is the process of explicitly representing the variations by specifying inputs as probability distributions. By describing the process as a probability distribution, which has its origins in experimental/measurement continuous data, an outcome can be sampled from the probability distributions, simulating the actual physical process/measurement.

This process requires a collection of actual measurements and determining the best fits to the data using the goodness of fit tests (GOF). GOF tests measure the compatibility of a random sample with a theoretical probability distribution function. Three most common GOF tests are (EasyFit, 2008):

- Kolmogorov-Smirnov
- Anderson-Darling
- Chi-Squared

The details of the probability distributions, GOF tests and random selection of design parameters can be found in readily available statistical modelling and reliability engineering publications (e.g., Harr, 1987 and Wolstenholme, 1999).

The stochastic modelling technique has been widely used in Civil Engineering, mainly in slope stability analyses. The development has not yet reached this point in the field of underground roof support design. Possible reasons for this were (i) defining a model which describes both the strength and the load acting on rock and (ii) extensive input parameters required in the analyses. Considering that AAMC underground operations collect a vast amount of geotechnical and geological information and the roof behaviour is well understood, it may be possible to apply the stochastic modelling in the design of roof support systems.
EVALUATION OF AAMC’S ADVANCED ROOF SUPPORT DESIGN METHODOLOGY

The above summarised roof support design methodology has been applied to a well-defined case study at Moranbah North Mine (Figure 3). The 2.0 m wide, 63 t shields rated at 1 750 t installed in August 2009 required a large installation road for LW 108. The install road width varied from 9.5 to 10.7 m increasing up to 11.5 m wide towards the tailgate (TG) end and up to 10.7 m wide towards the maingate (MG) end (Figure 4).

Figure 3 – MNC mine plan
In the past, the longwall install roads at MNC have been stable with slow long-term creep depending on stand up time before longwall installation. Past experience also indicated that the following factors are significant in an install road support design at MNC:

- Stress direction and magnitude/face road orientation;
- Development displacements and height of softening;
- Required roadway dimensions;
- Roof stratigraphy;
- Structures;
- Stand up time before longwall installation;
- Primary support density;
- Sequence of widening/floor brushing etc;
- Primary and secondary support positions;
- Monitoring regime;

In general, a complete install road roof support design should consider the following areas:

- Install road
- Wide areas in the TG and the MG sides of the install road (i.e. shearer stable, MG shields etc)
- Three-way intersections within the install road (i.e. cut-throughs (C/T) between the bleeder road and the install road)
- MG and TG intersections (including stubs), and
- Wide areas due to offline cutting or for another reason.

In the case of a back bleeder road (as in this case), the stability of the pillar located between the install road and the bleeder road should also be considered, as an under designed pillar may result in increased levels of roof deformation in the install road.

In addition, a Trigger Action Response Plan (TARP) should be developed for each install road to ensure a timely and quality response to changing conditions.
INPUT PARAMETERS USED IN THE ROOF SUPPORT DESIGN FOR LW 108 INSTALL ROAD

In order to conduct AAMC’s roof support design methodology, the following input parameters are required.

- Stress environment (principal, intermediate and minor stress directions)
- k-Ratio (horizontal stress to vertical stress ratio) to be used in numerical modelling
- Depth of cover
- Roadway width
- Height of fracturing/softening into the roof
- UCS of rock at long tendon anchorage horizon
- Elastic modulus at long tendon anchorage horizon
- Fracture spacing at long tendon anchorage horizon
- Development displacements
- Roof bolt capacity
- Long tendon capacity
- Roof bolts’ pre-tension
- Long tendons’ pre-tension.
- Coefficient of friction between the layers (assumed to be negligible in this case)
- Unit weight of immediate roof and overburden
- Bond strength of long tendons

It is evident from the above list that there are many input parameters that need to be taken into account in the design and almost all of these parameters inherently vary. In addition, experience has shown that the support installation practice as well as the performance of support consumables can also vary significantly and ideally, their variation should also be taken into account in the design. It is therefore considered that it is nearly impossible to conduct a sensitivity analysis on all of these parameters using a deterministic design approach.

Immediate Roof

MNC extracts the Goonyella Middle (GM) Seam within the Bowen Basin Coalfield, Central Queensland. The seam thickness varies from 5.0 to 6.5 m; the development thickness in the gateroads, main headings and the install roads is approximately 3.5 m. The cover depth of current MNC workings varies from 100 to 300 m; the cover depth associated with LW 108 install road is approximately 300 m.

The roof of MNC is generally characterised by a weak immediate roof overlain by moderately bedded, stronger siltstone and sandstone units. Figure 5 shows the general stratigraphic column in and around LW 108 install road.

An analysis of 58 boreholes over MNC current and past workings indicates a relatively consistent Coal Mine Roof Rating (CMRR) of 40. Based on the study conducted by Molinda and Mark (1994), this roof can be classified as “moderate to weak”. A CMRR value of approximately 40 is also a reasonable estimate for the LW 108 install road.

As part of routine geotechnical investigations, MNC geotechnical / geology department conduct numerous laboratory tests on roof and floor samples. These indicate that the UCS of the sandstone unit at long tendon anchorage horizon varies from 15 to 50 MPa with an average of approximately 30 MPa (Figure 6). The fracture spacing of this sandstone unit has an average of 374 mm (Figure 7) with a variation of 43 mm to 1 200 mm. Figure 8 shows the available elastic modulus test results in this database with respect to the target GM Seam.
With regard to the unit weight of the immediate roof, the laboratory test results indicate a unit weight of 0.018 to 0.026 MN/m³ for the immediate roof horizon (i.e., within 6.0 m top of the seam). In addition, it is assumed in the calculations that the overburden will have a constant unit weight of 0.025 MN/m³.

**Height of Softening**

Experience gained in previous longwall install roads at MNC indicates that the height of roof softening (the height into the roof where the deformation/separation is minimal) may increase to 4.5 m (on average) into the roof. Table 1 summarises the sonic probe extensometer measurements obtained for LWs 105, 107, 201 and 202 where detailed roof and rib monitoring programmes were carried out. It is evident from this table that in all previous cases the height of softening was equal or greater than the primary roof length of 1.8 m.

**Stress Environment**

The information regarding the stress environment is required in numerical modelling as well as in determining the Stress Concentration Factor (Gale and Matthews, 1992) to estimate the anticipated horizontal stress levels in the roof in and around the install road. The aim of numerical modelling in this case is to verify that the empirically calculated stress levels in analytical models are in accordance with numerical modelling.
Figure 6 – Distribution of UCS from the GM Seam floor

Figure 7 – Fracturing spacing distribution at long tendon anchorage horizon
Figure 8 – Distribution of roof elastic modulus data with respect to the GM Seam

Table 1 – Summary of height of roof softenings measurements obtained in previous longwall install roads at MNC

<table>
<thead>
<tr>
<th>Hole ID</th>
<th>Initial Displ. (mm)</th>
<th>Final Displ. (mm)</th>
<th>Height of Softening (m)</th>
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<tbody>
<tr>
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<td></td>
<td></td>
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<td>105.80</td>
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</tr>
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<td>4.5</td>
</tr>
<tr>
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Gale and Matthews (1992) linked the stress concentration factor (SCF) with the angle between the gateroad drivage direction and that of the major horizontal stress as shown in Figure 9. This model is also utilised to calculate the anticipated horizontal stress levels in the buckling failure model.

In order to determine the stress environment, a series of stress measurements were conducted at MNC. Figure 10 shows the details of the compiled MNC’s stress map. This figure indicates a strong NNE-SSW stress direction across the MNC, which is consistent with regional Bowen Basin experience. The stress measurement data also indicates a stress orientation of approximately 10° East of North for MNC.

A summary of MNC in situ stress measurement data is presented in Table 2. Using this data it is possible to calculate the tectonic stress and the major horizontal stress using the methodology given by Nemcik, Gale and Mills (2005) to utilise in buckling failure model.

![Figure 9 – Relationship between SCF and angle of gateroad to stress direction (after Gale and Matthews, 1992)](image-url)
A series of short encapsulated pull tests were conducted at MNC as part of the quality control procedures. The long tendon tests were conducted using 4 m long cables with 300 mm encapsulation. The roof bolt short encapsulated tests were using the standard roof bolt length of 1.8 m with 300 mm encapsulation. The results indicated that:

- long tendon pull out resistance varies from 0.3 to 1.43 kN/mm (calculated as the maximum load achieved/encapsulation length) with an average of 1.2 kN/mm.
- roof bolt pull out resistance varies from 0.3 to 0.6 kN/mm

Based on these variations and also that an initial analysis indicated that the impact of this variation is insignificant, the pull out resistance of roof bolts and long tendons are entered as single values of 0.3 kN/mm rather than probability distributions. It should however be noted that in areas where the pull out resistance is critical, bond strengths of roof bolts and long tendons should also be entered as probability distributions.

Probability Distributions of Input Parameters

As mentioned above, the roof support design methodology presented herein requires the input parameters as probability distributions rather than single values. In order to determine the representative probability distributions of input parameters, a series of GOF tests were run for each parameter. A summary of the GOF test results is summarised in Table 3.

Note that the results presented in Table 3 are based on a limited number of data points and the limits of the software utilised to conduct the Monte Carlo simulations. Therefore, some of the best fit probability distributions obtained from GOF tests are only marginally better than the others. It is also of note that in some areas, the roadway width was slightly wider than the planned 9.5 m, therefore the probability distribution for roadway width is assumed.

It should also be noted that the roof bolt and long tendon pre-tensions of 50 kN and 150 kN are also assumed in the calculations respectively.
Table 2 – Summary of *in situ* stress measurement data

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Table 3 – Summary results of GOF tests

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<th>Parameter</th>
<th>Representative probability distribution</th>
<th>Scale Parameter</th>
<th>Shape/location Parameter</th>
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<td>Fracturing spacing</td>
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<td>UCS</td>
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<td>Unit weight</td>
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SUPPORT DESIGN AND EVALUATION

Numerical Modelling

AAMC has developed mine-wide numerical modelling layouts for all underground operations for detailed geotechnical investigations. An example of MNC’s modelling layout is shown in Figure 11. The aim of numerical modelling in this study was to demonstrate that the magnitudes of horizontal stress notching calculated using empirical modelling (i.e., the methodology of Gale and Matthews, 1992) are not significantly different than numerical modelling and also the fact that the surrounding mining may have an impact on the magnitudes of the horizontal stress notching. In order to achieve this comparison, a detailed elastic numerical modelling study was conducted using Map3D.

Map3D is a 3-dimensional (3D) fully integrated stability analysis package based on indirect boundary element numerical modelling computational method. It is used extensively in mining applications for stress and displacement analysis. The elastic version of Map3D incorporates simultaneous use of fictitious force (FF) and displacement discontinuity (DD) boundary elements. This facilitates the definition of vast mining areas, where computing resources can be optimised by using DD elements for large mining areas away from the areas of interest and then using FF elements to construct detailed and true representation of the three dimensional mining geometry at the areas of interest (van Wijk, 2009). The input parameters used in the numerical modelling study are summarised in Table 4.

Table 4 - Map3D modelling input parameters

<table>
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<th>Parameter</th>
<th>Value</th>
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<tr>
<td>Elastic Modulus – Coal</td>
<td>3 GPa</td>
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<td>Poisson’s Ratio – Host Rock</td>
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<td>Poisson’s Ratio – Coal</td>
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<tr>
<td>Vertical Stress Gradient</td>
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<td>Major Horizontal to Vertical Stress Ratio</td>
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<td>Major Principal Stress Trend</td>
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</table>

Figure 12 shows the numerical modelling results at the final stage (following the development and widening) of the install road. Note that the grid plane where the results are shown is located approximately 2.0 m into the roof. It is evident from this figure that stress magnitudes of approximately 20 MPa are reasonable to expect in this case.

Using (i) the stress measurement data, (ii) the methodologies of Nemcik, Gale and Mills (2005) and Gale and Matthews (1992) and (iii) same input parameters used in numerical modelling, an average stress level of 20 MPa is also obtained in this study to utilise in the buckling failure model. Based on these results it is concluded that the results obtained from the empirical model is in accordance with numerical modelling.
Analytical Modelling

As mentioned previously, AAMC roof support design methodology requires that all four analytical models (i.e., shear, buckling and suspension tendon and suspension bond failures) are run in a critical area in order to ensure the stability of roof. In the case of LW 108, all areas of the installed road, including the TG and MG intersections and wide areas were evaluated. However, for the sake of simplicity of demonstration, only the standard 9.5 m wide area is presented in this paper. The probability distributions of input parameters used in analytical models are presented in Figure 13.

Figure 14 shows the probability distribution of expected horizontal stress notching. This indicates that stress magnitudes of up to 40 MPa may be expected.
An initial evaluation of required roof support densities was conducted using the analytical modelling. This evaluation study indicated that for 9.5 m wide areas of the install road, the following support patterns provided an acceptable distribution of factors of safety (Figure 15):

- First pass primary support development: 6x1.8 m long X-grade roof bolts installed at 1.0 m spacing.
- First pass secondary long tendon support: 3x8.1 m long tendons (nominal 48 t) installed at 2.0 m spacing.
- Second pass primary support: 6x1.8 m long X-grade roof bolts installed at 1.0 m spacing.
- Second pass primary support long tendon support: 2x6 m long tendons (nominal 45 t) installed at 2.0 m spacing.
- Second pass secondary support: 1x8.1 m long tendon (nominal 48 t) support installed at 2 m spacing.

The distributions of the factors of safety for different failure mechanisms are presented in Figures 16 to 19. Using the average values of input parameters, this support pattern indicated the following average factors of safety:

- Buckling failure 3.73
- Shear failure 1.60
- Suspension tendon failure 1.52
- Suspension tendon bond failure 2.91
Figure 13 – Probability distributions of input parameters

i) Roadway width

ii) Poisson's ratio

iii) Elastic modulus

iv) UCS

v) Fracturing spacing

vi) Displacement

vii) Height of softening

viii) Unit weight of immediate roof
The resultant likelihood of failures (the areas of under the curve of the factors of safety of <1 in distributions of factors of safety) of the overall system and individual failure mechanisms evaluated are as follows:

- Overall system (i.e., failure in any one of the mechanisms) \(3 \times 10^{-6}\)
- Buckling failure 0.0023
- Shear failure 0.00004
- Suspension tendon failure 0.0005
- Suspension tendon bond failure 0.0007

The above results demonstrate that while the average factors of safety of different failure mechanisms failure are acceptable, there is a likelihood of failure of 0.0003% of the system due to one or more of these mechanisms. Although this value of likelihood of failure alone may not indicate the associated risks without the calculations of exposure and financial costs, it demonstrates the fact that factor of safety alone cannot give an indication of exposed risks.

**CONCLUSIONS**

This paper summarises the AAMC’s roof support design methodology, which includes analytical, numerical and empirical modelling. The aim of this so called “combined support design methodology” is to ensure the stability of roadways at AAMC underground operations. This methodology currently uses the deterministic approach (calculation of a single factor of safety). The limitations of this design methodology with respect to using a single factor of safety are presented in this paper.

A summary of an improved design methodology, based on stochastic modelling, is also presented. The main advantage of this methodology is that as the design is based on probability distributions of input parameters, the outcome is based on a distribution of factors of safety rather than a single factor of safety.
A demonstration of application of this approach from Moranbah North Coal Mine is presented. The application of this design methodology to LW 108 install road indicated that while the resultant factors of safety of different failure mechanisms against roof falls using the average values of input parameters are acceptable, there is always a likelihood of failure, even though it is very small. This likelihood of failure may also be used to determine quantitative risks (safety and/or financial) associated with the development, widening and installation of the install road. As this methodology allows the user to determine these associated risks, it is considered that this design methodology based on a stochastic modelling is a step forward in the design of roof support systems.

ACKNOWLEDGEMENT

Anglo American Metallurgical Coal is acknowledged with gratitude for the permission to publish this paper.
Figure 16 – Probability distribution of buckling failure mechanism (9.5 m wide areas)

Figure 17 – Probability distribution of shear failure mechanism (9.5 m wide areas)
Figure 18 – Probability distribution of suspension long tendon failure (9.5 m wide areas)

Figure 19 – Probability distribution of suspension bond failure (9.5 m wide areas)
REFERENCES


AMCMRR – AN ANALYTICAL MODEL FOR COAL MINE ROOF REINFORCEMENT

Mark Colwell\(^1\) and Russell Frith\(^2\)

**ABSTRACT:** An analytical model for coal mine roof reinforcement (AMCMRR) has been developed. AMCMRR utilises a Factor of Safety (FOS) approach which is commonly used in all forms of engineering. The starting point in the development of AMCMRR was an existing analytical roof behaviour and roof support design methodology/model originally developed by the second author. This technique was being successfully utilised in the Australian underground coal industry for roof support evaluation and design prior to and during the course of the ALTS 2006 research project and when used was essentially calibrated on a site by site basis.

It has long been recognised that bolts and longer tendons can modify the behavior and load bearing capacity of the reinforced roof via the concept of *beam building*. The break-through in the development of AMCMRR was combining the original analytical model with the ALTS database to effectively quantify this reinforcing effect and this in turn provides the “platform” by which this analytical model can be calibrated for the entire Australian underground coal industry.

The two techniques (analytical-AMCMRR and empirical-ALTS 2009) now work together as a part of an overall and more robust roof support/longwall gateroad design methodology. This paper focuses on the application and use of AMCMRR and the analyses undertaken to quantify the reinforcement offered to the immediate roof via the concept of beam building.

**BACKGROUND**

At the beginning of 2006, Colwell Geotechnical Services commenced a research project entitled, “The Future Development and Integration of ALTS & ADRS for Improved Underground Roadway Design”. The project was funded directly by several of the major coal producers as well as individual Australian collieries. The ultimate goal of the ALTS 2006 Project was to provide the colliery Strata Control/Geotechnical Engineer with user friendly and interactive state-of-the-art software tools and subsequent training/support to assist them with their design requirements and in their strata management role. While the software packages were the major deliverables from the project, it was in fact the geotechnical research which was the main focus of the project and drove the development of the software.

A major component of the project was to develop an analytical model for coal mine roof reinforcement that would complement the ALTS Design Methodology (Colwell and Frith, 2009). This has been achieved and the analytical model is called the, “Analytical Model for Coal Mine Roof Reinforcement” or AMCMRR.

The starting point in the development of AMCMRR was an existing analytical roof behaviour and roof support design methodology/model originally developed by the second author (as discussed by Colwell et al, 2008). This technique was being successfully utilised as a consulting tool in the Australian underground coal industry for roof support evaluation and design prior to and during the course of the project. The basis for this model (and for AMCMRR) is that slender beam behaviour or buckling is typically the dominant behavioural mechanism initially occurring within the immediate coal mine roof measures subject to elevated horizontal stress conditions.

Hoek and Brown (1980) reported that studies conducted by Australian Coal Industry Research Laboratories (ACIRL) in the 1970’s (utilising physical models) clearly indicated buckling to be a dominant failure mechanism within the roof and floor of a layered deposit subject to elevated horizontal stress conditions. Uncontrolled roof behaviour of this type may then lead to other failure mechanisms occurring and to large scale roof displacements or roof falls.

\(^1\) Principal, Colwell Geotechnical Services  
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It follows that this dominant failure mechanism (i.e. buckling) should be accounted for in any empirical, analytical or numerical approach to coal mine roof support design. It simply cannot and should not be ignored. While it has long been recognised that bolts and longer tendons can modify the behaviour and load bearing capacity of the reinforced roof via the concept of beam building (Mark, 2000), the major problem faced by the authors in the development of AMCMRR was how to quantify the effect that bolts and longer tendons have on the load bearing capacity of the roof strata via said concept.

MODEL OVERVIEW

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

\[
FOS = \frac{\text{load bearing capacity of the structure}}{\text{applied load}} \quad (1)
\]

This approach is commonly used in coal pillar design worldwide with the UNSW Pillar Design Procedure (Galvin et al, 1999) being one such example. In this case the structure is the coal pillar where the strength of the coal pillar is given by a specific equation that has been determined empirically, based on an industry database of stable and failed pillar cases, typically under reliably inferred full tributary area loading conditions. The FOS is essentially a risk based measure of the likelihood of the design being inadequate with acceptable values being related to the likely consequence of the design being inadequate and the associated impacts (business, safety or otherwise).

In using AMCMRR it is critical that the user understand that the Factor of Safety has the following general definition:

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

It is not a FOS against a roof fall occurring as (a) the conditions under which a roof fall finally occurs are not well defined and (b) practical mining considerations requires that the roof be maintained as stable as possible during longwall retreat so as to minimise any potential impact on face production. Furthermore productivity and safety can be adversely affected by simply excessive roof convergence trapping equipment or deteriorating roof conditions necessitating the installation of additional roof support. Risk-based mining considerations dictate that designing roadway roof support against the occurrence of a roof fall is inappropriate as we need to be designing roof support against the triggering of the colliery's Trigger Action Response Plan (TARP).

The consequence of an inadequate design when using the above-defined FOS is logically the triggering of the colliery's TARP as a part of its strata management plan and the installation of additional support. It is not the imminent occurrence of a major roof fall and this always needs to be kept in mind when considering the actual magnitude of an adequate design Factor of Safety.

Consistent with the above, AMCMRR does not incorporate standing support (i.e. timber cribbing, link-n-locks, tin cans etc) as standing support is not used to reinforce the rock mass in the same way as encapsulated/tensioned bolts and cables. In terms of standing supports' contribution to overall roof stability/roadway serviceability (as a part of a roof support strategy); standing support is, in practical terms, utilised to prevent a potentially failed rock mass becoming a fall. It therefore “supports” rather than reinforces the rock mass.

It is noted that cables can also act in a similar manner to standing support, in that they can support a “dead weight” load where the rock mass has yielded or has essentially lost its ability to resist the applied horizontal stress. Some practitioners still design bolt/cable installation patterns in this manner where the dead weight load is directly related to an assumed height of softening. However this is an example of designing against a roof fall because if the design fails, the roof falls in. AMCMRR evaluates roof stability and support requirements much earlier in the instability process when reinforcing support can be applied. It is the authors’ contention that this is vastly preferable in terms of tendon support (i.e. reinforcing) effectiveness and more importantly, design reliability and robustness.
Within AMCMRR, bolts and cables are considered within the design process (and in the calculation of the FOS) to firstly increase the load bearing capacity of the strata (which is limited by the material's yield strength) and where longer tensioned cables are employed they also contribute to overall roof stability by directly resisting the applied load via the concept of mechanical advantage (MA – Frith, 2000). AMCMRR provides for a static load-balance design, where the goal is to design a tendon roof support strategy which allows the reinforced roof to maintain the horizontal stress acting across the roof so as to prevent the onset of a process of delamination and/or buckling that (otherwise left “unchecked”) could lead to a major roof fall.

As previously indicated; in terms of roof support design in some instances factors of safety have been calculated based on the height of softening and a related dead weight load supported by the cables. However it is important to note that in this instance the “structure” under consideration when calculating the FOS is in fact the cables not the roof. Also with this approach it is accepted that the horizontal stress has exceeded the yield strength of the roof material and essentially a “failed” rock mass is being suspended by the cables.

Whilst it is far less common to do so, there is no obvious technical reason as to why roof stability in mine roadways cannot be evaluated and designed for where the reinforced roof is the structure under consideration in equation 1. The problem has always been in being able to reliably assign magnitudes or quantities to the various components of the equation. However as with the approach taken for coal pillar design (i.e. the use of an empirically derived strength equation rather than one based on first principles), industry or individual mine site experience can potentially be used to “calibrate” various elements of the problem and so allow an industry or site specific FOS approach to be adopted.

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

\[
FOS = \frac{\text{load bearing capacity (reinforced roof strata) + load resistance via MA}}{\text{applied load}}
\]

As will be discussed, the load-bearing capacity of the reinforced roof strata is a function of the installed roof support and the roof strata itself. It is noted that this combination of the roof strata and roof support into a single load-bearing entity (as a part of equation 2) is not the same as reinforced concrete for example. The use of steel reinforcement in concrete is specifically aimed at overcoming concrete’s poor performance in tension rather than increasing its load bearing capacity in terms of compression. In the case of the analytical model, the applied load is compressive as a result of the horizontal stress acting. The roof support via the roof reinforcement mechanism of beam building increases the load bearing capacity of the strata (up to its yield strength) to resist the horizontal compressive forces and so limit vertical strata movements.

APPLICATION OF THE MODEL

The analytical design model is focused primarily on the design of primary and secondary tendon roof support for the gateroads associated with longwall mining. Figure 1 is a plan schematic of a typical Australian longwall mining layout, depicting a fully extracted longwall panel, one currently being extracted and a third where the twin gateroads (A and B Headings) are still to be completed to fully delineate the longwall panel.

AMCMRR deals with those cases where the horizontal stress acting can be realistically calculated (as a result of industry research) and importantly the roadway width remains reasonably constant. Therefore the analytical model cannot currently be used with respect to tailgates subject to double (or 2\text{nd}) pass longwall extraction and ALTS 2009 should be employed as state-of-the-art design for such cases. Realistic estimates of the horizontal stress acting is a critical input parameter and with respect to tailgates subject to double (or 2\text{nd}) pass longwall extraction it is assessed that further industry research is required in relation to this issue. Also effective roadway widening due to rib spall/softening is far more variable when a ribline has been subject to both maingate and tailgate loading (Positions c and d – Figure 1).
AMCMRR considers the following four cases:

1. Roadway development and the maintenance of a stable roof prior to any increase in horizontal stress as a result of adjacent longwall extraction (position a – Figure 1).

2. Maingate belt road roof support design to deal with the increase (or notching) of the in situ horizontal stress about the belt road intersection with the travelling longwall face (position b – Figure 1).

3. A tailgate where there is no adjacent goaf; for example where the tailgate is the first in a series of longwall panels (e.g. Tailgate 1 – Figure 1). In this instance the tailgate roof is subject to an increase (or notching) of the in situ horizontal stress, which is referred to as a Tailgate - Single Stress Notch.

4. The 4th scenario is a transient, but not uncommon situation where a tailgate roof is subject to a Super Stress Notch (position e – Figure 1). To occur, the longwall commences inbye of start-line of the previous LW panel, in this case LW 2 in relation to LW 1. Significant horizontal stress increase occurs as the faceline of LW 2 approaches and passes the start-line (or installation face) of LW 1.

In terms of the use and application of AMCMRR there are four basic components:

1. Evaluation of horizontal stress acting across the roof within individual roof units at various key points in the mining process (Colwell et al, 2008 and Colwell & Frith, 2009).

2. Determination of the material properties (including Young's modulus, Poisson’s ratio, UCS as well as “beam" thickness and length) associated with the immediate roof units, which are required both in terms of Point 1 above and in evaluating the load bearing capacity of the reinforced roof strata.

3. Determining the reinforcing effectiveness of the installed roof support (bolts and/or long tendons) in terms of their impact on controlling the horizontal stress acting. For primary roof bolts this is done by evaluating their ability to increase the effective beam thickness of the roof strata within the bolted interval. For long tendons they are evaluated both in terms of their contribution to beam building within the bolted interval and the control of buckling via the concept of Mechanical Advantage (Frith, 2000).

4. Utilising a load-balance approach (which incorporates aspects of slender beam behaviour and mechanical advantage) the FOS is calculated. Engineering judgement needs to be applied in
selecting a suitable FOS for design purposes this being a risk-based consideration that is always discussed with mine management as part of finalising design outcomes.

The following specifically discusses the analyses undertaken to quantify the reinforcement offered to the immediate roof via the concept of beam building.

LOAD BEARING CAPACITY

As previously discussed, roadway roof behaviour and associated instability is primarily based around the uncontrolled buckling of slender horizontal beams under the action of horizontal stress. Therefore at “its core” the analytical model is based on slender beam behaviour and that the bolts and cables can modify that behaviour via the roof reinforcement mechanism of beam building. The concept being that the bolts and cables create “thicker” beams within the reinforced section (or the bolted interval) and that a thicker beam will have a greater load bearing capacity than a thinner beam.

A clear example of this concept of beam building is demonstrated in Figure 2, which illustrates the behaviour (or response) of a section of main gate roof during and subsequent to longwall retreat. Under the action of horizontal stress, bedding and or weakness planes can be forced apart and thinner discrete beds or beams of roof material start to form. This inevitably results in discernible roof displacements and roof softening (i.e. delamination) for a distance into the roof.

The dashed horizontal line on Figure 2 represents the top of the 1.8m primary bolted interval and there is an obvious difference in roof behaviour at this location within the roof. The response of the roof within the bolted interval is that of thicker beams as compared to the roof material overlying this interval up to the extent of the roof softening, which is approximately 4m.

![Figure 2 - Roof behaviour adjacent to longwall extraction](image)

The starting point in developing a relationship to quantify the reinforcement concept of beam building is first to understand the behaviour and load bearing capacity of the unreinforced rock units (designated as $P_{\text{unit}}$) within the immediate roof and the input parameters required. Once this is understood a relationship is then required which relates the reinforced beam thickness (RBT) to the initial or unreinforced beam thickness which is referred to as the effective fracture spacing ($F_{\text{eff}}$). The $F_{\text{eff}}$ for an individual rock unit is derived from the individual rock Unit Rating, which is a part of the Coal Mine Roof Rating (CMRR) calculation (Colwell, 2009).
Evaluating the Load Bearing Capacity of the Unreinforced Roof Units ($P_{unit}$)

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

Behaviour outside the Euler range can be defined by a number of different structural concepts. For the purpose of this model use will be made of what is termed as the Johnson formula (see Foley, 2009 or Beer, Johnston and DeWolf, 2006 for more general information on this topic). Utilising these concepts of beam behaviour under axial load in conjunction with the roof’s material & physical properties an estimate of its load bearing ability ($P_{unit}$) can be deduced.

Structural elements that are loaded axially are generally referred to as columns and in terms of buckling have typically been divided into three general types:

(i) Short (stumpy) columns
(ii) Intermediate columns, and
(iii) Long (slender) columns.

Long (slender) columns fail by buckling i.e. large lateral deflections and the failure is elastic. A short beam will not fail due to buckling, as the ratio of the beam length to the effective cross sectional area is too small. Rather a short, ‘thick’ beam, axially loaded, will fail in simple compressive failure; that is when the load/area of the beam exceeds the allowable stress. Columns of intermediate slenderness exhibit a combined failure mode involving both yielding and large lateral deflections.

The critical or allowable stress associated with long beams/columns is governed by equation 3 (Euler Formula).

$$\sigma_{crit} = \pi^2 \frac{E}{[12(L_{eff}/d)^2]}$$  \hspace{1cm} (3)

where $E$ is Young’s Modulus, $L_{eff}$ is the effective beam length and $d$ is beam thickness.

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

There are a number of semi-empirical formulas for buckling in beams/columns in the intermediate length (and short) range. One of these is the J.B. Johnson Formula (refer Foley, 2009). The J.B. Johnson formula is the equation of a parabola with the following characteristics. For a graph of stress versus slenderness ratio, the parabola has its vertex at the value of the yield stress on the y-axis. Additionally, the parabola is tangent to the Euler curve at a value of the slenderness ratio, such that the corresponding stress is one-half of the yield stress.

The Johnson equation for the allowable stress is as follows:

$$\sigma_{crit} = [1 - (L_{eff}/f)^2/(2 C^2)] \sigma_y$$  \hspace{1cm} (4)

Where, $f$ is the beam’s Radius of Gyration and $C$ is the beam’s critical slenderness ratio

$$f = (\frac{I}{A})^{0.5} \text{ and } C = (2 \pi^2 \frac{E}{\sigma_y})^{0.5}$$

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

Essentially when the beam’s slenderness ratio ($L_{eff}/f$) is greater than the beam’s critical slenderness ratio ($C$) then equation 3 is used to calculate the beam’s load bearing capacity and when the beam’s slenderness ratio less than $C$ then equation 4 is invoked.
Therefore in undertaking these analyses the information required is modulus (E) and \( \sigma_y \) (where \( \sigma_y = 0.7 \times \text{UCS} \)) of the rock unit as well as the beam’s effective length (\( L_{\text{eff}} \)) and thickness (d). In terms of the individual beams that will form within the roof; firstly it is assumed the end fixing condition is pinned and therefore \( L_{\text{eff}} \) equals the roadway width and secondly the beam thickness is equal to \( F_{S_{\text{eff}}} \).

**Evaluating the Load Bearing Capacity of the Reinforced Roof Units (RBT)**

As previously discussed the analytical model is based on slender beam behaviour and that the bolts and cables can modify that behaviour via the roof reinforcement mechanism of beam building. The concept being that the bolts and cables create “thicker” beams within the reinforced section (or the bolted interval) and that a thicker beam will have a greater load bearing capacity than a thinner beam. Therefore in terms of an equation this concept is expressed as:

\[
RBT = f \left( F_{S_{\text{eff}}} \& \text{a measure of the installed level of roof support} \right) \tag{5}
\]

Essentially the only practical way that the above relationship can be established is empirically via back-calculation of a database with reasonable assumptions made. It was decided to use the ALTS primary roof support database (comprising 109 cases) in an attempt to establish this relationship. Primary support is defined as roof and rib support which is installed off the continuous miner or off a mobile bolter as part of a cut & fill operation. The main reasons for using the primary roof support database were:

1. There is far less variability with respect to the installation of primary bolts as compared to secondary cables. A very large proportion of the primary bolts now utilised in Australian are of x-grade (or close to x-grade) steel, are installed off the miner near the face as part of a “cut and bolt” operation with a two-speed resin system and are then tensioned within a range of approximately 5 t to 10 t. Furthermore by far the predominant lengths used in Australia are the 1.8 m and 2.1 m bolts (comprising 96 of the 109 cases).

2. As indicated above, in Australia the vast majority of primary support is installed near the face as a part of a “cut and bolt” operation. While the distance between the last line of support and the face may vary it will be within an operationally viable range and therefore similar across the industry. Furthermore there is some level of flexibility in that distance at a mine site level where for example the miners will have the option (if the ground conditions warrant) to “close up” that distance to limit roof displacement at the time of installation. The level of roof movement at the time of installation is a critical aspect with respect to the effectiveness of roof reinforcement and it is assessed that the level of roof movement associated with the installation of primary support is far less variable (across the industry) when compared to that of longer cables.

3. There is greater confidence with respect to the estimate for the horizontal stress acting perpendicular to the roadway orientation on development (i.e. \( \sigma_{R-\text{Dev}}, \text{Colwell and Frith, 2009} \)) as compared to those scenarios associated with a notching of the horizontal stress during longwall retreat (e.g. maingate stress notching).

Prior to undertaking the analyses a logical reduction of the primary roof support database was necessary to both simplify the analyses and to recognise certain geotechnical limits. Furthermore one important assumption was required as a part of that process. The assumption being that for development, Australian longwall operations typically aim to achieve a minimum reinforced roof FOS of approximately 2 (albeit unknowingly in most cases as there is no FOS based design for primary support to refer to) and that the eventual trendline associated with the database analyses will represent a reinforced roof FOS = 2. Experience would suggest that the Australian industry prefers “Table Top” roof conditions on development and a minimum reinforced roof FOS of 2 would reflect that choice.

While this assumption plays a crucial role in terms of the analyses it is not a critical assumption in terms of utilising AMCMR. It has always been the authors’ intent that the analytical model resulting from this project should initially be used and calibrated on a site by site basis. This in fact is typical of how many (if not most) analytical and numerical models are utilised by experienced geotechnical engineers. Furthermore any suggested Factors of Safety emanating from the database analyses will be relative to this assumption. The database was reduced in the following manner:

1. It was decided to initially only include those cases where the individual units within the bolted interval were geotechnically very similar i.e. similar UCS and \( F_{S_{\text{eff}}} \), such that the individual rock Unit Ratings
were similar. This ensured that the reinforcement mechanism of beam building was predominantly being analysed and not being complicated by other roof reinforcement mechanisms such as when a bolt is anchoring into a “strong bed”. This reduced the database from 109 to 97 cases.

2. While the assumption is that collieries aim for a reinforced roof FOS of 2, in reality there a quite a number of collieries where the unreinforced roof FOS on development (i.e. $\frac{P_{unr}}{\sigma_{R-Dev}}$) is already greater than 2 and to a degree the bolts (in terms of overall roof stability on development) are essentially simply “holding the roof mesh in place”. It was found that this typically related to those collieries where the CMRR > 50 (i.e. the strong roof category). Including such collieries in the analyses, where the unreinforced roof FOS > 2, is simply of no benefit and actually detrimental to the analyses in terms of developing a realistic relationship between RBT and $F_{Seff}$ & the installed level of roof support. Furthermore to ensure that there is essentially a “gap” between the unreinforced and reinforced roof FOS, it was decided to eliminate those cases where the unreinforced roof FOS > 1.5. This reduced the number of cases from 97 to 75 cases.

3. There are those instances where no matter what level of roof support is installed the reinforced roof FOS cannot be greater than two (2) due to the limiting factor being the yield strength of the material. For example if the rock unit’s UCS is 20 MPa then its yield strength is taken to be 70% of the UCS or 14 MPa and if $\sigma_{R-Dev} \geq 7$MPa then independent of the number of bolts installed (and the resultant RBT), the fact is that in this situation the reinforced roof FOS can never be greater 2 as the reinforcement offered by the bolts cannot make the material stronger than its yield strength. Accordingly it was decided to eliminate those cases where 2 times $\sigma_{R-Dev} \leq 0.7$ UCS. This reduced the database from 75 to 60 cases.

Finally only the “headings” section of the primary support database was used as the beam length is a critical factor in terms of the back-analyses. While there is relatively good control of roadway width with respect to headings throughout the industry, the formation of intersections varies from colliery to colliery and there is significant variation in the effective control of pillar corners. Therefore there are operational issues that also impact on the resultant level of primary support utilised within and adjacent to “intersections”.

Based on the assumptions previously discussed and the logical reduction of the database, it was found that an exceptionally strong relationship ($R^2 = 0.91$) existed between the ratio of RBT to $F_{Seff}$ and the primary roof support rating (PRSUP, refer Colwell and Frith, 2009) which is illustrated in Figure 3. It is worth noting that the individual components of the PRSUP calculation (i.e. length of the bolt, ultimate tensile strength of the bolt and density of the pattern) were also separately and collectively significant predictors of the RBT/$F_{Seff}$ relationship.

When calculating the RBT based on the installed level of primary support (as measured by PRSUP) and $F_{Seff}$ (derived from the geotechnical logging/geomechanical testing of the core) the trendline displayed on Figure 3 can be expressed as:

\[
RBT = 0.28e^{0.05341 \times PRSUP \times F_{Seff}}
\]

In terms of AMCMRR the load bearing capacity of the reinforced rock units (designated as $P_{RBT}$) is then calculated in the same manner as that previously outlined for $P_{unr}$, while now utilising RBT rather than $F_{Seff}$ for the effective beam thickness. Once again it is important to note that irrespective of the resultant RBT the beam’s load bearing capacity will be limited by the material’s yield strength (refer equation 4).

Where additional roof support is installed subsequent to development and prior to longwall extraction then equation 6 is still utilised to calculate RBT, where PRSUP is replaced by the total ground support rating (GRSUP, refer Colwell and Frith, 2009). However it should be noted that where longer cables are installed then their incorporation within the PRSUP and GRSUP calculations has been modified to account for the impact of any additional roof movement subsequent to installation of the primary roof bolts.
DISCUSSION

The reinforcement mechanism or concept of beam building (as discussed by Mark, 2000) associated with the installation of roof bolts has long been recognised in the underground coal mining industry. While numerous researchers (e.g. Peng 1998, Gale et al 1992 and Seedsman et al 2009) have discussed the various mechanisms by which the bolts act to “create thicker beams” (i.e. by maintaining friction on bedding planes etc) this is the first such study that has in a practical way attempted to quantify the beam building effect and then incorporated that effect within an analytical model.

Slender beam behaviour (i.e. buckling) is the dominant failure mechanism within a coal mine roof subject to elevated horizontal stress conditions and should be accounted for in any empirical, analytical or numerical approach to coal mine roof support design. It simply cannot and should not be ignored! So why has it taken so long for a model to be developed which directly includes this mechanism and has also attempted to quantify the increase in load bearing capacity of the reinforced roof via the concept of beam building?

By 1980 it was clear via observation, measurement and the use of physical models that buckling was a dominant failure mechanism associated with a coal mine roof, rib and floor, however its incorporation in geotechnical models associated with coal mine roof and rib reinforcement virtually disappeared for some 15 years in Australia. At that time the geotechnical engineer had essentially four forms of modelling open to them for both design and research purposes being empirical, analytical, physical and numerical. However with the increased power of computers there became (and to some degree there still is) a clear preference within the worldwide rock mechanics fraternity to utilise numerical modelling for research purposes and physical modelling has all but vanished.

Unfortunately numerical modelling (then and now) cannot truly represent the geotechnical environment in sufficient detail so as to take account of dominant behavioural and associated failure/reinforcement mechanisms associated with a coal mine roof and tendon reinforcement. Because of these limitations such models require manipulation of the host rock’s material properties in an attempt to mimic the behaviour of the unreinforced and/or reinforced rock mass. Because numerical modelling could not account for buckling most numerical modellers simply ignored this fundamental mechanism occurring within the roof, ribs and floor. The use of numerical modelling in this manner has actually been
detrimental to advancing our knowledge and understanding of both the unreinforced and reinforced rock mass behaviour.

AMCMRR utilises a FOS approach which is commonly used in all forms of engineering and it was intended that the ALTS database would provide the basis by which certain aspects of the model could be improved and/or calibrated and indicative design FOS values could be provided. The prospect was that in combining these two design/evaluation techniques (empirical – ALTS 2009 and analytical - AMCMRR) an even more robust roof support design procedure would result, this being analogous to the pioneering work of Salamon and Munro (1967), and later the University of New South Wales (Salamon et al, 1996), in developing coal pillar design for bord and pillar workings to its current level of reliability.

CONCLUSIONS

The authors’ experience in developing geotechnical models and design tools for the Australian underground coal industry suggests that the geotechnical environment and the way in which roof and rib support interacts with the rock mass are complex issues and without prudent simplification, the complexity of the problem will overwhelm all current geotechnical methods of modelling. Without question judicious simplification is at the heart of all engineering design as it is in all branches of science (termed reductionism). However the problem should not be oversimplified (i.e. the dominant failure mechanisms or critical data input parameters should not be ignored).

Empirical modelling has its limitations and an important maxim in the application of any empirical model is that the model should only be utilised within the bounds of the database from which it was developed. However, the great advantage of this approach is its firm links to actual experience and a worldwide proven track record in the underground coal industry of providing solutions to complex mining issues in a timely and cost-effective manner and therefore if judiciously applied (as all models should be), it allows for credible design within the rigours of a well constructed strata management process.

The authors contend that a clear understanding of the dominant mechanics of the geotechnical problem under consideration is required before a credible model (utilising any one or a combination of the three forms of modelling) can be developed for geotechnical design purposes. This paper demonstrates that a combined empirical and analytical approach is currently the most practical way of developing credible geotechnical design tools for the Australian or indeed any other underground coal industry.

REFERENCES

MANAGING THE GEOTECHNICAL ASPECTS OF LONGWALL FACE RECOVERY

David Hill¹

ABSTRACT: Longwall face recovery is almost certainly the most involved recurring geotechnical problem faced by operators, with major loss potential should problems occur. The outcomes of an industry sponsored (ACARP) research project on the geotechnical issues associated with conventional longwall recoveries is presented with updates the experiences gained.

A number of critical features of the geotechnical environment, support design and mining geometry have a pronounced impact on ground control during take-off. A model of roof behaviour at the take-off point has been developed and validated. Key geotechnical issues include low roof competency (ie weak roof), an adverse weighting environment, geological structure and horizontal stress concentrations at the gate ends (generally the maingate). All of these are identifiable either at the support design stage or, at worst, prior to the start of powered support removal, which is the critical stage of the take-off process.

Key aspects of the geometry and process are the ability to maintain powered support resistance during bolt-up and take-off, the direction in which the powered supports are removed, the impact of take-off chutes and the speed of the powered support removal process.

The author presents case studies that illustrate these issues and the associated ground behaviour.

THE ACARP PROJECT

Longwall face recovery is almost certainly the most involved recurring geotechnical problem faced by operators, with major loss potential should problems occur. However, unlike most geotechnical aspects of coal mining, very little research has historically been undertaken on the issue of conventional longwall recoveries. In 2006, Strata Engineering completed an ACARP-sponsored research project aimed at defining and minimising the geotechnical threats during take-off (Strata Engineering, 2006).

The project had the overall objective of developing guidelines for the specification of ground control strategies, so as to minimise the likely geotechnical threats relating to the safety, operational costs and production delays associated with the recovery and relocation of a longwall face.

The project commenced with an industry survey of longwall relocation practice. The resulting database covered issues such as the geotechnical environment, support practices and ground control experiences, including any difficulties encountered. Fieldwork aimed at geotechnical characterisation covered a range of environments in NSW and Queensland, drawing also on existing data from a number of mines. Longwall take-off monitoring data was obtained from 24 face recoveries across all the major coalfields. The fieldwork identified a number of critical features of the geotechnical environment, support design and mining geometry that have a pronounced impact on ground control during take-off. A cantilever model of roof behaviour at the take-off point was developed and validated by the data collected. The roof cantilever acts to transfer load to the solid abutment, the primary support element.

Four parameters were identified as the main geotechnical hazards, namely:

i. low roof competency (i.e. weak roof),
ii. an adverse weighting environment,
iii. geological structure and
iv. horizontal stress concentrations at the gate ends in deeper mines (generally the maingate).

All of these are identifiable either at the support design stage or, at worst, prior to the start of powered support removal, which is the critical stage of the take-off process.

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Four aspects of the geometry and process were seen to be particularly significant, in terms of their impact on roof stability and the success of the overall operation, namely:

i. the ability to maintain powered support resistance during bolt-up and take-off,
ii. the direction in which the powered supports are removed,
iii. the impact of take-off chutes and
iv. the speed of the powered support removal process.

The general rule with regard to the geometry of the take-off face and overall process is that the preparation done prior to the start of powered support removal should result in a high degree of reliability in terms of roof control, minimising the likelihood of subsequent difficulty and delay during the shield removal phase. The benefits of a relatively modest amount of preparation (eg in terms of floor repairs or concreting, roof cabling, cavity remediation and shield canopy contact) greatly outweigh the time and costs involved and moreover are dwarfed by the costs related to any remedial support action that inadequate preparation might subsequently necessitate (never mind the associated delays).

Several distinct ground control difficulties were identified by the project, namely goaf tightening / ingress, roof sag and cavity formation in the tip-to-face area. Although often related, the means of addressing these issues can vary (eg regarding the chock removal sequence). In recent years, the application of cables has become common in reactively managing areas of deterioration, as well as pro-actively reinforcing zones of identified potential difficulty.

A number of visual and quantitative triggers can be used to guide strata management. Again, the focus must be on maximising the likelihood of success prior to chock removal, rather than modifying the support and/or process in response to subsequent difficulties. Visual indicators include face breaks, cavities, guttering, rib spall, powered support loading, tendon loading (bolts and cables), as well as difficulties with tendon installation or tensioning. Quantified triggers relate primarily to creep rates and evidence of beam breakdown; guidelines were developed by the project with regard to the likely impact of various creep rates, in the critical period prior to the start of powered support removal.

The project formulated a support design process that provides a framework for managing the geotechnical threats in a pro-active manner, including a mechanism for quantifying support (bolt and cable) requirements. Software developed as part of the ACARP project aids this process, enabling alternative options to be identified and rationalised.

**THE DESIGN PROCESS**

The following summarises the design process applied by Strata Engineering for initial roof support design on the longwall take-off face:

i. Characterise roof competency in the take-off area, specifically in terms of Coal Mine Roof Rating (ie CMRR).

ii. Utilising CMRR and the key geometrical parameters of longwall face width and depth, apply the design equations provided in the ACARP project report to derive an indicative appropriate roof support density (Reinforcement Density Index or “RDI”). Unless quantified local experience suggests otherwise, a minimum probability of success of 0.99 is set (implying a high degree of support system reliability in the tip-to-face area). The software developed during the ACARP project assists in the derivation of RDI.

iii. Using the RDI as input, derive a practical bolting pattern, taking into consideration operational requirements. Typically:
   a. The final tip-to-face distance should be limited to a maximum of 3m (even allowing for the practice of staggering chock positions).
   b. The zone of bolted roof should extend from the final face to within 1m of the rear of the canopies, which typically implies a minimum of seven rows of bolts.
   c. The roof bolts in the tip-to-face area (at least) should be X grade steel.
   d. The bolt density in the tip-to-face area should be ≥ 1 bolt / m².
iv. If the roof is weak (i.e., a CMRR of <45), the support design should almost certainly incorporate systematic cabling in the final tip-to-face area. The overall design RDI should again be used to derive the appropriate cable density. Cables should generally be:
   - at least 6m long,
   - configured to anchor outside of the likely roof failure zone and
   - post-grouted at least 24 hours prior to the start of powered support removal using a high strength thixotropic grout.

v. Specific cable designs should be developed for intersections, including chutes, taking into consideration actual gate road roof behaviour under conditions of longwall extraction, the strategic importance of the excavations involved and the required machine movements (e.g., setting up of buttress chocks at the tailgate intersection).

vi. Specific secondary roof support designs should also be provisionally developed for expected or known areas of geological structure.

vii. Polyester roof mesh (or “geogrid”) should be employed, with a heavy grade (i.e., $\geq 60t$ capacity) suggested from the rear of the canopies to the face (i.e., for the control of both goaf flushing and the tip-to-face area).

viii. The buttress or walker chock configuration should be determined with due regard to the geometry of the powered supports and the take-off area as a whole (e.g., support length and final tip-to-face distance), taking into consideration local experience. Configurations involving three walker chocks are particularly susceptible to goaf side sag and require specific consideration of the support design above the canopies of the line chocks; often this involves additional cabling.

ix. The design of standing support for goaf edge takes into consideration local experiences and operational preferences, captured and formalised in some form of trigger-response plan (“TARP”).

During the actual take-off, this preliminary design should be ratified or refined with the aid of the following:

i. A review of powered support leg pressure monitoring on the approach to take-off, to identify any cyclic loading and to define zones of distinct loading along the face (e.g., heavier conditions towards mid-face).

ii. Mapping to identify and/or confirm geological structures, areas of adverse roof behaviour (e.g., cavities or face breaks) and geometrical issues (e.g., horizon errors, floor steps and poor canopy contact).

iii. Roof monitoring to determine the ongoing rate of displacement (i.e., creep) and any signs of cantilever breakdown.

iv. A review of anticipated versus actual conditions prior to the commencement of powered support removal and the implementation of tertiary support if necessary (e.g., at geological structures and cavity areas).


Finally, the documented LW recovery outcomes, both from an operational and geotechnical perspective, should be reviewed and used to refine future practice.

**RECENT EXPERIENCES**

Since the ACARP project was completed at the end of 2006, Strata Engineering has extended the original database to over 30 mines in Australia and overseas, including over 100 monitored case studies. The expanded database is currently being analysed, with the primary aim of re-assessing the impact of face height and width, as well as the significance of powered support capacity/design and the role of standing support. Future longwall recovery practice must take cognisance of the trend towards wider faces with larger powered supports, as well as increasing depth at some mines.
Of the outstanding issues flagged by the ACARP project report, the appropriateness and use of buttress / walker chocks remains topical. The most common operational difficulty related to ground control on a take-off face is the inability to advance the buttress chocks, due to roof deformation (ie sag, particularly on the goaf side). Goaf side tightening and sag are only issues in the context of the need to be able to continually advance the buttress chocks; without the buttress chocks, the practical impacts of the ground movement would be materially reduced, if not insignificant. This is particularly the case given trends with regard to improved support products (eg stronger mesh and higher roof support densities, including more routine use of cables). In this regard, the few mines which continue to employ systems based on standing support only tend to have a significant advantage, in that in heavy conditions it is usually a simple matter to increase the quantity of support at the goaf edge and continue with chock removal.

This problem can be exacerbated by the use of three as opposed to two buttress chocks. The use of three buttress chocks requires particular attention to roof and goaf edge control in the vicinity of the goaf side buttress chocks; this often involves cabling towards the rear of the canopies. It is sometimes possible to achieve adequate roof control in difficult ground conditions in the tip-to-face area, but still have ongoing problems with goaf side sag. The potential for deterioration on the goaf side during powered support removal is increased in the event of adverse weighting or presence of geological structure, noting that the slow rate of retreat during bolt-up tends to exacerbate the associated impacts.

A second issue flagged by the ACARP project was an increasing interest in the use of pre-driven recovery roads. Since that time, a number of pre-driven recovery roads have been successfully utilised in relatively aggressive ground conditions and difficult circumstances, demonstrating that a reliable methodology does exist (Thomas, 2008). Some of the issues often flagged as specific hazards with regard to pre-driven roadways are also factors influencing the success of conventional recoveries; examples include an adverse weighting environment and weak roof. The relativities of the risks associated with the two methodologies should be appraised on a mine specific basis, with equivalent levels of scrutiny. Along with the increased use of full pre-driven recovery roads in recent years has come a greater interest in the use of partial face-parallel stubs. Relatively short (ie 20m to 60m long) stubs driven along the stop line at either the maingate or tailgate end of the face exploit the protected ends of the face (with locally reduced weighting potential) and offer a number of operational advantages, primarily the removal of a significant portion of the bolt-up task from the critical path of the recovery process. The time required for bolt-up is becoming an increasingly significant issue with the trend to wider longwall faces.

CONCLUSIONS

Future recovery practice must take cognisance of the trend towards wider longwall faces and larger powered supports, as well as increasing depth at some mines. The result will tend to be increased densities of tendon support and a need to review the overall longwall recovery process. The ACARP project provided a rationale for addressing the related geotechnical issues and support design; this foundation is now being updated as a wider range of experiences becomes available.

The challenges presented by the abovementioned operational trends increase the emphasis on finding effective solutions, with a focus on the more efficient use of support elements, as well as strategies to limit the time associated with bolt-up and subsequent shield removal.

REFERENCES

STABILITY ANALYSIS AND OPTIMUM SUPPORT DESIGN OF A ROADWAY IN A FAULTED ZONE DURING LONGWALL FACE RETREAT - CASE STUDY: TABAS COAL MINE

Ali Sahebi¹, Hossein Jalalifar², and Mohammad Ebrahimi¹

ABSTRACT: Stability analysis of a longwall, East 1 tailgate (E1TG) of Tabas underground coal mine is presented. The mine extracts coal by both longwall and room and pillar methods. The mine is designed to produce 1.5 million tonnes of coal annually. The roadways have a rectangular profile of 4.5m width and 3.5m height. The field investigations and the geomechanical characteristics of rocks showed that the rock masses are weak, requiring a suitable support system. The roadway is intersected by a major fault zone. To investigate the effect of the fault zone on roadway stability, in particular during the face retreat, extensive numerical simulations were carried out. It was found that the stability of the tailgate was severely affected by major structures such as faults and crushed zones. In addition to this, it was revealed that the situation gets worse during the face retreat. An optimised support system was determined.

INTRODUCTION

Tabas underground coal mine is located some 85km south of Tabas town, Yazd province, Iran. Figure 1 shows the mine location. This mine is the first fully mechanised coal mine in Iran that produces 4000 tonnes of coal per day.

A 4.13 m long roof core sample taken in E1TG revealed a frequency of siltstone and sandstone layering above the coal seam. The coal seam was up to 2.2 m thick (average thickness of 2 m) dipped in the range 14°–26°. At the end of year 2007 the first longwall panel was commissioned. The longwall face was operated with a double drum shearer and shield supports. The rock mass rating (RMR) based on the roof core and rock mass confinement method (Daws 1992) was used to estimate the appropriate bolting pattern. Prior to bolt installation, the roadways were supported with conventional props and bars, which underwent severe deformation with the approaching longwall face. Figure 2 shows location of the extraction panels (East1 and East2).

LONGWALL MINING IN TABAS COAL MINE

Site description

The E1 longwall panel had a face width of 180 m and panel length of 1200m. The C1 working seam thickness varied from 1.8 to 2.2 m with dip varying between 11° to 26°. The roof of the coal seam contained 0.1 to 0.2 m mudstone, siltstone/sandstone interfaces and sandstone. The C1 seam had a uniaxial compressive strength of less than 5 MPa. The other seams in the vicinity of the C1 seam were C2 and D1 above and B1 and B2 below (IRITEC 1992). A 4.13 m long roof core taken in E1TG at MM of 180.7 revealed sequential layers of siltstone, sandy siltstone and silty sandstone immediately above the roof in the tailgate. The core data is summarised in Table 1 together with other observed parameters to calculate RMR values (Taghipoor 2008). Figures 1 and 2 show the location of the mine and the plan of the extraction panels East 1 and East 2. Figure 3 shows double drum shearer and shield supports in East 1 panel.

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The excavated roadways serving the longwall face were generally of cross sectional area which varied between 15 m$^2$ and 18 m$^2$. The basic support pattern system consisted of 13 x 2.4 m long AT roof bolts (7+6 pattern) per metre length of the roadway. The ribs in the tailgate were supported with four 1.8m fibreglass bolts in the right hand side and three 1.8 m AT bolts in the left hand side. Several trial sites were established to examine the performance of different bolting patterns. The first site was a 15 m length of the roadway containing 13 roof bolts plus IPB-160 steel frames, set one at 1m spacing. The sides of the roadway were packed with corrugated iron sheets and sand bags. The second trial section, some 40 m long, used a basic pattern of roof and side bolts with IPB 160 steel frames set at 1 m spacing. In the third trial site section of about 37 m length, the same bolting pattern was used plus frames set at 2 m spacing (Taghipoor 2008).

**DEVELOPMENT OF A FDM MODEL**

FLAC$^{2D}$ software, based on a FDM$^1$ analysis, was specially developed to calculate 2D stresses and displacements induced by underground excavations. FLAC can be used to solve a wide range of mining and civil engineering problems. Materials in the model can be linear elastic and non-linear (Mohr–Coulomb and Hoek–Brown failure criterion) and discontinuities may be defined into the model. This feature was used to model the movements of blocks in the roadway.

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$^1$ Finite Difference Method
Table 1 - Core data and RMR parameters

<table>
<thead>
<tr>
<th>Depth into roof (m)</th>
<th>Rock Type</th>
<th>UCS (MPa)</th>
<th>RQD (%)</th>
<th>Discontinuity Spacing (m)</th>
<th>Discontinuity Condition</th>
<th>Ground Water</th>
<th>Discontinuity orientation</th>
<th>RMR</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 2.8</td>
<td>Siltstone</td>
<td>32</td>
<td>29.2</td>
<td>0.06 – 0.2</td>
<td>Slightly rough, separation&lt;1mm</td>
<td>Dripping to dry</td>
<td>Very unfavourable</td>
<td>Class III–IV (35 – 46 )</td>
</tr>
<tr>
<td>Rating</td>
<td></td>
<td>4</td>
<td>8</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.8 – 3.7</td>
<td>Sandstone</td>
<td>73</td>
<td>69.3</td>
<td>0.06 – 0.2</td>
<td>Slightly rough, separation&lt;1mm</td>
<td>Dripping to dry</td>
<td>Very unfavourable</td>
<td>Class III–IV (47 – 58 )</td>
</tr>
<tr>
<td>Rating</td>
<td></td>
<td>7</td>
<td>17</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.7 – 4.13</td>
<td>Siltstone</td>
<td>32</td>
<td>34</td>
<td>0.06 – 0.2</td>
<td>Slightly rough, separation&lt;1mm</td>
<td>Dripping to dry</td>
<td>Very unfavourable</td>
<td>Class III–IV (35 – 46 )</td>
</tr>
<tr>
<td>Rating</td>
<td></td>
<td>4</td>
<td>8</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Geometry of the model

The geometry of the area modelled was 40 m by 40 m with a roadway width of 4.5 m and height of 3.5 m. The coal seam was modelled as 2 m thick and dipping at 20°. The E1TG immediate roof stratification sequence consisted of siltstone and sandstone above the roof. The geometry of the model defined is shown in Figure 4.

Boundary conditions

The model assumes plane strain state, nil displacements at the boundaries and constant field stresses. If the model is used to simulate convergence without longwall influence, then it is assumed that only the stress due to the pressure of the overburden at that depth is acting.

\[
\sigma_v = \gamma \times H \\
\sigma_h = k \times \sigma_v
\]

Where \(\gamma\) is the unit weight (kN/m\(^2\)), H is the depth (m). E1TG was located at a depth of 150 m around the coring position. The vertical stress of 3.2 MPa and the ratio of horizontal to vertical stress \(k = 0.4\) were determined for the site, according to the tectonic history of the region.

FLAC 2D was used to analyse the efficiency of the old and new roof bolting patterns. To provide input parameters for the models, the RocLab program (working based on GSI classification, GSI=RMR-5) was used to estimate the parameters of rock mass surrounding the roadway (Rocscience, 2002) and to provide input parameters for the models. The results are listed in Table 2. Short encapsulation pull test results were used to define roof bolt bond properties as shown in Table 3.

Figure 4 – The model roadway profile, layers and rockbolt pattern numerical modelling
### Numerical Modelling

#### Table 2 - Intact rock and rock mass parameters using Roclab program

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Depth into roof (m)</th>
<th>Intact Rock</th>
<th>Rock Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UCS (MPa)</td>
<td>m*</td>
<td>GSI</td>
</tr>
<tr>
<td>Coal</td>
<td>0</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>Siltstone</td>
<td>0</td>
<td>-2.8</td>
<td>32</td>
</tr>
<tr>
<td>Sandstone</td>
<td>2.8-3.7</td>
<td>73</td>
<td>13</td>
</tr>
<tr>
<td>Siltstone</td>
<td>3.7-4.2</td>
<td>32</td>
<td>7</td>
</tr>
</tbody>
</table>

#### Table 3 - Bolt and bond properties (IRITEC 1992)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>unit</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>mm</td>
<td>21.7</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>GPa</td>
<td>207</td>
</tr>
<tr>
<td>Tensile Yield Load</td>
<td>tone</td>
<td>32</td>
</tr>
<tr>
<td>Shear Stiffness of Bond</td>
<td>KN/mm</td>
<td>65</td>
</tr>
<tr>
<td>Compressive Strength of Bond</td>
<td>MPa</td>
<td>5</td>
</tr>
<tr>
<td>Normal Stiffness of Bond</td>
<td>MPa</td>
<td>100</td>
</tr>
</tbody>
</table>

### MODELLING WITH FAULTED ZONE

In this section, the stability analysis of roadway E1TG within the faulted zone was carried out. Material properties such as friction angle, cohesion, dilation angle, bulk modulus, shear modulus and in-situ stresses were used as inputs into the model defined with displacements and shear strains as outputs. The simplest and best-known failure criterion for rocks, the Mohr-Coulomb criterion, was used. In FLAC²D, the Mohr-Coulomb plasticity model is one of the built-in constitutive material models. It is used for materials that yield when subjected to shear loading but the yield strength depends solely on the major and minor principal stresses; the intermediate principal stress has no effect on the yield. For the Mohr-Coulomb model, the following material properties are required: rock mass density, bulk modulus, shear modulus, friction angle, cohesion, dilation angle and tensile strength. Interfaces here mean joints, faults, or bedding planes in rock masses. In FLAC, interfaces are characterised by Coulomb sliding and/or tensile separation. They have properties of friction, cohesion, dilation, normal and shear stiffness, and tensile strength. For bedding planes to be modelled, slip along the plane and bed separation are the major desired features (Itasca Group 2000).

**Bolt representation**

There are several structural elements in FLAC for simulating structural supports. One of them is called cable element, which is a one-dimensional axial element. It can be point-anchored or grouted to the surrounding material so that the cable element develops forces along its length as the surrounding media deform. Therefore, cable element is the best structural element for modelling rock bolts. The cable element requires the physical and mechanical rock bolt parameters as input data. The properties associated with the grout are more difficult to estimate. In many cases, the following expression provides a reasonable estimation of $k_{bond}$ (Itasca Group 2000):

$$K_b = \frac{\pi G}{5L_n \left[1 + 2t/d\right]}$$

Where;

- $G$ = Grout shear modulus;
- $t$ = Annulus thickness;
- $d$ = Diameter of the bolt.
Given the failure of the bolting system occurs at the grout/rock interface, \( s_{bond} \) can be approximated by the following equation:

\[
S_{bond} = \pi (d + 2\tau_1) Q_B
\]  

(4)

Where;
- \( \tau_1 \) = One-half of the uniaxial compressive strength of the weaker of the rock and grout;
- \( Q_B \) = the quality of the bond between the grout and rock (\( Q_B = 1 \) for perfect bonding);

If it is believed that the failure occurs at the bolt/grout interface rather than at the grout/rock interface, then the shear stress should be evaluated at this interface by replacing \( (d + 2\tau) \) by \( d \) in Equation 4. For partially grouted bolts, the free portion of the bolt does not have any bond with the rock. Under such circumstance, the values of \( k_{bond} \) and \( s_{bond} \) are set to zero (Itasca Group 2000).

To investigate the underground stability the Sakurai and et al. (1994) method was used. The method evaluates the critical strain in the elastic region. Since the rock mass is under triaxial stress, it is logical using the maximum critical strain for investigation of tunnel stability. They suggested the following equations (Sakurai 1993).

\[
\log \epsilon_c = -0.25 \log E - 1.22
\]  

(5)

\[
\gamma_c = (1 + \nu) \epsilon_c
\]  

(6)

Where;
- \( E \) = Young’s modulus of intact rock \( \left( \frac{kgf}{cm^2} \right) \)
- \( \epsilon_c \) = critical strain in uniaxial compressive strength
- \( \gamma_c \) = critical strain
- \( \nu \) = Poisson’s ratio.

Critical displacement values based on the critical strain are obtained by following equation:

\[
\epsilon_c = \frac{U_c}{a}
\]  

(7)

Where;
- \( U_c \) = Allowable displacement
- \( a \) = radius of the roadway

Displacement vectors in Figure 5 show that higher deformations occur at the top of left rib and bottom of right rib of the roadway within the coal. Similar results can be found in the maximum shear strain increment plot (Figure 6). These two figures reveal the potential failure mechanism which can lead to failure of wedges at left hand side part of roof and top part of right hand side rib. The displacement around E1TG is high and shows that E1TG was unstable and needed to be supported. To find the optimum support system, different patterns and spacing of rockbolts were modelled. Using Sakurai’s equations and results of analysis it was determined that patterns 8+6 and No flexi 4+2 are better than other patterns in the faulted zone.

Figure 7 presents axial forces in rockbolts in the two patterns. Results of vertical and horizontal displacements and maximum shear strain around E1TG are summarised in Table 4 and Table 5.

**MODELLING WITH RETREATING FACE**

In the longwall mining method, the roof strata at the longwall face is undermined and allowed to collapse behind the longwall face shields. When the face is far enough advanced from the face starting position, the immediate roof collapses at a certain distance depending on the geological conditions. Failure of the roof continues until the roof and the caved material are in contact. The natural stress distribution in the rock strata is disturbed with the excavation; high pressure zones are created in the adjacent coal because of the transfer of stresses. Figure 8 shows state in the ground after the excavation of a panel (Yavuz 2004).
Table 4 - Horizontal and vertical displacements of around E1TG (mm)

<table>
<thead>
<tr>
<th>Model of fault</th>
<th>Roof</th>
<th>Right hand rib</th>
<th>Left hand rib</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Support</td>
<td>25.2</td>
<td>66.6</td>
<td>87.4</td>
</tr>
<tr>
<td>Pattern 6+3</td>
<td>5.96</td>
<td>20.1</td>
<td>41.6</td>
</tr>
<tr>
<td>Pattern 6+7</td>
<td>4.76</td>
<td>15.7</td>
<td>36.7</td>
</tr>
<tr>
<td>Pattern 6+8</td>
<td>3.38</td>
<td>12.9</td>
<td>16.3</td>
</tr>
<tr>
<td>Pattern No Flexi 4+2</td>
<td>2.70</td>
<td>9.95</td>
<td>12.2</td>
</tr>
<tr>
<td>Pattern Flexi</td>
<td>5.69</td>
<td>1.99</td>
<td>40.1</td>
</tr>
<tr>
<td>Critical Displacement</td>
<td>10.2</td>
<td>18.3</td>
<td>18.3</td>
</tr>
</tbody>
</table>

Table 5 - Maximum shear strain increment around E1TG ($\times 10^{-3}$)

<table>
<thead>
<tr>
<th>Model of fault</th>
<th>Roof</th>
<th>Right hand rib</th>
<th>Left hand rib</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Support</td>
<td>10.4</td>
<td>10.7</td>
<td>18.7</td>
</tr>
<tr>
<td>Pattern 6+3</td>
<td>0.05</td>
<td>1.61</td>
<td>33.4</td>
</tr>
<tr>
<td>Pattern 6+7</td>
<td>0.04</td>
<td>0.6</td>
<td>84.2</td>
</tr>
<tr>
<td>Pattern 6+8</td>
<td>0.07</td>
<td>1.4</td>
<td>4.93</td>
</tr>
<tr>
<td>Pattern No Flexi 4+2</td>
<td>0.07</td>
<td>0.7</td>
<td>4.30</td>
</tr>
<tr>
<td>Pattern Flexi</td>
<td>0.04</td>
<td>1.59</td>
<td>38.1</td>
</tr>
<tr>
<td>Sakurai Critical Strain</td>
<td>4.54</td>
<td>6.48</td>
<td>6.48</td>
</tr>
</tbody>
</table>
To investigate the potential influence of the face retreat, the stresses applied to the model were increased in several stages to simulate the increase in stress ahead of the retreating face. Since the roadways were aligned with the maximum horizontal stress, roadways were not expected to be subject to a severe stress notch during the face retreat. To reflect this, the applied increases were mainly vertical. The stress increases simulating face retreat were applied in several stages. The results indicated that large amounts of roadway closure, mainly in the form of rib squeeze and floor heave, could be expected ahead of the retreating face. With the roof horizon in the seam, there was a large increase in the displacements within the coal roof as the stresses were increased.

The displacements above the coal seam did not show a significant increase nor did the height of softening increase (Bigby, et al., 2004). Taking into account the influence of the retreating longwall, the stresses are approximately 2.3 times the pre-mining (or virgin) stress magnitudes. This is in agreement with the calculation results based on the Wilson’s theory (Wilson 1980). In this research, it was found that the stress increases to about 2.4 times the pre-mining values. An independent study suggested stress magnitudes of the order 2 to 3 times pre-mining stresses were to be expected (IRITEC 1992).

Figure 9 shows roof layer deformation due to the increasing stress around E1TG. The total displacements of ribs and roof of roadway under different condition are shown in Figure 10.

Results of vertical and horizontal displacements and maximum shear strain around E1TG are shown in Table 6 and Table 7. The results show patterns 8+6 and No flexi 4+2 perform better than other patterns in the faulted zone during the longwall retreat.

<table>
<thead>
<tr>
<th>Left hand rib</th>
<th>Right hand rib</th>
<th>Floor</th>
<th>Roof</th>
<th>Model of fault (240%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>85.2</td>
<td>137.2</td>
<td>2.82</td>
<td>31.9</td>
<td>No Support</td>
</tr>
<tr>
<td>36.4</td>
<td>25.8</td>
<td>1.31</td>
<td>13.2</td>
<td>Pattern 6+3</td>
</tr>
<tr>
<td>33.3</td>
<td>25.5</td>
<td>1.31</td>
<td>12.6</td>
<td>Pattern 6+7</td>
</tr>
<tr>
<td>23.9</td>
<td>14.9</td>
<td>1.11</td>
<td>10.2</td>
<td>Pattern 6+8</td>
</tr>
<tr>
<td>19.5</td>
<td>8.7</td>
<td>1</td>
<td>8.16</td>
<td>Pattern No Flexi 4+2</td>
</tr>
<tr>
<td>31</td>
<td>25.8</td>
<td>1.68</td>
<td>19.7</td>
<td>Pattern Flexi</td>
</tr>
<tr>
<td>18.3</td>
<td>18.3</td>
<td>10.2</td>
<td>10.2</td>
<td>Critical Displacement</td>
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<table>
<thead>
<tr>
<th>Left hand rib</th>
<th>Right hand rib</th>
<th>Floor</th>
<th>Roof</th>
<th>Model of fault (240%)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.7</td>
<td>13.5</td>
<td>1.55</td>
<td>13.7</td>
<td>No Support</td>
</tr>
<tr>
<td>11.8</td>
<td>2.14</td>
<td>0.14</td>
<td>0.1</td>
<td>Pattern 6+3</td>
</tr>
<tr>
<td>14.2</td>
<td>2.11</td>
<td>0.12</td>
<td>0.1</td>
<td>Pattern 6+7</td>
</tr>
<tr>
<td>4.55</td>
<td>1.87</td>
<td>0.13</td>
<td>0.2</td>
<td>Pattern 6+8</td>
</tr>
<tr>
<td>2.67</td>
<td>0.82</td>
<td>0.14</td>
<td>0.21</td>
<td>Pattern No Flexi 4+2</td>
</tr>
<tr>
<td>21</td>
<td>2.7</td>
<td>0.18</td>
<td>0.13</td>
<td>Pattern Flexi</td>
</tr>
<tr>
<td>6.48</td>
<td>6.48</td>
<td>4.54</td>
<td>4.54</td>
<td>Sakurai Critical Strain</td>
</tr>
</tbody>
</table>

* 240% of pre-mining stress values applied
Figure 9 - Displacement around E1TG with increasing stresses due to face retreat: (a) Roof displacement (b) Displacement in right hand rib (c) Displacement in left hand rib
Percentages for series indicate proportion of pre-mining stresses applied in modelling

Figure 10 - Displacement for various support patterns, at the faulted zone and the face in retreat state

CONCLUSIONS

Stability analysis and optimum support design of E1TG roadway in the faulted zone during face retreat in Tabas coal mine indicate that:

- There is a need to install strong support system to counter high ground deformation and low safety factor around the roadway.
- Floor heave of TG is independent of the reinforcement in ribs and roof and also face retreat.
- More roof bolts will be needed to control roof movement during face retreat.
- Left side rib will need more reinforcement during retreat. Fully grouted resin bolts are a good choice for this purpose.
- **FLAC** analysis indicate that patterns 8+6 and No flexi 4+2 are better than other patterns within the faulted zone.
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QUANTIFYING THE IMPACT OF COVER DEPTH AND PANEL WIDTH ON LONGWALL SHIELD-STRATA INTERACTIONS

Robert Trueman¹, Michael Callan¹, Rod Thomas¹ and David Hoyer ²

ABSTRACT: Results of a series of back-analyses of the interaction between longwall shields and strata at a number of mines are presented. The purpose of these back-analyses was to quantify the impact of cover depth and panel width on shield performance.

Recently developed shield load cycle analysis theories were used to quantify the interaction between shields and strata. A load cycle is the change in support pressure with time from the initial setting of the shield against the roof until the subsequent release and movement of the support, which typically corresponds to a single shear. Historical shield pressure data from five longwall mines in Australia and Europe were back-analysed, together with strata delay data for the longwall faces. An assessment of the geology of the near-seam overburden was also made for each site. The longwall panels incorporated cover depths ranging from 50 to 770 m and panel widths ranging between 168 to 319m.

Use was made of a modified version of the longwall visual analysis (LVA) software that was specifically extended for this project and provided maps of the critical load cycle parameters implicit to the utilised analysis methodology. The major extension of the software involved presenting the outputs on the basis of individual load cycles for every shield as opposed to a time or chainage basis, thus allowing load cycle analysis to be carried out. In total about 6.5 km of longwall retreat and over 2 000 000 individual load cycles were back-analysed. Together with the strata delay and geological data, this enabled the effects of panel width and cover depth to be quantified within the range of the data.

INTRODUCTION

A number of authors have concluded or inferred the need for a greater powered support capacity with increasing depth of cover and/or panel width (e.g., Medhurst and Reed, 2005; Frith and Creech, 1997; Tsang and Peng, 1994). Nevertheless the impacts of these factors on support loading are still debated. Shield loading is a complex interaction between: shield capacity and set pressure; the composition of the main and immediate roof; the presence or absence of leaking legs; extraction height; cycle time; panel width and depth of cover. It has proven very difficult to isolate all of these factors in the past.

Load cycle analysis theories have been recently developed, which were presented in the Coal 2008 Conference (Trueman, Lyman and Cocker, 2008), that enable the factors influencing shield loading to be isolated and quantified. Commercial software was specifically extended to enable load cycle analysis to be carried out on historical shield monitoring data from five Australian and European longwall mines. These mines represented a range of cover depths, panel widths and strata composition that allows the impact of these factors on shield loading to be quantified.

SHIELD-STRATA ANALYSIS METHODOLOGY

The analysis methodology depends upon both the extraction and visualisation of the critical load cycle features necessary to interpret how the shields are interacting with the strata. An off-line version of the LVA software has been extended to provide and visualise the following critical load cycle features for each leg of each support (where both legs are monitored):

- Time Weighted Average Pressure (TWAP) Map – note: a) the TWAP is calculated between the initial setting of the shields to the roof and the final release at the end of the load cycle, b) a value is calculated for each leg of each shield for every load cycle, c) zones of high loading are shown in red and zones of low loading in blue and d) this map enables a rapid overview to be

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² LVA Pty Ltd
made of the periodic weighting interval, problem areas that have occurred on the face such as roof falls and support maintenance issues.

- **Number of Yield Events Map** – note: a) the number of yield events in an individual cycle have been colour coded; blue indicates 1 to 3 yields and orange/red >8, b) the number of yield events in a single load cycle is a very good indicator of the intensity of loading and has been shown to correlate well with roof conditions experienced on a face, c) extensive back-analysis at a number of longwalls has indicated that, in general, if there are less than 3 yield events per load cycle then roof conditions do not deteriorate at most sites, d) a deterioration in roof conditions is normally seen between the support tip and the face after >3 yield events, with increasing severity as the number of yields increases and e) this illustrates that it is the number of yield events in a given load cycle that is important and not whether or not a support yields.

- **Low Set Pressure Map** – note: a) a set pressure which is too low, has been found to lead to roof control problems on the face, because naturally occurring and mining induced fractures are allowed to dilate resulting in a reduction in the mechanical interlock of the strata, b) previous experience shows that a set pressure of <40 t/m² is the typical threshold value at which roof control problems can result, c) set pressures of >60 t/m² have been found to ameliorate any potential roof control problems relating to set pressure and d) for each back analysis, maps highlighting set pressures equating to <40 t/m² have been generated.

- **Initial Loading Rate Map** – note: a) the map illustrates the loading rate in bar/min calculated in the first ten minutes after the powered support is set, b) previous experience suggests that the initial loading rate is a good indicator of the intensity of the loading conditions and c) loading rates of <10 bar/min in the first 10 minutes after the support is set generally results in relatively benign loading conditions, while loading rates >15 bar/min generally results in heavier loading with the intensity of loading increasing as the loading rate exceeds this threshold.

- **Load Cycle Time Map** – note: a) the length of the cycle is of particular importance when the shields are being overloaded or being set too low, b) in such instances the additional cycle time allows more roof convergence and in doing so increases the probability of roof control issues and allows more time for fractures to dilate and c) cycle time is of less importance where the shields are being adequately set and are stabilising the roof.

- **Anomalous Leg Pressure Map** – note: a) the software identifies differential loading rates between two legs on a single shield and flags the leg with the lower loading rate as potentially having faulty hydraulics, valves or sensors, b) where anomalous legs are grouped together, low set pressures on the anomalous shields and overloading of the adjacent shields can result and c) both scenarios have been found to result in roof control problems.

The data are presented as maps in which the x-axis represents the support number counting from the maingate end of the face. The y-axis represents the shear number in the direction of mining and is therefore proportional to mining advance. Each shear represents a single load cycle for a shield, which enables load cycle analysis to be carried out.

The plotted value is the value of the variable of interest (ie one of the critical load cycle features) and it is coded by colour. Using colour as the third dimension has been found to be the most effective way of enabling a rapid evaluation of the support-strata interaction. These critical load cycle maps are used to characterise the support-strata interaction.

**GEOLOGY OF THE CASE HISTORIES**

The features of the geology of the longwalls studied that have the greatest potential to influence shield loading are summarised in Table 1.
Table 1 - Geology of longwalls most relevant to shield loading potential

<table>
<thead>
<tr>
<th>Mine</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of Cover (m)</td>
<td>450-500</td>
<td>500-540</td>
<td>745-770</td>
<td>500-540</td>
<td>50-220</td>
</tr>
<tr>
<td>Thickness of Thickly Bedded to Massive Units (m)</td>
<td>30 &lt;br&gt; i) 0-10 &lt;br&gt; ii) 5-10 &lt;br&gt; iii) 0-5</td>
<td>0 &lt;br&gt; i) 5-10 &lt;br&gt; ii) 5-10 &lt;br&gt; iii) 15-22</td>
<td>i) 0-23 &lt;br&gt; ii) 0-17</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Height of Thickly Bedded to Massive Units above the roof (m)</td>
<td>50 &lt;br&gt; i) 0-5 &lt;br&gt; ii) 30-50 &lt;br&gt; iii) 60-80</td>
<td>N/A &lt;br&gt; i) 15-20 &lt;br&gt; ii) 40-50 &lt;br&gt; iii) 55-70</td>
<td>i) 9 &lt;br&gt; ii) 45</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CMMR of Immediate Roof</td>
<td>40</td>
<td>55</td>
<td>36</td>
<td>50</td>
<td>35-60</td>
</tr>
</tbody>
</table>

LONGWALL DATA

The relevant longwall data is summarised in Table 2.

<table>
<thead>
<tr>
<th>Mine</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shield Capacity (t)</td>
<td>i) 80x750 &lt;br&gt; ii) 23x720</td>
<td>1 000</td>
<td>850</td>
<td>1,000</td>
<td>i) 36x1 240 &lt;br&gt; ii) 117x940</td>
</tr>
<tr>
<td>Shield Support Density Before the Cut (t/m²)</td>
<td>i) 100 &lt;br&gt; ii) 81</td>
<td>110</td>
<td>99</td>
<td>106</td>
<td>i) 136 &lt;br&gt; ii) 103</td>
</tr>
<tr>
<td>Yield Pressure (bar)</td>
<td>430</td>
<td>430</td>
<td>430</td>
<td>430</td>
<td>465</td>
</tr>
<tr>
<td>Tip-to-Face (mm)</td>
<td>650</td>
<td>450</td>
<td>500</td>
<td>500</td>
<td>610</td>
</tr>
<tr>
<td>Set Pressure (bar)</td>
<td>330</td>
<td>320 (350)</td>
<td>300</td>
<td>330 (345)</td>
<td>320</td>
</tr>
<tr>
<td>Shearer Cutting Method</td>
<td>Uni-Di</td>
<td>Uni-Di</td>
<td>Bi-Di</td>
<td>Uni-Di</td>
<td>Bi-Di</td>
</tr>
<tr>
<td>Web Depth (mm)</td>
<td>800</td>
<td>800</td>
<td>800</td>
<td>800</td>
<td>900</td>
</tr>
<tr>
<td>Panel Width (m)</td>
<td>168</td>
<td>319.5</td>
<td>305</td>
<td>305</td>
<td>263</td>
</tr>
<tr>
<td>Extraction Height (m)</td>
<td>3-3.4</td>
<td>3-3.2</td>
<td>5</td>
<td>2.4-2.8</td>
<td>3.05</td>
</tr>
</tbody>
</table>

SHIELD-STRATA INTERACTION ASSESSMENT

Mine A

Shield pressure and delay data were provided for 885 m of longwall retreat, during which approximately 1 325 shield load cycles were detected by the software. A number of significant roof control delays were recorded by the mine at regular intervals for the full length of the analysed section of retreat. The shield-strata interaction can be quantified from the critical load cycle maps that were developed for this period.

The TWAP map (see Figure 1) clearly shows the periodicity in the loading, with the peaks of the periodic weighting showing up as red (high average pressure) horizontal stripes. The areas where significant roof control problems were experienced are also clearly distinguishable as areas of blue (low average pressures).

The yield count map (see Figure 2) shows that at most of the peaks of the periodic weighting cycles, a significant number of supports yielded. At some peaks, the supports only underwent one or two yield events within the load cycle, but in others several yields were noted. Cases where several yield events were recorded during a single load cycle were in general associated with relatively long cycle times. All of the roof control delays recorded by the mine occurred after a number of the shields recorded several yield events. The yield events shown in Figure 2 are indicative of supports that are experiencing high level periodic weighting. As will be noted later, the number of legs identified as having potential problems with the hydraulics would have contributed to the intensity of the yielding at the peaks of the periodic weighting.

Figure 3 is a map showing where set pressures below 180 bar was recorded, which equates to about 40 t/m². There are a number of vertical stripes on the map and these are most likely associated with support legs that are either leaking or where the sensors are failing or poorly calibrated. There are
additionally a number of clusters of localised low set pressures that correlate to the blue sections in the TWAP map mentioned previously. These areas are associated with cavities and roof falls. The extent of the cavities would undoubtedly have been increased with such low set pressures.

The loading rate in the first 10 minutes after the shield has been set can be observed in Figure 4. Loading rates at the peak of the periodic weighting cycles generally varied between about 10 and 15 bar/ min. The high loading rates corresponded to where periodic yielding was observed.

Figure 5 is a map of those legs that have anomalous pressure readings. From this figure it is evident that approximately 25% of the legs have been identified as anomalous. A manual check of the raw pressure data indicated that all of these legs had potential issues with the hydraulics. This particular longwall operates with guaranteed set which was observed to be constantly topping up the pressure in most of these legs. This would have ameliorated roof control problems associated with low overall support density by maintaining system pressure at all times the pumps were operating, noting that a support density of less than 40 t/m2 before the cut has been observed to result in roof control problems at other sites. Nevertheless, the fact that the maximum support pressures in most of these legs would have been no more than system pressure would have contributed to the yielding on the other neighbouring legs at the peaks of the periodic weighting cycle. This quantity of problematic legs would undoubtedly have contributed to the reported roof control problems.

The estimated cycle times are shown in Figure 6. As mentioned previously, it is noticeable that the periods where several yields were noted on a large number of shields in a single load cycle correlated to relatively long cycle times. It is also noticeable that the roof control problems often occur during or immediately after relatively long cycle times.

The maps indicate that high level periodic weighting is being experienced, which is resulting in yielding of a number of the supports at the peaks of the periodic weighting cycle. Several yield events are occurring at some of the periodic weighting intervals, usually when cycle times are relatively long. Such numbers of yields in a single load cycle has resulted in roof control problems at other sites and is indicative of supports that are being periodically overloaded. A number of roof control problems are noticeable on the critical load cycle feature maps and have been noted in the delay data that occurred immediately after these events. The reason that the supports are being periodically overloaded is probably related to the amount of thickly bedded to massive strata in the roof. Only one sonic log was available to determine the composition of the roof. The interpretation of this log suggested that there may be an up to 30 m thick competent bed located at a height of about 50 m above the seam. Competent beds of this thickness and height above the seam have been demonstrated to result in high level periodic loading at other sites.

The longwall is relatively deep by Australian standards but the marked periodicity of the shield loading points to thick competent beds as the cause of shield overloading rather than the effect of depth. As will be discussed, the fact that much lower shield loading was observed in a longwall that was significantly deeper but had no thick competent units in the immediate or main roof tends to support this argument.

The narrower panel at Mine A has not prevented the supports from being periodically overloaded in some of the peaks of the periodic weighting cycles. Nevertheless the narrow panel means that in general, cycle times will be less than if the panel was wider. This will have likely influenced the number of yields being experienced in some of the high periodic weighting cycles. The narrow panel width in this case would therefore have been expected to have had a positive influence on roof control in many of the weighting cycles. This is confirmed by the fact that roof control issues did not occur following the peak of the periodic weighting cycles in most of the cases but tended to occur when cycle times were long. In narrower panels the number of shields protected from full loading by the chain pillars is proportionally greater and as such, the length of the face that is exposed to overloading is also proportionally less. In the case of Mine A about 50% of the shields went into periodic yield, whereas on other longwalls with wider faces, a higher proportion of the shields were often affected by periodic overloading, as will be discussed later.

**Mine B**

Shield pressure and delay data were provided for 870 m of retreat. Only minor delays were reported due to roof control issues on the longwall face. Approximately 1,100 load cycles were detected during the period of analysis. As with Mine A, the critical load cycle maps were used as an aid to quantifying
the shield-strata interaction. These maps are not presented in the paper, rather a summary of the relevant findings only.

The TWAP map showed a clear periodicity in the support loading. At the peak of the periodic weighting cycles a number of supports yielded and on occasions a large percentage of the supports on the face yielded. However, in general the supports only underwent one or two yield events within the load cycle, even though load cycle times varied. This is indicative of supports that are experiencing low level periodic weighting. When yielding did occur, stabilisation of the roof occurred within the load cycle after less than three yield events.

The only instances of low set pressures that were observed were in the vicinity of faults. Because of the relatively high angle of the faults to the face, only a few shields were set too low in any individual load cycle. Nothing showed up in any of the maps or delay data that indicated any significant roof control problems were experienced in these areas.

Loading rates at the peak of the periodic weighting cycles generally varied between 5 and 10 bar/min. with a few load cycles at the periodic weighting peak in excess of this. The higher loading rates generally coincided to areas where an appreciable number of supports reached yield.

Mine B is relatively deep by Australian standards and is extracting a relatively wide panel. Nevertheless, the analyses indicate that the shields are coping well with the roof conditions with few roof control issues. Few instances of low set pressures or legs with hydraulic problems were identified. Maintenance and support operation were therefore not a contributor to roof control issues on this particular longwall.

Mine C

Shield pressure and delay data were provided for about 700 m of retreat. Approximately 870 load cycles were detected during the period of analysis. As with Mine B, the critical load cycle maps were used as an aid to quantify the shield-strata interaction but most of the maps are not presented in the paper, rather a summary of the relevant findings only. The yield count map has been included because it shows a case study where periodic yielding occurred with only a few yields, even when cycle times were long. Few strata delays were recorded for the period of analysis and these tended to be close to the gateroads.

The TWAP map indicated that shield loading had a marked periodicity despite the fact that there were no thick competent units in either the immediate or main roof. The periodic weighting interval was observed to be in the range 7 to 14 m. The periodicity was also observed in the yield count map which indicated that at some, but not all, of the periodic weighting cycles the shields across the majority of the face (about two thirds or more of the shields) reached yield pressure. However, in general the supports only underwent 1 or 2 yield events within the load cycle, even though load cycle times varied and a number at the peak of the periodic weighting cycle were quite long, see Figure 7. This is indicative of supports that are experiencing relatively low level periodic weighting, even though some yielding was observed to occur. As noted previously, it is the number of yield events in a load cycle, particularly in long cycles, that indicate the intensity of the weighting not just the fact that yielding occurs.

There was evidence of clusters of low set pressures, although strata delays were not recorded in the vicinity of them. Only one leg of the support was monitored on this particular longwall and the anomalous pressure map depends upon both legs being monitored. A manual check of the raw data did however indicate that there were some legs that appeared to have potential hydraulic problems.

Loading rates at the peak of the periodic weighting cycles generally varied between 5 and 10 bar/minute. The higher loading rates generally coincided to where an appreciable number of supports reached yield pressure.

Mine C is deeper than any existing Australian longwall, has a high extraction height and is extracting a relatively wide panel. Nevertheless, the analyses indicate that the shields were coping well with the roof conditions with few roof control issues. There was evidence of clusters of low set pressures in places and legs with anomalous leg pressures were observed. There was therefore potential for support operation and maintenance to contribute to roof control issues but no such issues were noted in the delay data.
Mine D

Shield pressure and delay data was provided for approximately 1 400 m of longwall retreat and approximately 1 800 load cycles were detected by the software. Numerous roof falls of varying magnitude were recorded during the analysis period, the size and frequency of the falls noticeably increasing over a 600 m length of retreat. The roof falls were clearly distinguished on the TWAP map as was the periodic weighting interval, which was in the range of 10 to 20 m.

At the peak of the periodic weighting cycles a number of supports yielded and on occasions a large percentage of the supports on the face yielded. Over most of the length of the retreat the supports only underwent 2 or 3 yield events with the occasional load cycle experiencing up to 4 yields. In the area of the face where most roof falls were recorded the number of yields increased markedly, with up to 8 in some load cycles. Initial loading rates, at 10 to 20 bar/min, also tended to be higher in this area of the face. Low set pressures were also evident over a number of load cycles in the vicinity of the roof falls. Maintenance issues did not appear to be a significant contributor to the roof falls, with only 3% of the legs showing anomalous pressure readings.

Based upon the above, the face appeared to be experiencing a transition between low and high level periodic weighting over most of the analysis area, with high level periodic weighting over the 600 m length of retreat where the majority of roof falls occurred. The thickly bedded to massive sandstone unit located 55 to 70 m above the workings, had a thickness of between 16 to 20 m over the majority of the panel, increasing to between 20 and 22 m where most of the roof falls occurred. So whilst this longwall is relatively deep by Australian standards and the face relatively wide, the major cause of the roof falls could be attributed to the presence of a relatively thick thickly bedded to massive sandstone in the main roof.

Mine E

Shield pressure and delay data was provided over a length of retreat of 2 625 m and approximately 2 900 load cycles were identified. This longwall is characterised by a large depth range (see Table 1) and a relatively high rate of retreat. Very few roof control issues were noted from the delay data.

Periodic weighting was variable across the panel length. In a number of areas support yielding occurred with the majority of shields across the face being affected. In some areas the supports experienced a large number of yields, up to eight in some cycles, whilst in others either no yielding occurred or where it did, the supports only underwent one or two yield events.

As with yielding, the initial loading rates at the peaks of the periodic weighting cycles were variable. Rates as high as 10 to 15 bar/min occurred in some areas of the face, whilst rates of between 3 and 8 bar/min were recorded in others. The higher and lower initial loading rates correlated to the areas of high and low numbers of yields.

There was evidence of clusters of low set pressures in places and about 4% of the legs were found to have anomalous leg pressures. There was therefore some potential for support operation and maintenance to contribute to roof control issues, although no such issues were noted in the delay data.

The above is indicative of a longwall that is experiencing both low and high level periodic loading of the supports. Although variable, the depth of cover is relatively shallow even at the deepest point (see Table 1). Importantly, there was no correlation between the intensity of the periodic weighting and the depth of cover. Rather there was a strong correlation between the thickness of thickly bedded to massive strata and the periodic loading of the supports. The highest loading rates and number of yields were experienced in the areas where a competent unit in the roof reached a thickness of up to 23 m. Conversely the lowest loading rates and minimal yielding was experienced in areas where no thickly bedded to massive sandstone units existed.

Where there were no competent units in the roof the initial loading rates at the peaks of the periodic weighting cycles on this longwall were noticeably less than for the much deeper Mine C, which likewise lacked competent units. This indicates that depth of cover is of some, albeit limited significance, in terms of shield loading.
IMPLICATIONS FOR SHIELD LOADING

The most significant impact on shield loading was found to be the presence or absence of thickly bedded to massive units in the immediate or main roof. High level periodic weighting leading to periodic shield overload was observed once thickly bedded to massive sandstone unit thicknesses exceeded 20 m. A transition between low and high level periodic weighting appears to occur once the thickly bedded to massive sandstones thicknesses exceed about 16 m. The height above the roof that these units influence shield loading can be quite high. Beds whose bases were up to 70 m above the roof were observed to be causing high level periodic weighting in this study.

The initial loading rate at Mine C at the peak of the periodic weighting cycles was higher than the areas in the much shallower Mine E that also lacked thick competent beds in the near-seam overburden. This is despite the fact that the shields at Mine E would have been appreciably stiffer, having a larger leg diameter and lower operating height. This indicates that in general supports in deeper deposits will carry a higher pressure, everything else being equal. Nevertheless the shields used at Mine C, which have a support density slightly below the average of the five mines in this study (see Table 2), were not being overloaded to a depth of 770 m. This indicates that, at least within the range of the data presented in this study, depth on its own does not appear to be a major factor in shield loading.

The potential for shield overload was observed at all the panel widths in this study – 168 m to 319.5 m. It must be noted that in none of the case histories did the strata bridge across the panel. Bridging longwalls have been found in previous studies to result in reduced shield loading (e.g., Frith and Creech, 1997; Bigby, 1988). Nevertheless a reduced panel width can have a significant impact upon roof control if periodic support overload occurs, as the reduced cycle time associated with narrower panels will reduce the number of yields and subsequent roof degradation. In addition, the number of shields on the face affected will be less as a greater percentage of them will be protected from full loading by the chain pillars located at either end of the face.

The analyses have also indicated the importance of shield maintenance and operation on the shield loading environment. Inadequately maintained shields can increase the load on adjacent legs and supports. Low set pressures when set conditions deteriorate can have a similar effect and can destroy the mechanical interlock of the strata above the supports, leading to roof control problems.

CONCLUSIONS

The greatest impact on shield loading has been found to be the presence or absence of thick units of thickly bedded to massive strata in the immediate or main roof. Although an increased depth of cover will generally result in higher shield loading, everything else being equal, modern capacity supports are capable of adequately controlling the roof in deep longwalls. Once full caving is initiated on or about a longwall face, narrowing the panel width cannot be relied upon to prevent shield overload. Nevertheless, where shield overload does occur there are benefits to narrower panels. Maintenance and support operation have both been found to potentially significantly influence the shield loading environment.

REFERENCES

| Figure 1 - Time weighted average pressure map, Mine A | Figure 2 - Yield count map, Mine A |
Figure 3 - Low set pressure map, Mine A

Figure 4 - Loading rate map, Mine A
Figure 5 - Anomalous pressure reading map, Mine A

Figure 6 - Load cycle time map, Mine A
Figure 7 - Yield count map, Mine C
CALIBRATED PARAMETERS FOR THE PREDICTION OF SUBSIDENCE AT MANDALONG MINE

Ross Seedsman¹

ABSTRACT: The consent conditions at Mandalong Mine require that subsidence deformations must not change the flood hazard category or subject a dwelling to deformation beyond safe surface and repairable (SSR) unless permission is granted by the effected landholder. The subsidence prediction in 2003 utilised an analysis of sag based on voussoir beams and of pillar compression based on foundation engineering principles. The model uncertainty for the sag analysis was assessed to be relatively high with a low parameter uncertainty, while for the pillar compression the model uncertainty was low but the parameter uncertainty was high. Up to June 2009, seven longwalls have been extracted. The consent conditions have not been breached. Both the voussoir beam and pillar compression models have been demonstrated to be valid. There have been changes in the way in which key input parameters are estimated.

INTRODUCTION

When Centennial Coal purchased the mine in 2003 the mine plan proposed panels of up to 250 m width and maximum subsidence of 2.98 m. Their review of the consent conditions raised concerns about risks to continuity of operations. The standard subsidence predictions methods available at the time indicated that panel widths of approximately 80 m would be required to bring the continued operations risk down to acceptable levels. Seedsman (2006) proposed an alternative prediction methodology that factored in the geotechnical conditions in the overburden and identified the likelihood that panels up to 175 m could be possible. The initial longwall panels were designed at 125 m and currently the panel width is 160 m. Whilst the panels are relatively narrow, the viability of the operation is underpinned by the thick seam extraction – up to 5 m.

The decision to start the mine with 125 m panels was based on the need to validate and calibrate the prediction methodology. A large number of survey lines have been monitored (Figure 1) and the data used to check key parts of the prediction. Figure 1 presents an interpretation of the subsidence bowls that was calculated using Surfer with an anisotropy factor of 3 aligned parallel to the panels. The maximum subsidence to the end of LW7 was about 1.2 m and this is located under the highest elevation which also corresponds to the greatest depth of cover of 360m. At the outbye ends of the panels (depths of about 160 m – 180 m) there are some variations to the overall patterns and these provide the basis for some of the discussion in this paper.

DESIGN IN 2003

Derivation of allowable subsidence

Currently, and also in 2003, the prediction of all surface subsidence deformations starts with a prediction of the vertical movement induced at the surface. The change in flood hazard category was relatively simple to define in terms of vertical subsidence (500 mm was selected as the maximum allowable).

The SSR criterion was not quantified in the consent conditions, and after a review of various reports and an inspection of the surface, the target values were set at 7 mm/m tilt and 4 mm/m strain. The step to vertical subsidence was still required. Noting that the panel width/depth ratios would be low, it was concluded that the K1, K2, and K3 curves (Holla, 1987) could not be used. Constant values of 0.65, 2.0 and 2.5 respectively were used for subsidence less than about 500 mm. It was assessed that a maximum vertical subsidence of 500 mm would apply at the SSR. Most of the dwellings are located on the flood plain so the vertical subsidence constraints applied simultaneously.

¹ Director- Seedsman Geotechnics Pty Ltd
Figure 1 - Interpreted contours of vertical subsidence overlain on topography

Sag

The geotechnical model for the spanning of massive units is shown in Figure 2. The data input requirements for the model are:

- Panel width – the rib to rib distance of the extraction panel.
- Interburden distance – the distance from the roof of the seam to the base of the massive unit. This is determined from borehole data.
- Goaf angle – the angle by which the panel width is reduced at the base of the massive unit, and by which the surcharge is also reduced. The design utilised a 12° angle, as determined by a back analysis of other subsidence events in the coalfield (Seedsman 2004). A standard deviation of 8° was identified in the back analysis.
- The thickness of the massive unit. This was determined from the core and geophysical logs, based on the presence of a continuous coarse sandstone/conglomerate with no mudstone band thicker than about 10mm (these being interpreted to be mudstone pebbles).
• The surcharge on the beam, as given by the depth to the top of the massive unit
• Uniaxial compressive strength (67 MPa) and Young’s modulus (18.8 GPa) – laboratory values not corrected for the rock mass given the requirement for the unit to be a massive unit without discontinuities.
• The model assumes that the goaf below the spanning unit does not provide any support to the beam.

![Figure 2 - Components of a model for assessing spanning](image)

**Pillar compression**

The pillars were designed with factors of safety greater than unity, and greater than 2.23 under the flood plain. Pillar stress was estimated using a simple inverted pyramid model and a loading angle of 21°.

Pillar subsidence is a function of the stresses that are developed, the width of the pillar, and the deformation properties of the coal, roof and floor strata (Figure 3).

![Figure 3 - Factors in pillar subsidence model](image)

The compression of the pillar itself was calculated with simple elastic theory and a modulus of the coal being set at 1.5 GPa, this value being at the low end of the range for large sized coal samples quoted by Medhurst and Brown (1998). The compression of the roof and floor was assumed to be the result of
the settlement of a rigid footing (Poulos and Davis, 1976), with the roof modulus being assumed to 15 GPa. The modulus values were based on laboratory values as at the time (2003) there was no appropriate way to estimate the deformation modulus of ‘soft rock’ masses. At that time, the state of the art was the 1999 paper by Hoek and Brown that proposed that the modulus could be obtained from:

\[ E_{\text{m}} \text{(GPa)} = \sqrt{\text{UCS/100}} \times 10^{(\text{GSI-10)/40})} \]

Where the GSI is the Geological Strength Index and the UCS is in units of MPa. This gives 125 GPa for an intact 50 MPa rock (compared to a typical laboratory value of 15 GPa). For a GSI of 50, a modulus of 22 GPa is obtained. For the floor the calculations were modified to account for the finite thickness and presumed drained modulus of low strength claystones of the Awaba Tuff.

**PROGRESSIVE IMPLEMENTATION**

In order to manage the risks inherent when introducing a new subsidence prediction method in a highly charged environment, a conservative strategy was recommended and adopted. Approval was sought for the first 2 longwalls, each 125 m wide with a 41 m chain pillar. The prediction for maximum vertical subsidence at the LW2 was 250 mm, 50% of what was believed to be the maximum allowable for SSR and flood damage. This was composed of 50mm of sag, an immediate pillar compression of 30 mm – 50 mm, and a longer term consolidation of the Awaba Tuff of about 150mm.

At the end of LW1 and prior to the extraction of LW2, when LW1 can be considered to be an isolated panel, the maximum subsidence in the inbye areas was 183 mm and outbye the maximum subsidence without fault influence was 70mm. At the end of LW2 in areas from known faulting, the maximum vertical subsidence recorded was 282 mm in the elevated ground and 160 mm under the flood plain. Table 1 compares the outcomes for LW1 and LW2 with the allowable levels interpreted from the consent conditions. It can be seen that the performance of the mine layout is well within the consent.

The behaviour around the thrust faults outbye was predicted but the location of the subsidence was about 100 m further outbye than predicted. The immediate pillar compression was higher than predicted but the longer-term compression did not develop. This result was not surprising given the recognition of the limitations in determining the deformation modulus values, and provided justification for the conservative implementation.

**Table 1 Performance against consent conditions for the 125m panels**

<table>
<thead>
<tr>
<th>Consent condition</th>
<th>Interpretation</th>
<th>Allowable</th>
<th>LW1 and LW2</th>
<th>End LW4</th>
</tr>
</thead>
<tbody>
<tr>
<td>SSR</td>
<td>Tilt at dwelling</td>
<td>5-7 mm/m</td>
<td>3.9 mm/m</td>
<td>2.6 mm/m</td>
</tr>
<tr>
<td></td>
<td>Tensile strains at dwelling</td>
<td>3-4 mm/m</td>
<td>0.8 mm/m</td>
<td>1.3 mm/m</td>
</tr>
<tr>
<td></td>
<td>Compressive strains at dwelling</td>
<td>3-4 mm/m</td>
<td>1.6 mm/m</td>
<td>1.8 mm/m</td>
</tr>
<tr>
<td>Flood category</td>
<td>Vertical subsidence under the flood plain</td>
<td>500 mm</td>
<td>160 mm</td>
<td>225 mm</td>
</tr>
</tbody>
</table>

Because of the timing of approvals, LW3 and LW4 were also extracted at 125 m width. Up to LW4, the subsidence deformations had been less than the 500mm and SSR constraints set by Centennial (Figure 1), so the decision was made to increase the face width. LW5 onwards have been 160 m wide. At the end of LW7, the maximum subsidence is in the order of 1.2m (Figure 1).

The differences in the subsidence patterns for the shallow and deeper areas of the mine and with 125 m and 160 m wide panels are dramatic. In the deeper areas (Figure 5), the pillar compression component dominates and the sag between the panels is a secondary feature. In the shallow areas (Figure 6) the sag component dominates and the difference between the 125 m panels and the 160 m panels is clear.
Figure 4 - Contours of subsidence after LW2 and LW5

Figure 5 - Inbye cross line results

Figure 6 - Outbye cross line results

KEY PARAMETERS

As mining has progressed, the opportunity has been taken to progressively improve the predictions. The basic models of sag and pillar compression have remained unchanged, but there has been a change in the way some of the key parameters are estimated. The model uncertainty is now considered to be low, and the parameter uncertainty has reduced such that the mine operates much closer to the 500 mm allowable limit.

Goaf angle

During the retreat of LW5, greater than predicted subsidence developed in a restricted area. Both
inbye and outbye of this area, the vertical subsidence along the panel centreline was within the predicted range. A fully cored borehole was in close proximity and this showed that the interpretation of the conglomerate thickness was valid. Underground, the area coincided with a pronounced roll in the seam (Figure 4) which had already been implicated in a number of ground control difficulties – at the face the overburden was noted to cave more readily. There was only one other subsidence line that crossed the trend of the roll and with hindsight it was possible to identify some atypical deformations.

It is proposed that the roll is characterised by greater jointing in the overburden such that the goaf angle would be reduced. It is noted that the back analysis had indicated that the goaf angle varied between \(-20^\circ\) and \(+20^\circ\), with the \(-20^\circ\) value being an outlier. Omitting the outlier, the average goaf angle was found to be \(12^\circ\) with a standard deviation of \(8^\circ\).

The impact of reducing the goaf angle is to increase the span at the base of the spanning unit. This may lead to increased deflection or in the worst case failure of the beam. The higher subsidence developed at a depth of cover of approximately 180 m and the beam thickness was confirmed to be 39 m. Figure 7 presents plots that show how the stability and deflection change progressively. Note that for typical conditions, this change of goaf angle represents an increase in effective span at the base of the conglomerate of about 40 m.

![Figure 7 - Stability and deflections changes with reducing goaf angle](image)

It is interesting to note that the author has applied the model to other coal fields and has found that goaf angles of \(20^\circ-25^\circ\) may apply to longwall layouts that are aligned at much higher angles (say \(40-45^\circ\)) to the dominant joint direction. At Mandalong the orientation is within \(10^\circ\).

**Rock mass deformation modulus**

After LW2 it was noted that the immediate pillar compression was much higher than anticipated and there were no signs of further movements that had been suspected due to the consolidation the Awaba Tuff. The total deformation was within the anticipated range.

In 2006, 3 years after the initial designs, a method for the estimation of the rock mass modulus based on the reduction of laboratory values was published. Reducing laboratory modulus values to represent field behaviour is standard practice in rock engineering. Hoek and Diederichs (2006) provide the following equation to estimate the deformation modulus of rock masses from the laboratory values (Ei):

\[
E_{rm} = E_i (0.02 + \frac{1 - D}{2 (60 + 15D - GSI)})
\]

Where D is a disturbance factor to account for excavation blasting damage (set at 0 for this application).

The reduction factor is of an S shape with little change in modulus for very blocky rock masses (such as the Mandalong Conglomerate) and large changes for rock masses with intermediate values of GSI.
(bedded and laminated roof and floor strata). Note that a change in GSI of 4 units can, for intermediate values of the GSI, lead to a change of 10% in the rock mass modulus.

For the Mandalong project, allocation of GSI values has been based on coal joints being rough and stone joints being smooth, and the West Wallarah coal and the roof sandstone being considered blocky and the other materials being very blocky. It is noted that these selections are based, in part, on a calibration to the subsidence outcomes to date. GSI values and rock mass deformation moduli for the key materials in the design are presented in Table 2.

**Table 2 - Modulus and GSI values**

<table>
<thead>
<tr>
<th>Material</th>
<th>Ei</th>
<th>RSI</th>
<th>Erm</th>
</tr>
</thead>
<tbody>
<tr>
<td>West Wallarah Seam</td>
<td>3</td>
<td>60</td>
<td>1.5</td>
</tr>
<tr>
<td>Fassifern and Pilot Seams</td>
<td>2</td>
<td>45</td>
<td>0.4</td>
</tr>
<tr>
<td>Floor stone</td>
<td>10</td>
<td>43</td>
<td>1.8</td>
</tr>
<tr>
<td>Roof sandstone</td>
<td>15</td>
<td>60</td>
<td>7.5</td>
</tr>
<tr>
<td>Roof mudstones</td>
<td>15</td>
<td>49</td>
<td>4</td>
</tr>
<tr>
<td>Mandalong Conglomerate</td>
<td>22.8</td>
<td>95</td>
<td>22.8</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

The prediction methods have performed well and the outcomes are consistent with the consent conditions. The engineering behaviour models on which the predictions are based are well established and details of the various calculations involved can be readily found in the engineering literature. Mandalong has provided a well documented case study their application.

The approach to subsidence prediction used at Mandalong can be readily transferred to other coal fields. Early recognition of the spanning capability of thick beams came from work on the Bulgo Sandstone in the Southern Coalfield. The author has applied voussoir beam theory to the Triassic sandstone in the Western Coalfield and also the Tertiary basalts in the Bowen Basin. In the Southern coalfield, mine design usually incorporates the onset of pillar yield at the tailgate and hence failure when fully goafed. There is a need to incorporate the post failure deformation of the pillars in the pillar compression calculation.

**REFERENCES**


MINE-SCALE NUMERICAL MODELLING OF LONGWALL OPERATIONS

Abouzar Vakili¹, John Albrecht¹ and William Gibson¹

ABSTRACT: Elastic three dimensional Boundary Element (BE) codes are commonly used in the coal industry to model the induced stresses and rock mass response to longwall mining. While these models are often easy to build and quick to run, it is questionable whether these elastic models are capable of accurately simulating the highly non-linear rock mass response observed in longwall operations, in particular the complex caving and goaf behaviour of the overlying strata and resulting surface subsidence.

This study presents a comparison between modelling results obtained from the finite difference (FD) code FLAC³D and elastic BE code Map3D for a generic longwall extraction sequence. These models are compared with regard to the extent of surface subsidence and associated stability of pillars.

INTRODUCTION

Abutment stability, cavability and surface subsidence are important geotechnical issues that need to be considered for most longwall operations. These issues involve significant rock mass yield and deformation, which may necessitate the use of inelastic numerical models to analyse these complex problems. While three dimensional (3D) mine-scale inelastic numerical modelling is now being routinely conducting in hard rock mines, the application of these models in the coal industry is limited, usually only conducted for research purposes and not for operational design.

Reluctance to use mine-scale inelastic 3D models by the industry has largely been due to hardware limitations, long processing times and difficulties in constructing accurate mine geometries. However, most of these limitations have been resolved through recent hardware advancements and the use of CAD software to speed up model construction times.

This paper discusses the aspects of mine-scale numerical modelling for longwall operations and presents a comparative study between elastic and inelastic codes, for a generic longwall extraction sequence.

In this study the modelling results from the finite difference (FD) code FLAC³D and elastic BE code Map3D for a generic longwall extraction sequence are compared. The accuracy of each model is compared with regard to the extent of surface subsidence and pillar stresses modelled. The ease of construction, skills required, computing efficiency and cost effectiveness of each method are also discussed.

FLAC³D MODELLING

FLAC³D (Itasca, 2006) is a three-dimensional explicit finite-difference program. Finite-difference is a domain method where the problem domain (or rock mass) is divided into geometrically simple sub-domains or elements.

FLAC³D has been commonly used for the longwall research purposes. Examples of recent studies using FLAC³D for longwall modelling include Badr et al. (2003), Yasitili and Unver (2005), and Tarrant (2006).

AMC Consultants Pty Ltd has developed a new approach for mine-scale modelling which involves the use of both Abaqus/CAE (Dassault Systèmes, 2008) and FLAC³D programs. In this approach, ABAQUS/CAE is used for geometry construction and meshing, and also for visualization of results. The numerical analysis is conducting using FLAC³D.

¹ AMC Consultants Pty Ltd, Perth, WA, Australia
The modelled generic longwall layout is shown in Figure 1-a. This model comprises six different material properties (Figure 1-b).

![Figure 1 - Overall layout of the modelled longwall panel](image)

One of the main difficulties involved in longwall modelling is the modelling of cave and goaf behaviour. In order to study the large-scale longwall caving behaviour, a computer model must be able to effectively simulate large order strain and the correct induced stresses caused by the compaction of the goaf material. This requires a thorough understanding of the post-peak behaviour of the rock mass and a representative constitutive material model. However, in small-scale and more detailed studies, there are many other factors that need to be modelled in order to effectively evaluate the caving behaviour. These factors include: detachment/rotation of blocks, frequency and pattern of discontinuities and bending/rotation of roof layers. For more detailed study on small-scale caving behaviour refer to Vakili et al, (2007,2008 and 2009).

The numerical formulation in FLAC 3D allows the use of small-strain and large-strain modes. In small-strain mode —unlike the large-strain mode— small displacements, displacement gradients and rotations are assumed. In that mode, node coordinates are not updated, and stress rotation corrections are not taken into consideration (Itasca, 2006). As the caving process in longwall operations involves large strain (including block rotation), the use of small-strain mode may not be realistic.

For this paper, the sensitivity of the model to different constitutive models and strain modes (small or large) are investigated. Elastic, perfectly-plastic and strain-softening constitutive models are compared. The post-peak response of the rock mass, in the strain-softening model, is taken from Badr et al. (2003). The extent of the yield zone for each mining step in the strain-softening model is shown in Figure 2.

The extent of the caving zone at step 6 is shown in Figure 3 for three material models used. Both perfectly-plastic and elastic models show a more or less symmetrical goaf formation. However, for the strain-softening model the caving zone forms asymmetrically, reflecting the effect of the stress redistribution around the longwall panel after each step.

One of the main difficulties with the elastic longwall modelling is associated with modelling of two neighbouring panels. As can be seen in Figure 3, unlike the inelastic models, in the elastic model a symmetrical interaction takes place between two panels. This is due to the reversible nature of the elastic deformation.
Figure 2 - Extent of yield zone (goaf) in strain-softening model

Figure 4 and Figure 5 show the extent of subsidence at the end of model steps 5 and 6. The strain-softening model shows the most non-linear subsidence behaviour. The non-linearity of this model is more obvious in Figure 4, where it can be compared with the linear subsidence profile of the elastic model. This correlates well with observed subsidence monitoring results. Compared with the perfectly-plastic case, the strain-softening model predicts less subsidence. This can be explained by the fact that goaf compaction and reloading is better represented in this model and therefore the compacted goaf act as an additional support in the system, which inhibits excessive subsidence.

Figure 6 shows the total volume of the caved material for the different material models. As expected, the strain-softening model has the maximum volume of caved material.

The assessment of abutment conditions (i.e. chain pillar stability) can be highly influenced by the choice of constitutive model, element discretisation and face advance interval. As shown in Figure 7, the strain-softening model is the only material model that can represent the true effect of goaf compaction/reloading and its associated influence on pillar stability. All of the other models underestimate the stress distribution in the pillar.

MAP3D MODELLING

Map3D (Mine Modelling Pty Ltd) is a three-dimensional Boundary-Element (BE) program. The BE is an integral method. In integral methods only the boundaries of the problem domain are divided and the domains are considered to be an infinite medium. BE programs are best suited for linear (elastic) and homogenous materials (Brady and Brown, 2004).

Map3D program is commonly used to address operational requirements in longwall mining. Examples of recent studies where Map3D was used for longwall geomechanics include Hatherly et al (2003) and Klenowski (2000).
Figure 3 - Interaction between two longwall panels in different constitutive models

Figure 4 - Subsidence after completion of first panel (step 5)
Figure 5 - Subsidence after completion of step 6

Figure 6 - Predicted and measured subsidence profiles (after Orchard and Allen, 1970)

Figure 7 - Volume of caved material in each model
In this study, similar longwall layout was modelled with Map3D and FLAC^3D. Similar discretisation was used for Map3D model to make both models comparable.

Map3D is generally best suited for linear elastic modelling. However as discussed in the previous section, caving in longwall operations introduces highly non-linear behaviour and this cannot be modelled realistically by an elastic model. In addition, because of the nature of boundary element methods, the effect of large displacements and the associated geometry changes cannot be included in the model. Longwall caving is associated with large deformations and geometry variation, and this has to be considered for a representative modelling study.

To address these problems, it is a general practice, in Map3D models, to include the goaf geometry with a different material property, with gravity load applied to represent the impact of goaf compaction.

To estimate the goaf material properties and goaf compaction characteristics, empirical methods have been generally used by researchers. Example studies include Yavuz (2003), Salamon (1990) and Xie et al. (1999).

For this paper, a ‘with goaf’ and ‘without goaf’ case were modelled. For the case with goaf geometry, the goaf dimensions (caving height/angle) were obtained from the FLAC^3D modelling results (strain-softening model). The goaf geometry is shown in Figure 9.

A range of goaf material properties and stress conditions were modelled to assess the sensitivity of results. For comparison purposes, the longwall panels were constructed using ‘Fictitious Force’ (FF) as well as ‘Displacement Discontinuity’ (DD) elements. The modelling results were compared in terms of pillar stability and overall subsidence.

The stress magnitude (defined using maximum deviator stress) in a selected chain pillar is shown in Figure 10. The modelling results for pillar stability indicate high sensitivity to goaf material properties. Both modulus and vertical stress magnitude can significantly change the state of stress on pillars. As expected, the ‘without goaf geometry’ model is more or less equivalent to the ‘small-strain elastic’ FLAC^3D model and produces similar results.
Based on these results, if the properties of the goaf material are not known, the recommended approach would be to exclude the goaf material and represent the longwall and roadway geometry using FF elements.

The subsidence results are shown in Figure 11. The results for cases where the longwall panels are modelled using FF elements are highly erratic. The results for the 'without goaf' geometry model using FF elements show significant ambiguity and are not presented here. This reflects the limitation of using FF elements in the boundary-element method when dealing with thin tabular geometries. This limitation is discussed in more detail in Watson and Cowling (1985).

Figure 11 shows that the use of DD elements results in a more realistic subsidence profile, more closely matching the FLAC3D subsidence profiles.

However, compared with the 'elastic' FLAC3D model, the MAP3D DD model indicates less overall subsidence. This can be associated with the general limitations of boundary-element method, which cannot model large displacements and the associated changes in problem geometry.

**COMPARISON BETWEEN FLAC3D AND MAP3D MODELLING APPROACHES**

To compare the suitability of the two programs, different aspects of the modelling process must be taken into account. These aspects fall into two main categories, general aspects and technical aspects.
For technical aspects of the modelling, the two programs were compared in terms of their ability to model the surface subsidence and pillar stability. The pillar stability comments are also relevant for the assessment of face and roadway stability.

Figure 11 - Overall subsidence predicted by Map3D model

Note that the comments for the FLAC\(^3\D\) modelling only apply to the improved modelling approach, which uses ABAQUS/CAE for model construction and visualization. The comparisons are listed in Table.

Table 1 - Comparison between MAP3D and FLAC\(^3\D\) programs with respect to general modelling requirements for longwall mine-scale modelling

<table>
<thead>
<tr>
<th>ADVANTAGES</th>
<th>DISADVANTAGES</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Map3D</strong></td>
<td><strong>FLAC(^3\D)</strong> (with ABAQUS/CAE)</td>
</tr>
<tr>
<td>Fast and easy model construction</td>
<td>Best suited for linear and homogenous materials</td>
</tr>
<tr>
<td>Minimum modelling expertise are required</td>
<td>Well-developed modelling expertise required</td>
</tr>
<tr>
<td>Easy post-processing of results</td>
<td>Relatively long solution times</td>
</tr>
<tr>
<td>Fast computing</td>
<td>Relatively more expensive modelling option</td>
</tr>
<tr>
<td>Weakness planes can be modelled implicitly</td>
<td></td>
</tr>
<tr>
<td>Generally considered as more cost effective</td>
<td></td>
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<tr>
<td></td>
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</tbody>
</table>
Table 2 - Comparison between MAP3D and FLAC3D programs with respect to pillar-stability modelling requirements

<table>
<thead>
<tr>
<th>ADVANTAGES</th>
<th>DISADVANTAGES</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Map3D</strong></td>
<td></td>
</tr>
</tbody>
</table>
| • Reasonable accuracy can be achieved in a large-scale global model provided sensible input assumptions are made | • Goaf geometry including caving height and caving angle must be known accurately  
| | • Goaf material properties including modulus and Poisson's ratio must accurately be known. If not known the goaf geometry should not be included in the model.  
| | • Goaf compaction/reloading effect must accurately be known to include the associated vertical stress component. |
| **FLAC3D (with ABAQUS/CAE)** |               |
| • The caving behaviour can be accurately modelled subject to application of an appropriate constitutive model  
| | • Sub-modelling technique might be required if a higher accuracy is required  
| | • No separate material property or stress condition is required for the caved material  
| | • The ground support can be modelled for stability assessment |

Table 3 - Comparison between MAP3D and FLAC3D programs with respect to surface subsidence modelling requirements

<table>
<thead>
<tr>
<th>ADVANTAGES</th>
<th>DISADVANTAGES</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Map3D</strong></td>
<td></td>
</tr>
</tbody>
</table>
| • Can provide a quick approximation of the overall subsidence profile, if the longwall panel is constructed using DD elements. | • The predicted subsidence profile can be very inaccurate in cases where high non-linearity is involved  
| | • Because of the nature of the program, the subsidence magnitudes are not reliable and must not be taken into account  
| | • Visualization of the final subsidence profile can be difficult in cases where complex topography is involved  
| | • FF elements should not be used for subsidence prediction |
| **FLAC3D (with ABAQUS/CAE)** |               |
| • Very complex and detailed topography can be included into the model  
| | • Calibration and back analysis may be required to obtain confidence about the material properties and the post-peak response of the rock mass  
| | • The non-linear subsidence behaviour can accurately be modelled  
| | • Given that appropriate constitutive model and material properties are used, the model can predict the subsidence very accurately  
| | • The subsidence profile can be visualized very easily in 3D  
| | • Small-scale subsidence effects, where detachment and shear slips are involved, cannot be modelled |
CONCLUSIONS

In this study the modelling results from the finite difference code FLAC$^{3D}$ and elastic boundary element code Map3D for a generic longwall extraction sequence were compared. These models were compared in terms of the extent of surface subsidence and associated stability of pillars.

In general, Map3D should only be used in cases where high confidence exists about the goaf geometrical characteristics (caving height and caving angle), its properties (modulus and Poisson’s ratio) and its compaction/reloading characteristics. This code is generally not suitable for subsidence analyses. Nevertheless the application of this code can be very easy and cost effective where its applicability can be justified.

The FLAC$^{3D}$ program, and in particular its combined application with ABAQUS/CAE, is generally more suitable for cases where less information is available about the caving and goaf behaviour. The program can be effectively used for subsidence prediction. This modelling approach may require higher level of expertise than Map3D and it can sometimes be slightly more expensive. However with recent improvements in hardware and software capabilities, the application of mine-scale 3D inelastic continuum models is becoming easier and more cost effective.

REFERENCES

STABILITY ANALYSIS OF TABAS COAL MINE ROADWAY USING EMPIRICAL AND NUMERICAL METHODS

Ali Sahebi¹, Hossein Jalalifar¹, Mohammad Ebrahimi¹ and Ali Abdolrezaee²

ABSTRACT: Tabas coal mine is located south east of Tabas city, in Iran. The mine is the first fully mechanized coalmine in Iran that produces 4000 tonnes coal per day. Method of extraction is retreat longwall. One of the main problems in this mine, is the stabilization of entry roadways. In this research, five different methods were used to calculate potential rock loading on roadways, and according to the predicted rock load two types of section arches; V29 and V36, were considered for stabilization. Finally, the designed support system was numerically evaluated. From the numerical analyses, it was concluded that the roadway East1 Maingate could reach to the stabilised using V29 section arch.

INTRODUCTION

Tabas Coal Mine No.1 is located in a remote rugged desert environment approximately 85 km south of town of Tabas in Yazd province in mid Eastern Iran. In 1998, the National Iranian Steel Company (NISCO) issued an international tender for Tabas Coal Mine and NISCO has selected the Joint Venture Partnership of Iran International Engineering Company (IRITEC) and IRASCO as the preferred bidder. At the time, IRITEC/IRASCO as a contractor excavated the East 1 Main and Tail Gate to commission the 1st retreat longwall coal face (the East 1 Panel) and produce 1.5 million tonne coal annually. The mine is working seam C1. The seam gradient is 1 in 5 to 1 in 2 (11° to 26°) in initial mining area. In E1 MG panel, the gradient has been observed to be between 19° and 29°. At E1 panel the seam thickness varied from 1.8 m to 2 m. The C1 seam coal has a uniaxial compressive strength of less than 6 MPa. There are some other seams C2 and D1 above and B1 and B2 below the C1 seam (IRITEC 1992). Figure 1 shows Mine No.1 and other districts of Tabas coal mine with location of the exploration shafts.

<table>
<thead>
<tr>
<th>Depth into roof(m)</th>
<th>Rock Type</th>
<th>UCS (MPa)</th>
<th>RQD (%)</th>
<th>Discontinuity Spacing (m)</th>
<th>Discontinuity Condition</th>
<th>Ground Water</th>
<th>Discontinuity orientation</th>
<th>RMR</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 2.12</td>
<td>Sandy Siltstone</td>
<td>32</td>
<td>18</td>
<td>0.06-0.2</td>
<td>Slightly rough, separation&lt;1mm</td>
<td>Dripping to dry</td>
<td>Very unfavourable</td>
<td>Class III–IV (30 – 41 )</td>
</tr>
<tr>
<td>Rating</td>
<td></td>
<td>4</td>
<td>3</td>
<td>8</td>
<td>23</td>
<td>4 - 15</td>
<td>-12</td>
<td></td>
</tr>
<tr>
<td>2.12-3.35</td>
<td>Silty Sandstone</td>
<td>73</td>
<td>26</td>
<td>0.06-0.2</td>
<td>Slightly rough, separation&lt;1mm</td>
<td>Dripping to dry</td>
<td>Very unfavourable</td>
<td>Class III–IV (38 – 49 )</td>
</tr>
<tr>
<td>Rating</td>
<td></td>
<td>7</td>
<td>8</td>
<td>8</td>
<td>23</td>
<td>4 - 15</td>
<td>-12</td>
<td></td>
</tr>
<tr>
<td>3.35-3.8</td>
<td>Sandy Siltstone</td>
<td>32</td>
<td>49</td>
<td>0.06-0.2</td>
<td>Slightly rough, separation&lt;1mm</td>
<td>Dripping to dry</td>
<td>Very unfavourable</td>
<td>Class III -IV (35 – 46 )</td>
</tr>
<tr>
<td>Rating</td>
<td></td>
<td>4</td>
<td>8</td>
<td>8</td>
<td>23</td>
<td>4 - 15</td>
<td>-12</td>
<td></td>
</tr>
<tr>
<td>3.8-4.75</td>
<td>Sandstone</td>
<td>73</td>
<td>43</td>
<td>0.06-0.2</td>
<td>Slightly rough, separation&lt;1mm</td>
<td>Dripping to dry</td>
<td>Very unfavourable</td>
<td>Class III -IV (38 – 49 )</td>
</tr>
<tr>
<td>Rating</td>
<td></td>
<td>7</td>
<td>8</td>
<td>8</td>
<td>23</td>
<td>4 - 15</td>
<td>-12</td>
<td></td>
</tr>
</tbody>
</table>

A 4.75 m long roof core taken in E1MG panel revealed that the roof strata was made of layers of siltstone, sandy siltstone and silty sandstone above the roof of the MG. According to Table 1, the sequence of the stratification above the coal seam and other details are as indicated. For simplicity in modelling, all sandy siltstone and silty sandstone were considered as siltstone and sandstone,

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respectively. The use of TH section arches are being considered for the roadway at Tabas coal mine and it is understood that both V29 and V36 section arches are under consideration. These notes examine the use of TH arches in this situation. Support of the immediate portal area, i.e. the first few arches set under the excavation lip is considered in a separate note.

![Figure 1 - Districts of Tabas coal mine and location of exploration shafts (not to scale, IRITEC 1992)](image)

ROCK LOADING

Five different methods were used to calculate potential rock loading on roadways. These were as follows:

- Airey loosened zone approach
- Geomechnics rock mass classification system
- National Coal Board (NCB) loosened zone approach
- Terzaghi design method
- Whittaker and Hodgkinson loosened zone approach

In all methods, a rock density of 2.6 (tonnes/m³) was assumed. Rock / Support interaction analysis was considered as a possible method of estimating support requirements but this was not pursued due to the lack of reliable geological / geotechnical data. If such data becomes available, estimates can be made then this approach may be re-considered as it offers a good system of design for standing supports.

Airey loosened zone approach

This assumes that a loosened zone exists above a mine roadway, created by the roof strata fracturing into a triangular shaped loosened zone governed by the angle of friction of the rock mass (Final report ECSC 1982).

Figure 2 shows the general principle of Airey Triangular Loosened Zone and this gives the following rock loads.

- Angle of friction (F1) = 23°.
- RMR = 40

In Table 2 result was shown.

\[
Hp = \frac{w}{2 \tan F_1} \tag{1}
\]

\[
P = \left(\frac{w}{2}\right) \times Hp \times \gamma \tag{2}
\]
The Geomechanics rock mass classification system allows a RMR to be determined for the given rock mass. One of the outputs from this system is a method of determining rock load, \( P \) (Bieniawski 1989). This is given as follows:

\[
H_p = \left(\frac{100 - RMR}{100}\right)W
\]

\[P = H_p \times W \times \gamma\]  

Where;

\( W \) = Roadway Width (m)

\( \gamma \) = Rock Density (tones / m³)

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National Coal Board (NCB) loosened zone approach

This assumes that a triangular loosened zone exists above the mine roadway which loads the stand in supports. Figure 4 shows general principle. For ease of calculation, the loosened zone is assumed to be triangular with a height of 1 to 1.5 times roadway width (National Coal Board MRDE 1970)

\[
H_p = (1 \text{ to } 1.5)W
\]

\[P = \frac{W}{2} \times H_p \times \gamma\]

---

Terzaghi design method

Using a combination of modal tests and observations of load on steel arch supported roadways, Terzaghi proposed a rock load classification system for steel arch supported roadways. He subdivided his classification into nine categories to cater for a variety of conditions from “Hard and Intact” to “Swelling” rock. The category chosen for this estimation is “Very Blocky and Seamy” as it is the most appropriate of the categories to suit the anticipated conditions at Tabas coal mine (IRITEC 1992). For this condition, the rock load height, \( H_p \) is given as follows:

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### Table 2 - Rock loads calculated by Airey Method

<table>
<thead>
<tr>
<th>Roadway Width (m)</th>
<th>Rock Load (tonnes/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>31</td>
</tr>
<tr>
<td>5</td>
<td>38.28</td>
</tr>
<tr>
<td>5.6</td>
<td>48.02</td>
</tr>
</tbody>
</table>

### Table 3 - Rock loads calculated, GRMC method

<table>
<thead>
<tr>
<th>Roadway Width (m)</th>
<th>Rock Load (tonnes/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>31.59</td>
</tr>
<tr>
<td>5</td>
<td>39</td>
</tr>
<tr>
<td>5.6</td>
<td>48.92</td>
</tr>
</tbody>
</table>

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Whittaker and Hodgkinson loosened zone approach

This is similar to the National Coal Board approach but assumes that the loosened zone is semi elliptical in shape and extends to a height equivalent to the width of the roadway. Figure 6 shows the general principle. (Whittaker and Hodgkinson 1971)

\[ Hp = W \]  \hspace{1cm} (9)

\[ P = \left( \frac{W}{2} \right) \times \left( \frac{\pi}{2} \right) \times Hp \times \gamma \]  \hspace{1cm} (10)

**ESTIMATION OF ROCK LOADS**

After using five different methods to calculate potential rock loading on different width roadways, the results of rock loads are summarized in Table 7.
Figure 6 – Whittaker and Hodgkinson
loosened zone method

Table 7 - Estimation of rock load based on various method

<table>
<thead>
<tr>
<th>Roadway Width (m)</th>
<th>Rock load for given Design Method (tonnes/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Airey Geomec h. (GRMC) NCB (1) NCB (2) Terzaghi (1) Terzaghi (2) Whittaker &amp; Hodgkinson Mean</td>
</tr>
<tr>
<td>4.5</td>
<td>31 31.59 39.48 32.76 102.96 41.35 33.94</td>
</tr>
<tr>
<td>5</td>
<td>38.2 39 32.5 48.75 38.61 121.55 51.05 41.91</td>
</tr>
<tr>
<td>5.6</td>
<td>48.0 48.92 40.76 61.15 46.3 145.74 64.03 52.57</td>
</tr>
</tbody>
</table>

Note on the above table:

- Only the Airey and geomechanics approach take geotechnical parameters into consideration.
- The Airey or geomechanics approach can be seen to give a good agreement with the mean and should be used if a quick approximation is required.
- NCB(1) – Triangle height equals roadway width.
- NCB(2) - Triangle height equals 1.5 times roadway width.
- Terzaghi(1) – Hp equals 0.35 times (Roadway width plus height).
- Terzaghi(2) – Hp equals 1.10 times (Roadway width plus height).
- Mean does not include Terzaghi loads as they are clearly outside the parameters given by the other methods.

ROCK LOADS FOR DESIGN PURPOSES

Apart from the immediate portal area which may be subject to dead loading, and is discussed elsewhere, there are two distinct areas along the declines to consider. These are the seismic zone and the remaining length inbye of this section (normal zone). A review of the methods for design of roadways in seismic active areas revealed that it is common practice to allow 15% addition to the static rock load in order to cater for seismic events. The following Table 8 gives rock loads for design purposes and incorporates this recommendation. It should be noted that no Factor of Safety has been incorporated into these rock loads.

Table 8 - Rock loads for design purpose

<table>
<thead>
<tr>
<th>Roadway Width (m)</th>
<th>Rock load for given section of the decline (tonnes/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Normal  Seismic</td>
</tr>
<tr>
<td>4.5</td>
<td>33.31  38.3</td>
</tr>
<tr>
<td>5</td>
<td>40.43  46.5</td>
</tr>
<tr>
<td>5.6</td>
<td>49.86  57.33</td>
</tr>
</tbody>
</table>
REQUIRED SUPPORT

The use of TH section arches are being considered for the declines at Tabas and it is understood that a support with 15.5 m² cross section is preferred. Both V29 and V36 section TH arches are considered here with a base width, internal, of 5 m giving a maximum excavated width of 5.6 m. The collapse loads for TH arches are summarized in appendix 1. These have been compiled from theoretical work and the results of actual laboratory tests. From appendix 1, the collapse loads for design purposes are as follows for these sections of TH arch. From the above information, the recommended spacing of the TH arches is as follows.

It should be noted that this spacing are the theoretical values and in practice conventional spacing would most probably be used (e.g. 0.5 instead of 0.58 etc.) although it would not be inconceivable to manufacture special struts for this project.

DISCUSSION

TH arches are of the yielding type and the load capacities quoted from the test results are obtained by ensuring that the yield clamps do not slip. This is usually achieved by welding them together. In underground use, of course, the clamps can slip and this type of arch is designed to close (i.e. reduce its internal cross section) as load is applied. Yielding arches can accept a higher strata movement than conventional rigid arches. However, high lateral movement or eccentric loads can result in the clamps locking which can lead to early failure of support. An even load distribution around the arch is critical if optimum performance is to be achieved with a TH arch. It could be argued that a long life decline is the place where yield and hence closure cannot be tolerated. In this case, a strong, rigid arch would be preferable. The report concluded that in drill and blast excavated roadways rigid arches provided better support and roadways conditions, with the possible exception of floor heave, than yielding TH arches of comparable size in conventional gate roadways. In general, the TH arches exhibited about 30% greater vertical closure than rigid arches. This trial also showed that TH arches had a slight advantage over rigid arches in machine cut conditions provided that the roof strata was strong enough to retain the cut profile and eliminate point loads.

The use of TH arches in the “seismic section” could be an advantage due to the yielding nature of the arch gives greater flexibility it their application. The arch will require to be well packed to the strata in order to function well, but this applies to any section of roadway supported by TH arches, particularly in a drill and blast section.

NUMERICAL MODELING

Usual support system of coal mines in Iran is steel arches of type TH section, so it was decided to design a suitable support system of this kind for E1MG roadway. Calculation the support pressure, were used to construct a model using the FLAC 2D software. The model results showed that; steel arch V29 with spacing of 1.0 m is the best support system for this type of roadway. Figure 7 shows the roadway East 1 main gate profile that is supported by steel arch V29.

ROCK MASS PROPERTIES

To provide input parameters (rock mass properties) for the numerical simulation, Roclab program (based on GSI classification, GSI=RMR-5) was used (Rocscience, 2002). Table 11 displays the intact rock and rock mass properties.
Table 11 - Intact Rock and Rock Mass Parameters (IRITEC 1992)

<table>
<thead>
<tr>
<th>Depth into Roof (m)</th>
<th>Intact Rock</th>
<th>Rock Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UCS (MPa)</td>
<td>m*</td>
</tr>
<tr>
<td>Coal</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>0 - 2.12</td>
<td>32</td>
<td>7</td>
</tr>
<tr>
<td>2.12 - 3.35</td>
<td>73</td>
<td>13</td>
</tr>
<tr>
<td>3.35 - 3.8</td>
<td>32</td>
<td>7</td>
</tr>
<tr>
<td>3.8 - 4.75</td>
<td>73</td>
<td>13</td>
</tr>
</tbody>
</table>

IN SITU STRESS AND SIMULATION

No field measured value for the in situ stress was available. The E1 MG was at a depth of about 210 m around the coring position. Then a vertical stress of about 5.7 MPa and the ratio of horizontal to vertical stress K=0.33 was considered for the site, according to tectonic history of the region (Taghipoor, 2008).

The numerical modelling package, FLAC 2D, was used to conduct the numerical simulations. The code is restricted to 2 dimensional problems, hence only cross sections through the roadway are presented. These problems were analyzed on the assumption of plane strain along the axis normal to the plane of the model. Figures 8 and 9 show the tunnel convergence and shear strain around the tunnel respectively. As it can be seen from the Figures 8 to 11 and Tables 13 and 14, the value of displacements and shear strain increment (especially in roof and floor of roadway) around the roadway are high. It means roadway needs to be supported.

NUMERICAL ANALYSIS

To investigate the tunnel stability the Sakurai method and et al. (1994) was used. The method evaluates the critical strain in the elastic region. Since the rock mass is under triaxial stress, it is logical to use the maximum critical strain for investigation of roadway stability. They suggested following equation (Lotfi, 1999):

\[
\log \varepsilon_c = -0.25 \log E - 1.22
\]

\[
\gamma_c = (1 + \nu) \varepsilon_c
\]

Where;
- \( E \) = Young's modulus of intact rock \( \left( \frac{kgf}{cm^2} \right) \)
- \( \varepsilon_c \) = critical strain in UCS state
- \( \gamma_c \) = critical strain
- \( \nu \) = Poisson's ratio

Critical displacement values based on the critical strain are obtained by following equation (Lotfi. 1999)

\[
\varepsilon_c = \frac{U_c}{a}
\]

Where;
- \( U_c \) = Allowable displacement;
- \( a \) = radius of the roadway;

The maximum horizontal and vertical displacements around the tunnel before the steel arch installation are shown in Table 13. Table 14 shows the critical strain values around the tunnel. As it shows, the strain value is more than the allowable strain values, which causes the instability of roadway. Table 12 shows the properties of V29 steel arch.
The numerical results after steel arch installation showed that TH Arch V29 is suitable to support the E1MG. After installation it was observed that the critical strain values on roadway walls and roof were less than the permitted values which demonstrated the roadway stability. Figure 11 displays the vertical displacement after installation of steel arch V29 in E1MG, which shows that there is a good agreement with the experimental result.

**CONCLUSIONS**

From the empirical and analytical methods and numerical simulations following conclusions can be inferred.

- Horizontal and vertical displacements appeared to be high which showed roadway needs to be supported.
- Elastic strains are good indications to show the roadway instability. It means Sakoraei method is quite applicable to predict the tunnel instability.
- The numerical simulations indicated that there is a good agreement between empirical, numerical and field monitoring data.
REFERENCES

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Final Report on ECSC Research Project No 7220, 1982, Theoretical and practical studies towards improved control of strata around mine roadways, MRDE.
Itasca Consulting Group Incl. 2000, FLAC version 5.0, Thresher square east, Minneapolis, Minnesota, USA.
IRITEC, 1992, Internal Reports of Tabas coal mine, pp 110-135
Taghipoor S, 2008. Application of numerical modeling to study the efficiency of roof bolting pattern in east 1 main gate of Tabas coal mine, 6th international conference on case histories in geotechnical engineering, Arlington, pp 2-5
### APPENDIX 1: COLLAPSE LOADS FOR TH TYPE ARCHES

<table>
<thead>
<tr>
<th>Arch Type</th>
<th>Arch Dia (m)</th>
<th>Theoretical Collapse Load (tonnes)</th>
<th>Collapse Loads from Practical Tests (tonnes)</th>
<th>Likely Collapse Load For Design Purposes (tonnes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V29</td>
<td>5</td>
<td>41.3</td>
<td>32.5 to 44.2</td>
<td>22</td>
</tr>
<tr>
<td>V36</td>
<td>5</td>
<td>57.75</td>
<td>43.6 to 75.3</td>
<td>35</td>
</tr>
</tbody>
</table>

Note on the above table:

- Theoretical loads given by equation, load = \(2.2 \frac{Z}{D}\) where \(Z\) = section Modulus (\(\text{cm}^3\)), \(D\) = Arch dia (m) and is taken (Sadler, 1984)
- British steel tests are given in the form of tables issued in about 1986. These tests were carried out for British Steel by the National coal Board (subsequently British coal). Some results quoted were extrapolated from tests on other sections using the section Modulus as the main criteria.
ASSESSMENT OF SEISMIC EVENTS IN GERMAN HARD COAL MINING – OCCURRENCE AND PREDICTION

Axel Preusse¹, Heinz-Jürgen Kateloe¹ and Anton Sroka²

ABSTRACT: Underground coal mining can cause seismic events. There is currently no way to predict such phenomena reliably, in particular the stronger events. That is why, given the current state of the art, empirical methods are employed in this field.

Some assessment criteria, which relate to the geological and mining situation and either stimulate or rather help to avoid seismic events, are presented in this paper. On a practical mining situation in Germany, these criteria are examined with a view to predicting possible seismic events.

INTRODUCTION

Mining-induced seismic events are part of underground mining operations (Preusse, 2008, Fritschen, 2002 and Sroka, 2008). They occur in different intensities, and the extent, to which they are perceptible on the surface, depends on several geological and mining operational factors.

According to international expertise it can be stated that the occurrence of mining-induced seismicity depends on natural and mining operational conditions. Natural factors include geological conditions as well as tectonic stresses. Besides the geological conditions, anthropogenic factors also play an important role, for example, multiple seam mining, existing mining boundaries, and residual pillars, as well as the mining method and the face advance rate at the time of the seismic event.

Fritschen (2002) demonstrated the static correlation between extracted coal volumes and released seismic energy. However, methods for detailed prognosis of seismic events are not available so far.

The currently used method to assess possible seismic events due to future mining operations in a certain part of a deposit, which is planned for mining, is an empirical approach, i.e. the prediction of seismic events in future mining fields is based on experiences made in other fields, whereas it is assumed, that the most significant parameters for the occurrence of seismic events do not change. The more similar the geological conditions between experienced and future mining situations are (e.g. thickness of seam, seam depth below surface and type of overlying rock strata), the more precise such a prediction would be.

On 23th February 2008, the German federal state of Saarland experienced an especially strong mining-induced ground vibration measuring 4.0 on Richter scale with a maximum vibration velocity of 93.5 mm/s (see Figure 1). Due to this event and the resulting endangerment for health and life of people, the mining authority of the provincial government imposed an immediate stop of the longwall that caused the tremors. Due to this seismic event, RAG Company decided to cease the entire production at Saar Colliery temporarily. In the following period, RAG investigated other opportunities to continue with mining operations in other mining fields with minimum risk of mining-induced seismic events. A conceivable alternative was to shift the production to Grangeleisen seam of the Nordfeld coal deposit, Saar Colliery. The mine is a multi-seam operations, which mines coal from separate levels and hence the term coalfield has been used in this context.

The risk of ground vibrations due to a continuation of panel 20.4 East, Grangeleisen seam, was analyzed with regard to geological and mining operational conditions (Preusse, 2008). The assessment was based on a general technical expertise and findings from former seismic events at Saar Colliery.

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DEFINITION OF ASSESSMENT CRITERIA FOR THE OCCURRENCE OF MINING-INDUCED GROUND VIBRATIONS

Geological and mining operational assessment criteria

The following geological and mining operational (i.e. anthropogenic) assessment criteria (see Figure 2) are verifiably responsible for the occurrence of mining-induced seismicity. They can be found in international literature (Gibowitz and Lilko, 1994) as well:

- **Geological assessment criteria:**
  - Rock structure (competence of rock; especially sand rate)
  - Depth of planned mining horizons
  - Mined seam thickness
  - Maximum horizontal stress / anisotropic stress field in rock mass

- **Mining operational assessment criteria:**
  - Mining concentration (initial panel / multiple seam mining)
  - Panel ribsides and residual pillar
  - Face length of longwalls
  - Face advance rate and its change
  - Already known zones of seismicity

Analysis of assessment criteria on the activation of ground vibration in German coal mines

The following factors are considered to influence the risk of various events:

*Sandstone percentage in layers above the underground workings*

Any thick layer with a high sandstone percentage above the extracted coal seams, increases the general risk of seismic events due to mining operations. The percentage of sandy particles included in rock layers above the workings at Saar Colliery was between 18 and 70 %. 
Mining depth

The probability of having mining-induced ground vibrations increases consistently with the mining depth. This assumption was proved by several seismic events that occurred in the past. The strongest event so far occurred due to mining operations of Saar Colliery at a depth of approximately 1,450 m.

Mined seam thickness

The mined seam thickness as well is an influencing factor on mining-induced seismicity. Based on experiences derived from mining activities in Saarland, it can be stated that a mined seam thickness between 3.0 and 3.5 m will most likely cause ground vibrations with a higher intensity. For the mining operations in the Grangeleisen seam, Nordfeld coal field, however, the workable thickness was about 2.3 m.

Horizontal stress/ anisotropic stress field in underground rock strata

A further geological criterion with an impact on increased seismicity is an anisotropic stress field in the rock strata. This means there is a significant difference between the horizontal and vertical stresses. Therefore, information about the range of horizontal stress is investigated in situ by drilling in an unaffected area.

Multiple seam mining

In the extensively mined parts of the deposit (multiple seam mining), it can be assumed that the structure of rock strata as well as the hanging side of the roof strata is weakened and loosened (Kratzsch, 1997). This increases the sequence of ground movements, which will unavoidably occur due to underground coal extraction. However, according to documents and findings at Saar Colliery, mining operations that were preceded by mining in two or three different seams above the recent workings have actually provided better situations. This is proved by seismic activities which occurred due to mining operations in other mining fields of Saar Colliery.

Undermining of panel boundaries and residual pillars

Undermining panel boundaries and residual pillars, which were left behind by upper seam mining, generally increases the risk of seismic events. This fact can be concluded from mining operations in panel 20.3 East in the Grangeleisen seam of the Nordfeld coal field (see Figures 2 and 3), where two previously mined seams were undermined.

Face length (single face/ double face mining operations)

Based on the experiences at Saar Colliery it can be concluded that double face mining operations (two directly adjoining longwalls with same face advance) have a negative impact on possible mining-induced seismic events. Consequently, any mining operation at Saar Colliery should be converted from double face to single face mining.

Face advance rate and its change

According to findings at Saar Colliery, fast face advance rates as well as significant changes of the face advance rate have negative impact on potential seismicity. This is why operations should advance moderately and constantly, without any long periods of stoppage (e.g. extended weekends).

Zones of seismic events which are already known

Previous seismic events, which resulted from mining activities in the past, might have had different reasons, e.g. high percentage of sandy particles in the surrounding rock, larger seam thickness and undermining of residual pillars. However, they are indicators for further seismic events which will possibly follow due to subsequent mining activities.

An analysis of the previous seismic events in other coal fields of Saar Colliery pointed out that the occurrence of these events concentrates on narrow vibration corridors and their intersecting areas. However, such findings are presently not on hand for operations in the Grangeleisen seam in Nordfeld.
ASSESSMENT ON POTENTIAL SEISMIC EVENTS DUE TO OPERATIONS AT SAAR COLLIERY

THE SITUATION AT SAAR COLLIERY IS AS FOLLOWS:

Until 2008, the coal fields of Saar Colliery were Primsmulde South, Dilsburg East, Dilsburg West and Nordfeld. The operations took place in Schwalbach, Wahlschied and Grangeleisen seams (from the upper to the lower seams). On 23rd February 2008, when the strong seismic event occurred and mining activities at Saar Colliery had to be stopped, the following seams were mined:

- Primsmulde South: Schwalbach seam, panels Prims1 and Prims2
- Nordfeld: Grangeleisen seam, panel 20.4 East

The operations in Schwalbach, Wahlschied and Grangeleisen seams of the Nordfeld are shown in Figure 3. Figure 4 focuses on the mining situation in Grangeleisen seam.

![Figure 3 - Mined panels in mining field Nordfeld at RAG Saar Colliery](image)

![Figure 4 - Plan view of panels in seam Grangeleisen as well as working boundaries and pillars in seam Wahlschied](image)
Concerning the continuation of mining in panel 20.4 East, Grangeleisen seam, the following results were derived from the analysis of the assessment criteria (see previous section), examining the activating conditions for mining-induced ground vibrations.

**Relatively favourable conditions are:**

- Low percentage of sandy particles in overlying strata (18%)
- Relatively low seam thickness (about 2.3 m)
- Extensive previous extraction in Schwalbach and Wahlschied seams located above
- Single face mining (face length about 360 m)

**Neutral conditions are:**

- Moderate face advance rate (up to 5 m/ day)
- Medium mining depth (between 1050 and 1170 m)

**Highly unfavourable conditions when encountering:**

- Panel boundaries
- Residual pillar in Wahlschied seam (south of panel 20.4 East)
- Seismic events already occurred in the past

Based on the above described assessment it was assumed that no stronger seismic events were to be expected while mining operations continued in panel 20.4 East, Grangeleisen seam, after it had been restarted gradually. Finally, panel 20.4 East was mined as planned, verifying the described assessment.

**SUMMARY**

Underground hard coal mining can cause seismic events. Currently, there is no method available to reliably predict such occurrences, especially in terms of stronger seismic activities. According to the state of technology, any assessment on that topic is thus based on empirical methods.

Several geological and mining-operational criteria, which either support or rather help avoid the occurrence of seismic events, are presented. They are examined for the given mining situations in Saarland, Germany, with the purpose of assessing possible mining-induced ground vibrations.

**REFERENCES**


SEISMIC ANALYSIS OF HORSESHOE TUNNELS UNDER DYNAMIC LOADS DUE TO EARTHQUAKES

Navid Hosseini¹, Kazem Oraee², and Mehran Gholinejad³

ABSTRACT: Due to seismic events, such as earthquakes, the elastic waves propagate through a medium. The impact of these waves on underground structures is to provide dynamic forces and moments that may affect the stability of underground structures. The aim of this paper is to analyse the effects of seismic loads on the stability of horseshoe tunnels. As a case study, the stability state of the main access entry to C1 coal seam of Tabas collieries in Iran are analyzed using Phase2 software in static and dynamic states. It is often assumed that the effect of earthquakes on underground structures such as tunnels is negligible but the results of this study show that the stress caused by seismic loads can be harmful to the tunnel stability. It is concluded that the stress and displacement balance of forces around the tunnel are adversely affected and due to redistribution of these forces that create undue concentration in some areas, instability occurs in the tunnel. The paper also concludes that increasing the stiffness of the support system can increase the effect of the seismic loads. The analysis provided in this paper together with the conclusions obtained can serve as useful tools for the tunnel design engineers, especially in areas susceptible to seismic phenomena.

INTRODUCTION

In the past it has always been assumed that earthquakes have no major effect on tunnels, however the study of tunnel behaviours on seismic loads and also the damage of these structures, emphasize the necessity of the stability study under dynamic loading generated by earthquake (Williams, 1997).

Tabas coalfield is a main coal reserve that is located in the central part of Iran. The coal is mined by mechanized longwall mining method based on physical properties and geometry of coal seam (Hosseini, 2008). Several excavation tunnels are needed when using the longwall method (Oraee, 2001). However, due to several faults in Tabas collieries\ the stability study of these tunnels under dynamic and seismic loading is needful. Therefore in this paper as a typical case the stability of the main access tunnel in C1 coal seam is studied.

THE EFFECT OF SEISMIC WAVES ON UNDERGROUND STRUCTURE STABILITY

Each earthquake wave has a different effect on tunnel stability; these are described as follow:

P-waves

P-waves are usually concomitant with horizontal S-waves. P-waves create the axial compressive and tension on underground structure, while the horizontal S-waves only create a horizontal vibration (Wang, 1993). Therefore the horizontal S-waves have the major effect on high structure while their effect on underground structure is poor. Tunnels and other flexible linear underground structures based on a flexible ring, such as the support system can tolerate the effect of horizontal S-waves. P-waves rapidly propagate in the ground, and are thus the first waves affecting the structure.

Vertical S-waves

Vertical S-waves are a principal kind of elastic waves and carry about 65 percentage of the released seismic energy. These waves cause vertical displacement of the structure system and seriously damage the major structure, but for tunnels and other underground structures the effects are negligible, since using the flexible support system will neutralize the effect of these waves (Ebrahimi, et al, 2006). The velocity of vertical S-waves is less than that of the horizontal waves. Therefore the periodical
interval between vertical and horizontal S-waves relates to the distance between the structure and earthquake hypocenter.

Rayleigh waves

In Rayleigh waves, the direction of the spinning motion in the highest zone and direction of waves are opposite; the path of particle motion is elliptical and the large diameter is perpendicular to the direction of wave propagation. Rayleigh waves like the vertical S-waves are critical for high structure damages (Wang, 1993). The underground structures are vertically displaced based on height as a consequence of these waves.

Love waves

Love waves are a special type of horizontal S-waves which result in horizontal displacement. This displacement decreases by increasing the depth of the structure. Generally, a love wave is an important factor in threatening the underground structure. Tunnels experience the lateral dynamic displacement due to impact by love waves; the effect of the impact is different on different parts of the structure (Ebrahimi, et al, 2006). If the stress added is more than that of the structure safety limit, the lateral stiffness of underground structure must be increased for coordinating with a new loading state.

THE MAIN ACCESS TUNNEL OF C1 COAL SEAM AND SURROUNDING ROCK MASS

One of the main coal seams of Tabas coalfield in Iran is named C1, having 2 meters thickness and is associated with sandstone, siltstone and mudstone layers. Based on studies (Hosseini, 2008, Oraee, 2009), the geo-mechanical properties of coal seam and surrounding rock mass are given in Table 1.

Table 1 - Geo-mechanical properties of coal seam and surrounding rock mass

<table>
<thead>
<tr>
<th>Rock type</th>
<th>( \sigma_{ci} ) (MPa)</th>
<th>( m_i )</th>
<th>GSI</th>
<th>( D ) (kg/m³)</th>
<th>( E_i ) (GPa)</th>
<th>( \nu )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone</td>
<td>16.1</td>
<td>17</td>
<td>29</td>
<td>2700</td>
<td>5.281</td>
<td>0.32</td>
</tr>
<tr>
<td>Siltstone</td>
<td>25.6</td>
<td>12</td>
<td>21</td>
<td>2730</td>
<td>2.838</td>
<td>0.31</td>
</tr>
<tr>
<td>Siltstone &amp; Sandstone</td>
<td>57.8</td>
<td>15</td>
<td>24</td>
<td>2715</td>
<td>4.885</td>
<td>0.31</td>
</tr>
<tr>
<td>Mudstone</td>
<td>10.1</td>
<td>9</td>
<td>19</td>
<td>2650</td>
<td>0.343</td>
<td>0.30</td>
</tr>
<tr>
<td>Coal</td>
<td>7.0</td>
<td>12</td>
<td>19</td>
<td>1350</td>
<td>0.260</td>
<td>0.29</td>
</tr>
</tbody>
</table>

In this table, \( \sigma_{ci} \) is a uniaxial compressive strength of intact rock, \( m_i \) is a constant of intact rock, \( GSI \) is a geological strength index, \( D \) is a density, \( E_i \) is a young’s modulus and \( \nu \) is Poisson’s ratio. In tunnel stability analysis also the estimation of further mechanical properties of surrounding rock mass are required. For this purpose, Rocdata software provided by Rocscience Inc. (2009) is used to estimate the full geo-mechanical parameters of rock mass by comparison to the main rock failure criteria such as Hoek-Brown, Mohr-Coulomb, Bratton-Bandis and Power Curve. Based on the data in Table 1, and by using the RocData software, the estimation of other rock mass parameters is presented in Table 2. Based on tunnel excavation method and engineering judgment (Hosseini, 2008), the selected disturbance factor (Oraee, et al, 2009) is 0.3.

In this table, \( C \) and \( \Phi \) are the cohesion and friction angles based on the Mohr-Coulomb criterion, \( \sigma_t \) is the rock mass tensile strength, \( \sigma_C \) is the uniaxial rock mass compressive strength, \( \sigma_{cm} \) is the global rock mass compressive strength and \( E_m \) is the rock mass modulus of deformation. Due to large deposit and using of mechanized longwall mining, the main access tunnel in C1 coal seam must be stable for a long time and even during the entire life of the mine. This tunnel is excavated into a horseshoe section shape, with width and height of 5 m and 3.5 m, respectively. The average of overburden density is calculated at 2.7 t / m² per cubic meter, and the tunnel depth of ground surface is 40.8 m (Oraee, et al, 2009). The in-situ stress state is calculated by equations (Sheoru, 1994) as follows:

\[
\sigma_v = \gamma \cdot h
\]
\[ k = 0.25 + 7E_h \left( 0.001 + \frac{1}{h} \right) \]  
\[ \sigma_h = k \sigma_v \]  

(2)  

(3)

Table 2 - The estimated rock mass geo-mechanical parameters by RocData software

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Mohr-Coulomb</th>
<th>Rock mass parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C (MPa)</td>
<td>( \phi ) (Deg.)</td>
</tr>
<tr>
<td>Sandstone</td>
<td>0.144</td>
<td>41.01</td>
</tr>
<tr>
<td>Siltstone</td>
<td>0.120</td>
<td>37.42</td>
</tr>
<tr>
<td>Siltstone &amp; Sandstone</td>
<td>0.189</td>
<td>47.01</td>
</tr>
<tr>
<td>Mudstone</td>
<td>0.071</td>
<td>27.32</td>
</tr>
<tr>
<td>Coal</td>
<td>0.045</td>
<td>32.00</td>
</tr>
</tbody>
</table>

Where, \( \sigma_v \) is the vertical in-situ stress, \( \gamma \) is the average density of overburden, \( h \) is the depth below ground surface, \( k \) is the ratio of horizontal to vertical in-situ stress, \( E_h \) is the average of horizontal deformability modulus and \( \sigma_h \) is the horizontal in-situ stress. Based on Equation (1), \( \sigma_v \) is calculated 1.081 MPa. It is safe to assert that \( E_h \) is underestimated and therefore \( k \) is less than the stated value, i.e. considered to be the worst possible scenario. Therefore, \( E_h \) is selected as 0.25 GPa and \( k \) is calculated as 0.3 using Equation (2). Therefore based on equation (3), \( \sigma_h \) is 0.32 MPa.

NUMERICAL MODELING

The Phase2 software produced by Rocscience Inc. (2009), is used for modelling of the main access tunnel of C1 coal seam. This software is a numerical code, based on the two-dimension finite element method. For studying the tunnel stability the Mohr-Coulomb criterion is selected due to geo-mechanical properties of rock mass (Brady and Brown, 2004). The affected zone is considered to be three times the dimension of the tunnel as in a Box Shape. The meshing is triangular but with getting nearer to the tunnel for increasing the analysis accuracy, the node density increases and therefore the mesh fines. The tunnel modelled by Phase2 is depicted in Figure 1. As seen in this figure, in Phase2 modelling the geometry and state of associated layers and the coal seam are defined relative to the tunnel.

Static and dynamic analysis

After tunnel modelling in the Phase2 code, the model is run to analyse the tunnel stability in static and dynamic conditions. The Phase2 software calculated the value of each mesh node based on two dimension finite element method, having the ability of pseudo dynamic analysis and hence can simulate the effect of earthquake on tunnel stability.

Using statistical methods and probability analysis based on studies in Tabas coalfield (Hosseini, 2008), the peak seismic acceleration due to earthquake by using field data and results of faulty studies in region, is calculated 0.29g for return period of 500 years. To study the tunnel stability state under such an earthquake act, in next stage after the static analysis, the horizontal seismic acceleration 0.29 is applied on the model. Therefore, the tunnel stability in static and dynamic conditions for horseshoe tunnels is analyzed. The maximum principal stress (\( \sigma_1 \)) and the minimum principal stress (\( \sigma_3 \)) in static and dynamic analysis is shown in Figure 2.
As seen in Figure 2, both the maximum and minimum principal stresses are increased after applying the dynamic loads. However the increase in $\sigma_1$ is more than that in $\sigma_3$.

In Figure 3, the displacement in static and dynamic conditions is shown. Due to the application of the dynamic stress the displacement state of tunnel periphery is changed, and the displacement in tunnel to the left side is more than to the right side. Also the strength factor is one of stability analysis criterion. The strength factor of tunnel periphery in static and dynamic analysis is shown in Figure 4. Although the strength factor by applying the dynamic stress of earthquake is changed, this variation is not significant.
Based on the initial stress analysis and considering the displacement, the design of a support system is required for tunnel stability. Therefore the shotcrete with the young modulus of 30 GPa, and the Poisson ratio of 0.2 is used as the support system. First, one shotcrete layer with 50 mm thickness is applied in the model, and the displacement in static and dynamic analysis is determined. Then the shotcrete thickness increases to 150 mm while the other conditions remain same. The displacement with 50 mm and 150 mm shotcrete in static and dynamic analysis is shown in Figure 5.

The stress and displacement of tunnel periphery shows that the increasing thickness of the shotcrete layer increases the effect of dynamic stress. The acquired result is verified based on study developments and several similar modelling. Moreover it is also approved that the effect of dynamic loads on tunnel stability increases with the increasing stiffness of the support system.

The results of numerical modelling of Phase2 software shows that after seismic loading the maximum axial force is 0.812 MN, the maximum bending moment is 0.016 MN per meter, and the maximum shear force is 0.11 MN - applied on tunnels that must add to static loads, before the tunnel stability analysis.

**CONCLUSIONS**

Although the damage of earthquake in an underground structure is less than that on the surface structure, the applied dynamic stress is not negligible. Among elastic waves of an earthquake the love wave is particularly dangerous to the underground structure. The result of the study shows that with applying the dynamic stress by earthquakes, the stress and displacement in tunnel periphery is increased. Therefore, for tunnel stability the support system must be reinforced. However with the
increasing thickness or stiffness of the support system, the inertia is increased and thus the tunnel flexibility is reduced. Consequently the effect of the dynamic stress on the tunnel increases. The symmetry of stress and displacement distribution of tunnel periphery is adversely affected due to dynamic loading. Based on the direction of the motion of the seismic wave, displacement on one side of the tunnel is more than that on the other side; therefore the balance is disrupted and the potential of instability increases. Due to increases of the axial force the bending moment and the shear-force applied on tunnel by seismic loading, the dynamic analysis and also static analysis for tunnel stability is required.

REFERENCES


Oraee Kazem, 2001. Underground coal mining, Polytechnic University Press, Tehran, Iran, p. 251


INNOVATIONS IN MINE ROADWAY STABILITY MONITORING USING DUAL HEIGHT AND REMOTE READING ELECTRONIC TELTTALES

David Bigby¹, Keith MacAndrew¹ and Ken Hurt¹

ABSTRACT: Rockbolting telltales are now an internationally established means of providing pre-emptive warnings of roof falls. The dual height telltale, providing an immediate visible measurement distinguishing between movement above and below the rockbolted height, is the most widespread version. The dual height telltale was first developed by British Coal in the early 1990's as rockbolting was introduced to replace steel arch support and the success of this support system in deep coal mines has been widely ascribed to the use of this safety device.

Since its adoption, many permutations and improvements on the basic design have been developed and applied worldwide to suit different mining circumstances; for instance, triple height telltales are commonly used where a combination of roofbolts and longer tendons are installed at the face of the heading. The choice of appropriate movement action levels is vital for safety. Experience has also shown that systematic management of the application of the telltale warning system is required to ensure that appropriate action (usually the installation of additional support) is taken in time when action levels are exceeded. In Australia, this is exemplified in the TARP approach.

Another major development has been an intrinsically safe remote reading dual height telltale system which allows up to 100 electronic telltales to be connected with a twin core cable and read, using either a portable readout, from the end of the roadway, or a surface PC via a telephone cable connection. In the latter configuration, a real time display of roof condition is obtained whilst retaining the immediate visual indication underground. A recent development is the “Autowarning” telltale. This provides a warning of impending goafing in depillaring operations via high visibility, flashing LEDs. The paper describes these and other telltale developments and provides case histories of their application worldwide, including UK, India, and USA.

INTRODUCTION

The term ‘telltale’ is used to denote a strata extensometer which incorporates a visual indication of strata movement into an excavation and is intended to provide a visible warning of excessive ground deformation.

The application of telltales is primarily in rockbolt supported mine roadways to give warning of excessive roof or rib movement. Rock Mechanics Technology (now part of Golder Associates (UK) Ltd) has developed telltale devices ranging from the simplest mechanical types through to the latest electronic and auto warning configurations. These products and their applications worldwide are described in this paper.

DEVELOPMENT AND APPLICATION OF THE MECHANICAL STRATA MOVEMENT TELTTALE

Single height telltales

The principal application of telltales is to monitor strata deformation in rockbolt supported roadways. This is necessary because visual indications of excessive movement are not always present.

At its simplest, a mechanical telltale consists of a strata movement indicator (usually with coloured bands and/or graduations) positioned in the mouth of a drilled hole and attached to an anchor installed up the hole. The earliest telltales were simply longer bolts, point anchored above the support bolt horizon, and left protruding from the roof to indicate movement within the bolted horizon. These suffered from the disadvantages of limited monitored height and false readings caused by roof shear, which can result in the telltale bolt being trapped along its length and pulled down with the roof. A

¹ Golder RMT, Golder Associates (UK) Ltd
A typical single height telltale now consists of a reference tube, an indicator tube, a stainless steel wire and a spring anchor positioned at twice the bolted height, as shown in Figure 1.

Figure 1 - The single height telltale

One of the problems with purely mechanical telltales installed in the roadway roof is the difficulty in reading the graduated scale in high roadways. This problem has been overcome for single height telltales by developing the Rotary Telltale (Figure 2). The device converts roof movement into rotation of a pointer round a dial and magnifies the movement by a factor of fifteen. This has been developed to meet a South African requirement for a routine monitoring system that is easy to install, easy to read, accurate to better than 1 mm and low cost. Small movements can be read easily with the reading visible from below, even in a 5 m high roadway. The dial face is subdivided into coloured bands corresponding to chosen action levels. These rotary telltales are currently being used in several South African mines. The rotary principle has been patented and a low cost design is currently manufactured under licence in RSA.

Figure 2 - The rotary telltale

Dual height telltales

A shortcoming of single height telltales is that they do not provide information on the position of any movement occurring within the monitored length. Movement above the rockbolted height is an important parameter, which can indicate that the bolt system is failing to provide effective support. The type of additional support required in response to movement above the bolted height will usually differ from that needed for deformation within the bolted height (long tendons or standing support for the former compared with additional bolts for the latter), and ideally a telltale should distinguish between movement in these zones.
Telltales with two or more anchor positions, including the most commonly used types in Australia, have been introduced to do this. However, although it is possible to determine the movement magnitudes above and within the bolted height using additional anchor positions, without the concentric indicator arrangement described below, it is still necessary to compare and subtract the readings in order to do this. This defeats the primary role of the telltale in providing an immediate visible warning, particularly of movement above the bolted height.

In order to facilitate the introduction of rockbolt support into UK coal mines, staff working for British Coal overcame this problem by devising the dual height telltale, which in 1992 was patented in Britain and coal mining countries worldwide, including Australia (Figure 3). The key feature of the dual height telltale is that it directly indicates movement both within and above the bolted height. This is achieved using two anchor positions in conjunction with the concentric design of the movement indicators (Figure 4).

The anchorage arrangement means that the A indicator directly displays roof deformation within the bolted height and the B indicator suspended below directly shows deformation above the bolted height, without any necessity for calculation by the observer. The A indicator anchor is normally positioned approximately 0.3 m below the top of the rockbolts to coincide with the boundary of the fully reinforced zone. The B anchor position is typically at least twice the rockbolted height. A full description of the development of this device is given by Bigby and DeMarco.

In the UK, dual height telltales are installed at maximum intervals of 20 m along all rockbolted roadways, with the B indicator anchored at least twice the bolted height above the roof horizon. The telltales are observed by the District Official at least once during his shift and any indicating excessive movement noted on his statutory report. Similar practices have been adopted elsewhere where rockbolting is used in relatively difficult mining conditions involving weak roof and/or high local rock stresses, for example in Europe and Canada. A ground control safety procedure based on the use of telltales has been widely credited as a major factor in the successful introduction of rockbolting in these conditions.

**Telltales action levels**

The use of telltales as an effective safety warning device requires procedures detailing how they should be deployed and read and specifying what action should be taken, depending upon the level of roof dilation measured. This is embodied in the Australian TARP (Trigger Action Response Plan) concept.

In the UK specific guidance is provided in a document published by the UK Health and Safety Executive (HSE). This document recommends that the Colliery Manager should specify procedures for auditing of routine monitoring devices in rockbolted roadways, actions to be taken and the person
responsible for taking the action. There must also be a “Schedule of measurement zones and measurement frequency”. In “Inbye” areas, telltales should be observed and reported each shift and measured and recorded at least weekly. In areas of known or suspected instability they need to be measured and recorded at least daily and in other areas they should be measured and recorded at least once each month. Figure 5 shows a typical graph of a set of weekly Dual Height Telltale data for a gateroad, showing where remedial action has been taken.

Figure 5 - Weekly dual height telltale data

Since their introduction the coloured bands on standard UK dual height telltales have been green (0-25 mm), yellow (25-50 mm), red (50-75 mm) and, by default, action to provide additional support has to be taken after each 25 mm of movement on either indicator. The rationale behind these action levels was drawn from a number of sources. Firstly, there was French experience that 100 mm of total roof movement could be tolerated before roof failure. Thus a warning at 25 mm and action at 50 mm would seem appropriate for an unstable time trend (as described by Stillborg). This was supplemented by sonic extensometer data from Australian mines, provided by ACIRL during the introduction of rockbolting to the UK, which indicated similar levels of movement prior to roof falls developing. Secondly, it takes into account the typical strain level to failure of coal measures rocks of around 4 mm/m. This translates into 10 mm displacement on either indicator for a 2.4 m rockbolt, if the strain were spread throughout the monitored height. The 25 mm transition to yellow thus indicates a mean strain of 10 mm/m throughout the monitored zone, which would indicate that significant zones of softening are likely to be forming in the monitored column.

As experience with rockbolting in the UK developed and, in particular, a large quantity of sonic extensometer and telltale data was gathered, more site specific action levels were developed. Examples of determining site specific action levels are given by Kent, Cartwright and Bigby. Although standard dual height telltales still use 25 mm coloured bands, some mines find it necessary to take remedial action at deformation levels below 25 mm and in these cases, specialised telltales with different widths of coloured band may be used. Lower action levels have been needed, particularly for the B indicator, where the roof is more brittle than usual and/or where the levels of in-situ horizontal stress are low. In lower horizontal stress conditions, loosened roof is less likely to be clamped in place by roof shortening in the way that it is under high horizontal stress, leading to a danger of roof falls at lower levels of roof dilation.
Shear detection

Although the wire used in the dual height telltale to suspend the indicators is less susceptible to being trapped by roof shear than the rod type telltales formerly used, this can still happen in actively shearing strata, resulting in false readings. A shear detection facility is now incorporated as standard in Golder RMT mechanical telltales to allow any trapping of the wire in high shear conditions to be easily detected. The addition of a spring attachment at the wire/anchor joint allows the indicators to be pulled downwards slightly, following which they should return to their initial position. If they resist this downward pull, then the wires are trapped.

Other mechanical telltale types

Since the adoption of the mechanical telltale, many permutations and improvements on the basic design have been developed and applied to suit different mining circumstances. The Triple Height Telltale (Figure 6), for example was developed in response to the increasing use in the UK of steel wire strand flexible rockbolts as an additional support to standard full column resin anchored rockbolts. A typical UK roof bolting pattern employing flexible bolts would include a mixture of 2.4 m full column resin anchored rockbolts (at a minimum density of 1 bolt per square metre) and 4 m, 90% column, resin anchored, flexible bolts. As its name suggests, the Triple Height Telltale has an additional concentric indicator when compared to the Dual Height Telltale. The A indicator (nearest roof) is anchored 0.3 m below the top of the rockbolts, the B indicator (middle) is anchored 0.5 m below the top of the flexible bolts and the C indicator (lowest) is anchored a minimum of 5 m above the roof horizon or 1m above the flexible bolts if they are longer than 4 m. This allows the mine personnel and particularly the support engineer to easily determine whether any measured roof dilation is occurring within the bolted height (A indicator), in the roof zone reinforced by flexible bolts alone (B indicator), or above the reinforced height (C indicator) and so allows the most appropriate type of remedial support to be applied where required.

Figure 6 - The triple height telltale

Many other permutations of the telltale are in use in smaller quantities for specialised applications. These include special extended reference tube designs to bring the indicators to a lower height in high roadways. The ribside indicator telltale is an example. Here a Bowden cable type mechanism is used to transfer the dual height indicators to the ribside to enable easy reading. In addition to a wire for each indicator position, a third wire is used within the cable sleeve to indicate the reference tube position. This eliminates any error due to wire/sleeve expansion or contraction.

With conventional telltales, wire tensioning relies on the weight of the indicator. This makes them less suitable for use in deforming ribsides and consequently spring loaded versions have been developed for this purpose.

Worldwide applications

Over the last twenty years the mechanical telltale has been adopted in a large number of deep coal mining industries worldwide, both in its standard dual height form and in a number of alternative forms developed to meet the specific local mining and geotechnical environments. They are now in use in a significant number of US mines. This includes the triple height telltale, used where multiple lengths of
reinforcement are installed concurrently. The Single Height Rotary Telltale is used widely in South African room and pillar coal mines where low levels of movement in high roadways must be identified and reacted to. The mechanical dual height telltale is also being used in significant numbers in Indian, Indonesian and Norwegian coal mines.

THE ELECTRONIC DUAL HEIGHT TELLTALE

Concept

Although the introduction of the dual height mechanical telltale represented a significant step forward in mining ground control safety practice, it is limited by the need for frequent visual reading and the associated difficulty in obtaining consistent and accurate readings especially in high roadways. In addition there is a requirement for remote monitoring of potentially unstable or inaccessible areas. Consequently Golder RMT has developed an intrinsically safe electronic extensometer and telltale system which includes an electronic version of the dual height telltale. The contactless electronic telltale combines high accuracy with low cost and local or remote reading options.

The design specification for the Golder RMT Remote Reading Telltale (RRTT) system contained a number of essential features, namely;

- low cost transponders and connectors, such that the overall cost of ownership remains acceptable,
- simple installation, preferably by the heading team, and as similar as possible to standard telttales,
- dual height transponders,
- retention of the visual reading characteristics of standard telltales, both by observation of the colour bands and reading of a millimetre scale,
- no batteries or separate power supplies for individual transponders,
- underground diagnostic features to allow easy identification of faulty transponders
- easy replacement procedures for faulty transponders and avoidance of the necessity to carry large stocks of spares
- automatic system recognition of additional transponders when they are connected
- automatic generation of warnings when action levels are exceeded.

The final system design achieved all the above requirements.

Measurement principle

The basic measurement principle used is that the inductance of a coil varies depending upon the position of a ferrite rod within that coil. The on-board electronics convert the inductance to a frequency which is transmitted down the line when the transponder is addressed. This has the advantage over potentiometric devices of being contact-less, requiring very low power and not being susceptible to moisture. The signal conditioning electronics are well suited to an analogue, frequency based interrogation system. Numerous forms of this electronic extensometer are available based on a single basic transponder design with a range up to 75 mm at 0.1 mm resolution. Transponders can be interrogated remotely or locally with a portable readout unit which also provides the power source. They can be read separately, or connected together by a single cable to a central monitoring point.

Telltale transponders

The telltale transponders (Figure 7) are available in single height or triple height versions as well as dual height. They are designed to fit into 35-45 mm diameter boreholes with a stainless steel wire attached to each borehole horizon using a simple spring anchor. These wires transfer the relative axial displacement of the rock to the measurement mechanisms located within the transponder body. They incorporate concentric visible indicators with 25 mm coloured bands and millimetre scales to allow
direct visual reading, as well as the electronic measurement system. In each transponder the two ferrite rods are directly connected to the A and B indicators respectively as shown in Figure 7.

Electronic telltale transponders are outwardly similar to Golder RMT’s visual telltales and are designed to be used in vertical holes only. The alternative wire extensometer transponders do not have the visual reading facility but have the advantage of a spring tensioning mechanism, allowing them to be deployed in non-vertical boreholes. These are available in one, two and four height models.

![Figure 7 - The electronic telltale transponder](image)

**Remote reading system**

The remote reading dual height telltale system allows up to 100 electronic telltales to be connected with a twin core cable and are read using either a portable readout, from the end of the roadway, or by a surface PC, via a telephone cable connection (Figure 8). A highly reliable frequency based system is used by the local communications unit to address and interrogate each transponder. Each transponder has a specific address represented by a tone. When that tone is placed on the line by the interrogation unit, the relevant transponder responds with two tones representing the A and B indicator readings.

The transponders can be installed using a simple crimp connection without the need for specialist staff. Underground, the only power supply required is a 12 volt supply to the interrogation unit. Individual transponders draw their low power requirements from the same line that carries the reading and addressing data signals. There can be up to 2 km of cable between the underground reading position and the most distant transponder. The remote PC can be up to 10 km from the underground monitoring unit. One PC can monitor up to 4 sets of transponders. In this configuration, a real time display of roof condition is obtained whilst the immediate visual indication underground is retained. The one, two and four height roof and rib extensometers and convergence meters can also be interfaced to the system.
Figure 8 - Remote reading telltale system

Instrument reading and data analysis

The main function of the RRTT system is to provide real time data on the state of all the connected telltales to allow remedial action to be taken before a roof fall develops. However it also acts as a data logger allowing historical data to be stored for subsequent analysis. This is achieved in a number of ways. The raw frequency data is stored in both text file and random access formats at the configured scan rate. This data can be accessed by the “Boltmon” software program running on the surface PC or read into an Excel spreadsheet for plotting and analysis. The Boltmon program obtains readings consecutively from the system transponders and, once configured, continues to run in the background with no need for further operator intervention. “Boltmon” also provides a graphical display of selected transponder readings in real time and other diagnostic tools.

The main user interface is provided by the “RRTelltales” program. This accesses the frequency data stored by “Boltmon”, converts readings to millimetres of movement and displays them on the screen in tabular form. The “RRTelltales” program also stores the recorded data in a random access file which can be accessed for subsequent uploading into a database by a purpose written program. An “alarm” level can be set for each of the monitored telltale indicators, for both absolute movement and rate of change. This is achieved through a set of global, default values set for the whole system which can be overridden with a local set of values for each individual telltale.

Intrinsic safety certification

The system received European ATEX Intrinsic Safety Approval in mid 2000 and has just received Australian IECEx approval for use in potentially explosive atmospheres.

Application examples

German coal mine roadway

The first full-scale application of the system, employing 67 telltales, was in a 1300 m longwall maingate access tunnel in a German coal mine in 2001. This was a rectangular section rockbolted roadway serving a retreating longwall face. The Remote Reading Telltale System was used in order to obtain detailed information on roof behaviour in the front abutment area and to help to understand the performance of the roof and support system behind the longwall. This particular configuration differed from a “classic” RRTT system installation in that:
The transponders were not installed at the face of the roadway during development, but in several batches after the roadway had been driven. However the existing monitoring systems (mechanical telltales) indicated that very little roof movement had occurred prior to installation.

The desire to maintain readings behind the face meant that great care would be needed to prevent damage to the transponders and cable in the face-end area.

An initial pilot installation with 10 transponders allowed any unforeseen problems to be identified and corrected prior to the full installation.

Following this, the full system was installed during June 2001 with a total of 67 transponders installed between the 123 and 1108 metre marks. The face commenced production on 25th June 2001. The face passed the last transponder on the system in October 2002. During the period of operation the scan rate was set at 20 minutes such that the system display was updated and data was recorded from all connected telltales 72 times a day.

An example of the main “RRTelltales” screen from this site is shown in Figure 9. The connected telltales are shown in the first column under “Telltale Details”. In this case the roadway name (1830) and the metre mark where each telltale is installed are shown. The right hand half of the screen shows the current condition of each telltale. This is shown in terms of A, B and Total movement in millimetres, rate of movement (ROM) for A and B, in millimetres per hour, and the alarm condition for A, B, Total and ROM. The background colour in the A and B reading columns corresponds with the visible coloured band on the transponder itself (< 25mm green, < 50mm yellow >25mm, red >50mm). The readings on this display are updated at the scanning rate set for both “Boltmon” and “RRTelltales”. The remaining columns on the left hand side of the screen show the roof displacements for each telltale for previous days during the current week. The operator can scroll back for up to one week to allow recent trends to be examined.

Maingate roof conditions ahead of the longwall were excellent throughout the face retreat. The face did not generally begin to affect the roof until it was less than 20 m away. Figure 10, which shows detail of telltale data from Station 52 at 862 m, indicates that movement did not begin in this case until the face was some 16 m away. This graph shows each 20 minute data point and it can be seen that this level of detail shows several interesting aspects of roof movement which could only be observed with a monitoring system of this type. In particular, the “B” telltale shows roof movement occurring in several
distinct steps of up to 4 mm. The “A” indicator also shows these steps but, except for the 4 mm step on 7th January, they are generally of a lesser degree.

This first trial demonstrated the potential of the new system in obtaining data on roof deformation processes as well as acting as a real-time warning system to prevent roof falls in strategic rockbolted roadways.

Relatively sudden steps in movement as the face approached were seen on many of the telltales in the roadway and similar patterns of movement have since been observed at other sites, confirming that they are a real phenomenon. An example from a current application site, where the telltales are scanned every 10 minutes, is shown in Figure 11. Here, a sudden immediate roof displacement of approximately 8 mm occurred, followed 70 minutes later by a second displacement of another 6 mm. No additional movement above the bolted height occurred.

UK Coal Mine – Monitoring of Sealed Access Road

A remote reading telltale system, comprising 13 dual-height telltales, was installed in July 2005 to monitor a faceline access road at 800 m depth in a UK mine. This roadway was to be temporarily sealed until required to prevent spontaneous combustion and the system was needed to remotely monitor the condition of the sealed section of rockbolted maingate.

The installed system operated reliably over the period when the roadway was inaccessible and the relatively low movements, confirming stable conditions, allowed the mine to leave the sealed off face unsalvaged until the face equipment was required.

The previously sealed lengths of gate road and face line were re-accessed during April 2006, allowing the telltale visual readings to be checked against those displayed on the surface PC. This confirmed that the system had maintained its accuracy to within a millimetre over the 9 month operational period during which the roadway had been sealed. The maximum movement in the rockbolted maingate had occurred 5 m outbye the face line junction. This was 46.5 mm on the A indicator (bolted height) and 1.1 mm on the B indicator (above the bolts). The face was successfully salvaged following re-access of the district and the installation of long tendon support in the face end junctions.
Figure 11 - Data from Current RRTT site showing sudden steps in roof deformation (10 minute scanning interval)

Figure 12 shows data for four of the telltales over the 9 months when the roadway was sealed, again illustrating short term increases in movement rate presumed to be caused by the failure of individual strata units.

Indonesian room and pillar coal mine

Remote reading telltales are currently being used to monitor partial pillar extraction operations in an experimental Indonesian room and pillar coal mine. In this case the portable readout is used to connect to the chain of transponders underground. The readings are then transferred to a spreadsheet for plotting.

The depth of working is 100m and the seam thickness 2.5 m. The mine is being developed as linked main roadways off which angled run outs are driven. Additional coal is obtained by mining out “pockets” from one side of the run outs. The stability of the run outs and laterals is being monitored as this process continues.

The support being installed consists of 2.1 m rockbolts with additional long tendons in junction areas, and the telltale anchors in the roadways are being installed to 1.8 m (A) and 8 m (B). Figure 13 shows the position of the instruments in the run outs and Figure 14 typical telltale results as mining proceeds. The Figure shows additional movement within the bolted height developing and then stabilising as pocketing of successive run outs takes place. No significant movement above the bolts has taken place.

US gas storage cavern

A remote reading dual height telltale system is currently being used to monitor roof stability during development of a gas storage cavern in the USA. Sixty four telltales have been installed at 15 m intervals in a room and pillar type layout. The system is currently live and can be monitored by the engineers in real time on the internet. Figure 15 shows a screen shot taken at Golder RMT of some of the telltale readings on 10th November 2009. The instrument readings confirm good stability with very low readings and none above action levels.
Figure 12 - Monitoring results from sealed access road in a UK coal mine

Figure 13 - Monitoring plan including remote reading telltales – experimental Indonesian coal mine
THE AUTO WARNING TELTTALE

The auto warning telltale is a development of the electronic telltale concept, still meeting intrinsic safety requirements, and providing an additional high visibility warning of roof movement via flashing light.
emitting diodes (LEDs). Intended applications include pillar extraction areas in room and pillar workings to give warning of impending goafing. This is based on the premise that goafing events are preceded by smaller scale roof dilation which will be detected by the telltale. It is used in preference over roof to floor convergence monitoring which is susceptible to triggering by floor heave. In order to incorporate this additional warning, the telltale design includes a 1.5 volt alkaline primary cell supply and LED configuration housed within the existing telltale’s plastic drip tray moulding (Figure 16).

The Autowarning electronic module powers the LEDs in a flashing sequence when a pre-set level of telltale movement is reached. A minimum of two LEDs are employed in the latest version with an option for four LEDs for well lit areas. The flasher module has an operational ‘flashing’ life of over a month.

The auto warning telltale is currently being used in the first fully mechanised pillar extraction bord and pillar mines in India. Telltales are being installed in each junction and roadway midpoint prior to extraction operations (Figure 17). The telltales used are single height types with the anchor position at 10 m into the roof and a trigger level of 5 mm. This combination of large monitored height and low trigger level is intended to ensure that the telltale warning is triggered prior to a major goafing event. The auto warning telltales are reported to be working as planned with the LEDs flashing as roof failure commences.

SUMMARY

The rockbolting telltale has been a major contributor to ground control safety since it was developed in its present form almost twenty years ago. The more recent development and proving of the electronic telltale, with its remote reading capability and improved accuracy, allows intensive monitoring of bolted roadway deformation to be undertaken automatically. The information gained, if properly assessed and used, can play a major role in ensuring safety, optimising support design and confirming the stability of strategic roadways. There is no doubt that the role of mining geotechnical instrumentation of this type will continue to grow, with the ultimate aim of eliminating all falls of ground.
REFERENCES


Bigby D, Lewis D, and Luttig F 2003 Application of RMT’s remote reading telltale system to monitor roof movement during face retreat at west colliery, Germany 22nd Int. Conf. On Ground Control in Mining, Morgantown WV
NON DESTRUCTIVE INTEGRITY TESTING OF ROCK REINFORCEMENT ELEMENTS IN AUSTRALIAN MINES

Wauter Hartman¹, Benoit Lecinq², John Higgs³ and David Tongue³

ABSTRACT: Non-destructive testing, used to study the integrity of the bolting systems in underground mining and civil construction industries, as an alternative method to the current hydraulic pull testing practice, is described. Non-destructive tests were carried out on a total of 227 bolts, comprising 89 rebar type bolts, 124 cable bolts and 14 split sets were tested in four mines across Australia. The purpose of these tests was to confirm the validity of the testing methodology for rock reinforcement systems used in mines and provide reassurance on bolt’s integrity, which could have been compromised during installation or affected by in-situ aggressive conditions causing corrosion. A complex stress wave analysis package, based on the processing of clear seismic signals imparted into the rock reinforcement element, was used. The seismic signals are processed by “Fourier Transform” into various criteria, which can be used to produce models of the elements, such as mechanical admittance, frequency spectra and velocity. These components are then used in the final modelling of the rock reinforcement element under analysis. The non-destructive integrity testing of rock reinforcement at these mines indicated that there is opportunity to further investigate the potential in effectively managing the risk of ground failure incidents in underground openings.

INTRODUCTION

The traditional pull out tests currently used for rock reinforcement testing is not considered an effective tool for the detection of compromised rockbolt systems used for ground control in underground mining and civil construction industries. It is acknowledged that pull tests have an important role to play in static and quasi static ground support designs in determining critical bond lengths through short anchorage testing. However anchorage capacity testing does not provide an underground operation with any reassurance regarding bolt integrity, which could have been compromised during installation or affected by in-situ aggressive conditions that cause corrosion.

Non-Destructive rock reinforcement integrity testing conducted at four Australian Mines (i.e. Western Australia, Queensland, New South Wales and Victoria) have shown that there is potential to optimise this testing method. A total of 199 bolts comprising 61 rebar type bolts, 124 cable bolts and 14 split sets have been tested to date. The non-destructive rock reinforcement integrity testing was conducted using a complex "Stress Wave Analysis" package based on the processing of clear seismic signals imparted into the rock reinforcement element that is being tested. The seismic signals are processed by “Fourier Transform” into various criteria which can be used to produce models of the element such as mechanical admittance, frequency spectra and velocity which are all being used in the final modelling of the rock reinforcement element under analysis. The non-destructive integrity testing of rock reinforcement at these mines indicated that there is opportunity to further investigate the potential in effectively managing the risk of fall of ground incidents at underground mine and construction sites.

BACKGROUND INFORMATION

In simple terms the modified shock test, which is described by Higgs and Tongue (1999), is a seismic test using a hammer blow as the force and a transducer to pick up the resultant vibrations. With the application of digital filtering techniques an accurate mechanical admittance vs. frequency plot is obtained which can then be interpreted using the concepts developed by Davis & Dunn (1974).

This non-destructive method by vibration has its origins from Davis and Dunn where they carried out various types of non-destructive pile tests on sites in Western Europe and other French speaking countries for “The Centre Experimental de Recherches et d'Etudes du Batiment et des Travaux Publics” (CEBTP) of France. This vibration method had also been used and described by Gardner and

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Moses in 1973, but British engineers had not exploited this technique to the extent that could have been useful to them because of a lack of knowledge and a degree of mysticism associated with the interpretation of the results.

Since vibration testing of piles was first started by the CEBTP, a considerable amount of theoretical work had been done which shed light on the interpretation. The experience of testing many thousands of piles led to the technique being applied with more confidence to testing piles on sites as a norm. The main function of the test was to detect any major defect, such as an open fracture or an important strangulation of the concrete, particularly in the upper portion of the pile (Davis and Dunn, 1974). The vibration method used for pile testing has been slightly modified for rock reinforcement testing. Thus instead of using a vibrator (i.e. pulse generator to excite the pile), velocity transducer and accelerometer the Modshock system only incorporates a tapping device in order to excite the bolt, low frequency geophones (i.e. horizontal, vertical and upside down transducers) and an analogue / digital converter which converts the signal from the transducer into a digital format.

The analysis for rock reinforcement is similar and is based on measuring the frequency and amplitude response of a rock reinforcement element known as impulse. This response, known as Mechanical Admittance (or mobility), contains all the information necessary to check rock reinforcement integrity and to analyse the surrounding influences (i.e. ground deformation). At higher frequencies the resonating harmonics of the rock reinforcement element are detected, whereas at low frequency the response is generally linear allowing measurement of the element-head stiffness.

The non-destructive rock reinforcement integrity testing analysis is conducted using a complex “Stress Wave Analysis” package based on the processing of clear seismic signals imparted into the rock reinforcement element that is being tested. The seismic signals are processed by “Fourier Transform” into various criteria which can be used to produce models of the element such as mechanical admittance, frequency spectra and velocity which are all being used in the final modelling of the rock reinforcement element under analysis.

In research and laboratory applications of modal analysis, particularly of complex machinery, dynamic excitation was often provided by a linear hydraulic or eccentric mass shaker. Experience gained in testing over 140 bridges indicated that simpler means of excitation are suitable for 90% of all bridges where attaching shakers to bridges were seen as a complex and costly method and is only practical for research purposes or for extremely complex structures (Higgs and Tongue, 1999). Similarly the application for rock reinforcement integrity testing it was found that a simpler method to excite bolts is adequate for the detection of defects.

The development of the Australian based testing method started in the late 80’s and has been used for the correct assessment on a large variety of elements, which now exceeds well over 1 000 000 tests for more than 20 yrs (Higgs, 1975). Integrity Testing Pty Ltd (i.e. developers of the Modshock system) has for over 15 yrs carried out testing of long length steel rods, either as strand or solid steel bars. The most notable project was for BHP, who owned the Whyalla steel works, where they tested the tie rods holding back the crucial steel pile wall of the coal handling jetty.

The rods were tested and not only were the defective rods identified but it was indicated at what point the rods had lost a large cross section. This was located at a point where the rods came close to the base of the coal handling pit and water was seeping onto the rods causing corrosion. Thus a large successful background in the testing of steel embedded elements is generally with the lengths in excess of 5 m.

**TESTING SET-UP AND METHOD**

There are four components to the system (Figure 1-4):

1. **A Toughbook / Notebook** - this is used to collect data and providing power via a USB cable for the (see Figure 1).
2. **Analogue/Digital Converter encased in closed unit** - this converts the signal from the transducer into a digital format. The converter is soft wired to the transducer (see Figure 2).

3. **Transducer** – *which is* held at the end of bolt (i.e. collar of hole – see Figure 3) during the test. A signal / pulse is obtained, which is generated by a small hammer.
Figure 3 - Photo showing transducer (vertical) held at the end of bolt (i.e. collar of hole) at Manadalong Coal Mine

4. A small hammer or tapping device (see Figure 4) - This has to make contact against the plate or nut of the bolt during the test.

Figure 4 - Photo showing tapping device that is used to make contact with the nut of resin bolt

TEST OUTPUT

A valid seismic signal (see Figure 5) is obtained through the Modshock system and is one of the main criteria by which a test is accepted or rejected. The graph displays velocity on the vertical axis and time on the horizontal axis. The blue line represents the seismic signal; where as the pink line represents the commencement of element analysis.
Table 5 - Diagram showing valid seismic signal obtained by transducer (geophone)

The operator in the field can at the time of testing identify which element (bolt) are serviceable and then concentrate on the rock reinforcement elements that have shown anomalies. For the elements with anomalies a two-dimensional graph of the test can be obtained and presented after the analysis has been carried out (see Figure 6).

Table 6 - Two dimensional graph showing structural stiffness of rock reinforcement element with diameter used as a guide

The two opposing curved black lines on the graph represent structural stiffness through good embedment or load transfer. The top (blue) and bottom (green) horizontal lines in the graph collectively represent the element’s full diameter. The structural stiffness presented in the two dimensional plot together with the element’s diameter are used to indicate whether any disturbance (i.e. bolt necking, bolt volume reduction through corrosion, bolt shearing (Hartman, 2003) and/or ineffective grout or resin embedment) or reflection point can be detected during testing. The graph is an example of disturbance (reflection) where grouting was deliberately placed towards the toe end of the bolt as part of a calibration program at the Sunrise Dam Mine (WA). The graph clearly shows good load transfer or embedment towards the toe end of the bolt.
One of the vital pieces of information obtained from the non-destructive test is the “Head Stiffness” as this is the basis of all the load predictions and it also indicates the serviceability of the total bolt system. The head stiffness is the “E” prime of the bolt, measured as a direct measurement of the first part of the “structural stiffness plot”, and is similar to a load/displacement graph for a pull out test.

The “bolt head stiffness (tonnes/mm)” is compared to the two model stiffness values “E” min and “E” max. “E” min is a bolt model with the bolt pinned at its toe (end anchored) but with no clamping (no resin or grouting) along its length. “E” max is a bolt model with an infinite rigid base and “clamped” (full column grouted / resin) along its length. These models are based on the work carried out by Davis & Dunn (1974).

The “Stiffness” value of the bolt is a good indicator of the serviceability of the bolt, but cannot be used in its entirety to give a serviceability rating for the bolt, as a number of factors come into affect when measuring the stiffness. The measurement of the stiffness can be affected by the fixity of the end of the bolt, the bonding effect of the resin/grout around the bolt and the bond from the rock to the resin/grout to ensure a fully encapsulated scenario of the bolt.

**RECENT FINDINGS**

**Detection of rock reinforcement element length**

In a recent test at the Fosterville Gold Mine cable bolt lengths were accurately depicted following confirmation from the mine. The test set-up incorporated 10 m and 8 m lengths as input parameters for the cable bolt testing. Most of the bolts were confirmed to be either 8 m or 6 m in length (see below Figure 7 for final interpretation of two dimensional structural stiffness plot).

![Graph showing confirmation of cable bolt length](image)

**Table 7 - Two dimensional graph showing confirmation of twin stand cable bolt 6m length following initial 8m input parameter**

The above graph clearly shows bolt length to be around 6 m following test input parameter set to 8 m. The bolt length was later confirmed by the mine to be a 6 m twin strand cable bolt.

**Detection of poor grout / resin installation and shorter anchor**

Both Figure 6 and 7 above are prime examples of poor grouting. Figure 6 is an example of a calibration bolt with known grouting embedment at the toe and collar. Figure 7 is a two-dimensional graph of a “full column grouted” twin strand cable bolt tested at the Fosterville Gold Mine showing clear signs of a defect (i.e. grout deficiency) between 3 m and 4 m. In addition, this graph also shows that the cable bolt length to be 6 m instead of 8 m.
Confirmation of good quality resin installation on solid rebars and Hi-Tens end anchored cable bolts

Test completed at Mandalong Coal Mine shows that the resin installation practices at Mandalong appear to be of good quality when interpreting the two dimensional graphs (see Figure 8 below) and calculated load. Figure 8 indicates good embedment (structural stiffness) along the length of the resin bolt.

![Graph showing good embedment](image)

**Table 8 - Two dimensional graph showing good embedment achieved along the length of the 2.1 m resin anchor installation**

The Hi-Tens cable bolts are installed using a 1 200 mm long slow set resin which is inserted into the hole using a conduit to push it right to the back of the hole. The Hi-Tens tendon is inserted into the hole until it reaches the base of the resin. The bolt is spun through the resin for about 30 seconds to activate the quick set resin. The plate, barrel and wedge are installed with a tensioner to 20 tonnes. Tests conducted on the Hi-Tens tendon showed an interesting two dimensional graph where the resin installation is limited to the toe area of the bolt / hole as per design. A free anchor length of around 4.2m is maintained with the 2D plot showing either signs of stress increase or noise close to the collar. We have reason to believe that it could be related to an increase in stiffness / strain in the bolt as this was previously noticed when testing cable bolts at the Sunrise Dam Gold Mine and other mines. This phenomena relates to the first part of the two dimensional graph (see below Figure 9 – Hi Tension tendon – Test No. 9 – Mandalong Coal).

In 2004 Martin et al. showed that a critical load is required before the cable bolt, at a given location, would sense any load. This was done through instrumented cable bolts loaded at the collar and plotted against recorded microstrain at individual gauge locations (see below in Figure 10 the load profile along the length of the cable at different collar loads). This implied that a gauge positioned 25.4 cm from the collar would sense load only when the collar load exceeds 25.4 × 2 043 N/cm.

Figure 10 shows some similarity to the load vs dissipation rate graph of a collaborative investigation, conducted in 2004, into the behaviour of cable bolts. The collaborative work was carried out at the University of Saskatchewan, Saskatoon; Itasca Consulting Group, Inc., Minneapolis; University of British Columbia, Vancouver (BC) and the National Institute for Occupational Safety and Health (NIOSH). The study provided valuable information regarding their loading and strain behaviour. The above phenomenon, however, would need to be confirmed with strain gauged cable bolts. The cable bolts could then potentially be subjected to various loads and simultaneously tested using the Modshock system to compare actual loads with analysed elastic loads for correlation.


Table 9 - Two dimensional graph showing confirmation of end anchor resin embedment and possible stress / strain increase in close proximity of hole collar

Table 10 - Collar load plotted against A, microstrain (load profile curve) and B, distance from head of cable (load correlation curve, Martin et al.(2004))

TESTS RESULTS

Non-destructive tests were carried out on a total of 227 bolts, comprising 89 rebar type bolts, 124 cable bolts and 14 splitsets were tested in four mines across Australia (See Table 1 below outlining different types of bolts tested).
Table 1 - Different type of bolts tested

<table>
<thead>
<tr>
<th>Stiff Splitsets</th>
<th>Splitsets or Friction Bolts</th>
<th>Resin Solid Rebar</th>
<th>Single Strand Cable Bolt</th>
<th>Multi Strand Cable Bolt</th>
<th>Hi Tensile Cable Bolt</th>
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<td></td>
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</table>

Of the 227 bolts tested 36 were calibration bolts comprising of resin bolts, twin and single strand cable bolts and split sets. 191 Bolts were tested for defects related to insufficient grout / resin which affect the anchorage or anywhere along the length of the bolt; bolts affected by corrosion displaying significant volume loss and reduced load transfer and/or bolts displaying low calculated stiffness indicating low load transfer or poor encapsulation. These defects or significant issues were presented through a simplified bolt serviceability classification system (see below Table 2).

Table 2 - Simplified bolt serviceability classification system

<table>
<thead>
<tr>
<th>Category 1.</th>
<th>A perfect bolt in perfect rock conditions – in our opinion this will rarely occur</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category 2.</td>
<td>A bolt which we consider is serviceable in that it has good anchorage, good embedment / load transfer along the length of the bolt and reasonable rock/grout/resin contact. Conform to design criteria e.g. end anchored resin bolts.</td>
</tr>
<tr>
<td>Category 3.</td>
<td>A bolt that has some deficiencies in reduced anchor strength, poor grout/resin/rock contact or loss of bolt section. The remarks section will identify the possible source of the deficiency.</td>
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<tr>
<td>Category 4.</td>
<td>A bolt that has either failed; is loose or at a point where additional load on the bolt could lead to failure; or a loss of bolt section which is critical e.g. anchorage area where the 400mm critical bond length has been affected</td>
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Table 3 - below outlines the bolt classification results for the 191 bolts tested

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<th>Poor Signal</th>
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<th>3</th>
<th>4</th>
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</tbody>
</table>

70% of the 191 bolts tested were classified as serviceable but with 49% of the bolts tested, some kind of defect (i.e. suspect anchorage, low load transfer and/or volume reduction) have been detected. Almost 30% of the 191 bolts tested were classified as non-serviceable with the majority of the bolts showing a deficiency in end anchorage as per mine design and/or overstressed bolts due to excessive ground deformation. 2% Of the 191 bolts tested have been classified as inconclusive. This relates to
bolts being identified as short bolts or very poor anchorage but completely out of character for the type of bolt.

CONCLUSIONS

Ground support quality control has been a high priority for most mines but remained a high risk due to the uncertainty in the current bolt integrity testing procedure of pull testing. The use of non-destructive technology to test for defects or poor quality installation techniques are showing an enormous opportunity in effectively manage this geotechnical risk. It has been found that verification of rock reinforcement designs (i.e. bolt lengths and full column resin/grout installations) and integrity confirmation are two of the biggest challenges for geotechnical engineers and mine management. We are confident that this non-destructive integrity testing technique is a step towards reducing the uncertainty in quality control of rock reinforcement installations.

It is acknowledged that in order to increase confidence in other data interpretation the following are required:

- Calibration testing to confirm the elastic load increase in tendons and solid rebars as referred to in this paper and
- Confirmation of two dimensional graph amplitude variance and descriptive analysis.

ACKNOWLEDGEMENTS

The authors would like to thank the following mines and their management for allowing us to publish some of the information and results i) Sunrise Dam Mine (Ashanti Gold, WA), Fosterville Gold Mine (Vic) and Mandalong (Centennial Coal, NSW). A special thanks to Mr. Lammie Nienaber (Senior Geotechnical Engineer – Sunrise Dam), Mr. Luke Kroise (Senior Rock Mechanics Engineer – Fosterville Gold) and Mr. Elliot Tembo (Geotechnical Engineer – Mandalong) for their arrangements and input.

REFERENCES


Shear Testing of 28 mm Hollow Strand “TG” Cable Bolt

Peter Craig¹ and Naj Aziz²

Abstract: Cable bolts were introduced to the coal mining industry in the early 1980’s mainly for roadway reinforcement as a secondary means of support. Their application is dictated by the nature of the stratification, ground stress conditions and the size of the opening. Double shearing tests were carried out on the 28 mm hollow Strand Jennmar “TG” cable bolt, two tests were conducted to evaluate the shearing characteristics of the bolt and to gain a better understanding of the shearing behaviour of the cable. The first test was limited by a 50 mm travel on the testing machine and produced a shear load of 900 kN (92 t) at the maximum 50 mm displacement, with axial load generated on the cable bolt reaching 238 kN (24.3 t). In the second test the machine travel was increased to 75 mm, cable failure due to shear loading was achieved at 1 354 kN (138 t) load and a vertical displacement of 59 mm, with cable axial load in the order of 385 kN (39.3 t). Analysis of the failure mode and loads achieved indicate that the cable strands bent and the concrete crushed along the shear plane, the shear loading across the concrete and grouted cable then reached the tensile strength of the steel wires.

Introduction

Cable bolts were introduced to the mining industry around 1970, initially to surface mining and underground metalliferous mining and later on to coal mining in the early eighties mainly for roadway reinforcement as a secondary means of support of the last century. Cable bolts have since been used as both primary and secondary supports. As primary support, Fuller et al (1994) described the application of cable bolts, known as FLEXIBOLT, for strata reinforcement in both Angus Place and Ellalong Collieries, in NSW. As secondary support, cable bolts have also been used as cable trusses which act to support the immediate roof in a sling like manner, Fabjanczyk and Tarrant (1988), Fuller, et al (1991), O’Grady and Fuller (1992) and for reinforcement at higher stratification and beyond the rebar bolt length, mainly for anchoring lower strata layers immediately above the coal seam to the higher and competent bedding formation above. Initially cable bolt anchorage was by cementatious grouting and since 1990’s by chemical resin. The dominant type of cable bolts used for secondary support in Australian underground coal mines are 588 kN (60 t) capacity cables which are point anchored, pre-tensioned and post grouted. The installation of these typically 8 m long cable bolts involves a lot of manual handling of the cables, lifting of heavy hydraulic tensioners, along with exposure to cementatious grouts. Table 1 shows the specification of various cable bolts currently marketed and installed in Australian underground coal mines.

Because roof deformation loads cable bolts both axially and in shear, it is necessary to test cables for both tensile and shear strength. Axial loading is tested by pull testing in the laboratory (Goris, et al, 1996) and in the field, while shear testing is only possible in the laboratory. Axial pull testing of the Jennmar TG cable produced a tensile strength of 618 kN (63 t) and this test was completed as part of the product development needs.

While attention to the strength of the cable bolt is generally focused on tensile strength, very little attention is paid to the cable strength in shear. Ironically failure in shear represents one of the most damaging aspects of the cables’ credibility particularly in longwall gateroads and when lateral deformation of the immediate stratification is at its more severe. Accordingly, and in an effort to study the effect of shear forces on cable bolt supports, this study deals with double shear testing of the cable bolt.

The TG bolt was developed in 2007 as a 618 kN (63 t) post groutable bolt. The hollow strand cable bolt is a 28 mm diameter, nine wire strand cable. Each element is 7 mm in diameter, which surrounds a 14 mm hollow steel core tube. Figure 1 shows a cross sectional and side view of the cable. The TG cable is considered to have the following advantages over other similar cable bolt products as marketed in Australia:

¹ Jennmar Australia
² University of Wollongong
• It has a central grout tube to achieve small grout annulus and therefore higher bond strength
• Its flexible but reinforced central tube which allows a barrel and wedge to be used on the strand
• Its steel strand chemical composition resistant to stress corrosion cracking
• It has central grout tube resistant to corrosion
• It has simple bayonet grout fitting for push and ¼ turn attachment.

**Table 1 - Post grouted cable bolts used in Australian coal mines**

<table>
<thead>
<tr>
<th>Cable Type</th>
<th>UTS Strand (t)</th>
<th>Drill Hole Diameter (mm)</th>
<th>Strand Diameter (mm)</th>
<th>Bulb Diameter (mm)</th>
<th>Bulbing in Grouted Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>TG Bolt</td>
<td>60</td>
<td>38 - 42</td>
<td>28</td>
<td>35</td>
<td>No</td>
</tr>
<tr>
<td>Bowen Cable</td>
<td>60</td>
<td>42</td>
<td>21.8</td>
<td>38 / 33</td>
<td>Yes</td>
</tr>
<tr>
<td>Bulbed SuperStrand</td>
<td>B338</td>
<td>42</td>
<td>21.8</td>
<td>38</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>B348</td>
<td>52 - 55</td>
<td>21.8</td>
<td>48</td>
<td>No</td>
</tr>
<tr>
<td>Megabolt / Megastrand</td>
<td>MB9D</td>
<td>60</td>
<td>42 - 45</td>
<td>31</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>MB8D</td>
<td>54</td>
<td>42 - 45</td>
<td>27</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>MB9B</td>
<td>60</td>
<td>42 - 45</td>
<td>31</td>
<td>No</td>
</tr>
<tr>
<td>Post Groutable Hi-Ten</td>
<td>60</td>
<td>45 - 48</td>
<td>21.8</td>
<td>n/a</td>
<td>No</td>
</tr>
<tr>
<td>Mambo</td>
<td>60</td>
<td>42 - 45</td>
<td>21.8</td>
<td>34</td>
<td>Yes</td>
</tr>
<tr>
<td>15.2mm Twin-strand</td>
<td>54</td>
<td>52 - 55</td>
<td>15.2</td>
<td>26</td>
<td>Yes</td>
</tr>
</tbody>
</table>

**Double Shear Testing of TG Bolt**

Shear testing of the 28 mm diameter hollow strand "TG" cable was performed for the unbulbed grouted section at Wollongong University. The tests were carried out in a newly constructed large double shearing apparatus containing a 50 MPa concrete mould. Each cable was installed in newly cast concrete mould using "hi-thix" cable bolt grout supplied by Jennmar Australia. A 500 t capacity servo controlled compression testing machine was used for the test. The aim of the study was to determine the shearing performance of the cable bolt under different lateral loading conditions, and to assess the failure characteristics of the cable bolts. Two cable bolts, each 2 m in length were tested. One cable bolt was pre-tensioned to an initial load of 50 kN (4.9 t), and the other to 90 kN (8.8 t).
Concrete Block Casting

Concrete blocks were cast for each double shearing test. Once mixed the concrete was poured into the greased marine plywood mould, measuring 1 050 mm x 300 mm x 300 mm. The mould was divided into three compartments separated by two metal plates. A plastic conduit 24 mm in diameter was set through the centre of the mould lengthways to create a hole for cable installation. The cast concrete blocks were left for the first 24 hours to set and harden in the mould. The blocks were then removed from the moulds and kept in a moist environment for a period of 30 days to cure. The central hole of the concrete block was then rifle shaped reamed to 42 mm diameter, ready for the installation of the cable with cement grout. The UCS value of the concrete was 50 MPa, determined from testing of the representative 100 mm diameter cylindrical concrete specimens cast at the time of concrete preparation and pouring.

Cable Installation

The installation and encapsulation of the cable in the concrete block was carried out using Conbextra CB “hi-thix” cable bolt grout. This grout was high strength thixotropic grout (PC-201095). The following procedure was used in the grouting of the cable in the concrete blocks:

i. Four 20 mm vertical holes were drilled from the top side of each of the concrete block moulds to reach the 42 mm cable installation hole as shown in Figure 2. The central two grout holes (A) were located on the central 450 mm long block and were used to pour the grout into the 42 mm hole. The other two side holes (B and C) on the side blocks were to act as bleeder holes during the grouting stage.

ii. The cable was inserted in the 42 mm hole and pretensioned to the desired load. Pretensioning was made possible by using special cable grips which were anchored at the ends of the cable bolt. These grips were supplied by Jennmar Australia. 60 t capacity load cells were used to monitor the axial load developed during the initial cable bolt pretensioning and later during the shearing stage. Prior to grouting the annulus space between the cable and the 42 mm holes at the either side of the concrete block were blocked with tight wrapping with sealant tape to stop the grout from seeping out.

iii. Grout was poured to fill the space between the axially drilled hole and the cable. The block was mechanically vibrated to remove any air trapped and hence reduce cavity formation.

iv. The concrete/grout/ cable was left to cure for a minimum period of not less than seven days prior to testing. In any case the time of testing was dependent on the availability of the testing machine and other facilities.

Testing

Testing with single load cell and maximum 50 mm vertical displacement

Figure 3 shows the general test setup with one 60 t capacity load cell which was used to monitor the initial axial pretension load of 50 kN (49 t) on the cable bolt and subsequent load build up due to cable shearing. The vertical shearing of the central block was carried out at the rate of 1 mm per minute.
Both the vertical load and vertical displacement were monitored automatically together and stored in data-loggers for further processing.

Figure 4 shows the overall results of the first test in which the total vertical displacement was limited to 50 mm. The graph contains the combined processed data of the applied shear load (blue graph) and the force developed along the axis of the hollow strand cable (green graph) during the shearing process.

The total vertical load applied was in the order of 900 kN (88 t). This load was only possible for the maximum allowable displacement of 50 mm (Figures 4 and 5). Points A on the shear load /vertical displacement is attributed to the possible initial deformation of the central cable’s hollow core tube as well as grout and concrete crushing at the sheared zones as shown in Figure 6. Point B indicates the effect of further and sudden deformation of the concrete and grouts as well as cable strands. The extent of concrete crushing, moving inwards towards the centre of the middle block was around 60 mm as shown in Figure 6. The maximum axial load attained by the cable was in the order of 238 kN (24.3 t) at D, corresponding to the total displacement of 50 mm (Figures 4, and 5).

The second test was carried out with two load cells monitoring the axial load generated on the cable. One cell was mounted on each side of the cable as shown in Figure 3. Initially the bolt was subjected to an axial load of 100 kN (10.1 t), during encapsulation period of the cable in concrete blocks. The
system was then left for three weeks to cure. During this period the initial pretension load was dropped to 90 kN (9.2 t). The total vertical shearing displacement was increased to a maximum of 75 mm, which was the maximum possible stroke travel of the compression testing machine. The testing condition was maintained similar to the first test with regard to load application and displacement monitoring frequencies.

Figure 5 - Vertical shear displacement and axial load on cable bolt

As seen in Figure 7, the strand elements of the hollow strand cable began to fail when the vertical shearing load exceeded 1354 kN (138 t) as depicted by Blue graph (graph A). This failure occurred when the bolt was sheared some 60 mm vertically. Points 1, 2, 3 are the points of the cable element strand failures. Strand elements 2 and 3 occurred at the same time and these two failures may have occurred simultaneously on either side of sheared central block. Table 2 shows strands failure load and vertical displacements.

The axial load developed on the cable bolt due to shearing was observed by two axial load cell readings B (green) and C (Brown) respectively. The maximum axial load developed in the second test was in the order of 385 kN. This occurred at a vertical displacement of 60 mm.

Figure 6 - Concrete / grout crushing and cable bolt deformation in the vicinity
Figure 7 - Double shear loading, vertical displacement and axial load generation on the cable bolt using two load cells with maximum vertical displacement range of 75 mm

The failure loads shown in column two in Table 2 are the maximum failure loads recorded for the first cable bolt element (strand) failure, during shearing of the cable at two shear planes, i.e., double shear. Thus the failure loads per side are shown in column three of Table 2. It must be noted that the failure loads shown are to overcome both the cable element strength as well as the shearing of the surrounding concrete shear planes, which are subjected to gradually increasing pretension load.

Table 2- Cable elements failure loads and displacment

<table>
<thead>
<tr>
<th>Element</th>
<th>Failure Load- double plane shear (kN)</th>
<th>Failure load per shearing side (kN)</th>
<th>Vertical displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1 354</td>
<td>677</td>
<td>59</td>
</tr>
<tr>
<td>2</td>
<td>1 353</td>
<td>676.5</td>
<td>66</td>
</tr>
<tr>
<td>3</td>
<td>1 163</td>
<td>581.5</td>
<td>66.3</td>
</tr>
</tbody>
</table>

Next, one 400 mm long cable strand and 485 length of the central hollow tube were tested for ultimate tensile failure. The failure load of the 7mm strand was 72 kN (7.3 t), and that of the hollow central tube was 50 kN (5.1 t).

Given that the tensile strength of the cable is 618 kN (63 t), and considering that the hollow central steel is of concertina shape and of ductile material, then the central core is unlikely to bear significant proportion of the applied shearing load as would the strands, which are designed to bear.

As seen from Figures 8-11, it is clearly obvious that the cable strands failures occurred in tension and not in shear. All strand failures were of cone and cup failure and necking which are a characteristic of the steel failure in tension (Figures 8-9). This is expected as the concrete was, in general, more deformable and softer than steel. This was also evident from the heavily crushed zone in the vicinity of the shear planes (Figure 6).

If the cables were realistically sheared, then the shearing load of the cable would be around 2/3 or 70% of the tensile load, based on the past tests of the ordinary 24 mm diameter steel bolts using the conventional guillotine shearing test, and is a common knowledge on steel strength properties.
Applying this scenario to the cable strand, the failure load of the cable strand would be in the order of 44kN (4.4 t) instead of 72 kN (7.3 t). It must be mentioned that only the central core tube will be likely to fail in shear as it was flattened at the time of vertical loading/shearing. Thus it can be concluded that the maximum load per strand can be between 44 (4.5 t) and 72 kN (7.3 t) depending on the nature of the failure.
CONCLUSIONS

The first test was limited by a 50 mm travel on the testing machine and produced a shear load of 900 kN at the maximum 50mm displacement, with axial load generated on the cable bolt reaching 238 kN (24.3 t).

In the second test the machine travel was increased to 75 mm, cable failure due to shear loading was achieved at 1 354 kN (138 t) load and vertical displacement of 59 mm, with cable axial load in the order of 385 kN (39.4 t). Analysis of the failure mode and loads achieved indicate that the cable strands bent and the concrete crushed along the shear plane, the shear loading across the concrete and grouted cable then reached the tensile strength of the steel wires.
ACKNOWLEDGEMENT

The authors wish to acknowledge the technical laboratory support provided by both Alan Grant and Frank Crabtree of the School of Civil, Mining and Environmental Engineering, University of Wollongong, during preparation and testing of the cable bolts.

REFERENCES


MODIFICATION OF FOUR-SECTION CUT MODEL FOR DRIFT BLAST DESIGN IN RAZI COAL MINE - NORTH IRAN

Mohammad Hossaini and Hadi Poursaeed

ABSTRACT: Four-section cut, a model similar to Swedish method, is an empirical method for blasting design in underground excavations. This method, often, has been used in excavating tunnels with cross section area of more than 10 m². Using the model for smaller tunnels needs some modifications to achieve proper quantity of the parameters. In this paper, four-section cut method has been modified for designing patterns for tunnels with cross section area of less than 10 m². The applicability of the modified version has been examined through several blasting cycles and the ultimate optimized blasting pattern has been obtained. The previous blasting pattern of Razi coal mine, near Ramian city in Golestan Province, has been replaced by the new pattern which was proved to be much more efficient.

BLASTING PATTERN ALREADY PRACTICED IN RAZI COAL MINE’S DRIFT

The cross section area of Razi coal mine’s drift is 9.2 m². Rock properties at advance face of the tunnel are as in Table 1. Table 2 introduces the specifications of dynamite types used in this excavation. Table 3 points on blasting properties of the dynamite types. Blasting pattern used in the drift is illustrated in Figure 1. Length of stemming in this pattern is about 0.15-0.20 m which seems inadequate and has led to long bootlegs and violent air vibration. Therefore, to avoid these outcomes the length of stemming had to be increased to more than 0.2 m. Result of conducting the practiced pattern is shown in Table 4.

Table 1 - Rock properties of drift’s advance face in Razi coal mine.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear strength (MPa)</td>
<td>13</td>
</tr>
<tr>
<td>Velocity of blasting wave (m/sec)</td>
<td>4000</td>
</tr>
<tr>
<td>Specific weight</td>
<td>2.7</td>
</tr>
<tr>
<td>Specific energy (Mj/m²)</td>
<td>1.55×10³</td>
</tr>
<tr>
<td>Impedance (kg/m².sec)</td>
<td>10760×10⁴</td>
</tr>
<tr>
<td>Uniaxial compress strength (MPa)</td>
<td>77.5</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>5</td>
</tr>
<tr>
<td>Relative weight strength with respect to ANFO (%)</td>
<td>121</td>
</tr>
<tr>
<td>Relative bulk strength with respect to ANFO (%)</td>
<td>181</td>
</tr>
<tr>
<td>Specific drilling (m/m³)</td>
<td>4.75</td>
</tr>
<tr>
<td>Specific charge (kg/m³)</td>
<td>1.51</td>
</tr>
<tr>
<td>Advance Efficiency (%)</td>
<td>80</td>
</tr>
<tr>
<td>Length of bootleg (m)</td>
<td>0.10 - 0.20</td>
</tr>
</tbody>
</table>

Table 2 - Specifications of dynamite cartridges

<table>
<thead>
<tr>
<th>Type of dynamite</th>
<th>Diameter (mm)</th>
<th>Length (m)</th>
<th>Linear charge concentration</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>25</td>
<td>0.20</td>
<td>0.625</td>
</tr>
<tr>
<td>b</td>
<td>23</td>
<td>0.23</td>
<td>0.543</td>
</tr>
<tr>
<td>c</td>
<td>22</td>
<td>0.27</td>
<td>0.46</td>
</tr>
</tbody>
</table>

Table 3 - Blasting properties of the dynamite

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific weight</td>
<td>1.215</td>
</tr>
<tr>
<td>Velocity of detonation (m/s)</td>
<td>5000</td>
</tr>
<tr>
<td>Relative weight strength with respect to ANFO (%)</td>
<td>121</td>
</tr>
<tr>
<td>Relative bulk strength with respect to ANFO (%)</td>
<td>181</td>
</tr>
</tbody>
</table>

Table 4 - Results of the practiced blasting pattern

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD: Specific drilling (m/m³)</td>
<td>4.75</td>
</tr>
<tr>
<td>SC: Specific charge (kg/m³)</td>
<td>1.51</td>
</tr>
<tr>
<td>AE: Advance Efficiency (%)</td>
<td>80</td>
</tr>
<tr>
<td>Length of bootleg (m)</td>
<td>0.10 - 0.20</td>
</tr>
</tbody>
</table>

1 School of Mining Engineering, University college of Engineering, University of Tehran, Iran
Figure 1 - Blasting pattern used in Razi coal mine drift.

BLASTING PATTERN BASED ON 4-SECTION CUT

The four-section cut which is close to Swedish model is based on the parallel hole cut. This model started with Langefors and Kihlstrom (1963) and has been further developed afterwards. Holmberg published the complete blast design model in 1982 (Holmberg, 1982) and was later updated by Persson et al (2001). The method suggests the experimental equations listed in Table 5. In this Table, E and X are drilling error and length of each quadrangle sides respectively. Due to relative easiness and precision in drilling of direct cutting holes E is taken as zero. The value of E for stopping and perimeter holes is calculated by Equation 1 (Konya, 1995):

\[ E = \alpha H + \beta \]  
\[ E = \alpha H + \beta = 0.03 \times 1 + 0.01 = 0.04 \text{ m} \]  
Where; \( E \) = drilling error, \( H \) = blast hole depth (equal to 1m), \( \alpha \) = angular deviation (equal to 0.03 m/m) and \( \beta \) = collaring error (0.01 m).

In this model, the holes in the face are divided into separate sections as cutting holes, stopping holes, perimeter (roof, floor and wall) holes.

Four-section cut method includes an empty hole in the centre as shown in Figure 2. If the number of empty holes is more than one, equivalent diameter is calculated by Equation 3 (Konya, 1995):

\[ \phi_{e2} = \sqrt{N} \phi_{e} \]  
Where; \( \phi_{e} \) = empty hole diameter and \( \phi_{e2} \) = Equivalent diameter of empty holes.
### Table 5 - Equations for blasting pattern design

<table>
<thead>
<tr>
<th>Sections</th>
<th>Burden (B)</th>
<th>Spacing (S,X)</th>
<th>Stemming (St)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First square cut</td>
<td>$B_1 = 1.5\phi_{e2}$</td>
<td>$X_1 = \sqrt{2}B_1$</td>
<td>$St_1 = B_1$</td>
</tr>
<tr>
<td>Second square</td>
<td>$B_2 = \sqrt{2} \times B_1$</td>
<td>$X_2 = \sqrt{2}B_2 \times 1.5$</td>
<td>$St_2 = \frac{\sqrt{2}}{2} B_1$</td>
</tr>
<tr>
<td>Third square</td>
<td>$B_3 = \sqrt{2}B_2 \times 1.5$</td>
<td>$X_3 = \sqrt{2}B_3 \times 1.5$</td>
<td>$St_3 = \frac{\sqrt{2}}{2} (\frac{\sqrt{2}}{2} B_1 + B_2)$</td>
</tr>
<tr>
<td>Fourth square</td>
<td>$B_4 = \sqrt{2}B_3 \times 1.5$</td>
<td>$X_4 = \sqrt{2}B_4 \times 1.5$</td>
<td>$St_4 = \frac{\sqrt{2}}{2} \left( \frac{\sqrt{2}}{2} B_1 + B_2 \right)$</td>
</tr>
<tr>
<td>Stopping</td>
<td>$B = 0.012 \left( \frac{2\rho_C}{\rho_r} + 1.5 \right) \times \phi_r - E$</td>
<td>$S = 1.1B$</td>
<td>$St_S = 0.5B$</td>
</tr>
<tr>
<td>Roof</td>
<td>$B = 0.012 \left( \frac{2\rho_C}{\rho_r} + 1.5 \right) \times \phi_r - E$</td>
<td>$S = 1.1B$</td>
<td>$St_r = 0.2B$</td>
</tr>
<tr>
<td>Wall</td>
<td>$B = 0.012 \left( \frac{2\rho_C}{\rho_r} + 1.5 \right) \times \phi_r - E$</td>
<td>$S = 1.1B$</td>
<td>$St_w = B$</td>
</tr>
<tr>
<td>Floor</td>
<td>$B = 0.012 \left( \frac{2\rho_C}{\rho_r} + 1.5 \right) \times \phi_r - E$</td>
<td>$S = 1.1B$</td>
<td>$St_f = B$</td>
</tr>
</tbody>
</table>

**Figure 2** - location of holes in 4-section cut (Persson et al 2001).

This model suggests that the diameter of empty hole to be more than 75 mm. To achieve this diameter three empty holes with 45 mm diameter is drilled. The equivalent diameter of empty holes is calculated using Equation 3 as follows (Konya, 1995):

$$\phi_{e2} = \sqrt{N} \phi_e = \sqrt{3} \times 45 = 78 \text{ mm}$$  (4)

The type of dynamites that must be charged into cut holes is determined by Equation 5 (Jimno et al, 1995):

$$q = \frac{55\phi_h}{PRP_{ANFO}} \left( \frac{B_1}{\phi_{e2}} \right)^3 \left( \frac{B_1 \phi_{e2}}{2} \right) \left( \frac{C}{0.4} \right)$$  (5)

Where: $q =$ Lineal charge concentration (kg/m), $\phi_h =$ drilling diameter (m), $\phi_{e2} =$ equal diameter of empty holes, $B_1 =$ Maximum distance between empty hole and holes in the first cutting quadrangle (m), $C =$ Rock constant, $PRP_{ANFO} =$ Relative weight strength of explosive with respect to ANFO.

Frequently, the possible values of lineal charge concentration are quite limited as there is not an ample variety of cartridge explosives (Jimno et al, 1995).
It’s obvious that quantity of q for stopping holes is less than that of cut holes and quantity of q for roof and wall holes is less than that of stopping holes. Also, the quantity of q for floor holes is more than those of the roof and wall holes. For holes of 1 m length and type of dynamites chosen from Table 6 the values of parameters calculated by Equations in Table 5 are listed in Table 7.

<table>
<thead>
<tr>
<th>Sections</th>
<th>Burden (m)</th>
<th>Spacing (m)</th>
<th>Stemming (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Firs square</td>
<td>$B_1 = 0.117$</td>
<td>$X_1 = 0.165$</td>
<td>$S_{t_1} = 0.117$</td>
</tr>
<tr>
<td>Second square</td>
<td>$B_2 = 0.165$</td>
<td>$X_2 = 0.351$</td>
<td>$S_{t_2} = 0.082$</td>
</tr>
<tr>
<td>Third square</td>
<td>$B_3 = 0.351$</td>
<td>$X_3 = 0.744$</td>
<td>$S_{t_3} = 0.175$</td>
</tr>
<tr>
<td>Fourth square</td>
<td>$B_4 = 0.744$</td>
<td>$X_4 = 1.578$</td>
<td>$S_{t_4} = 0.372$</td>
</tr>
<tr>
<td>Stopping</td>
<td>$B \approx 0.6$</td>
<td>$S \approx 0.7$</td>
<td>$S_{t_5} = 0.3$</td>
</tr>
<tr>
<td>Roof</td>
<td>$B = 0.6$</td>
<td>$S = 0.7$</td>
<td>$S_{t_3} = 0.12$</td>
</tr>
<tr>
<td>Wall</td>
<td>$B = 0.6$</td>
<td>$S = 0.7$</td>
<td>$S_{t_4} = 0.6$</td>
</tr>
<tr>
<td>Floor</td>
<td>$B = 0.6$</td>
<td>$S = 0.7$</td>
<td>$S_{t_5} = 0.6$</td>
</tr>
</tbody>
</table>

DEFICIENCIES OF THE 4-SECTION CUT METHOD FOR SMALL CROSS SECTION TUNNELS

Four-section cut is often applied to large tunnels with cross section area of larger than 10m$^2$. In order to apply this method to tunnels with area of less than 10m$^2$ some modifications to the equations was found to be inevitable. Applying the traditional model would lead to some miss estimation of the parameter values some of which was found to be as follows:

- $B_1$ in first cutting square is very small.
- $S_{t_1}$ is very small. The results of previously performed blasting pattern in Razi Coal Mine’s drifts show that the length of $S_{t_1}$ must be larger than 0.2 m. From the other hand $S_{t_1}$ value must be more than 2B$_1$ (Ostvar, 1999).
- Quantity of B$_2$ appears to be small.
- Comparing the values of B$_3$ and B$_4$ with smaller dimension of the tunnel cross section reveals that the third and fourth cutting quadrangles are to be eliminated.
- Spacing is estimated from S=1.1B. This amount is not appropriate for tunnels where control blasting is required. In such patterns S<B would be more acceptable.
- The length of stemming for holes of wall and floor with length of 1m seems to have been over estimated.
- Results obtained from previous blasting pattern show that stemming length of floor holes is smaller than required.

MODIFIED MODEL FOR SMALL CROSS SECTION AREA TUNNELS

Taking the above mentioned points into account the traditional model needs to be modified for tunnels with area of less than 10 m$^2$. Table 8 shows the equations suggested for this purpose. In this Table, stemming lengths of $S_{t_f}, S_{t_w}, S_{t_f}$ and perimeter holes spacing have been taken from Swedish method. The hole depths are 1 m and the type of dynamites are as indicated in Table 6. The parameters calculated from equations in Table 8 are as appear in Table 9.
Figure 3 illustrates the pattern designed based on parameters shown in Table 9. Due to restriction of tunnel dimensions, in practice, burden of stopping and perimeter holes should be reduced to less than those obtained in Table 9. Such a pattern, can be applied for tunnels with area of 8-10 m² in rocks having density of about 2.7 kg/m³. Blasting results of this pattern are shown in Table 10.

### Table 8 - Modified equations for small cross section tunnels

<table>
<thead>
<tr>
<th>Sections</th>
<th>Burden (B)</th>
<th>Spacing (S,X)</th>
<th>Stemming (St)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First square</td>
<td>$B_1 = 1.7 \phi$</td>
<td>$X_1 = \sqrt[]{2} B_1$</td>
<td>$S_1 = 10 \phi$</td>
</tr>
<tr>
<td>Second square</td>
<td>$B_2 = \frac{\sqrt[]{2} \times B_1 + 2X_1}{2}$</td>
<td>$X_2 = \sqrt[]{2} B_1 \times 1.5$</td>
<td>$S_2 = 10 \phi$</td>
</tr>
<tr>
<td>Stopping</td>
<td>$B = 0.012 \left( \frac{2\rho_c}{\rho} + 1.5 \right) \times \phi - E$</td>
<td>$S = 1.1B$</td>
<td>$S_{st} = 0.5B$</td>
</tr>
<tr>
<td>Roof</td>
<td>$B = 0.012 \left( \frac{2\rho_c}{\rho} + 1.5 \right) \times \phi - E$</td>
<td>$S = 0.8B$</td>
<td>$S_{ro} = 0.5B$</td>
</tr>
<tr>
<td>Wall</td>
<td>$B = 0.012 \left( \frac{2\rho_c}{\rho} + 1.5 \right) \times \phi - E$</td>
<td>$S = 0.8B$</td>
<td>$S_{wa} = 0.5B$</td>
</tr>
<tr>
<td>Floor</td>
<td>$B = 0.012 \left( \frac{2\rho_c}{\rho} + 1.5 \right) \times \phi - E$</td>
<td>$S = 0.8B$</td>
<td>$S_{fl} = 0.5B$</td>
</tr>
</tbody>
</table>

### Table 9 - Calculated parameters based on equations in Table 8

<table>
<thead>
<tr>
<th>Sections</th>
<th>Burden (m)</th>
<th>Spacing (m)</th>
<th>Stemming (m)</th>
<th>No &amp; type of dynamite per hole</th>
</tr>
</thead>
<tbody>
<tr>
<td>First square</td>
<td>$B_1 = 0.133$</td>
<td>$X_1 = 0.19$</td>
<td>$S_1 = 0.32$</td>
<td>3.5(a)</td>
</tr>
<tr>
<td>Second square</td>
<td>$B_2 = 0.29$</td>
<td>$X_2 = 0.6$</td>
<td>$S_2 = 0.32$</td>
<td>3.5(a)</td>
</tr>
<tr>
<td>Stopping</td>
<td>$B = 0.6$</td>
<td>$S \approx 0.7$</td>
<td>$S_{st} = 0.3$</td>
<td>3(b)</td>
</tr>
<tr>
<td>Roof</td>
<td>$B = 0.6$</td>
<td>$S \approx 0.5$</td>
<td>$S_{ro} = 0.3$</td>
<td>2.5(c)</td>
</tr>
<tr>
<td>Wall</td>
<td>$B = 0.6$</td>
<td>$S \approx 0.5$</td>
<td>$S_{wa} = 0.3$</td>
<td>2.5(c)</td>
</tr>
<tr>
<td>Floor</td>
<td>$B = 0.6$</td>
<td>$S \approx 0.5$</td>
<td>$S_{fl} = 0.3$</td>
<td>3(b)</td>
</tr>
</tbody>
</table>

Figure 3 - Blasting pattern based on the modified method
Table 10 - Blasting results of the pattern based on equations in Table 8

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Equation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD: Specific drilling (m/m³)</td>
<td></td>
<td>4.34</td>
</tr>
<tr>
<td>SC: Specific charge (kg/m³)</td>
<td></td>
<td>1.47</td>
</tr>
<tr>
<td>AE: Advance Efficiency (%)</td>
<td></td>
<td>90</td>
</tr>
<tr>
<td>Length of bootleg (m)</td>
<td></td>
<td>≤0.10</td>
</tr>
</tbody>
</table>

**FINAL BLASTING PATTERN**

Comparison of Tables 10 and 4 implies that application of the pattern shown in Figure 3 leads to much better results in compare to the pattern previously practiced. Although the model of Figure 3 looks satisfactory, search for getting lower amounts of specific charge and specific drilling continued by gentle practical modifications.

Therefore, the model was improved step by step in consecutive blasting runs. After several blasting cycles, ultimate optimized blasting pattern was obtained (Figure 4). As shown in Figure 4, in ultimate optimized blasting pattern, first cutting square of Figure 3 has been eliminated. As estimation of parameters of second cutting square is done by using the parameters of first square, the later is therefore, determined although not made in practice. Results of ultimate optimized and previously practiced blasting patterns are shown in Table 11. As shown in this table, great improvement has been achieved by the modified model.

![Figure 4 – Final optimized blasting pattern](image)

**TABLE 11 - COMPARISON OF FINAL AND PREVIOUS BLASTING PATTERNS**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Previous</th>
<th>Ultimate</th>
<th>Improvement, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific drilling (m/m³)</td>
<td>4.75</td>
<td>3.91</td>
<td>21.5</td>
</tr>
<tr>
<td>Specific charge (kg/m³)</td>
<td>1.51</td>
<td>1.28</td>
<td>18</td>
</tr>
<tr>
<td>Advance Efficiency (%)</td>
<td>80</td>
<td>90</td>
<td>12.5</td>
</tr>
<tr>
<td>Length of bootleg (m)</td>
<td>0.10-0.20</td>
<td>≤0.10</td>
<td>Average: 50</td>
</tr>
</tbody>
</table>
CONCLUSIONS

- Applying the traditional four-section model to small cross section tunnels would lead to some miss-estimations of the pattern parameters. A modification to the model is inevitable in such tunnels particularly for estimation of stemming, cutting holes burden and perimeter holes spacing.
- In four-section model the proper value of uncharged central hole for tunnels with cross section area of 8-10 m² is 75-80 mm.
- In case of small cross section tunnels the third quadrangle of four-section cut is not required.
- Although the specifications of the first quadrangle of the cut are determined, the holes of this quadrangle corners are not drilled.
- The modified model results in great improvement of blasting efficiency and cost saving.

REFERENCES

INTRODUCTION OF CONTINUOUS HAULAGE (4FCT)
AT THE CLARENCE COLLIERY

Allison Golsby

ABSTRACT: Clarence Colliery is exploring the 4FCT as a continuous haulage system. The 4FCT (Flexible Conveyor Train) is used as a continuous conveyor between the current continuous miner unit and the fixed boot end of the panel conveyor vs the three shuttle car system, currently in use in a continuous miner partial pillar extraction panel at Clarence Colliery. At Clarence the base case is three shuttle cars per continuous miner, using a cut and flit partial pillar mining method.

The Joy 4FCT has not been utilised in Australia before. The main questions are: What is a 4FCT? How does it function? What are the benefits? How do we introduce the 4FCT safely? Why Clarence Colliery? What else needs to change at Clarence to optimise the 4FCT? Moving forward?

Clarence Colliery needs to meet specific subsidence, water make, geological and mine design requirements. The 4FCT implementation offers Clarence colliery the opportunity to explore new technology, while testing Clarence’s analysis, measurement, assessment and continuous improvement processes.

The choice of the ‘better option’ in any analysis is not always made for monetary reasons. Often option choice decisions are made for safety, operational ease or engineering or mine design optimisation. Money is not the prime driver, but part of the decision making process.

INTRODUCTION TO CLARENCE COLLIERY

The Clarence Colliery is located at Newnes Junction on the Newnes Plateau at Clarence in the Australian State of New South Wales. The Clarence Colliery is 10 km from Lithgow off Chifley Road between Dargan and the Zig Zag railway and 140 km East of Sydney, as shown in Figure 1.

Coalex Pty Ltd (ACN 000 694 315, Clarence Coal Investments Pty Ltd (ACN 003 772 174), Japan Energy Australia Pty Ltd (ACN 003 919 668) and SK Australia Pty Limited (ACN 003 694 225) are participants in an unincorporated joint venture (the Clarence Joint Venture). Clarence Colliery Pty Ltd (ACN 001 680 584) is the manager of the Clarence Joint Venture.

With resources of approximately 230 mt, the Clarence Colliery (Clarence) has large reserves of good quality coal sufficient to support mining for more than 20 years. The marketable reserves, including a recent additional mining lease area, are estimated to be 48 mt.

Clarence is an underground mining operation and has been in operation since 1979. Throughout its history Clarence has mined The Katoomba Seam using the partial pillar extraction system technologies. The Katoomba Seam is the upper most coal seam in the Illawarra Coal Measures. Clarence Colliery partial pillar extraction system resultant subsidence is minimal, an example is shown in Figure 2. Clarence has achieved safety and innovation awards for the mine site. Careful design combined with environmental monitoring and safety systems has enabled Clarence to gain these awards.

Clarence has a production capacity of up to 2.1 Mtpa with Australia’s most productive continuous miner operation using three continuous miners. A high capacity continuous haulage mining system (Joy 4FCT) has been ordered for installation by April 2010. A place change mining method is used in development and in partial pillar extraction panels.

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1 Mine Engineer, Clarence Colliery, New South Wales, Australia
Clarence primarily produces low sulphur, thermal coal for sale and export to markets in Korea, Taiwan and Japan where the coal is used for power generation. The efficient coal preparation plant enables the development of various product types. A coal sizing plant serves the domestic market.

The mined coal is transferred to surface by conveyor and reduced in size (<50 mm). The coal is then graded and some washed (to remove additional ash) and screened before it is all stockpiled for loading. The export coal is loaded on trains at the Clarence rail loop on the Western line at Newnes Junction. The Clarence coal exports are shipped mainly through the Port Kembla coal-loader.

**Clarence Colliery seam characteristics**

Clarence Colliery only mines the *Katoomba Seam*. The seam characteristics described for Clarence Colliery reflect the *Katoomba Seam*. There are other coal seams at Clarence within the Illawarra Coal Measures that have not been exploited by Centennial coal.

The Clarence Colliery seam characteristics are typically:

- m to 4 m in height with few partings.
- The seam contains low ash, low sulphur and phosphorous.
The seam gradient dips at an average of 1° to the east by north east.

Depth of cover is generally less than 200 m, though can reach 270 m.

Small geological faults exist in the projected mining panels. The Clarence Colliery geological model has been calibrated against the advancing faces, with a good correlation between the predicted and actual values. The Clarence geological model has been built up from geological mapping projected forward and aerial magnetic surveys calibrated against actual mapping. The faulting found on the surface is vertical and generally extend to the seam.

- The Clarence Colliery experiences a low stress environment.
- The Katoomba Seam and adjacent strata contains negligible methane and negligible carbon dioxide gas.
- The seam has a below average water make.
- The roof is a competent sandstone, with a hard sandstone floor.

Clarence Colliery mine design criteria

Clarence Colliery uses essentially a bord and pillar mining method, using partial extraction. The partial pillar extraction design maximises coal resource recovery without fracturing the over burden. Clarence mitigates ground water inflow and disturbance to aquifers, by keeping the over burden as intact as possible.

Clarence Colliery uses the partial extraction mine design in conjunction with balanced development and extraction timings. This balance optimises conveyor belt utilisation and therefore coal clearance from underground.

The Clarence Colliery mine design results in minimised surface subsidence under sensitive surface features, such as shown below in Figure 3. The extent to which surface subsidence and disturbance can occur is set by the following parameters:

- Subsidence of less than 100 mm
- Tilts of less than 2 mm/m
- Strains of less than 1 mm/m
- Cliff face protection protocols

WHAT IS A JOY 4FCT?

A 4FCT (Flexible Conveyor Train) is used as a continuous conveyor (as shown in Figure 4) between the current continuous miner unit and the fixed section belt of the panel conveyor instead of the three shuttle car system (with an example shown in Figure 5) with a conveyor boot end, currently in use in a continuous miner partial pillar extraction panel at Clarence Colliery. At Clarence the base case is three shuttle cars per continuous miner, using a cut and flit partial pillar mining method.

The choice of the ‘better option’ in any analysis is not always made for monetary reasons. Often option choice decisions are made for safety, operational ease or engineering design optimisation. Money is not the prime driver, but part of a thorough decision making process, using an investment evaluation process model.

The 4FCT is designed for the hopper to sit under the continuous miner conveyor, moving the coal into the lump breaker to crush the coal to the desired size, then feeding the coal onto the flexible belt and then tipping coal onto the section belt.
Figure 3 – Surface features under which Clarence has mined

Figure 4 – Flexible Conveyor Train
Figure 5 – Shuttle car

The Clarence Colliery 4FCT (4FCT01) specifications are (Joy Manufacturing 2008):

- 4FCT Flexible Conveyor Train (as shown in Figure 7 and 8)
- Over the top discharge
- Initial train length 110 m
- Dynamic move up (DMU) and retreat with cam, rails, propulsion system and stacker
- Rated capacity - size and convey 1,400 t/h, or 65,000 t/m
- Maximum turning radius of 9.44 m
- Minimum mining height 2.3 m, optimal 2.5 m, max mining height 3.5 m
- Belt width 1 065 mm
- Compatible with Roof Bolter Joy 4 head multibolter
- Coal is the material to be mined
- Maximum Grade < 5°
- Umbilical cable length 6 m
- Australian compliant engineering
- Traction speed 0-23 m/min
- Belt speed 0-215 m/min
- Conveying rate 24.4 t/min
- Adjustable lump breaker (Global brand) (as shown in Figure 7)
- Compatible with Continuous Miner Joy 12CM12B
- Entry width 5.5
- Machine Input voltage 950 VAC (range 855 to 1045 VAC)
- Machine Input Frequency 50 Hz
- Traction drives – four (4)

The 4FCT is expected to provide for Clarence Colliery:
Continuous coal clearance for the continuous miner, making the miner truly continuous
The 4FCT can tram in advance and retreat, convey and deliver coal simultaneously
The 4FCT follows the miner to multiple drivage sequences
The 4FCT provides services (as shown in Figure 14) to the continuous miner, such as electrical power, water and communications
The 4FCT crushes the coal to a predetermined size before the coal is carried on a conveyor

WHY USE THE 4FCT AT CLARENCE COLLIERY?

There were several elements considered by Clarence Colliery; with safety the prime driver. Some of the elements considered are listed below:

Safety – removes operational personnel from interaction with equipment and shuttle car work environment (as shown in Figure 6), operating at a slower traction speed. The 4FCT removes the ergonomic issues associated with shuttle cars (Joy Mining Machinery and Centennial Coal, 2009). The ergonomic issues involve the operator interacting with rough wheeling roads, roadway clearances, seat belts and canopies approximately 200 times a shift. The 4FCT reduces manual handling significantly in cable handling and section service moves.

Capability – increased production, improved productivity, increased marginal revenue, with the extra coal sold to current or potential customers, the shuttle cars are the bottle neck in the Clarence colliery coal clearance system. The increased production is a result of eliminating shuttle car wait times and therefore producing continuously.

Compatibility – Clarence mining conditions are conducive to 4FCT operation, the 4FCT is a modular ‘add on’ to the rest of the Clarence current Joy upstream and downstream equipment and boltier. The existing coal clearance, preparation and handling facilities match to the 4FCT. The 4FCT is quicker and safer with an optimised maintenance process from variable speed components and interchangeable parts from other Joy machines (Burgess and Raines, 2008).

Flexibility – The 4FCT is less able to react to a mine design change than the current system of cut and flit with shuttle cars. On the other hand the 4FCT is able to adapt to development and extraction mining methods readily. The 4FCT comes in modular sections allowing for the 4FCT to be lengthened as the mine design changes. Clarence Colliery can still use continuous haulage in a bord and pillar mining method, with partial pillar extraction and meet the regulator’s approval requirements.

Sensitivity – designed for the underground coal mining environment such as at Clarence, as seen in the United States experience and yet to be tested in Australia Drotsky, 2006). The Clarence Colliery employees readily accept and move forward with new technology change.

Cost – The 4FCT has a higher capital cost compared to shuttle cars, with potentially lower overall pit operating costs. The 4FCT is new to Australia with most of the implementation identified as a research and development project for Clarence Colliery. The 4FCT presents less maintenance than the shuttle car fleet for the following reasons: non intrusive maintenance, low mean time to repair, compatibility with the current equipment used at Clarence and therefore minimising 4FCT life cycle costs.

Mining approvals – the 4FCT operation is conducive to controlling the surface subsidence in Clarence’s mining operation and improving management of ground water.

MINE PLANNING AND PANEL DESIGN

The mine plan will need to change to accommodate the turning angles and the length of the 4FCT. One of the optimal mine plan designed for the 4FCT is shown in Figure 9. This particular mine plan came from 20 options assessed on coal recovery, practicality of mining, geotechnical needs, approval considerations, the 4FCT parameters and logistics of the mining cut and flit system. Current mine plan relies on 90° cut throughs and 5, 7 or 9 heading layouts.
The proposed 3, 5 and 7 heading layouts have been analysed to assess potential productivity, recovery and safety in development and extraction. The Clarence 4FCT panel designs are driven by the depth of cover, factor of safety for the remnant pillars, geological features and coal properties are a few. The
factor of safety for the panel design shown in Figure 9 is 2.3, making these designs very conservative. The panel designs are required to support the roof span above panels. To maintain these spans the panels need to be slender, contain spine pillars for extra support or reduced extraction in less competent roof areas.

As the panel design as shown in Figure 10 has been assessed, so the sequencing needs to be analysed and developed to optimise the productivity, recovery and utilisation of the new 4FCT technology and Clarence Colliery mining operations. This figure shows the mining sequences chosen for the 4FCT in a three heading layout, considering bolter and miner moves, with other associated activity interaction. Currently, with the equipment at Clarence the maximum cut-outs are 15 m. Analysis will be undertaken to optimise this mining sequencing process.

As the panel design shown in Figure 10 has been assessed, so the sequencing needs to be analysed and developed to optimise the productivity, recovery and utilisation of the new 4FCT technology and Clarence Colliery mining operations. This figure shows the mining sequences chosen for the 4FCT in a three heading layout, considering bolter and miner moves, with other associated activity interaction. Currently, with the equipment at Clarence the maximum cut-outs are 15 m. Analysis will be undertaken to optimise this mining sequencing process.

Because of its length, the 4FCT will not be able to mine start off or mine the current panel designs at Clarence. The new 4FCT panel design to allow installation and start off the panel (as shown in Figure 11) differs radically from the current knowledge, affecting the predictability of subsidence, stability of the pillars, coal output and ventilation design. The change in panel design will need to have a new ventilation plan devised to ensure enough air reaches the mining face as well as the increased panel differential pressures. There will be a requirement for more effective ventilation, requiring the same ventilation flow to the miner as previous with less cross sectional area for the air to travel, increasing differential air pressures in the 4FCT panels. The increased differential air pressure in the 4FCT panel requires the current non pressure rated ventilation stoppings to be higher rated, which is a more expensive stopping requirement.

**ALIGNING MINING OPERATIONS WITH 4FCT METHODOLOGY**

Clarence Colliery needs to align the current mining operations on site with the new 4FCT mining methodology.

The 4FCT is a piece of equipment designed and built in America. Before being used in Australia, the 4FCT needs to be compliant with Australian legislation, Australian standards, regulatory standards and the Clarence Colliery standards. Risk assessments were completed in the early stages of the 4FCT evaluation, so outcomes could be used to improve the design. There is an identification process, with the original equipment being modified to meet all of the requisite standards. Changes to original plant can result in consequences that will affect facets of the 4FCT operational parameters.

The 4FCT panel will develop in a faster linear advance than the other panels developed at Clarence. To keep the 4FCT advancing, the belt maintenance (Belt take up and loop arrangement) needs to be upgraded to allow for greater belt run out lengths between belt inserts and to speed up belt moves.

The belt advance structure will need to be a fast installation designed. To reduce 4FCT downtime while waiting for a belt move to be completed, the fast installation will be a critical path task for the 4FCT to optimise its utilisation and continue its advance.

The faster panel advances require a method of moving transformers and other services forward to support the continuous miner and the 4FCT. A faster method has been identified. This new technology utilises a monorail support system to rail the services forward in a panel.

The monorail and DMU (Dynamic Move Up Unit as shown in Figure 12) (Sebeck, Freeman and Ziegler 2008) from the 4FCT has a higher profile compared to the shuttle cars. This profile increase reduces the space available to install and maintain an explosion protection system for a coal mine panel. Part of
Figure 9 – Three heading layout design with expected extraction shaded

Figure 10 – Planned sequencing of the 4FCT Mining Process (Clarence Colliery 2009)
the pre-4FCT explosion protection system was a water barrier. With analysis, a stone dust barrier was considered an alternative protection method and then risk assessed (Golsby, 2009). The new installation process needs a pod that is designed for the ‘bat bag’ installation, with new risks requiring control. One of the solutions is to develop a holistic explosion protection system based on a risk assessment and the regulatory requirements. Application to the regulator would be necessary for these modifications to be assessed and benchmarked against best practice methods used by other coal mines.

The 4FCT will require more electrical power and water, compared to the shuttle car base case. There is a need to replan and rearrange our services to meet the needs of the 4FCT.

The 4FCT has been envisaged to potentially carry more coal than the current conveyor belts are rated for. The solution will require the pit conveyors to be analysed to convey coal at the 4FCT coal clearance rate.

The existing bonus system for Clarence Colliery employees is predominantly coal production driven. The new 4FCT technology will require a modified incentive system to ensure that the outcomes desired from the new technology are encouraged.

Consultation with all Clarence personnel will assist the new technology methodologies to align with current practices. This will ensure that all the changes needed and the safety considerations required, have been identified, which may encourage ownership of the new technology. Ownership of the 4FCT by all those at Clarence is necessary to ensure the 4FCT succeeds. The new processes and systems require comprehensive competency based training. This training is an imperative, if the 4FCT is to function as expected, particularly as safety is the first consideration.

The task of aligning the current mining operations at the Clarence Colliery with the 4FCT methodology process requirements involves a degree of complexity. Considerations included:

- Risk assessing the process, the design, time scale, compliance and associated infrastructure. The risk assessment outcomes help identify gaps that can be filled before they become critical.
Change management principles were applied to the introduction of a continuous haulage system at Clarence. The change management process offers a structured path to prompt for system, process and practice development reducing unplanned issues.

- Competency based training in the commissioning procedure, standards and agreed method of resolution will promote safe and effective operation, maintenance, transport, communication and problem solving.

- The new mining process will need to be process driven and not time driven as is current practice. The time driven model allows for mining to progress with panel maintenance (belt moves and installations) only on one shift, no matter where the mining process is up to. The 4FCT will need to have the panel maintenance to occur at a certain point in the mining process. To be effective, the 4FCT will need belt moves and installations ‘as and when needed’. The 4FCT cycle will not be able to align with a certain shift. The culture at Clarence is very time driven, with the maintenance specialists all being on the night shift. With the new process driven culture, skills will need to be improved over all shifts.

- Outbye coal clearance system needs to be operated for reliability and efficiency. The Clarence Colliery coal clearance is at optimal efficiency when section advances occur every 24 hours.

- System monitoring and communications reduce downtime, damage, while optimising safety, coal clearance and maintenance.

- The 4FCT has brought the monorail services unit to Clarence and it has readily been accepted. The monorail, an example shown in Figure 13 speeds up section advances and service moves, while reducing manual handling issues.

- Materials and parts require commissioning, to test all the operational parameters and documented for commissioning to be a success. The commissioning outcomes can be used to ensure high standards and expectations from both sides are met.

- The gaps identified during the commissioning need to be remedied before delivery is accepted.

CONTINGENCIES

Contingency plans are developed to optimise the introduction of the 4FCT. The 4FCT could experience teething problems. These challenges, if experienced may require the following contingencies:
The original shuttle car fleet will need to be maintained and kept close to the 4FCT panel for fast deployment if the 4FCT was inoperable for any reason.

To prevent disruption to the mining cycle workflow, new ventilation and bolting processes or appliances may need to be implemented to reduce the installation change issues, as the panel becomes a process driven, not a time driven mining cycle. The support cycle is critical to the 4FCT continuous mining process in the cut and flit mining method at Clarence. If poor roof is encountered, the support cycle will lengthen, creating 4FCT wait times, reducing coal production.

Renegotiate the bonus scheme with the Clarence workforce to ensure all find the bonus scheme acceptable. Consult with all personnel to find a win-win solution.

Implement a culture change to allow the role changes needed to make the new system 4FCT process driven. Ensure that Clarence employees readily accept and apply the agreed processes for the new 4FCT technology changes.

Training and education from the supplier, using competent personnel. The development of quality quantitative risk assessments with specific outcomes. These outcomes will drive the training and equipment needs associated with the 4FCT mining process.

As with any other change, technology changes mean there will be a need for measurement of Critical Performance Indicators (KPIs), monitoring, review, analysis and the development of a continuous improvement process. Over time, Clarence Colliery aims to increase production and productivity, with the use of efficient continuous improvement practices.

Ensure safety is addressed with all personnel, especially relating to the changes and consequences surrounding the 4FCT.

The use of benchmarking to estimate the expected 4FCT behaviour at Clarence by visiting similar mine sites using similar processes.

### INVESTMENT EVALUATION PROCESS

An effective investment evaluation process reduces the risk to business and increases the chances of a project succeeding. The 4FCT project is moving through the five phases of the investment evaluation process model. The five phases are concept, prefeasibility, feasibility, execution and operation. These phases each have several steps, which need to be completed before moving onto the next phase in the investment evaluation process model. All projects need to go through an extensive evaluation process. The 4FCT is currently at the execution phase. The Clarence Colliery 4FCT is currently being fully built.
for testing at the Joy Workshops at Mossvale. Once the testing has been completed successfully, the 4FCT will be broken down and moved to Clarence Colliery for rebuilding and commissioning onsite.

Is the 4FCT the right choice? To ensure the analysis draws the right answers, three other coal transport systems were compared against the same criteria, with the 4FCT being the optimal product choice. The alternative systems were: shuttle cars, Bucyrus chain haulage bridge conveyors, Sandvik tear drop conveyor system or bridge type conveyor belt system. All of these alternatives were seen to be less reliable, less compatible (needs a cement drive or have a maximum 450tph output) or may not be a proven technology. A feasibility study identified these alternatives, studied them and found the optimal system for continuous haulage at Clarence to be a 4FCT (Hedges, Griffith and Hack, 2008).

Some of potential risks with the introduction of the 4FCT to Clarence Colliery, identified in risk assessments (Clarence Colliery 2009) are:

- The 4FCT does not function as expected or planned, an iterative design process, with a specific original scope of work, good cooperation between OEM and customer and a thorough contract management system has ensured that all stakeholders have the same understanding of the 4FCT expectations (Hargraves Martin 1993).
- The 4FCT does not appear to be compatible with the other parts of the cut and flit mining system.
- The 4FCT is not compatible with the culture and skills now available to the pit. Skills need to be transferred between employees and new skills attained.
- The mine plan is not as flexible as the current mining processes at Clarence. The mine plan can not deviate for geological features as readily as the original Clarence mining system.
- There will be a different interaction hazard between the equipment, rib and personnel.
- The ventilation may not perform as anticipated.

Since the 4FCT risk assessments have identified potential issues, these outcomes have been assessed with controls developed to reduce any adverse 4FCT outcomes.
COMPLIANCE?

The 4FCT has been designed and built to American standards and needs to be Australian compliant, an important component of the 4FCT introduction to Clarence Colliery. Effective communication and consultation with the Clarence employees improves the safety, production and system interactions into the future. Effective consultation and communication for the 4FCT project includes:

- Regular meetings with employees outlining plans for the implementation of the 4FCT coal haulage system
- Quarterly meetings outlining steps in design, construction, commissioning and operation of the 4FCT
- Ongoing involvement of the people from different shifts and disciplines in seeking contribution in dealing with potential changes in production, service, and specialist support activities
- Involvement of Clarence tradesmen and operators during the build and compatibility testing at the OEM, Joy Mining Machines Workshops at Moss Vale.
- Commence initial trades and operator training to enable the components to be transported underground and assembled.
- The OEM has readily moved the 4FCT through the Australian compliance process, providing support and cooperation to the regulator and the customer/mine operator.

MOVING FORWARD

The 4FCT will be assembled at Clarence Colliery, commissioned and will start production shortly thereafter. Moving into the future, there are still factors and requirements to be met for this project to be a success. Some of these are:

- A significant change in safety and productivity
- Opportunity for the OEM and customer mine operator to partner in design, development, and implementation of a safe and efficient system of work
- Recognise and action the requirement to re engineer existing methodologies for advancing and retreating panel services
- Review current production and maintenance process monitoring measurement and analysis for application to 4FCT operation
- Determine appropriate set of KPIs in establishing continuous improvement program for 4FCT operations at Clarence

The identified audit process requirements include:

- Delivery tests need to include dimensions, observation of operation, record all functions, check all fluid pressure levels, check overload levels, test emergency stops and pilots, test thermistors and RTDs, test fluid flows, review compliance with all regulatory requirements, check polarity, test operational parameters, vibration testing, compatibility with other equipment onsite and check signage.
- Providing a risk assessment for operational use with compliance to site procedures, such as isolation.
- Design and operational compliance to Australian Standards, Clarence Colliery standards and legislative requirements.
- Training compliance gap analysis
- Mine extraction actual compared to planned for the 4FCT panels
- Mine schedule actual advance compared to planned

Before the 4FCT was ordered, the elements and components Clarence required of the 4FCT, were identified, described, and measured. The standards set by these requirements were formatted into the
required audit system. The audit system will ensure that the 4FCT meets the Clarence Colliery needs, with a method to measure agreed milestones to identify gaps early and to keep the scope of the project on track (Chan and Mauborgne, 1999).

The Clarence expectations of the 4FCT are:

- Life cycle of the 4FCT capable of performing as specified
- High level of automation
- Ease of operation
- High level of reliability
- Meet the legislative requirements of Australia
- Design risk assessment, inspections and test plans to be supplied by manufacturer and may be audited by the mine operator.
- Installation of 4FCT managing hazards, installation procedures supplied, competency training by supplier, supply of drawings, parts manual, supervised installation by supplier using competent personnel, with fit for purpose tools and parts.

ACKNOWLEDGEMENTS

The Clarence Colliery wishes to acknowledge the contribution made by the following people:
Jack Blackadder – Mine Surveyor
Johan Blignaut – Technical Services Superintendent
Paul Bowldige - Electrician
Terry Coggles – Project Administration
Andrew Dellabosca - Deputy
Steve Durie – Miner
Paul Glasson – Manager Electrical Engineering
Allison Golsby – Mining Engineer
David King – Mining Engineer
Mick Komma – Miner
Steve Lemcke - Fitter
Kevin McCusker – Commercial Manager
Bob Miller – Business Improvement General Manager
Brian Nicholls – Production Manager
Graeme Owens – Maintenance Manager
Peter Raines – 4FCT Project Engineer
Barry Riley – Site Check Inspector
Justin Ryan – Manager Mechanical Engineering
Gregory Shields - Mine Manager
Grant Sullivan – Safety / Training Manager
Bernard Vandeventer – Mine Manager
Bob Williams – Project Manager

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OUTBURST THRESHOLD LIMITS – CURRENT RESEARCH OUTCOMES

Dennis Black and Naj Aziz

ABSTRACT: Since outburst threshold limits were imposed on Bulli seam mines in 1994 the occurrence of coal and gas outburst incidents have virtually been eliminated from the Australian coal industry. With the reduction in incidents and therefore hazard to the industry there has been a corresponding drop in research effort in this area. Some mines have in recent years reviewed and raised threshold limit values and non-Bulli seam mines accept the method of threshold limit value determination introduced by GeoGAS, based upon the DRI900 concept.

Detailed analysis of gas desorption data from a variety of Australian coal mines, representing different coal seams with variable gas content, gas composition, rank, type, structure, etc., has been undertaken and the results indicate significant relationships which impact the accepted method of outburst threshold determination.

The relationships are discussed and a new method of determining outburst threshold limits, applicable to non-Bulli seam mines is presented.

INTRODUCTION

During an analysis of Bulli seam coal sample gas content data, as part of broader research into the factors that impact the drainage of gas from coal seams, several relationships were identified. These factors offered new insight into the nature of gas emission from coal and the method of determining outburst thresholds for non-Bulli seam mines (Black et al., 2009). From the analysis of 930 samples it was found that gas composition had little impact on the relative proportion of the three components of total gas content, Q1, Q2 and Q3, and the relationship between total gas content and Q3 gas desorption rate index (DRI) was also virtually independent of gas content. The observed relationship was considered potentially significant given the use of DRI for the determination of outburst threshold limits applicable to non-Bulli seam mines. Additional data was sought to extend the data analysis to include additional Bulli seam and non-Bulli seam mines. The gas database was increased to 4 785 samples from eight mines representing the north-west and southern Sydney basin and the Bowen basin. From the complete dataset, DRI data was available for 3 824 samples. Using the new data the analysis was repeated and extended to further investigate the relationship between DRI and total gas content, the impact of gas composition and the potential impact on the method used to determine outburst threshold limits applicable to non-Bulli seam mines.

BACKGROUND

An assessment of the work undertaken by Lama (1995) that led to the recommendation of gas content outburst threshold limit values applicable to the Bulli coal seam has been presented previously (Black et al., 2009). The gas content values nominated as threshold limit values were considered appropriate as no recorded outburst had occurred below this level. The Department of Mineral Resources applied a further factor of safety to the threshold limits and prescribed the limits to all mines operating in the Bulli seam (Clarke, 1994).

Williams and Weissman (1995) proposed the use of gas desorption rate to determine applicable outburst threshold limit values (TLV’s) for mines operating in coal seams other than the Bulli seam. Underpinning this desorption rate proposal was an apparent relationship to the Bulli seam TLV’s previously proposed by Lama, shown in Figure 1. The DRI is determined by measuring the volume of gas emitted from a 200 gram coal sample after crushing for 30 seconds and relating the result to the total gas content of the sample (Williams, 1997). The figure shows the distribution of gas emission volume relative to the total gas content of each sample which suggests that a CO2 rich sample liberates gas at a faster rate than a CH4 rich sample of similar total gas content. The data suggests a linear relationship.

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relationship between total gas content \( (m^3/t) \) and desorbed gas volume \( (ml \text{ per } 200 \text{ g sample crushed for } 30 \text{ secs}) \) which may be represented by the equation, \( Y = \alpha X \), where \( Y \) is total gas content \( (m^3/t) \), \( X \) is desorbed gas volume \( (ml) \) and \( \alpha \) is variable being 0.01 for >90% CH\(_4\) and 0.0067 for >90% CO\(_2\).

Based on the outburst TLV's of 9 \( m^3/t \) and 6 \( m^3/t \), which correspond to gas compositions of 100% CH\(_4\) and 100% CO\(_2\) respectively, a common desorbed gas volume of 900 ml is liberated. Williams and Weissman (1995) concluded that subject to knowing the relationship between gas emission volume during Q3 testing and the total gas content an applicable outburst TLV is the gas content value corresponding to an emitted gas volume of 900 ml (DRI900).

Figure 1 – GeoGAS desorption rate (DRI900) relative to Lamà’s outburst threshold limit values

Figure 2A – DMR specified outburst threshold levels

Figure 2B – Revised thresholds – West Cliff and Tahmoor

In the years since the introduction of the TLV’s two Bulli seam mines, Tahmoor and West Cliff, have completed formal reviews of the outburst risk which, following implementing additional safety controls such as increased gas drainage drilling density and regular core sample analysis, led to increasing the TLV’s. The original Bulli seam TLV’s and revised Tahmoor and West Cliff TLV’s are shown in Figures 2A and 2B respectively (Black et al., 2009). The changes have effectively replaced the need for outburst mining procedures (bomb squad) with increased drilling, more frequent core sampling and restrictions on daily rate of advance. Where a core sample gas analysis result falls below the WCC-Level1 or Tahmoor-Unrestricted lines (Figure 2B) mining may proceed without the need for further control or restriction. Should the gas content result from core sample analysis increase above the lower TLV additional controls are required dependent upon the conditions encountered. For example, in the case of Tahmoor Colliery the TLV is lower when mining in close proximity to geological structures than in unstructured conditions. It is noted that since increasing the TLV’s and maintaining the required additional controls neither mine has incurred an outburst event.
RESULTS AND DISCUSSION

Data was sourced from eight separate coal mines representing a broad range of conditions and coal properties, such as variable gas content, gas composition, rank and coal type, permeability, etc. Of the eight mines, DRI data was available for six. Figure 3A presents the DRI and total gas content values for each of the 3,824 samples. The results are significant showing a strong relationship between the two values which is given by

\[ \text{Total Gas Content (m}^3/\text{t}) = 0.008 \times \text{DRI (ml)} \] (1)

Equation 1 holds for each of the six individual mine datasets which, given the range of conditions present at each location, suggests that the relationship between total gas content and DRI is independent of coal mine conditions, particularly gas composition. These results differ from those presented by Williams and Weissman (Figure 1), which indicate that for a given total gas content the DRI of a CO\(_2\) rich sample is 50% greater than that of CH\(_4\) rich sample.

The relationship between total gas content and the Q3 gas content component was also investigated. Figure 3B shows the relationship between total gas content and the estimated total volume of gas liberated during Q3 testing. The results indicate reasonable correlation but the relationship is not as strong as for DRI. The estimated gas volume liberated during Q3 testing is determined by multiplying the reported Q3 gas content (m\(^3\)/t i.e. ml/g) by the standard sub-sample mass which is 200 grams in the case of non-GeoGAS laboratories and GeoGAS laboratories to the end of 2007 and 150 grams for GeoGAS laboratories from the start of 2008 to present (Williams and Weissman, 1995, Williams, 1997 and Neilsen pers. comm., 2009).

Considering the relationship between DRI and total Q3 gas volume similar values are expected, particularly in the case of low total gas content samples, where the residual gas content is not expected to be high and the bulk of the gas present within the sample will be released rapidly, within the first 30 seconds of crushing. Under normal circumstances the volume of gas liberated within the first 30 seconds of crushing (DRI) is not expected to exceed the total volume of gas released during the complete Q3 test.

Figures 4A and 4B show the distribution of the DRI:Q3 volume ratio relative to each of Q3 and total gas content. For the majority of data the DRI:Q3 volume ratio ranges between 50% and 200%, indicating that the DRI gas volume could be as little as half to as much as double the volume of gas estimated to have been liberated during the complete Q3 test. Some data fall outside this range and are considered likely to be the results of erroneous data. This result was not expected and various reasons for the difference were investigated.
The possible use of non-standard coal sub-sample mass by the laboratory during Q3 testing would affect the estimated Q3 gas volume which in turn impacts the DRI:Q3 volume ratio, resulting in an over or under-statement of the estimated Q3 gas emission volume. However for the sub-sample mass to effect the ratio to this extent the mass would have to also range from half to double the laboratory standard.

The method used to determine DRI and the impact of gas content and gas emission rate of the coal sample was also considered to impact the DRI:Q3 volume ratio. Figure 5A shows the results of gas volume liberated during Q3 testing for three separate coal sample conditions, representing high, medium and low gas content with corresponding high, medium and low gas emission rates. As noted by Williams (1997), the total time that the coal sub-sample is crushed during Q3 testing is 7 minutes (540 seconds). This same test period has been used in the examples illustrated in Figure 5A.

Figure 5B shows the same three gas emission profiles as Figure 5A however the x axis has been changed to reflect the square root of elapsed time. A similar method is used in determining the Q1 lost gas component, as explained in AS3980:1999, which presents the emission curve in a more linear trend.

Although the details of the DRI are considered by GeoGAS to be proprietary (Neilsen pers. comm., 2009) and the exact details of the calculation are confidential, equation 2 is considered by the authors to represent a credible method for determining DRI.

\[
DRI = \frac{Q_{30}}{t_{i}}, \frac{Q_{f}}{t_f}
\]

where:

- \(Q_{30}\) = volume of gas liberated (ml) during the initial 30 seconds of crushing.
- \(Q_{f}\) = volume of gas liberated (ml) during the full 7 minute crushing period of the Q3 test.
- \(t_{i}\) = 5.5 (\(\sqrt{30 \text{ secs}}\))
- \(t_f\) = 23.2 (\(\sqrt{540 \text{ secs}}\)) i.e. square root of 7 minute total test duration.

Figure 5B illustrates the application of equation 2 in determining the DRI gas content/gas volume through extrapolating the gas volume \((Q_{30})\) liberated at time \(t_i = 5.5 \text{ secs} (\sqrt{30 \text{ secs}})\) out to time \(t_f = 23.2 \text{ secs} (\sqrt{540 \text{ secs}})\), which represents the full 7 minute crushing period of the Q3 test (Williams, 1997).

As shown, the gas content and emission rate does affect the DRI value and the relative difference between the two values. In this example the DRI value (dashed line) relative to the Q3 gas volume (solid line) for the high, medium and low gas content/gas emission rate samples is 124%, 99% and 42%, respectively.
Alternatively, projecting a line of best fit through each of the three Q3 emissions curves and recording the total Q3 gas emission / gas content at the completion of the 7 minute test period may also provide a DRI value for each case. Using this method the difference between DRI and Q3 gas volume is reduced with the difference between the high, medium and low gas content/gas emission rate samples being 115%, 111% and 98%, respectively.

Figure 6 shows the impact of gas composition on the relationship between total gas content and DRI. Although the slope of the CO₂ dataset is adversely impacted by scatter there is little difference between the average of both the CH₄ and CO₂ datasets. In this case the results of testing and analysis of 3573 indicates that gas composition has little impact on the average relationship between DRI and total gas content.

The fact that several Bulli seam mines are now safely operating at threshold levels greater than those presented by Lama and prescribed to industry by the Department of Mineral Resources, combined with the results presented in Figure 6 may have a potentially significant impact upon the widely accepted method for determining outburst thresholds applicable to non-Bulli seam coal mines.

Figure 7A shows the relationship between DRI and total gas content, presented by Williams and Weissman (1995), for each of CH₄ and CO₂ whereby the outburst TLV’s applicable at that time correspond to a DRI value of 900. Given the demonstrated ability of Bulli seam mines to safely operate at increased TLV’s, e.g. 12 m³/t for CH₄ and 8 m³/t for CO₂, a DRI value of 1 200 may be a more suitable value for use in determining TLV’s for non-Bulli seam mines.

This analysis of DRI and Q3 gas emission data from eight separate mines has however demonstrated that gas composition has little impact on the relationship between DRI and total gas content. Therefore a single line, shown in Figure 7B, represents the common relationship between DRI and total gas content which is seemingly applicable to all Australian coal mine conditions. Given this standard relationship an outburst threshold gas content value proven effective in the Bulli seam also applies to non-Bulli seam mines. For example if a TLV of 8 m³/t is proven for CO₂ rich Bulli seam conditions the DRI is 1 000 and this gas content is applicable to all non-Bulli seam mines as a TLV for CO₂ rich conditions. Likewise if a TLV of 10 m³/t is proven for CH₄ rich Bulli seam conditions the DRI is 1 200 and this gas content is applicable to all non-Bulli seam mines as a TLV for CH₄ rich conditions.

It is understood that the gas content at the sites where outbursts occurred in the CH₄ rich conditions at both Central and North Goonyella Collieries in 2001 was above the current 12 m³/t Bulli seam CH₄ TLV. If confirmed this supports the proposed direct transferability of the Bulli seam TLVs. Assessment of the conditions present at outburst sites in other Queensland mines to confirm gas content and composition above the Bulli seam TLVs would further validate the proposed relationship.
Having analysed such a large data set it has been possible to better understand the relationship between DRI and total gas content, and the use of this relationship to determine appropriate outburst TLVs for non-Bulli seam mines. For the reasons discussed above the use of DRI900 as the basis for determining outburst TLVs is no longer considered valid, as the method produces overly conservative results. Therefore TLVs applicable to the Goonyella Middle seam and German Creek seam (Middlemount/Tieri) of 7.0 m$^3$/t and 7.7 m$^3$/t respectively (Williams, 2002) are considered to be lower than necessary and may be increased without creating an adverse outburst risk. Whilst in relatively benign conditions overly conservative TLVs have no impact upon mine operations. However with increased depth and gas content, the mine operators may be forced into unnecessarily onerous gas content reduction programs in order to avoid production delays and potential loss of reserves.
CONCLUSIONS

An extensive analysis of gas analysis data from some 4700 coal samples representing eight separate Australian coal mine conditions has been undertaken. The method of determining outburst threshold limit values applicable to non-Bulli seam mines, proposed by Williams and Weissman (1995) was reviewed. Based on recent Bulli seam experience it is suggested that the DRI value of 900 (DRI900) is no longer applicable and 1200 (DRI1200) may be a more appropriate value to use in non-Bulli seam outburst threshold limit determination.

Further analysis of the DRI and total gas content data has confirmed the existence of a standard relationship which appears to be independent of location and site conditions. This relationship provides the basis for an alternate theory for determining outburst TLV’s applicable to Australian coal mine conditions. It is therefore suggested that a TLV applicable to the Bulli seam is also directly applicable to non-Bulli seam coal mine conditions.

ACKNOWLEDGEMENTS

The authors wish to thank and acknowledge the support of GeoGAS and the Companies that have granted access to gas data.

The support of ACARP, through project C18004, is also acknowledged and appreciated.

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PARAMETERS AFFECTING COAL SEAM GAS ESCAPE THROUGH FLOOR AND ROOF STRATA

Abouna Saghafi¹, Hoda Javanmard¹ and Douglas Roberts¹

ABSTRACT: Coal seams are compact gas reservoirs and can contain large volumes of methane (CH₄) and carbon dioxide (CO₂) which are the main constituents of coal seam gas (CSG). CSG is present in various volumes and concentrations across the mining regions in the coalfields of the Sydney and Bowen basins. The variations in actual gas volumes and relative concentration of these gases in coal could be due to different gas generation/accumulation rates and different adsorption capacity of the coals, but also because of the difference in the sealing capacity of the non-coal sediments enclosing the coal seams. It is postulated that the sealing capacity of the main roof and floor rocks at a coal seam could have a major effect on the volume of gas in place (gas content).

This paper reports some results of an ongoing investigation on the gas flow parameters which affect the sealing capacity and retention of gas in coal reservoirs. The results discussed here concern, in particular, the matrix permeability (or micro permeability) and the diffusivity of the non-coal horizons in the roof and floor of the coal seams. These properties could be limiting factors on the rate of gas escape from a coal formation to the surrounding strata.

INTRODUCTION

Coal seams are high capacity gas storage media which depending on their adsorption properties and formation depth can retain large quantities of gases such as CH₄ and CO₂ in free and adsorbed phases. Most of the gas in coal is stored in the micro pore system (few nanometre pore size) where it is adsorbed onto the large surface area available in the micro pores. In the macro pore system gas is also stored in free phase, where at high pressures the quantity of stored gas in this phase can also be of importance. The quantity of gas stored in adsorbed phase (or gas content) depends on the pressure exerted by free gas molecules in the pore void volume. Hence, any fall in the free gas pressure would cause desorption of the adsorbed gas and the reduction of the gas content of coal. The desorbed or free gas can then escape from the coal seams to the upper strata and through conduits to the surface.

The origin of gas and its accumulation in coal seams would be affected by the depth. In deep coal seams gas is generally of thermogenic origin where gas had been produced as a by-product of coalification. New accumulation of CH₄ gas in these seams can be as a result of migration. Igneous activities over geological time have also resulted in the injection of CO₂ into these coal seams (Faiz et al., 2007; Embleton et al., 1985; Facer and Carr, 1979). For shallow coal seams (i.e., <300 m), most gas is of biogenic origin. The stable carbon isotopes ratio analyses of CSG from coals from the Sydney Basin show that CH₄ is generated as a result of microbial activities (Faiz et al., 2003). To allow the movement and storage of the methanogenic micro-organism and nutrients the coal seams have to act as a permeable aquifer.

It is expected that for a given coal formation the actual gas content of the coal seams would depend, among other factors, on the rate of gas accumulation within the coal seams and the rate of gas loss through the non-coal strata enclosing that formation. The main source of gas generation/accumulation in shallow seams is through microbial activities and one of the main mechanisms for gas retention is the sealing capacity of the coal formation. This capacity can be expressed in terms of micro flow properties of the roof and floor of the coal formation.

In the next sections the results of a study undertaken to evaluate the sealing properties of non-coal strata in the roof and floor of a coal formation in the Sydney Basin are presented.

Coal and rock samples used in this study

The samples for this study were obtained from a coal formation in a coalfield in the Sydney Basin. Roof and floor cores as well as coal samples were collected from an exploration borehole drilled into this coal formation.

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Two major sedimentary rock horizons, a claystone horizon in the roof and a sandstone horizon in the floor, enclose the coal sequences in this coal formation (Figure 1). This formation belongs to a coal system that contains some of the gassiest coal seams of this coalfield. Though the depth of the coal formation at the location of the drilling is relatively close the surface, high gas contents occur in the coal seams at this location. These conditions indicate favourable seal quality of the coal system reservoir. Other parameters may have also contributed to the high gas content such as high rate of biogenic gas generation and high capacity for gas adsorption of the coal; however, this study is limited to the flow properties of the enclosing non-coal strata.

In addition to the roof and floor of the coal formation the interburden rock and carbonaceous sediments were also sampled and are currently being investigated for their micro flow properties. The rock types include shale, siltstone and mudstone.

![Figure 1 - The layout of the coal formation where samples were taken for this study](image)

### METHODOLOGY

The sealing capacity of non-coal strata may be quantified in various ways. For example, it can be quantified by the time required for the coal system to lose half of its initial stored gas. Any quantification, however, requires the knowledge of the long term behaviour of the coal reservoir in terms of flow properties of the roof and floor units. Note that coal seams can also act as sealing strata.

In order to relate the flow properties of roof and floor rocks to other physical properties that may affect the gas flow within these strata, density and porosity measurement were conducted. The density was measured using a helium expansion method. The mercury intrusion technique was used to measure the porosity for sub-samples taken from roof and floor rocks.

The results of measurements of these properties are shown in Table 1. As is seen these properties are not largely different for the roof and floor samples. Density is higher for the floor sandstone and porosity is higher for the roof claystone.
Table 1 - Results of density and porosity measurement of floor and roof rocks

<table>
<thead>
<tr>
<th>Sample</th>
<th>Porosity (%)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Claystone (roof)</td>
<td>8.80</td>
<td>2.70</td>
</tr>
<tr>
<td>Sandstone (floor)</td>
<td>8.20</td>
<td>2.90</td>
</tr>
</tbody>
</table>

Measurement of micro permeability

As part of this project an apparatus was designed and built to measure the micro permeability of coal and non-coal rocks. It is a modification of a previously built system for the direct measurement of diffusivity of coal (Saghafi, 2001; Saghafi et al, 2007).

Permeability is the measure of fluid conductivity in a porous medium. The Darcy equation is used to express the flux of gas in terms of permeability and the gradient of pressure. In this equation the permeability is the coefficient of proportionality between these two entities. In one dimensional space the equation of permeation (Darcy’s equation) is,

$$\psi_p = -k \frac{\partial p}{\partial x}$$

(1)

where $\psi_p$ is the permeation gas flux (m³/m per second), $\mu$ is the dynamic viscosity of the gas (Pa·s), $k$ is the permeability of the medium (m²), $p$ is the gas pressure (Pa) and $x$ is the length (m). Note that the volumetric flux $\psi_p$ is at pressure and temperature of gas at location $x$.

Measured permeability would depend on the size of the material used. This is because coal seams are highly fractured reservoirs where the length and aperture of fractures strongly depends on the dimensions of the material used. At each scale a different fracture system can be defined and therefore the permeability would be different. The fracture permeability (macro permeability) affects the initial rate of gas production or gas injection into the reservoir. However, the long term rate of gas production would depend largely on micro permeability (matrix permeability) of the medium.

For this study it is assumed that the matrix permeability is active when the size of the sample is in the order of 0.5 to 2.0 cm (the lower diameter is suitable for highly fractured coal). In Figure 2, a schematic of the CSIRO system for measurement of coal matrix permeability is shown.

![Figure 2 - Schematic of the apparatus to measure the matrix permeability](image)

In this system a tube is partitioned by the sample into two chambers. The two chambers are initially filled with the same gas at two different pressures. Gas flows through the sample from the high pressure chamber ($P_1$) into the low pressure chamber ($P_2$) until a final, almost constant, pressure ($P_f$) is reached. The pressure data from the two chambers are continuously monitored using pressure transducers which communicate the data into a computer where the data are recorded onto a spreadsheet via a data logging application. The measured pressure-time data for the two chambers are used to calculate the permeability.

As discussed, CSG consists mainly of CH₄ and CO₂. The rate of permeation of these gases in rock and coal are different and in this study the permeability of the rocks were measured using both gases. Variations of pore pressure, pressure gradient, and temperature can be used to evaluate the effect of different conditions on matrix permeability.
For the tests reported here the system was kept at a constant temperature (27°C). The initial pressure in the high pressure chamber, $P_1$, was varied from 500 to 1000 kPa while the initial pressure difference between the two chambers was about 100 kPa for all tests. The matrix permeability was determined first for helium (He) gas to determine the upper limit of gas permeability. Measurements were then taken with CH$_4$ and CO$_2$ gases respectively.

**RESULTS OF MEASUREMENT OF MICRO PERMEABILITY**

Measurements were undertaken on small discs of 0.6 cm in thickness and 1.6 cm in diameter, prepared from the roof (claystone) and the floor (sandstone) core samples. In order to investigate the heterogeneity of the flow in the two main directions, the sample discs were cut parallel and perpendicular to the bedding. However, at the time of writing this paper only the results from gas flow in the direction parallel to the bedding were available and are discussed in this section of the paper.

In Figure 3, a microphotograph of the floor sample is shown. The sample diameter is about 16 mm. Under the microscope it could be seen that the mineral constituents of this sample are quartz particles surrounded by illite clay.

![Figure 3](image)

**Figure 3 - Photograph of sandstone sample disc (floor of coal formation) for matrix permeability and diffusivity measurement**

In Table 2, the results of 5 sets of measurements on the sandstone sample (floor rock) using helium gas (He) are presented. In this table the micro permeability is expressed both in metric units of length squared (m$^2$) and in engineering units of micro darcy (µD). Note that 1 m$^2$ = 1.013 x 10$^{18}$ µD. The results show that the permeability of this rock to He decreases from 2.8 to 1.9 µD when the gas pressure increases from 550 to 950 kPa.

Similar sets of measurements were undertaken on this sample for CH$_4$ and CO$_2$. Results of measurements show that sandstone micro/matrix permeability to CH$_4$ varies from 0.68 to 0.83 µD. The matrix permeability for CO$_2$ varies from 0.40 to 0.60 µD. For the claystone roof the results show that the matrix permeability to CH$_4$ varies from 0.23 to 0.25 µD. The matrix permeability of claystone to CO$_2$ varies from 0.12 to 0.17 µD.

All results are illustrated in Figure 4. For each gas pressure at least two measurements were conducted to ensure the repeatability and reproducibility of the results.

The results show that while the permeability of both floor and roof rocks reduces with gas pressure, the CH$_4$ permeability is higher than CO$_2$ for all pressures and for both rocks. Note that for both gases the micro permeability of the floor rock is 5-6 times larger than the roof rock.
MEASUREMENT OF DIFFUSIVITY

The diffusivity of a porous medium is a measure of the ease of gas propagation in the medium under the forces of a molecular concentration gradient. Gas diffuses in the direction of the smaller concentration. We assumed that the diffusion flow follows Fick’s 1st law which states that the flux is proportional to the concentration gradient. The diffusivity is the coefficient of this proportionality. In one dimensional space the equation of diffusion is,

$$\psi_d = -D \frac{\partial c}{\partial x} \quad (2)$$

where $\psi_d$ is the diffusive gas flux across solid coal, $D$ is the gas diffusivity coefficient, expressed in terms of length squared per unit of time (cm$^2$/s) and $c$ is the gas concentration at position $x$ in space.
In order to measure the diffusivity a system similar to the one shown in Figure 2 was used. The details of the system are presented elsewhere (Saghafi, 2001; Saghafi et al., 2007). Note that this system measures directly the diffusivity of gas in solid coal/rock. The same sample discs used for matrix/micro permeability are also used to measure the diffusivity. Because of the directional dependency of diffusivity the measurements are generally undertaken for diffusion flow parallel and perpendicular to the direction of bedding.

For the measurement of diffusivity the system can be kept at a constant temperature. For the measurements reported in this paper the temperature was kept constant at (27°C). Gas pressure was also kept constant at 101.3 kPa gauge pressure.

RESULTS OF MEASUREMENT OF DIFFUSIVITY

In Table 3 the results of measurements of gas diffusivity for roof and floor rock samples are shown. The results are for CO$_2$ and CH$_4$ flowing in the direction parallel to bedding. The results show that the CO$_2$ diffuses faster than CH$_4$ in all cases. The ratio of the diffusivity of CO$_2$ to CH$_4$ for these measurements is about 1.17. In other words, the CO$_2$ diffusion flux is 17% higher than the CH$_4$ flux for both roof and floor rocks.

In terms of the magnitude of diffusivity for the roof and floor rocks, these measurements indicate that the rocks display similar behaviour. Though, the results show that the gas diffusivity is slightly higher for claystone sample.

Table 3 - Results of measurement of the diffusivity of roof and floor rocks in the direction parallel to the bedding

<table>
<thead>
<tr>
<th>Rock type/Gas type</th>
<th>Diffusivity ($\times 10^{-10}$ m$^2$/s)</th>
<th>Diffusivity ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CO$_2$</td>
<td>CH$_4$</td>
</tr>
<tr>
<td>Claystone (roof)</td>
<td>23.65</td>
<td>20.30</td>
</tr>
<tr>
<td>Sandstone (floor)</td>
<td>22.12</td>
<td>18.91</td>
</tr>
</tbody>
</table>

DISCUSSION

The results so far show that permeation and diffusion take place at different speeds depending on the type of gas. In the case of permeation flow, CH$_4$ is faster than CO$_2$. However, for diffusive flow the order is reversed, i.e. CO$_2$ diffuses faster than CH$_4$.

Furthermore, the flow depends on the type of rock. The permeation is several times faster in sandstone compared to claystone. However, the diffusive flow shows similar behaviour for both rock types with the diffusivity slightly higher for claystone compared to sandstone.

It should be noted that although the permeability and diffusivity sub-samples were obtained from the same section of core, they were not identical and some differences may be due to the heterogeneity of the core samples.

In addition to diffusivity and permeability of non-coal rocks, the interfacial properties of gas and rocks affect the magnitude of the capillary pressure which in turn influences the intensity of gas diffusion and permeation in micro fissures of the coal and non-coal strata. The relative wetting properties of the solid (coal) to gas and water would be also studied to quantify the capillary pressures.

This study, once completed, should assist in evaluating the gas containment property of non-coal strata in a CSG reservoir for a given period of time based on the quality of the roof and floor rocks of the reservoir, and the rate of gas generation or gas injection for gas storage purposes.
ACKNOWLEDGMENT

The authors wish to thank the CSIRO Division of Energy Technology and CSIRO Energy Transformed Flagship for financing this project. We extend our thanks to Amin Ghanizadeh, from Tehran University for assisting in laboratory measurement of permeability and to Paul Marvig and Alfredo Quintanar of CSIRO Energy Technology for preparing the samples and for measurement of porosity and density.

REFERENCES


RECOVERY OF STORED GAS IN COAL BY NITROGEN INJECTION – A LABORATORY STUDY

Raul Florentin¹, Naj Aziz¹, Dennis Black¹, Long Nghiem¹ and Kemal Barış²

ABSTRACT: With increasing worldwide concern on Green House Gas (GHG) emission and its reduction, significant interest is now directed toward finding a practical and economical ways of enhancing methane gas containment in coal deposits. Carbon dioxide sequestration has been tried successfully for the recovery of methane from coal measure rocks. A laboratory study was undertaken to examine the effect of displacing the adsorbed gases in coal with N₂ injection. To study the feasibility of removing the initially adsorbed gas from coal with another gas, tests were carried out using an in-house built multi-function outburst research rig (MFORR). Accordingly the following laboratory tests were carried out: sorption and desorption characteristics of CO₂/CH₄ mixed gas in coal, displacement characteristics of adsorbed mixed gases with N₂ injection, displacement characteristics of adsorbed CO₂ with N₂ injection, and displacement characteristics of adsorbed CH₄ with N₂ injection. The study revealed that CO₂ desorption increased by almost 30% as a result of N₂ injection, which is about double that obtained without N₂ injection. This finding has significant bearing in solving the drainage difficulties experienced at West Cliff Colliery and Panel 500 area, which had difficulty in draining CO₂ gas concentration zones.

INTRODUCTION

With increasing worldwide concern on Green House Gas (GHG) emission and its reduction, significant interest is now directed toward finding a practical and economical ways of enhancing methane gas release from coal deposits and the subsequence recovery of this gas from both mineable and unmineable coal deposits.

Carbon dioxide sequestration has been tried successfully, for some time now, for the recovery of methane from coal measure rocks. The increased attraction of the CO₂ to coal is commonly attributed to the coal’s affinity to carbon dioxide. As a result of CO₂ injection, the methane gas is stripped from its monolayer adherence to the coal matrix surface and retained in coal fracture space, which would then be readily driven out of coal by the reduction in gas pressure. A major drawback of the application of CO₂ to methane recovery is the hazards associated with coal gas outburst in underground coal mining. Therefore, this technique is limited to unmineable coal deposits.

In recent years, however, the use of N₂ injection has been tried for methane recovery, in a number of locations in the USA and in Canada, with reported success, the case study of Tiffany Project, San Juan Basin, USA, Reeves and Oudinot, (2004) and Koperna et al (2009), has indicated that the recovery of CH₄ gas from coal have increased between 10-20 % with N₂ injection. There has also been some growing interest in the possible use of CO₂/N₂ mixture as an alternative approach to enhancing methane recovery, particularly from mineable coal deposits. The injection of mixed gas is considered to possiibly provide a synergy of production mechanisms which would result in lowering of CO₂ in mine air (Reeves and Oudinot, 2004).

The storage of gas in general is dependent on the coal rank especially in virgin seams. Higher rank coals such as higher rank bituminous and anthracite coals retain methane in preference to other gases, but in some other coal seams, such as the Bulli seam of the Sydney basin, NSW, there are areas where the dominant seam gas is CO₂ rather than CH₄. In fact, CO₂ and mixed gas CH₄/CO₂ have been found in a number of locations in Tahmoor, Metropolitan, Appin and West Cliff Mines. A typical difficult to drain site is at West Cliff Mine and panel 500, where some sections of the longwall panels, with CO₂ gas, are extremely difficult to drain, despite the extensive gas drainage drilling programme. Field studies on the use of sand–propped hydraulic fracturing failed to increase the gas drainage from such sites, although the technique was proven in other coal deposits as reported by Mills et al (2006). It is suggested that the highly stressed and low permeability coal is preventing carbon dioxide from being effectively drained.

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Little has been reported in literature on the application of nitrogen for carbon dioxide stripping from coal, particularly from underground coal mines, and accordingly a laboratory study was undertaken by the gas and outburst research group at the University of Wollongong, to examine the effect of displacing the adsorbed gases in coal with $\text{N}_2$ injection.

**EXPERIMENTAL PROCEDURE**

**Equipment**

To study the feasibility of removing or displacing the initially adsorbed gas from coal with another gas, tests were carried out using an in-house built multi-function outburst research rig (MFORR). This apparatus is consists of a number of components which can be utilised in a variety of investigations, for example, it was initially built for the study of the influence of the gas environment on coal strength. The description and utilisation of the equipment have already been reported previously in various publications, Lama (1995), Aziz, Hutton and Indraratna (1996) and Aziz and Ming (1999). To reiterate, the integrated components of the MFORR include:

1. High pressure chamber, which has a load cell for measuring the load applied on the coal samples
2. Main apparatus support frame
3. Precision drill
4. Drill cutting collection system
5. Universal socket for vertical loading of coal sample in the gas pressure chamber
6. Flow meters
7. Gas chromatographer (GC)
8. Data acquisition System.

Figure 1 shows the schematic drawing of the apparatus. Figure 2 shows the general view of the apparatus. The gas pressure chamber is a rectangular prism of cast iron with removable front and back viewing plates. Its dimension is 110 mm x 110 mm x 140 mm. The viewing windows are made of 20 mm thick glass in a cast iron frame. Access to the chamber is possible by unbolting the front steel frame to the chamber. The chamber is made leak proof by inserting packers between the frame and the box as well as fitting O-rings around the loading shaft situated at the top of the chamber.

Housed in the chamber was a load cell with the capacity of 40 kN for monitoring the applied axial load. A pair of specimen loading plates with locating lips was used for holding a cylindrical specimen. Thus the mean features of the MFORR with regard to coal gas sorption studies include:

- application of stress,
- application of gas suction,
- gas pressure confinement (gas flooding),
- sample strain measurement, and
- gas flow rate measurement

Three flow meters connected in series were used to measure the flow characteristics of the escaping gas from the coal in the high pressure chamber. The flow rate range of different meters was 0-100 mL/min, 0-2 L/min, 0-15 L/min respectively. The composition of the discharged gases was measured by an online GC.
Sample Preparation

Coal samples used in this study were obtained from the Bulli Seam in Mine A. Bulk samples were taken from different locations along the longwall panel 519. Once collected the coal samples were sealed in plastic bags and transported to the University of Wollongong mine gas laboratory. In the laboratory the coal lumps were cut into regular and manageable sizes and immersed in water to minimise the effect of adverse climatic conditions.

Core samples (Figure 3) of 54 mm in diameter and height of 50 mm were then prepared for testing. The preparation of the coal core samples was carried out in accordance with the International standard for rock core sample preparation and testing (ISRM, 1981). A 2 mm diameter hole was then drilled through each sample for draining of the gas flowing through the coal core.

Figure 1 - Schematic view of experimental set-up.

Figure 2 - A photograph of MFORR apparatus and closer view of the high-pressure gas chamber.
Prior to the sorption test, each coal core sample was fitted with both axial and circumferential strain gauges to monitor the volumetric changes in the sample during the gas sorption and desorption processes.

TEST PROCEDURE

Each coal sample used for the injection study was placed in the high pressure gas chamber of the MFORR and sealed tight. In test 1, the sample was loaded axially to a predetermined initial axial loading of 200 kg which after saturation reached a 365 kg load. These loads were equivalent to a vertical stress of 0.87 MPa and 1.60 MPa respectively.

The general procedure used for testing was to initially saturate the coal sample with a specific gas (such as CH\textsubscript{4}) and then recharge the coal sample by injecting N\textsubscript{2}. The aim was to study the displacement characteristics of the initially saturated gas in coal. The applied confining gas pressure in the gas chamber was maintained constant at 3.2 MPa, thus creating a confining condition with the lateral to vertical pressures, acting on the coal sample, being in the order of 3.7:1 to 2:1 ratio. The maximum pressure ratio was marginally greater than the ground stress conditions in the Bulli Seam of the Southern coalfield of NSW.

All tests were carried under strict environmental and laboratory conditions. The room temperature was maintained constant at 22 °C throughout the experiment. This controlled experimental environment condition was considered useful with respect to coal bed methane production, carbon dioxide sequestration research, and for mine gas outburst control. Accordingly the following laboratory tests were carried out:

- Sorption and desorption characteristics of CO\textsubscript{2}/CH\textsubscript{4} mixed gas in coal (Test 1)
- Displacement characteristics of adsorbed mixed gases with N\textsubscript{2} injection (Test 3)
- Displacement characteristics of adsorbed CO\textsubscript{2} with N\textsubscript{2} injection (Test 4)
- Displacement characteristics of adsorbed CH\textsubscript{4} with N\textsubscript{2} injection (Test 5)

It is noted that sorption test 2 is not reported in this paper as the test was similar to test 1.

Adsorption and desorption characteristics of CO\textsubscript{2}/CH\textsubscript{4} mixed gas in coal

Prior to the N\textsubscript{2} injection test, a series of tests were carried out to examine the sorption (adsorption and desorption) behaviour of mixed gas (CO\textsubscript{2}/CH\textsubscript{4}) in coal when it is subjected to both axial and lateral confining pressures. During the first stage of the sorption test (Test 1) the coal sample was saturated with the mixed gas at a total pressure of 3.2 MPa. The gas saturation of coal was achieved by flooding the high gas pressure chamber initially with the mixed gas to the required pressure level. Figure 4 shows the fluctuation of gas pressure levels from the initial charged pressure level of about 3.2 MPa down to the final level of about 3 MPa over a period of around 5 days.
The pressure curve is a typical gas adsorption profile in coal with high micro porosity, which is translated to large volume of molecular structure (matrix). The initial CO$_2$/CH$_4$ composition was in a 52/48 ratio. The coal saturation level was monitored according to the minimum duration time required for full saturation with this gas type, and as reported by Florentin et al, (2009). It is interesting to note from Figure 4 that the gas pressure fluctuates during the saturation period of the coal, the intermittent movement of the gas in and out of coal structure indicates that the coal sample is in a continuous state of gas adsorption and desorption. The amplitude of the gas sorption reduces as the coal nears its full saturation.

At the equilibrium point the percentage of molecules adsorbed is equal to the percentage of molecules desorbed. The amplitude of fluctuation levels in the adsorption profile show how far the gas molecules move from the macro to the micro pores in the coal structure. In the first sorption fluctuation step shown in the Figure 4, the coal sample is almost fully saturated mainly with carbon dioxide. The volumetric strain profile confirms clearly that the sample never stop from swelling. In the following fluctuation step, the amount of methane adsorbed is increased whereas the carbon dioxide component decreases until it is almost the same as the methane. However, in order to reach the equal proportions of the gases sorbed in coal, it is necessary to run the experiment much longer. This means that time is a deciding factor for the coal to adsorb methane in equal amount that carbon dioxide.

![Figure 4 - Variation in saturation pressure of adsorbed CO$_2$/CH$_4$ (0.52:0.48) gas in coal with time.](image)

Figure 5 shows the effects of the room temperature on the confining gas pressure during the process of pressure equilibrium. Two thermocouples were located nearby the MFORR to monitor the room temperature periodically. The changes in temperature improve gas sorption (adsorption and desorption) in coal by varying the equilibrium pressure duration.

Figure 6 shows the changes in gas composition over time. The early part, of the released gas is due mostly to the confining chamber gas pressure and, the latter part is a combination of both the confining chamber gas and the desorbed gas from the coal sample. The initial gas composition of 52%/48% (CO$_2$/CH$_4$) ratio was changed to 51%/49% ratio during the first five days of coal sample saturation, and measured just prior to the release of the confining gas, which indicates a differential gas component adsorption in coal.
Figure 5 – Effect of the room temperature on confining gas pressure at saturation

The methane concentration is marginally higher than the carbon dioxide during the early stages of the adsorption process. Higher CH₄ concentration is attributed to the preferential adsorption of CO₂ in coal (CO₂ affinity to coal) and the fact that the marginal increase is due to relatively short sorption time. However, the mixed gas percentages returns to the original 52/48 level after some 30 min of gas confining pressure in the bomb being dropped down to around 500 kPa from the initial pressure level of about 3.2 MPa. The changes in mixed gas composition then reached a 56/44 ratio when the chamber pressure was almost at atmospheric level. Table 1 shows that at the initial desorption stage, CH₄ desorption was greater than that of CO₂. However, at the end of the test, both CO₂ and CH₄ desorptions increase by a similar amount of almost 15%.

![Figure 5](image_url)

**Table 1**: Breakdown of the desorption process

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>CH₄ (%)</th>
<th>CO₂ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
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<tr>
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<td></td>
<td></td>
</tr>
<tr>
<td>840</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 6 - Change in confining gas composition intermittently and pressure during Test 1

![Figure 6](image_url)
The gas composition measured at the GC is the sum of the confined and adsorbed gas. Confined gas refers to the combination of the free confining chamber gas, and the free gas in the cleat and fracture systems due to the flooded confined gas. At this short duration saturation level (about five days) it is difficult to predict accurately the composition of both adsorbed and free gases when released. Commonly, at high pressures the gas composition measured at the GC inlet is mostly from both free gases that pass through the coal sample. Most of the desorbed gas (from openings and matrices) is likely to be measured at low pressure as it takes a longer time to be released.

Figure 7 shows the decrease in both parallel and perpendicular strains to bedding planes, which is a clear indication that the coal sample volumetrically shrank as the coal sample starts to desorb its gas. The slope profile of the coal strain perpendicular to bedding decreases almost linearly due basically to the gas desorption. This was not the case of the strain parallel to bedding, which was increased while the confining pressure was reduced. Furthermore decreases in the axial load, confining pressure, and perpendicular strain gave a clear indication that gas was desorbing from coal. The changes in both perpendicular and parallel strains are due to the Poisson effect. The axial load falls because of the gas desorption which allows the coal sample to regain its initial structural shape. It is likely that coal shrinkage will occur mostly at the macro porosity level and very little at micro pores level. These volumetric changes occur particularly during the first 30 min of gas pressure drop, and until the confining pressure drops from around 3 000 kPa down to 500 kPa, which is the point where the parallel strain in desorption is maximum. After this point, the matrix desorption is significant with the confining pressure almost zero. Thus both perpendicular and parallel strains decrease due to the coal shrinkage mainly at molecular structure level.

### TABLE 1 - Gas composition in desorption (Test 1).

<table>
<thead>
<tr>
<th>% Desorbed</th>
<th>Period</th>
<th>Status</th>
<th>CO2/CH4 ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>CO2</td>
<td>N2</td>
<td>CH4</td>
<td></td>
</tr>
<tr>
<td>48.93%</td>
<td>-</td>
<td>51.09%</td>
<td>Start test</td>
</tr>
<tr>
<td>56.01%</td>
<td>-</td>
<td>43.88%</td>
<td>End test</td>
</tr>
<tr>
<td>14.47%</td>
<td>-</td>
<td>-14.11%</td>
<td>Improvement</td>
</tr>
</tbody>
</table>

The changes in both perpendicular and parallel strains are due to the Poisson effect. The axial load falls because of the gas desorption which allows the coal sample to regain its initial structural shape. It is likely that coal shrinkage will occur mostly at the macro porosity level and very little at micro pores level. These volumetric changes occur particularly during the first 30 min of gas pressure drop, and until the confining pressure drops from around 3 000 kPa down to 500 kPa, which is the point where the parallel strain in desorption is maximum. After this point, the matrix desorption is significant with the confining pressure almost zero. Thus both perpendicular and parallel strains decrease due to the coal shrinkage mainly at molecular structure level.
Figure 8 shows the variations of the flow rate and confining gas pressures decay with sorption time. There is a strong gas pressure dependency on the process of gas movement. For the same reason similar gas behaviour will occur in the coal permeability profile.

Both the flow rate discharge and pressure decay occurred at higher rates particularly during the first 10 min of desorption time where the confining gas pressure dropped almost to a third of its value. This is because the tested coal sample was highly fractured at macro pore level. Similar results were obtained from other samples from the same location. At a pressure of 3.2 MPa, the maximum gas flow through the coal sample was about 2.3 L/min (3.3 m³/D) with maximum cleat permeability measured at around 11.7 mD.

In this part of the study, nitrogen gas was injected into the high pressure chamber housing the initially saturated coal sample used in the study for the purpose of improving the pressure gradient and the gas concentration gradient in the coal, and to examine the displacing influence of \( \text{N}_2 \) on the initially adsorbed gas, which is considered important for methane gas recovery from the coal and for carbon dioxide sequestration. Three gas types were used for the initial saturation phase; they were \( \text{CO}_2/\text{CH}_4 \) (52/48) mixed gas, \( \text{CO}_2 \) and \( \text{CH}_4 \) respectively. The experimental procedure was carried out in two stages, referred as stage A and B. In Stage A, nitrogen gas was injected to a high pressure chamber housing the saturated coal sample with an initially adsorbed gas at a pressure of 3.2 MPa. The injected \( \text{N}_2 \) gas was maintained for a predetermined period until the confining nitrogen gas concentration as measured by the GC was almost 100%. As a result of \( \text{N}_2 \) the initial mixture gas composition was found to be reduced to almost zero.

Stage B commenced when \( \text{N}_2 \) gas ceases injection. The discharged gas (which is part free gas as a confining gas, and part as adsorbed gas) from the saturated coal samples was monitored for the rate of flow and gas composition by flow meters and gas chromatographer as described previously.

Figures 9, 10, 11 and 12 show the results of the \( \text{N}_2 \) injection tests carried out on coal samples initially saturated with different gases of \( \text{CO}_2/\text{CH}_4 \) mixed, \( \text{CO}_2 \) and \( \text{CH}_4 \) respectively. The coal sample saturation periods in different initially charged gases were different. Thus, the duration period for each of mixed \( \text{CO}_2/\text{CH}_4 \), \( \text{CO}_2 \) and \( \text{CH}_4 \) gases were 5.3, 10.8 and 4.0 days respectively. These periods of initial charging were carried out just prior to the beginning of nitrogen injection.
Figure 9 - Variation in gas composition and pressure in Test 3

As seen in Figure 9 (Test 3), as soon as N₂ was injected into the gas chamber in Stage A, its composition began to increase sharply. The rate of N₂ increases occurred at the expense of the mixed gas CO₂/CH₄ and the near zero reduction in the composition of the CO₂/CH₄ mixed gas occurred after 30 min of N₂ injection. At the same time the latter gas concentration increased to almost 100%. The increased rate of mixed gas dilution was due to the large volume of N₂ gas injection into the chamber at high gas pressure. The rate of the mixed gas decline was almost an opposite mirror image of the injected N₂ gas increase, and as a result the combined confined gas pressure remained constant at 3.2 MPa. It is interesting to note that the mixed gas dilution ratio measured in every four min during the first 90 min of the N₂ injection in Stage A was smaller than the initial 52/48 mixture ratio as shown in Figure 10. This is because CO₂ gas was predominantly adsorbed in coal during the early stage of the saturation. Note that the released gas was mainly the free gas -confining gas or gas located on the macro porse. Once N₂ gas injection was stopped, the mixed gas began to gradually reappear.

When N₂ gas injection was stopped, it marked the beginning of stage B. The confining gas pressure dropped quickly, reaching almost zero after some 40 min of injection stoppage. During the same period the mixed gas composition level began to increase and some 20 min later the proportion of the discharged mixed gas component began to diverge with the rate of CO₂ gas discharged reaching almost the double that of CH₄. The CH₄ composition stabilised at 20 %, after some 30 min, while the CO₂ increased to 40%. The CO₂ desorption increases by almost 30% which was the double that those obtained in Test 1 (without N₂ injection). Table 2 summarises the results of N₂ injection.

Since the coal still retained some gas in its matrices, it is likely that these ratios could change, over longer periods of testing, especially for N₂ and CO₂. It is worth remembering that in stage A the gases measured at the GC were mainly from free gases with a small amount being from desorbed gases, while in stage B it is supposed that the measured compositions were mainly from desorbed gases.

From Figures 9 it can be inferred that the CH₄ composition is due to molecules desorbing at a steady rate from the coal matrices. However, the CO₂ composition profile is more likely to be due to molecules desorbing from openings- cleats, cracks and fissures- which is the expected location for the adsorbed gas in a short period of adsorption time. The N₂ desorption profile is mostly due to the free molecules passing through the core sample and is decreasing with pressure. In summary; N₂ displaces CO₂ basically due to the improvement of the concentration gradient, and the location of the adsorbed molecules. However, CH₄ is apparently less affected by N₂ injection for the same reasons. In a mixed gas adsorption, most of the CH₄ is adsorbed in coal matrices, while the CO₂ component is adsorbed mostly in the macro pores with a small amount being stored in coal matrices because of the so called CO₂ affinity to coal. Thus CO₂ prevails over methane in obtaining the sweetest spots available on the macro pores.
Table 2 - Gas composition in desorption (Test 3)

<table>
<thead>
<tr>
<th>% Desorbed</th>
<th>Period</th>
<th>Status</th>
<th>CO2/CH4 ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>CO2</td>
<td>CH4</td>
<td>N2</td>
<td></td>
</tr>
<tr>
<td>44.13%</td>
<td>42.64%</td>
<td>13.23%</td>
<td>1.03</td>
</tr>
<tr>
<td>50.86%</td>
<td>49.14%</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td>39.74%</td>
<td>19.50%</td>
<td>40.75%</td>
<td></td>
</tr>
<tr>
<td>67.08%</td>
<td>-33.02%</td>
<td>Free N2</td>
<td>2.04</td>
</tr>
</tbody>
</table>

31.90% Improvement

Figure 10 - Variation in gas concentration without the contribution of N2 in Test 3

Figures 11 and 12 show the results of similar tests carried out separately with each of CO2 (Test 4) and CH4 gases (Test 5) respectively. As seen in Figure 11, N2 was injected for about 110 min when full N2 concentration was achieved. During the second phase (stage B), starting from 15 710 min, the CO2 concentration began to increase sharply, from 0 to 50% in 20 min, and at the same rate that N2 concentration dropped. Thus 50% concentration point was achieved at the 15 730 min mark. The level of CO2 concentration of the out-flowing gas passing through the GC approached 75% after 215 min of GC analysis and testing, which suggests that the injection of N2 gas into coal appears to have a significant influence on CO2 displacement and removal from coal. This level of N2 influence on CO2 is almost four fold greater than for CH4. During the early phase of stage B, the discharged N2 gas was mostly free confining gas. Later, after the 15 800 min, the small amount of nitrogen released appears to be an adsorbed gas. Also any amount of the released carbon dioxide is more likely to be a desorbed gas because the initial pure CO2 free confining gas was mostly diluted during the N2 injection process (Stage A) and passed through the coal sample.

As can be seen from Figure 12 the removal of CH4 was at best 20% which was achieved after 90 min of testing. Note that this methane composition was almost the same as that measured in Test 3. Hence the application of N2 for methane recovery appears to be not a viable method in the current laboratory environment conditions. This finding has a significant bearing in solving the drainage difficulties experienced at West Cliff Colliery Panel 500 area, which has difficulty to drain CO2 gas concentration zones.
CONCLUSIONS

The following were inferred from the experimental studies which were conducted under strict environmental and laboratory conditions;

1. Nitrogen appears to displace both carbon dioxide and methane; however the degree of displacement varies according to the gas type. N₂ gas appears to displace CO₂ more than CH₄.
2. Coal appears to adsorb greater CO\textsubscript{2} than CH\textsubscript{4} in a mixed gas adsorption. However, longer saturation duration improves CH\textsubscript{4} adsorption until it equals the CO\textsubscript{2} level.

3. Injection of N\textsubscript{2} gas resulted in changes in coal volume. These changes occur both perpendicular and parallel to coal layering/bedding. However, and based on the results of this study, coal swelling occurred only axially when N\textsubscript{2} injection was stopped.

4. Coal shrinkage or swelling is attributed to the nature of gas sorption in coal joints, fissures and cracks as well as in the coal matrix.

5. On the basis of this study, N\textsubscript{2} injection has greater effect on CO\textsubscript{2} removal than on CH\textsubscript{4} removal. This finding has significant bearing in solving the drainage difficulties experienced in Panel 500 areas at West Cliff Colliery, which has with difficult to drain CO\textsubscript{2} gas concentration zones.

ACKNOWLEDGEMENTS

Kemal Bařiš was a visiting fellow to the School of Civil, Mining and Environmental Engineering, University of Wollongong. He was a recipient of a TUBITAK scholarship, which enabled him to come to Australia and participate in research on mine safety.

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IMPACT OF COAL PROPERTIES AND OPERATIONAL FACTORS ON MINE GAS DRAINAGE

Dennis Black1 and Naj Aziz1

ABSTRACT: Many Australian underground coal mines have or are likely to encounter areas of increased gas content, which are difficult to drain. A number of factors have the potential to impact the overall efficiency and effectiveness of gas drainage from the mined coal seam. A mine based investigation was undertaken at an operating coal mine working in the gassy Bulli seam of Australia's Illawarra coal measures. Gas production data from 279 inseam gas drainage boreholes was evaluated relative to a variety of coal properties and mine operational factors to determine the impact of each on gas production performance. Whilst the design of the boreholes and the drainage time had some impact on gas production it was the coal properties that had the most impact. Coal rank, ash content, gas content, seam thickness and gas composition were all found to impact gas production. In particular, total gas in place and degree of saturation had the most significant impact on coal seam gas production performance. Various recommendations are suggested.

INTRODUCTION

Many Australian underground coal mines have, or are likely to encounter areas of increased gas content which are difficult to drain. Such conditions are quite common in mines working the Bulli seam and the impacts range from increased gas drainage drilling expenditure to coal production delays and in the extreme cases loss of coal reserves.

This study, undertaken at a Bulli seam mine, is made possible through the availability of gas production data, recorded regularly throughout the production life of the underground to inseam (UIS) boreholes used to drain gas from the coal seam ahead of mining. Of the many hundred UIS boreholes drilled throughout the mine 279 were deemed appropriate for inclusion in this study. Boreholes that had obvious interaction with adjacent boreholes were excluded from the dataset along with boreholes reported to have experienced problems, such as borehole collapse, and those regularly reported to be full of water. Figure 1(A) shows the location of the boreholes relative to the mine workings. The UIS boreholes are drilled in a fan pattern from a dedicated drilling stub. The 279 boreholes included in this analysis were drilled from 34 separate drill stubs. The mine experienced increased difficulty in draining gas from the inbye ends of the panels, shown as the left side of the figure. The mine’s response to the poor drainage performance was to increase the drilling density, with the toe spacing between boreholes reducing from approximately 25 m, at the start of the panels, to less than 12 m in the inbye zones.

It was generally accepted at the mine that the poor gas drainage was the result of increasing CO₂ seam gas composition. Using gas composition data derived from exploration core sample analysis a contour plot was prepared to illustrate the change in CH₄ to CO₂ ratio along the length of the panels. Figure 1(B) shows the change in gas composition from CH₄ rich at the panel entry to CO₂ rich at the inbye end of the panels.

Table 1 lists the various operational factors and coal properties analysed to determine their respective impact on coal seam gas production performance. The variables have been divided into two groups based on the mine operator’s degree of control and ability to influence and control each to improve gas production performance. Within the bounds of the mine and equipment design constraints the mine operator has scope to vary the operational factors whereas the coal properties are, for the most part, the result of coalification and subsequent geological changes and therefore not able to be altered by the mine operator.

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Table 1 – Operational factors and coal properties included in the gas production analysis

<table>
<thead>
<tr>
<th>Operational Factors</th>
<th>Coal Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole length</td>
<td>Carbon content</td>
</tr>
<tr>
<td>Borehole diameter</td>
<td>Volatile matter</td>
</tr>
<tr>
<td>Drilling density</td>
<td>Vitrinite reflectance</td>
</tr>
<tr>
<td>Orientation to cleat</td>
<td>Inertinite/Vitrinite content</td>
</tr>
<tr>
<td>Drainage time</td>
<td>Mineral matter</td>
</tr>
<tr>
<td>Suction pressure</td>
<td>Seam/Coal ash content</td>
</tr>
<tr>
<td>Apparent dip</td>
<td>Inherent moisture content</td>
</tr>
<tr>
<td>Stress</td>
<td>Seam thickness</td>
</tr>
</tbody>
</table>

GAS PRODUCTION

There is a steady decrease in the total volume and rate of gas production from UIS drilling along the length of the panels. Figure 2 shows the average total gas production (m³) from boreholes drilled from each of the 34 drill stubs relative to the position of the stubs along the length of each panel. Both figures confirm a consistent decline in gas production with distance into the panels.

Figure 2 – Total drill stub gas production relative to panel drill stub location

Figure 3 shows the total gas production from each of the 279 boreholes which, in addition to the high degree of variability, highlight the significant number of boreholes that achieved low production, nearly 50% achieving less than 100,000 m³ total gas production. In order to account for the effects of initial gas emission rate the total gas volume produced within the first 50 days (D50) of the borehole productive life was also assessed. The gas produced at D50 was also found to decrease with distance into the panels.
Review of borehole production records provided insight into factors contributing to the low production from many boreholes. Regular problems with UIS boreholes were reported which included; ‘borehole blocked’, ‘borehole full of water’ and ‘no suction’. Separate investigation into gas drainage system performance at Bulli seam Collieries (Black, 2007) found sections of the gas drainage pipe network were adversely impacted by accumulations of water and coal fines. Such accumulations lead to blockages and generally increase system resistance, thereby reducing production capacity.

UIS drilling was found to be a significant source of water and coal fines. Although water and fines management had been available at the collar of the borehole, it was found that, due to the design of the drilling pattern, some interaction existed between the boreholes, particularly within the initial ten metres where the boreholes are closely spaced. This interaction allowed drill fluid, along with coal fines, to flow into adjacent boreholes. If connected to the gas drainage system, water/fines would flow directly into the gas reticulation pipe network.

Although it is necessary to maintain effective water drop-out systems throughout the gas reticulation pipe network, every effort should be directed toward maintaining boreholes free from accumulations and preventing water/fines entering the network. Where system health is regularly monitored and maintained it was considered reasonable to expect many, if not all, boreholes to produce above 100 000 m$^3$.

Given the estimated cost an installed UIS drainage borehole is in the order of $20 000 there is a potentially significant benefit available to the mine operator through increasing the gas production from every gas drainage borehole metre drilled. Improving drainage effectiveness (m$^3$/S) also aids in avoiding potentially significant financial penalty associated with unnecessary drilling expense, production delays and potential loss of reserves, as well as greenhouse gas emission costs.

### OPERATIONAL FACTORS

Of the operational factors considered, borehole length, diameter, drilling density and applied suction pressure were found to have little impact on gas production.

The impact of borehole trajectory on gas production did not initially indicate a relationship between gas production and each of borehole orientation relative to cleat, stress and seam dip, as shown in Figures 4(A), 4(B) and 4(C) respectively. There was however zones within each where increased maximum gas production are evident. The zones of increased maximum gas production exist, as indicated in Figure 4, where boreholes are oriented between 5 and 60° to the dominant cleat (100/280°), 0 and 40° to the principal horizontal stress (075/255°), and have an apparent dip between 0 to +3.0°. Considering the potential benefit of maintaining borehole trajectory within the recommended limits the average total gas production increased 31%, from 151 500 m$^3$ (n=279) to 198 600 m$^3$ (n=107). This was 63% greater than the average 122 100 m$^3$ production of the boreholes outside this range (n=172). Also, considering the potential added benefits associated with borehole and system maintenance, the average total gas production could be further increased. Excluding the sub-100 000 m$^3$ boresholes the...
average total gas production of the boreholes falling within the recommended trajectory range increases by a further 54%, to 280 000 m³ (n=71).

Time on suction was found to have the greatest impact on total gas production. The drainage time provided to the boreholes within this mining area, shown in Figure 5(A), ranged from as little as one week through to one year, with an average of 157 days. It is worth noting that 25% of the boreholes have an effective drainage life of less than 100 days.

Figure 5(B) shows the relationship between total gas production and drainage time which indicates increasing gas production in response to increased drainage time. Further assessment of this relationship, considering the boreholes located within each of four 1 000 m zones along the length of the panels, shown in Figure 5(C), supports the relationship. It should also be noted that the boreholes located inbye of 30 c/t achieve low production where drainage time was less than 300 days.

In general gas production increases in response to increasing drainage time and at least 100 days should be provided in order to achieve reasonable gas production. In the case of the slower draining coal, located inbye of 30 c/t, significantly longer drainage time is required.

Therefore in addition to regular monitoring and maintenance of the gas drainage boreholes, and the overall gas drainage system, the mine operator has the ability to improve the effectiveness of the gas drainage program, thereby producing more gas per drilled metre, through controlling borehole trajectory to remain within the identified limits and increasing drainage time, particularly in the case of the slower drainage boreholes located inbye of 30 c/t.
COAL PROPERTIES

The rank of the coal within the Bulli seam at this mine is classified as medium volatile bituminous, with the indicators of carbon content, volatile matter and vitrinite reflectance ranging in value from 67.3 to 70.8% (69.0% average), 20.1 to 23.5% (21.7% average), and 1.26 to 1.32% (1.29% average) respectively.

Figure 6(A) shows increasing gas production corresponding to increasing coal rank. However, the higher rank coal happened to be located at the outbye, most productive area of the mine. Further analysis of the data within the four 1 000 m zones, along the length of the panels, shown in Figure 6(B), adds support to a relationship, albeit weak, between gas production and coal rank.

Petrographic analysis of 90 coal samples, sourced from within the mining area, was used to determine the distribution of maceral type and mineral matter. The average inertinite content was found to be 55.4%, with the range extending from a low of 47.0%, at the panel entry, to a high of 61.5% at the inbye end of the panel.

The mineral matter content was found to be variable throughout the mining area, averaging 3.3%, with a range of 2.4 to 4.6%. No relationship was found between total gas production and mineral matter content. From this analysis coal type is not considered to have a notable impact on gas production within this mining area.

Ash analysis of 94 coal samples was conducted at an independent laboratory to determine raw ash, through density separation, and coal ash, through proximate analysis. The raw ash, herein referred to as seam ash, ranged from 10.5 to 14.0%, with an average of 12.2%. The coal ash content was found to range between 8.3 and 10.7%, with an average of 9.7%. In both cases there was evidence of decreasing total gas production in response to increasing ash content.

Figure 7(A) shows the relationship between total gas production and coal ash content of the complete data set. Figure 7(B) shows the relationship between total gas production and coal ash content within each of the four cut-through zones along the panels.

Inherent moisture content data was available from proximate analysis testing on 91 coal samples sourced from within the mining area. The results were used to assess the impact of inherent moisture on the total gas production performance within the area. The average inherent moisture content was 0.9%, with a range of 0.8 to 1.0%. Within the data available no relationship was found to exist between total gas production and inherent moisture content.

The average thickness of the Bulli seam within this mining area was found to be 2.6 m, with the range extending from 2.3 to 2.9 m. Figure 8(A) indicates increasing gas production associated with increasing coal seam thickness. However the thicker coal is located at the outbye part of the mining area which has a higher gas production rate. Figure 8(B), shows the gas production and seam thickness within each of four cut-through zones, further supporting the relationship between gas production and coal seam thickness.
Within the mining area the gas composition (CH$_4$:CO$_2$ ratio) spans a broad range, from a low of 13% to a high of 98%, whilst the gas content span a much narrower range, from a low of 7.5 m$^3$/t to a high of 15.5 m$^3$/t. The relationship between gas composition and gas content, Figures 9(A) and 9(B), show the panel entry to be CH$_4$ rich with relatively high gas content, decreasing in both gas content and CH$_4$ composition with distance into the panels.
Figure 10(A) indicates increasing gas production in response to increased gas content. However, as shown in Figure 10(B), the relationship between gas production and gas content, within each of the four cut-through zones, is not particularly strong suggesting other factors may be impacting gas production.

Figure 11(A) shows that as the methane gas composition decreases there was a fall in average gas production, with an increase in the number of boreholes achieving very low total gas production. Figure 11(B) shows gas production relative to composition of the coal seam gas within each of the four cut-through zones. The data suggest a relationship between the two variables, independent of location. Further assessment of the data, divided into three groups on the basis of gas composition, found that gas production was again positively impacted by time on suction and to a lesser extent by gas content. Therefore the data suggests that, within this mining area, gas composition has impact on gas production.

Figure 12(A) shows the strong relationship that exists between total gas production and total GIP for each of the 34 drill sites. From the data shown it has been determined that on average 32% of the total GIP was removed from the coal through gas drainage. Figure 12(B) shows the distribution of gas production relative to GIP within each of the four cut-through zones. The positive relationship between total production and GIP is maintained in each zone along with evidence of the consistent decrease in total gas production with distance into the panels. Figure 12(C) shows the relationship between total production and GIP, grouped on the basis of gas composition. From the data presented, not only can it be seen that gas production increases in response to increasing GIP, within this mining area the more productive CH₄ rich zones have much greater total GIP than the CO₂ rich zones.
A coal holding the maximum possible amount of gas at current reservoir pressure and temperature conditions is said to be ‘saturated’, whereas a coal holding less than the theoretical maximum is referred to as ‘undersaturated’. The most successful coalbed methane production occurs in fields that are close to fully saturated (Lamarre, 2007). Slightly undersaturated coals behave similar to saturated coals with only a short delay prior to first gas production followed by a steady, strong, rising gas production rate. Deeply undersaturated coals behave quite differently and require extensive dewatering prior to initiation of gas production. In deeply undersaturated coal the critical desorption pressure, which is the pressure at which consistent gas production can be expected, is significantly less than the initial reservoir pressure and requires extensive dewatering prior to initiation of gas production. The result of the long dewatering (depressurising) period is that the peak gas production rate can be significantly less than that of an equivalent saturated coal.

Figure 13 shows a typical Bulli seam in situ gas condition and the relative saturation and difference in pressure reduction required to reach the respective critical desorption pressure for both CO₂ and CH₄ rich seam gas areas.

In a study of the economic impact of gas saturation on coals in the United States (Seidle and O’Connor, 2007) determined that as coal became less saturated, the gas production profile weakened exhibiting a longer dewatering time and lower peak production rate. Compared to a fully saturated coal, a coal that was 60% undersaturated required five times as long to reach the peak gas production rate and the magnitude was one sixth that of the saturated coal. Gas saturation is therefore an important coal property and its impact on gas production must be considered.
The degree of saturation (DoS) used in this analysis represents the ratio of measured to saturated gas content (Equation 1). The measured gas content ($V_{\text{meas}}$) is determined using the method described in Australian Standard AS3980:1999. The saturated gas content ($V_{\text{sat}}$) is calculated using the modified Langmuir equation (Equation 2), which requires prior knowledge of the Langmuir constants of volume ($V_L$) and pressure ($P_L$), determined during gas adsorption testing, and the initial reservoir pressure ($P_i$), determined through the use of pressure measuring devices, such as piezometers.

$$DoS = \frac{V_{\text{meas}}}{V_{\text{sat}}} \cdot 100 \text{ (\%)}$$

where:

- $DoS$ = degree of saturation (\%)
- $V_{\text{meas}}$ = measured gas content (m$^3$/t)
- $V_{\text{sat}}$ = saturated gas content (m$^3$/t)

$$V_{\text{sat}} = V_L \cdot \frac{P_i}{P_i + P_L} \text{ (m}^3/\text{t)}$$

where:

- $V_{\text{sat}}$ = saturated gas content (m$^3$/t)
- $V_L$ = Langmuir volume constant (m$^3$/t)
- $P_i$ = initial reservoir pressure (kPa)
- $P_L$ = Langmuir pressure constant (kPa)

The Langmuir equation can also be used to determine the critical desorption pressure ($P_d$) corresponding to a given measured gas content (Equation 3) and therefore the reservoir pressure reduction ($P_i - P_d$) required to reach the critical desorption point.

$$P_d = P_L \cdot \frac{V_{\text{meas}}}{V_L - V_{\text{meas}}} \text{ (kPa)}$$

where:

- $P_d$ = critical desorption pressure (kPa)
- $P_L$ = Langmuir pressure constant (kPa)
- $V_L$ = Langmuir volume constant (m$^3$/t)
- $V_{\text{meas}}$ = measured gas content (m$^3$/t)

Piezometers installed into the Bulli seam were used to record seam pressure changes in response to advancing mine working and gas drainage. Data was collected from 18 piezometers over an 11 month period from December 2006 to October 2007 and consolidated to provide a monthly average pressure response for each piezometer location. A contour plot was prepared to show the pressure distribution for each of the 11 months, providing valuable insight into hydrostatic pressure change and impact of both mine workings and gas drainage drilling. Figure 14(A) and 14(B) show the change in hydrostatic pressure over the seven month period, between February 2007 and September 2007, respectively.

Of particular significance is the fact that the hydrostatic pressure within the coal seam at the time of roadway development is approximately 1 000 kPa and appears to reduce at a slower rate from the inbye parts of the mine. Using the previous example of CO$_2$ rich coal with an *in situ* gas content of 10.5 m$^3$/t the critical desorption point was 570 kPa. In this case, where the hydrostatic pressure does not fall below 1 000 kPa during the life of the gas drainage program the reservoir pressure is at least 430 kPa above the critical desorption pressure which impedes gas desorption. The fact that the reservoir pressure exceeds the critical desorption pressure validates the low gas production from the inbye part of the mining area and highlights the need for significantly increased drainage time in these deeply undersaturated areas.
From the hydrostatic pressure contours shown above, it can also be seen that within the drilling range of the UIS gas drainage boreholes the pressure is typically no greater than 2 500 kPa at the time of drilling. This result is consistent with the findings of Marshall et al. (1982), who recorded a maximum gas pressure of 2 670kPa at a distance of 40 m in a UIS borehole drilled in the Bulli seam. The gas content measured prior to, or during UIS drilling, within each of three zones along the length of the panels, plotted at an initial reservoir pressure of 2.5 MPa, are shown in Figures 15(A), 15(B) and 15(C). The figures show the decrease in DoS from the slightly undersaturated outbye, CH4 rich zone, through to the deeply undersaturated inbye, CO2 rich zone.

The relationship between DoS and total gas production from all UIS boreholes in each of the 34 drill sites is shown in Figure 16(A) which indicates a positive relationship. However, given the variable number of UIS boreholes drilled from each drill site, the relationship between total gas production and the unit gas production rate (m3/m), shown in Figure 16(B), was also considered. The results indicate a strong relationship between gas production and DoS.

CONCLUSIONS AND RECOMMENDATIONS

This detailed analysis of the relationship between total gas production and various geological properties and operational factors has provided insight into the complex interactions that exist within this mining area. It was found that a significant portion of the UIS drilling effort yields little benefit. Of the 279 boreholes analysed, 124 (45%) achieved less than 100 000 m3 total gas production. A typical operational response to such low production is to drill additional boreholes, often allowing only a short drainage window.
At an estimated cost of $20,000 for each installed UIS drainage borehole, it makes good sense to improve the gas drainage effectiveness (m³/$) of every metre drilled. Improving drainage effectiveness also aids in avoiding potentially significant financial penalties associated with unnecessary drilling expense, production delays and potential loss of reserves, as well as greenhouse gas emission costs.

Gas production was found to positively correlate with coal properties such as rank, ash content and total gas in place, which represents the combination of gas content and volume of coal being drained. However, degree of saturation was found to have the closest and most significant relationship to gas production.

Of all the operating factors considered, time on suction was shown to have the most significant impact on total gas production. The drainage time ranged from one week to one year, with almost 25% of the 279 boreholes analysed having a drainage time of less than 100 days. It has been shown that potentially significant drainage time is required in order for the seam pressure to be reduced to the critical desorption point, particularly in the deeply undersaturated, CO₂ rich zones. The nature of UIS drilling requires it to be in close proximity to mining activity which affords quite a short effective drainage window. Where the degree of saturation is less than 50%, the drainage time required far exceeds the drainage window available through the use of UIS drilling. In such areas supplementary surface-based gas drainage methods should be used and provided upward of five years drainage time rather than five months.

Analysis of the factors able to be controlled by the mine operator demonstrated generally low impact on total gas production. However, the results suggest that increased production may be achieved through maintaining UIS borehole orientation within an identified ‘optimum’ range. Increased gas production was achieved from boreholes oriented between 5° and 60° to the dominant cleat, 0° and 40° to the principal horizontal stress, and drilled up-dip at an apparent dip between 0° and +3.0°. The 107 boreholes within the assumed optimum range achieved 198,600 m³ average total gas production, 63% greater than the average production of the boreholes outside this range, and 31% greater than the average production of the total dataset.

Analysis of applied suction pressure highlighted variability in suction pressure applied to the boreholes throughout their productive life. There was also evidence of a reduction in applied suction pressure with distance into the panel supportive of increasing resistance. Separate studies attributed the increase in resistance to accumulations of water and coal fines within the boreholes and gas reticulation network. Although not analysed in this study, it is accepted that maintaining system health has a potentially significant impact on gas production capability through avoiding conditions such as, ‘borehole blocked’, ‘borehole full of water’ and ‘no suction’. In order to maintain the efficiency and overall production capacity, the gas drainage system should be supported by effective design and ongoing maintenance.

Through taking action to eliminate the poor producing boreholes, by way of increased drainage time and maintaining the health of the UIS boreholes and broader gas drainage system, significant gas production improvement can be expected. In this case, the average total gas production of the 155
boreholes whose gas production exceeded 100,000 m$^3$ was 59\% greater than the average production of the total dataset.

Combining optimum borehole trajectory and increased drainage time with regular monitoring and ongoing maintenance of the UIS borehole and gas reticulation system would further increase the expected gas production. From the boreholes analysed in this study, 71 lie within the optimum trajectory range and achieved above 100,000 m$^3$. The average total gas production achieved by these boreholes was 280,000 m$^3$, 85\% greater than the average production of the total dataset.

**ACKNOWLEDGEMENTS**

The authors wish to acknowledge the assistance of Mr. M. Jurak and Miss K. Lennox in preparing the data used in this analysis.

The support of the Australian Coal Association Research Program (ACARP), through project C18004, is also acknowledged and appreciated.

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MINING GAS INITIATIVE NORTH RHINE-WESTPHALIA –
PART OF ENERGY AGENCY.NRW
PLATFORM FOR ECONOMY, ENGINEERING AND RESEARCH

Axel Preusse¹ and Joerg Kraemer¹

ABSTRACT: An overview of the Mine Gas Initiative NRW, part of the EnergyAgency.NRW in North Rhine-Westphalia (NRW), Germany, is presented. It covers the role of the initiative in areas relating to energy production from mine gas, enhancement of mine security through utilisation of mine gas and enforcement of climate protection by avoiding methane emissions into the atmosphere. Achieved objectives and future developments will be presented.

INTRODUCTION

Motivation: In an effort to utilize methane is based on mainly three reasons. Firstly, methane is an energy source comparable to natural gas, with an energy content of 13.0 kWh per kg. Secondly, methane remains a safety risk, especially to underground coal mining. And thirdly, methane is 21 times more harmful to the climate than CO₂, which makes its utilization a priority for environmental protection. Additionally, profits can be generated from certificates according to the Clean Development Mechanisms of the Kyoto-Protocol.

EnergyAgency.NRW: Since 2007, the EnergyAgency.NRW (in German: EnergieAgentur.NRW) is the joint umbrella organisation of the former NRW Energy Agency and the NRW State Initiative On Future Energies. This merger has created a strategic platform with wide-ranging competence in the energy domain from research funding, technical development, demonstration and market launch to energy consultancy and continuous vocational training.

The EnergyAgency.NRW offers companies in North Rhine-Westphalia, by means of competence networks, platforms for strategic alliances. Furthermore, energy consultancy services are provided in the form of administrative and contract services as well as information and training services for specialists and private households. The agency also present seminars to people using electric and electronic devices and facilities raising awareness of potential savings from the rational use of energy.

In the field of renewable energies great efforts are made to advance technological innovations and to forge ahead with knowledge between science and industry. Companies from NRW are supported in specific matters relating to foreign trade. Overall EnergyAgency.NRW acts as a central contact point for all matters related to the subject energy.

Clean Development Mechanisms: Given the global warming potential of methane is about 21 times greater than carbon dioxide, the mechanisms of the Kyoto-Protocol gain increasing importance concerning the utilization of mine gas. Especially in countries that do not have a supporting framework like the Renewable-Energy-Law in Germany (in German: Erneuerbare-Energien-Gesetz, EEG), the economical realization of mine gas projects can be made possible by additionally gaining revenue from emission trading.

CLASSIFICATION OF COAL BOUND METHANE

Common definitions in Germany divide coal bound methane into two main groups (Figure 1). The first group – Coal Mine Methane (CMM) – describes methane in combination with mining activities – current as well as past. To further specify different types of origin, other terms are common. Coal Seam Methane (CSM) describes methane in active coal mines. To provide safety for the workers, some of this gas is extracted prior to mining and by ventilation during mining operations. The methane extracted from the mine may be utilized on site. The gas from mine ventilation, known as Ventilation Air Methane (VAM), is considered too dilute for utilization. The term Abandoned Mine Methane (AMM) describes gas

¹ Mine Gas Initiative NRW, Institute for Mine Surveying, Aachen University
extracted from abandoned mines via shafts or boreholes. This gas may be used for on site power and heat production.

**Figure 1 - Definitions**

According to German legislation gas exploration and production is regulated separately to mining (i.e. in abandoned and virgin coalfields) requiring separate gas concession. Following the introduction of the Renewable-Energy-Law in 2000 there was a rush on gas licenses and at present almost 94 exploration and production licenses have been granted in North Rhine-Westphalia (Figure 2). According to provisional figures, 67 mining authorisations for mine gas utilisation had been awarded at the end of 2008.

**Figure 2 - 67 Mine Gas concessions for gas utilization from abandoned coal mines in North Rhine-Westphalia (27 concessions for exploration not shown) 
(Source: Bezirksregierung Arnsberg NRW Abt. 6)**

Figures 3 and 4 summarise technologies, installed capacity and production capacity along with current research involving active and abandoned mines as well as current CBM-operations in North Rhine-Westphalia (Figure 5).

At the end of 2008 the provisional figures indicate that there were 128 unit-type co-generation power plant modules as well as a turbine set in service with electrical capacity totalling 170 megawatt. The 843 million kWh of electrical power generated in 2008 and the heat supply of 79 million kWh are capable of supplying about 220,000 households. The total mine gas potential exploited resulted in a CO₂-reduction of about 3.7 million tons in 2008.
The second group of mine gas, known as Coal Bed Methane (CBM), refers to methane gas bound in virgin hard coal deposits. Due to strata composition, relatively deep coal layers and very moderate permeabilities in the German hard coal deposits, extraction of CBM has failed up to now, but is still a topic of discussion and research (see below).

**RESEARCH AND DEVELOPMENT (R&D)**

**CMM**

Apart from the conventional concept of utilization by power and heat production with co-generation plants, a great amount of research and development is still to be done.
Common aim of these efforts is an improvement in energy yield, a sustainable proceeding concerning the mine gas deposit in the sense of a deposit management and the recovery of new mine gas resources.

**CBM**

In times of rising costs for energy and a run out of resources it seems to be more necessary than ever to look for alternative energy sources. In this context CBM could become an important alternative to conventional natural gas. CBM provides an innovative usage of German hard coal deposits and could help Germany to become more independent from the import of fossil fuels. Due to that fact a multidisciplinary research community is exploring the productability of CBM in North Rhine-Westphalia in the R&D-project “CBM Muensterland” for one and a half year now.

This alliance already forecasted the technical and geological viability of CBM in the Muensterland area. The economical feasibility of CBM production will be assessed in the second phase of the project, which is currently anticipated.

Additional commercial CBM projects are also being considered. Figure 5 gives an overview of current CBM research activities in North Rhine Westphalia.

**Figure 5 - Current CBM research-activities in North Rhine Westphalia**

**MINE GAS INITIATIVE NRW**

Supported by the German Renewable-Energy-Law, utilization of mine gas in North Rhine-Westphalia has significantly increased since the beginning of the year 2000. In 2001, under the chairmanship of first Author, the Mine Gas Initiative NRW was founded as a part of the NRW State Initiative On Future Energies to further support this development. In 2007 the NRW State Initiative On Future Energies and the NRW Energy Agency were combined to form the new EnergyAgency.NRW.

The Mine Gas Initiative NRW supports the development of technology of mine gas extraction and utilisation and to intensify the technology transfer. The Mine Gas Initiative NRW is an interdisciplinary platform for information and communication for persons and groups from companies, governments and institutions of research and development, whose representatives meet regularly. It aims at the utilization of an innovative energy source, the environmental protection by reduction of harmful emissions and the improvement of mine safety. It also tries to introduce emission-trading into planning procedures.
INTERNATIONAL ACTIVITIES

The Mine Gas Initiative NRW provides national and international support for German companies by establishing and improving international contacts (Figure 6). It supports participations in national and international exhibitions. The Mine Gas Initiative NRW is open to all interested persons, groups and organizations. Selected members from the Mine Gas Initiative NRW are already partners or could be possible partners for international collaboration in the fields of mine gas extraction, mine gas utilization and emission trading.

In recent years the Mine Gas Initiative NRW focussed on the PR of China which led to the establishment of a "Frame Agreement between Shanxi-Province, PR China and Mine Gas Initiative NRW on the Topic of the Development of Mine Gas Projects in Shanxi Province..." in September 2009.

Furthermore RWE, as a representative enterprise of North Rhine-Westphalia, began to find cooperative partners in Shanxi province on Clean Development Mechanism projects since October 2008. After a year of negotiation and investigation of coal mine gas, RWE and Shanxi Dubao Energy Development Institute reached co-operation agreement. The intention of RWE and its cooperative partners in Shanxi Province is to establish a joint venture for clean energy and Clean Development Mechanism projects.

In 2008 an agreement between the University of Queensland, Australia, represented by the School of Engineering and the School of Physical Sciences/Earth Sciences, both part of the Energy & Environment Group, and RWTH Aachen University, Germany, represented by the Institute for Mine Surveying, Mining Subsidence Engineering and Geophysics in Mining and the Geological Institute, has been signed.

The Australian and German partners want to extend and intensify scientific co-operation in the field of Coal Bed Methane research and technologies. The co-operation also covers the exchange of expert personnel and will include research activities, development of technologies, information exchange and training.

The agreement falls under the umbrella of the Memorandum of Understanding between the State of North-Rhine Westphalia and the Australian States of Queensland and New South Wales, signed in November 2004. This Memorandum covers - among other things - collaborations concerning renewable energies.
SUMMARY

Mine gas is a new alternative energy source. There are large quantities of methane stored in abandoned and active mines. The extraction of the gas can be accomplished via old shafts or boreholes. From this energy source, electricity as well as heat can be produced. In Germany, mine gas is treated as a mineral resource under federal mining law and almost 70 mine gas concessions have been granted. The development of this new field of activity has shown encouraging results thus far. Interesting future fields are mine gas extraction from virgin coal deposits (CBM) and the implementation of the mechanisms according to the Kyoto-Protocol.

This paper explains the role of the Mine Gas Initiative NRW in North Rhine-Westphalia, Germany, part of the EnergyAgency.NRW (EnergieAgentur.NRW). Concerning mine gas utilization, it covers the motivation, definitions and fields of interest of economy, engineering and research – current and future. It also describes the work of the Mine Gas Initiative NRW, and its activities on all topics related to the utilization of mine gas, such as energy production, enhancement of mine security and enforcement of climate protection.

Hard coal mining in Germany has a long-lasting tradition spanning centuries. In this period huge expertise in dealing with all matters of underground mine gas problems in a safe way and in utilizing mine gas as energy source has been accumulated. To bring this expertise together the Mine Gas Initiative NRW was founded in 2001.

The Mine Gas Initiative NRW is open to interested persons, groups, companies and organizations. Selected members from the Mine Gas Initiative NRW are already partners or could be possible partners for international collaboration in the fields of mine gas extraction, mine gas utilization and emission trading.
COAL MINE GOAF GAS PREDICTOR (CMGGP)

Les Lunarzewski

ABSTRACT: Research has consolidated on the knowledge of gas emission characteristics of coal mine goaf in Australia into a form which enables emission predictions to be made using limited input data. The fundamental principles are encapsulated within the coal mine goaf gas predictor (CMGGP) professional engineering software. This easy to use tool is designed to predict the decline of methane emissions and calculate gas reservoir capacity of underground coal mine goaf. The method for estimating abandoned coal mine and goaf area methane potential is applicable to both dry and flooded mines and also takes into account coal mine methane utilisation options. The software will run on a PC with windows XP or vista operating system and can be used by individual specialists, coal mines and/or other institutions involved in coal mine closure, coal mine methane utilisation and greenhouse gas emissions reduction.

INTRODUCTION

When an underground coal mine ceases coal production methane gas continues to flow into the underground workings through the process of desorption from residual coal within strata disturbed by mining activity. For gassy mines this desorption process will continue for many years after closure and can resume when flooded mine workings are dewatered. The coal mine operator is therefore faced with potential long-term liabilities including explosion risks on the surface and possible dangers to the public as well as continuing greenhouse gas emissions. After stopping the ventilation system, pumping mine water and sealing mine outlets to the atmosphere (shafts, inclines, service boreholes etc.) gas release from goaf areas will continue until underground workings and macro-fractures are flooded.

Responsible operators are now looking at ways to exploit methane from closed/sealed underground coal mines where practical to reduce environmental emissions, minimise public liability, to take advantage of an energy resource that would otherwise be wasted, provide some continuing local employment and to gain added value from the mine before total abandonment.

A practical and scientifically-based prediction tool for assessing the methane production potential of underground coal mines goaf areas ranging in size from a single panel to whole-of-mine is presented.

BACKGROUND

In New South Wales and Queensland there are more than 50 underground gassy coal mines, which have ceased coal mining operations since after 1954 (Lunagas, 2004). Whereas various methods for predicting gas emissions in working mines are available and in regular use, no complete methodology has been established for predicting the decay of emissions once coal production ceases and the coal mine is no longer operating. However, extensive studies on gas emission from sealed goaves and abandoned coal mines have been undertaken by Lunagas Pty Limited using data predominantly from N.S.W & Q.L.D underground coal mines complemented by additional data from UK, Poland, Czech Republic, USA and Japan (Lunagas 2005). Thus, a method for forecasting long-term gas emission decay was developed for use by specialist industry practitioners in which predicted and measured data showed good correspondence.

The future development of coal mine methane utilisation from coal mines goaf in Australia will depend on the availability of suitable sites and their proximity to a customer. Viability will also depend on the ability to predict both the quantity and decline rate of coal mine methane from those mines, to meet customer requirements whilst at the same time generating a profit for the operator. The national and local energy market for gas and electricity use is continually changing and schemes must be flexible to accommodate these changes while maintaining commercial viability and customer satisfaction.

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It is therefore in the interests of coal mine operators to monitor and gather gas emission data from goaf areas to accurately quantify current emissions and to predict future emission for exploitation potential assessment.

**COAL MINE GOAF CLASSIFICATION**

All input data were gathered from five underground gassy coal mines (and their sealed longwalls and districts) located in New South Wales (4) and Queensland (1) and have been classified for coal mines goaf categories-stages related to the mining, hydro-geological and ownership conditions.

**Table 2 - Coal mine goaf classification**

<table>
<thead>
<tr>
<th>Classification</th>
<th>Mine entries &amp; surface boreholes</th>
<th>Ventilation</th>
<th>Coal production</th>
<th>Water pumping</th>
<th>Responsibility</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temporary closed</td>
<td>Opened, not permanently or partially sealed</td>
<td>Operating on reduced capacity</td>
<td>Ceased Possible future production</td>
<td>Optional</td>
<td>Mine operator (maintenance)</td>
<td>Not a factor</td>
</tr>
<tr>
<td>Closed</td>
<td>Partially or fully sealed</td>
<td>Terminated</td>
<td>Ceased No future production</td>
<td>Terminated Goaves gradually flooding</td>
<td>Mine operator</td>
<td></td>
</tr>
<tr>
<td>Decommissioned</td>
<td>Permanently sealed</td>
<td></td>
<td></td>
<td>Terminated Goaves gradually flooding</td>
<td>Transferred from mine operator to the relevant Government Authority</td>
<td>1-20 years</td>
</tr>
<tr>
<td>Abandoned</td>
<td>Permanently sealed</td>
<td></td>
<td></td>
<td>Termination</td>
<td>Mine operator</td>
<td></td>
</tr>
<tr>
<td>Sealed longwall or district</td>
<td></td>
<td></td>
<td></td>
<td>Optional</td>
<td>Mine operator</td>
<td>Not a factor</td>
</tr>
</tbody>
</table>

Note: For the purposes of this work, goaf is defined as the part or total mine from which the coal has been partially or wholly removed and is no longer ventilated to allow access.

**COAL MINE GOAF GAS RESOURCE MODELLING**

Gas is emitted from the worked seam ahead of mining, from unworked coal seams above and below a longwall as a result of the de-stressing caused by coal extraction and from any natural gas reservoirs disturbed by the mining activity (Lunarzewski 2001 & 2007). Once coal production stops, no further new gas release sources are activated but the existing gas sources continue to emit gas at a decreasing rate.

Gas emission from coal seams in the disturbed zone around a longwall occurs as a result of desorption from the coal matrix and pressure flow through the natural and mining induced fractures. Where there are few coal seams above and below the worked seams, methane emissions at mine closure tend to be very low. However, Australian experience is that an exception might occur where there has been extensive room-and-pillar mining, with no pillar recovery, in moderately permeable coal when residual gas in coal pillars can contribute significant gas flows.

Where de-pillaring is practised the situation is analogous to longwall extraction and gas flows from any adjacent seams will add to the emissions.

For the proposed model a specific input data and information have been selected which include the most typical and achievable mining, gas and geological records.

**COAL MINE GAS DECLINE PHENOMENON**

The “dry mine” case refers to the condition where underground water is being controlled by pumping and gas released to the underground workings is either vented to the atmosphere or recovered for methane utilisation purposes. During coal production activities, gas emissions include both;
‘Production gas’ released from the strata as a consequence of mining activities, and 'Background gas' which remains and is released to the goaves during mining activities after stoppage or cessation of coal production.

When mining ceases production gas rapidly declines, however, the background gas declines slowly and stabilises after mine is closed and/or sealed.

This occurs in a two phases of the decline process;
1. Rapid short-term decline phase, and
2. Slow long-term decline phase.

Background gas represents the theoretical maximum emissions from mine goaf, however, gradual reduction of desorption from gas sources can be accelerated under suction. The rapid decline of production gas depends largely on the continuity and rate of mining (longwall retreat) over the period of a few weeks prior to cessation of the mine coal production. The final methane make before decay starts is taken (if recorded) as the average of the last month of coal production. For other cases the last five years average tonnage and methane make (relative gas emission) is used.

**Rapid short-term decline phase**

The initial rapid decline and stabilization phases take place during the first three to twelve months after mine finishes coal extraction. During that period of time the coal mine production gas is substantially reduced to the level of 30%-70% of the final gas make as a consequence of terminating strata relaxation process and continuation of strata reconsolidation phenomenon.

**Slow long-term decline phase**

The background gas long term gas decline phase starts from year one after time of longwall/mine closure and will last up to 15 to 20 (30) years decline limit time'

---

**Figure 6 - Composite methane emission decay from a series of longwalls and the district (coal mine)**

**GAS DECLINE CURVES**

Gas decline curves - ‘Drysim’ and ‘Wetsim’ have been generated using multi longwall excel spreadsheet models for selected underground coal mines with long term and full range data and information (Lunagas 2005). Both curves represent coal mine methane quantity and decline rate.
A new concept has been developed and demonstrated in this paper for wider application in the general case where individual longwall and/or mine input parameters are not known in detail.

The computer simulation program encapsulating the concept uses basic key-data and information available for the mine. This engineering software integrates the exponential function and a proprietary curves fitting routine to derive the exponential decay constants for various mines category.

**’Drysim’ decline curves**

Mathematical formulae for the decline curves of each phase have been developed (Lunagas 2005) using long term empirical results from the selected longwalls and underground coal mines as a relationship between quantities of gas \( Y = \) litres \( \text{CH}_4 \) per second and time \( X = \) months. The formula includes two variable coefficients ‘\( a \)’ - methane emission intensity and in situ gas sources productivity and ‘\( b \)’ - the decline rate with time i.e. speed of decay.

\[
F(x) = a e^{-bx}
\]

Where:
- \( x \) - time (months)
- \( a \) - quantity constant
- \( b \) - decline constant

**’Wetsim’ decline curves**

Two different methods can be used for predicting coal mine gas emission decline rates in a flooding coal mines goaf:

1. Simulation,
2. Flooding time constrained empirical decline curve.

The Wetsim simulation method involves progressively reducing the decaying goaf emission sources contributing to a part or whole of a mine’s gas emission to mirror the isolating effect of rising water (Figure 2).

Data from all extraction panels in a closed mine are required for this method; but such information is not typically available in coal mines so was deemed not to be an appropriate method for use in the software model, however, if available could be calculated manually (Lunarzewski, Creedy, 2006).

**The constrained empirical decline curve**

The constrained decline curve methods make use of the characteristic exponential equation representing the slow long-term decline phase established for dry mine. Total coal mine gas resource is obtained by simulation using stratigraphy and in situ gas content. The basic expressions representing the background gas emission decline curve and the gas reservoir area graph shape which are respectively:

\[
(1) \quad F(x) = a e^{-bx} \quad \text{and} \quad (2) \quad F(x) = a e^{-(a/GR)x}
\]

This neat, simple concept illustrated in Figure 3 yields first order estimates when limited data are available.

The gas emission start and end with time and flow rates are well defined in the flooding case. Gas emission will be zero when the mine is finally flooded and the time when this occurs is estimated from the void and water inflow data.

The final 'Wetsim decline curve' becomes the combination of the above assumptions and the 'Drysim decline curve' mathematical formulae for the nominated conditions i.e. mine category (Lunagas 2005).
The time taken for the workings to flood is calculated using a void volume model and inferred water inflow data. This method is practical, intuitive and seems to yield reasonable estimates and was therefore selected for use in the software model (Lunarzewski and Creedy, 2006).

**EXAMPLES OF CASE STUDIES**

Prediction of coal mine methane emission decline quantity with time is determined by various decline curves which use the empirical coefficients defined for typical underground coal mines and consequently applicable to any coal mine goaf with a similar mining, gassy, geological, and hydrogeological conditions i.e. mine category (Table 2).
Figures 4 and 6 show maps-selected districts of coal mine goaf categories and selected examples of gas emission trend graphs (Figures 5 and 7) for individual longwalls, district and the whole mine.

**Figure 4 - Category I - ‘High gassy mine’ U/G workings**

**Gas emission phases and coal mine categories**

Following cessation of mine coal production the coal mine gas emission (production and background gas make) has been classified into three consecutive phases:

1. Production gas rapid decline,
2. Background gas stabilisation period, and
3. Background gas long-term decline.

The decline curve computes the quantity of coal mine gas (methane make) and its decline rate using the specific constants for selected category of the mine.

**Coefficients**

There are two major constants which classify and characterise the individual coal mine categories: quantity coefficient = ‘a’ and decline rate coefficient = ‘b’
Figure 5 - Category I mine - Longwalls and mine goaves methane emission versus time graph

Figure 6 - Category VI and VII - ‘District and longwall goaves’ U/G workings

Table 2 - Coefficients for various goaf categories

<table>
<thead>
<tr>
<th>Category</th>
<th>Description</th>
<th>Quantity coefficient</th>
<th>Decline rate coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>High gassy</td>
<td>a_i</td>
<td>b_i</td>
</tr>
<tr>
<td>II</td>
<td>Low gassy</td>
<td>a_ii</td>
<td>b_ii</td>
</tr>
<tr>
<td>III</td>
<td>Low permeability</td>
<td>a_iii</td>
<td>b_iii</td>
</tr>
<tr>
<td>IV</td>
<td>Temporary closed</td>
<td>a_iv</td>
<td>b_iv</td>
</tr>
<tr>
<td>V</td>
<td>Sealed district goaves</td>
<td>a_v</td>
<td>b_v</td>
</tr>
<tr>
<td>VI</td>
<td>Room &amp; pillar goaf</td>
<td>a_vi</td>
<td>b_vi</td>
</tr>
<tr>
<td>VII</td>
<td>Sealed longwall goaf</td>
<td>a_vii</td>
<td>b_vii</td>
</tr>
<tr>
<td>VIII</td>
<td>High permeability or shallow mine</td>
<td>a_viii</td>
<td>b_viii</td>
</tr>
</tbody>
</table>
Figure 7 - Category VI and VII- ‘District 1 & individual longwall goaves’ methane emission versus time graphs

**COAL MINE GOAF GAS PREDICTOR SOFTWARE**

The Lunagas Pty Limited Coal Mine Goaf Gas Predictor (CMGGP) is simulation software for predicting the decline rate of gas make and calculating the gas reservoir capacity of coal mine goaves (Lunagas, 2009). The software comprises three main sections.

1. The “Coal mine parameters” in which data are entered to facilitate calculation of the decline curves for both dry and wet mines.
2. “Gas reservoir characteristics” which allows entry of stratigraphical data for calculating the available gas reservoir.
3. “Methane decline curves and gas reservoir” charts (decline curves) showing the results of calculations made based on data in the above two sections.

**Flowchart**

The flowchart (including required input data) shows software application and availability for the nominated coal mine goaves.
Software inputs and outputs

Figure 8 - High gassy mine inputs for decline curves and gas reservoir
REFERENCES

**A NUMERICAL SIMULATOR OF OUTBURSTS OF COAL AND GAS**

Sheng Xue\(^1\), Gang Wang\(^{1,2}\), Yucang Wang\(^1\), and Jun Xie\(^1\)

**ABSTRACT:** An outburst of coal and gas in underground coal mines may occur when stress condition and coal failure combine with rapid gas desorption. A mechanical and fluid coupled numerical simulator, SimBurst, has been developed to simulate the initiation process of the outburst, as a first step to model the whole process of the outburst. This paper describes the simulator and a simple model set up with the simulator to model the initiation of an outburst in roadway excavation to illustrate the methodology and approach with the SimBurst. The model simulated the basic features of an outburst initiation process, including coal deformation, pore pressure and principal stress vector redistribution, and yield and tensile failure zone of coal.

**INTRODUCTION**

An outburst of coal and gas is a major hazard in underground coal mining. It could cause dangers of asphyxiation due to oxygen deficiency, of poisoning by noxious gases, and explosion by inadvertent ignition of the resultant explosive mixtures and of injury due to violence.

A number of theories have been proposed to explain the mechanism of the outburst. These include; cavity theory and the pocket theory (Shepherd *et al.*, 1981), the dynamic theory (Farmer and Pooley, 1967; Shepherd *et al.*, 1981), and the spherical shell destabilization theory (Jiang, 1995). It is generally accepted that mining-induced changes in strata stress and gas pressure cause deformation and failure of coal and the interaction between gas and coal results to a dynamic type of failure or outbursting. More specifically, there are two conditions which must be met for an outburst to occur: coal must be failed under an effective stress; gas must be able to desorp rapidly from the coal and the gas pressure must be large enough to push the failed coal into the mining opening instantaneously. In other words, the occurrence of an outburst is the result of combined effects of stress redistribution, gas desorption, coal property and time. Despite the generalized understanding of the outburst mechanism, quantitative prediction of the outburst is still a challenging issue as current outburst prediction methods are largely based on past practical experiences and empirical data.

Some attempts have been made to numerically model the process of the outburst. These include mainly a phase transformation model (Litwiniszyn, 1985), a gas desorption and flow model (Paterson, 1986), a boundary element model (Barron and Kullmann, 1990), an airway gas flow model (Otuonye and Sheng, 1994), a fracture mechanics model (Odintsev’s, 1997), a simple finite element model (Xu *et al*., 2006) and a plasticity model (Wold *et al*., 2008). Despite these great efforts there is still no single numerical model to simulate the whole process of the outburst. It has proven to be difficult because the outburst is in fact a two-step process (initiation and development) and each step has its own characteristics and requires different approaches. As a first step, a numerical simulator, SimBurst, is developed to model the initiation process of the outburst.

**SIMBURST DESCRIPTIONS**

SimBurst is a three-dimensional numerical simulator developed by CSIRO Earth Science and Resource Engineering. With this simulator, the process of mining-induced stress redistribution, changes of coal/rock permeability and pore pressure, and coal/rock failure (initiation of the outburst) can be simulated to gain a better understanding of outburst mechanism. It can also be used to understand the relative importance of the key contributing factors in the outburst initiation through parametric studies and help determine the threshold values for outburst risk.

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There are two important processes in outburst initiation: coal deformation and failure, and gas desorption and flow. The deformation and failure of coal are modelled with a widely used commercial code for geotechnical analysis of rocks (FLAC3D), and the gas desorption and flow are simulated with a well-established code for fluid flow and gas desorption modelling (COMET3). These two codes are coupled to model the outburst initiation.

**Modeling of the deformation and failure of coal**

Coal is treated as a continuous medium and its mechanical behaviour is numerically modelled as it reaches equilibrium or steady plastic flow. Application of the continuum form of the momentum principle yields the following equations of motion:

\[ \sigma_{y,j} + \rho b_j = \rho \frac{dv_j}{dt} \]  

(1)

Where \( \rho \) is mass per unit volume of the medium; \( b_j \) is body force per unit mass; and \( \frac{dv_j}{dt} \) is material derivative of the velocity.

Equation (1) is solved with a stress-strain law. The incremental stress and strain during a time step is governed by various elastic or elasto-plastic constitutive laws, which can be written in a general form as follows:

\[ \Delta \sigma' = H(\sigma', \dot{\varepsilon} \Delta t) \]  

(2)

Where \( H \) is a given material functions; \( \sigma' \) is the effective stress; \( \dot{\varepsilon} \) is infinitesimal strain-rate tensor and \( \Delta t \) is a time increment.

The effective stress in Equation (2) is related to the total stress in Equation (1) by

\[ \sigma' = \sigma + I \alpha P \]  

(3)

Where \( \alpha \) is Biot's effective stress parameter; \( I \) is the unit tensor; and \( P \) is pore pressure.

**Simulating gas desorption and flow**

Coal is considered as dual-porosity/single-permeability system. Gas diffuses from the discontinuous coal matrix blocks into the continuous cleat system in coal. The basic equations governing fluid flow in the coal cleats (fractures) are mass conservation for gas and water:

\[ \nabla \cdot \left[ b_g M_g (\nabla p_g + \gamma_g \nabla Z) + R_w b_w M_w (\nabla p_w + \gamma_w \nabla Z) \right] + q_m + q_g = (d/dt) (\phi b_g S_g + R_w \phi b_w M_w) \]  

(4)

\[ \nabla \cdot \left[ b_w M_w (\nabla p_w + \gamma_w \nabla Z) \right] + q_w = (d/dt) (\phi b_w S_w) \]  

(5)

Where \( \nabla \) is gradient operator; \( \nabla \cdot \) is divergence operator; subscript \( g \) and \( w \) stand for gas phase and water phase respectively; \( M_g = k k_{rn} / \mu_n \) is phase mobility, where \( k \), \( k_{rn} \), \( \mu_n \) are absolute permeability, relative permeability and viscosity respectively, \( \mu_n \) is viscosity; \( \gamma_n = \rho_n g \) is gas or water gravity gradient, where \( \rho_n \) is phase mass density and \( g \) is gravitational acceleration; \( S_n \) is degree of saturation; \( b_n = 1/B_n \) is gas or water shrinkage factor, where \( B_n \) is formation volume factor; \( t \) is time; \( Z \) is elevation; \( \phi \) is effective fracture porosity; \( q_g \) and \( q_w \) are the normal well source terms; \( R_{nw} \) is gas solubility in water.

Gas phase pressure \( P_g \) and water phase pressure \( P_w \) are related by capillary pressure \( P_c \).
\[ P_c = p_g - p_w \]  

(6)  

Water and gas saturation satisfy:

\[ S_w + S_g = 1 \]  

(7)  

Equations (4)-(7) make up four equations and contain four unknown variables \( P_g, P_w, S_g \) and \( S_w \), hence it is a solvable system. The volume of adsorbed gas in the coal matrix is described by the Langmuir adsorption isotherms:

\[ \frac{V}{V_L} = \frac{p_g}{P_L + p_g} \]  

(8)  

Where \( V \) is volume of gas adsorbed at pressure \( P_g \); \( V_L \) is Langmuir volume; \( P_L \) is Langmuir pressure.

The gas flow (rate) through the matrix is described mathematically by Fick’s first law of diffusion expressed in the form:

\[ q_m = \left( \frac{V_m}{\tau} \right) [C - C(p)] \]  

(9)  

Where \( C \) is average matrix gas concentration; \( V_m \) is bulk volume of a matrix element; \( p \) is gas pressure; and \( \tau \) is gas sorption time.

**Coupling**

Because FLAC3D and COMET3 are two separate codes, when coupled together, the resulted equations cannot be solved simultaneously; instead they are solved sequentially with coupling parameters passed to each equation at specific intervals. In the SimBurst, the FLAC3D and COMET3 codes are executed sequentially on compatible numerical grids and coupled through user-defined modules which serve to pass relevant information between the equations that are solved in the respective codes. The basic ideas of the coupling process are shown in Figure 1.

![Figure 1 - Basic process of coupling mechanical model with fluid model](image)

**AN EXAMPLE WITH SIMBURST**

A simple model is set up with the SimBurst to simulate the initiation of an outburst in roadway development to illustrate the methodology and approach (Figure 2). The model is 150m long in x-direction, 105m wide in y-direction and 4m in z-direction (Figure 3).
The roadway is 5 m in width and a total of five excavation steps (from 1 to 5) with 5m in each step are simulated with the model. The development rate is 13.75m/shift. Table 1 lists other major parameters used in the model.

Table 1 - Key parameters used in the model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mining depth</td>
<td>500</td>
<td>m</td>
</tr>
<tr>
<td>Coal UCS</td>
<td>10</td>
<td>MPa</td>
</tr>
<tr>
<td>Permeability in x-direction</td>
<td>0.4</td>
<td>mD</td>
</tr>
<tr>
<td>Permeability in y-direction</td>
<td>1.3</td>
<td>mD</td>
</tr>
<tr>
<td>Permeability in z-direction</td>
<td>0.2</td>
<td>mD</td>
</tr>
<tr>
<td>Langmuir constant $V_L$</td>
<td>21</td>
<td>m$^3$/t</td>
</tr>
<tr>
<td>Langmuir constant $P_L$</td>
<td>900</td>
<td>kPa</td>
</tr>
</tbody>
</table>

The modelled results presented here include mesh deformations, pore pressure contours, principal stress vectors and yielded and tensile failure zones around the excavated roadway.
The mesh deformation for each exaction step is shown in Figure 5. It indicates that marginal deformation occurs in the 4th step, and a large deformation occurs at the 5th step, indicating an initiation of an outburst. It should be noted that the deformation at the 5th step as shown in the Figure is taken at an instant when the outburst is initiated.

![Figure 5 - Mesh deformations with roadway development](image)

Figure 6 shows contours of the pore pressure distribution at the 5th step. The elliptical shape of the pore pressure distribution is caused by anisotropic permeability. The maximum pore pressure gradient occurs in the direction of minimum permeability at the face.

![Figure 6 - Pore pressure changes with mining advance](image)

Figure 7 shows the principal stress distribution when an outburst occurs. It indicates that a stress arch is formed about 4 to 5 m ahead of the face. Due to yield and tensile failure of coal failure, a destressed zone is formed around the excavated roadway, as shown in Figure 8.
SUMMARY

A numerical simulator, SimBurst, has been developed by coupling a geotechnical analysis code and a fluid flow and gas desorption model to simulate the initiation process of the outburst, as a first step to model the whole process of the outburst.

A simple model is set up with the SimBurst to simulate the initiation of an outburst in roadway excavation to illustrate the methodology and approach of the SimBurst. The model simulated the basic features of an outburst initiation process, including coal deformation, pore pressure and principal stress vectors redistribution, and yield and tensile failure zone of coal.

It should be noted that the current version of the SimBurst is limited to simulate the process of the outburst initiation mainly because it is based on continuum media and no explicit mechanisms of fracture and fragmentation of coal is included. The CSIRO outburst research team is currently developing a new version of SimBurst which will overcome some of the limitations of the current SimBurst. For example, coal will be considered as a non-continuum medium and two-way coupling between gas and fractured coal will also be taken into considerations.
ACKNOWLEDGEMENTS

The research was jointly funded by CSIRO and Huainan Coal Mining Group of China.

REFERENCES


BENCHMARKING MOIST COAL ADIABATIC OVEN TESTING

Basil Beamish\textsuperscript{1} and Rowan Beamish\textsuperscript{2}

Coal self-heating leading to spontaneous ignition is an on-going safety and environmental issue for the coal industry. To assist with developing an appropriate Spontaneous Combustion Management Plan (SCMP) it is necessary to assess the propensity of the coal to spontaneously combust. This paper presents preliminary results of a new test method that benchmarks moist coal self-heating using an adiabatic oven against actual site performance. The test is conducted on the coal in its as-received moisture state. There is a significant time separation to thermal runaway between each of the coals tested to date. Coals with moisture contents greater than 7% go through a temperature plateau region before reaching thermal runaway, whereas coals with moisture contents less than 7% steadily increase in temperature until reaching thermal runaway.

INTRODUCTION

The influence of moisture on coal self-heating has been investigated by a number of studies. It is generally accepted that there are competing influences of heat of wetting and moisture evaporation depending on the environmental circumstances of the coal (Bhat and Argarwal, 1996; Bhattacharyya, 1971, 1972; Guney, 1971; Hodges and Hinsley, 1964). Numerical model studies by Akgun and Essenhigh (2001) showed that moisture effects on self-heating in a broken coal pile situation are two-fold. In the case of low moisture content coals, the maximum temperature increases steadily with time. In the case of high moisture coals, temperature increases rapidly initially before evaporation dominates and the temperature reaches a plateau value (generally around 80-90°C). Once the coal becomes dry locally the temperature will increase rapidly towards thermal runaway. However, if the coal pile has been in a prolonged drying phase that is interrupted by a rain event and the water penetrates into the pile then additional heat can be generated from the heat of wetting effect as the coal re-adsorbs the moisture available to it. This effect can also lead to premature thermal runaway in the coal pile.

Development of a standard laboratory test to benchmark moisture effects on coal self-heating has not been achieved to date. Instead a number of tests have been developed to rate the propensity of coal for spontaneous combustion (Nelson and Chen, 2007). In the Australian and New Zealand coal industries there is one test that is routinely used. This is the R\textsubscript{70} self-heating rate test (Beamish et al., 2000, 2001; Beamish and Arisoy, 2008a), which has been used to show the effects on coal self-heating rate of rank (Beamish, 2005), type (Beamish and Clarkson, 2006), mineral matter (Beamish and Blazak, 2005; Beamish and Sainsbury, 2007; Beamish and Arisoy, 2008b) and moisture (Beamish and Hamilton, 2005; Beamish and Schultz, 2008). The R\textsubscript{70} self-heating rate is a low temperature oxidation spontaneous combustion index parameter that is measured on dried coal from a start temperature of 40 °C. The relationship of this parameter to thermal runaway performance of as-mined coal has been interpreted on an inferred basis by comparison with coals that have similar R\textsubscript{70} values and coal characteristics. As such a propensity rating scale has been developed for both New South Wales and Queensland conditions using this parameter.

This paper presents preliminary results of a new test method to benchmark moist coal performance from adiabatic oven testing against actual site performance of a range of coals from Australia and overseas that cover the rank spectrum from sub-bituminous to high volatile A bituminous.

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ADIABATIC OVEN TESTING

Coal samples

Details of the samples used in this study are contained in Table 1. Spring Creek Mine extracts coal from the Main Upper Seam of the Greymouth Coalfield, New Zealand using a bord and pillar system and hydro-mining. Coals A, B and C are from the Sydney Basin of Australia. Coal B is currently mined by opencut methods and coals A and C are from two longwall mines still under development. Coal D is imported into New Zealand from Indonesia to supply a local power station. This coal also has a history of self-heating during storage at the New Zealand port facilities if not reclaimed within 10-15 days.

<table>
<thead>
<tr>
<th>Sample</th>
<th>R70 (°C/h)</th>
<th>Volatile matter (% dmmf)</th>
<th>Calorific value (Btu/lb, mmmf)</th>
<th>ASTM rank</th>
<th>Ash content (%)</th>
<th>Moisture content (%)</th>
<th>Start temperature (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spring Creek</td>
<td>5.87</td>
<td>41.3</td>
<td>13749</td>
<td>hvBb</td>
<td>1.2</td>
<td>11.7</td>
<td>27.0</td>
</tr>
<tr>
<td>Coal A</td>
<td>5.95</td>
<td>37.7</td>
<td>13298</td>
<td>hvBb</td>
<td>15.3</td>
<td>7.3</td>
<td>27.3</td>
</tr>
<tr>
<td>Coal B</td>
<td>3.18</td>
<td>45.8</td>
<td>14664</td>
<td>hvAb</td>
<td>4.9</td>
<td>3.0</td>
<td>27.5</td>
</tr>
<tr>
<td>Coal C</td>
<td>3.08</td>
<td>39.3</td>
<td>14029</td>
<td>hvAb</td>
<td>4.4</td>
<td>5.0</td>
<td>27.4</td>
</tr>
<tr>
<td>Coal D</td>
<td>28.57</td>
<td>51.6</td>
<td>9755</td>
<td>subC</td>
<td>1.8</td>
<td>24.0</td>
<td>24.4</td>
</tr>
</tbody>
</table>

SELF-HEATING TEST PROCEDURE

The R70 testing procedure essentially involves drying a 150g sample of <212μm crushed coal at 110°C under nitrogen for approximately 16 hours. Whilst still under nitrogen, the coal is cooled to 40°C before being transferred to an adiabatic oven. Once the coal temperature has equilibrated at 40°C under a nitrogen flow in the adiabatic oven, oxygen is passed through the sample at 50mL/min. A data logger records the temperature rise due to the self-heating of the coal. The time taken for the coal temperature to reach 70°C is used to calculate the average self-heating rate for the rise in temperature due to adiabatic oxidation. This is known as the R70 index, which is in units of °C/h and is a good indicator of the intrinsic coal reactivity towards oxygen.

The major changes from the normal R70 method for moist coal testing are, testing the coal with its as-received moisture content from the ambient mine start temperature, an increased sample size of 200g and a decreased oxygen flow rate of 10 mL/min. Increasing the sample size to 200g provides a greater mass of coal to react that is still manageable without modifying the reaction vessel. Decreasing the oxygen flow rate to 10 mL/min reduces any cooling effect experienced by the coal from moisture evaporation as it self-heats. Effectively, these changes optimise the worst case scenario of developing a heating from as-mined coal.

RESULTS AND DISCUSSION

R70 self-heating rate values

The R70 self-heating curves for each sample are shown in Figure 1. Their respective R70 values are contained in Table 1. It can be seen that Coals B and C have a medium intrinsic spontaneous combustion propensity rating, Spring Creek and Coal A have a high intrinsic spontaneous combustion propensity rating and Coal D has an ultra high intrinsic spontaneous combustion propensity rating. These values and rating are generally consistent with the rank differences between the samples.
Moist coal performance and benchmark comparison

Results of tests using the new moist coal adiabatic method are shown in Figure 2. It is clear that Coals A, B and C are much less reactive than the Spring Creek coal sample when tested from the same start boundary conditions that incorporate the coal moisture. In addition, even though Coal D was started 3°C lower than all the other tests it is still by far the most reactive coal, consistent with its low rank and high R70 rating. The separation in time to reach thermal runaway between each coal is also quite distinct, as is the shape of the self-heating curves.

All of the high moisture samples (Coal D, Coal A and Spring Creek) show a visible temperature plateau region before proceeding to thermal runaway. This is consistent with the published moist coal numerical models of Schmal et al., (1985), Arisoy and Akgun, (1994), Monazam et al., (1998), and Akgun and Essenhigh, (2001). The two low moisture samples (Coals B and C) show no temperature plateau region and steadily increase in temperature towards thermal runaway. Again this behaviour is consistent with the published numerical models.

Coal A takes approximately two and a half times as long to reach a thermal runaway situation as Spring Creek coal (Figure 2) despite having a similar R70 value (Table 1). This is most likely a combination of both the way in which moisture is held in the pores of Coal A and the higher mineral matter content. These same coal features may also be contributing to the lower temperature plateau of 60°C for Coal A. In practical terms the prolonged temperature plateau at approximately 60°C for this coal provides a significant opportunity to detect and manage any hot spot development in loosely broken coal in an underground environment, which could be monitored based on the corresponding gas evolution pattern associated with the coal self-heating.

Coals B and C do not display a temperature plateau due to the low initial moisture content of the two coals. The time taken to reach thermal runaway for Coal B is approximately one and a half times that of Spring Creek coal. Interestingly, the R70 value of Coal B is half that of Coal A yet it reaches thermal runaway much sooner due to the combined effects of a lower moisture content and lower mineral matter content. Coal C however, takes approximately four times longer to reach thermal runaway than Spring Creek coal and over three times longer than Coal B, despite both Coal C and Coal B having similar R70 values. This may be because Coal B is slightly lower in rank than Coal C.
Figure 2 - Moist coal adiabatic self-heating curves for three high volatile bituminous coals from the Sydney Basin and a subbituminous coal from Indonesia compared with high volatile bituminous coal from Spring Creek Mine, New Zealand

Coal D reaches thermal runaway in approximately one quarter of the time taken by Spring Creek. Given actual heatings occur in loose stockpiles of this coal in 10-15 days it would suggest that heating issues would occur at Spring Creek between 40-60 days if a substantial broken coal pile formed. This is consistent with the mine experience of the heating that developed in July 2008 (Beamish and Hughes, 2009). The same analogy can be applied to the other coals. Therefore, Coal B would be expected to take between 60-80 days to develop a heating. Again stockpile experience with this coal confirms that heatings have not developed in less than 60 days and in fact records show that only mild warming has occurred in handling this coal to date.

CONCLUSIONS

Benchmarking moist coal self-heating performance against two known reactive coal behaviours has advanced the use of small-scale laboratory testing for rating spontaneous combustion propensity. There are clear distinctions between high moisture content and low moisture content coal behaviours. Coals tested to date with moisture contents above 7% as-received show a temperature plateau region where the balance between heat generated from coal oxidation and heat lost through moisture evaporation reaches a temporary equilibrium, until the coal becomes sufficiently dry locally. Once this stage is surpassed the coal temperature rapidly goes into thermal runaway and spontaneous ignition is inevitable. In contrast, coals with moisture contents lower than 7% appear to steadily increase in temperature until reaching thermal runaway. These differences have practical implications for spontaneous combustion management. For example, the high moisture situation appears to provide a larger window of opportunity to detect and control an underground heating.

ACKNOWLEDGEMENTS

The authors would like to thank Solid Energy New Zealand Ltd for their continued support of spontaneous combustion benchmarking and UniQuest Pty Limited for granting permission to publish this paper.
REFERENCES

APPLICATION OF FAULT TREE ANALYSIS TO COAL SPONTANEOUS COMBUSTION

Basil Beamish\textsuperscript{1,2}, Tony Sutherland\textsuperscript{3}, Michael Coull\textsuperscript{4}, David Walker\textsuperscript{3}, Gary Day\textsuperscript{3}, Craig Shales\textsuperscript{3}, Roger Craker\textsuperscript{3}, John Rowland\textsuperscript{5} and John Smith\textsuperscript{4}

This method has been applied to the analysis of a spontaneous combustion event for the five year mine plan of the new Abel Mine, Newcastle, Australia. A team of people with diverse backgrounds provided input to the analysis, which was facilitated by a risk management consultant. The team consisted of the Technical Services Manager – Underground Operations, the Project Manager, The Undermanager, the Ventilation Officer, the local Check Inspector (representing the underground operators), a ventilation consultant and a coal spontaneous combustion expert. The resulting fault tree has provided the mine with a clear set of controls that can be incorporated into a spontaneous combustion management plan. This approach sets a new benchmark for the coal industry and produces a generic model that is robust enough to be transferable to any mine by adjustment of the site specific parameters. Some of the branches of the fault tree will be presented in this paper to raise the awareness of the coal industry to the comprehensive nature of this approach to risk management.

INTRODUCTION

All underground coal mines need to assess the risk of a spontaneous combustion event occurring. In New South Wales leading industry practice is the development and implementation of a Spontaneous Combustion Management Plan (SCMP) that complies with the Spontaneous Combustion Management Code (MDG1006). A formal risk assessment is the keystone to developing the SCMP and there are a number of approaches that can be taken to do this. To assist mining operations with this process, the New South Wales Department of Primary Industries and Mineral Resources has issued a Risk Management Handbook (MDG1010) that also incorporates a set of compliance guidelines in the form of a Review Checklist (MDG1014).

In July 2009, the Abel Mine in Newcastle organised a risk assessment session with the objective to identify, analyse and assess the potential for a spontaneous heating event. Hence, the risk assessment considered the risks to Abel Mine and all site personnel associated with a spontaneous combustion event at the mine. It was considered that the most suitable risk assessment method for this task was Fault Tree Analysis (FTA). The focus of the risk assessment was therefore on identifying all possible “direct” and “contributing” causes of spontaneous combustion at Abel Mine, identifying the underlying “root” causes for each and then considering the adequacy of existing controls as preventative measures. The risk assessment covered planned operations in the Upper Donaldson Seam for the next five (5) years, including SMP Area 1, the South Mains to 47c/t and the area to the East. The West Mains were excluded from this risk assessment.

This paper presents a summary of some of the key elements of the fault tree analysis applied to the Abel Mine as it is not possible to present the details of each of the branches determined from the full risk assessment session. The final report for this work was 27 pages long and the completed fault tree is best viewed as an A0 size drawing, which was included as an appendix to the report.

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\textsuperscript{4} Dallas Mining Services Pty Ltd, NSW
\textsuperscript{5} HMS Consultants Australia Pty Ltd, Newcastle NSW 2300
RISK ASSESSMENT

Risk assessment team

A spontaneous combustion risk assessment needs to be undertaken as a team effort with input from diverse backgrounds. Leading practice dictates that a risk assessment session should be managed by a facilitator who is expert in risk management methods to ensure that things are done systematically. In the case of the Abel Mine session a mix of site personnel with relevant experience and external consultants were involved (Table 1). The site personnel covered a range from senior management to face personnel.

Table 1 - Risk assessment team composition

<table>
<thead>
<tr>
<th>Position/ Title/ Qualifications</th>
<th>Industry Experience</th>
</tr>
</thead>
<tbody>
<tr>
<td>Senior Consultant</td>
<td>28 years</td>
</tr>
<tr>
<td>Risk Management Facilitator</td>
<td>28 years</td>
</tr>
<tr>
<td>Consultant</td>
<td>2 years</td>
</tr>
<tr>
<td>Risk Management Scribe</td>
<td>2 years</td>
</tr>
<tr>
<td>Undermanager</td>
<td>23 years</td>
</tr>
<tr>
<td>Operator</td>
<td>11 years</td>
</tr>
<tr>
<td>Local Check Inspector</td>
<td>12 years</td>
</tr>
<tr>
<td>Ventilation Officer</td>
<td>12 years</td>
</tr>
<tr>
<td>Ventilation Consultant</td>
<td>34 years</td>
</tr>
<tr>
<td>Spontaneous Combustion Consultant</td>
<td>30 years</td>
</tr>
<tr>
<td>Technical Services Manager</td>
<td>25 years</td>
</tr>
<tr>
<td>Project Manager</td>
<td>32 years</td>
</tr>
</tbody>
</table>

Risk assessment methodology

The risk assessment was undertaken in accordance with the requirements of AS/NZS 4360:2004 and MDG 1014, using a Fault Tree Analysis approach with spontaneous combustion as the “top level” undesirable event. Team members initially held a “brainstorming” session to identify all issues associated with the development of spontaneous combustion. Following this, a fault tree was developed to display the logical structure of events and situations which could lead to a spontaneous combustion event. The “Primary”, “Secondary” and “Intermediate” causes were identified and considered by the Risk Assessment Team, with each level in turn further analysed to find the “root” causes in each case.

At this point the current controls for each root cause were examined to consider their adequacy. Further (additional) risk reduction controls and actions were then identified by the team to reduce the final risk to an acceptable level, or “As Low As Reasonably Practicable” (ALARP).

FAULT TREE ANALYSIS

Primary causes of a spontaneous combustion event

The initial branches of the spontaneous combustion fault tree are shown in Figure 1. Three primary causes of a spontaneous combustion event developing have been identified. These are self-heating (in response to the mining process), less than adequate dissipation of heat and a less than adequate monitoring system. All three of these branches must occur for an event to escalate to a dangerous level – thus requiring the “AND” gate as shown in Figure 1.
Self-heating

Exposure of coal surfaces to air

Sufficient time of exposure for self-heating to take place and adsorption of oxygen by the coal (Figure 1). All three of these must occur for this branch to be part of the root cause of an event (once again designated by the “AND” gate in Figure 1). The mining factors leading to exposure of coal surfaces are coal left in the goaf, accumulation of coal elsewhere in the mine and fractured coal pillars. It only requires one of these factors to be present for the branch to come into play leading to an event (this is represented as an “OR” gate in the fault tree structure). Each of these intermediate causes can be subdivided further. For example fractured coal pillars can be subdivided into the following four root causes:

- Geological anomalies (Fault, dykes, sedimentary channels etc);
- Less than adequate pillar design;
- Pillar life cycle; and
- Pillar loading.

Any one of these root causes can lead to an event (once again this would be represented by an “OR” gate in the fault tree structure). Having determined the root causes at the bottom of this branch on the fault tree, controls can be identified for each. The current controls for geological anomalies are exploration to date, development of a geological model and hazard mapping. Additional controls for geological anomalies are update the geological model and maps as the mine develops as well as identification of sheared coal zones as part of the mapping process.

Current controls for less than adequate pillar design are pillars to be designed by a competent geotechnical engineer, strata monitoring and monthly strata management meetings. These same controls also apply to the pillar life cycle and pillar loading.

The mining factors associated with exposure over time are a slow extraction retreat rate, failure to seal quickly and effectively and prolonged ventilation over the goaf. Again any one of these intermediate causes can lead to an event. These factors can also be broken down into root causes and controls identified for each.

Adsorption of oxygen by the coal requires both the presence of an air supply and that the coal is susceptible to oxidation. There are multiple possible sources of air into the underground environment that can lead to a spontaneous combustion event. These are:
- Shallow depth of cover;
- Ventilating over the goaf;
- Elevated oxygen levels in the sealed goaf;
- Less than adequate ventilation appliances;
- High pressure differential; and
- Boreholes.

Current controls to mitigate against shallow depth of cover are no pillar extraction at < 50m and surface subsidence monitoring for cracks and remediation. For any ventilation over the goaf, monitoring for signs of oxidation products would be necessary. The current controls for elevated oxygen levels in the sealed goaf are gas monitoring and analysis; no pillar extraction at < 50m; surface subsidence monitoring for cracks and remediation; and identification of the signs of oxidation. In terms of ventilation appliances, the current controls require that these be rated appliances installed by an external supplier. An additional control is the implementation and maintenance of an individual seal installation permit and history file. High differential pressure areas are currently highlighted by Ventsim modelling and analysis for the five year plan. The borehole status database identifies boreholes that need to be checked for integrity.

Less than adequate heat dissipation is a combination of less than adequate cooling and the coal susceptibility to spontaneous combustion. The control for the latter is to provide a thorough assessment of the coal susceptibility (Beamish and Arisoy, 2008) to cover the five year plan. Less than adequate cooling may arise from broken coal thick enough to provide insulation (reducing heat loss) or reduced ventilation or changing humidity of the ventilation current. The reduced ventilation may stem from either sealing issues, regulator issues or goaf fall issues. Appropriate controls for each of these root causes can be implemented as part of the normal management practice of the mine.

Less than adequate monitoring can contribute to an event as a result of either less than adequate monitoring resources or less than adequate response to alarm conditions. In the case of the latter this can be either failure to respond to an alarm or failure of the alarm itself. All of these branches need careful consideration to ensure appropriate controls are in place. For example, failure of the alarm may be as a result of either hardware or software. Current controls for hardware failure include, appropriate maintenance systems, fit for purpose equipment and adherence to standards of engineering practice. Additional controls for hardware failure are TARP to include procedure in case of failure of part or all of the monitoring system and backup power supply. Additional controls for software failure include confirming the monitoring system purchase includes backup software for rebooting or re-initialising software and access to service personnel. Consideration is also given to training mine personnel in software systems.

CONCLUSIONS

Risk assessment is routinely used in the coal industry to develop plans that are appropriate to the management of hazards. Leading practice at all coal mines requires the implementation of a Spontaneous Combustion Management Plan (SCMP). The initial step in developing this plan should be to identify the root causes of an unwanted spontaneous combustion event. This can be achieved using the Fault Tree Analysis (FTA) risk assessment method. Controls can then be constructed from this analysis to form the SCMP. However, to perform this analysis correctly requires additional input from site personnel to capture the experience and diversity required for identifying site specific conditions and controls, thus obtaining the best possible outcome in terms of risk management. This process has been successfully applied to the new Abel Mine in Newcastle to develop a comprehensive SCMP for the five year mine plan. Three primary causes of a spontaneous combustion event have been identified as self-heating (in response to the mining process), less than adequate dissipation of heat and a less than adequate monitoring system. The root causes of each of these branches on the fault tree have been identified with appropriate current and additional controls as preventative measures.
ACKNOWLEDGEMENTS

The authors would like to thank Donaldson Coal, HMS Consultants Australia Pty Ltd and UniQuest Pty Limited for granting permission to publish this case history paper.

REFERENCES

CASE STUDY OF ETHANE EMISSIONS AT MANDALONG MINE

Cambridge Claassen¹ and Basil Beamish²,³

ABSTRACT: Gas chromatograph analysis of sealed goaf areas at Mandalong Mine show that ethane concentrations of 30 ppm are present in older areas and up to 225 ppm have been recorded in more recent areas. Such high levels of ethane would normally be considered an indication of spontaneous combustion for many mines. However, at Mandalong Mine despite these high ethane levels, no significant carbon monoxide is recorded, indicating that the ethane being measured is a result of gas desorption from the coal over time. Generally, there is a sympathetic evolution pattern between ethane and methane, with some areas of the mine being richer in ethane than others. This paper presents results from the mine showing trends in ethane emissions from the sealed longwall panels that have been extracted to date and discusses the implications of the historical goaf seal data for reviewing the existing TARP.

INTRODUCTION

Mandalong Mine is located 50 km south of Newcastle, New South Wales, Australia. The mine operates a 150 m wide retreat longwall system in the West Wallarah Seam of the Newcastle Coalfield. The seam varies in thickness from 3.5 to 6.5 m and has moderate gas content up to 6 m³/t. The predominant seam gas constituent is methane, but ethane is also present as a subordinate component in appreciable amounts. In-seam gas drilling and drainage is applied to the seam to lower the gas content to sufficient levels to prevent statutory limits being exceeded in the mine general body gas make.

The presence of ethane at elevated levels has often been used as an indicator of coal self-heating in the underground environment. However, where ethane may be present as a seamgas constituent the use of ethane within the TARP system becomes more problematical and a considered approach is necessary to the interpretation of gas atmospheres under such circumstances. This paper presents a case history study of the work in progress at Mandalong Mine to resolve this issue.

GAS SAMPLING AND RESULTS SUMMARY

Summary of gas data

Gas chromatographic results of gasbag samples from sealed goaf areas at Mandalong Mine have been analysed graphically to determine and interpret trends in ethane emission data. The main goaf areas investigated to date are MG7, MG6, MG5, MG4, MG3, MG2 and MG1. Methane emission data have also been added to the same plots to establish any sympathetic relationships between the two gases. A summary of the long term averages for ethane, methane, carbon dioxide and oxygen is contained in Table 1. To standardise the interpretation of the results all values have been plotted on an air-free basis and outliers and erroneous data points have been excluded from the analysis. The oxygen values are also contained in Table 1 to show the efficiency of the goaf seals.

Sealed goaf trends

The ethane and methane emission trends for MG1 goaf are shown in Figure 1. The historical long term average for ethane is below 25 ppm (Table 1). However, for the past two to three months this has equilibrated at 30 ppm. Methane and ethane trends appear to be responding sympathetically. Oxygen remains below 5%, but the long term average shows a higher oxygen concentration on the Flank side of the goaf. The carbon dioxide is approximately 7% for both seals (Table 1). No carbon monoxide is present in the gasbag samples indicating the gas accumulation trends are controlled by seamgas desorption and not coal oxidation.

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² B3 Mining Pty Ltd, Kenmore QLD 4069
³ The University of Queensland, School of Mechanical and Mining Engineering, Brisbane QLD 4072
Table 1 - Average gas results for maingate panel seals at Mandalong Mine

<table>
<thead>
<tr>
<th>Goaf seal</th>
<th>Ethane (ppm)</th>
<th>Methane (%)</th>
<th>Carbon Dioxide (%)</th>
<th>Oxygen (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FLANK 1HDG 0-1CT (8/01/08 – 12/10/09)</td>
<td>24 ± 7</td>
<td>46.73 ± 7.97</td>
<td>7.25 ± 0.30</td>
<td>3.93 ± 1.76</td>
</tr>
<tr>
<td>MG1 2HDG 1-2 CT (6/12/05 – 24/08/09)</td>
<td>22 ± 7</td>
<td>37.78 ± 9.05</td>
<td>6.94 ± 1.02</td>
<td>3.05 ± 1.72</td>
</tr>
<tr>
<td>MG1 1HDG 1-2CT (8/01/08 – 14/09/09)</td>
<td>26 ± 9</td>
<td>44.52 ± 6.80</td>
<td>6.79 ± 0.59</td>
<td>2.41 ± 0.78</td>
</tr>
<tr>
<td>MG2 2HDG 1-2 CT (2/06/06 – 22/09/09)</td>
<td>34 ± 5</td>
<td>46.16 ± 4.34</td>
<td>5.96 ± 0.78</td>
<td>2.75 ± 0.89</td>
</tr>
<tr>
<td>MG2 1HDG 1-2 CT (19/10/06 – 12/10/09)</td>
<td>32 ± 12</td>
<td>45.42 ± 8.82</td>
<td>6.34 ± 0.51</td>
<td>2.75 ± 1.19</td>
</tr>
<tr>
<td>MG3 2HDG 1-2 CT (22/11/06 – 7/10/09)</td>
<td>35 ± 10</td>
<td>43.64 ± 9.67</td>
<td>4.54 ± 1.54</td>
<td>3.93 ± 1.57</td>
</tr>
<tr>
<td>MG3 1HDG 1-2 CT (28/06/07 – 12/10/09)</td>
<td>52 ± 14</td>
<td>56.65 ± 8.02</td>
<td>5.10 ± 0.71</td>
<td>2.46 ± 1.04</td>
</tr>
<tr>
<td>MG4 2HDG 1-2 CT (15/05/09 – 7/10/09)</td>
<td>44 ± 6</td>
<td>52.09 ± 3.90</td>
<td>5.34 ± 0.20</td>
<td>1.76 ± 1.26</td>
</tr>
<tr>
<td>MG4 1HDG 1-2 CT (13/05/09 – 13/08/09)</td>
<td>82 ± 20</td>
<td>63.30 ± 4.29</td>
<td>2.99 ± 0.65</td>
<td>1.60 ± 0.76</td>
</tr>
<tr>
<td>MG5 2HDG 2-3 CT (8/04/09 – 8/08/09)</td>
<td>84 ± 14</td>
<td>59.88 ± 4.01</td>
<td>3.78 ± 0.43</td>
<td>1.60 ± 0.82</td>
</tr>
<tr>
<td>MG5 1HDG 2-3 CT (4/05/09 – 12/10/09)</td>
<td>217 ± 16</td>
<td>74.39 ± 3.15</td>
<td>1.99 ± 0.34</td>
<td>2.80 ± 1.27</td>
</tr>
<tr>
<td>MG6 2HDG 1-2 CT (11/04/09 – 11/10/09)</td>
<td>212 ± 7</td>
<td>74.26 ± 3.06</td>
<td>2.42 ± 0.41</td>
<td>2.47 ± 1.49</td>
</tr>
<tr>
<td>MG6 1HDG 1-2 CT (9/06/09 – 10/10/09)</td>
<td>216 ± 5</td>
<td>70.83 ± 1.59</td>
<td>1.65 ± 0.10</td>
<td>2.02 ± 1.36</td>
</tr>
<tr>
<td>MG7 2HDG 1-2 CT (12/06/09 – 7/10/09)</td>
<td>171 ± 4</td>
<td>63.13 ± 1.76</td>
<td>1.99 ± 0.09</td>
<td>2.54 ± 0.20</td>
</tr>
</tbody>
</table>

The ethane and methane emission trends for MG2 goaf are shown in Figure 2. The historical long term average for ethane is above 25 ppm (Table 1). However, for the past two to three months this has equilibrated at 40 ppm. Methane and ethane trends appear to be responding sympathetically. Oxygen remains below 5% and the carbon dioxide is between 6 and 7%. No carbon monoxide is present in the gasbag samples indicating the gas accumulation trends are controlled by seamgas desorption and not coal oxidation.

The ethane and methane emission trends for MG3 goaf are shown in Figure 3. The historical long term average for ethane is above 30 ppm (Table 1). However, for the past two to three months this has equilibrated at 40 ppm. Methane and ethane trends appear to be responding sympathetically. Oxygen remains below 5% and the carbon dioxide is currently at approximately 6%. No carbon monoxide is
present in the gasbag samples indicating the gas accumulation trends are controlled by seamgas
desorption and not coal oxidation.

The ethane and methane emission trends for MG4 goaf are shown in Figure 4. The historical long term
average for ethane is approximately 50 ppm (Table 1). However, this has recently risen to 60 ppm.
Methane and ethane trends appear to be responding sympathetically. Oxygen remains below 5% and
the carbon dioxide is between 5 and 6%. No carbon monoxide is present in the gasbag samples
indicating the gas accumulation trends are controlled by seamgas desorption and not coal oxidation.

Figure 1- Ethane and methane emission data from MG1 goaf

Figure 2- Ethane and methane emission data from MG2 goaf

The ethane and methane emission trends for MG5 goaf are shown in Figure 5. The historical long term
average for ethane is approximately 80 ppm (Table 1). However, MG5 2HDG seal has recently risen to
100 ppm. Methane and ethane trends appear to be responding sympathetically. Oxygen remains well
below 5% and the carbon dioxide is between 3 and 4%. No carbon monoxide is present in the gasbag samples indicating the gas accumulation trends are controlled by seamgas desorption and not coal oxidation.

The ethane and methane emission trends for MG 6 goaf are shown in Figure 6. The historical long term average for ethane is between 210 and 220 ppm (Table 1). However, MG 5 1HDG seal has recently risen to over 250 ppm, but the latest gasbag sample shows the values returning to the long term average (Figure 6). Methane and ethane trends do not appear to be responding sympathetically. Oxygen remains below 5% and the carbon dioxide is between 2 and 3%. No carbon monoxide is present in the gasbag samples indicating the gas accumulation trends are controlled by seamgas desorption and not coal oxidation.
The ethane and methane emission trends for MG 7 goaf are shown in Figure 7. Since the end of July 2009 the average for ethane from MG 6 1HDG seal has been between 210 and 220 ppm (Table 1). However, MG 7 2HDG seal is currently averaging just over 170 ppm. Methane and ethane trends do not appear to be responding sympathetically. Oxygen remains below 5% and the carbon dioxide is between 1 and 2%. No carbon monoxide is present in the gasbag samples indicating the gas accumulation trends are controlled by seamgas desorption and not coal oxidation.

![Figure 5- Ethane and methane emission data from MG5 goaf](image)

![Figure 6- Ethane and methane emission data from MG6 goaf](image)
DISCUSSION OF SPONTANEOUS COMBUSTION TARP FOR SEALED GOAF

Current spontaneous combustion TARP for sealed goaf

The current spontaneous combustion TARP for sealed goaf is shown in Table 2. This appears to be overly complex, but at the time it was originally developed the TARP was meant to cover all perceived combinations that may represent an event at the mine. The high level of ethane used to set the normal TARP level has been based on the recognition that ethane would be present even under normal circumstances due to its presence as seamgas. However, as the mine has progressed it has become apparent that this level does not capture the nature of the ethane emission in the goaf and recent goaf seals are now reaching the trigger purely on the seamgas desorption response with no other signs of oxidation products. Conversely, the fact that no carbon monoxide is being detected in the sealed goaf suggests that the LEVEL 1 trigger for carbon monoxide is set far too high and by the time this is reached a serious event would more than likely be underway. Therefore, based on the historical sealed goaf gas trends a simpler and logical TARP can be developed.

Table 2 - Current Mandalong Mine spontaneous combustion TARP for sealed goaf

<table>
<thead>
<tr>
<th>Normal</th>
<th>LEVEL 1</th>
<th>LEVEL 2</th>
<th>LEVEL 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>CO &lt;20 ppm</td>
<td>CO &gt; 20 ppm and O2 &gt;10% OR CO2 &gt; 10% and O2 &gt;10% OR H2 &gt; 30 ppm and O2 &gt; 10% OR Ethylene &lt;5 ppm</td>
<td>CO &gt; 50 ppm (and upward trend doubling or more over previous 24 hours) and O2&gt;10% OR CO2 &gt; 15% and O2&gt;10% OR H2 &gt; 50 ppm and O2 &gt; 10% OR Ethylene &gt;5 ppm OR Temp &gt;50°, seal sweating, heat haze, tarry smell</td>
<td>CO &gt;200 ppm and upward trend doubling or more over previous 24 hrs OR CO2 &gt;20% OR CO &gt;200 ppm (and upward trend doubling or more over previous 24 hrs and H2 &gt;100 ppm OR Ethylene &gt;10 ppm (and rising) OR Smoke or open flame</td>
</tr>
</tbody>
</table>
Modified spontaneous combustion TARP for sealed goaf

A modified spontaneous combustion TARP for sealed goaf is shown in Table 3. This TARP is much simpler than the existing one and the levels have been set to match the historical data of the first seven longwall panels. The major elements of the TARP are as follows:

- Values for carbon monoxide, carbon dioxide, ethane and ethylene are on an air-free basis.
- Generally \(<2% O_2\) indicates no potential to lead to a serious rise in temperature for any coal as the rate of oxidation is too low, but Mandalong historical data would suggest this value is \(5% O_2\) due to the low reactivity of the coal.
- At NORMAL level ethane, carbon dioxide, methane and hydrogen are monitored to interpret the state of the goaf with respect to seamma gas desorption.
- LEVEL 1 historical averages can be calculated automatically in SMARTMATE and an Excel macro can be added to automatically alarm at the appropriate trigger level. In addition, the historical average for ethane and carbon dioxide should be reviewed at the fortnightly ventilation meeting. By doing this the ethane and carbon dioxide historical averages and standard deviation (SD) information can be provided to mine personnel as a fortnightly update.
- At LEVEL 2 ethane, carbon dioxide and hydrogen values would have increased substantially above the historical average for the goaf.
- The progression from one level to the next takes into consideration the doubling effect of accelerated coal self-heating.
- To confirm an alarm requires repeatable results from an additional two gasbag samples taken from the same location within two hours.

### Table 3 - Modified Mandalong Mine spontaneous combustion TARP for sealed goaf

<table>
<thead>
<tr>
<th>Normal</th>
<th>LEVEL 1</th>
<th>LEVEL 2</th>
<th>LEVEL 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>CO &lt;5 ppm</td>
<td>OR</td>
<td>CO ≥5 ppm AND O₂ ≥5% OR Ethane &gt;2SDs above that particular goaf seal historical average AND CO ≥2ppm OR CO₂ &gt;2SDs above that particular goaf seal historical average AND CO ≥2 ppm OR Ethylene 1 ppm</td>
<td>CO ≥10 ppm OR Ethylene ≥2 ppm OR Temp &gt;50°, seal sweating, heat haze, tarry smell</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

Ethane emissions into the sealed goaf areas of Mandalong Mine have been reviewed in conjunction with other gas data. As a result it is clear that more recent areas of the mine are subject to higher concentrations of ethane from seamma gas desorption. This phenomenon has prompted a review of the existing spontaneous combustion TARP levels, which appear to be quite complex and may well result in an underestimation of the development of a heating in the sealed goaf. Consequently, a modified TARP is proposed that can be validated against the historical goaf seal gas data from the first seven longwall panels. A similar procedure is being investigated for other areas of the mine.

**ACKNOWLEDGEMENTS**

The authors would like to thank Centennial Coal and in particular Mandalong Mine for their continued encouragement to review the existing mine data.
IMPROVING EMERGENCY MANAGEMENT IN UNDERGROUND COAL MINES

David Cliff and John Grieves

ABSTRACT: The findings of ACARP funded research project C17008 - Optimising the collection of information for effective use in the event of an emergency at an underground coal mine is reported. The aim of this ACARP funded research project was to identify ways of optimising the information collection and reporting processes used in emergencies in underground coal mines to ensure rapid and effective response, minimising the risk to life. This was to be achieved through evaluating the current emergency management systems at mines, identifying good practice and also areas that needed improvement. There were three areas of focus for the project: the control room, senior mine official on site and the incident management area. The control room in particular is a key area where accurate information is required during an incident especially in the early stages until a senior mine official can take charge. The control room remains the first point of contact during an incident for most personnel. Speedy evacuation and in seam response is predicated upon knowing what is happening and where everyone is located. A number of mines in NSW and Queensland were visited and their emergency management systems were analysed. In Queensland this was undertaken as part of the involvement in the level one emergency response exercises (LOERS).

A series of twenty recommendations for further action have been compiled and are listed in the final report submitted to ACARP. The main findings of this project were:

- The emergency management system (EMS) often seemed to be no more than a paper document that had not been properly tested.
- Most mines had not formally identified what information would be necessary in an emergency particularly what would be required to ensure rapid re-entry for rescue purposes.
- There is an urgent need to define the minimum information requirements.
- There is a need to define an industry wide competency for control room operators (CRO).
- Mines need to significantly increase the training carried out in emergency preparedness and response especially in the management of incidents.

INTRODUCTION

Ineffective and time consuming information collection, display and analysis continue to be major problems identified in the level one emergency preparedness exercises run each year in Queensland underground coal mines. The events in the USA over recent times at Sago, Aracoma and Darby mines also bear testament to this problem.

In recent years there has been renewed interest on rapid response to emergencies. In 2006 for example, the Queensland Department of Mines and Energy sponsored a workshop on “Fight or Flight, the first five hours”. This seminar highlighted the need to respond using mine resources quickly as external aid will probably take five hours to be ready to deploy. This workshop has spawned three committees to drive the process further. Subcommittee three has been tasked with investigating emergency support and identifying research issues. Information management is one of the seventeen key areas being studied by the group.

The ACARP funded research project C12017 - Mine Integration of Robust Gas Monitoring and Communication and continued as C14026 Ethernet-Based Mine Communications and Information Systems set out to import gas, geo-technical and personnel monitoring data sourced from new and existing subsystems in order to analyse the levels of risks, and produce generic modules that employ an open access protocol as part of an integrated safety system that can be installed at other mines.
(Rowan et al, 2007). In addition a real-time risk management system (RTRMS) was developed. The uptake of this technology has not been rapid and at the recent level one emergency exercise at Grasstree mine the NEXSYS system was not utilised by mine staff. It will take a significant time period before these systems are able to be installed in other mines. The systems are still being bedded down and debugged. In addition due to the cost, the leap in technology and understanding necessary to properly operate these systems, there will be delays in implementing this new way of doing things. This mirrors the development and lack of uptake of the ECAS system (Nemes-Nemeth, 1991) developed more than a decade previously in an attempt to provide expert system support in emergencies.

Previously ACARP has provided funding to improve the capabilities of underground mines and mines rescue services to rapidly and accurately investigate any abnormal gas concentration detected in a mine, through projects C11031 and C13010. These projects promoted the use of computers and electronic systems to collect, display, analyse and communicate information, using standard off the shelf computer software and hardware. In addition readily available decision making assistance systems were showcased and promoted. Despite wide distribution of the project outcomes and software there has been virtually no take up of this technology at mine sites. One reason put forward to explain this non acceptance is that it is designed to be used only in an emergency situation and personnel do not use it on a day to day basis. They are therefore unfamiliar with its operation.

In recognition of this, rather than import new systems to a mine only to be used in an incident, C17008 sought to capitalise on the systems used day to day at the mine for use in an emergency, identifying any improvements or modifications necessary. The types of information to be investigated included the location of persons and equipment on site, the ability to reliably characterise state of the underground environment, and the resources on site available to respond to an emergency.

C17008 aimed to improve the quality of information collection, analysis and dissemination at underground coal mines, the reduction in time taken to acquire the information and make decisions, and the associated improvement in emergency response capacity.

WORK PROGRAM

The objectives of this project were:

- To identify how information relating to the location of personnel and equipment is maintained during routine mine operation.
- To identify how the mine environment is monitored during routine mine operation.
- To identify how this information is accessed and utilised during an emergency by key personnel, in particular by the control room operator, the senior mine official on site and the incident management team.
- To identify what improvements are necessary for effective information usage during an emergency at an underground coal mine, especially during the first few hours of an incident.
- To identify ways of improving the information management at underground coal mines
- To provide practical examples of methods to improve information collection, analysis and dissemination.

From experience in analysing mine emergency response systems, the best way to analyse the information system requirements in an emergency is to step through the process at the mine when an emergency is triggered and ask a series of basic questions.

1. How is an emergency defined? How does an individual know to initiate an emergency response process?
2. Who declares the emergency?
3. When an emergency is initiated what initial information should be collected and by whom?
4. If senior staff is summoned for assistance what information do they require?
5. When is this information required?
6. Who will provide it?
7. In what form will it be provided?

From these observations it was intended to identify mechanisms to improve these processes to meet requirements by:

- Identifying required characteristics
- Identifying current processes
- Identifying gaps and problem areas
- Consulting relevant research eg past ACARP projects
- Identifying practical solutions and improvements

At each mine visited current routine information collection and reporting processes were identified and information flow maps were created to identify:

- How the location of persons and equipment at the mine were monitored
- How the mine environment was monitored, analysed and reported, and
- Who has access to this information, in particular in the control room, the senior mine official on site and if/when an incident management team was convened.

Observations from the level one emergency response exercises were analysed in order to:

- Characterise information requirements
- Compare the requirements with the actual processes uses
- Identify gaps and problem areas

Experience gained from participating in the management of a number of actual incidents was also included for analysis.

**BASIS OF ANALYSIS**

In addition to legislative requirements, and based upon the technical literature, the analysis of the management of a number of significant incidents at underground coal mines in Australia over the past 20 years, and the findings from the level one emergency exercises in Queensland over the past ten years, it was concluded that there are some basic tenets that any effective emergency response system should follow:

1. It must be robust ie. It must function at any time of day or night and with the resources that are available or easily accessible. Historically too many systems are designed only to operate on day shift when a full complement of personnel are available or worse are dependent on

2. It must be simple to implement and not dependent upon the availability of specific personnel. The decision to evacuate a mine must be clear and able to be made by any responsible person not just the mine manager.

3. Specified actions should be clear and prioritised. Duty cards are usually read sequentially so those items at the top of the list get done first.

4. The response must be consistent with that specified within any major hazard management plans

5. The time taken to respond must be minimised. In an underground coal mine the time for effective response is immediate, the longer the delay the harder it is to respond and the higher the risk to personnel. Response time is crucial for mines that are isolated from support facilities.

6. Paperwork should be minimised and remove duplication of forms or functions.

7. The key to effective response is information – knowing what is going on, what will happen, who is where and who and what are available to assist.
8. Automatic actions and responses should be optimised. Computer based alarm systems can reduce the time needed to make decisions. Automated call-out systems are much quicker than manual systems.

9. Leave nothing to chance, be sure of actions – check completion, check response. In any call out system the response from those called out must be known i.e. are they coming and how long before they get to site.

10. Minimise the possibility for corruption of information. Multiple handing of information, especially by word of mouth or on hand written notes, significantly increases the possibility for misunderstanding and misinterpretation. The number of forms being used should be kept to a minimum and designed for optimum use.

11. People in key roles must be familiar with their roles, capable of carrying them out and have the resources available to carry them out. Often the Incident Management Room is not equipped to allow the incident management team (IMT) to operate effectively – inadequate communications, uncontrolled access, too much distraction from prime function, inadequate information display and monitoring.

12. Adequate support for key roles. A key feature is in the continuity of management of emergencies over time, this requires that suitably trained personnel are alerted to take over roles at designated times. This means that not only must they be rested prior to starting their stint but also the information is maintained to brief them quickly and effectively when the changeover of roles is affected. Staff should be changed over in a staged manner to ensure continuity of operation.

13. Interaction with offsite stakeholders needs to be carefully managed to allow the IMT to focus on managing the emergency. This requires adequate resourcing of suitably skilled individuals and that they be kept informed of all developments.

RESULTS AND DISCUSSION

General

At most of the mines visited the emergency response system appeared to have been mainly developed in response to the legislated requirements. The systems thus focused on evacuation and fire fighting. Incident management did not receive much attention. There was a general lack of recognition of what information is required to manage an incident and how to obtain it. At all the level one exercises and in the course of actual incidents much time was lost in identifying what information was required to manage an incident, where to get it from and who could generate it. There is an urgent need to develop an industry-wide guideline that identifies the scope and quality of information that is needed to effectively manage an emergency. Queensland Mines Rescue Service (QMRS) has established a project in cooperation with the NSW Mines Rescue Service to quantify the information needs for reentry and how best these needs could be satisfied. The project has been initiated to identify the gaps between what mines currently collect and what is needed. An ACARP grant application has been made by the QMRS to take this gap analysis further and identify a specification for a system (involving software and hardware) to meet this need.

Mine environmental monitoring systems

All mines visited had modern mine monitoring systems, or were in the process of installing them. The information required for effective incident management is often collected by such systems but is not commonly displayed on a single easily interpreted screen. Instead the information is fragmented, and displayed on a multitude of screens, reflecting the disparate monitoring systems that operate at a mine. Mine environmental monitoring systems should offer a readily accessible data display screen showing the information required for effective emergency management. This should include the status of mine services, the fan and atmospheric conditions.

In a number of cases it was found that mines were not maintaining their mine environment monitoring systems consistent with the appropriate Australian Standards. It was found that there were inconsistencies between the gas concentrations determined by the differing types of monitoring equipment. It is important that all gas monitoring equipment is properly maintained and calibrated. Basic information such as the gas concentration range and accuracy of each type of detection technique should be common knowledge at a mine site.
It was surprising to find how often the triggers in trigger action response plans (TARPs) did not reflect the alarm settings of mine environment monitoring systems. Modern systems should allow for the actions required when these triggers are reached to be displayed electronically and automatically.

Training and competency

It was found that there were generally inadequate levels of training in the management of incidents and in the incident management roles at mine sites. The level of training that mine site personnel receive in emergency preparedness needs to be significantly increased. In particular emphasis should be placed on testing the whole emergency response system, especially under worst case scenarios such as night or weekend. There is an opportunity for providers of training in emergency management to develop courses for mine site personnel in emergency management for personnel other than for the major roles. This would generate a wider understanding of what happens in an emergency and what needs to happen in what order.

Any whole of mine training in emergency management plans (EMP) should include the post incident analysis and investigation that may be required by the regulator. The training should also consider how the EMP would be implemented and maintained for an incident that continues for more than one shift.

Another area of concern, regularly identified in the level one exercises and during the current mine site visits, was the lack of consistency in the requirements and responsibilities for the control room operator. A number of mines were in the process of defining training requirements but most still relied upon a mentoring process with little formal training. There is a need to define a competency for a CRO. This definition needs to be consistent industry-wide.

Analysis of mine emergency response plans indicated that few if any had actually been put to the test of a full scale emergency exercise or incident. Many contained glaring errors and inconsistencies. The best way to validate an emergency response system is to test it. Mines need to regularly carry out emergency training exercises that require the convening of incident management teams and the interaction with offsite stakeholders.

Roles and responsibilities

Another example of the artificial nature of EMP was the impracticality of the functions specified in the EMP for the CRO. It was often impossible for him to carry out all his designated tasks in a timely and effective manner. Testing the plans would quickly identify whether or not this duty card can be carried out effectively or not.

EMP also gave inadequate consideration to the pivotal role played by the ventilation officer (VO). It is imperative that they have the resources and capacity to carry out their role in an incident. It was often found that the VO was the only person on site capable of operating the gas chromatograph; he was required to report on the mine environment to the IMT, generally he was also a member of a mines rescue team, and he would be required to generate ventilation simulations to analyse and develop incident management scenarios for evaluation.

The level of understanding and the number of site personnel competent to operate mine environmental monitoring systems need to be significantly improved with suitable training and practice in using the systems. There needs to be a basic guide to the operation of such systems readily available in an emergency so that these systems can be accessed without requiring the presence of specialists.

Duty cards remain poorly utilised. In a number of instances key personnel completely ignored their duty cards. Duty cards should be developed as aids to personnel to check that they are carrying out their designated roles. The personnel should be properly trained in their roles and only use the duty card as a check or information source. Duty cards are most useful when assigning personnel roles that they are not familiar with at a junior level – portal guard, site security, etc. Some mines had over thirty duty cards. Often the duty card contained a great deal of spurious information and documentation that was not referred to during the incident. Unfortunately often relevant information such as key contact numbers or mines rescue guidelines was often missing from the duty card kits.

The role of a process checker was shown to be valuable at several of the level one exercises and
should be considered for inclusion in EMP.

Emergency response systems

In too many cases the emergency management systems (ERS) and plans were found to be paper documents compiled to demonstrate compliance with legislation rather than real systems. The most common omission from Emergency Response Plans were:

- The training requirements for site personnel were not specified.
- Regular practice of emergency response at panel, site and wider were not specified
- No on call roster of suitably trained and experienced personnel
- Information collection, analysis and management not dealt with.
- Not linked to principal hazard management plans (QLD) or major hazard management plans (NSW)
- No effective process needs to be established to collect witness statements and debriefs and ensure that the information contained within them is transmitted promptly to the relevant personnel
- No systematic information collection and reporting processes in place.
- Normal communications and task planning and allocation systems at the mine were not utilized.

Incident action plans (IAP) must include actions, person responsible for carrying out the action, and the status of the action. IAPS must be reviewed at each IMT meeting including status of outstanding actions.

The most effective emergency management systems are those that build upon those systems in daily use.

Miscellaneous

None of the sites observed during this project or during the level one exercise were able to demonstrate robust personnel location systems. Some were trialing electronic tagging systems with limited success. Tag boards augmented by white boards were universally found to be unreliable and required surface personnel to manually check and update on a regular basis. Control Room Operators regularly complained that underground workers were not reporting their movements. In the level one exercises it would often take in excess of five hours to identify who was where in the mine and who was unaccounted for. Personnel location monitoring systems need to be improved and be made reliable.

There was a wide variety in the quality of incident management rooms. A number were multi-function meeting rooms which at least meant the facilities were kept in a reasonable state of repair. On a number of occasions during level one exercise, the designated IMT room was not used as it proved to be either ill equipped or too far from other key areas of operation. IMT rooms generally did not adequately record and report key incident information, leading to poor or delayed decision making. Incident management rooms need to be equipped with operational equipment and be in a suitable location so that they will actually be used in an emergency. Previous research projects have highlighted the value of electronic and smart white boards.

Information transfer at mines was usually a combination of post-it notes, verbal briefings based upon memory and messages written on the back of hands. Far too often during level one exercises and incidents was key information lost or transferred incorrectly. Greater use of more systematic systems including email on site should be made to distribute information, particularly debrief information, incident action plans and status reports.
CONCLUSIONS

Currently mine evacuations occur once or twice a year across the whole underground coal mining industry in Australia. Incident management teams form probably about twice as often in order to deal with incidents at mines. Thus it is not likely that the average coal mine will experience the need to activate its emergency management plan in order to manage an incident. As such emergency response is not given the priority that it needs to be effective. Most of the shortcomings identified during this research project were created because the emergency response systems had not been fully trialled and evaluated.

The main findings of this project were:

- The emergency management system (EMS) often seemed to be no more than a paper document that had not been properly tested.
- Most mines had not formally identified what information would be necessary in an emergency particularly what would be required to ensure rapid re-entry for rescue purposes.
- There is an urgent need to define the minimum information requirements.
- There is a need to define an industry wide competency for control room operators (CRO).
- Mines need to significantly increase the training carried out in emergency preparedness and response especially in the management of incidents

ACKNOWLEDGEMENTS

The support of the mines involved in this study is gratefully acknowledged. This report would not have been possible without the continued support of the ACARP monitor Bevan Kathage and industry monitors Barry Robinson and Bruce Dowsett.

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MINES RESCUE GUIDELINES: THE NEXT GENERATION

Geoff Nugent¹, Seamus Devlin², John Grieves³, David Cliff⁴ and Darren Brady⁵

ABSTRACT: The procedures under which the coal mining industry based mines rescue organisations for Queensland and New South Wales operate have been developed over many years of challenging training, exercises, rescues, recoveries and, sadly, fatalities. The New South Wales Mines Rescue Service and the Queensland Mines Rescue Service are working together to underpin their operating procedures and guidelines with risk management logic while taking heed of lessons from the past. The initial focus for this undertaking is a crucial aspect of mines rescue operations: the emergency mine entry and re-entry. A three-phase process is being used for the development of new guidelines for emergency mine entry and re-entry to facilitate integration with operations’ emergency response systems and day-to-day operations. The first phase is the assessment of risks and determination of appropriate controls for Rescue Services effecting a mine entry or re-entry. The second phase is the conversion of the risk assessment into the practical guidelines (“Emergency Mine Entry and Re-entry Guidelines”), capturing the necessary controls identified in the risk assessment. The third phase is converting the guidelines into systems that mining operations and mines rescue organisations alike, together with other key industry stakeholders (the Inspectorate, Industry Safety and Health Representatives, Industry Check Inspectors etc), can use for effecting mine entry or mine re-entry responses. A particular emphasis in this third phase is the collection and analysis of information in a timely manner and appropriate format to support decision makers, technical support and crews effecting responses. While these efforts focus on the mines rescue organisation provided services and emergency responses, it is clear there are benefits for operations in having systems ready to support the Emergency Mine Entry and Re-entry Guidelines, as there are significant overlaps between information required for most types of emergency responses involving mines rescue organisations and the information operations require in managing their principal hazards.

INTRODUCTION

Mines rescue organisations have been in existence in Australia for 100 years. The development of these organisations, and the protocols under which they respond to incidents, has been driven by experiences in response to a range of incidents and the development of new technologies.

The New South Wales and Queensland coal mining industries have embraced risk management logic into their legislation and safety cultures. This is equally reflected in the training and response to emergencies under the guidelines used by the New South Wales and Queensland mines rescue organisations. Despite this, the guidelines have until recently not been the subject of a comprehensive, formal risk assessment process.

The underground coal mining industry internationally has had many emergency response experiences, ranging from minor incidents through to significant disasters, where investigations of the events indicated significant improvements could have been made in the emergency response. A common theme in many of these investigations is that the emergency response, and ultimately the outcomes, could have been considerably enhanced by the implementation of appropriate systems for: the collection and interpretation of appropriate data and information; and the management of the obtained knowledge for decision making.

The clear link is that for a response to be effective, those making decisions regarding the appropriate response in an emergency must have adequate information, supplied in a timely manner and suitable format.

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MINES RESCUE ORGANISATIONS

Queensland

Mines Rescue in Queensland

2009 marks the centenary of mines rescue organisations in Australia. With coal mining expanding on the West Moreton field (surrounding Ipswich, Queensland), the then Queensland Department of Mines initiated the establishment of a Mines Rescue Brigade in November 1909 (Hanrahan, 2009). The Department suggested that three miners from each mine be trained in first-aid and rescue methods. Three committees appointed soon after considered: appropriate premises for rescue apparatus; appropriate methods for affording instruction to the colliers who may become honorary members of the brigade; and the smoke helmets best adapted for the district.

In 1910, four coal miners (who had been members of the Queensland Ambulance Transport Brigade) and six new volunteers made up a ten man team to undergo a course of instruction in first-aid and the use of rescue apparatus. They became the first formal mines rescue team in Australasia (Strang and Mackenzie-Wood, 1993).

In 1912, a Rescue Station was erected at North Ipswich on the property of the Ambulance Brigade. The first permanent rescue instructor (who had trained in rescue in Britain) was appointed this same year (Hanrahan, 2009). The first fully equipped Mines Rescue Station was subsequently built at Booval (Ipswich, Queensland) in 1923 (Strang and Mackenzie-Wood, 1990).

Queensland Legislation

The Queensland Mining Act Amendment Act of 1920 made a formal, legislated provision for the establishment of a rescue organisation in Queensland (Strang and Mackenzie-Wood, 1990), which carried into the (now repealed) Coal Mining Act 1925. Key in the repealed legislation was not only the need to establish a rescue brigade to afford assistance in the case of emergency in any coal mine, but also to ensure that through the brigade at all times there would be available a sufficient number of suitably qualified and trained persons suitably equipped to allow the brigade to properly discharge its function (Queensland Government, 1996).

In 1997 the Queensland Coal Legislation Amendment Act 1997 amended the Coal Mining Act 1925 and fundamentally changed the relationship between the then Queensland Mines Rescue Brigade and the coal mine operators. This legislation made each coal mine operation responsible for the provision of a mines rescue capability and for being party to an agreement with an accredited corporation (Queensland Government, 1999). The accredited corporation had the function of providing “mines rescue services”, including: helping each underground mine operator to provide a mines rescue capability; providing underground mines rescue training programs; and providing adequate and appropriate staff and equipment (Queensland Government, 1999). These legislative provisions continued, with some extensions and minor modification, into the current Coal Mining Safety and Health Act 1999.

The Coal Mining Safety and Health Act 1999 defines a mines rescue capability as “the ability to provide a suitable number of trained persons and maintained equipment to allow continuous rescue operations to take place and to help the escape or safe recovery of anyone from a mine if it has, or may have, an irrespirable atmosphere” (Section 221, Queensland Government, 2009a).

While the legislation clearly places responsibility on operations for their mines rescue capability, Section 174(d) of the Queensland Coal Mining Safety and Health Regulation 2001 places responsibility on the accredited corporation for developing appropriate mines rescue procedures: “A mines rescue agreement for an underground mine must state the operational procedures developed by the accredited corporation to be followed by the corporation in carrying out the mines rescue services at the mine” (Queensland Government, 2009b).

Queensland Mines Rescue Services Limited was formed in 1998 and remains the only corporation accredited under the mines rescue provisions in Coal Mining Safety and Health Act 1999. The head office for Queensland Mines Rescue Services Limited is located at Dysart, Queensland, with stations at Dysart and Blackwater.
New South Wales

*Mines Rescue in New South Wales*

Strang and Mackenzie-Wood (1990) note no organised rescue facilities were available for disasters at the Bulli Colliery (Illawarra District, NSW, 1887), Mount Kembla (Illawarra District, NSW, 1902), Stanford Merthyr Colliery (South Maitland, NSW, 1902) and Bellbird Colliery (South Maitland, NSW, 1923).

After the Bellbird Colliery disaster, breathing apparatus had been used at several incidents. The continuing public reaction to the Bellbird Disaster, together with confidence gained in the use of breathing apparatus during the re-entry operations, prompted the New South Wales Government to introduce legislation to enable the establishment of a mines rescue station in the four coal mining districts (Strang and Mackenzie-Wood, 1993): South Maitland (Abermain), Newcastle (Boolaroo), Southern (Bellambi) and Western Coalfields (Lithgow) (Mines Rescue Board NSW, 1999). The South Maitland Rescue station has since moved to Singleton and has been renamed the Hunter Valley Mines Rescue Station.

*New South Wales Legislation*

The New South Wales Mines Rescue Act 1925 was passed subsequent to the Bellbird Colliery disaster and provided for mines rescue stations being established in the four main coal mining districts. The regulations accompanying the 1925 Act and subsequent versions have covered matters such as the duties of rescue station personnel, training standards, rescue procedures, rescue station facilities, equipment and vehicles and rescue facilities at the mines (Mines Rescue Board NSW, 1999).

The current legislation controlling New South Wales mines rescue activities is the Coal Industry Act 2001, through which the Mines Rescue Act 1994 was repealed. The former Mines Rescue Board was dissolved under this legislation, replaced by the provision for one or more companies to provide a range of health and safety services, including mines rescue functions (New South Wales Government, 2009). Coal Services Pty Ltd is the current “approved company” under this legislation, with its subsidiary Mines Rescue Pty Ltd being the current “mines rescue company”.

The Coal Industry Act 2001 notes the mines rescue company has the following principal mines rescue functions in connection with underground coal mines in the State: making available rescue services and facilities to deal with emergencies in those mines and, in particular, ensuring that the (Mines Rescue) Brigade has the capacity to deal with any such emergencies; ensuring that adequate rescue equipment (such as breathing apparatus) is available to enable members of the Brigade to deal with emergencies in those mines; training members of the Brigade in mine rescue procedures at those mines and, in particular, in the use of breathing apparatus; establishing appropriate procedures and arrangements for ensuring the mobilisation of members of the Brigade and the supply of rescue equipment in response to emergencies in those mines; ensuring that persons with an adequate knowledge of mine rescue work are available to provide technical advice to the owners of those mines if emergencies should arise in those mines (New South Wales Government, 2009).

*Other Australian States and Overseas*

It is acknowledged that mines rescue organisations for other states in Australia (Tasmania, Victoria and Western Australia) and overseas have been formed and in many instances continue to service their areas. This paper focuses on New South Wales and Queensland mines rescue organisations servicing their states’ underground coal mining industries, as underground coal mining (and associated legislated rescue capability) in Australia outside of these states is limited.

**Mines Rescue Guidelines: The Past**

The term guideline in relation to mines rescue organisations is used to capture the operating procedures, protocols and standards endorsed and used in both training and emergency response by the Queensland and New South Wales mines rescue organisations. While the guidelines encompass some strict boundaries within which mines rescue members, managers, superintendents and associated mines rescue personnel must operate, the term also reflects that there are aspects of operation that must be adapted to suit the particular incident or training exercise.
Emergency responses in the mining industry have been conducted over many centuries, but formal guidelines are a relatively recent development. The first recorded use of breathing apparatus for rescue in a mine was at the Seaham Pit Disaster, North-East England, 1880. An updated version of these Fleuss breathing apparatus was used in the recovery of bodies following the Mount Kembla Disaster in 1902 (Hanrahan, 2009), necessitating training in the use of the apparatus.

Mining disasters, advances in technology and research appear to have been the biggest drivers of guideline development and revision for the New South Wales and Queensland mines rescue organisations, supplemented by learnings from training and emergency exercises and the formal and informal communications between mines rescue organisations.

Some significant drivers of guideline development include:

- Disasters preceding legislation and formal rescue organisations for Queensland and New South Wales: Bulli Colliery (1887), Mount Kembla (1902), Stanford Merthyr Colliery (1902) and Bellbird Colliery (1923);
- The 1972 Box Flat No. 7 Colliery major explosion, near Ipswich, Queensland. 18 men were fatally injured, including mines rescue members, following efforts to fight an underground fire;
- The 1975 Kianga No. 1 Mine underground explosion, southern Bowen Basin, Queensland, following a spontaneous combustion event. 13 men were fatally injured;
- The 1986 Moura No.4 explosion, southern Bowen Basin, Queensland. 12 men were fatally injured in an underground explosion attributed to an ignition caused by a flame safety lamp;
- The 1994 Moura No. 2 Mine explosion, southern Bowen Basin, Queensland. 11 men were fatally injured due to an underground explosion attributed to spontaneous combustion behind a recently sealed section of the mine;
- The advent and implementation of successive generations of breathing apparatus, including the Fleuss breathing apparatus and Dräger BG174 and BG4 units;
- Updated portable gas monitoring devices for more rapid and accurate analyses of mine environments; and
- The implementation of emergency exercises in the Queensland underground coal mining industry, notably the annual industry-wide “Level 1” exercises since 1998.

Each disaster and emergency exercise has brought about a host of recommendations that have been incorporated into mines rescue guidelines.

**MINES RESCUE GUIDELINES: THE PRESENT**

The inquiry following the Moura No. 2 Mine explosion of 1994 triggered significant legislative changes for the coal mining industry. One of the most critical changes was the adoption of management plans and procedures, underpinned by risk assessments. Risk management was formally becoming entrenched in the coal mining industry, driven by the need for risk to a person from coal mining operations to be at an acceptable level, defined for Queensland coal mining operations as being within acceptable limits and as low as reasonable achievable (section 29, Queensland Government, 2009a).

**Risk Management in Mines Rescue**

While the majority of the protocols by which mines rescue operations are undertaken in New South Wales and Queensland remained the same, rescue efforts from the late 1990s began to incorporate risk management, and particularly formal risk assessments, as part of undertaking emergency responses. The guidelines for the New South Wales and Queensland mines rescue services captured the need to risk assess specific emergency responses while the guidelines set the boundaries for rescue operations. The controls in the guidelines however were not subject at this stage to a formal risk assessment.

Given the co-operative spirit between the two states’ rescue organisations, similarities in the apparent intent of legislation and the same hazards being present across the underground coal mining industries...
for the two states, it is not too surprising that there are many similarities in the mines rescue guidelines from the New South Wales and Queensland mines rescue services.

The intent section of the Queensland Mines Rescue Service guidelines notes: “Mines Rescue guidelines are achieved with the underpinning risk management philosophy in all that is done to minimise and mitigate the challenges, hazards and threats to personnel. However the nature of the underground coal mine environment and situations in which mines rescue teams are called to operate, these guidelines only serve to give direction and guide the decision making process. Decisions are made within risk management practices and therefore are taken by the team leader and team to achieve objectives within the framework of risk based logic. These guidelines serve as a guide to that process” (Queensland Mines Rescue Service Limited, 2007).

The intent section of the New South Wales Mines Rescue guidelines notes: “These guidelines have been developed through detailed risk assessments and consultation with industry and mines rescue experts both within Australian and Overseas. Ongoing annual reviews will be conducted taking into account underground mine emergencies, simulated emergencies and general application of the guidelines to ensure that they remain both functional and practical. Due to the number of variables in an underground coal mine emergency situation the procedures and limits / barriers in the guidelines may not always be appropriate or practical. Should this occur then IMT [Incident Management Team] and MRS [Mines Rescue Service] officers must adopt a documented risk management approach referencing the guidelines to identify likely risks associated with the proposed operation / actions and the barriers to be implemented” (Mines Rescue Pty Ltd, 2009).

The intents in these guidelines clearly point to the use of risk management, and specifically the use of documented risk assessment tools, as part of mines rescue.

Risk Management: Mines Rescue Guidelines

Queensland coal mining legislation can be interpreted as requiring the same application of risk management processes for mines rescue procedures (which would include the guidelines) as is required for the development of procedures for operations. By virtue of the mines rescue agreement stating procedures (such as “the operational procedures developed by the accredited corporation to be followed by the corporation in carrying out the mines rescue services at the mine”) and links to each operation’s Emergency Response Principal Hazard Management Plan, the Queensland Mines Rescue Service guidelines can be considered a part of each operation’s Safety and Health Management System. This is further reinforced by the Queensland Coal Mining Safety and Health Regulation 2001 (refer sections 359, 360, 366) where the use of mines rescue trained personnel, with mines rescue equipment and working under mines rescue procedures, can enter irrespirable atmospheres.

Following this principle, and in the absence of specific regulations or recognised standards, safety and health obligations for mines rescue can only be discharged by taking reasonable precautions and exercising proper diligence (section 38, Queensland Government, 2009a). Underpinning mines rescue guidelines with formal, documented risk assessments is an effective way of demonstrating reasonable precautions have been taken and proper diligence has been exercised.

While the above has focussed on Queensland Mines Rescue Service guidelines satisfying Queensland legislation, the same conclusions can be drawn for New South Wales Mines Rescue guidelines with reference to Duty of Care provisions under the Occupational Health and Safety Act 2001 and the need for “emergency management systems” under the Coal Mine Health and Safety Act 2002 (refer sections 44-47).

MINES RESCUE GUIDELINE REVIEW: EMERGENCY MINE ENTRY/RE-ENTRY

A core mines rescue activity is that of entering or re-entering a mine as part of an emergency response: the emergency mine entry/re-entry. While there are many aspects of the mines rescue guidelines to which risk management philosophy can be applied, the emergency mine entry/re-entry is such a crucial and wide-reaching aspect of mines rescue operations, with significant overlaps with how operations manage their principal hazards and emergency responses, that underpinning emergency mine entry/re-entry processes with risk management will have significant benefits and is a logical starting point for a fundamental review of mines rescue guidelines.
The review of emergency mine entry/re-entry activity was partly in response to the recommendation from the report for the 2007 Queensland Level 1 Exercise (held at Grasstree Mine) that the Queensland Mines Rescue Service “should formalise the guidelines by using a risk based approach to develop a set of mine re-entry TARPS based on explosibility rather than percentage of UEL and LEL of explosive gases” (Alexopoulos et al, 2007).

It is important to note that although there is considerable overlap between information essential for emergency mine entry/re-entry and a “no lives at risk” mine entry/re-entry, risk tolerance and planning processes would differ as recognised in the New South Wales Mines Rescue guidelines: “The re-entry and exploration within a mine for the recovery of bodies or restoration of operations is not normally considered an emergency situation. These activities should be a pre-planned operation, using a risk management approach (with reference to the guidelines to identify the likely risks associated with the proposed operation), and under the direction of mine management” (Mines Rescue Pty Ltd, 2009).

Phase I: Risk Assessment

The first stage of reviewing the emergency mine entry/re-entry process was a thorough and comprehensive risk assessment involving relevant industry stakeholders. The risk assessment team initially undertook a brainstorming process to assist with identifying the potential hazards or barriers which could prevent a mines rescue team entering a mine or part of a mine considered dangerous to coal mine workers. While the brainstorm process identified a number of external barriers which could prevent re-entry, the team consciously focused on the potential hazards and barriers existing at a mine, in what the team regarded as known-unknown information (unquantified hazards) to the rescuers and decision makers (Incident Control Team).

From the brainstorm process, twelve critical hazards were identified for the risk assessment team to analyse, specifically for how they could occur and why they would occur. The risk assessment techniques that were used to assist in the process were a customised semi-quantitative risk assessment tool based on the Minerals Industry Risk Management Guidelines and Queensland Mines and Energy Recognised Standard 02 (“Control of Risk Management Practices”). Due to the risk assessment not being mine specific, the team agreed that no current controls would be applied which therefore ranked all hazards as extreme. Proposed controls and hazard specific barriers where then provided by the group for each hazard to mitigate its risk.

This process was completed through a major industry risk assessment conducted over four days, facilitated by the Queensland Mines Rescue Service (QMRS), with participation from: the NSW Mines Rescue (NSWMR); Queensland Department of Mines and Energy (now Queensland Mines Energy, part of the Department of Employment, Economic Development and Innovation); the Construction, Forestry, Mining and Energy Union (CFMEU) Industrial Safety and Health Representatives; Simtars, mines rescue volunteers and third party industry stakeholders. The assessment reviewed key hazards and addressed specific issues in relation to the deployment of mines rescue crews in emergencies. It was highlighted by the risk assessment that mine hazards must be able to be assessed accurately and efficiently, not only to determine and analyse what is known, but to identify what (if any) further information is required for sufficient understanding of mine conditions to the level necessary for sound, risk-management based deployment and management of resources.

This phase of the guideline review was completed November 2008.

Phase II: Guideline Development

The second phase, developing the Mine Entry/Re-Entry Guideline, is in progress. The objective here is to develop the results and controls from the risk assessment into a guideline incorporating checklists and flow charts for emergency mine re-entry. The intention is to establish a tool which can be utilised by both mines rescue services and operations with the aim of efficient and effective management of emergency responses.

A task group, the Mine Re-Entry Task Group, was formed early in 2009 to develop a framework for the implementation of action items from the Mine Entry/Re-entry Risk Assessment. The core members of the task group are: Geoff Nugent (Queensland Mines Rescue Service), Seamus Devlin (New South Wales Mines Rescue), Darren Brady (Simtars), Assoc Prof David Cliff (Minerals Industry Safety and Health Centre, Sustainable Minerals Institute, University of Queensland) and John Grieves (New Hope Corporation Limited).
Under the guidance of the task group, a guideline is being developed, supported by checklists and flowcharts, for knowledge management in the event of an emergency. This guideline will detail what information is required to support an emergency response and how such information can be attained. Part of this guideline development is to scope opportunities for software and hardware solutions suitable for emergency response for the management of information, and to test draft guidelines at emergency exercises.

Specific actions within this second phase are:

1. Classifying the controls from the Risk Assessment:
   - Identify responsibility for collection/interpretation of information (site, mines rescue, external provider);
   - Determine the ability to collect and maintain information prior to a response;
   - Determine the information type eg automatic generation, manual collection; and
   - Determine the information importance to assessment of risk i.e. Rank its level of criticality.

2. Conducting post-mortems of previous emergencies and emergency exercises applying controls from the risk assessment.

3. Developing audit tools from the risk assessment to conduct gap analysis between what information/processes are commonly/typically available at an operation (Qld and NSW) and what is required to comply with developed guidelines.

4. Seek key stakeholder feedback on draft guidelines via: Operators Forums (Qld and NSW); Queensland Safety & Health Conference presentation/workshop; Queensland Mines Rescue Service Technical Advisory Committee; New South Wales Mines Rescue Standards committee; and the Mine Managers Association of Australia.

5. Disseminate guidelines to industry

6. Test guidelines within Level 1 or 2 Emergency Exercise

**Release of the draft guideline is planned for first quarter 2010.**

The task group has conducted the classification process and developed an audit tool to conduct gap analyses at selected underground operations in Queensland and New South Wales. Gap analyses have been conducted at Anglo Coal Australia’s Moranbah North Colliery (Bowen Basin, Queensland), Peabody’s Metropolitan Colliery (Southern Coalfields NSW), Caledon’s Cook Colliery (Bowen Basin, Queensland), Rio Tinto’s Kestrel Mine (Bowen Basin, Queensland), Xstrata’s Tahmoor Colliery (Southern Coalfields NSW) and BHP Billiton’s Dendrobium Mine (Southern Coalfields NSW). Additionally the Task Group has conducted smaller assessments at Xstrata’s Oaky North Colliery (Queensland) and Centennial Coal’s Mandalong Colliery (NSW).

Through these gap analyses, the task group has identified some common but important trends in relation to emergency response information management:

- Information requested is captured but not readily available within an acceptable time;
- The supply of critical (and sometimes basic) information is reliant on one or two key people being available;
- Some information monitored is not understood by people monitoring;
- Some required information (particularly for validation) is not monitored or measured at all.

**Gap Analyses Results**

The gap analyses have provided a wealth of information that assists with the development of the guidelines and provides examples of high quality systems that not only effectively manage principal hazards, but that also provide high levels of support in the event of an emergency response. Equally, the gap analyses have identified areas across a number of the operations where more effective systems would undoubtedly provide better management of principal hazards and superior results in the event of an emergency response. Many of these areas have been identified in previous Queensland Level 1 exercises.
Some examples of high quality systems worthy of consideration at all operations are:

- Up-to-date registers of ventilation control devices, implemented under a regime of routine device inspections;
- The use of a “process checker”, a person not directly involved in decision making and action taking, but auditing events against emergency response plan requirements, providing reminders to key personnel on necessary functions that may otherwise be missed;
- Comprehensive mine environment and ventilation monitoring systems with redundancy and due regard for providing information to assist self-escape and aided-escape efforts – consider how a surface controller or control room operator knows where to direct those escaping from underground through the appropriate route; and
- Thorough understanding of seam gas makes and behaviour including the impact on districts and whole of mine environment when change occurs through planned or unplanned ventilation interruptions and barometric influences.

Common shortfalls in emergency response systems, generally also identified in previous Queensland Level 1 exercises, include:

- Inconsistent debriefing processes that fail to capture or pass on information from key eyewitnesses;
- Debriefing process does not utilise targeted questions to determine last known status of localised and general mine conditions such as ventilation and devices, manual atmospheric monitoring, other environmental conditions (eg visibility), roadways and panel layout.
- Control room operators juggling multiple duty cards and under extreme time pressure;
- Heavy reliance on technical people for appropriate responses (eg ventilation officers, electrical engineer) with little or no redundancy;
- Fundamental information for status of the mine environment and systems are not automatically or manual maintained to convey the relevant information to emergency response teams in a clear and concise format.
- Under utilisation, or lack of awareness, of some environmental monitoring software analysis capabilities to provide preset charting with trigger levels for less common ratios and trending rate of change, particularly for potential explosive atmospheres.
- Limited consideration of current location, status and accessibility of other interconnecting airways (including boreholes) for use as alternative means of monitoring, communication and ventilation during an emergency.
- General lack of recognition of specific sensor ranges for handheld, real-time and tube bundle monitoring systems (critically at control room operator and ventilation officer level), coupled with systems failing to indicate or alarm where sensors are returning results that are out of range;
- Automated alarm settings inconsistent with triggers as specified in management plans and associated trigger action response plans (TARP);

Informal feedback has been given to the operations at which the gap analysis has been conducted.

**MINES RESCUE GUIDELINES: THE FUTURE**

**Phase III: Guideline Implementation**

The completion of constructing the Mine Entry/Re-Entry Guideline, supported by appropriate control check-sheets and checklists, represents a major step forward for mines rescue operations. Discussion and feedback on the draft guidelines will likely take the guideline development through to mid 2010. The effectiveness of the guidelines and how well the guidelines are incorporated into operations’ emergency response systems can be tested through the following: audits through mines rescue organisations; emergency exercises; mines rescue training exercises; and emergency responses.
The logical extension of this process is to continue the review process through the remainder of the mines rescue guidelines.

While the guideline implementation focuses on emergency response, there are clear benefits to operations using guideline systems relevant to everyday operations and day-to-day management of principal hazards. Such opportunities for integrated systems, which promote familiarity for operators and management alike and give the best probability of effective emergency response, are obviously favoured.

**Critical Information Management: Implementing the Guidelines**

The Moura No. 2 Inquiry Task Group 4 (Mines Rescue Strategy Development) report stated: “Knowledge of conditions in a mine following an incident is essential in planning any rescue effort. Information systems must be provided to support implementation of the most appropriate rescue measures” (Moura No. 2 Inquiry Task Group No. 4, 1994). This same report contained the recommendation that “Industry should develop an effective computer-based emergency decision support system for incident management and training”.

In the event of an underground coal mine emergency, the rapid and accurate collection of data relating to mine hazards and the efficient assessment of such data are crucial to the safe deployment and management of resources responding to such an event. Various reports and forums, including the September 2006 Queensland “Fight or Flight” Seminar, have recognised the first five hours of an emergency response as critical for implementing effective strategies for the best outcomes. Analysis of industry emergencies and emergency exercises has repeatedly proven that the site data required to determine an appropriate course of action post-incident and adequately assess the risks in effecting appropriate emergency responses is rarely available in a timely manner and suitable format.

The Queensland Mines Rescue Service and New South Wales Mines Rescue, supported by the task group, have identified a suitable support project for guideline implementation: the “Emergency Response: Mine Entry Data Management” project. The Australian Coal Association Research Program (ACARP) supports this project (reference C19010) and further work will be forthcoming during 2010. ACARP project C19010 will commence in 2010.

The aim of this project is to develop a functional specification for data collection and management systems suitable for the efficient, risk-assessed management of mine hazards in the event of an emergency response. The use of risk-management logic provides adequate control of risks in effecting emergency responses while maximising response efficiency. This project is targeting a quantum leap in information management for emergencies by the development of functional specifications for systems that facilitate the “Mine Entry/Re-entry Guideline”.

The objectives of this project are to: develop a functional specification for an information collection and management system appropriate for efficient, effective implementation of the Mine Entry/Re-Entry Guidelines; and to raise industry awareness of Mine Entry/Re-Entry Guidelines and information collection and management systems appropriate for emergency responses.

The “Emergency Response: Mine Entry Data Management” project differs from previous research undertaken in the emergency response area by linking risk-management logic underpinning Mines Rescue emergency response procedure development to site emergency response information requirements. The results from this project will be a targeted response to the key recommendation from the forthcoming ACARP C17008 Project (“Optimising the Collection of Information for Effective Use in the Event of an Emergency at an Underground Coal Mine”) report: “There is an urgent need to develop a guideline that identifies the scope and quality of information that is needed to effectively manage an emergency. This should [be] consistent industry-wide” (Cliff, 2009).

The major benefits of this research are: industry will have relevant and functional specifications for information management systems that will offer clear directions for current and future mine monitoring and analysis systems so that information relevant to effecting an emergency response is readily available in suitable formats during a mine emergency; information management systems will suit “Mine Entry/Re-entry Guidelines”; and a key priority for underground research from ACARP, “reviewing the adequacy and effectiveness of emergency response measures leading to practical solutions for industry implementation” will be researched.
Key components of this project to manage emergency response information management issues are: researching and developing software and hardware specifications (with paper based equivalents for sites where electronic systems are not justified); targeting information appropriate to specific incident types; and building on research completed to date (including systems such as ACARP funded integrated data management system NEXSYS – refer Rowan et al, 2007 – and the NERDDC funded expert computer software system ECAS – refer Nemes-Nemeth and Aubrey, 1991).

The key risk areas for the project are: ensuring developed systems are appropriate for “Mine Entry/Re-entry Guidelines”; and ensuring the development of useful, appropriate specifications that integrate with the variety of information management systems (paper and computer-based) already in utilised by operations. The experienced project leaders representing the key Queensland and New South Wales mines rescue organisations, combined with other key task group members, and the level of support offered from key industry stakeholders and operations minimise exposure of this project to these risks.

Adoption of Guidelines by Industry

Updating the guidelines is of little value if the results and key learnings are not disseminated to industry, or if the guidelines are not absorbed into the fabric of emergency response strategies within the coal mining industry.

Efforts to achieve widespread adoption include:

- The Mine Entry/Re-Entry guidelines will be incorporated into Queensland Mines Rescue Service and New South Wales Mines Rescue procedures, promoting standardisation;
- Integrating the processes documented under the guideline and information management tools into existing industry emergency management training programs, such as the Queensland Mines Rescue Service’s Mine Emergency Management System (MEMS) and Coal Mines Qualifications Board Emergency Management Course;
- Reviewing competency standards for emergency management and ventilation officers to identify opportunities for improvement based on the guideline and developed technology; and
- Promoting guideline and information management tools through industry forums via presentations and workshops.

CONCLUSIONS

The Emergency Mine Entry/Re-entry Task Group believes that the development of the Emergency Mine Entry/Re-entry Guideline, appropriately implemented and coupled with a knowledge management tool (founded on risk management logic), would significantly assist emergency responses and decision makers in real and simulated emergency situations through:

- Taking reasonable precautions and demonstrating proper diligence in the decision making process;
- Determining and understanding existing risk within a constrained time, promoting effective planning and strategies;
- Developing and implementing plans and strategies that do not place rescuers at an unacceptable level of risk.
- Minimising delay to rescue operations during information collection and assessment;
- Reducing the likelihood of abandonment of any attempt of rescuing affected coal mine workers.

The process of establishing the updated guidelines, underpinned by risk assessment, also facilitates the review process for future refinement. The information management systems have obvious benefits for day-to-day and routine operations in the management of any operation’s principal hazards.

Completion of the processes outlined in this paper will mark a quantum leap forward in industry emergency response, emergency preparedness and information management.
REFERENCES


APPIN COLLIERY EXPLOSION REASSESSED

Bob Kininmonth

ABSTRACT: The Judicial Inquiry into the Appin Colliery explosion on 23rd July 1979 made a specific finding that the fan starter box was the location of the initial ignition of gas. The subsequent Coronal Inquiry recognized the possibility of the fan starter box being the source but also recognized the possibility that the Deputy’s flame safety lamp could have been the trigger. Neither Inquiry detailed the mistakes that must have been made for either of those options to be the cause. It is contended that the flow of factual information at such Inquiries can be influenced by the high-profile legal representation used by the interested parties. It is important to ask whether an Inquiry before an independent technical expert would give the industry a better explanation of the factors that led to and caused the event.

INTRODUCTION

At 11pm on Tuesday 24th July 1979 an explosion of methane gas in K Panel of Appin Colliery resulted in the deaths of 14 workers. This disaster was associated with a changeover of ventilation that was intended to create a situation whereby a central intake, in a three heading development panel, would be shielded from intake gas by returns on each side. Prior to the changeover the panel had two intakes and one return. The then Minister for Mineral Resources directed that a Judicial Inquiry be held. His Honour Judge A.J. Goran conducted the Inquiry and delivered his report on 9th May 1980 (Goran, 1980). Subsequently a coronial Inquiry under the Coroners Act was held before Stipendiary Magistrate J. Hiatt and a report delivered on 19th December 1980 (Hiatt, 1980).

PLANNED VENTILATION CHANGEOVER

Ventilation conditions in K Panel prior to the changeover and the intended affect of the changeover are shown in three stages in Figure 1 as follows:

Stage 1 shows the panel layout on the morning of Tuesday 24th July with A and B headings as intakes and LW8 Maingate as the return. The overcast at the intersection of C/T 3 and A heading was being built. The LW8 Maingate stub was brattice ventilated. An auxiliary fan in C/T 4 provided ventilation for A heading while B heading that had been advanced 70 m as an intake stub, was ventilated on brattice

Stage 2 shows the post-changeover ventilation flow. This situation was planned to be reached, after completion of the overcast, by erection of the temporary brattice in B heading and removal of the brattice stopping in C/T 3.

Stage 3 was to be the completion of the changeover with a fan, that had been placed in B heading being used to ventilate the stub.

Near the end of the afternoon shift the overcast had been completed and the brattice in B heading had been erected. The Deputy who was meant to remove the C/T 3 brattice did not do so because he noticed leakage from the overcast. It was near the end of his shift and he left the panel. During the inquiries it could not be established what messages were passed to the evening shift Deputy, but it is clear that for some undetermined time the whole of B heading inbye of C/T 3 was virtually unventilated.

1 Retired Senior Inspector of Coal Mines
INQUIRY FINDINGS

The report by Judge Goran (1980) included the following statements:

1. 'It is now obvious that I cannot accept as any form of probability the proposition that the deputy’s safety lamp caused the first ignition.'

2. 'I am therefore left, as a result of the whole of the evidence, with the conviction that the explosion began by an ignition in the fan starter-box. I do not suspect that the deputy’s lamp contributed in any way to the explosion.'

3. 'My finding necessitates a finding of gas in the starter-box and around the back of the fan. I am unable on the evidence to say in any precise manner how this collected, there being no eye witnesses and the evidence itself having been largely destroyed by the explosion. The following means are, however open on the evidence:

   i. The failure of adequate ventilation of B stub, because of the non-removal of the No3 cut through brattice.

   ii. A possible failure of ventilation due to an occurrence such as a fall in a stopping outbye.

   iii. The substantial leakage in the overcast and through B heading stopping creating a serious deficiency of air available to the B heading fan.'
iv. The failure, deliberate or accidental, of the B heading stopping’.

Coroner Hiatt (1980) stated:

‘I have spent considerable time on my deliberations concerning the question of the source of ignition and referred to many aspects both in support and against each of the alternatives advanced. Other alternatives were excluded on consideration of the evidence and another raised by Mr. Lloyd as to a spark caused by friction on the use of the steel wedge has been left up in the air, however I have determined that such a proposition is less likely as a probability than the fan starter switch in B heading or the Oil Flame Safety Lamp in the possession of Mr. Rawcliffe. If it was the former then it is probable that some person now deceased was at fault. If it was the latter, such act or omission not being deliberate or without exercise of reasonable care, the cause of the ignition could have been without fault or accidental.

An examination of the evidence in respect of those two alternatives discloses:

In one, the fan starter switch chamber, a higher probability of an ignition source and less probability of gas being present in explosive proportions with a readily ascertainable flame path to the face of the headings and in the other, the Deputy’s defective Oil Flame Safety Lamp, there is a higher probability of gas in explosive proportion being present with a readily exposed flame path (the vent tube or layering) with a less probable source of ignition than that available in the case of an alive electrical source in my view neither can be responsibly excluded on the evidence before this Court.

The evidence adduced does not enable me to say what was the source of the ignition which caused the explosion of methane gas at the face of the heading. Therefore I am not able to determine what the proximate or direct cause was of the explosion but I have concluded that there was a condition existing for a period of time whereby the B heading stub was not properly ventilated. The precise reason for that has not been disclosed on the evidence but on the balance of probability there is evidence that the acts and omissions of persons on the previous shift contributed to inadequate ventilation.’

ESTABLISHED INFORMATION

At the time of the explosion ten of the crew members were in the cribroom in the intake roadway and four employees were in the face area. The Undermanager, the Deputy and the electrician were near the fan.

The following significant matters relevant to the incident appear to have been accepted:

1. The starter box of the fan was not in a flameproof condition, most of the studs had been removed. It was considered that the fan had been started and found to be running in reverse. Change of direction could be achieved from inside the starter box but should not have been attempted with the power on.

2. At the time of the explosion the fan cable was live.

3. There was a distinctive pattern mark inside the fan starter, which later tests indicated could only have been formed by an internal ignition.

4. The Deputy was equipped with a flame safety lamp for gas detection. The Undermanager, who arrived in the panel at a late stage, had both a flame safety lamp and a methanometer. The Deputy’s flame safety lamp, which was damaged in the explosion, was shown by later examination to have some defects. In particular it was found that the relighter key was missing.

5. There was a brattice stopping in C/T 3, which should have been removed to allow flow of air into the return, it was not removed as expected and the time of its removal could not be determined.

6. The roadway in which the fan had been installed was almost completely unventilated for an undetermined period but possibly as long as five hours.

7. Scientific investigation indicated that the explosion was initiated at the face end of the standing stub entry. It was considered that flame had traveled up the vent tubes leading to the gas accumulation at the face.
8. The inspectorate did not enforce the requirement of General Rule 1 of the Coal Mines Regulation Act 1912, that intake gas levels should not exceed 0.25 per cent.

Comments

In assessing the circumstances that led to the explosion there are a number of matters that are fundamental to an understanding of the event.

Gas control

There were three levels of methane covered by legislation in New South Wales: a level of 0.25 per cent in the intake, a level of 1.25 per cent when electric power would be disconnected and a level of 2.5 per cent when workers would be withdrawn.

In addition it is important to acknowledge that methane in air will burn on a flame and can be recognized between 1.25 percent and 4.5 per cent. At 5 per cent an ignition occurs. When present at higher percentages methane is most easily ignited at 7.5 percent and has its maximum explosive strength at 9.8 percent.

Flameproof enclosures

A flameproof enclosure is designed so that the lid of the enclosure fits on a wide edge that absorbs heat and prevents an internal explosion being transmitted with sufficient energy to ignite an external explosive mixture. It is essential that the lid is tightly held in place –usually by a number of screwed studs. If a flameproof enclosure is to be opened where gas may occur it is important to disconnect the power supply.

After the explosion at Appin the fan starter-box was found to be in a non-flameproof condition with only one of the 24 studs in place. The starter-box immediately became suspect as the source of the ignition.

Flame safety lamps

Apart from the ability to show the presence of methane, safety lamps have also been regarded as being an important tool in providing warning by being extinguished in low levels of oxygen. They have the characteristic that when they are placed in contact with methane at 5 per cent an ignition inside the lamp extinguishes the flame and the flame does not pass outside because of the protective gauze construction.

A post-explosion investigation showed that many safety lamps at Appin and in the Illawarra, had minor defects. A series of tests of safety lamps with these defects indicated that in some circumstances they could transmit an ignition to an outside explosive mixture. MacKenzie-Wood (1980) reported:

‘Glasses with ‘non-parallel’ ends, found in colliery stock fitted to a safety lamp in otherwise good condition, were found to ignite an external flammable atmosphere with internal ignition’

The safety lamps issued to Deputies and other officials at Appin were of the Protector type with a pyrophor relighting facility. The approval for that type of safety lamp contained a condition that a lamp should not be relit in a place where gas was likely to be present.

Judge Goran (Goran, 1966) in his report on the Bulli Colliery fire commented.

1. ‘The oil flame safety lamp has a long and successful history in mines as an instrument of safety. Its flame can readily detect carbon dioxide or methane. Despite any criticisms which have been levelled at it during my inquiry and elsewhere it is truly the miners friend and, in my opinion cannot at present be supplanted.’

2. ‘There has been demonstrated before me an improved oil flame safety lamp manufactured in Great Britain which has a relighting device within it, whereby the lamp can be relit by use of a lever which generates by friction a spark in the safety of the lamp itself, without the lamp being unlocked for this purpose it should not be relit except in the main body of the air current. This is
because there may be an explosive mixture within the lamp itself. A repeated explosion would render the lamp ineffective, since its gauzes would become red hot.'

Strang and Mackenzie-Wood (1990) wrote:
‘If a lamp is defective then it is during the act of relighting that the moment of greatest potential danger lies.’

It is of interest to note that safety lamps have always been suspect in conditions of high air velocity and that the relighting of lamps can be a source of danger.

Pamely (1898) wrote in the following terms:
‘Again if the lamp were extinguished in any other way, and subsequently filled with an explosive mixture, the sudden flash of re-lighting and the internal explosion might cause sufficient pressure within the lamp to force the flame through the gauze and fire the external gaseous mixture.’

Platt (1924) wrote:
‘The most severe test that can be made on a lamp in a still atmosphere is the ‘internal ignition test’ and while it should be made on all types of lamp, it is particularly appropriate to lamps fitted with internal relighters for it is during the act of relighting that the moment of potential danger lies if the lamp is defective.’

Strang and Mackenzie-Wood (1990) continued:
‘The pyrophor relighter lamps were banned in Germany, their country of origin, in 1953.’

Following the explosion at Moura No4 Mine in 1986 an Inquiry (Queensland Government, 1986) concluded that safety lamps were not considered to be safe in atmospheres containing methane and coal dust and they were withdrawn from general use in Queensland coal mines.

Although approval for the use of safety lamps was not revoked in New South Wales they were gradually removed from service and it appears there are now none in use.

Removal of accumulations

The use of a fan to remove an accumulation of gas from a roadway is not an unusual event. The essential features of such use are:

1. The area around the fan should not contain more than 1.25 per cent of methane.
2. The exhaust gas passing through the fan should contain less than 2.5 per cent methane.

It would be the duty of the deputy to ensure the area was free of gas and to arrange for the vent tubes to be separated at a convenient point so that enough fresh air entered the tubes to ensure the exhaust was kept below 2.5 per cent. The vent tubes inbye of the separation point carry high levels of methane and these are diluted by fresh air feeding in. When the fan stops the methane-rich flow will not stop immediately.

The Appin circumstances

The changeover from two intakes to one intake and two returns was an unusual procedure for Appin. It was not possible to establish whether each of the on-site officials was kept in touch with progress in the changeover period. It is clear however that the final stages of the changeover had been reached by the start of the evening shift on the day of the explosion.

On the previous shift the overcast in 3 cutthrough was completed and a temporary brattice stopping was erected in B heading outbye of C/T 3. It was planned that the brattice stopping in C/T 3 would be removed so that B heading would become a return. The deputy who was on-site did not remove the C/T 3 brattice because he became aware that there was leakage of intake air through the overcast. It was not possible to establish when or if the C/T 3 brattice was removed. It is obvious however, that until it was taken down B heading was almost completely unventilated.
The brattice for the stopping in B heading was found after the explosion to have been erected on the wrong side of the props. Assuming this did not affect its efficiency and that the leakage from the overcast was not as significant as suggested, removal of the C/T 3 stopping was all that was necessary to establish B heading as an operational return.

Once this was done the final step to be undertaken was the establishment of fan ventilation for the standing stub of B heading inbye of C/T 4.

The Deputy’s role

Apart from his responsibility to ensure safety in his panel the Deputy should have carried out the following actions before starting the fan:

1. Test for gas in the area around the fan.
2. Assess that there was an adequate flow of air down the newly opened return.
3. Arranged for the vent tubes to be separated so that when the fan was started the exhaust from the fan would contain less than 2.5 per cent methane.

Operation of the fan

It was accepted that the fan was started and was found to be running in reverse. In reverse the fan could still pass air in the correct direction but at a much lower volume (approximately 40 per cent of its rated capacity). Correction of reverse running can be achieved by disconnecting the power, opening the starter box and making suitable adjustments. It is, of course, important to reclose the starter box and ensure it is in a flameproof condition before power is restored.

Blast pattern

After extensive testing at Londonderry and elsewhere it was agreed that the blast pattern in the starter box could only have been formed by an internal ignition. Even though the cover was only held in place by one stud the explosion in B stub could not have caused the pattern in the box.

It was accepted that the trigger for the explosion was an ignition of methane that travelled inside the vent tubes. If the starter box provided the spark for the ignition then, apart from the fact that power was left on to the fan, the following events must have occurred:

1. The flow of air in B heading was insufficient to prevent recirculation (possibly even when the fan was running in reverse and passing a low flow volume).
2. The recirculation was undetected.
3. An explosive mixture was allowed to pass through the fan perhaps because the vent tubes had not been separated.
4. The cloud of recirculated air remained stationery around the fan and the starter box during the time taken to open the starter box.
5. Both the Deputy and the Undermanager, who were equipped with safety lamps, must have failed to notice or failed to act on the fact that their safety lamps were extinguished by the presence of the explosive mixture.
6. The starter button must have been pressed while the fan was not in a flameproof condition.

There is, however, one alternative possibility that is supported by the fact that B heading was left virtually unventilated for an extended interval. During the period before the C/T 3 brattice was removed gas continued to accumulate. The build-up may well have extended into the zone between C/T 3 and C/T 4 and also into the starter box. Under that condition the first application of power, that is when the fan was run in reverse, would have led to ignition in the box and could have led to formation of the blast pattern at a time when the box was in a flameproof condition.

After the explosion the body of the Deputy was found in the shuttle car. There is no logical reason for him to be there unless he was either on the boom of the miner or in the shuttle car controlling the gap in
the vent tubes and ensuring that the fan exhaust was carrying a low percentage of methane. It is possible that he did not realise he was in a spill of methane from the separated vent tubes and caused the ignition by attempting to relight his lamp. Withdrawal of the key can cause a spark sufficient to ignite methane in the lamp. This proposition is reinforced by the fact that the Deputy’s lamp was found without the relighter key. It is difficult to imagine how the force of an explosion could remove the key from a lamp if the key was fully home.

Inspectorial tolerance

For a number of years prior to 1979 the problem of maintaining intake gas levels at less than 0.25 per cent had been discussed with the management of the colliery. Intake quantities had been increased without solving the problem. Trials with methane drainage holes had been carried out but were not at that time considered to be successful in controlling the make of gas.

The preamble to the General Rules of the Coal Mines Regulation Act (NSW Government, 1978) contained the statement that those rules shall be observed as far as is reasonably practicable. It was in the spirit of that statement and in recognition of ongoing efforts to control gas-make that mining continued. Although an increase in intake gas levels was bound to make face ventilation more difficult there was never any attempt to accept higher than prescribed levels at the face or elsewhere.

When it became obvious, during the Judicial Inquiry, that inspectors had allowed levels higher than 0.25 per cent in the intakes some concern was expressed and Judge Goran referred to the matter as “inspectorial tolerance”.

Goran (1980) recorded two relevant and somewhat contradictory statements about that matter:

1. At page 87 ‘I hasten to say that the Minister and the Undersecretary were quick through Counsel to deny knowledge of the practice and to disassociate themselves from it’
2. At page 91 in reference to an exemption granted before the explosion ‘…the Minister saying that the quantities of methane being liberated in the colliery were giving him cause for considerable concern’

A letter to the manager requesting details of action he proposed to take to control the intake gas situation was included in a Departmental file. That file which also contained follow-up reports by inspectors was available to Counsel but was not presented to the inquiry. When asked about this, Counsel assisting the Inquiry indicated that though he was representing the Minister he was not representing Departmental officers. That fact, together with a further comment at page 87 by Goran (1980), puts into question the objectivity of the Department. The comment, by Goran, was:

‘I have received a document from Inspectors in the Department who are not coal mining Inspectors and are appointed under different legislation. They wish me to make it clear that they should in no way be confused with those Inspectors whose duty it is to enforce the Coal Mines Regulation Act.’

When notice of the Coronial Inquiry was received a letter signed by six Inspectors requested approval for one of their number to be their representative.

Approval was granted by the Coroner.

INQUIRY RECOMMENDATIONS

In a press release, the Minister for Mineral Resources, (Mulock, 1980), identified 27 recommendations from the Judicial Inquiry. Kininmonth (1981) listed 14 of the ones that may have had an immediate or major effect. It is appropriate at this time to consider an abbreviated version of all 27 with the Minister’s comments and an indication of the current position:

- Official circulation of specific material on limitation of explosions.

  Minister’s comment: Instructions given for booklets and films to be prepared and distributed.
Current position: A document Guideline for coal dust explosion prevention and suppression was prepared (NSW Department of Primary Industries, 2001).

- Reference should be made to overseas work in determining the adequacy of legislation dealing with fires and explosions.

Minister’s comment: Stonedusting and control of flammable gas will be given specific attention in the current review of the Coal Mines Regulation Act.

Current position: This matter is covered in the Coal Mine Health and Safety Regulation 2006 (NSW Government, 2006).

- All collieries should review whether their precautions against propagation of an explosion are sufficient.

Minister’s comment: Chief Inspector to commence an examination. If necessary, action will be taken to require management of mines to revise their precautions.

Current position: This is covered by Hazard Management Plans in Regulations 35 and 36 of the Coal Mine Health and Safety Regulation 2006 (NSW Government, 2006).

- Legislation should be strong in areas of known danger to attempt to prevent any foreseeable risk of an incendive nature.

Minister’s comment: The need for legislation to prevent any foreseeable risk of an incendive nature is indisputable.

Current position: A booklet titled “Preventing Frictional Ignitions” was made available to the industry.

- The gas problem will become apparent to most deep-mining projects. The Department should act to inspect and advise collieries on new techniques.

Minister’s comment: The department is considering the need to employ a “gas control engineer”

Current position: The Department does not have such a staff employee.

- General Rule 4 reports are vague in the extreme and give no real information as to actual gas conditions or ventilation.

Minister’s comment: Action has been taken on the revision suggested.

Current position: There is now no standard form.

- There should be some check upon the Deputy’s safety inspections. The ideal officer to perform this task is the Federation’s Check Inspector.

Minister’s comment: This issue will require detailed consideration. The parties concerned will examine ways in which the recommendation may be given effect.

Current position: Although there is no specific supervisory role in regard to deputies, the Coal Mines Regulation Act 1982 (NSW Government 1982) made provision for District Check Inspectors to suspend operations if they became aware of a condition of danger or a breach of regulations.

- The Deputy must be given a methanometer as well as his safety lamp.

Minister’s comment: I agree. Specific training programmes for the Deputies on the proper use of methanometers will be introduced.
Current position: Since the withdrawal of flame safety lamps all deputies are issued with multi-gas detectors.

- I would recommend the appointment of a ventilation officer perhaps with a part-time but prime responsibility whose duty it would be to supervise the whole question of ventilation in a mine.

Minister’s comment: Action is being taken to determine the legal mechanism to define the duties of such an officer.

Current position: All mines are required to have a Ventilation Officer and to have a biannual audit from a Ventilation Engineer

- The system of policing an Act designed to keep mines safe must be kept as tight as possible.

Minister’s comment: The act dealing with health and safety must be observed. It is unfortunate that it must be policed.

Current position: The Act and Regulations are enforced by Departmental Inspectors.

- All experiments (related to an Inquiry) should be departmentally official, permitted and conducted under the supervision of one director.

Minister’s comment: Any such appointment will be made by me.

Current position: The Department now has an Investigation Unit and there is provision in section 147 of the Coal Mines Health and Safety Act 2002 for the appointment of investigators (NSW Government, 2002).

- The dearth of competent Inspectors. Inspections sometimes are separated by months and then do not involve the whole mine.

Minister’s comment: I believe that there will continue to be difficulties in recruiting competent Inspectors because of the competing demands of an expanding coal mining industry.

Current position: It is apparently still difficult to recruit and retain Inspectors. Their functions are supplemented by the appointment of Mine safety Officers who have limited powers but do carry out audit and inspection duties.

- There appears to be a totally inadequate number of Electrical Inspectors.

Minister’s comment: Immediate action be taken in relation to this particular staffing matter.

Current position: Present staff levels appear to be satisfactory.

- No record of the result of an inspection (by a local Inspector) seems to be left at a mine.

Minister’s comment: The fact that no record of the results of inspections is left at the mine is an obvious deficiency in the current system and I have arranged for this matter to be corrected immediately.

Current position: Although reports of inspection are not made as a routine procedure any matter of concern is covered by a notice in writing.

- A duplicate copy of exemptions should be sent to the local check inspector.

Minister’s comment: I agree that the check inspector should be supplied with a copy of exemptions which may be granted in respect of the mine at which he is employed.

Current position: This is now done.
I feel that it is dangerous to raise the present statutory limit of .25%. If there is any tolerance, it should be limited to a low departure from the statutory provision and only given on written application to the Chief Inspector for exemption.

**Minister’s comment:** It is my intention that the present statutory limit should be maintained. If there is a situation where tolerance may be required, it will only be given on written application to the Chief Inspector.

**Current position:** This is now standard procedure.

I feel that it would be an advantage for collieries to be graded in terms of gassiness.

**Minister’s comment:** I have asked the Chief Inspector to determine what advantages would accrue from adopting the recommendation. I will require urgent action.

**Current position:** Gas monitoring is used to determine the standard of stonedusting to be used.

I specifically recommend that every lampman be supplied with an illuminated magnifying glass for the inspection of faults in gauzes.

**Minister’s comment:** The Department has commenced action towards preparation of a code for the use and maintenance of oil flame safety lamps.

**Current position:** this was done until the withdrawal of safety lamps.

In the more gassy mines there needs to be a requirement that an automatic monitoring device of a sufficiently portable nature be installed at strategic points to give a continuous reading of CH₄, CO and O₂.

**Minister’s comment:** The matter of automatic monitoring in gassy mines needs to be separated into two areas of concern. One is the matter of general monitoring of the mine atmosphere by way of automatic devices. The second is that of provision of an individual air sampler which can provide an automatic alarm to the person carrying the device.

**Current position:** Gas monitoring is widely used and Deputies carry automatic alarming instruments.

It appears that there is a grave danger in driving a lengthy stub in a gassy panel and leaving it stand on brattice ventilation alone particularly on the intake side. As a suggestion only I put forward the figure of 50 metres as the limit.

**Minister’s comment:** I have directed the Chief Inspector to investigate as a matter of urgency the manner in which the recommendation can be given effect.

**Current position:** The overall problem has been reduced by improvements in methane drainage. There is also some use of supplementary compressed air ventilation at times of power failure.

I have been asked to make a special recommendation that the Regulations make provision for not removing any (stone dust or water) barrier once it has been placed in position.

**Minister’s comment:** I have directed that a complete review be made of the present requirements in respect of stone dust and water barriers.

**Current position:** This matter is dealt with in the Guideline for coal dust explosion prevention and suppression. (NSW Department of Primary Industries, 2001).

There is need for proper policing of those parts of the Act and Regulations which deal with the opening of flameproof enclosures under voltage.
Minister’s comment: I have directed that all regulations dealing with the safety provisions of flameproof enclosures under voltage must be placed under continual review.

Current position: This matter was already covered by Reg. 31 of the 7th Schedule of the 1912 Act (NSW Govt., 1978). It was adopted into the 1982 Act and is now covered by Reg. 19 of the Coal Mines Health and Safety Regulations, 2006 (NSW Govt., 2006).

- Regulation by legislation is required for a breaker system to prevent any flameproof enclosure being opened without the automatic disconnection of power. This kind of device and the necessary circuitry are already known to the Departmental electrical inspectors and it should be an essential requirement.

Minister’s comment: I have no hesitation in agreeing with this recommendation.

Current position: Not all flameproof enclosures have interlocks. Electrical equipment is now required to meet IEEE Standards. It appears that those Standards do not insist on interlocks.

- Advocates addressing me have expressed dissatisfaction with the qualification of two distinct classes of mining people:
  i. the man with a certificate from abroad who receives endorsement of his certificate of competency without the necessity to show sufficient local competency;
  ii. the method of recruitment of “new starters” and their induction training.

Minister’s comment: There will be a requirement for all persons registered for practice in New South Wales to obtain a qualification in New South Wales mining practices and law. I have noted the comments made and particularly those attributed to the Miners’ Federation, that they consider the current training requirements to be inadequate. I support this view

Current position: There is now a mutual recognition scheme covering all Australian States and New Zealand. Holders of certificates from other areas are required to sit for the full relevant examinations.

- I am still concerned, however, about the discomfort self-rescuers cause to those who are forced to use them.

Minister’s comment: I have asked the Chief Inspector, in co-operation with the Director of the Chemical Laboratories of my Department to investigate the current status of research and development into this type of equipment overseas and whether or not it is possible by local innovation, to introduce an improved model.

Current position: Most mines now provide oxygen producing self-rescuers.

- It would be a pity, however, if experimenting remained the sole use of the Department’s research and testing centre at Londonderry, even though this ranks highest in the galleries importance. It should be used as a teaching aid for men in the industry.

Minister’s comment: For some years the department has run training sessions for officers from the coal mines in this state. During these courses the facilities at the Chemical Laboratories and the Londonderry Centre have been used as a training and demonstration aid. Their use will continue.

Current position: Londonderry is now not part of the Department. It is used as a central meeting point but not for training purposes.

- The Judge referred to separate comments, which he made privately on the draft of the legislation designed to replace the existing Coal Mines Regulation Act.

Minister’s comment: A number of these comments have already been referred to my Department for consideration.
Current position: It is impossible to comment on such private submissions.

THE ROLE OF INQUIRIES

Goran (1980) in the report of his Inquiry made the following statements as part of his preliminary observations:

1. Most inquiries into fires, explosions and disasters in coal mines which occur in the United Kingdom are non-judicial inquiries, the investigation itself being conducted at a senior administrative level. The resultant report takes the form of a description of the colliery and the relevant equipment, followed by a narrative setting out the events. Conclusions are in short form and there is a list of recommendations for future improvements in coal mines generally.

2. I make no criticism of this method of reporting. The report which I now tender, however, is the report of a Court which has investigated an explosion. The conclusions drawn are based upon a mass of evidence tendered before me and tested in the greatest detail by the cross-examination of learned counsel.

3. I have adopted standards of proof as any Judge must do, accepting some evidence and rejecting other evidence at times which experience in judgment has told me I cannot accept.

4. It must always be borne in mind that the duty of this Court is to assist in so regulating the industry that events such as that which occurred at Appin Colliery are not repeated and that men who work in this industry should do so with such safety as can be afforded to them by legislation or by proper practices.

There is no doubt that many of the recommendations of the Appin Inquiry have led to significant changes in the NSW coal industry. Since 1965 there have been four Judicial Inquiries in NSW; Bulli, West Wallsend, Appin and Gretley. Looking at these from an industry perspective, there are valid questions that may be asked about the economic cost of such inquiries and whether legal representation is given on the basis of a search for the facts or in an attempt to protect clients.

Carver (1981) speaking from his experience as a retired Chief Inspector in Great Britain said:

‘Some twenty years ago the representatives of the Unions, Management, and Inspectorate decided they would not employ lawyers in future Inquiries and that each interested party would appoint someone from within its own ranks to present its case and cross-examine witnesses. The reason for this interesting decision was to keep the Inquiry on a purely technical basis without the introduction of legal niceties and even ‘red herrings.’

Hargraves (1996) in a letter to the Bulletin of The AusIMM, included the following comments:

1. ‘As expected with their background of generally adversarial work, the thread of the Moura No2 Explosion Inquiry conducted essentially by members of the legal profession had elements of laying blame and appearing to attempt to have some witnesses contradict themselves.’

2. ‘Inquiry barristers have been known to withhold evidence ‘because it will not help our case’. So much for the threefold aim of, how, why and non-repetition. There are some in the Industry who consider that the careers of others have been unfairly hurt entirely due to partial ignorance behind some attitudes and decisions of legal participants.’

3. ‘Perhaps it is time to consider more seriously a purely technical inquiry on the UK pattern,.

It should be noted that whereas the Coal Mines Regulation Act 1912 as amended (NSW Government, 1978) made provision for Judicial Inquiries the Coal Mine Health and Safety Act 2002 (NSW Government, 2002) in section 113 makes provision for a person (obviously, including a Judge) to be appointed as a Board of Inquiry. Despite the fact that, under section 114 (9), evidence given by a witness is not admissible in any criminal proceedings, there is still the following provision of section 113 (5):

...
'If the Board of Inquiry agrees, an agent (including a legal practitioner) may represent a person or body at the inquiry'.

It is not clear how involved that representation could be.

CONCLUSIONS

A disaster, such as that at Appin, has a devastating effect on the mine and the mining community. It can only be small compensation for those affected to be given an understanding of how the event occurred but it is important to give assurance that the cause is understood and that action will be taken to prevent a recurrence.

The current legislation provides for use of Boards of Inquiry but does not remove the possible involvement of legal representation. The fact that many recommendations from past Inquiries have led to improvements in mine safety does not alter the feeling of many in the industry that legal representation sometimes prevents the achievement of believable conclusions about the cause of disasters. It may be time to look, in retrospect, at the Appin disaster and to ask if a technical investigation would have given a clearer understanding of the event and led to better procedures being adopted.

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DESIGNING EXPLOSION RATED VENTILATION SEALS FOR COAL MINES USING HIGH-FIDELITY PHYSICS-BASED COMPUTER MODELLING

Ian Verne Mutton\(^1\) and Alexander Remennikov\(^2\)

ABSTRACT: Questions have been raised about the effectiveness of ventilation control devices (VCDs) to safely resist explosions during their intended life. This functionality depends on the ability of the VCDs and in particular seals to withstand changes in the behaviour of the strata, particularly where longwall abutments influence the stress regime in and around the chain pillars. As a consequence of an explosion impact on a seal, the surrounding strata could experience increased loads possibly resulting in permanent deformation and requiring grout consolidation. These aspects of seal design have been investigated using advanced numerical analysis.

Globally since the early 20\(^{th}\) century, to protect underground personnel, ventilation seal designs have been required to be tested at an internationally recognized explosion test gallery to achieve pressure ratings required by legislation. The last two decades has seen advances in materials technology and engineering of structures. It has become accepted practice to use numerical methods to provide engineering ratings for mine seals in line with other industries where the elimination of prototype testing provides more rapid product introduction to the market. Before presenting the results of numerical analysis, structural aspects of seal design are simply explained including arching behaviour and the contribution of dynamic magnification due to impact loads.

High-fidelity physics-based computer simulations using software LS-DYNA were able to predict the results from physical testing of mine based seals in a most realistic way. Test data from live gas/coal dust deflagration explosions at Lake Lynn, PA, USDA along with pressure-time curves recently developed by the National Institute of Occupational Safety and Health as a result of the study of explosive atmospheres, were used to simulate a realistic loading environment caused by 138 kPa (20-psi) and 345 kPa (50-psi) explosions in physics-based models of seals.

INTRODUCTION

Explosions of gases and of coal dust have always been a basic hazard in coal mines and to this day continue to be the cause of disasters in coal mines. The advancement of knowledge in seal design and construction has tended to be driven by these disasters. In response to the alarming number of fatal explosions and fires in U.S underground coal mines the Bureau of Mines was set up on July 1\(^{st}\), 1910 (Tuchman and Brinkley, 1990) and likewise in Poland, Experimental Mine Barbara conducted live tests on mine seal designs typically constructed in coal mines since 1925. Various experimental mine facilities around the world conducted live explosion tests in the absence of mathematical models that could adequately describe seal response to such explosions. There was also no means to physically measure and define seal response to real time explosion impulses. It was in 1930 that experimental work involving measurement of seal response to explosions by the U.S Bureau of Mines started an understanding structurally of what influenced the performance of ventilation seals when subjected to an explosion overpressure.

It is important to define what attributes a seal requires before discussing the finer details of structural design and load bearing capacity. During the normal course of underground coal mining, it sometimes becomes necessary to install permanent seals to isolate abandoned or worked out areas of the mine. This practice eliminates the need to ventilate those areas. Seals may also be used to isolate fire zones or areas susceptible to spontaneous combustion. To effectively isolate areas within a mine, a seal should

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control the gas air exchanges between the sealed and open areas to prevent toxic and/or flammable gases from entering active workings and oxygen from entering the sealed areas.

be capable of preventing an explosion initiated on one side from propagating to the other side, and

continue its intended function when subjected to a fire test incorporating a specific (AS 1530.4-1990) time-temperature heat input (Tuchman and Brinkley, 1990).

The 1994 explosion at Moura No. 2 Mine renewed the focus on Ventilation Control Devices (VCD) within Australian coal mines with closer examination of the design and construction of seals. Prior to the enactment of new regulations on 16 March, 2001 in Queensland, introduction of the Queensland Mines Department Approved Standard for Ventilation Control Devices provided prescriptive ratings for seals and stoppings and required live testing of seals and stoppings in an “internationally recognized mine testing explosion gallery”. As part of the enormous amount of research undertaken at this time after the recommendations of Task Group 5 (Oberholzer and Lyne, 2002) in establishing practical design criteria to assist mining engineers to minimize the risks of seal failure, Tecrete Industries introduced explosion rated shotcrete based Meshblock seals with an overpressure capacity of 138 kPa (20 psi) and 345 kPa (50 psi). Gateroad seal design more or less conformed to seal ratings used in the United States since 1971 where it was stated in 39 CFR 75.335 (Mine Safety and Health Administration – Title 30 Code of Federal Regulations, 1997) requires a seal to “withstand a static horizontal overpressure of 138 kPa (20 psi). Previous research by the former U.S Bureau of Mines (Weiss et al, 1999) indicated that it would be unlikely for overpressures exceeding 138 kPa to occur very far from the explosion origin provided that the area on either side of the seal contained sufficient incombustible and minimal coal dust accumulations.

Recent tragic accidents at Sago, WV and Darby, KY Mines in 2006 caused by methane explosions behind sealed off areas brought the issue of safety of mine seals to the attention of regulatory authorities. Following the enactment of the 2006 Miner Act and MSHA’s issuance of the emergency temporary standard (ETS) more stringent performance standards have been adopted for mine ventilation seals. There is a minimum standard of 345 kPa (50 psi) (designed, constructed and maintained) for a specific pressure-time curve, when the atmosphere inside the sealed volume is monitored and maintained inert. In the United States more commonly pressure rated seals have a capacity for 827 kPa (120 psi) in line with the findings of the NIOSH study entitled, “Explosion Pressure Design Criteria for New Seals in U.S Coal Mines” (Zipf, Sapco and Brune, 2007). The findings of this report have challenged globally established beliefs in seal design and explosion propagation.

It is in the light of these stringent new standards and questions asked by mine operators in Australia, that the design of Minova’s Meshblock 138 kPa (20 psi) and 345 kPa (50 psi) overpressure rated shotcrete seals were investigated early in 2009 using computer based numerical simulations to investigate such complex phenomena as behaviour of seals under explosion loads and the effect of strata convergence on explosion rating. Engineer designed steel access hatches are used for degassing purposes in some Australian mines and the effect of these hatches on seal integrity is also investigated. It is now normal practice in most industries to use numerical methods for the design of critical structures although the integration of software in the design process has a long way to go. A recent example is the Boeing 777 which was digitally designed using 3D solid modelling technology that included integrating spatially three million parts with CAD software and finite element modelling of components.

STRUCTURAL DESIGN CONSIDERATIONS FOR VENTILATION SEALS

There are several existing simplified methods that can be used to provide ventilation seal design. However only high-fidelity physics-based computer simulations are able to predict the results from physical testing of mine based seals in a most realistic way. Explosion testing is still extensively used to test existing designs and NIOSH’s relatively new hydraulic test facility provides a cost effective method to develop stress-strain response data using a water load as an alternative to full-scale (Sapco, Harteis and Weiss, 2008) explosion testing. In order to provide a “fit for purpose” seal design the conditions that the seal will be subject to for its intended life must be defined.
IMPOSED STRESS CHANGES ON VENTILATION SEALS

- Imposed stress changes that may affect the structural integrity of the seal (and surrounding strata) and hence its ability to safely resist an explosion load are as follows:

- Gate road seals are subject to changing longwall abutment loads during coal extraction as the face moves past the seal located in the chain pillar cut-through. The roadway periphery and hence the seal is subject to convergence conditions and increased vertical loads due to dilation of coal mine strata.

- Chains pillars experience increased vertical load and lower stiffness coal plies can be crushed within the chain pillar increasing stresses within the seal material.

- Aquifers that are breached by caving during extraction and water from the longwall equipment can flood the seal. Water will leak through the path of least resistance. This water could pass across the boundary between the seal and the enclosing strata, through a porous seal or through the surrounding coal mine strata along cleats, joints and bed separation. At certain pressures it is possible that surrounding plies could be hydraulically separated providing a leakage path.

- The existing primary and secondary support affects the load that the seal experiences due to convergence.

- Mobilization of joint sets and fault planes due to mining induced stress changes.

Considerations of structural behavior

The U.S. Bureau of Mines (Rice, Greenwald and Howarth, 1930) conducted a series of explosion tests and found that restraining the edges of a seal caused a dramatic increase in seal strength to a level much higher than that predicted by plate theory. As the seal experiences an explosion load, it bends and pushes outwards on the surrounding strata. The development of this strength-enhancing mechanism will depend on a number of parameters including the stiffness and strength of shotcrete material and mine strata, seal thickness, and the height of crosscuts.

Basically there are two structural engineering approaches to designing seals to resist explosions, two – way arches [described in this paper] and plug-type failure, with both possible failure modes dependant on the structural reaction of the surrounding strata. The arching mechanism for wall behaviour is most applicable when the wall thickness to wall height ratio [T:H] ranges from 1/15 to ¼ (Zipf, Sapco and Brune, July 2007) and fits in with Meshblock seal thicknesses that ranges from 200-600 mm. For lower T: H ratios a flexural failure mode applies and for higher ratios (found in plug-type seals) a shear failure mechanism along the roadway contact is more applicable.

Arching behaviour in shotcrete seals

The arching mechanism (Refer to figure 2) is thoroughly described in study undertaken in an ACARP (Pearson et al, 2000) report which described the structural response of explosions on 325 mm thickness Meshblock seals. As the seal is subject to increasing horizontal loads a compression arch forms within the thickness of the seal. A compressive stress is imposed on the surrounding strata. The strength of the seal is limited by the crushing strength of the shotcrete and the response is essentially independent of the steel reinforcement. As the seal is increasingly loaded, tensile cracks form along yield lines (see Figure 3) to essentially form plastic hinges. The intact shotcrete is initially in the form of a shallow arch. As the load increased the cracks deepen increasing the depth of the arch and essentially increasing the compressive forces on the intact shotcrete until compression failure occurs.

The arching mechanism will not develop if the roof and floor rocks have very low stiffness and strength and lower strength bending will predominate. The formation of plastic hinges is shown from test work at NIOSH's Lake Lynn laboratory in 1997 where crack propagation lines are shown after a seal has experienced an explosion. Crack propagation commences at the seal centre and migrates outwards in a horizontal line and up into the corners in the pattern shown.
Meshblock seals are practically unreinforced when compared to, for example, concrete slabs which normally require a mesh of reinforcement on the tension side. The vertical and horizontal bolts were installed at mid-section of each seal, hence not contributing to its flexural strength significantly. In spite of such arrangements, the obtained capacities are many times larger than those obtainable in reinforced concrete slabs designed in accordance with modern codes of practice (e.g. AS3600-2009). For such (and larger) seal height: seal width ratios it is conservative to assume that the applied pressure is carried entirely by strips spanning in the vertical direction. The attainment of large ultimate capacities for such strips can be attributed to development of significant lateral restraints $H$ (see Figure 4(a)) exerted by the mine strata. Figure 4(b) shows that the lateral compressive force $H$ on the cross-section at mid-span significantly reduces the tensile stresses caused by bending, hence delaying the initiation of cracking.

The static strength of 325 mm Meshblock seals was determined by ACARP study (Pearson et al., 2000) using numerical modelling and compared with data from the Lake Lynn Experimental Mine (LLEM) explosion tests undertaken by Tecrete Industries in conjunction with BHP Coal in 1997. Figure 5 shows the ultimate capacity of the 2.74-m high seals from calculations and shows good agreement with the LLEM test results. In the example considered by Pearson et al. (2000), the seal deflection of 6 mm at the mid-point translated to only 1.3 mm between roof and floor at the arch supports. These results will be later verified with recent 138 kPa (20 psi) and 345 kPa (50 psi) Meshblock seal design using LS-DYNA software.
One of the approaches to predicting response of mine seals to explosion loads is based on an equivalent dynamic modelling technique. Equivalent dynamic modelling is based on the fundamental premise that the structural response obtained by conducting a pure static analysis, with applied load increased from the original level by a certain factor, will be identical to a full dynamic analysis of the same structure conducted with the actual load. The magnification factor used for such analysis is called the Dynamic Load Factor (DLF).

Currently, the DLFs used in coal mine seal designs are derived from Single-Degree-of-Freedom (SDoF) models assuming elastic behaviour. When structural response is assumed to be elastic, then the theoretical maximum DLF is 2.0 (for triangular load pulse with zero rise time). Therefore, most of the “equivalent” dynamic models use a factor of 2 to increase the peak dynamic load before conducting static analysis on a mine seal. Because multiple modes of structural response could contribute to the response of seals, the DLFs estimated from SDoF models may not be applicable for coal mine seals. Furthermore, the worst-case DLF = 2 corresponds to elastic behaviour of the seal. When a portion of
the seal gets damaged and becomes plastic or softens due to concrete failure, the elastic DLFs may not be conservative for predicting the response of seals.

Figure 6 presents a maximum response of an elastic SDoF system for a triangular explosion pulse with finite rising time. This shows the range of dynamic magnification responses that explosion test pressure-time (P-T) curves at LLEM could produce. $T$ is the rise time of the explosion and $T_n$ is the natural period of the seal and it is seen that when the ratio is close to 1 the maximum impact and acceleration of the seal occurs.

The data from explosion testing of Meshblock seals (Weiss et al, 1999) also indicated that in some cases the recorded times at peak pressures did not match the times of seal failure. This signifies the need for time-history dynamic analysis, an important aspect of this study. Further examination of the pressure time histories reveals an impact-type input of energy to some seals when comparing their natural periods in bending response (computed in the pre-test round of analyses) with the duration of pressure pulses. In such cases the dynamic response amplification (e.g. maximum seal displacements) will depend on the ascending and descending portion of the pressure-time diagram.

**HIGH-FIDELITY PHYSICS-BASED MODELLING OF SEALS**

**Explosion pressure-time curves for seal analysis**

The test data from live gas/coal dust deflagration explosions at Lake Lynn, PA, USDA can be used to simulate a realistic loading environment caused by 138 kPa (20 psi) and 345 kPa (50 psi) explosions in physics-based models of seals. Figure 7 presents examples of 134 kPa (20 psi) and (50 psi) 345 kPa experimental pressure-time curves that can be used for dynamic analyses of seals.

Figure 7(b) also includes the 345 kPa (50-psi) design pressure-time curve, recommended by NIOSH (Zipf et al, 2007) with the rise time (time to reach peak pressure) of 0.1 sec. From the comparison with the experimentally derived curve (Lake Lynn Experimental Mine) in Figure 7(b), one can observe that the design curve is characterised by the pressure rise rate that is more conservative than indicated by experimental gas explosions. In this paper only the 20 psi experimental curve is used to analyse the example 300-mm thick Meshblock seals.
Finite element analysis software

LS-DYNA, a general purpose transient dynamic finite element program (LS-DYNA, 2008) was used to develop the finite element models in this study. LS-DYNA is used to solve multi-physics problems including solid mechanics, heat transfer, and fluid dynamics either as separate phenomena or as coupled physics, e.g., thermal stress or fluid structure interaction. LS-DYNA is an industry accepted dynamic first-principle based code for analysis of structures under extreme loads generated by blast and impact events with the ability to compute large deformations due to flexure, shear, and material failure.
Model description

As an example, the shotcrete seal which is 3.4 m high and 300 mm thick is analysed in this paper. Due to the symmetry of the seal, the boundary conditions, and the loading about the central vertical plane, the model includes only one half of the seal allowing for a model width of 2.7 m. The model includes roof and floor skeleton bolts (650 MPa steel) of 21.7 mm diameter that are placed at 600 mm centres around the periphery. The 200-mm deep rib keys are modelled for 300-mm thick seals. The rib keys are modelled with a single row of 1200 mm long bolts with 600 mm tails protruding and 600 mm full encapsulation.

To simulate the seal-rock interfaces, floor, ribs and roof are explicitly modelled as large solid bodies surrounding the seal. The overall thickness of the floor and the roof in the model is 2.5 m. The Meshblock seals have 1.8 metres of coal in the roof and 0.6 metres of coal in the floor. The remaining depth is filled with the rock materials. Figure 8 shows the components of the seal model used in this study.

In the finite element model, solid elements with a single integration point were used to model the shotcrete seal and the surrounding coal and rock materials. Overall model dimensions and the sizes of finite elements were determined from a mesh convergence study. The mesh convergence study included a number of runs of the model with variable model dimensions and increasing levels of mesh refinement. In the final model, the concrete seal was modelled with 50-mm cube solid elements, and the surrounding rock was modelled with 250-mm cube solid elements.

Beam elements were used for the skeleton bolts in the ribs, roof and floor. Each beam element shared two of the solid element nodes to model the strain compatibility between the steel and the concrete. As a result, slip between the steel reinforcement and the concrete was included explicitly in the model. Slip occurs as a function of the failure of the concrete attached to the reinforcing bars. Reinforcing bars were extended 600 mm into the ribs, roof and floor to provide sufficient anchorage length. The bond between the steel bars and the rock was modelled using constrained conditions provided by LS-DYNA for connecting meshes of dissimilar densities.

Figure 9 shows the finite element model of the rib keys. The rib key is modelled by extending the concrete seal model into the body of the coal ribs. Interaction between the key and ribs is simulated.
using surface to surface contact surfaces. The full model of the seal consists of 127,050 nodes, 336 beams, and 114,000 solid elements.

**Figure 9 - Modeling the keys for the ribs and the skeleton bolts**

**Material Models**

The concrete model employed for modelling the shotcrete seal was model 159 in LS-DYNA implemented in keyword format as MAT_CSCM_CONCRETE for Continuous Surface Cap Model. The model formulation includes a smooth and continuous intersection between the failure surface and hardening cap. The model includes isotropic constitutive equations, yield and hardening surfaces and damage formulations to simulate softening and stiffness reduction. A rate effects formulation increases strength with strain rate. The model has been thoroughly tested by several US Governmental agencies (Murray and Lewis, 1995; Murray, 2007) for predicting damage in concrete under severe impact and blast loads, which has demonstrated its reliability and accuracy. Default input values for model parameters were used in this study. Default material parameters are generated by the model based on the specification of the unconfined compression strength. In this study, the unconfined compression strength of 50 MPa was used based on the test data from testing of Hanson shotcrete in Queensland.

Roof, floor and ribs were modelled using Material Type 173 based on Mohr-Coulomb criterion in LS-DYNA. The material has a Mohr Coulomb yield surface, given by \( \tau_{\text{max}} = C + \sigma_n \tan(\phi) \), where \( \tau_{\text{max}} \) = maximum shear stress on any plane, \( \sigma_n \) = normal stress on that plane, \( C \) = cohesion, \( \phi \) = friction angle. The tensile strength is given by \( \sigma_{\text{max}} = C / \tan(\phi) \). After the material reaches its tensile strength, further tensile straining leads to volumetric voiding. Material 173 is intended to represent soils, rock and other granular materials.

The appropriate material modelling parameters for roof, floor and ribs are summarised in Table 1 for the boundary roadway conditions investigated in this study. It should be noted that coal mine strata are variable in geomechanical properties with adjustments required when considering bulk properties as compared to laboratory test results of intact cored specimens. Coal shows (directional) compressive strength variations due to variable cleat, moisture and gas content changes, stone partings, varying macerals shown in laminae found in a vertical seam section and changing ash content. Table 1 material properties represent values that have been used when modelling mine strata for ground support and chain pillar design.
### Table 1 - Material properties for models of roof, floor and ribs

<table>
<thead>
<tr>
<th>Boundary Roadway Condition</th>
<th>Material</th>
<th>Young’s Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
<th>Friction Angle (deg)</th>
<th>Cohesion (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>Coal</td>
<td>3,000</td>
<td>0.4</td>
<td>30</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Stone</td>
<td>5,000</td>
<td>0.2</td>
<td>35</td>
<td>5.0</td>
</tr>
<tr>
<td>Floor</td>
<td>Coal</td>
<td>3,000</td>
<td>0.4</td>
<td>30</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Stone</td>
<td>5,000</td>
<td>0.2</td>
<td>35</td>
<td>5.0</td>
</tr>
<tr>
<td>Ribs</td>
<td>Coal</td>
<td>3,000</td>
<td>0.4</td>
<td>30</td>
<td>1.0</td>
</tr>
</tbody>
</table>

**Predictions of response of the 300-mm Meshblock seal to 20-psi explosion loading**

Based on the finite element model shown in Figures 8 and 9, and the loading and material properties described above in this section, non-linear transient dynamic analyses were carried out for the example Meshblock seal design. Crack patterns for the seal are visualised using the contours plots representing damage levels from zero to one calculated by the concrete model. A contour value of zero indicates no damage, so concrete strength and stiffness are those originally specified as input values. A contour value of one indicates maximum damage and severe cracking, in which the concrete strength and stiffness are reduced to zero.

Predicted crack patterns in the 300-mm Meshblock™ seal at about 1.0 sec after the explosion are shown in Figures 10 and 11. The seal displaces laterally about 6 mm at about 0.1 sec explosion duration and reaches residual permanent deformation of about 2 mm at about 1.0 sec, as shown in Figure 12. It can be noted that the concrete damage is mainly located on the outbye side of the seal (non-impact side) and is characterised by tensile cracks forming a typical yield line pattern characteristic of the rectangular panels with all four edges simply supported. This result confirms that the seal responds as a two-way slab where the keys, the bolts and the interface friction provide effective supporting boundary conditions to the seal. Damage contour values between 0.5 and 0.8 indicate that the concrete strength and stiffness along the damage regions have significantly reduced but the level of damage is not severe. Moreover, no elements have eroded in the calculations. This indicates that the overall integrity of the seal was maintained after being exposed to the 20-psi explosion loading. Crack pattern shown in Figure 3 for the similar Meshblock seal design explosively tested in the Lake Lynn Experimental Mine in 1997 provides experimental validation of the numerically simulated results for the 300-mm seal example.

Closer examination of the computed results indicates that the rib keys play a significant role in the response of the seal to blast loads. Figures 13 and 14 show principal compressive stress and maximum shear stress distributions in the coal ribs near the keys. It can be noted that the ribs experience large bearing stresses with the maximum value of about 3.9 MPa at the mid-height level of the seal. From Figure 14, the maximum shear stress in the ribs near the keys is about 1.4 MPa which exceeds the shear strength of coal of 1.0 MPa. Large shear stresses extend up to 120 mm into the rib.

Figure 15 shows the contours of maximum shear stresses in the roof and floor strata. The maximum shear stresses reach about 1.0 MPa in the roof and floor within the area of about 150 mm wide on both sides of the seal and about 120 mm deep. The results of high-fidelity physics-based analyses can be used to support the grouting program for seal construction if required. They can provide a justification for Polyurethane or cement grouting of the potential yield zones in order to increase compressive, tensile and shear strength of the rib, floor and roof materials in the immediate strata.
Figure 10 - Concrete damage contours on the outbye side of 300-mm Meshblock seal (load was applied to the inbye surface)

Figure 11 - Concrete damage contours on the inbye surface of a 300-mm Meshblock™ seal
Figure 12 - Time-history of peak deformations of a 300-mm Meshblock™ seal

Figure 13 - Principal compressive stress distribution in coal rib

Figure 14 - Maximum shear stress distribution in coal rib
A history of coal mine disasters at the beginning of the 20th century which saw increasing production rates and mechanisation, led to an increased research effort into the protection of underground miners from the explosive potential of coal dust (stone dust applications coal dust control)and gas concentrations. In the absence of advanced structural engineering techniques and materials science knowledge many researchers around the globe concentrated on live testing of VCDs using controlled explosions. Despite advances in understanding structural engineering aspects of seal design (Rice, Greenwald and Howarth, 1930), it is only in the last two decades that attempts have been made to physically measure seal response to explosions and to simulate seal behaviour with advanced structural techniques. Explosion testing still seen as an important safeguard for proving seal ratings is now used in conjunction with numerical methods using computer based tools that have been developed for other industries using well understood construction materials. Queensland Mines Department Approved Standard for Ventilation Control Devices was introduced in 1996, and during this period several generic ventilation seal systems were being introduced into coal mines, some being unique to Australia.

One such seal, Meshblock, introduced into Australian mines in 1994 is constructed of cement based shotcretes. Meshblock has been subjected to explosion test programs with outcomes previously summarised in an engineering model, however new questions such as the effect of strata convergence on explosion rating need to be answered.

In this paper, a high-fidelity physics based finite element model for the explosion rated Meshblock ventilation seals was developed. The model is suitable for computing dynamic responses of ventilation seals in coal mines subject to explosion loading. The seal model includes the concrete material model that incorporates many important features of concrete behaviour, such as tensile fracture energy, shear dilation, effects of confinement, and invariant failure surfaces. Damage metric is used to gauge the evolution of the concrete’s behaviour from elastic to elasto-plastic, and to softening or fracture.

Numerical modelling and simulation of the explosion rated ventilation seals can be undertaken in stages to determine their resistance to explosion loads, the combined effects of explosion loads and roof to floor convergence and finally to establish the ultimate capacity of ventilation seals and their overall response. Detailed investigation of the interface stresses between the seal and the surrounding strata can provide important information for the grouting program for seal construction.

ACKNOWLEDGMENT

The authors wish to thank Minova Australia for permission to present this paper and gratefully acknowledge the assistance and advice provided by various mine operators, and consultants during the defining of likely seal boundary conditions preceding numerical analysis.
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ADVANCED NUMERICAL MODELLING METHODS OF ROCK BOLT PERFORMANCE IN UNDERGROUND MINES

Chen Cao¹, Jan Nemcik¹ and Naj Aziz¹

ABSTRACT: Current developments in numerical modelling techniques are presented to investigate the load transfer between the steel bolt surface and resin encapsulation. Underground measurements of steel bolt performance indicate that amongst many parameters bolt profile configuration plays an important role in load transfer capacity between the bolt and resin that encapsulates the steel bolt. The short encapsulation pullout tests of roof bolt capacity can be successfully modelled and directly compared to the insitu pullout tests. Using the numerical modelling techniques, changes in the bolt profile can be studied in detail to provide a better understanding of the bolt reinforcement mechanism. In particular, emphasis is placed on the bolt profile geometry and its influence on the load transfer characteristics. Even though this work does not discuss any results, it is intended to outline the procedures and methods employed as part of the continuing research at the University of Wollongong.

INTRODUCTION

During mining, stresses and displacements of strata are constantly changing. Stress conditions in strata just ahead of the coal face typically exceed the rock strength and initiate fractures that lead to strata displacements and typically need steel bolt reinforcement. Over the past two decades, there has been a growing interest on the application of numerical modelling to bolt/resin/rock interaction with the aim of better understanding of the load transfer mechanisms for effective strata reinforcement around underground excavations. Blumel, Schweiger and Golser (1997) carried out numerical simulation of the bolt load transfer characteristics with the main aspect of the analysis being to investigate the difference in the bolt behaviour versus the rib geometry and in particular the spacing between the ribs. The numerical simulation was based on using finite element mesh to study the load transfer mechanisms which was aimed to be incorporated in future interface modelling. Further studies by Aziz and Jalaifar, (2007, a and b) extended the work to include modelling of bolt profile configuration under axial and lateral loading conditions. Aziz and Jalaifar simulated short encapsulation pull and push tests and compared the results with the laboratory and field tests. The work presented here outlines the refined techniques available to conduct sensitivity studies on various bolt rib profiles and their spacing to enable selection of the optimum bolt profile geometry.

NUMERICAL MODELS OF MINE EXCAVATIONS

As part of the numerical models that are extensively used to simulate mine excavations (FLAC, UDEC and other available software packages), standard rock bolt reinforcement can be routinely modelled to study the strata performance. Simplified assumptions of bolt reinforcing capabilities are used in these models, requiring bolt properties as the data input. Typical rock bolt elements with axial and bending behaviour are available within the ITASCA software to model steel bolts for ground reinforcement. As described in the FLAC manual, the shear behaviour of the modelled grout annulus is represented as a spring-slider system located at the nodal points along the reinforcing member. The maximum shear force developed in the grout is a function of the cohesive strength of the grout and the confining stress-dependent frictional resistance of the grout (Itasca, 2005). The effective confining stress that develops normal to the rock bolt depends on the bolt profile and other parameters such as the grout annulus size.

The routine bolt functions available in these packages cannot be used to study the effectiveness of the bolt profiles. In order to successfully determine the optimum bolt profile geometry it is essential that only a small part of the bolt-resin-rock is modelled and appropriate sensitivity studies are carried out to evaluate the optimum load transfer between the bolt and the resin/strata interface.

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NUMERICAL MODELLING OF BOLT PROFILES

To maximise the load transfer between the steel bolt and the surrounding rock mass, detailed investigations of interaction between the bolt profile, grout and the rock interface need to be undertaken. The in-situ pullout tests of various bolt profiles indicate that the bolt profile and rib spacing have a significant influence on the bolt to ground load transfer (Blumel, 1997 and Aziz, 2007). Such field results are depicted in Table 1 while a typical bolt rib spacing is shown in Figure 1.

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

In order to optimise the bolt profiles with respect to their load transfer capabilities, sensitivity studies of various profiles and their spacing need to be undertaken. To depict an accurate geometry of the bolt profile embedded in the resin, fine 3-dimensional mesh of suitable elements needs to be constructed. An example of the generated fine mesh for detailed simulation is shown in Figure 2.

Minute changes in the bolt profile geometry can be studied and various parameters such as the location, magnitude and type of the grout failure, displacements and stress distribution recorded. The final outcome of such study is to gain new knowledge how the change in bolt profile geometry and spacing influence the grout failure and what mechanisms contribute to achieving the optimum load transfer between the steel bolt and the surrounding strata. The modelled results can then be compared to the laboratory and field trials.

To study the bolt profile embedded in the resin and its influence on the load transfer between the bolt and the resin, a numerical model with a very fine 3-dimensional mesh was constructed using the 3-dimensional ANSYS finite element method software (ANSYS USA). The model showing the details of the bolt profile, resin annulus and the surrounding rock can be seen in Figure 3.
Figure 2 - Generation of the fine mesh for detailed simulation of rib profiles

It is envisaged that several different software packages such as 3-dimensional FLAC, UDEC, PFC, ABAQUS and other finite element models may be trialled for comparison and to ensure accurate assessment of the bolt-grout system behaviour. The sophisticated laboratory tests must be conducted to understand the grout post failure behaviour in detail. To simulate the exact behaviour of small granular particles within the failed grout zone is challenging and may require skilful programming.

To optimise the bolt profile spacing with respect to the maximum achievable load transfer and to understand the reason how the bolt profile spacing influences the load transfer, it is necessary to look at the details of resin failure as it occurs during the simulated pullout test. To replicate the insitu grout behaviour in the numerical model, the sophisticated laboratory tests must be conducted to understand the grout post failure behaviour in detail. The triaxial tests are needed to measure the intact and post failure properties of resin under various loading conditions including Young’s Modulus and Poisson’s Ratio, compressive and tensile strength, cohesion, angle of internal friction, grout bulking at various points of loading and failure stage and a record of detailed stress strain relationship.

The load transfer is proportional to the compressive stress that develops normal to the failed surface located either at the bolt-resin boundary or within the resin itself. An increase in normal stress perpendicular to the sliding surfaces caused by the bolt profile displacement, bulking of the failed resin and location of the failure zones holds the key to the load transfer.

The drilled hole profile in rock strata is usually rifled with spiral grooves between the resin and surrounding rock. This profile is desirable and it becomes particularly important when considering load transfer in weak strata. Poor cohesion that may develop between the resin and the host strata is aided by the wedging mechanism of an uneven hole diameter and surface roughness. In a similar manner, the numerical modelling is useful to predict what hole profiles are desirable in weak strata to minimise poor load transfer between the strata and the resin surface.

Future work may include the detailed model of the bolt system that can also be used to maximise the quality of the resin mix and encapsulation. The model can simulate the dynamic motion of the rotating bolt and the installation process within the hole. The bolt profile, speed of the bolt rotation, hole size and resin viscosity can be optimised to maximise resin mixing capabilities and viscosity requirements during bolt installation.

CONCLUSION

This study shows how the numerical modelling methods can be successfully used to optimise the load transfer between the bolt and the surrounding strata. The study indicates that the standard rock bolt reinforcing elements commonly used in the numerical simulation of the supported underground
excavations cannot be used to optimise the load transfer capabilities of the bolt. A detailed model of the bolt profile must be constructed, loaded to failure and compared with other profiles to find the optimum bolt profile with maximum load transfer capabilities between the bolt and host strata.

Numerical modelling is an extremely versatile, useful, cheap, fast and accurate tool to enable decision making regarding to the choice of the best reinforcement for a particular application. This method is becoming more accepted in the mining industry and further studies are desirable to provide the industry with cheap and reliable tools that can be used in future on the routine basis.

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CFD SIMULATION OF UNDERGROUND COAL DUST EXPLOSIONS AND ACTIVE EXPLOSION BARRIERS

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\textbf{ABSTRACT:} Computational fluid dynamics (CFD) is being applied to the study of coal dust explosions and their suppression in underground coal mines. As part of an ACARP funded project to develop a practical active explosion barrier, CFD is being used to simulate the explosion dynamics in simple mine roadways before examining the design requirements for an active explosion barrier. Results of these simulations will be used to develop the specifications for a prototype active explosion barrier with a reduced requirement for large scale testing.

Results to-date are very encouraging with validation of the model behaviour against a range of explosion conditions in the Simtars Siwek 20 L chamber and the CSIR's 200 m explosion tunnel in South Africa. This paper presents the results of a number of simulations with comparison against data obtained from the 200 m tunnel and preliminary modeling of an active barrier. This modeling provides an opportunity to examine explosion dynamics at a level not seen before.

\textbf{INTRODUCTION}

Coal dust explosions have always and will continue to represent the most significant hazard in an underground mine. Much effort has been expended in developing methods of prevention and suppression and these generally centre on the use of passive processes, such as adding stone dust to accumulations of coal dust to prevent its ignition. Traditional methods of investigating underground explosions have generally been limited to observations of staged explosions in facilities such as Bruceton (USA), Buxton (Britain), Barbara (Poland), Tremonia (Germany) and Lake Lynne (USA) experimental mines or the Kloppersbos (South Africa) explosion tunnel.

Many of these facilities are now closed and, despite the undoubted value of the knowledge gained from their operation, there is still much to learn regarding the nature of coal dust explosions and their suppression.

One aspect of research pursued by SkillPro at the Kloppersbos facility in South Africa was that of the demonstration and development of an active explosion barrier. With support from ACARP, Projects C8010 and 9008 did produce a successful result in showing the operation of a system to extinguish a coal dust explosion with an electronically initiated system of suppression dispersal ahead of the explosion flame. For various reasons it was not possible not to progress the demonstration. There was however a significant desire in Australia to continue the research effort in this area. It was therefore proposed to develop Computational Fluid Dynamics methods for modeling of coal and methane explosions in underground coal mines and ultimately the performance of active explosion barriers in an effort to minimize the large scale testing required for these systems. ACARP has again supported the work described here and BMT WBM has collaborated with SkillPro in the development and analysis of the modeling and its outcomes.

\textbf{METHODOLOGY}

Any numerical modeling effort is only as good as the accuracy of the predictions it is able to make. For the purpose of this project, a substantial selection of test results for explosions carried out in the 20 L Siwek spherical chamber at Simtars and CSIR Kloppersbos explosion tunnel in South Africa was available for validation purposes. In earlier ACARP funded projects (C8011 and C9011), SkillPro had investigated the minimum inerting requirements of a range of Australian coals using the small scale 20 L chamber and the 200 m long explosion tunnel. It was decided to make use of this data to validate the CFD model developed by firstly modeling the Siwek chamber dispersal and explosion and then to

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repeat the process with the data from the Kloppersbos tunnel. It was considered essential to obtain reasonable agreement with the modeled and actual explosion characteristics with these methods of testing before proceeding to modeling of active explosion suppression systems.

The Kloppersbos Explosion Tunnel

As the modeling of the Siwek chamber was an intermediate step in the model development, the 20 L chamber will not be described in this paper, but it is desirable for the reader to understand the nature of the Kloppersbos facility. Consisting of a steel pipe 200 m long and 2.5 m diameter, the explosion tunnel is mounted on concrete blocks on the surface (see Figure 1). Originally developed by Cook, the tunnel has been used to examine the minimum inerting requirements of coals from South Africa and Australia, and the suppression of coal dust explosions by the CSIR bagged barriers and active explosion systems. The tunnel is equipped with a series of pressure and flame transducers at regular intervals along its length to allow analysis of the explosion characteristics.

Explosions are staged by igniting a small zone of methane/air mixture at the closed end of the tunnel (see Figure 2). This lifts up and provides the ignition source for various combinations of coal and coal/stone dust mixtures distributed inside the tunnel. The configuration commonly used to test inerting requirements is the “double strong” explosion in which 35 kg of pulverised coal is loaded on six shelves (three on each side of the tunnel) between 20 m and 50 m from the closed end. This is repeated for a second set of shelves running from 64 m to 94 m from the closed end. To examine the minimum inerting requirement of a coal, a mixture of progressively higher incombustible content is loaded onto the second set of shelves until there is no flame propagation through this zone. Another type of explosion is the “Seminar” explosion in which the same quantity of coal is placed on the floor of the tunnel. This explosion is used on industry training days and always produces a spectacular result (see Figure 3).

The most recent efforts in this project have been aimed at validating the modeling outcomes against the results of a wide range of “double strong” explosion results gathered during ACARP Project C9011 and then proceeding to examining the performance of a water based active barrier in a “double strong” explosion.
Figure 2 - Kloppersbos tunnel layout – double strong coal dust explosion (not to scale)

Figure 3 - Results of a ‘Seminar’ coal dust explosion at Kloppersbos

**CFD MODELING**

The CFD model is detailed and complex. The full description of the theory and mathematics is beyond the scope of this paper. In this section we present a brief summary of the content of the model is presented without going into mathematical details.

**Compressible flow solver**

At the heart of the CFD model is a transient compressible flow solver evolving total gas density, temperature, pressure and velocity all as a function of time. The k-epsilon Reynolds Averaged Stress turbulence model is employed.

**Gas chemistry**

Kinetic gas phase chemistry and coal char surface reactions with oxygen deliver the energy required for the ignition and propagation of a coal dust explosion. The majority of the gas phase chemistry is related to the combustion of CH₄ and H₂ with oxygen to produce CO₂ and H₂O. This process is modeled by tracking mass fractions of N₂, O₂, CH₄, H₂, CO₂, and H₂O of the total gas density on a cell-by-cell basis, and employing the simplified single step irreversible reactions:

\[
\begin{align*}
\text{CH}_4 + 2\text{O}_2 & \rightarrow \text{CO}_2 + 2\text{H}_2\text{O} \\
2\text{H}_2 + \text{O}_2 & \rightarrow 2\text{H}_2\text{O}
\end{align*}
\]

Single rate Arrhenius equations are used to describe the molar conversion rates for these two reactions, with the equation coefficients tuned to yield the correct laminar flame speeds for these reactions at stoichiometric fuel-air ratios and standard temperature and pressure.
More advanced chemistry models are possible but are not realizable given computational constraints. The above simplified models yielded good pressure-time comparisons against test data for the combustion of CH\textsubscript{4} and H\textsubscript{2} within a 20 L "Siwek" spherical test chamber.

**Coal dust**

The evolution of position and velocity of particles of coal dust is calculated by integrating the velocity and acceleration of the particles. The acceleration of the particles is computed using the relative velocity between the particles and the gas, and a drag model which transitions from Stokes law at low Reynolds numbers to constant drag coefficient at high Reynolds numbers.

The evolution of the temperature of the particles is calculated using the Ranz-Marshall heat transfer model and the difference in temperature between the particles and the local gas around them. Heat input/loss from the particles due to radiation effects (section 3.4) is also accounted for.

Both the momentum and thermal coupling between the particles and the gas is bi-directional. Total momentum and energy of both particles and gas is conserved.

The devolatilisation of the coal particles is modeled with a single kinetic rate equation of Arrhenius form. All volatile species are assumed to evolve at the same rate. This is well known not to be the case in reality, but this reduction in complexity is not considered to be detrimental to the model. The surface oxidation of the char particles is modeled with a single kinetic rate equation:

\[
C + O\textsubscript{2} \rightarrow CO\textsubscript{2}
\]

The rate of reaction is proportional to the square root of partial pressure of O\textsubscript{2} at the particle surface (i.e. the reaction is of order 0.5), which is factored down from that in the far field according to the diffusion law and also the emission of volatiles from the particle. Importantly, the energy yield of the reaction in the first instance heats the particle and not the far field gas. The temperature of the particle is then controlled by the conduction and radiation heat transfer processes.

As the coal particles are only microns in size, it is not possible to track the evolution of every individual particle of coal, as there are billions of particles. Instead the evolution of a computationally more tractable number 'parcels' is modeled, where each parcel represents a collection of individual particles. The exchange of momentum, heat, and gas species with the gas phase is scaled according to the number of particles within a 'parcel'.

To model the physical process of entrainment of the coal dust from the floor of a tunnel or roadway into the gas flow within the CFD simulation requires a very fine mesh near the coal laden surface in order to resolve the flow boundary layer. This represents a significant computational expense that may be avoided through the use of an entrainment model. Such a model was developed for these simulations. It appears to yield an intuitive result in the animations, and has not presented a stumbling block with regard to model calibration against available test data.

**Radiation field**

The transfer of energy ahead of the flame front by means of radiation plays an important role in a dust explosion. Radiative heat transfer is accounted for using the "P1" model in which the radiation intensity is assumed to be isotropic, and its distribution is diagnostically solved for at each time step according to the volumetric absorptivity and emissivity of the dust cloud and gas phase combined.

**Water spray**

The evolution of droplets of water injected in the vicinity of the flame front is solved for in much the same way as for the coal particles. Again the ‘parcel’ approach to modeling particles is employed, but the devolatilisation and surface reaction models are replaced by an evaporation phase change model. Again total mass, momentum, and energy for both water droplets and gas is conserved.

The evaporation model used is based on that of Bird. Of particular importance is the effect known as ‘Stefan flow’ in which the evolution of gaseous vapour from the particle surface acts to shield the particle from the thermal conduction processes heating the particle. Hence the temperature of a droplet
of liquid injected into a hot gas flow asymptotes to the boiling point of the liquid as it progressively shields itself from the hot gas. Neglecting this effect can cause the cooling effect of the liquid spray to be overestimated by a factor of 4 or more.

**CFD Mesh**

The CFD Model uses a three dimensional (3D) hexahedral mesh of the tunnel with a cylindrical expansion volume at the end of the tunnel to provide a realistic representation of the pseudo wave-transmissive pressure and velocity boundary condition that exists at the end of the tunnel. Figure 4 shows the cross-section of the main tunnel mesh, which is duplicated at 0.125 m intervals for the length of the tunnel, totaling over 300,000 cells including the expansion volume. A plane of symmetry on the centerline of the tunnel was utilized.

The cells highlighted in blue in Figure 4 are voided to create the shelves on which the coal dust and stone dust is placed for the “strong” explosions. The shelves in the test facility are constructed of wire mesh, hence they will provide some resistance to the flow in the longitudinal direction, but offer little resistance in the vertical direction. This was modeled in the CFD mesh by ‘perforating’ the shelves with a 50/50 duty cycle for cells voided / cells present.

**Computational considerations**

The CFD mesh is not large by way of CFD models, but the fine dust particles require a small time step to follow the fast time scales at which the heat transfer and combustion processes occur. The large number of time steps combined with a moderate size CFD mesh, chemistry calculations, and tracking hundreds of thousands of dust parcels, has necessitated the need to run the model on a large multiprocessor computer.

**RESULTS**

**Calibration**

Prior to modeling the Kloppersbos tunnel, a significant amount of effort was directed to modeling coal dust combustion within a 20 L ‘Siwek’ spherical chamber. Having obtained reasonable calibration with test data the work progressed to modeling the Kloppersbos tunnel.

Figure 5 shows flame sensor and pressure transducer data during a typical ‘Double Strong’ test in which coal dust is loaded onto the shelves in both fuel zones. The salient features to note are the accelerating flame front and the high pressures outbye of the second fuel zone. Also note that the flame sensor data is presented in volts as sampled by the instrumentation system as there is no known calibration to either gas temperature or radiation intensity. From the photocell datasheet and the circuit geometry we estimate that 5V output corresponds to the range of about 1500-1600K black body temperature within the tunnel, but this has not been confirmed.

Figure 6 shows the results of the CFD model in the same format, except now the temperature is in Kelvin direct from the model. The salient features of accelerating flame front and outbye pressure pulse are present. Further improvement in calibration may perhaps be obtained with improved dust entrainment models, better coal combustion models, more detailed chemistry, and so on. However, the authors are of the opinion that the CFD model is of sufficient accuracy to allow investigations into active barrier concepts, bearing in mind that any promising concept will be significantly tested against real explosions during its development.
Active Barriers

The first active barrier concept modeled was that of a ring of water injectors located at 60m down the tunnel, just inbye of the second set of shelves. Figure 7 illustrates the results of the CFD simulation from \( t = 0.56 - 0.59 \) s, just as the flame front passes through the ring. The coal dust is coloured with the black body radiation spectrum in Kelvin (lower left scale), and the water droplets are blue. The iso-surface is a temperature contour marking the approximate position of the flame front, coloured according to gas velocity in m/s (upper right scale). The dynamics of the flow are striking, particularly the motion of the water sprays as they cool the passing flame front.

Figure 8 shows temperature contours as a function of distance and time for this single ring active barrier geometry, with mono-disperse droplets at 20 \( \mu \)m and 200 L/s volume flow rate into the tunnel. As can be seen the ring suppresses combustion in its local vicinity, but a bubble of hot products passes through allowing the flame to extend into the second fuel zone from which the explosion is re-established.
Figure 7 - Initial active barrier concept

Figure 8 - Single ring, 20 μm droplets, 200 l/s flowrate
The obvious route to improving the performance of the barrier is to increase the volume flowrate. But before resorting to brute force, we wished to investigate the effect of spray geometry on the performance of the barrier. Figure 9 shows the results for a triple ring active barrier geometry, with mono-disperse droplets at 100 \( \mu \text{m} \) and 200 L/s total volume flow rate into the tunnel. In this model the rings were spaced 10 m apart and only occupied the upper two thirds of the tunnel to better represent what may be more practical for use in a real roadway. The explosion is successfully prevented from propagating beyond the second fuel zone, and this for the same total flowrate and larger droplet size than the single ring design.

The effectiveness of the barrier is strongly dependent on the droplet size in the spray. Theory predicts that for a given volume flowrate, the total evaporation rate is inversely proportional to droplet diameter squared. Hence if the droplet size can be halved the barrier need only inject water at a quarter of the rate to be equally effective.

![Figure 9 - Triple ring, 100 \( \mu \text{m} \) droplets, 200 l/s flowrate](image)

**FUTURE DIRECTIONS**

The results presented here are somewhat preliminary, but offer a promising picture as to what might be achievable in terms of a re-locatable explosion barrier that will allow normal mine traffic to pass unhindered, yet prevent both incipient and mature dust explosions from propagating past the barrier.

Future work will involve the design and construction of a prototype system for evaluation in the Kloppersbos test facility. The test data gathered will enable further calibration of the CFD model, which then in turn may be used to progress the designs for systems suitable for real roadways.

**CONCLUSION**

In summary SkillPro and BMT WBM have jointly developed a highly advanced capability in simulating the dynamics of coal dust explosions. Further, the software is able to predict the impact of injected explosion inhibitors on the propagation of the explosion, and therefore assess the effectiveness of active explosion barriers.
The software has been validated to the extent possible with test data from a dedicated coal dust explosion test facility, and has been used to investigate possible prototype barrier designs for use in this facility.

It appears that a violent coal dust explosion may be prevented from propagation with the injection of a fine water spray in quantities of less than 20 L per square meter of tunnel area, provided reasonable requirements for droplet size, nozzle velocity, and water flow rate are met.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the financial support of the Australian Coal Association Research Programme and the encouragement and support of the ACARP project monitors Mr. Guy Mitchell and Dr Bevan Kathage.

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THE USE OF CFD MODELLING AS A TOOL FOR SOLVING MINING HEALTH AND SAFETY PROBLEMS

Ting Ren¹ and Rao Balusu²

ABSTRACT: Many safety and health problems in the coal mining industry involve the understanding and analysis of the mechanism of fluid or gas flow, including goaf gas migration, face ventilation and dust dispersion. Computational Fluid Dynamics or CFD has become a powerful tool to assist mining engineers and find solutions to these problems. The use of CFD modelling in goaf gas management and drainage, goaf inertisation for heating control, and longwall dust control strategies is summarised.

CFD MODELLING

CFD is commonly accepted as the broad topic embracing mathematics and numerical solution, by computational methods, of the governing equations which describe the motion of fluid flow, the set of the Navier-Stokes equations, continuity and any additional conservation equations, such as energy or species concentrations. Today CFD become a powerful tool in almost every branch of fluid dynamics and engineering.

CFD modelling has been used in the mining industry in a number of areas, including control of methane and spontaneous heating (Creedy and Clarke, 1992; Tauziede et al, 1993; Ren and Edwards, 2000), dust control (Afiz et al, 1993; Sullivan et al, 1993), mine fires and explosions (Lee, 1994), auxiliary ventilation layouts in rapid heading development (Moloney et al, 1998). More recently, CFD codes are being used in Australia for development of goaf gas control (Balusu et al, 2002, 2004) and goaf inertisation strategies (Balusu et al, 2002, 2005), dust control on longwalls (Ren and Balusu, 2005; 2007; 2009), heating gases dispersion in the goaf (Ren and Balusu 2008) and goaf heating simulations (Yuan and Smith, 2008).

The development of CFD models involves a number of complex steps, depending upon the specific problems to be addressed. The most important step is to understand the issues to be investigated, the engineering concepts and the results to be expected from the modelling studies. For mining related health and safety problems such as gas management, this would typically require visits to the mine site to discuss the problem with the ventilation engineers and examine relevant data to clarify any technical issues before developing the CFD models. In summary, the following steps would be involved in the process of any CFD models:

- Field studies to obtain fundamental data and information – this would typically involve the collation of mine design plans, ventilation layout and gas monitoring data, gas drainage systems, geological and geotechnical reports, depending on the problems to be studied;
- Construction of CFD model geometry and computational mesh/grid – The raw data collected from the field studies has to be simplified to produce the model geometry and grid (2D or 3D) which has to be conceptually sophisticated enough to represent the real case to be investigated. This is typically done by the use of a CAD style mesh generator or pre-processor of a CFD package;
- Setup of CFD model – The above computational mesh will be brought into the CFD processor or the solver for the definition of computational models, boundary conditions, flow properties and other functions that need to be performed by user defined programs or functions (UDF); This step may also involve the refinement of meshing, mesh quality and boundary checking, and in some case, the complete re-meshing of the initial model;
- Initial model simulations – Once the CFD model setup has been completed, an initial run of the CFD model with a few iterations is often performed to check the stability and convergence of the model. 'Engineering judgement' is needed at this stage to examine if the model is producing meaningful results. In many cases, modifications are needed to modify boundary

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conditions, computational models, UDF functions and mesh improvements. A good base-
model should be characteristic of good convergence and meaningful results independent of
computational mesh;

- Base-model simulation and validation using field or experimental data – The above base-
  model needs to be calibrated and validated against real data, typically ventilation survey data
  or gas monitoring data. The base model will be fine-tuned to produce results that show
  reasonable good agreement with the real case;

- Parametric studies to investigate various ‘what if’ scenarios of the problems and solutions –
  The fine-tuned base model can now be used to carry out a wide range of parametric or
  sensitivity studies to investigate the problems when changing one or two design parameters
  and develop solutions or optimum strategies.

- Figure 1 shows the CFD model geometry and computational grid of a longwall panel using
  perimeter ventilation system.

Figure 1 - CFD model geometry and computational grid of a longwall panel

GOAF GAS MANAGEMENT AND DRAINAGE

The fundamental understanding of goaf gas migration characteristics is important for modern longwalls
with high gas emission, and in seams liable to spontaneous heating. Knowledge of the impacts of
various operational and geological factors on the accumulation of goaf gas in the goaf is essential for
the design of effective goaf gas management and gas drainage system to reduce fugitive gas (mainly
methane) reporting to longwall and return ventilation. Figure 2 shows the goaf gas distribution patterns
for an Australian longwall panel using perimeter ventilation. The face is ventilated with around 80 m$^3$/s
of airflow, with goaf gas emissions between 1 500 L/s and 2 000 L/s consisting of 65% carbon dioxide
(CO2) and 35% methane (CH4). The model results indicate that a gas-rich zone of high concentration
goaf gas (CO2 and CH4) starts to accumulate close to the TG side of the goaf at about 100 m behind
the face and at lower level of the goaf due to the heavy goaf gas (mainly CO2). The modelling results
also indicate that oxygen ingress into the goaf can be up to 400 m behind the face on intake side of the
goaf. Oxygen levels in the vicinity of the goaf perimeter road are in the range of 5 to 10%, and oxygen
tends to layer towards the top of the goaf as a result of the heavier goaf gas.
In addition to the use of perimeter ventilation system, goaf gas drainage is needed to reduce goaf gas reporting to the longwall and return ventilation resulting from high goaf emissions. Figure 3 shows the use of surface goaf wells for goaf gas drainage. Modelling results show that the strategy of goaf gas drainage from goaf holes near the face in combination with continuous gas drainage from deep holes would substantially improve the overall performance of the gas drainage system and reduces tailgate gas concentrations. Depending upon the geological conditions in the panel, these holes should be located at about 50 m from the rib and close enough to the seam level in elevation to avoid air ingress and maximise the capture of high concentration CO$_2$ gas.

Modelling study and field application experiences demonstrated that the optimum goaf gas drainage strategy should include the following:

- Goaf holes should be drilled ahead of mining on return side of the goaf, preferably at 30 to 70 m from gateroad depending on the longwall caving conditions, and at 100 to 300 m spacing depending on the goaf gas emissions and other conditions,
- Goal holes diameter should be in the range of 250 mm to 400 mm and cased using both competent and slotted steel pipe according to strata and predicted caving conditions to improve borehole stability and maximum gas capture,
- Borehole bottom elevation should be adjusted according to the goaf gas compositions. Goaf hole should be drilled close the seam if the goaf holes aimed to drain heavier goaf gas such as CO$_2$ in the goaf,
Total capacity of the goaf gas drainage plants should be around 2 to 3 times the expected goaf gas emissions in the panel to achieve optimum performance of the gas drainage system and provide better gas control on the face,

Goaf gas drainage should include a combination of goaf holes near the face and deep goaf holes in the panel in order to improve the overall gas drainage efficiency and to reduce the effects of barometric pressure changes on tailgate gas levels,

The strategy of continuous operation of deep goaf holes at moderate capacity should be implemented. i.e., intermittent operation of deep goaf holes at high capacity may not improve the overall efficiency and may lead to problems,

Ventilation system in the panel should be designed to minimise oxygen ingress into the goaf, including immediate sealing-off all the cut-throughs behind the face, in order to improve overall gas drainage efficiency,

Oxygen concentration at all goaf holes should be continuously monitored and controlled at less than 5% to reduce the risk of spontaneous heating in the longwall goafs,

Goaf gas drainage should be carried out from more goaf holes at optimum capacity. It is preferable that gas drainage is carried out by 3 to 4 holes, rather than 1 to 2 holes, this would reduce oxygen ingress into the goaf.

In combination with field trials, the CFD modelling results have been used to design the goaf hole parameters and the development of optimum goaf gas management and drainage strategies. Significant improvements in goaf gas capture have been achieved in several coal mines in Australia (Balusu et al, 2002; 2004).
CFD modelling has been used to optimize the design of roof boreholes and overlying gas drainage road in Chinese coal mines using ‘Y’ ventilation system (a bleeder road between a retaining wall in the goaf and solid coal rib). Figure 4 shows the CFD modelling results of roof boreholes drilled at different angles to intercept seam gas from overlying seams. Results show that boreholes drilled at 65° ~ 70° are more effective to intercept high concentration goaf gas than boreholes at 45° ~ 50°, and it is equally important to case the boreholes up to 30 m and maintain borehole operation beyond 200 m behind the face.

(a) Roof boreholes intercepting high methane gas at 30m above roof

(b) Roof boreholes intercepting high methane gas at 40m above roof

Figure 4 - CFD modelling of roof boreholes intercepting seam gas from overlying seams
SPONTANEOUS HEATING AND GOAF INERTISATION

A number of heating incidents have occurred in recent years leading to major production losses and safety risk for a number of mines in Australia. The coal seams in these mines areas are generally thick, and the risk of goaf heatings increase significantly due to the large quantities of broken coal left behind the chocks and its exposure to deep oxygen penetration in the goaf due to increased ventilation volumes. A snapshot of the typical goaf gas distribution behind the longwall face using perimeter road is presented in Figure 5. Gas monitoring results show that intake air ingress on the MG side of the panel was high, with the oxygen level at more than 17% even at 400 m behind the face. Even on the TG side, oxygen ingress appears to extend up to 100 m behind the face. High oxygen penetration around the start-up area can also be observed. This high oxygen ingress could lead to heating development in the goaf, particularly during face stoppage or slow face retreat in the panel.

Figure 5 - Typical gas distribution in a longwall goaf

CFD modelling was used to investigate oxygen ingress into the goaf and identify ‘oxidation zones’ where spontaneous heating of coal is most likely to take place. Figure 6 shows the CFD modelling results of oxygen penetration pattern into the goaf and the mapping of ‘oxidation zones’ (with oxygen concentration between 5~18%) in the goaf using iso-surface of oxygen concentrations. Results show that goaf heating is likely to occur in goaf areas at about 50~200 m behind the face, along the goaf edge of MG side, and in the vicinity of face start-up line.

To minimize the risk of goaf heating, a common practice in Australian longwall mines is to inject inert gas such as nitrogen behind the face to reduce the spatial areas of ‘oxidation zones’ thus suppress the onset of potential heatings. CFD modelling was used to identify inert gas injection strategies behind the face to achieve the most effective goaf inertisation. Figure 7 shows the effect of inert gas injection at different locations behind the face. Results show that inert gas injection at a location immediately behind the face line will only have negligible impact on goaf inertisation, as most of the inert gas injected will simply disperse into the main ventilation stream and disappear into the return airflow. Even at some 60m behind the face line, the injection of inert gas at a rate of 0.5 m$^3$/s would only have marginal effect on goaf inertisation. CFD results indicate that the most optimum injection locations for the modelled cases should be within the range of 150-400 m behind the face line on the maingate side. The results indicated that inert gas flow rate of 0.5 m$^3$/s would be required for most cases, although in some cases lower flow rates might be sufficient or in some other cases higher flow rates might be required to achieve the desired effect.

In case of access problems in underground workings, such as after evacuation of personnel from underground workings, the only possible means of goaf inertisation may be via surface drilled boreholes or those existing surface goaf holes for goaf gas drainage. CFD simulations results indicate that an improved goaf inertisation effect could be achieved by injecting inert gas via surface boreholes. By injecting inert gas on both side of the goaf via surface goaf holes would produce a better inertisation result than simply injecting inert gas on one side of the goaf. Similarly it is also important to understand goaf gas distribution patterns so that the correct injection locations can be identified to achieve the maximum goaf inertisation effectiveness.

The main findings from CFD modelling have been implemented in field applications and excellent goaf
inertisation results have been achieved, including longwalls in Australia and Blasting Gallery (BG) panels in India.

(a) ‘Oxidation zones’ liable to spontaneous heating – plan view

(b) ‘Oxidation zones’ liable to spontaneous heating – 3D view

**Figure 6 - CFD modelling of oxygen ingress and ‘oxidation zones’ in the goaf**

(a) Base model – no inert gas injection

(b) Inert gas injection at 60m behind the face

(c) Inert gas injection at 220m behind the face

**Figure 7 - CFD modelling of goaf inertisation**
LONGWALL DUST CONTROL

Management of dust on longwall face is a challenging issue for mine operators, especially for the new generation of longwall faces. The airflow and dust dispersion patterns in a longwall are complex as many factors such as ventilation, cutting machine/chock movements as well as dust control devices (e.g. water sprays and curtains) are involved. Three Dimensional CFD models were built to represent longwall faces in thin, medium and thick seams. Figure 8 shows the layout of the CFD model. These models consist of a section of the full scale coal face and the maingate, and embody the major longwall components such as chocks, shearer, spill plate, BSL/crusher and conveyor. In addition, dust scrubbers, shearer clearer, venturi sprays and curtains were incorporated into the models to investigate the effect of various dust control options.

To establish the base airflow patterns on the longwall face, CFD simulations were carried out with a range of ventilation rates between 30 m$^3$/s to 80 m$^3$/s. Field information and data were used to establish the geometry and boundary conditions of the longwall faces representing 4.5 m thick seam, 3.0 m medium seam and 2.1 m thin seam. Results from the base-case CFD models were calibrated and validated against field data obtained from three Australian longwalls. The validated models were then used for extensive parametric studies involving changes in air flow rates, shearer Clearer/sprays, the position of scrubbers and curtains etc. These parametric studies can be used to investigate the effect of various controls on dust flow patterns and dust capture on longwall faces.

A particular application of CFD modelling has been the development of a new shearer scrubber system. CFD was used as a design tool to optimize parameters including the locations of both inlet and outlet (airflow discharge direction), and the capacity of the scrubber in relation to the face airflow rates. Figure 9 respectively shows the modelling results for the dust scrubber system. Modelling results indicate scrubber inlet located towards face ventilation offers improved advantage of capturing a large portion of the dust particles from roof/chock movements ahead of the shearer as well as some of the dust from the spalling area. This scrubber inlet location is also effective for confining the dust particles from these sources to the face. In practice, this option also has the advantage of maintaining the clearance of the scrubber inlet(s) from the direct falling and stacking of coal lumps from the cutting drum and spalling area. Modelling studies showed that it is important to correctly position the scrubber outlet location and discharge direction to improve the overall diversion of escaped dust particles away from the face walkway. As shown in Figure 9 (c), scrubber outlet discharge tilted slightly towards the face helps the confinement of dust particles to the face and the overall diversion of dust clouds away from the walkway.
In collaboration with EnviroCon, CFD modelling results have been used in the design optimisation process for the new shearer dust scrubber system, as shown in Figure 10 (a). The final design included the following key features:

- The scrubber was designed as a compact modular unit to fit the limited space between the ranging arm and the shearer body;
- Scrubber intake duct facing the ventilation direction;
- Fine water sprays embedded into the intake duct to increase scrubber collection area and additional function of 'air curtain' to suppress and streamline dust particles;
- Scrubber exhaust under the ranging arm and tilted 15° towards the face and 45° toward the tailgate.

Field trials of the scrubber system were carried out at BHP Billiton Mitsubishi Alliance’s mine at Broadmeadow in Queensland’s Bowen Basin. Dust monitoring results indicated that the dust reduction rate varied from 43 per cent (with average dust concentration falling from 1.35 mg/m³ to 0.77 mg/m³) to 56 per cent (with average dust concentration falling from 1.59 mg/m³ to 0.70 mg/m³). Figure 10 (b)
shows the impact of shear scrubber system at 4th Chocks Outbye of Shearer Operator’s positions. The results were better than expected and indicated a positive advance in mine safety.

(a) Field trials of shearer dust scrubber system

(b) Impact of shearer dust scrubber system on longwall dust reduction

Figure 10- Field trials of longwall shearer scrubber system and its impact on dust reduction

CONCLUSION

This paper has provided a summary of the application of CFD modelling techniques as a tool for solving mining health and safety problems, including goaf gas management and drainage, goaf heating and inertisation and longwall dust controls. The usefulness of CFD has been demonstrated in the
development of optimum strategies for handling these issues and significant benefits to mine operators both in Australia and overseas such as China and India. CFD modelling results must be validated against field data and engineering judgments, and used as a tool of integrated system combining other computing and experimental methods.

ACKNOWLEDGEMENTS

The author is highly grateful to ACARP for funding most of the research work presented in this paper. The strong support from many mines staff and management team for the field trials of EnviroCon and their active involvement in the development of the shearer scrubber system is acknowledged.

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APPLICATIONS OF RFID AND MOBILE TECHNOLOGY IN TRACKING OF EQUIPMENT FOR MAINTENANCE IN THE MINING INDUSTRY

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ABSTRACT: Supply Chain Management (SCM) is a crucial factor in reducing the down time in equipment maintenance in the mining industry. The paper describes a tracking and verification system using Radio Frequency Identification (RFID) and mobile RFID technology developed for an SME in the UK to monitor the assembly of parts for a manufacturing company. The system uses low cost passive tags (costing few cents) to provide information in real time using TCP/IP protocol which is internet compatible and can be viewed anywhere in the organisation worldwide to provide more effective management control. The RFID technology and mobile RFID equipment is able to operate in a manufacturing and fabrication ‘metal environment’ with read/write distances of up to 6m. The information can be linked to a CAD system and/or Witness Quick 3D to provide visualisation and simulation of the shop floor in terms of equipment and personnel movement. This information can also be linked to digital imagery and used to provide evidence and visualisation for agile management systems in mining machinery workshops and stores.

INTRODUCTION

Radio Frequency Identification (RFID) is an automatic identification technology which has received attention recently because of its potential in asset tracking and logistics support. It is widely thought that RFID technology is causing a revolution in Supply Chain Management (SCM) by providing a substitute for barcodes. The rapid development of RFID technology actually provides business operations with a chance to gain competitive advantage, particularly for SME’s (Small or Medium Enterprise) involved in SCM operations, along with its potential for tracking equipment in maintenance operations.

RFID technology offers an efficient means of logistic management especially for agile and lean management. The concept of lean production, originally derived originally from Toyota Production Systems, means eliminating any waste of resources or effort in the system, for example using less of everything, including human effort, manufacturing space, investment in tools and engineering time to develop new products (Womack et al., 1990). Lean management is linked to the concept of “zero inventory” and the Just In Time (JIT) approach (Fan et al., 2007). The concept was promoted by the Laccoca Institute of Lehigh University in 1991 and is based on the idea of “flexible manufacturing systems” (Fan et al., 2007). Agility means using market knowledge and a virtual corporation to exploit profitable opportunities in a volatile marketplace (Naylor et al., 1999). In fact, lean and agile management share a common objective: to meet customer demands at the least total cost (Goldsby et al., 2006).

This paper describes the application of RFID technology in SCM and the concept of agile management in a manufacturing SME in the UK to monitor the assembly of components. The design of an RFID system is discussed and the performance of two main types of tags used in RFID technological systems is examined and tested. The results presented demonstrate the feasibility of using RFID technology in supporting agile management systems. This technology can be used to monitor and track equipment in engineering workshops used in the mining industry.

RFID IN LOGISTIC MANAGEMENT

There are many ways to use RFID technology for logistic management, but the basic idea is to track and mark the container (usually this is a pallet or container) rather than tracking each product component which may cause extra cost and other issues, for example metal components may block the signal resulting in failure to track the components’ movement.

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The application of RFID technology is restricted by many factors, such as the physical environment, production workflow and the financial investment available to the company. The case study was conducted at a company through collaboration arrangements. The company is involved in the fabrication and manufacture of metal assemblies for numerous customers. Its environment and complex workflow represent a typical manufacturing SME which would benefit from the use of emerging RFID and mobile technology.

Challenging issues in the SME case study company

The case study company is a fabrication/manufacturing firm that provides metal manufacturing and assembly services, including metal cutting, machining, drilling, fabrication, welding and also uses subcontractors if any ‘powered’ or ‘paint coating’ is required for the finished product before dispatch from their premises to the customer. The large fabrication service offered by the company results in diverse workflows and each individual order is usually specific to that customer, resulting in complex work schedules.

The company’s final product usually contains several components that typically have diverse workflow systems. The company has a relatively large factory with several workshops and its current Manufacturing Resource Planning (MRP) system is not effective in providing timely information on the work process, job progress and particularly in locating components. This sometimes results in components going missing when they are needed and the staff must spend unproductive time either locating or re-producing the component. The company has realised that its employees spend too much time locating components, which has a negative impact on efficiency and performance, and has resulted in a financial impact on the company.

Consequently there is an urgent need for a practical system to solve these issues, and RFID technology can provide a suitable solution. However, all the products produced by the company are metal components, which affect the propagation of radio waves used by this technology. This paper discusses and describes the experiment for evaluating the feasibility of using RFID technology in this manufacturing environment.

Design of the RFID Technology for Tracking and Monitoring

The manufacturing company uses wooden pallets as temporary containers to hold the components and semi-finished products as they are used and/or fabricated and moved around their workshops. The basic idea of the application is to use passive RFID tags attached to the individual pallets to track the movement of the pallets and indicate the logistics of products movement and storage within the company via ‘choke points’ also referred to as ‘gates’ or ‘doors’.

The RFID passive tags could be affected by the metal environment of the manufacturing facilities, and must be carefully attached to the pallets. Figure 1 illustrates the attachment of the tag to the corner of the pallet in order to try and avoid interference from the metal products which ideally need to be placed in the centre of the pallet.

![Figure 1 - RFID tag location and arrangement in relation to the pallet](image)

In addition, each pallet could have more than one tag either to reduce erroneous readings and/or be used to represent different product components on the same pallet as illustrated in Figure 2. For example, the tags shown in the illustration on the same pallet can have different ID’s but be linked to
the same product in the database so that once a tag on the pallet is read electronically, the information on its location and contents can identify where the pallet is located in the workshop and/or warehouse facilities.

![Multiple RFID tags attached to a pallet](image1)

**Figure 2 - Multiple RFID tags attached to a pallet**

In order to receive the tag’s signal, and therefore the location of each pallet, each gate or door in the plant has to be installed with an antenna and corresponding reader as shown in Figure 3. The tags will then be captured as they pass a gate and the pallet location can then be determined in relation to the layout of the workshop. Figure 3 gives an example: - the pallet moves through a gate entry (in this case the punch machine room), and as only this gate has captured the tags (reading distance on a passive can be attenuated between 0.2 to 6m), the location of the pallet must be the punch machine room. In addition, the tags can also be scanned by handheld devices such as specially adapted mobile phones (which are ubiquitously carried by individuals) and this provides more flexibility in monitoring and controlling the tags, for example when checking information or updating/ modifying records.

![Read/Antenna gate installed in the workshop](image2)

**Figure 3 - Read/Antenna gate installed in the workshop**

If a longer read-range of the tags is required, then an active RFID tag can be used. Active tags are usually more expensive costing approximately A$ 8 each (depending on scale of purchase) compared to the passive tags which only cost a few cents. However, if the pallet is dispatched to the customer or supplier together with the product the tags can be removed from the pallet and reused. This would only need to be considered when using active tags because of their higher cost. The tags removed from the pallet can be assigned a new ID for a new workflow. The performance of passive and active tags will be discussed later in the paper.

The application of RFID technology can be based on TCP/IP network protocol; a popular network protocol, used with WIFI, Local Network, and the Internet. This means it can provide long-distance monitoring and control from any place in the world providing there is Internet access; another benefit is that the system can be easily linked with the WIFI network and supports wireless control (Zhang et al., 2009a; Zhang et al., 2009b).

The information can be linked to a CAD system and/or Witness Quick 3D (Lanner, 2010) to provide visualisation and simulation of the shop floor in terms of equipment and personnel movement to provide agile management concepts. The data from the RFID equipment can be adapted and used in simulation software such as Witness to identify and determine on-line ‘bottle necks’ in the workflow, as illustrated in Figure 4 showing that the simulation can be represented in an initiative 3D format for visualisation purposes (Zheng et al., 2007b).
RFID EXPERIMENTS IN THE MANUFACTURING COMPANY

In order to determine the ‘field’ performance of RFID technology in a complicated environment a number of experiments were undertaken in the case study engineering company using RFID technology. The case study company premises comprise a turn of the century ‘type’ building with heavy gauge structural support stanchions and reinforced flooring containing a large metal fabrication environment with numerous workshops: the worst case scenario for interference when using RFID technology. The first aim of the experiment was to determine the communication range in the factory, which is full of metal sub assembles and heavy manufacturing equipment. The second aim was to set up a RFID embedded door/gate to test the feasibility of recording the movements of pallets in the factory.

Equipment

In this experiment, the performance of both the passive RFID and active RFID was tested. The passive equipment was an ALR-8800 Reader with two circuit antennas from Alien Technology, and the Active equipment was a 217002 Reader with built-in antenna from GAO RFID Inc.

ALR-8800 is a UHF reader and operates in the 865.7-867.5 MHz range, following the EPC G2/C1 RFID standards. The reader supports two communication interfaces: RS-232 for serial connection and RJ-45 for TCP/IP communication. The system configuration needs at least two external antennas for reading: one is for sending the power to the tag, and the other for receiving the signal. In this experiment, one reader with four antennas was used to build the two gates. The RFID equipment with its antenna is shown in Figure 5. The tags used in this experiment were AL=964x, which is the best performing tag that could provide about an 8-9 meter communication range (based on previous laboratory testing).

The active equipment was from GAO RFID Inc: operates in the 2.45 GHz and applies an 802.11b wireless communication standard (WIFI). It also has two communication interfaces, either RJ-45 or a WIFI interface which can wirelessly communicate to the computer.
Testing procedure and results

The aim was to test the readability of the passive tag (worst case scenario) when attached to a metal skip, and the result shows that it supports up to a 6 meter read range in open areas as shown in Figure 6. The tag was attached to the top of the skip, which was filled with metal ‘off cuts’. The tag was kept 2 cm from the skip and the RFID antenna was moved as far as possible from the tag to determine the maximum read range.

To test a more complex environment the tags were attached to a wooden pallet which was full of metal sub assembles. In this situation, the communication range was dramatically affected, but there was still a practical read communication range of 1.5-2 meters. This result proves that passive RFID technology can track components in a factory environment, particularly metal sub assembles used in maintenance workshops etc. The passive tags used in the tests could provide sufficient communication range for the internal logistic management purpose of tracking and monitoring and could give longer distances in open work areas. The experimental details are shown in Figure 6.

The testing for active tags was a simpler procedure because of its longer communication range and signal penetrability. Even when the tags were fully covered by metal components, the equipment could still read the tag at a distance of more than 10 meters because of the stronger signal strength and wider frequency band. In this test, the reader was fixed and the tag was moved further away until the signal was lost: testing was then repeated 5 times. Figure 7 shows that the final result was about 35 meters a much better result than the passive tags.

The second test was to evaluate the tracking and monitoring function of RFID systems. For this purpose, a software application has been developed for monitoring and automatically generating the database records when movement of the tags is detected at the gates.

Figure 8 shows the four antennas working as two pairs to simulate two gates in the manufacturing company. A pallet with metal sub assembles is the tracking target and five tags are laid on the metal components. The target pallet is moved by a trolley and navigated though the two gates and then brought back through the gates.
During the movement of the pallet, the system picks up the tags four times, and generates three records which are shown in Figure 9. In the demonstration the antennas are numbered to represent the location, and because only two antennas receive signals (the other two are for sending the signal), the locations are respectively marked as “0” and “2”. For ease of understanding, the records of one tag (ID “BEEF0007”) are shown in Figure 9. The first records indicate that the tag is first captured by the antenna in location 0, therefore, the Last location column is marked “N/A”, and the second location shows the same tag picked up by the antenna in location 2, which means that the target pallet has moved to location 2. Then, the pallet is moved back and picked up by the same antenna, but no records are generated. The third record shows that the pallet has been moved back to the starting position which is location 0. This scenario demonstrates that RFID technology can work in the manufacturing company to provide management information for tracking and monitoring. More importantly it shows that it can work in one of the worst case environments (regarding RFID interference) as previously outlined, and therefore undoubtedly can work in any similar manufacturing and maintenance company.

The experimentation in the manufacturing company shows the feasibility of using RFID for tracking and monitoring via gates or choke points to provide logistical management information which could help improve efficiency and performance and solve some of the issues related to the case study. In addition, the reader and IT equipment in this experiment are linked by WIFI interface, i.e. are wireless providing more flexible control and deployment for users in the company.
DISCUSSION OF RFID TECHNOLOGY AND SOFTWARE DEVELOPMENT

RFID is an automatic identification technology that allows a small radio device attached to an item to carry an identity of that item (Glover et al., 2006). An RFID system has at least has three components: the reader, tags and middleware. Its working procedure can be briefly outlined as: the reader under the control of the middleware transmits adequate energy through the antennas to power up the tags and communicate with the tag to request and receive the identifier (Landt, 2005; Lehpamer, 2008). As an emerging technology receiving continual attention, many different types of RFID tags and associated technology have been developed. In general the tags are usually classified into passive RFID and active RFID (Landt, 2005), and in the experiments outlined in the paper, both of them are tested in an industrial environment.

The ALIEN equipment used in the experiments is a passive system which follows the EPC standards, from the EPCglobe (Electronic Product Code globe) organization which promotes the concept "Internet of Things", which means an open and shared infrastructures for auto-identifiable networked objects (Roussos, 2008). A passive tag devices has no power supply built into the tag and only works when it is inductively powered by the antenna, which allows it to transmit its information back to the reader. The power from the reader/antenna is usually limited and consequently the communication range of a passive system is shorter than the active system, and is dependent on the output power of the antenna. The communication range of passive RFID has increased dramatically in recent years, and can provide up to 5-10 meters, which is sufficient for practical logistic tracking, compared to a few years ago, when item level tracking was less than 1 meter.

The RFID equipment from GAO RFID is an active system, and the experiment outlined in the paper demonstrates that it provides about a 35 meter communication range in the metal environment of the case study company, previous tests have shown that in open areas, the equipment operates up to a range of 50 meters. An active system usually has an on-board power source, typically a battery and performs a specialized task (Behera et al., 2008). This structure has some advantages in terms of functionality and capability but drawbacks in terms of its cost and maintenance aspects. For example, the simplest tag from GAO RFID inc. costs about A$ 8, and only has a battery life of 3 years.

Figure 10 outlines two aspects firstly the conventional information in terms of tracking and monitoring of components within the manufacturing and workshops zone and secondly that this information can be used to provide knowledge management by using data mining techniques and simulation to provide potential competitive advantage. The latter is of particular importance to an SME in terms of reducing production costs linked with agile management concepts to improve performance.
The data transmitted from the RFID equipment will be stored in a database and the records of logistic movement will be extracted from the information and stored in a central database. The CAD system can be linked to the database to provide additional information either from real time monitoring and/or data analysis. The information can be directly used to monitoring the movements of assets or products and help to realize lean manufacture, such as JIT methods. Alternatively knowledge management systems can be developed and the data updated to a data warehouse; the information stored can then be used for data mining purposes to improve the company’s performance. For example, the information can be used to support knowledge management systems, including an intelligent decision-making system or other knowledge-based system to optimize the work process. This is extremely important particularly for an SME in terms of reducing production costs linked with simulation-based management concepts to improve performance (Yu et al., 2002; Zheng et al., 2007a).

CONCLUSIONS

This paper describes the emerging technology of RFID used in SCM in order to reduce waiting time and improve performance of the workflow. A tracking and monitoring system is described which was developed for an SME in the UK to monitor the assembly of components for a large excavating manufacturer. The system designed uses RFID technology to acquire information to track and monitor products which can be linked to a CAD system and/or Witness Quick 3D to provide visualisation and simulation of the shop floor in terms of equipment and personnel movement. Serial experiments were undertaken to evaluate the design of the system for both passive and active RFID systems. The tests show the feasibility of RFID technology and mobile RFID equipment, which be operated in a manufacturing and fabrication ‘metal environment’ with read/write distances of up to 6m. In addition, a brief discussion of RFID technology is introduced in order to compare the two types of RFID tags and associated equipment. The tags can also be scanned by handheld devices such as specially adapted mobile phones which these days are ubiquitously carried by individuals, and can provide more flexibility in monitoring and controlling the tags, such as in checking information or updating/ modifying records.

Application of RFID technology is more complex than widely imagined. It requires the correct specification and customisation to be set up and aligned to the business application in terms of the readers, antenna, middleware and software for individual applications. The requirements for technical advice at the feasibility and implementation stages are crucial to the success of the application. This technology would provide useful applications to mining machinery workshops and stores in terms of both tracking and monitoring and also in cost accountancy in relation to reliability of modular components and usage requirements.
Currently work is being developed in the application of RFID and mobile technology to support several business applications including asset and document tracking in a hospital complex, application in the hospitality industry, construction waste recycling and medical waste tracking and verification systems using customised hardware and purpose-developed software systems.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the support of EPSRC Collaborative Training Account (CTA) in respect to this project.

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Towards an Integrated Information Infrastructure in Coal Mining Asset Management Application

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Abstract: Maintenance is recognised as the largest controllable cost among direct mining costs (Lewis and Steinberg 2001). Ironically, the ideal architecture for information systems and technologies for optimising maintenance functions has not been extensively studied. While many maintenance management approaches recognise the importance of understanding equipment functionality and performance monitoring, based in an integral range of information, extensive study to attain proper organisation of the information have not been undertaken. Degradation detection and prediction applications found in recent literature are generally for specific applications rather than generic (Lee et al 2006)

The current study is intended to address Longwall mining equipment integrated data infrastructure. The expected outcome of this study is to come up with a centralised information architecture that will utilise all the relevant information in the organisation to optimise asset management functions in a results based framework.

Introduction

Even though in practice it is generally considered a non value generating activity, asset management (Bignell 1997), can have a larger impact in the organisation than other central activities. On average maintenance costs represent 40 percent of total operating costs for the business. Downtime for longwall mining machines is recognized as one of the highest in the industry, reaching around 35 percent (Bongers, 2004). Considering that average longwall production is valued at $100,000 per hour; required efforts to improve machine downtime seem evident. Moreover there is a group of costs associated with physical asset management that are difficult to be perceived, difficulties that have resulted in certain costs not being considered. Just to mention some, decrease in productivity or product quality due to asset malfunction are difficult to perceive, usually the equipment fault is made evident only when the equipment stops functioning. The study presented in this paper is one that aims for a measurable improvement in longwall production, by decreasing equipment downtime and proposing an information management framework designed to achieve a higher utilization of the installed capacity by measuring an operational performance.

Longwall asset management function is generally judged in terms of equipment availability, and operations uptime, but rarely is it assessed in terms of the throughput or productivity of the equipment the function is responsible for. This fact alone generates several misunderstandings and lost opportunities for improvement given the lack of awareness. It is recommended through the presented framework to consider the importance of throughput, rather than the availability itself, in longwall sites and other mining sites this recommendation will prove beneficial since the metrics based in this premise will be aligned towards the real goal of the organization, increase profitability.

Information and Architecture

Why look at information and architecture?

Perceiving and being aware of what is happening with mine throughput and mine assets performance is needed and is needed on time, in order to encourage a proactive frame of mind. Management of information is of vital importance for achieving this. Information may be the only source of evidence of an operations performance.

The right management and the required awareness to obtain it will be achieved only if the importance of information management is perceived; thereby the information infrastructure should be planned and managed. The right information should be in the right hands at the right time.

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Some organisations that do not manage information can’t even explain the cause of the asset faults or loss. Events are not related to outcomes, and faults are not related to information that could evidence possible causes for such faults. And the paradox is that generally the information is available, but is not organised and/or utilised. If is not organised then basically it doesn’t exist. As an example imagine having the phone number of a friend in an agenda written 10 years ago without alphabetical order; or even worse not knowing in which agenda or book the number is written. Getting to use this information would involve some kind of random coincidence. In fact this is how the IT landscape looks like as this paper is being written, even if it is not accepted by most IT experts; this happens in mining and in many other industries. There are seas of information and we have to mine (datamine) it as if it was as difficult to obtain as gold is. The main reason is the abundance of Silos that exist operationally and technically. The information landscape is conformed by several information systems and devices generating information.

The study being performed demonstrates that the data generated in operations process is underutilised. The real potential of data is not known on most of the mine sites. The excess of information available has been rather a disappointing factor when initiatives to analyse the information appeared. The time required to perform such analysis has been a major turn down. What will be the benefit if the results are obtained even days after the fault presented itself? Barely triggering corrective measures.

This paper is a claim to weight the benefits that managing integrated information accordingly would signify to longwall mining operations and highlights the importance of finding the right vision of how systems are desired to be designed. The systems should be architected.

**ACTUAL LANDSCAPE OF MINE ASSET MANAGEMENT**

According to Lewis and Steinberg (2001) currently the mining industry is slowly evolving from reactive to proactive maintenance philosophies. Undoubtedly implementing one of the current structured maintenance approaches will result in cost reductions and productivity improvements. Hartman (1992) also mentioned how maintenance is gaining a strong emphasis in the mining industry due to awareness that improved equipment availability is the key for obtaining a return on mine investment.

It is also important to highlight that Hartman (1992) described the evolution of the equipment with a trend to larger, more costly and complex equipment as time advances. Heavy mobile equipment has been considered costly for many years, but the increase in technologic advances in the last decades is making it even more complex and therefore more costly for materials and the qualified personnel to maintain it.

**RELEVANT FACTS OF INFORMATION IN ASSET MANAGEMENT**

- An overwhelming amount of data is produced by sensors and systems. According to Lee et al. (2006) “Many sophisticated sensors and computerized components are capable of delivering data about the machine’s status and performance. The problem is that little or no practical use is made of most of this data”. In longwall sites massive amounts of information should be recorded if all the sources of raw data were to be stored. That accounts to approximately 4Gb per week. (Bongers, 2004) That makes historical analysis costly since the mentioned growth of information will require a constant increase in data storage.

- Process and Technologic Silos. Although there is communication among different organisational areas, it is difficult to accept that objectives are shared between them and that the work is performed to improve the whole performance. As an example we can cite the relationship between operations, maintenance, quality and engineering. Issues are generally solved individually when the cause of problems come from another area it is difficult to perceive and solve. The same happens with technological efforts. Individual efforts are the most common when developing new systems or buying new equipment. Information is also recorded in stand alone sources for analysis, such as Excel sheets or individual Access databases.
• Information systems lack of flexibility. Evans (2007) found that organizational change usually disrupts the process and how the work is performed in this process, but it will not change the associated computer application and usually no information recording process is changed to mirror the process.

• Information systems are not planned to work interconnected. Devices and systems rarely communicate to each other. As a sole example redundancy of information was found in certain processes on the longwall, manual recording of stoppages and equipment data recorded evidencing stoppages. Bongers showed that both sources of data where rarely congruent (Bongers 2004, p7), which diminish value of having two sources of the same information. Data obtained redundantly from a process will only be useful if reconciliation of the information is performed. A lack of integration of data and process make this task almost impossible to perform continuously.

• Lack of reliability of information. Communication problems cause incompleteness of information. Underground condition cause problems in communication of data, resulting in a large amount of information loss (Bongers, 2004) Bongers in his thesis recommends a framework to deal with this problem as a part of his fault detection and isolation solution.

• The lack of process compliance also results in unreliable information. An interview made by Pancucci (2000) to the Industrial analyst Cambashi suggests that the main barrier for a meaningful computer maintenance management system (CMMS) implementation generally is the lack of adequate data that will allow implementing the practice. One of the main reasons, he explains, is a lack of discipline in process compliance.

• Big software and equipment providers try to push towards unique software provider solution not contributing with system integration.

• Outsourcing and third parties involved in daily process. One of outsourcing practices risks is the fact that diagnostic records may not be analysed in conjunction with plant site management records-critical historical information is lost when this happen; this fact is identified by Evans (2007).

INFORMATION INTEGRATION FOR ASSET MANAGEMENT STATE OF THE ART

The main studies regarding integration between the maintenance function and other areas are explained below.

Artificial intelligence is used to integrate maintenance management system taking into consideration not only equipment condition but also production quality, efficiency and costs, this is discussed by Zhang and Yeung(1997).


Kans (2009 confirms an important fact for this paper, "while other areas of the business were integrated into enterprise wide systems such as Enterprise Resource Planning (ERP) systems and Computer Integrated Manufacturing (CIM) systems, maintenance has not been well represented". Most of the ERP and CIM systems where used but most of these systems doesn’t include maintenance modules Nikolopoulos (2003). Pancucci (2000) also reported how the rush to keep production up and running is still impacting the actual integration of maintenance with planning.

According to Kans (2009) "having a holistic perspective on maintenance enables predictive-proactive maintenance".

PROPOSED ARCHITECTURE

The author suggests a continuous performance measurement strategy that will involve performance data from both operations and maintenance. This architecture is reliant in a continuous process and not a one time activity, holistic performance measurement and analysis should become an automated task.
The main feature of the presented architecture design recommendations is that it is centred in obtaining value for the business, rather than using the highest technological advances. This ensures using only that what provides beneficial outcomes.

**INTEGRATED INFORMATION ARCHITECTURE ROADMAP**

For making a broader use of the information it has to be holistic and reliable. The recommended roadmap to achieve this asset management practice is presented below. The roadmap of the architecture was kept independent of the technology or processes necessary to support it, since technology and processes change, they are not to be part of an information architecture guideline. The guideline is to be used as a vision to trace the architecture of information systems, considering a broad and complete spectrum.

Measure Performance: monitoring trends in performance and detecting degradation of performance in variables such as:

- Productivity
- Utilization
- Condition
- Maintenance function.

Performance should be continuously monitored and stored. For a practical framework on maintenance performance analysis refer to De Groote (1995), although individual asset performance measurements are not covered.

**Key questions:** Is the throughput of the mine continuously monitored? Is the throughput of individual sections of the longwall being measured? Is there any procedure to detect performance degradation in equipment other than programmed inspection or the fault made visible? Are there production/energy consumption metrics implemented?

Integrate Information: having a unique repository of information where data could be continuously analysed in a time feasibly way and considering all the involved factors.

- Plan systems ahead for enabling interoperability and standardise. Otherwise build reliable interfaces
- Develop or acquire required information infrastructure
- Business process oriented solutions to enable interoperability

**Key questions:** Is sensors information being used? How many stand alone devices, PLCs or SCADAs are working in mine operations? What information is recorded and daily analysed from such systems? What benefit is acquired from historical information obtained from such devices and equipment?

Analyse: creating the respective analysis methods and procedures. Data should be used to model optimal operational condition and performance for assets, processes and systems. Organisations usually fail in completing the follow-up analysis as concluded by Campbell and Jardine (2001) and the reporting that is expected to be continuous generally is neither regular nor timely.

- Determine the assets that impact more on the total system output
- Determine the right variables to monitor
- Track performance trends and analyse
- Detect evidence of degradation of performance/condition before the occurrence of a fault.
- Analyse undertaken actions and results

**Key questions:** Are the reasons for a decrease in mine throughput known? Are those continuously analysed? What is the optimal throughput of each of the critical assets in the Longwall? Is the throughput kept constant?
Execute: Execution of the measures necessary to solve the detected evolving degradation of performance or implementation of a continuous process to eliminate the root of the failure.

- Process compliance
- Manage teams and institute continuous processes
- Knowledge should trigger actions

Key questions: Which is the ratio of reactive maintenance vs. proactive maintenance? Is manual information recorded consistently? Are stoppages and time documented? Are CMMS systems predictive capability used to trigger inspections or maintenance?

If any of the stages in the cycle is not accomplished the cycle is not complete and the whole effort will not generate the desired value.

The information integrated architecture specific application for Asset Management was named Sensible Maintenance and is explained in the following sections with an illustrative case for longwall mine equipment.

SENSIBLE MAINTENANCE

Sensible maintenance is an asset management practice only enabled by the use of integrated information with the purpose of detecting degradation in asset performance and/or condition. Based in holistic information from operations and business and obtained from the system and equipment data. The concept aims at determining the relationship between equipment degradation and the degradation of performance and using this analysis for early detection of asset degradation and prevention of fault. Sensible maintenance model is shown in Figure 1.

![Figure 1 - Sensible Maintenance Model](image)

CASE STUDY INTEGRATED INFORMATION ARCHITECTURE APPLIED IN ASSET PERFORMANCE DEGRADATION DETECTION. (SENSIBLE MAINTENANCE)

In order to illustrate sensible maintenance, the concept obtained from applying an integrated information architecture for Asset Management purposes, a single application in longwall coal mining was selected.

Longwall shearer maintenance related literature, and performance information will be utilized to create a model for monitoring the performance and condition of this type of equipment. The recommended framework is based on the possible encountered failures in the equipment, its components and
subcomponents, although this study scope only covers the range of information that has to be monitored to detect the failure and encourages further study of the relationship of actual equipment condition with equipment performance.

A complete classification of shearer failures in terms of the root of the failure and component of the system where the fault is explained and organized as a fault tree by Gupta, Ramkrishna and Bhattacharya (2006). Only the section specifying the causes of failure of cutting-drums will be used for this case, which is presented in Figure 2.

![Figure 2 - Cutting-drum fault tree (Source Gupta, Ramkrishna and Bhattacharya 2006)](image)

**Detecting relevant variables to monitor**

Based in a hierarchical organisation; relevant information that can evidence degradation in the functionality of the system or its components can be selected. If we examine specifically the cutting drum component we will find that the basic sub-components that may fail are the picks and the water hoses and therefore the degradation can be evidenced by monitoring those components.

*Selected Component: Shearer Picks*

**Asset Function:** The main function of the picks is to cut the coal out of the coal bed.

**Environment variables:** Include variables that are not controlled and affect the accomplishment of the asset function as coal hardness or flatness of the coal bed.

**Controlled variables:** Include variables that are controlled by the operator or configured previously that relate with the accomplishment of asset function as cutting motor power of the leading drum, cutting depth, current rotation speed of the leading drum, shearer travelling speed.

**Internal uncontrolled variables:** are the variables that are measured from the components as condition monitored for example cutting drum motor temperature, components oil state, cutting drums vibration and ranging arms vibration.
Output variables: Are basically variables that are uncontrolled and are a result of the system operation as in this case metric tons of material per hour, electric consumption, dust in the environment, coal size and coal quality.

Timely detection of degradation of the condition and/or performance of any of the sub components may prove to be useful, since it increases the time available before the total failure of the shearer system takes place.

In fact it was detected with this analysis, that there are some system components from which no condition monitoring is being made, and in those components we found that a related performance measurement could still be made. This fact opens the possibility to monitor certain variables related to the piece of equipment for evidencing degradation in equipment from which it was obviously impossible to monitor its condition.

Designing the system of correlation of information

The main premise to design the system is:

The right correlation of parameters acquired from inputs and the environment should have a predictable outcome.

There by: the capacity of the system and asset should be predictable.

In practice the right system of correlation of information can only be obtained with real data, recorded when the equipment is achieving its expected performance under the operational context. But for the study case a model will be generated considering a possible correlation of variables.

Studied input parameters will be classified in environment variables, controlled input variables and condition variables:

- Environment variables such as coal hardness, composition of the strata, thickness of the coal bed, flatness of the coal bed, geological obstacles.
- Controlled input variables such as cutting motor power of the leading drum, cutting depth, height of leading drum, current rotation speed of the leading drum, shearer travelling speed.
- Condition variables such as cutting drum motor temperature, components oil state, components age, cutting drums vibration, ranging arms vibration.
- Performance variables such as metric tons per hour, electric consumption, dust in the environment, coal size, coal quality.

The relationship between these variables will be stored as numerical data. A combination of certain environmental conditions and intended controlled input should result theoretically in a throughput ranging between A and B, that is somehow predictable, after gathering data of operations utilising optimal equipment.

After gathering enough information about each variable mentioned in a holistic and correlated manner, we will have a notion of what can be considered a standard behaviour, and if performance is as expected then we have a numerical evidence of what can be considered an optimal behaviour.

Consequently the system analysis will lead to an understanding that if all the input variables stay constant then there should be no reason for an abnormal output. And that if a decrease in the metric tons per hour is detected and the current rotation speed of the leading drum and the shearer travelling speed remain constant and all the variables controlled remain constant probably there is a decrease of performance in some component that is related with the final output or the equipment is not being manipulated properly.

So for exemplifying the above stated, the example of the picks condition will be used:

If all the input variables stay constant, including the current rotation speed of the leading drum (measurement that is obtained by a rotation sensor or rotameter), cutting depth and shearer speed, the
average metric tons per hour is supposed to be predictable in certain degree. But if data measurements prove that average production has decreased under specified expected outcome (value that will be obtained from historical analysis of the system) it leads to the conclusion that degradation of the picks has taken place, because the last component before the studied component, the leading cutting drum, is working under expected rates of performance. Then the probability of needing maintenance of the picks is high, when any other possible causing factor hasn’t been identified. It has been explained by Tiryaki (2004) that actual problems related to clearance ring picks in longwall equipment cause excessive wear to the backplate, excessive vibrations of the machine and increases the amount of respirable dust and fines. Most of these are indicators that can be monitored and were placed among the variables to be considered since a relationship between those variables and the condition of the equipment is likely to exist and provide help in the early detection of equipment degradation.

The system analysis also lead us to one question, if all the variables in the system are not being considered as a whole in any known longwall site, is productivity degradation being accurately perceived for longwall equipment?

Bongers (2004) work proved that by adequately monitoring the condition of the equipment in a holistic framework it is possible to detect developing failures in longwall systems, even without the inclusion of performance measurements. Developing failures that can be detected with anticipation include: “blockage/overload stoppages, temperature trips and dupline faults” (Guan, Gurgenci and Meehan, 2005). Including performance monitoring to the state of the art of condition monitoring is expected to add value and certainty to actual Failure Detection and Isolation (FDI) practices and with time to maintenance management process itself.

CHALLENGES

Interoperability between information systems is a technical challenge that has not yet been completely managed. Several organizations are working on standards to close this technical gap between systems. The purpose of systems architecture initiative is to trace a vision of the ideal systems architecture oriented towards the purpose of the organisation.

Information systems can have a lack of flexibility. Evans (2007) found that organizational change usually disrupts the process and how the work is performed in this process, but it will not change the associated computer application and usually information recording process isn’t changed to mirror the process. This is one of the factors that should not be underestimated when implementing the systems architecture.

CONCLUSIONS

Longwall asset management function is generally judged in terms of equipment availability, and operations uptime, but rarely is it assessed in terms of the throughput or productivity of the equipment the function is responsible for. This fact alone generates several misunderstandings and loss opportunities for improvement given the lack of awareness. It is recommended through the presented framework to consider the importance of throughput, rather than the availability itself. In longwall sites and other mining sites this recommendation will prove beneficial since the metrics based in this premise will be aligned towards the real goal of the organization, increase profitability.

Currently operations and industrial recording processes and practice are not being studied extensively, and the range of possibilities this field offers have not been exploited to the maximum extent possible. Specifically in Longwall mining sites, data that is recorded is rarely used for fault detection and isolation. This could only contribute to a reactive rather than proactive framework of mind.

CMMS are not being implemented widely and its full potential is not being exploited probably because benefits are not recognized. Whenever corrective and scheduled maintenance work are practiced, a new approach will not be accepted unless the gained benefits are not only measurable but easy to perceive. Current proposed initiative creates the possibility to measure the results of every action undertaken by a maintenance function, since the architecture is focused in equipment performance management.

The current paper recommends a simple formula; equipment degradation can be early detected through monitoring process-throughput information and comparing it with the expected condition and
performance. Every piece of equipment or component has attributes or behaviours and for each diagnosis methods can be abstracted and used in a simple model for the optimization of maintenance functions. Yet this model requires a certain level of integration and organization of the information. The architecture is the enabler of such integration.

Things are not static, but on the opposite side absolutely dynamic. It was found that information architecture to be successful has to be conceived so it can support any change whether it is in process, structure, information, strategy or purpose.

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Pancucci, D, 2000b. Taking maintenance out onto the web, *Manufacturing Computer Solutions*. Pg. 25
PRODUCTIVITY IMPROVEMENT FROM ECONOMIC CONCEPT TO AN ENGINEERING TOOL

Kazem Oraee¹, Navid Hossein², Mohammadreza Soltani³, and Mehdi Amirafshari⁴

ABSTRACT: In broad terms, productivity is defined as the ratio of all outputs to all inputs. In this paper, productivity is distinctly used as an engineering tool and from practical viewpoint. With such engineering look, productivity is an adequate tool for evaluation of the advancement in competitive market, work assessment, profit/loss analysis, decision and even developing or changing the activity. Productivity measurement seems to be an easy task but this is a misconception. In fact the concept remains to be one of the most complex and unknown criteria. It is for this reason that attempts have been made here to accurately define productivity and hence simplify its measurement. A case study has been adopted and the productivity of Eastern Alborz Coal Mines in Iran has been calculated for years 2001-2008. The resulting values and the component models are then subject to analysis. These results are examined in terms of practicability and it is shown that the method prescribed is a pragmatic approach in all similar system situations.

INTRODUCTION

Productivity is the effective use of each factors of production which is defined as output to input (Oraee, 1998). With measurement of productivity during time, the trend of changes is defined. Increases or decreases of productivity are directly proportional to profit of company. Moreover, determination of productivity defined the ability of companies in competitive markets (Oraee, 2006). Productivity can be computed as partial and total. The partial productivity describes the ratio of output to each of inputs, including manpower, capital, energy, and etc., while the total productivity is the ratio of output to sum of inputs (Oraee and Pymander, 1998). The productivity is calculated for various purposes ((Oraee and Pomander, 1998), (Oraee, 1996, 2006) such as:

- The strategic, in competitive markets for survival and/or improvement.
- Technical, for verifying performance of various divisions.
- Planning, to verify profit/loss and the necessary decisions.
- Management, for development or change in kind of activities.

The productivity as a standard uses for estimation of efficiency and profitability. It helps the optimum allocation of resources. Therefore, the productivity can act as a parameter in forecasting and planning (Oraee, 1998).

In this study, the productivity of Eastern Alborz Coal Mines (EACM) is calculated. For this propose the total of outputs and inputs are computed for calculation of the total productivity and partial productivity including productivity of manpower, energy, and capital.

MINING OUTPUT

The first step in computing productivity is the measurement of outputs. The output is something that is produced, hence in mining definition, the output is the minerals. The output for EACM should be calculated in physical units (tons). The output also can be calculated as monetary units (US dollars), but this type of measurement is mostly for mines that produces the different types of minerals (Oraee, 1998). Table 1 (IMPASCO, 2009 and Soltani, 2009) shows the production of EACM during 2001 to 2008. To accurate analysis, selling price of coal also included. Moreover, in order to eliminate the

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inflation, based on inflation price index of wholesale of Iran (Price Index and Inflation, 2009), the prices are inflation adjusted. For this reason, the year 2001 is selected as a base and the index of each year is divided to the base year and the result is multiplied in to the selling amount of the same year.

Table 1 – The total output in EACM during 2001 to 2008

<table>
<thead>
<tr>
<th>Year</th>
<th>2001</th>
<th>2002</th>
<th>2003</th>
<th>2004</th>
<th>2005</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extracted coal (ton)</td>
<td>467,000</td>
<td>532,000</td>
<td>512,000</td>
<td>537,000</td>
<td>650,000</td>
<td>681,000</td>
<td>670,000</td>
<td>691,000</td>
</tr>
<tr>
<td>Concentrate (ton)</td>
<td>268,000</td>
<td>290,000</td>
<td>284,000</td>
<td>279,000</td>
<td>321,000</td>
<td>281,000</td>
<td>313,000</td>
<td>324,000</td>
</tr>
<tr>
<td>Price per ton of coal (US $)</td>
<td>68</td>
<td>73</td>
<td>83</td>
<td>101</td>
<td>113</td>
<td>126</td>
<td>133</td>
<td>186</td>
</tr>
<tr>
<td>Total sales (1000 US $)</td>
<td>18,000</td>
<td>21,000</td>
<td>24,000</td>
<td>28,000</td>
<td>36,000</td>
<td>35,000</td>
<td>42,000</td>
<td>60,000</td>
</tr>
<tr>
<td>Wholesale price inflation index</td>
<td>175</td>
<td>192</td>
<td>211</td>
<td>242</td>
<td>265</td>
<td>297</td>
<td>341</td>
<td>422</td>
</tr>
<tr>
<td>Inflation adjusted (1000 US $)</td>
<td>18,000</td>
<td>19,100</td>
<td>20,000</td>
<td>20,200</td>
<td>23,800</td>
<td>21,000</td>
<td>21,600</td>
<td>24,900</td>
</tr>
</tbody>
</table>

In Figure 1 the trend of production growth and inflation adjusted selling price of coal in each year compare to previous, during 2001 to 2008 is depicted. As seen in this figure, the trend of production and income (from selling coal) show significant changes and in some cases no harmony. For example, in 2006, in spite of growth in production (as compared with 2005), the income is decreased, on the contrary in 2007 although production decreased, but the income has increased.

Figure 1 – The trend of production and inflation adjusted selling price of coal in EACM during 2001 to 2008

1. Mining input

The mine input is divided into various divisions that their summation is the total mine input. In this research, also the mine input of EACM is divided into three groups: 1) manpower, 2) energy, and 3) capital input. Accordingly each input is considered, separately.

The manpower input

The manpower input has an important role in productivity calculation. The accepted unit for manpower is either working hours (time) or number of workers (Oraee, 1996). While the level of education, technical knowledge, expertise, service records, and similar criteria effect on labor cost. Obviously the efficiency and therefore the cost of inexperience labor in compare to an expert one, and or the comparison of each expert labor with an engineer is far different. Therefore, if the manpower is defined by number or working hours it is required the labor cost be defined based on particular index. Obviously, such a calculation is not easy and accurate. Thus, calculations of manpower input based on labor cost solve this problem. As, the salary difference always is a criteria in position of a personnel. In other words, using the costs to calculate the manpower input, relying on market-power to adjust the
value of each person with various skills. In the use of monetary unit for labor input must also be done inflation adjusted. Therefore, the consumer price index of Iran (Price Index and Inflation, 2009) has been used. The manpower data (Soltani, 2009) of EACM during 2001 to 2008 is presented in Table 2.

In Figure 2, variation in number and costs of personnel during 2001 to 2008 is shown. As can be seen, although the number and cost of personnel during these years have been declining, but reduce in rate of personnel number is more than personnel costs. In other word, the costs per person increases, that probably is due to technical knowledge and skills of personnel, or the policies of labor laws during 2005 to 2008. The first reason causes growth of total productivity, while the other's reduces the total productivity.

Table 2 – The manpower input in EACM during 2001 to 2008

<table>
<thead>
<tr>
<th>Year</th>
<th>Total of personnel</th>
<th>Number of underground personnel</th>
<th>Number of coal face workers</th>
<th>Total wages and salaries (1000 US $)</th>
<th>Consumer price inflation index</th>
<th>Inflation adjusted (1000 US $)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2001</td>
<td>2,248</td>
<td>1,679</td>
<td>319</td>
<td>13,000</td>
<td>178</td>
<td>13,000</td>
</tr>
<tr>
<td>2002</td>
<td>1,983</td>
<td>1,562</td>
<td>298</td>
<td>9,000</td>
<td>206</td>
<td>7,800</td>
</tr>
<tr>
<td>2003</td>
<td>1,692</td>
<td>1,484</td>
<td>207</td>
<td>10,000</td>
<td>238</td>
<td>7,500</td>
</tr>
<tr>
<td>2004</td>
<td>1,466</td>
<td>1,275</td>
<td>206</td>
<td>9,000</td>
<td>275</td>
<td>5,800</td>
</tr>
<tr>
<td>2005</td>
<td>1,240</td>
<td>1,056</td>
<td>161</td>
<td>10,300</td>
<td>308</td>
<td>5,900</td>
</tr>
<tr>
<td>2006</td>
<td>1,117</td>
<td>964</td>
<td>174</td>
<td>10,000</td>
<td>339</td>
<td>5,300</td>
</tr>
<tr>
<td>2007</td>
<td>1,058</td>
<td>922</td>
<td>149</td>
<td>10,000</td>
<td>402</td>
<td>4,700</td>
</tr>
<tr>
<td>2008</td>
<td>973</td>
<td>846</td>
<td>150</td>
<td>13,300</td>
<td>504</td>
<td>4,700</td>
</tr>
</tbody>
</table>

Figure 2 – The variation in number and costs of manpower in EACM during 2001 to 2008

Energy input

The energy is a part of the raw materials (Oraee, 2006), but in this research the energy input is calculated, distinctly. Because, the cost of energy is an important part of raw materials costs and in Iran, the variation of energy cost is not depend on variation of market prices and the laws of supply and demand. Energy is measured by the unit such as kilo-Watt per hour, kilograms, and litters. The summation of different type of energy the BTU unit is used. Since, the summation all inputs is for calculation of total productivity, in this study the total cost of energy is used. The electricity and fossil fuels are the main energy consumption in EACM, which in Table 3 is given based on costs (Soltani, 2009). These costs by using consumer price index of Iran (Price Index and Inflation, 2009) for various years are inflation adjusted.

The trend of inflation adjusted prices variation during 2001 to 2008 is given in Figure 3. Although, the final price is decreased, however the sinusoidal fluctuations indicate the change in productivity during these years.
Table 3 – The energy input in EACM during 2001 to 2008

<table>
<thead>
<tr>
<th>Year</th>
<th>2001</th>
<th>2002</th>
<th>2003</th>
<th>2004</th>
<th>2005</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
</tr>
</thead>
<tbody>
<tr>
<td>Costs of fuel and electricity (1000 US $)</td>
<td>390</td>
<td>540</td>
<td>560</td>
<td>520</td>
<td>590</td>
<td>630</td>
<td>820</td>
<td>780</td>
</tr>
<tr>
<td>Consumer price inflation index</td>
<td>178</td>
<td>206</td>
<td>238</td>
<td>275</td>
<td>308</td>
<td>339</td>
<td>402</td>
<td>504</td>
</tr>
<tr>
<td>Inflation adjusted (1000 US $)</td>
<td>390</td>
<td>470</td>
<td>420</td>
<td>340</td>
<td>340</td>
<td>330</td>
<td>360</td>
<td>280</td>
</tr>
</tbody>
</table>

Figure 3 – The trend of inflation adjusted energy cost in EACM during 2001 to 2008

Capital inputs

The capital includes buildings, machinery, equipment, and the amount of reserves in a particular point of time, such as the end of calendar year (Oraee, 1996). Building maintenance and repairs are needed, machinery and equipment get depreciated and the new investments will be necessary. Therefore, the calculation of capital inputs is difficult. If the buildings and machineries be rented by company, the calculation of capital costs is quite easy (Oraee and Pomander, 1998). Otherwise, in productivity calculation the depreciation costs as a capital costs is used. In this research, the depreciation of building by straight-line method with life of 30 years is computed. The industrial machinery by declining-balance method with depreciation rates of 35 percent is depreciated. The light vehicles by straight-line method with life of 6 years have been calculated. Also, the properties and working equipments by declining-balance method with depreciation rate of 15 percent and the office supplies and furniture by straight-line with life of 15 years have been depreciated. Moreover, properties, machineries and equipments based on final price are included in the calculation. The cost of overhaul and maintenance and the minor repairs on occurrences are included as the current cost of the same year. The depreciation costs are given in Table 4 (Soltani, 2009).

The costs of spare parts, industrial parts and timber, arc and other tunnel equipments for the years of 2001 to 2008 given in Table 5 (Soltani, 2009). In addition, the wholesale price inflation index of Iran (Price Index and Inflation, 2009) has been used for inflation adjusted. The costs of transportation and maintenance of EACM during 2001 to 2008 is shown in Table 6 (Soltani, 2009). The costs for the said years based on the consumer price inflation index of Iran (Price Index and Inflation, 2009) are inflation adjusted.

Table 4 – The depreciation costs in EACM during 2001 to 2008

<table>
<thead>
<tr>
<th>Year</th>
<th>2001</th>
<th>2002</th>
<th>2003</th>
<th>2004</th>
<th>2005</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depreciation costs (1000 US $)</td>
<td>700</td>
<td>700</td>
<td>1,400</td>
<td>1,300</td>
<td>1,400</td>
<td>1,900</td>
<td>2,100</td>
<td>2,600</td>
</tr>
</tbody>
</table>
Table 5 – The costs of spare parts, industrial parts and timber, arc and other tunnel equipments in EACM during 2001 to 2008

<table>
<thead>
<tr>
<th>Year</th>
<th>2001</th>
<th>2002</th>
<th>2003</th>
<th>2004</th>
<th>2005</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spare parts, industrial parts (1000 US $)</td>
<td>1,000</td>
<td>1,000</td>
<td>900</td>
<td>1,100</td>
<td>1,200</td>
<td>1,400</td>
<td>1,100</td>
<td>1,000</td>
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<tr>
<td>Timber, arc and tunnel equipments (1000 US $)</td>
<td>39</td>
<td>16</td>
<td>8</td>
<td>2</td>
<td>8</td>
<td>18</td>
<td>18</td>
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<tr>
<td>Total (1000 US $)</td>
<td>1,000</td>
<td>1,000</td>
<td>900</td>
<td>1,100</td>
<td>1,200</td>
<td>1,400</td>
<td>1,100</td>
<td>1,000</td>
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<tr>
<td>Wholesale price inflation index</td>
<td>175</td>
<td>192</td>
<td>211</td>
<td>242</td>
<td>265</td>
<td>297</td>
<td>341</td>
<td>422</td>
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<tr>
<td>Inflation adjusted (1000 US $)</td>
<td>1,000</td>
<td>900</td>
<td>700</td>
<td>800</td>
<td>800</td>
<td>800</td>
<td>600</td>
<td>400</td>
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</tbody>
</table>

Table 6 – The costs of transportation and maintenance in EACM during 2001 to 2008

<table>
<thead>
<tr>
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<th>2001</th>
<th>2002</th>
<th>2003</th>
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<tr>
<td>Maintenance costs (1000 US $)</td>
<td>700</td>
<td>600</td>
<td>800</td>
<td>900</td>
<td>2,200</td>
<td>2,400</td>
<td>1,500</td>
<td>2,600</td>
</tr>
<tr>
<td>Transportation costs (1000 US $)</td>
<td>600</td>
<td>700</td>
<td>500</td>
<td>800</td>
<td>1,400</td>
<td>600</td>
<td>1,800</td>
<td>2,200</td>
</tr>
<tr>
<td>Total (1000 US $)</td>
<td>1,300</td>
<td>1,300</td>
<td>1,300</td>
<td>1,700</td>
<td>3,600</td>
<td>3,000</td>
<td>3,300</td>
<td>4,800</td>
</tr>
<tr>
<td>Consumer price inflation index</td>
<td>178</td>
<td>206</td>
<td>238</td>
<td>275</td>
<td>308</td>
<td>339</td>
<td>402</td>
<td>504</td>
</tr>
<tr>
<td>Inflation adjusted (1000 US $)</td>
<td>1,300</td>
<td>1,100</td>
<td>1,000</td>
<td>1,100</td>
<td>2,100</td>
<td>1,600</td>
<td>1,500</td>
<td>1,700</td>
</tr>
</tbody>
</table>

Also, the costs of work equipments, stores, bank loans, and the other capital expenditures with annual interest rate of 15 percent (Soltani, 2009) are given in Table 7 (Price Index and Inflation (2009)).

Table 7 – The costs of work equipments, stores, bank loans, and etc. in EACM during 2001 to 2008

<table>
<thead>
<tr>
<th>Year</th>
<th>2001</th>
<th>2002</th>
<th>2003</th>
<th>2004</th>
<th>2005</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
</tr>
</thead>
<tbody>
<tr>
<td>Work equipments and store (1000 US $)</td>
<td>800</td>
<td>500</td>
<td>2,400</td>
<td>2,000</td>
<td>1,900</td>
<td>1,800</td>
<td>1,600</td>
<td>1,400</td>
</tr>
<tr>
<td>Bank loans (1000 US $)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>300</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Other capital expenditures (1000 US $)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total (1000 US $)</td>
<td>800</td>
<td>500</td>
<td>2,400</td>
<td>2,000</td>
<td>1,900</td>
<td>2,100</td>
<td>1,600</td>
<td>1,400</td>
</tr>
</tbody>
</table>

Table 8 shows the total of capital inputs of EACM during years of 2001 to 2008 which are calculated based on tables 4 to 7.

Table 8 – the total of capital inputs in EACM during 2001 to 2008

<table>
<thead>
<tr>
<th>Year</th>
<th>2001</th>
<th>2002</th>
<th>2003</th>
<th>2004</th>
<th>2005</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total of capital costs (1000 US $)</td>
<td>3,800</td>
<td>3,200</td>
<td>5,500</td>
<td>5,200</td>
<td>6,200</td>
<td>6,400</td>
<td>5,800</td>
<td>6,100</td>
</tr>
</tbody>
</table>

In Figure 4, the trend of capital inputs of EACM is depicted. According to this figure, the capital inputs showing the increasing trends, which can reduce the capital productivity or even the total productivity.

In Figure 5, the share of each cost in total costs during 2001 to 2008 is given in percentages. As can be seen, the share of personnel costs are reduced during these years, but the share of capital costs with the same ratio has increased. While, the energy costs is almost constant.
Thus, the total inputs of EACM including the manpower, energy, and capital are as in Table 9.

Table 9 – Total inputs in EACM during 2001 to 2008

<table>
<thead>
<tr>
<th>Year</th>
<th>2001</th>
<th>2002</th>
<th>2003</th>
<th>2004</th>
<th>2005</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manpower input (1000 US $)</td>
<td>13,000</td>
<td>7,800</td>
<td>7,500</td>
<td>5,800</td>
<td>5,900</td>
<td>5,300</td>
<td>4,700</td>
<td>4,700</td>
</tr>
<tr>
<td>Energy input (1000 US $)</td>
<td>390</td>
<td>470</td>
<td>420</td>
<td>340</td>
<td>340</td>
<td>330</td>
<td>360</td>
<td>280</td>
</tr>
<tr>
<td>Capital input (1000 US $)</td>
<td>3,800</td>
<td>3,200</td>
<td>5,500</td>
<td>5,200</td>
<td>6,200</td>
<td>6,400</td>
<td>5,800</td>
<td>6,100</td>
</tr>
<tr>
<td>Total (1000 US $)</td>
<td>17,200</td>
<td>11,500</td>
<td>13,400</td>
<td>11,300</td>
<td>12,400</td>
<td>12,000</td>
<td>10,900</td>
<td>11,000</td>
</tr>
</tbody>
</table>

PRODUCTIVITY CALCULATION IN EACM

Productivity calculated according to the either monetary units or physical units. Using monetary units in define of output and input indicating the economic productivity or profitability (Oraee and Pomander, 1998). At this point, by using the ratio of output (as monetary unit) per total inputs, the total productivity and next with using the inputs of personnel, energy and capital, the partial productivity of the EACM during 2001 to 2008 are calculated and presented in Table 10.

Figure 6 shows the trends of total and partial productivity including personnel, energy, and capital based on monetary unit during 2001 to 2008. According to this figure, the trend of the personnel productivity changes and the capital productivity are compatible with the total productivity. These three productivity indexes reduce significantly in 2003, but showing almost constant onwards. The sinusoidal fluctuations in trends of productivity are a sign of system instability.

The trends of the energy productivity during these mentioned years were instable. As such after dramatic decrease in 2005 and 2006, reaching to a constant trends in 2007, but shows the sudden increases in 2008. According to the Figure 6, the increases of the energy productivity in 2008, is an unusual result as the capital productivity decreased and also the global price of energy has been increased in the same year.
Table 10 – Productivity at EACM based on monetary units during 2001 to 2008 ($/§)

<table>
<thead>
<tr>
<th>Year</th>
<th>2001</th>
<th>2002</th>
<th>2003</th>
<th>2004</th>
<th>2005</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total productivity (1000 US $/1000 US $)</td>
<td>1.0</td>
<td>1.7</td>
<td>1.5</td>
<td>1.8</td>
<td>1.9</td>
<td>1.8</td>
<td>2.0</td>
<td>2.3</td>
</tr>
<tr>
<td>Labor productivity (1000 US $/1000 US $)</td>
<td>1.4</td>
<td>2.4</td>
<td>2.7</td>
<td>3.5</td>
<td>4.0</td>
<td>4.6</td>
<td>4.3</td>
<td>5.3</td>
</tr>
<tr>
<td>Energy productivity (1000 US $/1000 US $)</td>
<td>46</td>
<td>41</td>
<td>48</td>
<td>59</td>
<td>70</td>
<td>64</td>
<td>60</td>
<td>89</td>
</tr>
<tr>
<td>Capital productivity (1000 US $/1000 US $)</td>
<td>4.7</td>
<td>6.0</td>
<td>3.6</td>
<td>3.9</td>
<td>3.8</td>
<td>3.3</td>
<td>3.7</td>
<td>4.1</td>
</tr>
</tbody>
</table>

Figure 5 – The share of each cost component in EACM during 2001 to 2008

Figure 6 – Variations in total productivity and partial productivity based on monetary units during 2001 to 2008 ($/§)
The productivity of EACM based on physical units (physical per monetary unit) during 2001 to 2008 is given in Table 10.

Table 10 – Productivity at EACM based on physical and monetary units (ton/1000 US $) during 2001 to 2008

<table>
<thead>
<tr>
<th>Year</th>
<th>2001</th>
<th>2002</th>
<th>2003</th>
<th>2004</th>
<th>2005</th>
<th>2006</th>
<th>2007</th>
<th>2008</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total productivity (ton/1000 US $)</td>
<td>27</td>
<td>46</td>
<td>38</td>
<td>48</td>
<td>52</td>
<td>57</td>
<td>61</td>
<td>63</td>
</tr>
<tr>
<td>Labor productivity (ton/1000 US $)</td>
<td>36</td>
<td>68</td>
<td>68</td>
<td>93</td>
<td>110</td>
<td>128</td>
<td>143</td>
<td>147</td>
</tr>
<tr>
<td>Energy productivity (ton/1000 US $)</td>
<td>1,197</td>
<td>1,132</td>
<td>1,219</td>
<td>1,579</td>
<td>1,912</td>
<td>2,064</td>
<td>1,861</td>
<td>2,468</td>
</tr>
<tr>
<td>Capital productivity (ton/1000 US $)</td>
<td>123</td>
<td>166</td>
<td>93</td>
<td>103</td>
<td>105</td>
<td>106</td>
<td>116</td>
<td>113</td>
</tr>
</tbody>
</table>

In Figure 7 also the trend of total productivity and partial productivity that includes productivity of manpower, energy, and capital based on physical unit per monetary unit are presented. As can be seen, the trend of total productivity variation with productivity of manpower and capital are similar, but the trend of energy productivity is different and shows the sinusoidal behavior.

Figure 7 – Variations in total productivity and partial productivity based on physical and monetary units (ton/$) during 2001 to 2008

The Figure 8 shows the trend of total productivity based on monetary unit (US dollars/US dollars) and physical unit (ton/US dollars) during 2001 to 2008. As expected, until year 2004 these two indexes of total productivity are similar, but in 2005 the trend of total productivity based on monetary unit extremely decreases in opposite of the other index! In 2006 it reaches to lowest limit and increases progressively for the two next years!

CONCLUSION

Although, the productivity is a simple economic concept defines as an output to input ratio, but from an engineering view is a critical tool. Such a tool is a key role on evaluation, analysis, comparison, and generally in decision-making processes for knowledge-base economy. The trend of variation of total productivity based on physical per monetary unit shows that the EACM may lead to an unsuitable economic status. While the trend of changes on total productivity based on monetary unit indicating a partial growth in recent years. Such a growth may be either based on inflation index stated by central bank of Iran during these periods or based on engineering and economic concept. Perhaps, growth in partial productivities such as productivity of manpower, capital and particularly, energy may be essential for improvement of EACM. In this connection, mechanization is key factor as it improves productivity beside the other important factors like administrative and management. To ignore the above said factors would reduce the competitive ability of EACM in market. With increases the
production costs especially, energy cost (based on real price, not subsidies) producing of coal may not have economic justification, ever.

Figure 8 – Variations in total productivity and profitability at EACM during 2001 to 2008

REFERENCE

Oraee, K., Mining Economics, Amirkabir University Press, 1996.
### INDEX TO AUTHORS

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