Proceedings of the 2021 Resource Operators Conference

Naj Aziz  
*University of Wollongong, naj@uow.edu.au*

Ali Mirzaghorbanali  
*University of Southern Queensland*

Follow this and additional works at: [https://ro.uow.edu.au/coal](https://ro.uow.edu.au/coal)

**Recommended Citation**

Research Online is the open access institutional repository for the University of Wollongong. For further information contact the UOW Library: research-pubs@uow.edu.au
EDITORIAL BOARD

NAJ AZIZ
JAN NEMCIK
ISMET CANBULAT
JASON EMERY

ALI MIRZAGHORBANALI
JOHN HOELLE
ROGER BYRNES
ZHENJUN SHAN

Typeset by
Zhenjun Shan
University of Wollongong

ADVISORY BOARD

Naj Aziz, University of Wollongong
Bob Kininmonth, Illawarra Outburst Committee
Belle Bharat, Anglo American
Jan Nemcik, UOW
Basil Beamish, B3 Mining Services, Aust.
Roger Byrnes, Wollongong
David Evans, DSI Australia
Dan Payne, BHP, Brisbane
Martin Watkinson, Simeants Queensland
David Cliff, University of Queensland
Frank Hungerford, University of Wollongong
Ismet Canbulat, UNSW, Australia
John Hoelle, Braemar Geotech, Australia
Stuart MacGregor, SCT Operations
Patrycja Sheffield, Centennial Coal
Andrew Seccombe, BlackRock, Qld

Jacqui Purcell, LDO Group, NSW
Richard Campbell, Queensland
Rod Doyle, Hume Coal Pty Limited, NSW
Kevin Marston Aus IMM-Illawarra Branch
Paul Hagan, UNSW, Australia
Robert Hawker, Minova Australia
Jason Emery, Optimum Geotechnics
Russell Frith, Advance Mining
Peter Craig, Jennmar Australia
Ray Tolhurst, AusIMM Illawarra Branch
Terry Medhurst, PDR Engineers
Ali Mirzaghordanali, USQ
Ross seedsman, Seedsman geotechnic
Gavin Lowing, Peabody Energy
Shahin Aziz, Wollongong

REVIEWERS

Naj Aziz, University of Wollongong (UOW)
Bob Kininmonth, Illawarra Outburst Committee
Basil Beamish, B3 Mining Services, Australia
Yvette Heritage, SCT
Jan Nemcik, University of Wollongong
David Cliff, University of Queensland
Ismet Canbulat, UNSW, Australia

John Hoelle, Braemar Geotech, Australia
Peter Craig, Jennmar Australia
Ali Mirzaghordanali, University of Southern Queensland
Kevin Marston- Aus IMM Illawarra Branch
Patrycja Sheffield, Centennial Coal
Ross seedsman, Seedsman geotechnic
SPONSORS AND EXHIBITORS

SPONSORS

[Austmine logo] [Blue Haulers logo] [DSI logo] [Jennmar logo]

[Master Builders Solutions logo] [Minova logo] [SCT logo] [Quarry Mining logo]

SUPPORTERS

[AusIMM logo] [University of Wollongong Australia logo] [Mine Managers Association of Australia logo]

[BBUGS logo]
CONTENTS

PROCEEDINGS OF THE 2021 RESOURCE OPERATORS CONFERENCE .............................................. i
EDITORIAL BOARD .................................................................................................................. ii
SPONSORS AND EXHIBITORS ............................................................................................... iii
CONTENTS ............................................................................................................................... iv
PREFACE ................................................................................................................................. vii
CONFERENCE BOOK COVERS ............................................................................................ viii

The Changes and Future of Australian’s Resources Sector and the Advancements the Industry Has Made in Relation to Safety
Ian Macfarlane ....................................................................................................................... 1

Hydrogeological Properties and Mining Impacts on Groundwater in the Bowen, Styx and Galilee Basins
George Klenowski and John Bernal ...................................................................................... 6

Stress Change Near a Major Geological Structure during Longwall Mining
Baotang Shen, Xun Luo and Joey Duan ................................................................................ 17

The Fundamentals of Modern Ground Control Management in Australian Underground Coal Mines
Jason Emery, Ismet Canbulat and Chengguo Zhang ............................................................. 26

Thin Spray-on Liners: a Historical overview and Their Future Potential as Support in Underground Coal Mines
Claire Morton, Zhongwei Chen and Mehmet Siddik Kizil ..................................................... 42

Geotechnical Aspects of the Pike River Mine Drift Recovery
Chris Lee, Stuart MacGregor and Dinghy Pattinson ............................................................. 50

Developing an Innovative Protective Structure on Continuous Miners Against Coal Burst Hazards
Ting Ren, Alex Remennikov, Xiaohan Yang, Dulara Kalubadanage and Peter Holt ............... 63

Using Remote Reading Instrumentation to Improve Safety, Productivity, and Support Design in Underground Coal Mines
Samantha Watson, Nathan Owen and Claire Morton ............................................................. 71

A First-Principles Causation Hypothesis for Pillar Bursts in Underground Coal Mines
Russell Frith ......................................................................................................................... 78

Backfill Grouting for Mining Subsidence Prevention
Habib Alehossein, Baotang Shen and Zongyi Qin ................................................................. 91

Further Insights into the Mechanics of Multi-Seam Subsidence from Ashton Underground Mine
Ken Mills and Stephen Wilson .............................................................................................. 101

Goaf Gas Distribution near the Tailgate Under Three Gateroad Conditions
Rao Balusu, Bharath Belle and Krishna Tanguturi ................................................................. 115
Active Roadway Explosion Barrier Evolution in Coal Mining

Arend Späth and Bharath Belle ................................................................. 124

Why Slender Beam/Column Behaviour Should not be Ignored for Effective Ground Support Design

Mark Colwell and Russell Frith, ................................................................. 138

Coastal Reservoir-A Technology to Supply Sufficient, High-Quality and Affordable Water to Industry with Minimum Environmental/Social Impact

Shu-Qing Yang ......................................................................................... 158

Water Tracer Technologies to Defect Sources of Seepage and Protect Environmental Assets

Wendy Timms, Devmi Kurukulasuriya, Bill Howcroft, Ellen Moon and Karina Meredith ......................... 167

Filter Requirements for Graham’s Ratio Oxygen Deficiency

Snezana Bajic, Sean Muller and Mladen Gido ........................................... 177

Dynamic Model of Fault Slip and Its Effect on Coal Burst in Deep Mines

Jan Nemcik, Gaetano Venticinque, Zhenjun Shan and Libin Gong ..................... 186

Benchmarking Study by Laboratory Load Transfer Testing along Full Resin Encapsulated Rock Bolts

Sabitha Sasi and Peter Craig .................................................................. 194

Effect of Pretension on the Mechanical Behaviour of Bolted Rock

Mahdi Saadat and Abbas Taheri ................................................................ 203

Introduction to New Methods of Static and Dynamic Pull Testing of Rock Bolts and Cable Bolts

Sina Anzanpour, Naj Aziz, Jan Nemcik, Ali Mirzaghorbanali, Jordan Wallace, Travis Marshall and Saman Khaleghparast .............................................................. 210

Numerical Approach towards Dynamic Double Shear Testing of Tendons Using LS-DYNA

Saman Khaleghparast, Alex Remennikov and Naj Aziz ................................ 218

Pretension Effect on the Performance of Cable Bolts in Underground Coal Mines

Xu Li, Guanyao Si, Joung Oh, Zizhuo Xiang, Naj Aziz and Ali Mirzaghorbanali ................. 229

Angled Shear Testing of 15.2 mm Seven Wire Cable Bolt

Naj Aziz, Sina Anzanpour, Saman Khaleghparat, Ashkan Rastegarmanesh, Ali Mirzaghorbanali, Alex Remmenikov, Jan Nemcik, Joung Oh and Guanyao Si ........................................... 239

Respirable Coal Dust and Silica Exposure Standards in Coal Mining: Science or Black Magic?

Nikky LaBranche, David Cliff, Kelly Johnstone and Carmel Bofinger ....................... 251

Laboratory and Field Testing of Surfactants Used to Meet New Workplace Exposure Standards for Respirable Dust in Coal Mines

Neil Alston, Ping Chang, Zidong Zhao and Apurna Ghosh .................................. 261

Beyond Blasting: Evaluation of a Disc-Based Rock Cutting System for Surface Coal Mining

Isaac Dzakpata, Dihon Tadic, Amin Mousavi and Mehmet Kizil ........................................... 270

Determining Rock Elastic Parameters Using a New True-Triaxial-Based Technique

University of Wollongong, February 2021

V
Robert Purser, Mutaz El-Amin Mohmoud, Mehdi Serati and Zhongwei Chen.................................................285
Axial Behaviour of Rock Bolts–Part (A) Experimental Study
Hadi Nourizadeh, Sally Williams, Ali Mirzaghorbanali, Kevin McDougall, Naj Aziz and Mehdi Serati.....294
Axial Behaviour of Rock Bolts–Part (B) Numerical Study
Hadi Nourizadeh, Ali Mirzaghorbanali, Kevin McDougall, Naj Aziz and Mehdi Serati .........................303
Fracture Propagation Model of Coal Under Indirect Tensile Stresses
Mehdi Serati, Hamid Roshan, Ali Mirzaghorbanali, Mutaz El-Amin Mahmoud and Thejaswee Valluru .311
A Case of Longwall Coal Mining Productivity & Safety Optimisation
Sahar Ardehali, Naj Aziz, Habib Alehossein, Matthew Bowerman and Matt Robbins..........................317
Numerical Modelling on the Stability of an Underground Research Facility Excavation in Czech Republic
Libin Gong, Petr Waclawik, Kamil Soucek, Martin Vavro, Jan Nemcik, Sahendra Ram and Radovan Kukutsch
........................................................................................................................................................................325
Application of Monte Carlo Simulation to Quantify Uncertainties of First Weighting Interval Estimation
Sadjad Mohammadi, Mohammad Ateai, Ali Mirzaghorbanali and Naj Aziz ..............................................345
Improving Rock Mechanical Properties Estimation Using Machine Learning
Ruizhi Zhong, Matt Tsang, Gift Makusha, Ben Yang and Zhongwei Chen.................................................353
Characterization of the Fracture Mode in Asphalt at Varying Temperatures
Mehdi Serati, Thejaswee Valluru and Ian Van Wijk ..............................................................................362
Effect of Surface Profile on Axial Load Transfer Mechanism of Tendons
Ashkan Rastegarmanesh, Joel Misa, Ali Mirzaghorbanali, Naj Aziz and Kevin McDougall.....................369
Mining Equipment Human Factors Design for Workforce Diversity
Danellie Lynas, Gary Dennis and Robin Burgess-Limerick ......................................................................383
INDEX TO AUTHORS ........................................................................................................................................391
PREFACE

On behalf of the organising committee, we welcome you to the Resource Operators’ Conference (ROC2021), a rebranded name of the well-known Coal Operators’ Conference. The conference is hosted jointly between the University of Wollongong (UOW) and the University of Southern Queensland (USQ) and is supported by the Illawarra Branch of the Australian Institute of Mining and Metallurgy and the Mine Managers Association of Australia.

Holding the conference under such difficult conditions has presented many challenges; remaining up-to-date and considerate of the current COVID-19 pandemic - no matter how scary – has felt somewhat controllable. Dealing with climate change activists on the other hand has been difficult and daunting, to say the least.

In this conference 30 papers out of the total of 37 papers were presented over the two days online conference. The conference focused on innovations and best practice in the areas of worker safety and reducing the environmental impact of the mining sector. Additionally the conference addressed issues relating to various aspects of modern mining operations, both surface and underground, bringing industry experts and university researchers together to share their expertise and knowledge to help the industry be safer and more efficient.

While organising a conference during a pandemic and then switching it to a new format presented many challenges, the end result was well worth the effort and provided a valuable forum for sharing best practice and innovation in safety and environmental performance for the mining sector. The Hon Ian Macfarlane, Chief Executive of the Queensland Resources Council was guest speaker. Addressing the conference on the topic of “The challenges and future of Australian’s resources sector and the advancements the industry has made in relation to safety”, his speech is included in this proceeding.

Thanks are extended to; Professor John Bell, DVC Research from the University of Southern Queensland, for his encouraging words and Professor Valerie Linton for her full support and encouragement, which energised us to make this conference go ahead after a year’s hiatus. Thanks also to BBUGS, NUGS and the conference sponsors. Finally, a big thank you to the organising team, which made this conference possible, in particular Johlene Morrison of the UOW for her exceptional organisational and editorial skills, Dr. Ali Mirzaghorbanali of USQ, Kevin Marsden, Dr. Zhenjun (Ricky) Shan, and the conference advisory committee members. Finally, sincere thanks to authors and participants, who form the backbone of the conference success. The online zoom connection was facilitated by the University of Southern Queensland.

All papers are peer reviewed to maintain the conference’s high standing and recognition. All proceedings are available online through the University of Wollongong Library Research Online http://ro.uow.edu.au/coal

Professor Naj Aziz
Conference executive chairman

Dr. Ali Mirzaghorbanali
Conference executive co-chair
CONFERENCE BOOK COVERS
THE CHANGES AND FUTURE OF AUSTRALIAN’S RESOURCES SECTOR AND THE ADVANCEMENTS THE INDUSTRY HAS MADE IN RELATION TO SAFETY

Ian Macfarlane

Ladies and gentlemen, it is my very great privilege to acknowledge the traditional custodians of the land on which we live and work, and of the many different nations across the length and breadth of our great state.

We pay our respects to the Elders, past, present and emerging as the holders of the memories, the traditions, the culture and the spiritual wellbeing of the Aboriginal and Torres Strait Islander peoples across Queensland and the nation.

I’m delighted to be with you, albeit virtually, at this early hour. It’s disappointing that we can’t all be together in person, though, as we’ve all discovered, there are upsides to meeting online, such as being able to sit there in your casual clothes and slurp your coffee. Please remember to mute your microphone if this is you.

Ladies and gentlemen, today I want to share my excitement about the bright future for Queensland’s resources sector. And it is a bright outlook, despite the apparently imminent death of the coal industry that seems to dominate media headlines.

To paraphrase the words of Mark Twain, “The reports of the death of the coal industry are greatly exaggerated.” I’m delighted to be with you, albeit virtually, at this early hour. It’s disappointing that we can’t all be together in person, though, as we’ve all discovered, there are upsides to meeting online, such as being able to sit there in your casual clothes and slurp your coffee. Please remember to mute your microphone if this is you.

Ladies and gentlemen, today I want to share my excitement about the bright future for Queensland’s resources sector. And it is a bright outlook, despite the apparently imminent death of the coal industry that seems to dominate media headlines. To paraphrase the words of Mark Twain, “The reports of the death of the coal industry are greatly exaggerated.”

I’m here to tell you Queensland’s mineral wealth has never been in greater demand. The world wants and needs the valuable resources that our resource-rich state can provide.

While our industry has been faced with some trading challenges involving China, fortunately they are not the only country in the world who wants to lift its people out of poverty with the help of our abundant, high-quality coal, gas and minerals.

Resources companies have been busy shoring up new market opportunities and benefiting from the large increase in global coal prices on the back of market disruptions.

In fact, you might be surprised to learn that despite 2020 being the Year of COVID, the Queensland resources industry continues to have plenty of things to smile about. Such as the latest data from the Australian Bureau of Statistics that shows Queensland achieved a record number of coal job for the November quarter, and in fact reported the highest number of resources jobs since the record was set in 2013.

Coal jobs rose by an incredible 40 percent during the quarter - increasing from just over 28,000 to almost 40,000 - which demonstrates how crucial the resources sector is to Queensland’s economy and to employment. It also demonstrates jobs in coal, and jobs in resources, are far from dead. Our industry will continue to transition, as it always has, from one year and one decade and one century to the next, influenced by market forces, community expectations and political decisions. All resources companies are part of the world’s transition to a lower emissions future, and exciting innovations are

1 Guest speaker, Queensland Resource concil (QRC) Chief Executive
happening in this space driven by the resources sector. But it won’t happen overnight, and in the meantime our industry and the people who work in have a very important role to play in supporting life as we know it on this planet.

The latest ABS jobs data shows total Queensland resources jobs increased 23 percent over the previous quarter to reach more than 78,000 jobs. This is 18 percent higher than the same period in 2019, which was unaffected by the global pandemic. This means nearly 12,000 additional jobs were added in the August to November quarter.

Oil and gas jobs have also recovered strongly, increasing 147 percent between the August and November quarters. Added to that, December trade data shows the value of Australian coal exports rose by 26 percent over the previous month, demonstrating that coal continues to be an important part of the global energy and industrial mix, and will be for some time to come.

December was also a very good month for Queensland in terms of tonnage, with state coal exports up 19 percent on November, increasing from 16 million tonnes to just over 19 million tonnes. Resources companies continue to perform and exceed expectations, despite COVID-19, which benefits every Queenslander through taxes, royalties, exports, jobs and business opportunities. I’d like to mention here the opportunities a strong resources sector can and does bring to Queensland’s Indigenous community. With many of our operations on the doorstep of Indigenous communities, our industry has an important role to play in providing career opportunities to develop expertise and economic independence.

Over many years, the QRC and our industry have worked in partnership with Indigenous communities and government bodies to create sustainable economic development through employment and business opportunities to help build strong Indigenous families and communities in Queensland. That’s why we were delighted to welcome in October last year the extended commitment from the Department of Aboriginal and Torres Strait Islanders to work with the QRC and its members to boost employment opportunities for Indigenous Queenslanders.

This partnership, forged 12 years ago, helps ensure the resources sector continues to be a leading provider of opportunities for Aboriginal and Torres Strait Islanders.

Throughout the state, our member companies are providing game-changing opportunities for Indigenous Queenslanders, not just for jobs, but for fulfilling careers and business opportunities.

We’re proud to say the proportion of Indigenous Queenslanders working in the resources sector is on par with the proportion of Aboriginal and Torres Strait Islanders living in the state, which is 4 percent. So, we’re clearly getting a whole lot of things right, with more to come. That said, the unfortunate Juukan caves destruction in the Pilbara in Western Australia last year marked a radical change in the relationship between traditional owners and resources companies.

Public scrutiny following this event is leading to a complete overhaul of the way some companies approach relationships with Indigenous communities, and vice versa.

The subsequent Senate inquiry, having concluded its investigation into the handling of that incident, will soon turn its attention to the cultural heritage frameworks of the various states and territories.

The challenge for the resources sector in Queensland lies in maintaining our excellent record on cultural heritage and evidence of the skills and attributes indigenous people bring to our workforce and to the quality and diversity of leadership and performance across our industry is showcased every year at the presentation of QRC Indigenous Awards.

Indigenous engagement and cooperating with governments to ensure any changes to the cultural heritage framework continue to allow companies to operate under best-practice guidelines.

Some in the community may be surprised to know resources companies are also leading the way in exploring and investing in new-economy minerals, technologies and renewable energy projects. This is so our industry can continue to responsibly operate and contribute to the state economy and to jobs in the long term.

It’s certainly an exciting time to be part of the resources sector and Queensland is in prime position to benefit from our industry’s long-established reputation for resilience and innovation. When you think back to the very early - and very tough - days of mining in Queensland, and look at where we are now,
you would never doubt our industry’s ability to constantly move forward and improve our performance across every measure.

Projects such as the $1.7 billion copper string 2.0 - an 1100-kilometre transmission line from Cloncurry to Townsville to connect the North West Minerals Province with the National Electricity Market - will dramatically boost Queensland’s minerals exports if it comes to fruition. These exports are currently worth close to $10 billion a year according to Queensland Treasury figures.

Cheap and reliable access to energy is the key to driving further investment in our highly prospective North West Minerals province. And speaking of energy, our sector is well placed to take advantage of the growth in demand for renewable energy.

The International Energy Agency’s latest prediction is that 80 percent of growth in electricity demand will be in renewables by 2030. That means more worldwide demand for critical minerals, which Queensland has in abundance. More about that in a moment.

Our members are also carefully watching the potential for development of a new hydrogen industry in Queensland. Hydrogen is a clean, renewable fuel that can be used in transport, power supply and a range of industrial processes. It’s already important for a range of industries that provide vital inputs into manufacturing processes and our mining and agricultural sectors.

Hydrogen can be produced from a variety of sources including renewables such as solar, wind and biomass. The QRC supports the Queensland Government’s Hydrogen Industry Strategy, which includes a $15 million industry development fund to support hydrogen projects in the state, and we look forward to watching this initiative unfold.

Of course, while the outlook for our sector on purely economic terms looks extremely positive, we are not without our challenges. The number-one concern for our members is consistently our social license to operate. No surprises then that global consulting firm EY’s survey of mining and metals companies identifies license to operate as the number-one risk for mining and metals in 2021.

As we all saw with the last federal election, the support of the community is key to influencing government policy that values and supports mineral exploration and development.

It was truly heartening during the federal campaign to see such grassroots acknowledgement of the jobs and economic wellbeing the resources sector brings to local communities.

And it’s all true. Every year for the past 11 years, the QRC has promoted the economic contribution of Qld’s resources industry to the state’s economy through an annual release of data gathered directly from our members.

I think the $82.6 billion figure plus the 420,000 jobs across the sector needs to be here. This data showed that in 2019-2020 QRC members supported almost 1300 community organisations right across Queensland. It showed our sector works hard to support our local communities by providing jobs across Queensland, with 83 percent of our direct workforce living outside of the Brisbane region.

Our members recognise the many benefits of a diverse workforce and this is why the QRC puts a lot of time and effort into our Women in Resources Action Plan, first established in 2006. This includes celebrating and promoting the exceptional women working in our sector and the people and companies who champion diversity in our annual Resources Awards for Women presented at our International Women’s Day Breakfast each year. Last year it was the biggest crowd ever, with 1,000 attending the event in Brisbane and another 1300 watching via live webcast. If you get in quickly, you might just get a seat for this year’s event on 10 March, but I wouldn’t delay as it’s usually a sell-out and it’s a fantastic event.

Adding to our efforts to encourage more women and young people in general into our sector is the QRC’s Queensland Minerals and Energy Academy, also known as the QMEA.

Now with 80 schools on board across Queensland, this partnership between QRC, its members and the Queensland Government highlights the career opportunities available in resources to high school students through hands-on workshops and events involving staff from our member companies.
Our QMEA/Women in Mining and Resources Queensland (WIMARQ) and QRC/WIMARQ Mentoring programs also aim to attract and retain women in our sector. Outcomes from these programs show it’s a strategy that’s working.

Another major social licence challenge for our sector is, of course, managing carbon emissions. What many people might not realise is that our sector is already using and developing technology to reduce its emissions.

Increasing use of renewable energy options by resources operations has also helped to reduce the sector’s carbon footprint and energy costs, which we will continue to do.

One example is Anglo American and Macquarie Capital working together to develop an integrated solar-powered, green hydrogen production and mine vehicle project at the Dawson Mine in Queensland.

In Weipa, Rio Tinto invested in a 1.7 megawatt solar farm to displace its diesel power use. And, since 2006, Australia’s black coal producers have committed $550 million through Low Emissions Technology Australia (LETA) to identify, research and develop low-emission technologies for the production of energy, steel and other materials. LETA works with like-minded organisations to enable the industry to progress to a net-zero carbon emissions future.

Social license is also predicated on keeping our people safe. I know the health and safety of our employees is the one thing, above all else, that keeps many a CEO awake at night.

And technology has had a role to play here too. Our sector is a big user of and innovator in technology, and an example of this is the new Resource Centre of Excellence in Mackay, which was established to connect the brightest minds in research, technology, education and METS to shape the resources sector of the future.

Now with 80 schools on board across Queensland, this partnership between QRC, its members and the Queensland Government highlights the career opportunities available in resources to high school students through hands-on workshops and events involving staff from our member companies.

Our QMEA/Women in Mining and Resources Queensland (WIMARQ) and QRC/WIMARQ Mentoring programs also aim to attract and retain women in our sector. Outcomes from these programs show it’s a strategy that’s working.

The sector’s annual Queensland Mining Industry Health and Safety conference brings together employers, employees, unions and the government to share safety information, technology and ideas and hear from leading global experts. And QRC members have been working for many years on innovations such as remotely operated equipment, virtual reality, and augmented reality technologies to take people out of harm’s way. The issue of safety and health has thrown up some specific challenges in the past few years. The re-emergence of Coal Workers’ pneumoconiosis, (CWP) or Black lung in 2015-16, the ministerial safety resets in 2019 and COVID-19 in 2020 are just some examples. However, the industry’s response to all these challenges has been outstanding.

As an industry, we remain committed to supporting government responses to COVID-19 to keep our communities and workers safe, while ensuring regional communities and the Queensland economy can continue to function and flourish. The efforts of the QRC and our member companies have ensured the resources sector has continued to operate safely through the pandemic, delivering enormous economic benefit to Queensland when it was needed most.

Over the past 20 years, since a risk-based approach to legislation in Queensland was introduced, there have been no multi-fatality disasters, and a reduction in fatalities annually. However, even one fatality or serious injury is one too many, which is why the QRC and our members supported the government’s safety reset last year to double down on their efforts to keep our people safe. More can, and is being done. So, ladies and gentlemen, I know I have covered a lot of ground here today, but there is a lot going on in resources.

What to take away from this morning? Well, firstly that Queensland is richly endowed with mineral wealth, much of it still untapped, and we have world-leading talent right here in this state to develop it. The economic outlook for resources is incredibly positive. We’re attracting more women and young people to work in our industry to ensure we have the skilled workforce we need for our technologically
advanced sector. We're also continually raising the bar to make sure our people are safe and return home to their families each day. We are also increasingly contributing to the lives and careers of Indigenous people in our resource communities and beyond.

And, we have strong, supportive communities based around our resources operations who value the jobs and economic security we bring to their region. We don't take any of this for granted. Green 'lawfare', misinformation in the blogosphere, the relentless war against fossil fuels and shareholder activists are a real and present danger.

We need to be vigilant in keeping our positive messages out there to ensure that the regulatory environment is one that continues to encourage investment and the responsible development of the state’s resources for the benefit of all Queenslanders.

Thank you ladies and gentlemen.
HYDROGEOLOGICAL PROPERTIES AND MINING IMPACTS ON GROUNDWATER IN THE BOWEN, STYX AND GALILEE BASINS

George Klenowski¹ and John Bernal²

ABSTRACT: The results of extensive hydrogeological investigations in the Bowen, Styx and Galilee Basins are described in this paper. These are coal mining areas with saline groundwater resulting from predominantly marine deposition. Small aquifers occur in Quaternary and Tertiary deposits and within coal measures strata. Tertiary Basalt flows sometimes contain larger aquifers. Mining results in localised groundwater drawdown. Testing has included determination of in situ permeability, pump-out flow rates, salinity and pH values of groundwater. Piezometer monitoring of aquifer drawdown during mining has been completed. Computer modelling and inflow calculations have been used to determine inflow rates during longwall mining. Accurate permeability results are required to obtain design parameters for mine dewatering systems. Mining results in aquifer drawdown, surface subsidence and the formation of open tension cracks. Because significant leakages can occur into underground workings from overlying water bodies within the critical tensile strain zone, extensive modelling has been completed to obtain accurate inflow values. Results indicate that only minor aquifers occur within coal measures strata. The groundwater is generally highly saline and is classified as contaminated. It can only be used for washing coal. The main impact from mining is potential inundation of underground workings from subsided, overlying water bodies. Groundwater drawdown recharges back to original levels following mining.

INTRODUCTION

Comprehensive groundwater investigations have been completed in the Bowen, Styx and Galilee Basins, where economic coal deposits occur. Pump-out flow rates, permeability and conductivities have been determined. Aquifers within coal measures are generally small and highly saline because the depositional environment was predominantly marine /deltaic. This water is classified as contaminated and cannot be discharged offsite. It can only be used for washing coal or purified by reverse osmosis prior to discharge. Water with a salinity of greater than 3000 ppm total dissolved solids cannot be discharged off a mining lease. Local, saline aquifers occur within emplaced igneous sills. Small perched aquifers, which are present in overlying Tertiary and Quaternary deposits include sandy alluvium in water courses and gravel beds at the base of Tertiary Clay. The water quality varies from fresh to saline. Significant freshwater aquifers can occur in Tertiary Basalt flows which overlie coal measures.

DETERMINATION OF GROUNDWATER PARAMETERS

Aquifer constants

Flow rates from aquifers obtained by air lift pump-out testing of drill holes are measured using a V-notch weir (The Australian Institute of Mining and Metallurgy, 2011). Transmissivity and co-efficient of storage values are determined by pump-out testing of production bores with monitoring of drawdown and recovery in observation bores. Calculations are completed using the Jacob or This Method (Australian Mining Engineering Consultants, 2000). Division of transmissivity by aquifer thickness gives permeability.

Accurate permeability values can be obtained by pump-in testing. The open-end test is normally done in unconsolidated or weathered material (Australian Mining Engineering Consultants, 2000). The hole is fully cased and the amount of water retained by the ground through the open bottom is recorded. The permeability is then obtained from:

\[ K = \frac{Q}{5.5 \times \pi \times r \times H} \]

¹ Manager, Australian Mining Engineering Consultants. Email: amec1@bigpond.net.au Tel: +61 4 1718 7149
² Geologist, Waratah Coal Pty Ltd. Email: j.bernal@waratahcoal.com Tel: +61 4 4985 0642
where
\[ K = \text{permeability (m/sec)} \]
\[ Q = \text{constant flow rate into hole (m}^3\text{/sec)} \]
\[ r = \text{internal radius of casing (metres)} \]
\[ H = \text{differential head of water (metres)}. \]

A drilling rig is normally used to complete packer tests (Figure 1). In these tests a section of hole is isolated with a single (downstage testing) or double (upstage testing) packer arrangement (Figures 2 and 3). Packers are either inflated hydraulically or pneumatically (Figure 4). Water is then pumped down the hole within drill rods and the flow rate and pressure are recorded. Five consecutive tests, each of ten minutes duration are normally completed for each stage, with the first three tests being done at increasing pressures and the last two tests done at decreasing pressures. Permeability values can then be interpreted in terms of laminar flow, turbulent flow, dilation, wash-out and void filling (Houlsby, 1976), and appropriate values selected for inflow calculations. Prior to determining permeability values, the differential head values need to be corrected for friction losses in the system, which include flow meter, packer and rods (Figure 2).

Figure 1: Drilling rig used for packer permeability testing

Figure 2: System for determining packer losses

Figure 3: Testing packers

Figure 4: Monitoring equipment and nitrogen cylinder used for packer permeability testing.

The formulae for this test (Australian Mining Engineering Consultants, 2000) are:

\[ K = \frac{Q}{2\pi x L x H} \log e \ X \frac{L}{r} \text{ where } L \geq 10r \]

\[ K = \frac{Q}{2\pi x L x H} \sin h^{-1} \ X \frac{L}{2r} \text{ where } 10r > L \geq r \]
where \( K \) = permeability (m/sec)
\( Q \) = constant flow rate into hole (m\(^3\)/sec)
\( L \) = length of test section (metres)
\( H \) = corrected differential head of water (metres)
\( r \) = radius of hole tested (metres)
\( \log_e \) = natural logarithm
\( \sinh^{-1} \) = arc hyperbolic sine.

The relationship between permeability and aquifer drainage is described in Figure 5.

**Figure 5: Permeability and drainage systems**

Inflow calculations

A number of assumptions need to be made when using steady state type inflow equations for dynamic situations involving tensile fracturing during extraction mining. Calculated inflows should only be used as a guide. More accurate calculations can be obtained using models with accurate permeability values and sophisticated computer programs.

Darcy's equation can be used to calculate inflow from subsurface aquifers, as follows:

\[ Q = K_i A \times 1000 \]

where
\( Q \) = flow rate (litres/sec)
\( K \) = permeability (m/sec)
\( i \) = hydraulic gradient
\( A \) = cross-sectional area of aquifer (m\(^2\)).

Inflows into a longwall mine can be computed using the formula for flow into a slot from a two line source (Australian Mining Engineering Consultants, 2000) as follows:

\[ Q = K (2 \times Hr \times Hw - Hr^2)/2L \times l \times 10^3 \]

where
\( Q \) = rate of inflow per panel length (litres/sec)
\( K \) = weighted permeability of overburden (m/sec)
\( Hr \) = height to rockhead (i.e. base of Tertiary or \( 2/3 \) Hc, whichever is less (m))
\( Hw \) = height to water table (m)
\( L \) = drawdown function
\( = 5 \times Hw \)
\( l \) = panel length
\( Hc \) = critical caving height (height at which maingate and tailgate caving angles intersect the surface).

For first panel extraction the above value of \( l \) is doubled to allow for inflow from both sides. For subsequent panels inflow occurs from one side, the other side being goaf.

**INFLOW MONITORING**

Monitoring of inflows into open cut and underground workings has been completed. Sources of water include surface storages, aquifers and inundated underground workings. Surface ponded water which
can flow into underground mines originates from open cut final voids, internally draining spoil piles, subsidence troughs, natural depressions, flowing creeks, dam reservoirs and ungrouted boreholes in topographically low areas. A number of monitoring techniques have been used including V-notch weirs, flowmeters on pump pipelines, piezometers and measurements of water level falls in surface ponds of known volumes. A method has been developed to calculate goaf storage volumes (Australian Mining Engineering Consultants, 2000). The main aim of inflow monitoring has been to measure water inflow rates from sources located at different heights above underground workings and relate the inflow rates to measured and calculated tensile strain values. Although it is known that tensile strain and inflow rates decrease with increasing overburden thickness, quantification of these values is required to improve inflow predictions.

MINING INDUCED GROUNDWATER CHANGES

Mining causes fluctuations of the groundwater regime due to dewatering of aquifers. These changes are reversible and post-mining recharge results in the return to original water levels with salinities similar to previous values.

Open cut mining

Minor seepage of saline groundwater occurs during open cut mining, mainly from coal seams. Local aquifers occur at the base of Tertiary Clay and within weathered coal measures strata. The water table generally occurs between 10.0 m and 30.0 m below the surface with some seasonal fluctuation. Ponded, saline water in final voids can have pH levels as low as 2.5 (Australian Mining Engineering Consultants, 2000), due to the presence of oxidisable pyrite which reacts to form sulphuric acid.

Groundwater drawdown curves adjacent to open cut voids are generally steep due to the very low permeability of the strata. Drawdown curves can be generated by computer modelling using programs such as the SEEP/W portion of the GeoStudio package. Detailed sections with accurate permeability values are required to produce valid results (Figure 6). Results from Australian Mining Engineering Consultants, 2020, show that during mining groundwater drawdown occurs for about 40 m outside of the excavated profile after exposure for one year and about 550 m from the excavated profile after 13 years. During post-mining recharge the phreatic surface would return to the original level with similar salinity.

Figure 6: Plan and section computer model, Central Queensland Coal Project
Underground mining

Longwall mining is the most common underground extraction method. Localised bord and pillar mining is also practiced. During longwall mining the overburden subsides and longitudinal and arcuate tensile fractures form (Australian Mining Engineering Consultants, 2000). Goafing results in a primary caving zone where the failed material is rubbly and an overlying, secondary caving zone when the overburden subsides en-masse along longitudinal fractures, parallel to the gateroads (Figures 7 and 8, Australian Mining Engineering Consultants, 2000). In the critical tensile strain zone which occurs to a height of about 180 m above the mining horizon, tensile fractures are interconnected, allowing downward flow of groundwater or ponded water into the goaf (Figure 8). Above this zone water does not flow into the goaf.

![Figure 7: Diagrammatic representation of longwall subsidence](image1)

![Figure 8: Section through subsided igneous sill aquifer](image2)

Computer modelling has been used to calculate inflow rates for increasing depth of cover. Figure 9 is a typical geotechnical section. A program based on MAP3D, which has been developed incorporates all the major components of the rock strata that are required to simulate ground response to underground multi-seam mining (Australian Mining Engineering Consultants, 2000). The program used to investigate transient and steady state water flow mechanisms is the 3-D finite element program London University Stress Analysis System (LUSAS) developed by Finite Element Analysis (Australian Mining Engineering Consultants, 2000).

Inflow rates have been calculated using the MAP3D and LUSAS seepage flow models. Results are tabulated in Table 1 and are compared with maximum recorded inflows.

<table>
<thead>
<tr>
<th>Depth of cover (m)</th>
<th>Seam thickness (m)</th>
<th>Inflow rate (l/sec)</th>
<th>Maximum recorded inflow (l/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>70</td>
<td>2.8</td>
<td>165</td>
<td>140</td>
</tr>
<tr>
<td>100</td>
<td>2.7</td>
<td>76</td>
<td>83</td>
</tr>
<tr>
<td>120</td>
<td>2.8</td>
<td>39</td>
<td>45</td>
</tr>
<tr>
<td>140</td>
<td>2.8</td>
<td>30</td>
<td>27</td>
</tr>
<tr>
<td>175</td>
<td>2.7</td>
<td>7</td>
<td>2</td>
</tr>
</tbody>
</table>

Computer modelling indicates that there is a good correlation between predicted and maximum recorded inflow rates. The modelling has been refined using ongoing subsidence, strain and inflow monitoring data. The inflow rates in Table 2 require assessment because they are affected by the type of ponded floor material, degree of clogging of open tension cracks and hydrostatic head.

In the Styx Basin computer modelling of groundwater drawdown during mining has been completed using the SEEP/W portion of the Geostudio package. Two dimensional finite element seepage analyses were performed. Computed drawdown was minimal.
Figure 9: Geotechnical section – German Creek Mines

### Table 2: Recent inflow monitoring data

<table>
<thead>
<tr>
<th>Mine</th>
<th>Date</th>
<th>Location</th>
<th>Depth of cover (m)</th>
<th>Inflow rate (litres/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oaky No.1, panel 19</td>
<td>2000</td>
<td>Highwall mining entries, Stuart Pit</td>
<td>110</td>
<td>45</td>
</tr>
<tr>
<td>Grasstree</td>
<td>2011</td>
<td>Grasstree North Pit above Southern Colliery</td>
<td>105</td>
<td>65</td>
</tr>
<tr>
<td>Oaky North, 400’s panels</td>
<td>2011 – 2015</td>
<td>Oaky Creek</td>
<td>160 – 180</td>
<td>Negligible</td>
</tr>
<tr>
<td>Oaky No.1, panel 32</td>
<td>2013</td>
<td>Aquila Low, A7 Pit</td>
<td>105</td>
<td>30</td>
</tr>
<tr>
<td>Oaky No.1, panel 36</td>
<td>2014</td>
<td>Highwall mining entries, Pit A6 South</td>
<td>105</td>
<td>20</td>
</tr>
<tr>
<td>Oaky North, panel 308</td>
<td>2014</td>
<td>Grasstree South Pit and spoil piles</td>
<td>130</td>
<td>23</td>
</tr>
<tr>
<td>Oaky No.1, panel 33</td>
<td>2015</td>
<td>Aquila Low Pit</td>
<td>105</td>
<td>20-30</td>
</tr>
<tr>
<td>Oaky No.1</td>
<td>2016</td>
<td>Tailings in Talagai (A4) Pit</td>
<td>120</td>
<td>60</td>
</tr>
</tbody>
</table>

Subsidence monitoring indicates that longitudinal tension cracks occur parallel to gateroads along zones of maximum tensile stress. Full subsidence generally occurs above the panel centre line and is related to height of overburden, panel width and extracted seam thickness. Due to the panel widths in Queensland mines, strata bridging does not occur, except for minor goaf hang up. Yield pillars crush under abutment load.
TESTING RESULTS FOR THREE BASINS

Extensive testing has been carried out in the Bowen and Styx Basins. Preliminary hydrogeological work has commenced in the Galilee Basin. Groundwater chemistry, pump-out testing flow rates and permeability have been determined.

Bowen Basin

Hydrogeological investigations were completed at the German Creek and Oaky Creek Mines prior to longwall mining and are ongoing.

Groundwater chemistry

Salinities of local, near surface aquifers range from 2000 to 6000 ppm. Coal measures and spoil piles groundwater salinities range from 12 000 to 25 000 ppm (Australian Mining Engineering Consultants, 2000 and Klenowski and Phillips, 1988). Ponded, saline water in final voids can have pH values as low as 2.5 (Australian Mining Engineering Consultants, 2000 and Capricorn Management, 1993). At the German Creek Mines about 5000 megalitres of acid water with a pH of 2.5 was dosed with lime to increase the pH to 6.9, prior to being used for washing coal (Capricorn Management, 1993). Post-mining infilling of final voids with rejects, tailings or spoil is recommended, prior to topsoiling and seeding. At the Moranbah North Underground Mine groundwater salinity ranges from 8000 to 9000 ppm (Shell Company of Australia, 1995).

Pump-out testing flow rates

The maximum recorded pump-out flow rate in coal measures at the German Creek and Oaky Creek Mines was 10 litres/sec in the Fairhill Formation. At Southern Colliery a flow rate of 37.5 litres/sec was measured during pump-out testing of saline water in the Aquila Sill aquifer. Pre-drainage was completed prior to longwall undermining (Klenowski & Phillips, 1988). Throughout the Bowen Basin flow rates from coal measures are low and the groundwater is generally highly saline. At the German Creek and Oaky Creek Mines pit water can only be used to wash coal and the latter mine has installed a reverse osmosis plant to treat contaminated water.

Permeability

A typical stratigraphic section with permeability for the German Creek and Oaky Creek Mines is shown in Figure 10. These results are typical for the Bowen Basin and recent data from the Styx and Galilee Basins are compatible. Measured permeability values for surface alluvium, Tertiary Clay and weathered bedrock are $1.3 \times 10^{-5}$, $2.3 \times 10^{-6}$ and $8.5 \times 10^{-7}$ m/sec respectively (Figure 10, Australian Mining Engineering Consultants, 2000). Coal measures permeability ranges from $1.7 \times 10^{-6}$ to $4.1 \times 10^{-8}$ m/sec.

At the Moranbah North Underground Mine and the Goonyella / Riverside Project measured permeability is $5.0 \times 10^{-5}$ m/sec for Tertiary and Quaternary Clay and $1.5 \times 10^{-6}$ to $3.0 \times 10^{-8}$ m/sec for coal measures strata (Shell Company of Australia, 1995). The low permeability results indicate limited groundwater drawdown during mining with rapid, post-mining water table recovery.

Styx Basin

The Styx Basin is being investigated for the production of PCI (pulverised coal injection method) coal. Extensive groundwater investigations have been completed at the Central Queensland Coal Project (Australian Mining Engineering Consultants, 2020).

Groundwater chemistry

Coal measures groundwater is highly saline with salinities ranging from 8 400 ppm to 15 700 ppm. Minor seasonal fluctuation of groundwater occurs (Australian Mining Engineering Consultants, 2020). The pH is neutral (6.8 to 7.6, Australian Mining Engineering Consultants, 2014). Isolated water pools which occur along water courses have salinities ranging from 135 ppm to 950 ppm (Australian Mining Engineering Consultants, 2020). The pools dry up due to evapotranspiration and are recharged by rainfall runoff.
Pump-out testing flow rates

Pump-out testing has been completed in five drill holes. Air-lift flow rates ranged from 0.002 litre/sec to 0.15 litre/sec, indicating very tight, coal measures strata. All discharged water was highly saline.

Permeability

Recent permeability testing has been completed for the full mining horizon at the Central Queensland Coal Project (Australian Mining Engineering Consultants, 2020). Surface clay and weathered
bedrock have permeability values ranging from $3.7 \times 10^{-8}$ to $1.0 \times 10^{-9}$ m/sec. Permeability of fresh surface bedrock is $3.73 \times 10^{-7}$ to $1.33 \times 10^{-7}$ m/sec. Fresh coal measures strata have permeability of $2.6 \times 10^{-8}$ to $3.96 \times 10^{-8}$ m/sec. All permeability is very low indicating limited groundwater drawdown during mining (Figure 6). Post-mining recovery would be rapid.

**Galilee Basin**

Economic thermal coal deposits occur in the Galilee Basin. Construction has commenced at the Carmichael Mine and investigations are proceeding at the China Stone, Kevin’s Corner and Galilee Coal Projects.

![Figure 11: Galilee water bores drilled in 2019](image)

**Groundwater chemistry**

Original groundwater investigation was completed at the Galilee Coal Project in 1994 (Bridge Oil Limited, 1994). Work continued in 2007 (Australian Mining Engineering Consultants, 2007) and is ongoing. Groundwater salinity is variable ranging from 170 ppm to 19 300 ppm. Where sandstone beds are more permeable, groundwater tends to be less saline. The Colinlea Sandstone Member is a typical example. Seven groundwater investigation holes were completed in 2019 (Figure 11). Salinities ranged from 290 ppm to 19 400 ppm. Measured pH values were 7.45 to 8.7. These results are preliminary and ongoing testing is proceeding.

**Pump-out testing flow rates**
Previous pump-out testing at the Galilee Project gave values ranging from 1.5 litres/sec to 12.6 litres/sec with an average flow rate of 6.5 litres/sec (Bridge Oil Limited, 1994). Maximum flow rate was in the DL2 to E interval. No pump-out tests were completed in the seven holes completed in 2019 because flows were too small to measure.

Permeability

Permeability results for the Galilee coal measures range from $6.0 \times 10^{-6}$ to $8.0 \times 10^{-6}$ m/sec (Bridge Oil Limited, 1994). Recent testing of coal seams gave an average permeability of $1.3 \times 10^{-7}$ m/sec (Ward pers. comm (2020)).

SUMMARY OF TEST RESULTS

A summary of test results is included in Table 3. Pump-out flow rates are determined by air-lift testing. Results are for coal measures strata. Low pH values occur where oxidisable pyrite is present.

Table 3: Summary of test results for the Bowen, Styx and Galilee Basins

<table>
<thead>
<tr>
<th>Basin</th>
<th>Maximum pump-out rate (litre/sec)</th>
<th>Salinity range (ppm)</th>
<th>pH range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bowen</td>
<td>10.0</td>
<td>12 000 to 25 000</td>
<td>2.5 to 8.5</td>
</tr>
<tr>
<td>Styx</td>
<td>0.15</td>
<td>8400 to 15 700</td>
<td>6.8 to 7.6</td>
</tr>
<tr>
<td>Galilee</td>
<td>12.6</td>
<td>290 to 19 400</td>
<td>7.45 to 8.7</td>
</tr>
</tbody>
</table>

CONCLUSIONS

Hydrogeological investigations in the Bowen, Styx and Galilee Basins indicate that the groundwater in coal measures is generally highly saline and is classified as contaminated. Such water cannot be discharged off mining leases but can be used for washing coal or purified by reverse osmosis, prior to release. Ponded pit water can have pH levels as low as 2.5 due to the presence of oxidisable pyrite. Acidic water can be dosed with lime for neutralisation, prior to being used for washing coal. Final voids should be backfilled with rejects, tailings or spoil.

Coal measures strata have very low permeability with local, minor aquifers. Larger saline aquifers can occur in emplaced igneous sills. Small, freshwater aquifers sometime occur in overlying Tertiary and Quaternary deposits. Significant freshwater aquifers may be present in Tertiary Basalt flows.

Groundwater drawdown around open cut voids is localised due to the low permeability of the strata. Post-mining recovery of saline water is rapid.

The critical tensile strain zone which forms during longwall mining occurs for a distance of about 180 m above the mining horizon. Within this zone, subsidence tension cracks are interconnected to the goaf allowing downward flow of water. Flow rates into the goaf from an overlying water body decreases as the overburden thickness increases. Monitoring indicates that following mining the saline, phreatic surface returns to its original level.

REFERENCES


Australian Mining Engineering Consultants, 2007. Preliminary investigations of hydrogeology and Belyando River water resources for the Alpha Coal Project.
Capricorn Management, 1993. A new integrated tailings disposal and water management system for the German Creek Mine.
STRESS CHANGE NEAR A MAJOR GEOLOGICAL STRUCTURE DURING LONGWALL MINING

Baotang Shen¹, Xun Luo² and Joey Duan³

ABSTRACT: Stress state and geotechnical conditions often change significantly near major geological structures (e.g. faults, shear zones, dykes) in underground coal mines, which is the cause of most major mine instability and/or safety hazards including coal burst, roof falls, water inrush and gas outburst. In order to understand and quantify the stress state near major geological structures, an integrated study had been conducted in the vicinity of a dyke in an Australian underground coal mine. The field monitoring program included installing microseismic geophones, stressmeters and extensometers in the roadway roofs and coal pillars, aiming to obtain seismic and stress change data during longwall mining. The monitoring results indicate that the stress regime was clearly different on the inbye and outbye sides of the dyke. The inbye side had a much higher stress than the outbye side before and during the longwall mining. This study provided quantified field evidence that the stress concentration occurs near major geological structures. This stress concentration could lead to high strain energy concentration in the rib of a roadway, and hence increase the risk of coal burst.

INTRODUCTION

Coal burst (also called coal bump) is a violent collapse of coal walls and/or roofs occurring in underground coal mines. It may happen in development roadways, at the longwall face or at the chain pillars. Because it occurs suddenly with no or very little early warning, coal burst is particularly dangerous to mine personnel and mining equipment. Coal burst is a long-standing issue for underground coal mines in many countries all over the world. The first reported coal burst was recorded in the UK in 1783. Since then many countries including Germany, USA, Canada, Poland, Russia, India, China and Australia have experienced coal burst events, some sadly with the loss of lives. In Poland, 60% of coal mines have experienced coal burst with about 20 coal burst events per year (Kleczek and Zorychta, 1993). In China, the largest coal producer in the world, over 140 underground coal mines currently are coal burst prone. During 2011-2013, over 60 fatalities occurred in 17 coal burst related accidents in China (Pan, et al., 2013)

The occurrence of coal burst is relatively rare in Australia as the majority of underground mines are still relatively shallow. However, as mines in Australia are going deeper and moving into more challenging geotechnical conditions, the risk of coal burst is certainly on the rise. Some mines have experienced several apparent coal burst events where the mining depth has reached between 500 m - 600 m. Mines with strong massive roof conditions have also experienced pressure bumps and major weighting on the longwall support system.

Many coal burst events in both Australian and overseas mines have occurred near faults, dykes and other geological structures. Dynamic roadway instability events have occurred in several underground mines in New South Wales near dykes, thrust faults, and normal faults. Fatal coal burst incidents were reported overseas due to geological structures. For instance, the Qianqiu coal burst incident with 11 fatalities in China in 2011 was a direct result of the stress elevation by a thrust fault during mining. Faults, dykes and other geological structures are considered to be some of the key contributing factors to coal bursts.

The study of stress anomalies and geotechnical conditions near major geological structures is one of the key research areas for hazard identification and management in underground mining, not only for coal burst risk but also for broader applications in risk assessment and management (such as gas outburst, roadway stability).

¹ Senior Principal Researcher, CSIRO Mineral Resources, Brisbane, QLD 4069 Australia
Email: Baotang.shen@csiro.au; Tel +61 7 33274560

² Senior Principal Researcher, CSIRO Mineral Resources, Brisbane, QLD 4069 Australia
Email: Xun.luo@csiro.au; Tel +61 7 33274551

³ Researcher, CSIRO Mineral Resources, Brisbane, QLD 4069 Australia
Email: Yi.duan@csiro.au; Tel +61 7 33274433
This paper presents a summary of the recent systematic study on monitoring the stress state near a major geological structure in an Australian underground coal mine. The work is part of a research project sponsored by the Australian Coal Association Research Program (ACARP) and the CSIRO. More details of the study can be found in Shen, et al., (2019) and Shen, et al., (2020).

FIELD MONITORING DESIGN AND INSTALLATION

Monitoring site

A comprehensive field monitoring program was designed to collect stress and seismic data in the vicinity of an identified major geological structure during actual mining. It is ideal that the monitoring site may be coal burst prone so that the monitoring data represents possible conditions found prior to coal burst. An Australian underground coal mine (referred to as “Mine A” hereafter) had been selected for the field monitoring.

Mine A uses the longwall extraction method to mine a 9 m thick coal seam where only the bottom 3.5 m – 4.0 m of coal is extracted. The current longwall panels are typically 400 m wide and 3 km long. The overburden depth ranges from 160 m to 380 m. The roadways are typically 5.5 m wide and 3.5 m high. Mine A was identified as a suitable site for a case study due to its unique geological and geotechnical conditions. The following key geological and geotechnical conditions exist in Mine A:

- A normal fault with a displacement between 1 m and 5 m and a dyke with a thickness ranging between 0.5 m and 3 m exist.
- Pressure bumps of various magnitudes have been recorded in the underground workings.
- A massive 15 m-20 m thick conglomerate exists in the immediate roof which influences the loading (stress) behaviour and periodic weighting is present in the main roof.

Monitoring plan

The selected monitoring site is located in Longwall (LW) 107 at Maingate B heading between cut-through (CT) No.9 and No.10 in an area of the panel intersected by the dyke. A vertical borehole was drilled from the surface to lower the instrumentation cables. Two logging stations were setup at the surface, one for microseismic data and the other for stress/displacement data. The stress/displacement data were remotely collected from the CSIRO’s office in Brisbane in real-time through an optical fibre network, while the microseismic data were recorded in real time and collected manually on site on a weekly basis. Underground, two monitoring stations were established on either side of the dyke (inbye and outbye) as shown in Figure 1. Figure 2 shows the sensor arrangement in a vertical cross-section at the inbye location, and those at the outbye location are very similar.

![Figure 1: Plan view of inbye and outbye monitoring locations. Sensors are numbered in red](image)
At the inbye monitoring station, two extensometers were installed, one in the roof, and one in the rib. Similarly, two geophones were also installed, one in the roof and one in the rib. To monitor the stress change, one uniaxial and one biaxial stressmeter were installed in the rib. The outbye monitoring station mirrored the inbye monitoring configuration. A total of 4 tri-axial geophones, 4 stressmeters and 4 extensometers were installed at the two monitoring stations. The discussion in this paper will be focused only on the stress and microseismic monitoring results due to page limitations associated with this paper. The displacement monitoring results are consistent with results reported in Shen, et al., (2019).

**MONITORING RESULTS**

One of the key objectives of the field monitoring program was to obtain quantitative evidence of the stress change around a major geological structure since the stress concentration is considered to be a key contributor to coal burst. In Mine A the targeted major geological structure was a sub-vertical dyke, dipping to the south at an approximate dip angle of 75° - 80°. It had a width of approximately 1.5 m at the monitoring location. Two identical sets of monitoring sensors (stressmeters, extensometers and geophones) were installed both sides of the dyke at a distance of 10-20 m. The aim was to directly compare the stresses and geotechnical conditions at the two sides during longwall mining, and hence quantify the influence of the dyke on the risk of coal burst.

**Stress change**

*Uniaxial stressmeters*

Two uniaxial stressmeters were installed in the pillar between roadway headings B and C, in two inclined boreholes drilled from the B heading. Stressmeter #11 was located inbye of the dyke at a distance of 10 m, whereas the Stressmeter #12 was outbye at a similar distance. Both stressmeters were installed at the borehole depth of about 6-8 m from the B heading. The uniaxial stressmeters only measure one-dimensional stress change. Because the stressmeters were inclined at an angle of 30° upwards, they actually measured the equivalent stress change in the direction of 60° from the horizon.

Figure 3 shows the measured stress change in the pillar in the period from the 25th November 2017 to the 23rd April 2018. This period covers the critical period when the longwall face travelled from 422m in front of the dyke through to when it was 226m past the dyke. The black curve in Figure 3 shows the longwall distance to the dyke, and the blue and red curves are the measured stress at the inbye and outbye locations.

Stress changes in the pillar could be observed starting from the 19th January 2018 when the longwall face was about 150m from the dyke. As the longwall mining approached the dyke, the stress change at both monitoring locations increased exponentially until the 14th February 2018 when the face passed by the dyke at a distance of 30m. The mining was then stopped for 12 days, and consequently no significant stress changes were recorded during this period. This demonstrates a close correlation between the mining progress and the pillar stress change.
When longwall mining was resumed on the 26th February 2018, the measured stress change continued to increase. It is interesting to note that, during the period from the 5th to the 15th March 2018 when the longwall face was some 80-90 m past the dyke, the outbye stress change dropped by 0.2 MPa while the inbye stress change continued to increase. This could have been caused by the shearing along the dyke during the caving process, which could have released the vertical stress outbye and increased the load inbye. The stress change rate slowed down gradually as the longwall face passed beyond the dyke by some 130 m. An important observation is that the measured pillar stress increase at the inbye location is significantly higher than that at the outbye location. When the longwall face was 30 m past the dyke, the stress increases at the two locations were 1.2 MPa and 0.7 MPa, respectively. When the longwall was 200 m passed the dyke, these values were 2.4 MPa and 1.6 MPa. The overall difference in the stress magnitude at the inbye and outbye sides of the dyke was 30-40%. This highlights the significant influence of the dyke on stress redistribution in its vicinity. During longwall mining, the dyke could have acted as a barrier for stress transfer due to the shear movement along the dyke. The dip direction of the dyke had a major influence on the stress concentration pattern around the dyke. The “hanging wall” of the dyke would take more stress than the “footwall”.

**Biaxial stressmeters**

Two biaxial stressmeters were installed, one at each of the two sides of the dyke. They were installed in two sub-horizontal boreholes at a depth of 4 m into rib, hence measuring the two-dimensional stresses in the pillar in a vertical cross section parallel to the roadway axis. Figure 4 shows the measured major principal stress change (P) and minor principal stress change (Q) in the rib from the outbye location. In this case, P was approximately in the vertical direction and Q in the horizontal direction. Unfortunately, the biaxial stressmeter at the inbye location appears to have malfunctioned and did not return any valid reading.

The measured stress changes from the outbye biaxial stressmeter in Figure 4 showed a similar trend to those in Figure 3 from the uniaxial stressmeters, but the magnitude of the stress changes was much higher. The measured vertical stress change was 8.5 MPa from the biaxial stressmeter, compared with the 1.6 MPa from the uniaxial stressmeter when the longwall face was beyond the dyke by some 226 m, both at the outbye side of the dyke. This huge difference could be partially caused by two
factors: (1) The biaxial stressmeter was located in the roadway rib where a high stress concentration existed due to roadway excavation. The uniaxial stressmeter was installed in the roof coal above the rib, where the stress concentration should be much less. (2) The biaxial stressmeter measured the vertical stress whereas the uniaxial stressmeter only measured the stress at a 30° angle from the vertical direction.

Based on this comparison, the magnitude of vertical stress change in the rib could be about five times the magnitude of measured stress changes in the roof coal using the uniaxial stressmeters. If the same proportion holds for the inbye location as for the outbye location, the total stress increase at the inbye location could be as high as 12 MPa. This would be on top of the existing rib stress at an overburden depth of 280 m at the monitoring location, which is greater than the in situ vertical stress of 7 MPa. The resultant total stress of >19 MPa in the rib is close to or exceeds the uniaxial compressive strength of the coal, and hence may cause localised failures.

![Biaxial Stressmeter](image)

**Figure 4:** Measured stress changes in the pillar at the outbye of the dyke using a biaxial stressmeter. P – Major principal stress change (vertical), Q – Minor principal stress change (horizontal)

**Seismicity**

During the monitoring period, two major groups of seismic events were identified. The first group was associated with the longwall caving and they were located near the longwall face. This group of events triggered both geophone arrays underground and additional geophones located on the surface. Event location results have shown that the majority of these events occurred near the longwall face and a clear decrease in the time difference between the Longitudinal Wave (P) and Shear Wave (S) arrivals could be observed as the longwall face approached the monitoring network. The dominant frequency domain for this type of events was from 25 Hz to 75 Hz. The magnitude of this type of event was estimated to be from -3.5 to -3.0 Richter Scale.

The second group was close to the dyke. This group of events triggered four underground geophones installed close to the dyke. Event location results have shown that most of these events occurred around the dyke area. These events also showed clear arrivals of P and S waves, which indicates that they were associated with the shear fractures in the rock mass around the dyke. The dominant frequency domain was from 150 Hz to 300 Hz. The magnitudes of these types of events were estimated to be from -4.0 to -3.5 Richter Scale.
The seismic event location was applied to events with clear P wave and S wave arrival-times and recorded by four underground geophones. In total, there were 528 events selected for location. The locations of the selected events are shown in Figure 5 in plan view. The majority of the events occurred in the immediate roof and coal seam.

Because the underground geophones were closely deployed around the dyke, to avoid significant bias in the locations, the location for the seismic events related to the longwall was only applied to the data recorded from the 15th January to the 31st March 2018, when the longwall face was within 200 m of the dyke.

It can be seen from Figure 5, that most of the significant seismic events located (larger red dots) were related to the longwall and the moderate seismic events (smaller red dots) were mainly clustered around the dyke. Among the seismic events near the dyke, there were rock fracturing events both inbye and outbye.

Interestingly, the seismic events around the dyke were observed even when the longwall face was 400 m ahead of the dyke. While the longwall face was approaching the dyke, the fracturing events in the rock mass continued to propagate. Stronger seismic events were detected when the longwall was mining through the dyke. Also, the rock fracturing events near the dyke were constantly captured until the longwall production was completed in July 2018.

![Figure 5](image.png)

**Figure 5:** Plan view of the mine plan and locations of the 528 selected events.

Figure 6 shows the accumulative amplitude of seismic events received by each underground geophone for the duration from the 27th November 2017 to the 8th April 2018. Geophones 7 and 8 were located 5 m into the roadway rib at a location of 10 m inbye and outbye the dyke, respectively. They were at approximately the same locations as the biaxial stressmeters, making the comparison with the stressmeters more relevant. Geophones 5 and 6 were located 9 m into the roadway roof at a location of 20 m inbye and outbye the dyke respectively.

The accumulative amplitude curves shown in Figure 6 represent the total energy from all the seismic events received by each geophone without considering their locations. Stronger events from a far distance away may have received similar amplitude to a weaker event at a close point. In reality, these curves are more likely to capture the weak seismic events occurred in close vicinity to each geophone but in a great number.

The results in Figure 6 clearly show that the two rib geophones recorded many more seismic events than the two roof geophones. The accumulative seismic amplitude from the rib geophones was above 4 times that of the roof geophones. It was also noted that the geophones at the inbye locations received more seismicity than the outbye locations, although with respect to the locations from the dyke they were not very different.
The seismic monitoring results are consistent with the stress monitoring results. All these results showed that, at the inbye location of the dyke, the measured stress change and seismicity were significantly higher than that at the outbye location. At the same roadway location, the measured seismicity in the rib was greater than that in the roof.

From the monitored data, it can be concluded that the inbye and outbye sides of the dyke had different stress regimes. The inbye side experienced much higher stresses than the outbye side. At the same location, the rib experienced higher stresses than the roof. This is clear evidence that the dyke had changed the stresses and hence the mining conditions. The risk of coal burst could have been increased at the inbye side of the dyke.

**Correlation between stress change and seismicity**

The vibrating wire type stressmeters used in the field monitoring have a resolution as high as 0.002 MPa. Previous experience (Shen, et al., 2013) indicated that, the rate of stress change can reflect the longwall caving events with a success rate of more than 80%. Here the rate of stress change and the accumulative seismic amplitude are used to investigate the precursor of pressure bumps or coal burst.

The rate of stress change is defined as:

$$ S' = \frac{\Delta S}{\Delta t} = \frac{S - \text{Average (S)}}{\Delta t} $$  \hspace{1cm} (1)

where:

- $S'$ Rate of the stress change (MPa/min),
- $S$ Stress magnitude at any given time from the stressmeter readings (MPa),
- Average (S) Average stress magnitude over the previous 10 readings (MPa),
- $\Delta t$ Time interval between two data readings (= one minute).

Using the rate of stress change, any small sudden stress variation can be picked up and plotted clearly. Figure 7 shows the comparison between the rate of measured stress change and the maximum seismic amplitude. At the inbye location, the stress change in the rib measured by the uniaxial stressmeter and the maximum seismic amplitude per minute recorded by the rib geophone are plotted (top figure). At the outbye location, the stress changes from both the uniaxial and biaxial stressmeters (vertical stress only) are plotted against the maximum seismic amplitude by the correspondent geophone (bottom two figures).
In general, both the rate of stress change and the seismic amplitude increased rapidly when the longwall face approached to the monitoring location and shortly passed it. There were many spikes in the stress rate plots and the seismic amplitude plots, many of them were closely correlated. For instance, on the 31 December 2017, all the curves showed a series of sudden stress changes and seismic events, albeit their magnitude were small. The most obvious one was on the 9 April 2018 when nearly all the curves showed maximum spikes. The longwall face reached the dyke on this day.

Additional analysis was also conducted for much short time windows to examine the correlation between stress and microseismic results. Overall, there was a very close correlation between the stress change rates and the seismic amplitude. For every stress spike, there was a corresponding seismicity spike. It is also interesting to note that the stress spikes from all three stressmeters occurred at the same time although they were located at different locations. This indicated that they all responded to the same stress event very accurately.

The cause of these stress and seismicity spikes are of great interest, as they could present geotechnical events or pressure bump events. The relationship between the stress and seismic events was examined with respect to the longwall shearer position when the longwall face was close to the dyke location. It was found that an excellent correlation exists between the shearer position and the measured stress rate and seismicity amplitude. When the shearer was moving from the tailgate to the maingate, both the stress rate and seismicity amplitude were increasing and they reached their maximum values when the shearer reached the maingate. When the mining stopped and the shearer stayed still, there were only very limited stress changes and seismicity events.

![Figure 7: Comparison between the rate of stress change and the maximum seismic amplitude.](image)

It is believed that the progressive caving events after chock advance following the shearer movement was the main cause for the recorded stress change and seismicity events at the monitoring location. These caving events had changed the overall stress distribution in the vicinity of the longwall face and caused abutment stress increase. These stress changes could be very small if the longwall face was some distance away. However, with the very sensitive stress and seismic sensors, they could still be received and identified. These results demonstrate that the vibrating wire stressmeter has the potential to be used together with geophones to monitor rock fracturing or failure events at a distance.

**CONCLUSIONS**

In this study, (1) a comprehensive field monitoring program in the vicinity of a major geological structure was undertaken in a selected mine site; and (2) detailed analysis of monitoring data to identify the stress anomalies near the geological structure was analyzed and evaluated to determine the possibility of using the monitoring tools for coal burst forecasting. Mine A was selected as the
monitoring site where pressure bumps had been experienced in roadways due to the existence of a strong conglomerate unit in the overburden strata. Overall, the monitoring program has been successful, and it recorded valuable stress and seismic data during mining. The key conclusions from the field monitoring program are:

**Seismicity:** A significant number of microseismic events at the monitoring site were induced by longwall mining of LW107. Seismic activities near the longwall face and the dyke were both captured during the monitoring period. Most of the significant seismic events were located near the longwall face and most of the moderate seismic events were clustered around the dyke. Rock fracturing on both inbye and outbye sides of the dyke was identified. The seismic energy released on the inbye side was significantly greater than that on the outbye side. The seismic signals of these events showed clear arrivals of S-waves, which is associated with shear failure, indicating the existence of shear fractures in the rock mass. Most of the seismic events were in the immediate roof and coal seam.

**Stress regime:** The stress monitoring results indicate that the stress regime was clearly different on the inbye and outbye sides of the dyke. The inbye side had a much higher stress than the outbye side before and during the longwall mining. It is apparent that the dyke had caused stress redistribution in its vicinity, which led to stress concentration in the “hanging wall” and stress release in the “footwall”.

Monitoring system sensitivity: Both the stressmeters and microseismic systems were highly sensitive and had picked up subtle stress or fracturing activities caused by mining and caving. When the longwall face was at a distance of 60 m from the dyke, the stressmeters and the microseismic system were able to detect the movement of the shearer (or the roof caving related to it). In most cases, the monitored stress changes and the seismicity were closely correlated, and they both responded well to the same geotechnical events (such as caving or rock fracturing). This makes it possible to use one or both of the two techniques to monitor the change of rock mass conditions for forecasting rock burst.

In summary, this study provided quantified field evidence that stress concentration occurs near major geological structures. This stress concentration could lead to high strain energy concentration in the rib of a roadway, and hence increase the risk of coal burst.

**ACKNOWLEDGEMENTS**

The authors would like to thank the mining companies involved in this study for their in-kind support in providing data and assistance in field monitoring, without which the key part of the project work would not have been possible. The authors are grateful to ACARP project coordinator Peter Bergin and project monitors Owen Salisbury, Sharif Burra, Brad Elvy, Roger Byrnes, Paul O’Grady, Peter Corbett, Bharath Belle, Raelene Obrien, Russell Thomas, Frank Fulham and Ian Stone, who have provided very valuable comments and suggestions relevant to the study during several review meetings. We also thank my colleagues Matt van de Werken and Bongani Dlamini for their major contributions in the monitoring and microseismic data analysis.

**REFERENCES**


THE FUNDAMENTALS OF MODERN GROUND CONTROL MANAGEMENT IN AUSTRALIAN UNDERGROUND COAL MINES

Jason Emery¹, Ismet Canbulat¹ and Chengguo Zhang¹,∗

ABSTRACT: Underground coal mining is inherently hazardous, with uncontrolled ground failure regarded as one of only several critical risks for multiple fatality events. Development, implementation and management of overarching systems and procedures for maintaining strata control is an important step in mitigating the impact of ground failure hazards at a mine site operational level. This paper summarises the typical Proactive Ground Control Management System (PGCMS) implemented in various Australian underground coal mines. Australia produces approximately 100 million tonnes a year of metallurgical and thermal coal from approximately 30 of the world’s safest longwall mines operating in New South Wales and Queensland. The increased longwall productivity required to achieve both high levels of safety and profitability, places significant emphasis on the reliability of proactive ground control management for longwall mining operations. Increased depths, adverse geological conditions, elevated variable stress regimes and weaker ground conditions, coupled with an industry wide need for increased development rates continue to make ground control management challenging. Ground control management is not only about ground support and pillar design though but is also a structured process that requires a coordinated effort from all levels of the workforce to both minimise the occurrence of adverse geotechnical events and mitigate the potential risks when they do occur. The PGCMS presented in this paper is proven to provide both a safer and more productive mine environment through minimisation of unplanned delays. The critical elements of the method are presented in detail and demonstrate the utility and value of a ground control management system that has potential for implementation in underground coal mining globally.

INTRODUCTION

Coal has been the main source of energy for producing electricity in Australia for over 200 years and in more recent times a top five export commodity in terms of revenue. The Department of Industry, Innovation and Science estimates that saleable black coal production in Australia was over 440 million tonnes in 2014-15, which is more than 50 per cent higher than a decade ago and over 150 per cent higher than 1990-91 (Department of Industry, Innovation and Science, 2016). Open-cut coal mining accounts for approximately 77 per cent of coal production in Australia. Currently, approximately 100 million tonnes a year of coal is produced from 30 longwall mines operating in New South Wales and Queensland. The geological and geotechnical conditions vary significantly both between and within these operations and successfully managing such variability requires an integrated, Pro-active Ground Control Management Strategy (PGCMS).

Australia’s coal mining safety record outperforms that of the US, China and other major coal mining countries (Harris, et al., 2014), with the fatal risks associated with strata failure typically well managed. Large ground control failures resulting in unplanned delays to production and lost revenue do still occur relatively frequently but are often only reported on internally within organisations for commercial reasons. Over the years, many mines have developed various ground control strategies to minimise or eliminate uncontrolled strata failures as the loss of control poses significant safety and financial risks. Since the decline of coal prices from 2012 (Minerals Council of Australia, 2015), a typical internal ground control strategy has evolved utilising in-house geotechnical engineering groups within the company and/or mine site technical services departments. This has resulted in most geotechnical designs being conducted by mine site geotechnical engineers whereas in times gone by this was often provided by an independent third party, or consultant engineers. Although this new approach is sound practice and acceptable if completed by suitable competent persons, it should still incorporate some form of peer and/or external third-party review. But rather than rely on the third party, the PGCMS has evolved from a desire for mining companies to continue to manage the risks on a daily basis throughout the life cycle of an operation to ensure it operates as safely and productively as possible.

¹ School of Mining Engineering, University of New South Wales, Sydney, Australia
∗ Corresponding author: Email: chengguo.zhang@unsw.edu.au, Tel: +61 2 9385 5524
The PGCMS involves many critical elements. It is not only an understanding of the impacts of the geotechnical environment on likely ground behaviour to allow the mines to extract underground reserves safely and economically, but also how to predict, communicate and escalate the expected conditions in a timely manner to the appropriate audience so its impact is suitably mitigated. This paper provides an overview of the foremost PGCMS in Australia, describes the critical elements and main features of the system, and offers guidelines for all coal mine operators to consider incorporating into a ground control management plan.

CRITICAL ELEMENTS OF A PRO-ACTIVE GROUND CONTROL MANAGEMENT STRATEGY

A PGCMS is a practical stepwise, systematic process that has evolved within the Australian coal mining industry to ensure that no person is exposed to an unacceptable level of risk from an uncontrolled strata failure. This approach is based on a standard Plan–Do–Check–Act (PDCA) methodology (or similar) originally developed as a tool to control and continuously improve processes in manufacturing, otherwise known as the Demming Wheel (Demming, 1950).

![Figure 1: PDCA Model (after Demming, 1950)](image)

The following principles form the PDCA model in an iterative manner:

- **Plan** – planning and documentation of objectives, expected outcomes, systems, processes and activities
- **Do** – acceptance and implementation of the plan
- **Check** – measurement and analysis that is understood and accepted
- **Act** – review and management follow-up, enact a response to changing conditions and implementation of improvement initiatives that are sustainable.

For implementation of a successful PGCMS, it is vital to ensure that all relevant stakeholders are involved in its formulation, and once formulated, the process is integrated into the mine’s “Safety and Health Management System” (SHMS). It is equally important that the PCGMS is communicated and made available to the entire workforce. An underlying requisite is that the process is owned not only by mine management (usually the Site Senior Executive), but the entire workforce. In many cases, implementation of a ground control management plan may be less than optimal as personnel fail to complete their roles defined within the plan. This may not be due to negligence but maybe the strategy itself is flawed, the person was unaware of their requirements in the plan or several other valid reasons not involving disregard. Involving people at all levels of the organisation in the process as required by a PGCMS, creates ownership at all levels. Hence the first point is worth iterating again; the relevant persons at the mine should all be involved in the development of the PCGMS so that they have ownership of the final document, understand its components and why they were included.

The elements of a typical ground control plan used in Australian underground coal mines can be grouped into four main categories: a) procedures and standards, b) data collection and design, c) implementation and monitoring and d) review and investigation. The following critical elements are included in those four categories:
• strata control principal hazard management plan
• ground control standards
• geotechnical design manuals and programs (software and calculators)
• geotechnical design reports
• geotechnical data and associated calculations
• geological and geotechnical mapping and hazard plans
• investigation of incidents and hazard reports
• trigger action response plans specific to various stages of mining
• monitoring regime (observation, extensometry, powered roof support monitoring etc.)
• risk management and risk assessments
• permit to mine process (also commonly referred to as Authority to Mine Process)
• periodic geotechnical testing and sampling maintained in an accessible database
• periodic third-party audits of high-risk zones ("critical areas")
• ground support product standard specifications, evaluation and testing procedures
• weekly reporting system
• training for employees and geotechnical engineers
• definition and appointment of suitably qualified geotechnical engineers
• audits and reviews of ground control management process

The above list is not exhaustive; there are other processes that Australian mines may use, and some listed that are not used. It is up to the individual mine operator to determine what is most suitable for their operation, often via a risk assessment process. While some mining companies develop the above elements as standalone processes, others combine them into an overall “ground control management plan” or into the “principal hazard management plan” in a hierarchical structure with subordinate documentation referenced in the overarching plan. The following sections present a practical guideline for the combination of those elements.

Procedures and standards

Strata control principal hazard management plan

The Principal Hazard Management Plans (PHMP), also commonly referred to as principal mining hazard management plans or ground control management plans are required by legislation in Australia as part of each mine’s safety and health management system. The PHMP must provide for the following basic elements so far as is reasonably practicable:

• risk identification and assessment
• hazard analysis
• hazard management and control
• reporting and recording relevant safety and health information and;
• recording of data and any calculations made.

It is a common practice that each principal hazard is individually risk assessed before the commencement of mining and that the principal hazard management plans are developed in accordance with that assessment to mitigate the risks identified. As discussed previously PHMPs are unique to each operating mine, involve a comprehensive and systematic investigation and analysis of all aspects of risk to health and safety associated with the principal hazard. Following commencement of mining the underlying risk assessments are reviewed periodically prior to the PHMP being reviewed, based on several criteria typically documented within the PHMP (including when a major loss of control event occurs at the mine).

The PHMP must address hazard identification, control selection, control management, review, audit and corrective action to manage risk associated with the principal hazard to within acceptable limits. In general, principal hazard management plans include:

• legislative requirements
• background information about the mine including history of any significant loss of control events
• major strata control risks to the operation
• underlying risk assessments
• geotechnical design guidelines
• review requirements for the principal hazard management plan or equivalent
• geotechnical characterisation (domains, zones, districts etc.)
- roles and responsibilities in accordance with the mines organisational structure
- all relevant Standard Operating Procedures and Standard Work Procedures
- the site’s critical controls process including methods of assessing and recording the quality of implementation.

**Ground control standards**

The aim of ground control standards is to ensure that the ground control processes at the mines are carried out to a minimum acceptable standard to ensure safe and economic extraction of reserves and to provide a set of consistent and auditable outputs.

These standards also provide a framework for compliance with the relevant government statutory bodies and internal corporate regulations of the operator (such as Anglo American Fatal Risk Standards, 2008; BHP Billiton Fatal Risk Control Protocols, 2003). Regular audits are conducted at the mines to check that as a minimum these standards are met, and appropriate controls are in place. In general, the requirements of fatal risk standards, which form part of the ground control standards, are summarised in the following risk areas: a) plant and equipment requirements, b) system and procedural requirements and c) people requirements, which are regularly audited six-monthly or annually (Anglo American Fatal Risk Standards, 2008; BHP Billiton Fatal Risk Control Protocols, 2003).

**Geotechnical design manuals, software and calculators**

Since the decline of coal prices, most coal companies in Australia employ geotechnical engineers within their technical services departments to conduct detailed geotechnical designs. These designs are often complex and require specialist skills only attained from specific training and experience. It is an important requirement of these designs that a standard process is followed, and all assumptions and calculations are transparent and auditable, as required by legislation in both New South Wales and Queensland. The internal design manuals and programs must be aligned with these requirements as the minimum unless otherwise determined by risk assessment.

Over the years, numerous pillar and roof support design methodologies have been developed in Australia and elsewhere. These methodologies are based on empirical, analytical and/or numerical methods. As the underlying principles and the databases used in the development of these methodologies vary, their applicability to geotechnical environments also vary, often requiring engineering judgement to determine their suitability. Geotechnical manuals summarise the recommended design methodologies and make recommendations on the applicability of them and the minimum design process maps as well as the acceptable standards that need to be used at mines by all personnel, both employees and contractors. In addition, design manuals make recommendations for monitoring, mapping and hazard plans to ensure that the design can be analysed, reviewed and adjusted if required in a timely manner.

A premise of having a design manual is that a standard design process is followed which is auditable and repeatable. An example of such a design and evaluation process is presented in Figure 2. The design manuals typically contain the following sections:

- overall geotechnical design process (i.e. flow chart)
- roof support design – recommendations on the input data, serviceability requirements, roof support design strategy in standard and in critical areas, e.g. longwall install and recovery roadways, design criteria, implementation and communication (i.e. support plans), review process and data storage
- pillar design – design process, pillar types and serviceability requirements geological and geotechnical data, design methodologies, design criteria (i.e., factor of safety and probability of failure), implementation and communication, review process and storage of data
- monitoring – ground deformation monitoring, roof support performance monitoring, stress measurements, critical area audits, surface subsidence monitoring, longwall powered support monitoring, implementation and communication, and monitoring data collection and analysis
- mapping and hazard plans – methodology, data requirements and mapping, currency of data, hazard plan presentation and communication.
In terms of ground support, there are no universally accepted roof and rib support design methodologies in Australia. Therefore, many mines tend to use a “combined support design methodology” which considers several methods and/or uses one method for the design and then one or more methods to back analyse and check the design. The following methods are generally used:

- analytical methods for buckling, shear and dead-weight loading
- field testing and monitoring (including underground observation)
- numerical modelling
- rock mass classification and empirical analysis.

For pillar design, there is more uniformity amongst the geotechnical fraternity, who rely on the following methods to design coal pillars based on loading and serviceability requirements:

- UNSW pillar design methodology (Salamon et al., 1996)
- Analysis of Longwall Pillar Stability (Mark, 1990)
- Analysis of Longwall Tailgate Serviceability (Colwell, 1998)
- 2D and 3D numerical modelling using both Finite Element Method and Distinct Element Method.

Worthy of special mention is the ALTS design methodology, which was initially provided to the Australian coal industry in early 1999 (Colwell, 1998) and over a ten year period was continually refined and updated such that the latest version, ALTS 2009 (Colwell et al, 1999 and Colwell, 2010) and associated software package, has grown to be the prevalent technique for chain pillar and gateroad ground (roof and rib) support design at most operating longwall mines in Australia.

Figure 2: Ground control standard process map (Anglo American Coal, 2014)
This is largely because the outputs from ALTS 2009 most accurately reflect the design requirements to provide serviceable gateroads associated with longwall extraction. In addition, ALTS 2009 is relatively quick and straightforward to use allowing typically time poor mine site geotechnical engineers to conduct in-house design work with high levels of accuracy, improving both safety and productivity at those mine sites. However, like all design methodologies the geotechnical environment needs to be properly characterised so that data input parameters (such as the Coal Mine Roof Rating - CMRR and in situ stress levels) and their potential variation across the area under design consideration is well understood and therefore data input can be selected using appropriate/prudent judgement.

Geological and geotechnical mapping and hazard plans

Mapping and hazard plans are integral parts of an effective ground control management strategy requiring a consistent and standardised process integrated with the daily operations of a mine. The mapping and geotechnical hazard plan standards are usually linked to the operational and planning cycle of the mine assisting in reducing uncertainty around the nature of the rock mass and its impact upon the mine schedule.

In development sections, the geological mapping is conducted regularly immediately behind the development face, typically on a weekly basis. The longwall mapping is more variable depending on the difficulty of the seam, with some mines only mapping the gates before the panel starts while other mines map the longwall face after every shear is taken. Development mapping can be a good indicator for areas of increased risk due to geological and mining induced features but is heavily dependent on its quality and consistency. Best practice requires a second underground inspection by a geotechnical engineer to verify mapping and check for ongoing signs of deterioration prior to utilising the data for design purposes.

Development hazard plans typically use data from mapping, borehole cores, borehole geophysics and remote sensing such as surface seismic reflection surveying and aeromagnetic surveying. In general, development hazard plans consider the following information:

- thickness of the seam and seam split
- stress environment
- depth of cover,
- roof competency: uniaxial comprehensive strength and coal mine roof rating
- floor competency: uniaxial comprehensive strength and slake durability
- presence, persistence and magnitude of discontinuities (faults, joints, shears)
- presence and nature of igneous intrusions
- interaction between geological structures
- overlying competent rock thickness and strength (e.g. sandstone or basalt channels)
- dip of the seam
- water or water bearing strata

Hazard plans for secondary extraction refer to:

- geological structures (reverse or thrust faults, mid-angled structures, structures aligned at a shallow angle to the roadway and areas where two or more geological structures intersect)
- direction of minor and major geological structures
- presence and nature of igneous intrusions
- roof competency: uniaxial comprehensive strength and coal mine roof rating
- floor competency: uniaxial comprehensive strength and slake durability
- roadway size
- roof slabbing, falls and guttering
- roof displacements following the development
- horizontal stress direction, magnitude and notch
- mine site-specific hazards (i.e., depth of cover, in-seam and multi-seam interactions, installed densities of support, rib spall, changes in seam dip and sandstone or conglomerate channels)
- installed support densities
- off-line cut areas
- installed support
It is imperative that all available data (historical and recent) is presented on mapping and hazard plans, which are provided prior to the start of any underground development and any secondary extraction. For pre-feasibility and feasibility studies hazard plans are also provided to the project teams. It is also imperative that hazard plans are routinely updated with the most recent information.

**Risk management and risk assessments**

In the context of this paper, risk management refers to co-ordinated activities to direct and control an organisation with respect to risk; and risk assessment is the overall process of risk identification, risk analysis and risk evaluation (Standards Australia, 2009).

The overall risk management strategy of coal mines is outlined in the mines’ health and risk management plans i.e. PHMP. Risk assessments are an important part of this plan and are used extensively throughout the Australian mining industry to underpin the strategy. There are many
publicly available publications on risk management and assessment procedures and standards. This paper explains how risk assessments are used in coal mining ground control.

Risk assessments in ground control are conducted in the following stages of mining:

- during pre-development studies, e.g., pre-feasibility, feasibility, pre-development design studies
- when preparing long term mine plans
- prior to the development of gateroads or mains sections
- prior to the extraction of longwall panels
- in all other circumstances when a specific assessment is warranted, for example prior to mining through a structurally disturbed zone or following a major change in mining circumstances since a previous risk assessment.

The aim of the risk assessment is to identify all potential hazards, to rank them and implement the appropriate controls to reduce their impact on safety and productivity. The risk assessments conducted prior to the start of development and the longwall consider the following information:

- geology and geotechnical – all potential structures, for example faults, dykes, seam thinning and thickening, seam rolls, competent layers in overburden, change in roof and floor competency, dip of seam and potential stress environment
- ventilation
- gas
- spontaneous combustion
- surface infrastructure, cultural heritage, surface vegetation, water bearing structures (e.g., dams)
- ground water and underground water hazards
- previous workings
- hazards associated with operating the mining equipment
- other hazards as deemed appropriate.

Many of these hazards are not identified with confidence prior to the start of development. However, many are observable before the commencement of secondary extraction (i.e., longwall retreat) and should be included in the new risk assessment.

Trigger action response plan

A Trigger Action Response Plan (TARP) is an essential element of any PGCMS. A TARP is designed and implemented for a specific geotechnical area or domain to deliver a simple set of rules to provide guidance on support requirements, and other actions required as a response to specific visual and/or monitoring ground behaviour. TARPs typically categorise the geological and geotechnical conditions in a “traffic light” system to indicate different risk levels. In addition, TARPs refer to the, required responses and responsibilities of all relevant people such as the deputy, mine manager, miner, geotechnical engineer, geologist etc. This may also include the appropriate level of support to be installed. An example of a longwall strata control TARPs is presented in Figure 3, which shows the conditions and trigger levels in different geological and geotechnical conditions for the longwall face and the gateroads.

Australian coal mines use ground control TARPs for development, outbye areas, longwall face, longwall gateroads, installation roadways and longwall recovery. To be an effective trigger action response plans should define:

- different levels of ground behaviour (triggers), based on key parameters
- responses to triggers (changes in monitored parameters and associated actions)
- individual responsibilities.

The TARP should be as short and simple as possible and ideally not longer than one page. The number of relevant parameters should be distilled to the minimum required to reflect the range of ground behaviour experienced locally. Ultimately, production personnel must have significant input to the documentation and the system, so that common ownership exists.

Data collection

Routine geotechnical testing and sampling

Australian mines rely on extensive geological and geotechnical data for geotechnical designs and for overall ground control management. To ensure that the required data is provided adequately and in a
timely manner, mining companies developed guidelines for geotechnical testing and sampling for underground and open cut operations.

<table>
<thead>
<tr>
<th>Level</th>
<th>Green - Level 1</th>
<th>Orange Level 3</th>
<th>Red Level 4</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Condition</strong></td>
<td><strong>GATE CONDITIONS</strong></td>
<td><strong>GATE CONDITIONS</strong></td>
<td><strong>GATE CONDITIONS</strong></td>
</tr>
<tr>
<td></td>
<td><strong>ROOF (&lt;10m OB of face)</strong></td>
<td><strong>ROOF (&lt;10m OB of face)</strong></td>
<td><strong>ROOF (&lt;10m OB of face)</strong></td>
</tr>
<tr>
<td></td>
<td>- Minor faulting of roof rib corner</td>
<td>- Minor faulting of roof rib corner</td>
<td>- Minor faulting of roof rib corner</td>
</tr>
<tr>
<td></td>
<td>- No additional gatering</td>
<td>- No additional gatering</td>
<td>- No additional gatering</td>
</tr>
<tr>
<td></td>
<td>- No signs of additional loading on support</td>
<td>- No signs of additional loading on support</td>
<td>- No signs of additional loading on support</td>
</tr>
<tr>
<td></td>
<td>- No roof talk</td>
<td>- No roof talk</td>
<td>- No roof talk</td>
</tr>
<tr>
<td></td>
<td><strong>RIBS (&lt;10m OB of face)</strong></td>
<td><strong>RIBS (&lt;10m OB of face)</strong></td>
<td><strong>RIBS (&lt;10m OB of face)</strong></td>
</tr>
<tr>
<td></td>
<td>- Minor additional rib spall &lt;50mm</td>
<td>- Minor additional rib spall &lt;50mm</td>
<td>- Minor additional rib spall &lt;50mm</td>
</tr>
<tr>
<td></td>
<td>- Installed rib support controlling rib conditions</td>
<td>- Installed rib support controlling rib conditions</td>
<td>- Installed rib support controlling rib conditions</td>
</tr>
<tr>
<td></td>
<td><strong>TELLTALES (&gt;10m from face)</strong></td>
<td><strong>TELLTALES (&gt;10m from face)</strong></td>
<td><strong>TELLTALES (&gt;10m from face)</strong></td>
</tr>
<tr>
<td></td>
<td>- Additional movement &lt;10 mm since stable conditions</td>
<td>- Additional movement &lt;10 mm since stable conditions</td>
<td>- Additional movement &lt;10 mm since stable conditions</td>
</tr>
<tr>
<td><strong>FACE</strong></td>
<td><strong>FACE</strong></td>
<td><strong>FACE</strong></td>
<td><strong>FACE</strong></td>
</tr>
<tr>
<td></td>
<td>Face spall &lt;0.5m</td>
<td>Face spall &gt;1m for more than 10 checks</td>
<td>Face spall &gt;1m for more than 10 checks</td>
</tr>
<tr>
<td></td>
<td>Minor spall ahead of head drum</td>
<td>Spall &gt;10 checks ahead of head drum</td>
<td>Spall &gt;10 checks ahead of head drum</td>
</tr>
<tr>
<td><strong>ROOF</strong></td>
<td><strong>ROOF</strong></td>
<td><strong>ROOF</strong></td>
<td><strong>ROOF</strong></td>
</tr>
<tr>
<td></td>
<td>Minor fresh cracking</td>
<td>Fresh fracturing evident</td>
<td>Fall of roof ahead of face in either gate</td>
</tr>
<tr>
<td></td>
<td>Additional faulting of roof/rib corner 10-20m OB Face</td>
<td>Additional faulting of roof/rib corner 20m OB Face</td>
<td>Fall of rib greater than rib bolt length and further than 5m OB face</td>
</tr>
<tr>
<td></td>
<td>Additional gatering 200-300mm &lt;5m OB Face on one side only</td>
<td>Roof bolts and plates deforming, Tendons breaking or heavily deformed plate</td>
<td>Rib bolts and plates deforming, Tendons breaking or heavily deformed plate</td>
</tr>
<tr>
<td></td>
<td>- Relatively infrequent roof talk</td>
<td>Numerous broken bolts, Standing support showing significant loading</td>
<td>Numerous broken bolts, Standing support showing significant loading</td>
</tr>
<tr>
<td></td>
<td><strong>GATE</strong></td>
<td><strong>GATE</strong></td>
<td><strong>GATE</strong></td>
</tr>
<tr>
<td></td>
<td>Additional rib spall 100-150mm (&gt;10m OB but &lt;20m OB Face)</td>
<td>Additional rib spall &gt;150mm (&gt;20m OB Face)</td>
<td>Additional rib spall &gt;150mm (&gt;20m OB Face)</td>
</tr>
<tr>
<td></td>
<td>Rib bolt plates beginning to buckle or break</td>
<td>Rib bolt plates buckled and breaking</td>
<td>Rib bolt plates buckled and breaking</td>
</tr>
<tr>
<td></td>
<td><strong>TELLTALES (&gt;10m from face)</strong></td>
<td><strong>TELLTALES (&gt;10m from face)</strong></td>
<td><strong>TELLTALES (&gt;10m from face)</strong></td>
</tr>
<tr>
<td></td>
<td>- Additional movement &lt;10 mm but &lt;40mm since stable conditions</td>
<td>- Additional movement &lt;10 mm but &lt;40mm since stable conditions</td>
<td>- Additional movement &lt;10 mm but &lt;40mm since stable conditions</td>
</tr>
</tbody>
</table>

**Figure 5:** An example of a longwall Trigger Action Response Plan. Note: the responses and the roles and responsibilities of relevant people are not included in this example.

These guidelines provide a framework for collecting geotechnical data during drilling in exploration programs to ensure that geotechnical data are measured and recorded systematically to a common standard. This ensures the data can be used reliably for the assessment of rock mass and the evaluation of mine design parameters. The frequency of geotechnical testing is also specified in the guidelines.

The guidelines are usually used in conjunction with mandatory guidelines for core logging to ensure that:

- Samples will provide adequate representation for the area of interest and/or over the entire lease.
- Samples are taken from within the correct/target horizons and lithology.
The correct tests are undertaken on the samples, to allow for analysis of the conditions likely to be encountered during mining.

The correct number of tests is conducted.

All other data is recorded to allow for the assessment of the materials, and later calculation of rock mass rating systems, such as coal mine roof rating.

Standard, reliable testing procedures are used in testing and data collection so that minimal uncertainties are introduced into the design during these processes.

Data storage is adequate.

It is well-accepted by the Australian coal mining industry that collecting adequate geological and geotechnical data through monitoring, instrumentation, drilling from the surface or underground and through 2D and 3D seismic surveys is necessary to minimise the potential for an uncontrolled fall of ground. Further the consensus is that the cost of the data acquisition, although high, is paid for many times over by improvements to productivity from reduction in uncertainty. Therefore, coal mines usually collect substantial amounts of geological and geotechnical data throughout the life of the mine operations.

**Roof bolt and accessories – standard specifications, evaluation and testing procedures**

The Australian mining industry has access to a variety of ground support products. A concern with many support products is that the mines must make sure that the products supplied meet the minimum specifications outlined in ground support selection and testing standards. Therefore, in recent years, there has been an increasing emphasis on quality assurance and quality control testing.

Ground support selection and testing standards specify the dimensional, material and testing requirements for roof bolts, cables and accessories used at mines, including the steel bars, cables, nuts, plates, resin and mesh for use in a complete assembly. These standards also specify the requirement for the suppliers to conduct routine tests to ensure that the roof support components comply with the minimum standards.

There are several factors that contribute to the under-performance of installed ground support elements. These factors should be controlled to specified tolerances by regular, systematic quality control procedures. The factors that can affect the performance of a roof bolt support system can be classified into two areas: indirect controllables and direct controllables.

The indirect controls are related to suppliers’ quality control procedures, such as metallurgical properties of roof bolts, deformation pattern of roof bolts, and chemicals used in the manufacturing process of resin capsules and the consistency of these properties. The standards require that the suppliers’ quality control procedures are audited routinely and that the manufacturer’s quality control procedures should comply with the relevant ISO and/or Australian Standards. All quality assurance and quality control test results are also provided to the site geotechnical engineers for random checking and record keeping.

The direct controllables can be divided into three distinct groups: ground support and accessories; compliance with the design; and quality of installation.

The minimum specifications of the standards include:

- roof bolts and cables – chemical composition, length, profile, straightness, finish, colour coding, colour coding, nut break out, mechanical performance, plates (washers), nuts, drill bits, rods and spanners
- resin – capsule size, shelf life, gel and setting time, bond strength and system stiffness, colour coding, packaging, uniaxial compressive strength, elastic modulus, creep, shear strength, push test, capsule diameter, capsule length, freedom from leakage
- mesh and straps – compliance with relevant Australian standards, grade, fire resistance, profile, dimensions, yield strength and storage.

For the above specifications, regular testing and audit requirements are also prescribed in the standards. These standards are also used in supply contracts to ensure that the standards are obligatory. All required tests are usually conducted by the suppliers.

Many mines may also require on site quality assurance and quality control testing conducted by independent companies to ensure that all support products comply with the minimum standards.
Monitoring

Regular Monitoring

Monitoring is probably the most important element of a PGCMS to prevent uncontrolled falls of ground. Therefore, not surprisingly, ground deformation and support effectiveness monitoring are also a requirement of the Australian legislation. In Australian coal mines, strata monitoring consists of both observation (qualitative) and measurement (quantitative). Both methods are required primarily for design verification but also for identifying areas of non-conformance (such as unpredicted anomalies) so that remedial measures can be applied in a timely manner.

Monitoring provides different qualities of data to different users (i.e., operators for decision making and geotechnical engineers for design purposes). Ground monitoring is undertaken (Galvin, 2016):

- aid in exploration
- establish benchmark data for environmental approval and licensing purposes
- determine properties for input into mine design
- validate mine design
- validate the quality of ground support hardware
- validate the quality of ground support installations
- research the unknown
- provide timely warning of deviation from predicted ground conditions and design performance, both in the short and long term
- identify, quantify and verify mining effects, impacts and consequences.

Australian mines conduct extensive monitoring to understand the ground behaviour and to measure all parameters that can result in strata problems. These include:

- pre-mining stresses
- stress changes
- displacements of roof, ribs, floor and pillars
- reinforcement installation procedures
- permeability of strata
- longwall shield loading
- performance or condition of pillars
- installed support.

Ideally, monitoring systems need to be designed and implemented to provide timely, fail-safe warning of the development of critical ground conditions so that personnel and equipment are not exposed to burial, entrapment, windblast, dust, and noxious and flammable atmospheres. Gaps in knowledge and technology currently prevent these monitoring goals from being fully achieved (Galvin, 2016). The use of real time strata control displacement monitoring (extensometers) is continuing to grow in popularity as the technology develops, however it is still not common or accepted practice.

Permit to mine process

A Permit to Mine (also known as Authority to Mine or PTM) is a site-based process that identifies the principal mining hazards and controls for each new mining area. This typically includes expected ground conditions, ground support requirements, gas drainage and ventilation compliance requirements, inrush potential, surface structures and restrictions. A Permit to Mine is developed before mining takes place in any area. The process originated as a tool to assist in controlling the risk of outburst but has since evolved to cover all principal hazards.

All relevant information is listed, reviewed and authorised by all parties (i.e., mine gas and ventilation engineer, geotechnical engineer, geologist, surveyor, development and/or longwall crews and mine manager) to indicate that the identified risks are considered and controlled. This allows the mine manager to make a well-informed decision on the expected hazards and ensure the appropriate controls are in place prior to approving mining to commence. If used appropriately, this system is powerful in identifying and mitigating risks before any mining takes place, hence it is universally adopted and covers all technical risk factors including ground control.
Regular geotechnical critical area inspection

Historically, strata control failures in mine access roadways that cease production are not uncommon, and often may have been preventable with earlier identification and intervention. Regular geotechnical critical area reviews ensure that certain critical areas of the mine are regularly inspected so that the ground support in those areas is in line with the mine’s minimum support requirements and any ongoing deterioration is recognised. A critical area is defined as areas of the mine where strata deterioration or failure may cause process delays or expose people or equipment to potential harm.

![Figure 6: A Two-Anchor Remote Reading Tell Tale Schematic (Buddery, et al., 2018)](image)

For active panels, there are routine processes in place, as stipulated in principal hazard management plans and mine inspection regimes, to manage the hazards associated with ground control. Therefore, the critical area inspections are specifically for outbye areas of the mine where inspection is less frequent.

Critical area inspections involve simple visual observations of ground conditions and identification of deteriorating ground conditions so that those areas are included in the mine’s maintenance scheme e.g. replacing corroded or damaged support elements. The inspections are usually performed by an independent third party who are not familiar with the area so that an objective and unbiased inspection of the ground conditions is conducted and recorded.

Weekly reporting system

Weekly reporting systems are another valuable tool in ground control management to ensure that all development and secondary extraction panels are inspected by geologists and/or geotechnical engineers and a standard inspection sheet is filled in. Following the inspections, a standard weekly report is produced and distributed to all relevant parties (i.e., development crews, longwall crews and management) to indicate the areas of non-compliance with ground support designs given in trigger action response plans. The frequency of response plan triggers and installed support are also mapped to ensure that the trigger levels in the response plans are appropriate for the conditions in the panel.

The geological mapping of the panel is also conducted during these inspections and a mine plan with geological structures, installed support and general geotechnical conditions in the panel are included in weekly reports.

Training

Ground control management in Australian mines involves many critical steps and requirements. To ensure that these steps are well-understood, and the requirements are met internal and external training programs are provided to geotechnical engineers as well as the workforce. A training scheme is also a requirement of Australian legislation.
All Australian mines have a system in place to ensure that all personnel working underground are competent, trained and authorised to perform the geotechnical tasks assigned to them. There is also an on-the-job training and assessment process for mine workers. All employees are also trained by a geotechnical engineer in the following areas:

- support design principles
- principal hazard management plan requirements
- identification of geological anomalies which contribute to weaker ground conditions
- trigger action response plans.

Geotechnical engineers are usually responsible for ensuring that refresher training courses are provided regularly to all employees. The training of junior geotechnical engineers involves in-house training sessions and external courses. In-house training sessions involve training in ground control management strategy and the geotechnical design processes. External courses are usually structured around new developments in geotechnical engineering. Registered professional engineers are required to complete and demonstrate Continual Professional Development (CPD) to a level determined by the governing body to remain registered and audited regularly.

Critical controls

As risk management has evolved over time so have the checks and balances used to assess the health of the system. One aspect that is now elementary to a PGCMS is the implementation of a critical controls monitoring or critical control verification process. A Critical Control (CC) is defined as a risk control that is either crucial to preventing an event from occurring or mitigating the consequences of an event (ICMM, 2015). Each mining company has its own slight variation on the CC process however it will typically consist of a series of verification activities to be performed periodically on the identified CCs. This verification will be completed by the risk owner within the site management team and compliance reported through to corporate. Where there are deficiencies identified action plans must be developed and assigned to relevant persons to ensure the deficiency is rectified. An example for ground control is the critical controls associated with geotechnical design. A universal critical control for geotechnical design is that each design is completed and peer reviewed by a competent person in accordance with the PHMP and ground control design guidelines. Evidence of this process must be available for each design currently being implemented at the mine (typically a peer review sign off form). Another universal CC is the monitoring of underground excavations according to a scheduled inspection regime. Documentation must be supplied as evidence that both the monitoring, and the appropriate responses to this monitoring, are being carried out in accordance with the relevant documentation.

![Figure 7: Typical Critical Controls Process Flow Chart (ICMM, 2015)](image_url)
Review and investigation

Investigation of accidents and incidents

In ground control management, accidents and incidents are related to fall of ground which is defined as an unplanned movement of ground that results in a failure within the ground control system with the potential to affect safety and production or has a business cost.

Investigations of accidents and incidents are required by Australian legislation. Queensland Coal Mining Safety and Health Regulation (2013) states that a coal mine’s safety and health management system must provide the following:

a. the procedure for investigating accidents and incidents at the mine
b. making the investigation findings available to the mine’s workers
c. implementing corrective action for accidents and incidents.

Many accident and incident investigations involve using the Incident Cause Analysis Method, which provides logic towards incident and accident causation and supports the notion that most incidents and accidents are rarely caused by a single act or condition, but rather by a number of factors working together (Mining Industry Resource Management, MIRMgate).

Audits and review of ground control management process

A PGCMS has many elements and it is a live process. The implementation of this strategy is not a simple task; it requires resources and time. To ensure that all operations are at comparable levels in implementation of the strategy, regular internal and external audits are conducted by mining companies.

Every element of a ground control strategy is also reviewed regularly to ensure each element is still effective and applicable to the environment the mine is operating in. In an event of a major failure (such as fall of ground), reviews of the complete process are also conducted, and this requirement is included within the ground control management plan.

Strata Defect Hazard Register

Although the risk of ground control failure is highest when within a certain distance of the active mining face, the deterioration of ground support over time has also become a key element of a PGCMS. With large-scale modern mines in operation for many decades the deterioration of ground support and the associated conditions increases with time due to weathering of the ground, weathering of support elements and damage due to impact from mobile equipment. Due to this deterioration over time and an absence of response there have been several large failures generally in outbye areas of mines that incurred significant business losses and unacceptable levels of exposure to coal mine workers. Many operations now utilise a system that includes the regular reporting, inspection and remediation tracking for identified defective strata support in outbye areas of the mines. These strata defects are also tracked in global information system (GIS) enabled maps so that the defects may be identified prior to planning tasks being undertaken in certain areas.

Figure 8: Example Strata Defect Hazard Map
CONCLUSIONS

Ground failures pose a high-level risk to both individuals and production in underground coal mines. Therefore, Australian mines have developed over time what is considered the best practice pro-active ground control management strategy globally, to provide work areas both safe for employees and to minimise unplanned delays to production.

This paper summarised the typical best practice pro-active ground control management strategy used in Australian underground coal mines and detailed the critical elements. A pro-active ground control management strategy is not only about roof support and pillar designs. Applications of these steps vary significantly by both the size of a mine and the size of a mining company. Yet all ground control strategies are required to comply with and demonstrate compliance with the relevant Australian legislation. Although not all Australian coal mines apply all the elements outlined above, most of the mines do have similar systems that they utilise in daily ground control management. This requires a high level of onsite geotechnical knowledge and skills with most companies now employing several geotechnical engineers and geologists at each mine site to ensure the PGCMS is implemented effectively.

As research into ground control continues to improve, so does the application of ground control strategy and its elements with emphasis on roof and rib support designs, and technology for instrumentation and monitoring. The material presented in this paper gives guidance to mining engineers in other countries for achieving safe and productive coal extraction similar to that being achieved in Australia through a PGCMS.

REFERENCES


THIN SPRAY-ON LINERS: A HISTORICAL OVERVIEW AND THEIR FUTURE POTENTIAL AS SUPPORT IN UNDERGROUND COAL MINES

Claire Morton¹, Zhongwei Chen² and Mehmet Siddik Kizil³

ABSTRACT: Thin Spray-on Liner (TSL) is a fast setting, multi-component, polymeric material that is designed to be spray applied to a rock surface and provide areal support. TSLs are widely used in hard rock mining and some civil applications; however, they are yet to become a preferred support element in coal mining operations. Understanding of how TSL products behave to provide support to a coal rock mass and how they can be tested and monitored in-situ, is not well enough understood for them to be included as a routine support system. This presents a current challenge to the industry as how the assessment of a TSLs suitability or comparability to an existing support method can be done. This paper provides a broad overview of how TSLs have been used and tested to date and offers some insights into the future direction for establishing their use in underground coal mines.

BACKGROUND

To date, the application and design of TSLs in underground coal mines has been based on experience, field observations, engineering judgement and assumptions. It presents a current challenge to the industry as to how the assessment of a TSL’s suitability or comparability to an existing support method can be done. In order to accurately quantify the parameters that are critical to a TSL to adequately support a rock mass under the effects of fracturing, induced stresses and rock mass instability, the mechanisms by which the liner actually supports the rock need to be reviewed, understood and incorporated into an engineering support design (Saydam, et al., 2003).

TSL properties have been estimated and measured in the laboratory with testing methods continually being adapted and refined. In some material property testing, TSLs have proven to outperform other spray liners such as shotcrete (Yilmaz, 2011). However, coal specific laboratory and coal mine site testing and observations are underrepresented in practice and in literature. Thus, there is little use of TSLs as a support mechanism in underground coal mines, where mesh, both steel and cuttable, is used as the primary areal support for both roof and ribs. Using TSLs as part of the support method can potentially improve the advancing speed of development faces, in addition to offering a reduction in manual labour and reduced exposure to ground conditions for personnel, if it can be demonstrated that its technical performance is as good, or better, than current systems.

The onsite geotechnical engineer has the challenge of ensuring that any type of support that is introduced or used in a support design in lieu of an existing support type, meets or exceeds the performance specifications for the existing methods. The use of TSL as a support product has previously presented a challenge to this requirement as the behaviour and performance is quite different to any other support types in use.

While TSLs have been in use successfully in various types of tunnelling and mining environments (Yilmaz, 2011), their use in coal mines has not progressed as quickly. This may be in part because of the actual mechanical properties of coal and its ability to maintain structural integrity in laboratory testing. Pan et al. (2013) describe how the mechanical properties and behaviour of coal is different to rocks and governed by the rank of the coal and its microstructure. Furthermore, they found that the ability of coal to resist deformation is dependent upon the orientation of forces applied relative to the cleat alignment and the coal’s microstructure. The direction of mining relative to cleat and fracture orientation cannot always be planned to provide the most favourable outcomes for adhesive support systems. Localised variation within coal measures means cleat and fracture orientation may vary over

¹ Claire Morton, School of Mechanical and Mining Engineering. The University of Queensland. Email: c.morton@uq.edu.au
² Dr Zhongwei Chen, Senior Lecturer, School of Mechanical and Mining Engineering. The University of Queensland. Email: zhongwei.chen@uq.edu.au
³ Assoc. Professor Mehmet Kizil, Mining Program Leader, School of Mechanical and Mining Engineering. The University of Queensland. Email: m.kizil@uq.edu.au
short distances. Perhaps this adds additional complexity when trying to determine the effectiveness of a spray-on liner whose effectiveness relies on its ability to penetrate open spaces with the coal rock.

Figure 1: Photo of roof support in an underground coal mine showing mesh, bolts and cable support

The Australian mining industry developed a keen interest in the potential use of TSLs in both hard rock and coal mines due to the knowledge of their successful use abroad (Potvin and Nedin, 2003). A surge in interest initially resulted in the development of many TSL variations by various product manufacturers. However, many of the products were deemed unsuitable due to the inadequate mechanical or chemical properties. Since then, many of these products have been redesigned or new ones developed, but in the literature still no TSL system has achieved success to act as a standard mode of support in underground coal operations (Guner and Ozturk, 2018).

WHAT IS TSL

TSLs have generated interest since the early 1900’s when the first cement gun was used to spray an early type of shotcrete. The ease and speed of application of this type has widespread appeal for many industries, and TSLs are widely used in hard rock and civil applications throughout the world. Common TSLs, also referred to as skins or membranes, are reactive or non-reactive polymer or water-based materials formed from a combination of a multi-component, polymeric material, cement and sand, or cement only, that are sprayed onto the rock surface at a thickness between 3 mm and 5 mm, and form part of a surface support system (Yilmaz, 2011). TSLs are applied close to the mining face to contain spall and unravelling of rock. The use of flexible support membranes prevents rock degradation and structural failure of excavations by mobilizing and conserving the inherent strength of the rock mass immediately about the excavation surface (Tannant, 2001). By applying a liner to a freshly exposed rock face, fractures can be prevented from dilating and any potential blocks secured in place. Early restriction of movement of this type could offer improved strength of the rock mass and overall increasing the stability of the excavation. Research has also been conducted into the ability of TSLs to mitigate the impact of rockburst in controlled environments. This work has produced positive results, indicating that TSL may have application in mines that are prone to outburst type conditions (Archibald and Dirige, 2006).

Kanda and Stacey, 2019 investigated the performance of TSL in various hard rock mines in South Africa and collated their findings to present varied uses of TSLs at different operations. They reported that various TSLs had been used for reasons such as preventing falls of ground, reduce scaling of pillars, mitigate the need for barring down and as a secondary support element. Pending the performance of TSL in underground coal, if proven to be a successful support material this indicates that the potential uses of TSL are wide spread, depending on the properties of the individual product.

HOW TSL WORKS

A quality liner will adhere strongly to the surface on which it is sprayed and provide containment to the rib side. As with all reinforcing support schemes, a supporting TSL is designed to develop forces in response to ground movements to minimise deformation and displacement. To be effective in this, the liner must have sufficient integrity to limit the movements of individual blocks of rock. Coal is inherently a heavily jointed rock mass, which tends to unravel easily once spalling has begun. If a liner can act to prevent the initial spalling and subsequent unravelling of blocks of coal in the rib or roof, then the coal
blocks are forced to interact with each other essentially creating a stable beam. This is a critical point of differentiation between mesh and a TSL, where mesh is designed to allow deformation and spall to occur behind it, a TSL is designed to prevent it from occurring as shown in Figure 2. The greatest benefit of this type of support action is best realised prior to the rock undergoing significant displacements (Tannant, 1999), once significant spall or movement has occurred, the use of other restraining methods such as mesh may be more appropriate support options to employ.

**Figure 2:** Penetrating of open joints and fractures between TSL and rocks (Saydam et al, 2003).

The following table (Table 1) presents the support mechanisms relevant to TSLs acting as a support mechanism, from Tannant (2001) and identifies their importance in relation to underground coal mining environments. Understanding the relative importance of the properties tabled, and their criticality in determining the performance of a TSL as a support method is still an opportunity for the coal mining industry to examine.

**Table 1: Support mechanisms of TSL with their relevance to underground coal (from Tannant, 2001)**

<table>
<thead>
<tr>
<th>MECHANICAL PROPERTIES</th>
<th>SUPPORT MECHANISM</th>
<th>DESCRIPTION</th>
<th>IMPORTANCE FOR COAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear adhesion</td>
<td>Interlocking of coal blocks</td>
<td>For a TSL to offer interlock between discrete blocks of coal, it must be well bonded to the surface and have sufficient tensile strength to withstand block movement, which is demonstrated in Figure 2. In this instance, it has been shown that shear movement between the coal (or shown on laboratory manufactured substrate materials) can be prevented and the movement of blocks constrained.</td>
<td>This is a critical mechanism to consider when applying TSL to an underground coal environment. Coal is made up of many discrete blocks, bounded by joints and natural cleats.</td>
</tr>
<tr>
<td>Tensile adhesion</td>
<td>Gluing Action</td>
<td>Often working in conjunction with the restriction of block dilation, the action of spraying the TSL achieves penetration of the sprayed material into discontinuities or jointing which can be prevalent in coal as illustrated in Figure 1. By penetrating these openings and bonding to the surfaces, movement and rotation of blocks can further be restricted by this gluing action.</td>
<td>Coal is inherently jointed and cleated, providing opportunity for movement along these planes.</td>
</tr>
<tr>
<td>Shear strength</td>
<td>Containment/ Basket Mechanism</td>
<td>In instances where rock has failed behind a sprayed TSL, and the liner has sufficient tensile strength, a basket may form which acts to contain failed rock. This type of mechanism can occur when using mesh, shotcrete or TSL; however the amount of rock that is able to fail behind a tenaciously adhering TSL or shotcrete may be minimised when compared to movement behind mesh.</td>
<td>Buckling, bulging, fretting are all common failure modes in coal that could be constrained by this mechanism.</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>Skin/ membrane Action in conjunction with washers and bolt plates</td>
<td>Bolt plates are an important part of the support system and act to distribute the applied load from a bolt (or cable) to the surrounding strata. A sprayed liner can extend the area of influence of the face plate.</td>
<td>Increased face coverage can result in lower rate soft coal drop out, buckling and spall in coal ribs.</td>
</tr>
<tr>
<td>Ductility/ flexural strength</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
All the aforementioned support mechanisms may act independently or in combination with each other. In addition to the supporting mechanisms of TSLs, if sprayed to provide full coverage on rib or roof and rib, liners will potentially prevent the ingress of air and water into the coal and reduce deterioration associated with the weathering. While limiting ingress, the liner also acts to reduce egress of gas and water out of the coal, thus may provide other operational benefits to an operation by reducing gas emission or water make into workings.

**POTENTIAL BENEFITS IN UNDERGROUND COAL MINES**

With the potential to offer considerable operational benefits with their fast application rates and much reduced product per square metre compared to shotcrete, interest in TSLs and their development exists throughout the coal mining industry. TSLs’ advantages are extensively detailed throughout the literature and summarised by Yilmaz (2011):

- fast application rates,
- rapid curing times ranging from seconds to hours,
- reduced materials handling compared to shotcrete,
- high tensile strength with high areal coverage,
- high adhesion which enables early reaction against ground movement, and
- ability to penetrate joints.

Stacey and Yu (2004) suggest that these advantages have the potential to lead to improved cycle times, increased mechanization, and improved safety. If it can be implemented, the advantages of applying a thin spray-on liner at the coal face include, but not limited to, (i) better control over coal ribs close to the development face, (ii) reduced exposure for mine site personnel, and (iii) less manual handling operations – particularly if the system can be incorporated into existing surface to seam delivery infrastructure. The benefits of using these products from original product developers and listed by Li et al. (2016) are:

1. confine ground movement as soon as possible
2. unravelling of rocks is reduced compared to mesh – provide stability
3. provide immediate support to personnel compared to conventional support methods, i.e., bolt and mesh
4. active support method by constraining rock movement
5. reduce rib displacement and improve roadway serviceability
6. can achieve significant area coverage
7. surface to seam delivery can remove manual handling and assist in achieving Zero Harm
8. no rust / corrosion
9. fast set – obtain properties quickly
10. potentially improved cycle times for bolting constrained operations
11. progression towards automated system with rock support without personnel intervention.

**Full coverage support**

Bolting density and placement are critical considerations for any support regime. Strata failures or unravelling of coal, can occur in between support elements, and therefore the idea of a full coverage support that can be applied at the face and contain and control rock movement has widespread appeal. In addition to these benefits and specific to the support of coal mine roadways, a full coverage areal support that can be applied directly at the coal face could drastically improve the condition of roadway rib conditions.

Using conventional methods, full rib coverage with the use of rib bolts and mesh at the coal face is constrained heavily by the time it takes and the equipment in use. Minimum distances from the floor to
the lowest rib bolt can be as much as 1.2m for some continuous miner operations. This gap of unbolled, unmeshed rib can fail or spall, and in most cases is rebolted or shotcreted as part of a remedial support process to minimise personnel exposure and improve roadway conditions, as shown in Figure 3. If this lower rib is unstable, this can also cause unravelling or destabilisation of the rib above as the lower rib frets away, eventually requiring additional support to a much larger portion of the rib than just the initially unsupported section. TSL also has the advantage of being able to be sprayed into areas that may be awkward or hard to reach with bolts and mesh due to spall, geological structure or cavities.

Figure 3: A rib with intersecting structures and roof cavity showing coverage provided by bolts and mesh.

TSL roof to floor has the potential to eliminate any lower rib exposure for personnel and deterioration of the roadways serviceability, thus reducing the requirement for remedial support. Developing a specific suite of material testing criteria so that a design method can be followed for the application of TSLs as a primary rib support, and with a potential for the future as a primary roof support, will be critical for the benefits of this product to be usable.

Automation of TSL application

Manual handling is one of the major impediments to achieving Zero Harm in the mining industry. The ability to automate the application of spray-on support is an enabler to full automation of development operations. The continued success of automated processes centre on having the ability to quickly and easily provide a way to support the rock face, without requiring people to do the task.

Success and advances in technology in recent years of surface to seam outfits, delivering products for strata support, ventilation control and general mine maintenance could further reduce the required handling of a spray lined. This type of delivery system could provide a unique opportunity to be able to deliver a spray support from the surface directly to a freshly cut coal face within minutes.

Ease of application

Other immediate benefits of a successful TSL delivery system at the coal face include TSL’s potential to be sprayed in outbye areas, particularly difficult to access areas including belt roads, chambers and tailgate roadways. A surface delivery system that can provide spray on areal coverage support would offer solutions to many areas of mines that require remedial support in difficult to access areas. The erection, mobilisation, and access to scaffolding for the bolting re-support of these locations is often prohibitive. Figure 4 from Jjuuko and Kalumba, 2014 seeks to demonstrate the logistical differences between spraying TSL compared to the equipment required to spray shotcrete. Thicker linings such as shotcrete also have suitability issues in areas with minimal space such as alongside a belt road and still require significant room to manoeuvre their application equipment.
Investigations into the application time may also reveal productivity gains with the widespread use of TSLs compared to shotcrete or even meshing and bolting systems.

![Figure 4: Photos of TSL application and shotcrete (Jjuuko and Kalumba, 2014)](image)

**BRIDGING THE GAP FOR MAKING TSL MORE USEABLE**

The role of roadway support in an underground coal mine is to achieve an acceptable level of risk to coal mine workers while excavating roadways for mine access. Designing required support to maintain stability and ensure roadway serviceability and personnel safety is a multi-faceted process. Support design must be specific to an application, incorporate the full suite of available geotechnical information and local anecdotal evidence-to-hand.

Support elements or hardware undergoes a rigorous testing and quality analysis process before being included into accepted design principles at each mine site. Each type of hardware, such as roof or rib bolts, bolt plates, cable bolts and plates, standing support elements or mesh type are included in the design process and must meet minimum technical requirements. Quality Assurance (QA) and Quality Control (QC) processes are employed by OEMs and mine operators to ensure that all support equipment being used performs to these requirements or the minimum specifications as stated by the OEM. For TSLs to be accepted as a trusted part of the support regime, further work needs to be done on understanding and measuring the support it will provide to an underground coal excavation. Yilmaz (2011), used the testing results of hundreds of samples of TSL and proposed a function for categorising the strength of a TSL based on its measured tensile strength. This is shown in Figure 5. Developments such as these are steps forward in establishing the usability or suitability of a TSL for various applications.

![Figure 5: Tensile strength categories (Yilmaz, 2010)](image)

Different support types play individual roles within the system to provide roadway stability. Bolts and cables provide active support which can clamp together rock to form beams for stability. Cables can also be installed to provide suspension type support. Mesh is installed as a skin control and provides a restraining force upon a rock face once the rock face mobilises or delaminates (i.e., passive support).
The mesh will offer a restraining force up to its strength limit and provide reduced rock dilation once the rock has mobilised to rest against the mesh and therefore reduce the area to allow further movement. Mesh also offers certain advantages such as supporting crushed, damaged or weak coal; however it cannot offer active resistance to movement until its strength is initiated by rock movement onto it. When considering the use of TSLs as part of the support system to replace mesh, the understanding of how mesh works, and further understanding of how TSLs work to provide support is critical. The two support types differ markedly in several key technical areas, which could form the fundamental basis for support choice and design in any given application.

In addition to the mechanical and technical properties of a support method (mesh or TSL), it is most probable that the way in which the support can be applied or installed will ultimately determine suitability for most operations. This is a critical aspect for consideration when designing a testing or monitoring program for TSLs. Certain 'mix and pump' products which are generally water-based polymer mixed with specialised cementitious powder are not practical to apply at the coal face. In this case certain variables such as rate of gain or curing time become less important to the application when compared to a product that meets the criteria to be able to be safely sprayed off the miner and used as a support element for advance mining. For example, the properties of shotcrete make it a suitable support product to be sprayed in outbye areas and assist with long term roadways stability. However due to the large logistical effort and machinery required to be utilised to spray shotcrete, its rate of curing and its slumping properties, shotcrete is not a support type that could be considered for spraying at the coal face and provide improved safety and production, whereas TSL should be considered.

The routine use of TSLs as part of a mine support strategy will need to be complimented by the overall operational strategy of the mine or mining company dedicated to the development of automation. Spray-on support that can be applied as part of a cutting cycle, such as bolts and mesh, represents a significant change to current work practices in Australian coal mines. The downstream benefits of a change like this can only be truly realised where the vision of an operation is aligned with research and development towards automated processes. The commitment to the development, research, and actual implementation of making a spray on support available at the coal face is large, but potentially yields considerable downstream economic and social benefits.

CONCLUSIONS

Determining an appropriate methodology for designing and measuring the performance of TSLs as a support to underground coal mine roadways is not an easy task. The effectiveness of the support mechanisms that can influence how effective a liner will be, as well as the relative involvement of each of these parameters in varying conditions requires further testing and analysis. Developments, in hard rock mining and civil applications, over the past 30 years in understanding and modelling of TSL behaviour and performance have improved the situation. However, the gaps in real world field testing and the development of a reputable and reliable design process remains an opportunity to be investigated and presented to underground coal operations as a valid support method.

REFERENCES


Tannant, D D, 2001. Thin spray on liners for underground rock support. 17th International Mining Congress and Exhibition of Turkey, 57-73. ISBN 975- 395-417-4
GEOTECHNICAL ASPECTS OF THE PIKE RIVER MINE DRIFT RECOVERY

Chris Lee¹, Stuart MacGregor² and Dinghy Pattinson³

ABSTRACT: The Pike River mine exploded on the 19 November 2010. Thirty-one (31) men were working underground at the time of the explosion and only two men were able to escape. The Pike River Recovery Agency was established in January 2018 to conduct a safe manned re-entry and recovery of the Pike River mine drift to gather evidence to better understand what happened in 2010.

SCT Operations Pty Ltd (SCT) have been engaged to assist with the management of strata control hazards as part of the planning and implementation phases of the drift recovery. Initial geotechnical assessments comprised review of available historical geological and geotechnical information to develop a geotechnical baseline report and hazard map to assist with future planning and risk management. A range of controls have been implemented to manage geotechnical risk to acceptable levels and to ensure that adequate levels of inspection, mapping, monitoring, assessment, and review are maintained at all stages of drift recovery. Additionally, 3D FLAC modelling, surface tunnelling simulations and field loading trials have been conducted to support proposed tunnelling through a Rocsil plug located at the top end of the drift to provide access the rock fall area which marks the end of the mandated drift recovery. Given most of the drift had not been physically inspected following the explosion a range of drillhole assessments comprising downhole camera and laser scanning was also conducted to improve understanding of the drift environment both prior to and during re-entry.

As part of operational implementation and continuous improvement processes modifications were made to both ground support systems and bolting equipment which significantly improved support cycle installation times. SCT also supplied real-time roof monitoring instrumentation to the site which supplies an almost continuous data feed to the mine control room for interpretation and automatic alerting if Trigger Action Response Plan (TARP) threshold levels are exceeded.

INTRODUCTION AND BACKGROUND

Pike River Mine is an underground coal mine, located 46 km north-northeast of Greymouth on the South Island’s West Coast of New Zealand (Figure 1). Access to the mine workings is through a single 2.3 km stone drift which is developed to the rise at an average grade of approximately 1 in 9. The tunnel was driven through metamorphic Gneiss basement rocks to a point where it intersects the approximately 600 m throw Hawera Fault at around 2100 m. Inbye the Hawera Fault the tunnel passes into geologically structured coal measures to intersect the Brunner Coal Seam near the top of the drift about the rockfall location.

An explosion at 3:44 pm on Friday 19 November 2010 resulted in the loss of twenty-nine (29) miners. Two men, who were in the access drift some distance from the mine workings, managed to escape.

On 24 November 2010, a second explosion occurred, and this suspended any consideration for re-entering the mine. A third explosion occurred at 3:39 pm on the 26 November and a fourth explosion occurred on the 28 November at 1:55 pm.

In January 2011, the mine was sealed by Mines Rescue personnel, and the recovery attempt abandoned. No one has entered and physically inspected the drift following the explosion except for mines rescue personnel who reached around 300 m as part of assessment and construction of the 170 m seal. A large rockfall, which occurred sometime following the first explosion, is located at the top end of the drift around 2260 m and prevents access into the mine workings.

¹ Senior Geotechnical Engineer, Strata Control Technology. Email: clee@sct.gs Tel: +64 27 2948960
² Managing Director, Strata Control Technology. Email: smacgregor@sct.gs Tel: +61 4 19 972 138
³ Chief Operating Officer, Pike River Recovery Agency. Email: Dinghy.Pattinson@pikeriverrecovery.govt.nz
The mine was sold by the receivers of Pike River Coal Ltd in 2012 to Solid Energy New Zealand Ltd (Solid Energy) with the Government agreeing to fund a re-entry project on the basis any plan was safe, technically feasible and economically viable. The detailed plan developed by Solid Energy was rejected on the basis that it did not satisfy those criteria and the mine was subsequently sealed at 30 m inbye the portal by Solid Energy in November 2016.

The Pike River Recovery Agency Te Kāhui Whakamana Rua Tekau mā Iwa (the PRRA) was established by the NZ Government as a stand-alone government department on 31 January 2018 to work in close partnership with the Pike River families and other key stakeholders to plan for decisions on the safe manned re-entry of the Pike River mine drift.

The PRRA strategic objective is to conduct a safe manned re-entry and recovery of the Pike River mine drift (access tunnel) to:

- Gather evidence to better understand what happened in 2010 with an eye to preventing future mining tragedies and promoting accountability for this mining tragedy;
- Give the Pike River families and victims overdue closure and peace of mind; and
- Recover remains where possible.

The PRRA works in close partnership with the Family Reference Group and their technical experts who together play a central role in the planning, decision making and implementation on the safe manned re-entry of the Pike River Mine drift.

The evidence gathering processes are directed by the NZ Police Investigation team who provide onsite oversight to all the forensic processes conducted in the drift and on surface as part of the staged drift recovery. The PRRA also operates in a publicly transparent and open fashion with rigorous assessment of risks and control measures associated with manned re-entry of the drift.

SCT Operations Pty Ltd (SCT) were engaged by the Agency to assist with the management of strata control hazards as part of the planning and implementation phases of the drift recovery process. This paper summarises the geotechnical aspects of the Pike River Mine drift recovery.

**INITIAL PLANNING WORKSHOPS**

Between April and October 2018, the PRRA consulted a wide range of technical advisors from the UK, Australia and New Zealand to develop and risk assess potential re-entry and recovery options for the Pike River Mine drift. The drift recovery comprised recovery of the drift up to a large rockfall located at around 2260 m as well as an additional approximately 600 m of side tunnels which formed the Pit Bottom in Stone (Figure 2). The workshops involved family group members, advisors, Worksafe NZ and independent reviewers who developed three distinct operational re-entry plans. The planning
process included rigorous assessment and integration of the forensic examination processes to be conducted at every stage of the drift recovery process.

![Figure 2: Pike River Mine workings and drift recovery extents](image)

The three options considered for safe drift re-entry included:

- **Option A** - Driving a short, small diameter access tunnel into the drift (near Pit Bottom in Stone) from a suitable location on the side of the drift to provide a second means of egress and ventilation circuit to ventilate the drift with fresh air.

- **Option B** - Single-entry option (using the existing drift) without a new access tunnel or large diameter borehole and using forced ventilation with alternative safety provisions such as a refuge chamber and facilities for refilling breathing apparatus.

- **Option C** - Drilling a large diameter borehole of 600mm or greater from above, and into, the drift to provide an emergency escapeway. The borehole would not meet the legal requirements of a second means of egress.

The PRRA assessed each option and preferred re-entry of the single-entry drift which was considered both technically feasible and safe. Relative to the other options it was also considered the simplest method and the preferred approach to ventilation and egress whilst the drift work was being conducted.

On 14 November 2018, the Minister for the PRRA formerly confirmed the Government’s intent to recover the Pike River Mine drift and confirmed Single Entry (Option B) as the preferred re-entry option. Risks associated with single entry drift recovery are managed through best practice risk management and planning processes.

As part of the single-entry drift recovery a forcing ventilation system was considered the safest and most effective option. This resulted in the drift acting as the mine return. In addition, and as part of more detailed planning, a Rocsil plug was remotely placed into the drift down a borehole at the top of the drift to assist with mine ventilation and to separate, via a physical barrier, the mine workings from the drift. The Rocsil plug was installed approximately 10 m outbye the rockfall. Due to the ventilation system adopted an exemption was sought from the mining regulations that specifies that an escapeway from the mine must be in an intake airway. Drift recovery efforts could not proceed past
the mines rescue seal located at 170m until the exemption was approved. The exemption was approved, following additional risk assessment and planning, in December 2019 for drift recovery up to the Rocsil plug. The 170m seal was breached in late December 2019 with drift recovery operations commencing in January 2020.

GEOTECHNICAL BASELINE REPORT

A range of geotechnical assessments have been conducted to assist the PRRA with safe and efficient drift recovery. Prior to drift recovery efforts commencing a factual Geotechnical Baseline Report (GBR) was compiled by review of readily available information. The GBR formed an important part of the workflow as there was uncertainty surrounding the actual ground conditions, stability of the drift and the integrity of the installed support elements. The GBR included review of construction records and shift reports including bolt installation audits, detailed geotechnical mapping records and rock mass classification data captured during drift development. The GBR was developed to reduce uncertainty, assist with more detailed planning and allow for the identification and management of several key strata control risks.

The key hazards identified as part of the GBR which required suitable management as part of the drift recovery included:

- The single-entry nature of the drift and the subsequent risk of entrapment due to rockfall.
- The very poor to extremely poor ground associated with faulted and geologically structured areas.
- Areas, particularly about mapped faults, that appear to be under supported for the rock conditions encountered.
- Potential for significant fire related damage to the roof (including potential for falls of ground) in the weaker Coal Measure Strata.
- Time dependant deterioration (i.e. corrosion) of black steel bolts, given the drift was constructed 14 years prior to the recovery, and reduced capacity of support elements including shotcrete.
- Areas of poorly installed support quality; resin mixing issues were commonly noted in various shift reports and, in the coal measure section of tunnel, highlighted several concerns in relation to both the quality of the installed support and deviation away from the recommended mining system, support design and tunnel monitoring plan.
- Areas of over dimension roadway noting the nominal design profile was rarely achieved due to over break in the gneiss and coal measure section of the tunnel.
- The potential for time dependant weathering of the rock mass particularly in areas of shallow cover (<50 m) which may provide for modes of ground failure associated with the blocky rock mass.
- Areas that exhibit large signs of actual or visual deformation resulting in a higher instability risk.

The GBR provided input for operational implementation of safe manned re-entry including the following controls; initial ground support design, design assistance and procurement of fit for purpose bolting rigs and bolting systems, the development of geotechnical principle hazard management plans, operating procedures, geotechnical Trigger Action Response Plans (TARPs) with included provision for bolt integrity testing, hazard plans and workforce training material. The information contained in the GBR was also summarised and included in the mines Permit to Recover documents for the various stages of drift recovery.

DRIFT DEVELOPMENT AND GEOTECHNICAL CONDITIONS IN THE DRIFT

Construction of the 2.3 km long Pike River drift commenced in September 2006 and was completed in late 2008. The drift was designed with a nominal width of 5.5 m and a nominal height of 4.5 m with an arched profile across the crown. Between chainage 1880m and 2035 m a series of approximately 600 m of additional tunnels were developed off the main drift to establish pump stations and coal
handling infrastructure - this area is known as the Pit Bottom in Stone. The tunnel profile in the pit bottom in stone varied to suit the installed infrastructure with resultant profiles ranging from 5.5 m to 8m wide and from 4.5 m to 11 m high. The drift was constructed almost entirely by conventional drill and blast methods using a twin boom jumbo and conventional explosives. Figure 3 shows the jumbo in the drift highlighting the typical tunnel profile, water inflows and the often-wet tunnelling conditions.

![Figure 3: Pike River drift during construction highlighting the ground conditions, water inflows and often-wet conditions](image)

The entire tunnel East (outbye) of the Hawera Fault has been excavated in Metasedimentary Gneiss from the portal to chainage 2098 m where the Hawera Fault was intersected in the drift (Figure 4). Field estimates of the Gneiss strength along the drift are typically moderately strong (50 MPa) to very strong (150 MPa). Some areas of low strength, weak to very weak rock is noted in the tunnel mapping records particularly around areas of faulting and shearing with the rock often described as broken and fragmented. Notably the Gneiss section of tunnel was impacted by 3 to 4 main joint sets, many containing clay infill, with numerous shear zones and a total of 11 faulted areas and associated fault gouge consisting of clay infill and occasionally altered rock fragments. The zones either side of the fault structures were noted as being broken, very blocky and heavily sheared.

![Figure 4: Geological section of Hawera Fault and surrounding geology showing drift location and the structured nature of the ground](image)

The Hawera Fault is a large 500 m to 600 m throw reverse fault that hades over the coal measure section of tunnel and is orientated at a near 45° angle to the alignment of the drift. At the Hawera Fault, Cretaceous Paparoa Coal Measure sediments (>100 m thickness) are found, juxtaposed
against older Palaeozoic rocks belonging to the Metamorphic Gneiss, due to reverse downward faulting of the Hawera Fault (Figure 4).

The Coal Measure rocks encountered towards the end of the drift are folded upwards due to drag on the Hawera Fault which overturned the Brunner Seam near outcrop. This deformation resulted in the section of tunnel between the Hawera Fault and first coal being driven in the Paparoa Coal Measure Strata underlying the Brunner Coal Seam. These rocks typically comprise very weak to moderately strong well interbedded sandstones, mudstones, carbonaceous bands, altered grit, conglomerate and coal. Given its proximity to the fault and deformed and structured nature the ground conditions were found to be exceptionally poor to poor and containing steep bedding, many slick joints, sheared zones and areas of clay alteration and associated gouge.

The overall rock quality encountered in the drift was of significantly poorer quality than anticipated from the pre-development surface mapping and Geological Report and required the installation of higher levels of support than expected to manage ground conditions. The rock mass classification and support classes for the drift as constructed are highlighted in Figure 5. Typically, the drift was supported with 6 x 2.1 m bolts at a 1.5 m spacing in Class IV rock with Class V rock incorporating up to 120 mm of fibre reinforced shotcrete and a 1.0 m bolt spacing. Figure 6 highlights some of the poorer ground conditions encountered during coal measure development.

![Figure 5: Pike Drift mapped Q values, faulted zones, and support classes, 0 to 2000m](image)

![Figure 6: Paparoa Coal Measures east of the Hawera Fault showing the typically weak and tectonically deformed nature of the ground](image)

**CONTROL OF GEOTECHNICAL RISK DURING DRIFT RECOVERY OPERATIONS**

The drift recovery process was advanced in stages not exceeding 20 m intervals to allow for adequate levels of ventilation, forensic inspection, and geotechnical assessment. Safety of the workers and underground environment was a priority throughout recovery and forensic inspections.
Given the uncertainties relating to the condition of the drift and support elements prior to re-entry the management of geotechnical risk during the drift recovery was an essential aspect and was achieved through multiple controls which included:

- Having fit for purpose strata support equipment hardware and support plans.
- Ensuring people are trained and competent noting that the PRRA was a new organisation that employed mine workers immediately prior to the re-entry who needed training in all aspects of the recovery processes and systems.
- The drift re-entry TARP which included provision for drift conditions, support condition and density, explosion related impacts comprising blast and heat impacts, a seismic response and incorporation of roof monitoring data.
- Regular drift inspections and assessment of the integrity of the existing support which included a geotechnical mapping sheet and bolt testing plan (pull testing and over-coring) being incorporated into the daily statutory shift reports.
- Development of a Strata Management Team who conducted monthly meetings to discuss and approve on any changes required to achieve safe drift recovery.
- Development of suitable controls to manage the risk the single-entry nature of the drift including entrapment which comprised CABA, laying air and communication pipes on the floor of the drift and advancing a refuge chamber and roof fall recovery gear in the operational zone.

GROUND SUPPORT DESIGN FOR DRIFT RE-ENTRY

Ground support design to support the re-entry processes was initially assessed from review of construction records compiled in the GBR including mapped rock mass classification data, bolt testing data and information contained on the engineers shift reports. Additionally, geotechnical information obtained from selected boreholes were reviewed as part of the coal measure support design assessments. Zones of higher relative geotechnical risk were established and noted on hazard maps based on assessment of ground conditions, the adequacy of the existing support design and quality of the installed support.

The installation of additional ground support during re-entry was recommended and was broadly determined and as follows:

- Prescribed areas for rock bolting on advance in all faulted areas inbye the 170 m seal where ground conditions were extremely poor to exceptionally poor and specific areas where the installed ground support quality appears poor based on review of shift information in the gneiss section of drift;
- Systematic re-support of all areas of drift from the Hawera Fault inbye in Coal Measures during drift recovery; and
- Following a Geotechnical TARP response and based on observation and monitoring of encountered geotechnical conditions and results of ground support testing in the drift during re-entry.

Gneiss Support Design

In the Gneiss sections of drift, the faulted areas are typically associated with poorer rock mass conditions than unstructured areas of drift and were considered zones of higher relative geotechnical risk which required specific assessment for support design. Review of the mapping data (Q-system) indicated that in the poorest areas associated with faulting (where Q ≤ 0.1) the drift was under-supported on the basis of the consistent use of Excavation Support Ratio (ESR) value of 1.6 used in the initial design. An ESR of 1.0 was used for design of re-entry support in the faulted areas, consistent with the Q-system recommendations but to also provide a lower risk profile as part of the single entry nature of the recovery, and to mitigate potential uncertainties relating to the installation quality of the existing support in the wet, weak and faulted ground noting that resin “mixing and taking” issues were noted on some shift reports.
Unwedge analysis was also conducted on each fault zone using mapping data that indicated the support design would provide adequate levels of safety for both static and dynamic loading cases. The initial ground support design assessment comprised 6 x 3 m long grouted CT bolts at a 1.0 m spacing which was later optimised as part of continuous improvement processes following underground inspection, more detailed review of site specific conditions and various bolting trials. Subsequently recommendations were made for prescribed and systematic support to all faulted areas in the drift with 5 x 2.7m long resin encapsulated Posimix bolts at a 1.2 m to 1.3 m spacing dependant on site conditions.

**Coal Measure Support Design**

The coal measure section of drift contains very weak to moderately strong inter-bedded sediments that have been structurally deformed and adversely impacted by the Hawera Fault. Conditions both through and inbye the fault were mapped as poor to exceptionally poor with low to very low RMR values and estimated CMRR values relative to most coal mines. Existing support design in Coal Measures was based on an empirical system which comprised the Q Index classification system with verification using the RMR system. Following review of construction data it appeared that the coal measure section of tunnel had not been, in many locations, supported to the design standards, and it was difficult to accurately comment on the adequacy of the installed support given the lack of any quality roof monitoring and mapping data. Given the exceptionally poor conditions benchmarking against other sites was considered difficult and potentially extended empirically based design tools outside their intended usage domains. That said, SCT adopted a first principles design approach to coal measure support design coupled with operational and support design experience in reasonably similar conditions at the nearby Spring Creek Mine. This included assuming a 5m high softened zone exists in the roof strata, consistent with the estimated roof fall height (the roof fall is estimated to be approximately 5 m above the tunnel roof as observed in drillhole PRDH50) and dead weight loading assumptions estimated as 75 t/m which were also adopted for standing support design for areas inbye the Rocsil plug.

A support plan was developed and incorporated 8 m long high capacity (60 t) pre-tensioned cable bolts on a 2:1 pattern at a 1.0 m spacing. Grouted cables were expected to provide better levels of support in the weak, broken ground and are likely extend above the likely height of roof softening. Grouted cable bolts were also expected to avoid most of the potential performance issues commonly associated with resin-based primary support systems in weaker clay rich ground and the quality of installation (i.e. pre-tension and grout quality and grout return) could be easily assessed and noted on bolt audit sheets. The installation of additional primary support for the roof and ribs was not prescribed in revised coal measure support plans. This was based on operational experience that showed adequate levels of skin control were frequently being provided by the existing primary support elements and shotcrete where installed. Installation of additional primary support is installed as required following inspection and assessment of conditions and linked to a revised Coal Measure Geotechnical TARP.

**BOLTING SYSTEMS AND CONTINOUS IMPROVEMENT PROCESSES**

Two bolting rigs were procured by the PRRA for the drift recovery:

- **ALFABs Boar Bolter** - a hydraulic powered (via LHD) QDS platform rotary bolting rig without a Temporary Roof Support (TRS); and
- **Clarke Drifter** - an air over hydraulic powered crawler type bolting rig with rotary percussive drilling capabilities and a TRS fitted.

The rigs were specifically configured for Pike River Mine based on the support types to be installed in the tunnel, variation in rock types along the drift and the drift geometry which dictated the operating envelope for bolting machines. No power was installed in the drift as part of the recovery which limited gear selection to hydraulic (LHD powered) and air powered machines only. The rigs were designed to bolt 90% of the drift's length noting that drift height ranged from approximately 4.5 m to 6.0 m based on as-built construction drawings.

The Clarke Rig was initially envisaged to be the prime bolting machine in the hard rock sections of tunnel and was fitted with a rotary percussive drifter to improve drilling performance and increase the
durability of drilling consumables in the hard and aggressive rock conditions. The ALFABs was envisaged to be the prime bolting unit in the softer coal measure rock with typically smaller diameter rotary drilling capabilities.

It quickly became apparent that less than adequate air pressures being available in the drift were impacting the Clarke Rigs drilling performance. In addition, the specified mast length made operating the rig difficult as the tunnel opening was typically a lot smaller than that shown on the as built construction drawings as up to 600 mm of gravel had been placed on the floor. This resulted in the initial hard rock section of drift being supported with the rotary Boar Bolter which proved time consuming and hard on drilling consumables. As bolting was one of the key elements in the drift recovery process a solution was needed to improve both drilling and bolting cycle times. After some onsite optimisation work comprising different drilling configurations cost estimates and design solutions were sought from ALFABs to replace the rotary drill head on the Boar Bolter with a percussive drifter unit. At the same time various resin based bolt types were trialled as the grouted CT bolts being used at that time were adversely impacting the water quality leaving the mine which required additional treatment at the portal prior to being discharged into the Pike Stream which now forms part of the Paparoa National Park. Resin was also now preferred to grout as it reduced the potential for contamination of any forensic evidence which was a key aspect and consideration at all stages of the recovery work.

A Montabert HC25 hydraulic drifter was recommended for the ALFABs drill head replacement which, along with a resin based Posimix bolting system, delivered the following key benefits:

- Faster drilling and bolt installation times (no requirement for post grouting, smaller hole diameter and faster drilling rates);
- Better, more ergonomic work platform for operators (elevated work platform);
- Standardise hard rock bolting consumables and processes across the site; and
- Deliver cost savings by using fit for purpose hard rock drilling consumables.

Bolt installations were routinely measured during support cycles in the drift. In the hard rock section of drift, a total of 844 bolts were installed as part of the drift recovery; 569 bolts were prescribed with the balance (275) installed as a TARP response typically into mapped shear zones, areas of blocky and spalling rib and at other locations where the quality of the existing installed support was proven as less than adequate via testing (pull testing) and observation.

Notably a 400% reduction in bolting time was observed in prescribed areas following the changes to the bolting systems. This result was largely contributed to the bolting plant and bolt type modifications and to a lesser degree improved operator experience and familiarity on the bolting rig noting that some of the operators had no or limited experience with this type of bolting plant prior to the project commencing. The rotary drill head was placed back on the ALFABs once the Coal Measures were reached and a head replacement cable feeder fitted to assist with safe and efficient installation of 62 t megabolts up to the Rocsil plug.

REAL TIME MONITORING AND ALERTING

A roofAlert remote Tell-Tale monitoring system (Figure 7) was installed in the drift as part of the re-entry to continuously monitor roof movements. A total of thirty-two (32) instruments were installed during the re-entry advance at approximately 70 m centres or located to monitor higher risk areas including faulted zones and larger excavations. The Tell-Tales provided both automatic and visual read out that could be monitored manually if power supply were disrupted.
Given the nature of the drift recovery conservative trigger levels were established. The wired system comprised a backbone cable and junction boxes at each Tell-Tale which sent an almost continuous data feed (Tell-Tales were read every 15mins) back to standalone computers set up in the control room. The computers in the control room were configured to send out email and text messages if trigger levels were exceeded and to sound an audible alarm that would allow suitable control room response. A remote Tell-Tale was also wired into the system on surface that could be manually “displaced” to allow weekly testing of automatic alerting as part of the quality assurance processes. The continuous remote monitoring capability was also an important aspect of the sites COVID-19 response when the tunnel could not be physically inspected during the extended lockdown period. Coupled with the sites remote gas monitoring systems it provided almost uninterrupted underground monitoring data and confidence for the safe restart of drift recovery operations and the end of the extended lockdown period.

ROCSIL TUNNEL TRIALS AND GEOTECHNICAL ASSESSMENTS

An approximately 10m thick Rocsil plug was installed into the drift remotely down drillhole PRDH48 some 10 m outbye the roof fall (Figure 8). Access to the fall requires breaching the plug, wearing breathing apparatus in an irrespirable atmosphere, with a small tunnel of approximately 1.0 m wide by 1.8 m high excavated using hand tools. Geotechnical assessment formed part of the planning processes for the seal breaching activities and consisted of surface tunnelling simulations, 3D FLAC modelling and field loading trials.

Figure 7: roofAlert Tell-Tale monitoring system

![RoofAlert Tell-Tale monitoring system](image)

Figure 8: Cross section about Rocsil plug and roof fall locations

![Cross section about Rocsil plug and roof fall locations](image)
Rocsil Tunnelling

As part of small tunnelling field simulations, a small tunnel was excavated through Rocsil that had been pumped into a 6 m (20 ft) shipping container on surface between 12 and 13 March 2020. The tunnelling trial was conducted to simulate the proposed excavation geometry and methodology to be used underground during the Rocsil plug breach. This involved two workers using BG4 breathing apparatus to advance a small tunnel of 1.0 m width and 1.8 m height with an arched roof profile using hand tools. The trial geometry is the planned excavation size to be used when tunnelling through the Rocsil plug. The Rocsil material was easily excavated using a hand saw, pelican pick and spade with excavated material initially loaded into a skip and subsequently wool bags to simulate the proposed handling and storage methods underground. The approximately 6 m long Rocsil tunnel was fully excavated quickly, in around 4hrs, given the very weak material strength (average 60 kPa), easy cuttability and very low density (37 kg/m3) which made for efficient material handling. A bulking factor of around 1.5 was estimated from the trial. Figure 9 shows the tunnel being excavated through the shipping container and the tunnel at completion.

Field Scale Load Testing and Validation of 3D FLAC Modelling

A field scale loading test was completed over the tunnel cut through the Rocsil foam in the shipping container on 3rd June 2020. The field test was, in part, used to validate a 3D FLAC modelling assessment and to better assist with an understanding of Rocsil foam deformation characteristics about the tunnel opening under various loading conditions. Information collected during the trial was also used for development of TARPs to enable adequate response to changing conditions during Rocsil breaching operations.

Loading of the ~600 mm thick tunnel roof septum was conducted using a thick 750 mm diameter plate and calibrated hydraulic ram fitted to an engineered support frame (Figure 11). Loading of the roof septum was completed incrementally until complete failure occurred with measurements of both load and displacements captured during the testing.
The Rocsil foam septum above the tunnel was able to withstand peak loads of 80.6 kPa equal to 8.2 t/m² or 3.1m of rock head (dead weight loading) at around 15mm convergence. Notably the load – displacement graphs show the tunnel roof septum was able to sustain relatively high residual loads, following peak load being obtained, for reasonably large displacements. High residual loads of approximately 72.5 kPa were maintained to 55mm of roof convergence. Following this the load (strength) of the material dropped off very quickly with high angle shears observed on the rib lines and a “loss of structural integrity” noted during the test. The results suggest that the Rocsil material is able to resist load post peak strength for relatively high levels of displacement but once critical levels of stability are reached (as evident by the development of high angle shear failures) strength is rapidly lost and the load carrying capacity or the Rocsil roof septum is substantially reduced (Figure 11).
Importantly the Rocsil material was noted to deform in a behaviour consistent with expectation and the 3D modelling results and provided confidence that the in-situ behaviour of the Rocsil plug would behave in a reasonably predictable fashion which allowed for the development and implementation of TARPs with conservative trigger levels. The TARP included deformation aspects established from the field test comprising displacement (roof convergence), visual deterioration (tension / radial cracks and shear failures) and noise as highlighted in Figure 12.

Qualitatively, the deformation observed in the field trial was markedly aligned with anticipated deformation modes simulated by the 3D FLAC Pike River Rocsil Tunnel model. This was identified through similarities indicating an onset of high angle extensional shear surfaces along the rib lines coinciding with maximum support load prior to ultimate failure being reached. In addition, back analysis of field test results indicated material property strength values that were comparable to those used in the modelling which were originally derived from a relationship of the laboratory UCS testing results of Rocsil cube samples obtained from the shipping container.

**Downhole Camera and Laser Scanner Assessments**

Prior to breaching the Rocsil plug with the small tunnel a downhole camera and C-ALS scanner was placed down PRDH51 as the drift recovery approached the Rocsil plug. This information was used to assist with more detailed planning relating to better understanding the drift geometry for standing support placement, the assessment of ground conditions, including the potential for any time dependant deterioration, about the rockfall area. This aspect of the drift recovery was considered a high risk area for the recovery operations given the proximity of the rockfall and potential for further instability and needed a high level of detailed planning given the crews would be working in breathing apparatus in a low oxygen (~2%) atmosphere to reach the rockfall.

**SUMMARY**

The Pike River drift recovery process has been a complex and challenging task that has required detailed preparation and risk assessment at all stages of planning and execution. The main purpose of the drift recovery was to gather forensic evidence to assist with the police investigation. The forensic aspects have required careful consideration at all stages of planning and execution.

Development of a factual Geotechnical Baseline Report assisted with detailed planning and allowed for the identification and management of several key strata control risks. The GBR was important as there was uncertainty relating to both drift and support conditions and the drift had largely not been physically inspected following the explosion. The GBR provided input for operational implementation of safe manned re-entry including; initial ground support design, design assistance and procurement of fit for purpose bolting rigs and bolting systems and the development of geotechnical principle hazard management plans, operating procedures, geotechnical TARPs, hazard plans and workforce training material. Routine geotechnical assessment, mapping, monitoring and a bolt testing regime also formed part of the statutory reporting during the staged recovery process to ensure operations progressed safely.

Several geotechnical efficiencies were implemented following operational experience and inspection of conditions. This included optimising bolting systems which delivered significant improvements to bolting cycle times whilst minimising potential for environmental and forensic contamination.

In addition, surface tunnelling simulations and field load testing were conducted on Rocsil foam that was placed into a shipping container to assist with Rocsil seal breaching activities. This was supported by 3D FLAC modelling of the proposed Rocsil tunnel breach which was validated by the field load testing. Boreholes intersecting the drift were used to obtain information, and to inform decision making, via downhole camera inspections and 3D laser scanning technology.
DEVELOPING AN INNOVATIVE PROTECTIVE STRUCTURE ON CONTINUOUS MINERS AGAINST COAL BURST HAZARDS

Ting Ren¹, Alex Remennikov², Xiaohan Yang³, Dulara Kalubadanage⁴ and Peter Holt⁵

ABSTRACT: As mining progresses into deep ore deposits in Australia, geo-hazards such as coal burst and outbursts are becoming a major concern for mine workers. The occurrence of these hazards involved the ejection of coal lumps and sometimes large volumes of hazardous gases such as methane and carbon dioxide. Whilst it is extremely important to de-stress and de-gas the seam and adjacent strata before roadway development, and install competent support systems such as steel mesh and bolt, the last line of protection will be the installation of a protective canopy on the Continuous Miner to shield mine workers from these deadly dynamic impact of coal and rock resulting from a burst or outburst. The aim of this paper is to introduce the design, manufacture, and testing of an innovative protection structure on Continuous Miner in underground coal mines.

INTRODUCTION

Underground extraction of coal deposits involves the development of a network of roadways (gateroads) and longwall faces (or room and pillar). In Australia, the development of these roadways is typically achieved by the use of Continuous Miners (CM) in conjunction with other transport and roof support equipment. Figure 1a shows a typical CM (Alpine Bolter Miner ABM 25) that is used in some Australian underground coal mines. As the mining overburden increases, more challenging gateroad development conditions are being encountered, these include high geo-stress loading and hazardous gases (methane and carbon dioxide). Gas drainage using boreholes has to be conducted before a roadway can be developed using CM to avoid the hazards of gas/coal outbursts. Recently a major rib/sidewall pressure burst occurred in a longwall development roadway during mining operations at a coal mine in NSW, resulting in the tragic death of two workers on the CM. Figure 1b shows the gateroad heading after the burst incident. The burst coal can be seen on the left of the miner, where the installed mesh is displaced outward, beneath the ventilation ducting. The two men were engulfed by material ejected from the ribline during the pressure burst and died.

![Figure 1: (a) Continuous miner (ABM25) used for underground roadway development and (b) the site conditions following the pressure burst incident at a coal mine in NSW (Bruce & Jim 2017)](image)

Whilst extensive investigations and studies have been conducted following the incidents to better understand the circumstances of the incident, the industry is interested in developing a protective system that can be installed on the CM as the last line of defence should such an incident occur and other mitigation measures being compromised. The design of such a protective system needs to better

¹ A/Professor, University of Wollongong. Email: tren@uow.edu.au Tel: +61 2 4221 4186
² Professor, University of Wollongong. Email: alexrem@uow.edu.au Tel: +61 2 4221 5574
³ PhD Student, University of Wollongong. Email: xy987@uowmail.edu.au Tel: +61 4 8455 7784
⁴ PhD Student, University of Wollongong. Email: dmk824@uowmail.edu.au Tel: +61 4 2133 3228
⁵ Managing Director, Ironclad Mining Machinery. Email: pholt@stratalinings.com.au
understand the likely dynamic loads resulting from such incidents and correspondingly develop an innovative structure that would absorb and withstand such impact to protect the workers on the CM.

This paper presents an innovative design of a protective system for Continuous Miners capable of withstanding high-velocity impacts of large coal fragments. The protective system has a modular structure of individual protective panels and the frame elements. The effectiveness of the developed protective system has been validated via an extensive experimental program of testing of the protective panels and the full protective system assembly against large impact loads replicating high-speed coal fragments.

DYNAMIC HAZARDS

Dynamic hazards such as coal burst and outbursts can cause a great threat to mining safety as the dynamic hazards generally are associated with massive energy release and high-velocity coal ejection. Understanding the dynamic loads (forces and energies) for such incidents is a highly technical and specialist area, which is a key input into the design of the protective structure to be installed on the CM as the last line of protection. It has been mentioned by many researchers that the ejected coal blocks can reach 20 m/s high velocity (Frith et al. 2020). To determine the impact force and energy carried by ejected high-velocity coal blocks during dynamic hazards, the drop coal test (see Figure 2a) was conducted in the laboratory. As shown in Figure 2a, coal blocks are dropped from 6 m height and the load is measured by a load sensor. The high-speed camera (HSC), which can take 4000 pictures per second, is placed in front of the drop coal system to film the drop process of coal blocks. Figure 2b shows the images captured by the HSC during the drop coal test.

Figure 2: (a) Schematic diagram of the drop coal test setup (b) the impact of a coal block during the test captured by high-speed camera

\[ P_{\text{max}} = 1.765 \times G^{3/5} \times \lambda^{2/5} \times R^{1/5} \times H^{3/5} \]

The impact load caused by falling coal can be calculated according to equation 1 (Labiouse et al. 1996), where \( P_{\text{max}} \) is the impact force in kN, \( G \) is the weight in kN, \( R \) is the equivalent radius of dropped coal in m, \( \lambda \) is the Lame’s constant which is related to the material of coal and steel cushion in kN/m², and \( H \) is the drop height in m. The energy carried by coal can be determined by the work-energy theorem. The correlation equation between drop weight and impact load has been established based on test results. It has been determined that the impact force and energy caused by 175 kg (0.7 m diameter) coal block with 20 m/s velocity are 406 kN and 25658 J, respectively.

DESIGN OF CM PROTECTION SYSTEM

The CM protection system mounted on continuous miner (CM) is an effective measure to control coal burst and outburst hazards. The design and material of the protection system are important for providing efficient energy absorption capacity and impact load resistance. To develop an innovative and efficient protection system on CM for coal burst control, a honeycomb panel was designed and tested in the laboratory. The honeycomb panel includes two parts: 0.7 mm thickness core and 5 mm thickness surface plates. A core is cut into pieces by water-jet cutting and assembled according to a specific arrangement (see Figure 3a). Then the two steel faceplates are welded with the core part (see Figure 3b).
Figure 3: (a) Core of the honeycomb panel (b) Top surface of the honeycomb panel

The CM protection system consists of two main components: honeycomb panels and steel frame. The honeycomb panels are slid into a steel frame and connected to the frame members using bolted and welded connections. The frame can be installed on both sides of CM. The total height of protective structures is 1.5 m, which can provide enough refuge space for operators when dynamic hazards occur.

TESTING OF CM PROTECTION SYSTEM

The preliminary numerical models were developed for the honeycomb panel and the full protection system (PS) with honeycomb panels to optimise the thicknesses of individual components and maximise the protection levels. However, it is important to carry out full-scale experiments on the developed protection system to calibrate and validate the performance of the PS.

The drop hammer impact tests were performed for individual honeycomb panels (HP) and full PS with HPs. HP was mounted on a specially designed steel frame with fully fixed support conditions and tested by a drop hammer system with 6.40 kN weight and 4 m dropping height as shown in Figure 4. The falling hammer is gravitationally accelerated to over 8 m/s with 25,088 J impact energy. The designed HP was able to resist the impact load by absorbing energy via plastic deformation and crushing of the honeycomb core as shown in Figure 5.

Figure 4: (a) Camera and data logger outside the drop hammer system (b) drop hammer test set-up for HP (c) schematic diagram of test set-up of HP with displacement sensor
Figure 5: Experimental results for drop hammer impact test for HP: (a) deformed HP and (b) centre displacement history and impact load

Even though the PS is intended to be installed as a vertical barrier on the CMs, the fabricated prototype PS was fixed in a horizontal direction in the drop hammer facility for impact testing. The experiments were performed to evaluate the PS’s performance under the impact of falling 600 kg mass from 1 m and 3 m heights onto the central HP and top HP, respectively. The PS was instrumented with two high-speed laser displacement sensors and two strain gauges as shown in Figure 6.

Figure 6: Experimental set-up for full-scale testing of PS and instrumentation

The PS successfully resisted the applied impact loads with reasonable deformations in both tests. The HP panel initially absorbed energy via core crushing and plastic deformation and then the square hollow sections (SHS) and steel straps in the PS absorbed most of the impact energy via plastic deformation. Figure 7 shows the experimental results of the 1 m drop test on the PS. The far-end of the SHS showed around 26 mm of permanent displacement after the test.

The impact test on the top HP of PS can be considered as the worst-case scenario as it could cause more damage to the PC. Interestingly, the PS resisted the impact load from 3 m height very well as shown in Figure 8. The peak dynamic displacement of the centre of the top HP was 170 mm and permanent displacement was around 95 mm.
Figure 7: Experimental results for 1 m drop test on PS: (a) deformed PS; (b) displacement histories of the PS; (c) impact load and (d) strains of the PS.

Figure 8: Experimental results for the 3 m drop test on PS: (a) deformed PS; (b) displacement history of the PS; (c) impact load and (d) strains of the PS.
NUMERICAL MODELLING OF CM PROTECTION SYSTEM

The numerical models were developed for the drop tests of individual HPs and drop tests of full PS using LS-DYNA R10.0 finite element code. The HP was modelled using Belytschko-Tsay four-node shell elements while the clamping plates of the supporting structure were modelled using eight-node constant stress solid elements as a rigid structure. The drop hammer falling process was omitted and instead the velocity of the drop hammer just before the impact was defined. 2-mm size shell elements were found to be balanced the results and the simulation time through a mesh sensitivity study. Both top and bottom clamping plates were fully fixed for all the degrees of freedom to provide fixed boundary conditions to HP similar to experiments. Mechanical testing was performed for dog-bone specimens that were cut from 5 mm and 0.7 mm steel sheets. Steel material behaviour was modelled using *MAT_PIECEWISE_LINEAR_PLASTICITY constitutive model in LS-DYNA and was fed with experimental mechanical properties. The strain rate effect was not considered for numerical models as it was found that the numerical results well agreed with experimental results without strain rate effects. The comparison of numerical results with experimental results for the 4 m drop hammer impact test is shown in Figure 9. It can be observed that HP showed large plastic deformation and simulation well captured it. However, the experimental HP showed asymmetric nature of deformation but numerical deformed symmetrically. The numerical model reasonably captured the displacement histories and impact load histories.

![Figure 9: Comparison of numerical model results with experimental results for HP testing at 4 m height: (a) deformed HP panel; (b) centre displacement histories and (c) impact load histories](image)

The SHS and steel straps and other remaining steel flat plates in the PS were modelled using eight-node constant stress solid elements while HPs were modelled using 2 mm size shell elements in a similar method as explained earlier. The solid element sizes of 4 mm and 5 mm were used. The welded connections and bolted connections were modelled using *CONTACT_TIED_SURFACE_TO_SURFACE contact algorithms, while *CONTACT_AUTOMATIC_SURFACE_TO_SURFACE contact algorithms were used to define the friction contacts between the HP panel and the frame.

The drop hammer falling process was omitted as described earlier. Strain rate effects were not considered for the PS. Figure 10a compares the displacement histories of the top central HP on the PS under the impact of 600 kg from 3 m height onto the top HP. It can be observed that the numerical
model underpredicted the central displacements. However, it is important to note that the PS in the experiment already had a permanent deformation of about 0.05 m from the previous impact tests which could increase the peak response of PS in this test.

The numerical model predicted the impact load reasonably close to the experimental impact load curve as shown in Figure 10b. However, there is a slight deviation of the response timing. The numerical model will be further calibrated using additional experimental results in the future. Figure 11 shows the PS’s deformation as predicted by the numerical model in comparison with experiments. Effective stress contours are also shown in Figure 11b.

![Figure 10](image1.png)

**Figure 10:** Comparison of numerical and experimental results for drop test on top HP of the PS: (a) central displacement histories and (b) impact load histories

![Figure 11](image2.png)

**Figure 11:** Comparison numerical model predicted deformation with experiment: (a) Experimental deformed shape of the PS and (b) Numerical model predicted deformation

**CONCLUSIONS**

An innovative protective structure on Continuous Miner against coal burst hazards is developed and investigated in this paper. The following conclusions can be drawn:

(1) The impact force and energy caused by ejected coal block with 20 m/s velocity can reach 406 kN and 25658 J, respectively.
(2) Both the numerical and experimental results have verified that the individual honeycomb protective panels can resist up to 500 kN impact load and 25,088 J impact energy.

(3) The tests and numerical modelling of the Continuous Miner Protective System (PS) have demonstrated that the PS is capable of absorbing the energy of multiple large impacts with controlled deformations and limited damage to the main energy-absorbing components of the protective system.

ACKNOWLEDGMENTS

This research is funded by the Australian Research Council Industrial Transformation Research Hub for Nanoscience-based Construction Material Manufacturing (IH150100006). The authors acknowledge all technical staff at the Faculty of EIS, University of Wollongong, involved in this research, in particular Mr. Alan Grant, for their support during the manufacture and testing stages. The authors also thankful to Mr Evan Brown from the Facility for Intelligent Fabrication at UOW for the development of an automated welding procedure of the honeycomb protective panels.

REFERENCES


USING REMOTE READING INSTRUMENTATION TO IMPROVE SAFETY, PRODUCTIVITY, AND SUPPORT DESIGN IN UNDERGROUND COAL MINES

Samantha Watson¹, Nathan Owen² and Claire Morton³

ABSTRACT: Accurate and timely monitoring of strata stability in underground excavations is a critical activity that forms part of the fundamental daily mine safety processes and is mandated by law to be carried out in all operations. Coal mine strata monitoring includes the use of instrumentation and observations carried out by personnel. Mechanical tell-tales are an instrument widely used throughout the coal mining industry and are used to measure displacement of roof rock horizons. They comprise of either 2 or 4 reading indicators that are connected by stainless steel wires to anchors secured up a purposely drilled hole and require personnel to read them individually and record displacements manually.

Prompted by the desire to improve safety and provide a system that could offer continuous monitoring of areas within the mine, without the requirement for manual recording of tell-tale data, Moranbah North Mine (MNM) initiated installation of a remote reading monitoring scheme in 2012. The monitoring system is entirely automated, and the data is automatically collected and transferred to the surface via an optical fibre cable. Alarms are pre-set to defined triggers and alerts are sent out automatically if triggers are reached. The real time data can be accessed anywhere with available internet connections and log on details. Significant amounts of data, previously not accessible, has been recorded and collected since the initial remote reading system was installed. This has provided critical information for support design work, remedial works, and development of accurate Trigger Action Response Plans (TARP). The data is increasingly being used for optimising mine support. Upgrades to the remote reading technology in recent time mean that the available system uses cutting edge technology and can be manipulated to meet different applications.

All Anglo American underground metallurgical coal mines within Australia now employ Remote Reading Tell-Tales (RRTTs) in longwall gate roads. The system is also being adopted by numerous other coal mines in other mining groups both in Australia and Globally.

INTRODUCTION

Australia's coal mining industry accounts for 27 percent of total revenue for the Australian mining industry with longwall mining providing around 90 percent of Australia’s underground coal production (CSIRO, 2018). Longwall mines are characterised by development of roadways that constrain longwall panels, rapid retreat of longwall faces which extract all the coal in each longwall panel which then allows overburden strata to collapse into the resulting void. This means that the stress regime is constantly changing, and that the geotechnical environment can change within hours or even minutes (Guo et al. 2012).

The coal mining industry primarily uses manually read tell-tales as the main method of monitoring roof behaviour (Jayanthu et al). These instruments are often only read once per shift by underground officials, with readings recorded on paper sheets and sent to the surface to be entered into an online database. This process often limits the extent to which the system can act as a safety tool and as a means of assessing strata behaviour to determine the effectiveness of the roof support. With respect to safety, movement may go undetected due to the instruments not being continuously read. With respect to strata behaviour, the extent of the dynamic geotechnical environment ahead of the longwall and as the longwall approaches may not be captured as manual readings represent only a snapshot in time. MNM initially installed RRTTs and Remote Reading Strain Gauges (RRSG) in its surface to

¹ Geotechnical Engineer, Anglo American. Email: samantha.watson@angloamerican.com Tel: +61 7 4968 8846
² Managing Director, Nome Services. Email: nathan.owen@nomeservices.com.au Tel: +61 7 5648 1315
³ University of Queensland. Email: c.morton@uq.edu.au Tel: +61 7 2968 8846
coal seam inclined drifts to continuously monitor strata movement in these critical excavations. These initial installations were simplified as the drifts are fixed and non-gaseous. Realising that RRTTs had great potential to improve safety and data acquisition around longwall panels, MNM worked with the provider to develop a system that could be applied in roadways which were retreating and had the potential to contain flammable gas. The initial system designed was based around two-anchor tell-tales and was limited to approximately 40 instruments in a 4km long roadway.

The results from the initial system were extremely encouraging, providing a full picture of strata behaviour as the longwall approached. A significant safety benefit was a reduced requirement for physical examination of the tailgate roadway. This not only reduced the exposure of underground officials to an environment typically elevated in gas and dust, but in doing so increased daily production time by up to an hour - a significant production time gain at a longwall mine operation.

Following initial success, the system has been developed to improve reliability, accessibility and to integrate with other existing technology at MNM. The new system is based on two and four-anchor tell-tales that, in addition to the electronic interface, can be visually read by all underground personnel. Each instrument can be readily reset if displacement exceeds the tell-tale range and the system can support up to 150 instruments along 10km of roadway.

Figure 1: Installed RRTT in an underground Gypsum mine (courtesy NOME Services)

THE REAL TIME MONITORING SYSTEM

Real Time Monitoring Instrumentation

Individual RRTT instruments are simple to install and connect. They can be installed at the development face off the continuous miner in place of standard mechanical instruments. Once connected in the advancing development panel, a complete record of strata movement can be collected which enables primary development support and secondary longwall support to be assessed for effectiveness, which then in turn enables ground support systems to be optimised.

Data from the RRTTs is transmitted to software installed on a remote server on the surface. Alarms are set in the monitoring software to alert when any trigger levels for total movement or rate of movement are reached. The software then sends data to site Supervisory Control and Data Acquisition (SCADA) interfaces in real time. The SCADA interfaces are now also included in the sites digital transformation using underground tablets. This enables underground officials to receive alarm triggers relating to their panel in real time, allowing remedial measures to be put in place as necessary to manage safety risks and prevent operational delays e.g. longer cable support, standing support and pre-consolidation.

A variety of monitoring instruments are in use at MNM for remote and continuous monitoring of critical excavations in outbye areas and at the active coal faces.
Strain Gauges

HMA 3050 strain gauges with Campbell Scientific datalogger are fixed directly to installed steel sets and shotcrete lining in the Drifts. The strain gauges are a proven technology and have been used in the mining, civil and materials testing industry for many years. The device design is simple and robust, with no moving parts and can be monitored remotely or by the manual readout box. The devices are proven to be corrosion resistant, waterproof, dust proof and the strain induced by temperature change is corrected for automatically. The electric coil is detachable without damaging the gauge, providing a degree of flexibility in the event of cable damage.

The devices themselves are a vibrating wire strain gauge, mounted to the surface of the structural member (steel set or shotcrete lining), where any deformation of that substrate produces a change in the wire tension and corresponding change in its frequency of vibration. The frequency is measured through an electronic coil, connected through a signal cable to a data logging system and all measurements are made in terms of micro strain.

![Figure 2: Workings of strain gauge](image)

![Figure 3: Strain Gauge installed](image)

2 and 4 anchor Extensometry/ Tell-tales

The RockMonitor XR tell-tales are the latest version of the RRTT and are configured with both 2 and 4 measurement heights at MNM and currently the 4 x height RockMonitor design forms part of the active monitoring program in the mine. These tell-tales are being installed at the cutting coal face on continuous miner operations as part of the primary support process. The cable installation and connectivity is able to be achieved as part of the standard panel advancing. Because the newer
version of the tell-tales have a manual user face they can be used as mechanical tell-tales until they are connected to the remote system.

![Figure 4: User face of RRTT are able to be read manually](image)

Data Collection and Storage

Typically, the equipment set up for the powering and connecting the RRTT system is installed in a life of mine areas such as the outbye mains. A power and data enclosure containing an intrinsically safe power supply and associated equipment is installed alongside an intrinsically safe certified controller. The equipment requires a 110-240V power feed and an ethernet cable that runs to a network switch inside any belt starter of transformer that already has a communication backbone. This allows the communication link to run to a virtual machine or server on the surface of the operation where the user software, called CORE, is installed. The software then communicates directly to the controller underground and receives continuous data packets. Up to 10km of underground roadways can be monitored from a single controller.

![Figure 5: Rockmonitor Controller and Telltales](image)

Accessing and Storage of the Monitoring Data

Access to all instruments is quick and easy via plug and play connectivity. Users are created with the CORE software package and level of access can be assessed and provided based on the role of the individual. When SCADA exporting is being used any personnel on the surface or underground with access to a SCADA interface can visualize the system and access data trends/alerts in real time. Exporting data into Excel XLS or CSV is also available.

The data can be stored several ways and if required it can be easily secured on an offsite server for added security. On small systems not requiring connection in real time to the surface, a portable reader can be put into auto wake up mode and left underground connected to the main trunk line. It will wake up at predetermined intervals from 30 minutes up to 12 hours and take a log of every instrument reading along the line.
Figure 6: Monitoring Dashboard

BENEFITS AND APPLICATION OF CONTINUOUS REMOTE READING

Benefits of RRTT

Using remote monitoring systems at MNM has provided significant advantages and advances in monitoring of the strata across the mine. Visibility on the behaviour of critical excavations such as widened or high roadways has given feedback on support performance as well as provided opportunity for fast response times when needed. Some of the advantages of using the RRTT from a strata monitoring perspective at MNM are listed in Table 1.

Table 1: Strata Control Benefits of RRTT at MNM

<table>
<thead>
<tr>
<th>STRATA MONITORING BENEFITS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuous monitoring of strata displacement</td>
</tr>
<tr>
<td>Strata displacement measurement up to 150mm (without reset)</td>
</tr>
<tr>
<td>Two or four monitoring points (anchor heights) between 0.6m and 15m</td>
</tr>
<tr>
<td>Fully customizable alarm settings including rate of movement and absolute displacement</td>
</tr>
<tr>
<td>Automated notifications via software or e-mail for improved response time</td>
</tr>
<tr>
<td>Detailed monitoring and display of data using powerful application software</td>
</tr>
</tbody>
</table>

Logistically, the system is relatively simple to have installed with some features including; the ability to monitor up to 10km with a single system, flexibility configuration – with spur or daisy chain available, plug and play installation, integration with existing systems such as SCADA, and connection of up to 150 devices per controller.

Applications of RRTT

The RRTT system was initially introduced to MNM following a roof fall in the mine’s entry drift. The following recovery programme included the installation of support in both the P&E and Conveyor Drifts to a minimum Factor of Safety (FOS) value of >2. In addition to this support review, it was recommended that ongoing observations and measurements be undertaken in the drifts by way of a real-time monitoring program which monitors displacement trigger levels. Both tell-tales and strain gauges were installed throughout the drift to provide continuous monitoring along the length of both roadways. Figure 6 shows the location of the tell-tales installed along the drift, in cross section.
Due to the success and benefits of this installation, the system has since been applied throughout the mine in outbye critical excavations and active production areas. Some notable optimisation strategies and safety improvements have been executed due to the use of the system, including capturing movement exceeding the capability of manual telltales or other systems, real time convergence monitoring and constant awareness of conditions in restricted or no go zones.

- In Longwall 112 an integrated monitoring system using convergence pogos was installed in the tailgate roadway to record the continuous ground movement outbye of the retreating longwall face and better understand both the roof and floor movement. This was able to be done without requiring any personnel to access the tailgate, which typically represents a limited access area due to respirable dust exposure, elevated gas during cutting and poor roof and rib conditions.

- Faceline 111, an exceptionally wide roadway designed specifically for installation of large longwall equipment, experienced extended standing time which resulted in ground movement exceeding the design expectations. The installed RRTT monitoring system was used to closely monitor and allow the geotechnical engineers to respond accordingly and in a timely manner to conditions that were occurring but were not necessarily detectable by visual observations. Having the continuous visual on ground movement in this critical excavation provided the mine confidence and reassurance that operations could continue despite being delayed.

- Faceline 112 excavation support was designed using the experience of Faceline 111, which had recorded magnitudes of movement that could not have been obtained without the RRTT system as they exceeded the limitations on other available systems. The strata characterisation and experience from Faceline 111 indicated that the support in Faceline 112 should be denser and extend above the height of softening. This provided precedent during the support design process that benefited the overall success of Faceline 112.

FUTURE AND NEW TECHNOLOGY

SMART Connectivity

SMART junction boxes are currently being designed and certified to allow for automated disconnection of instruments during longwall retreat. This will further reduce the need to put personnel at risk when disconnecting devices and will allow for personnel to remain and analyse data from remote operating centres.

Technology Transfer

Currently there is collaboration between Bond University and NOME to incorporate machine learning in order to develop artificial intelligence techniques for the prediction of strata movement in underground mines. The current intention is for this system to be capable of forecasting future strata movement in real-time based on current and past movement data, as well as key geological and mine features. Primarily, the purpose of this module would be to improve safety and reduce downtime by predicting and pre-empting catastrophic strata movement before it occurs.
Increased Design & Response Accuracy

The work being done on machine learning and strata movement prediction will eventually allow the incorporation of predictive TARPS for proactive strata planning in critical roadways.

Personnel Safety Systems

Collision avoidance and Proximity detection is planned to be incorporated into as part of the suite of real time monitoring systems offered with the system. This will also include the means to alert work groups of trigger points and eliminate the exposure of personnel once these situations occur, improving safety by reducing risk to personnel.

CONCLUSION

Using remote monitoring systems at MNM has provided significant advantages and advances in monitoring of the strata across the mine. With continually improving technologies and advances in machine learning, the system has the potential to offer more safety benefits in addition to strata monitoring. As with the implementation of any new system to a mining operation, the successful onboarding requires a financial investment in addition to the resources to be allocated to ensure its success; personnel involvement at the implementation, maintenance and ongoing data analysis takes time and dedication to this success. MNM has invested the resources to integrate the system into its routine processes and routinely benefits from the ability to monitor remotely. With underground coal mining operations getting increasingly deeper and with more and more roadways that require continued serviceability, the option of remote monitoring can help improve support design, responses, and overall safety.

ACKNOWLEDGMENTS

We wish to thank Wesley Noble for consultation approving use of Moranbah North Mine data for the purpose of this paper.

REFERENCES


Jayanthu, S., Singh, T. N., Singh, D. P. (1998): A critical study of strata behaviour during extraction of pillars in a thick coal seam, Proceedings of 17th Int. Conf. on Ground Control in Mining, West Virginia University, 4-6th Aug’98.

A FIRST-PRINCIPLES CAUSATION HYPOTHESIS FOR PILLAR BURSTS IN UNDERGROUND COAL MINES

Russell Frith

ABSTRACT: A review of published literature reveals case histories whereby the entire periphery of a coal pillar has "burst" out as a single event during first workings, the associated energy and material release causing significant damage to adjacent roadways and any equipment/infrastructure located within said roadways. Such events are distinct from coal burst events during first workings such as that at the Austar Mine in 2014, or those linked to overburden bumps related to either horizontal stress-driven slip along major faults and/or thick, massive strata units in the overburden or floor of the coal seam. The paper considers as to how the necessary "unstable" conditions for a pillar burst event could conceivably be generated, based on established coal pillar mechanics, specific pillar loading conditions and the shear-restraint of horizontal planes according to both cohesion and friction. The associated hypothesis is applied to a published example to test its veracity.

The longer-term objective of this type of back-analysis is to provide a “cause and effect” list of geological, geotechnical and mine layout circumstances that can and indeed have resulted in entire coal pillar bursts during underground coal mining activities, being able to predict the likely propensity for such events prior to mining being a mandatory requirement in an effective prevention or consequence mitigation process.

INTRODUCTION

The rapid expulsion of coal material into mine workings is both a potentially destructive, in terms of mine safety and production, and relatively poorly understood phenomenon in underground coal mining. Coal industries from around the world, particularly those with deep mines, report such events on an infrequent basis, with the principles of fundamental cause and effect remaining elusive. As recently as 2017, Dr Chris Mark published a paper with the provocative title “Coal Bursts that Occur During Development: A Rock Mechanics Enigma” (Mark 2017) which clearly confirms the basis of the preceding opening statement. Within Mark 2017 a series of what are described as “pillar bursts” case histories are discussed whereby the entire periphery of one or more coal pillars has reportedly undergone bursting as a single event relatively soon after roadway development. Figure 1 from the Manalapan No.17 Mine in Kentucky indicates two such reported pillar bursts, as described by Newman (2002):

The bump impacted six pillars with the greatest damage centered in the belt entry, two breaks out by the active face. Roughly 3–4 m of the pillars on either side of the belt entry ailed. The belt entry two breaks out by the face was filled with fine coal in the center of the entry…The coal in the remaining portion of the pillar was separated from the roof creating a void space, approximately 3 m deep into the pillar.

The height of the void space was roughly 0.3–0.5 m at the edge of the remaining portion of the pillar, grading back to contact with the roof…..Although no roof falls were visible, wide spans in excess of 12 m resulted from the void space that formed between the top of the bumped pillars and the immediate roof.

It was stated that no unusual geological features were noted in the vicinity of the burst sites, Figure 2 showing roadway conditions following the pillar burst. Mark (2017) commented that the underlying workings are first workings with limited areas of extraction, and that the burst pillars in the overlying Kellioka Seam have back-analysed ARMPs Stability Factors (SF) in the order of 2, which would normally suggest a stable pillar system. Demonstrably then, another failure mechanism must be at work that has no direct link to traditional coal pillar design.
What is immediately obvious in Figure 1 is that the areas of burst pillars in the Kellioka Seam align almost exactly with remnant pillars in the underlying Harlan Seam workings. Furthermore, the burst pillars coincide with the cover depth being more than 457 m (1500'). The reported response of the mine was to impose mining restrictions above certain cover depths, the final upper limit being 300 m.

![Figure 1: Locations of Burst Events in the Manalapan No. 17 Mine (Black) relative to underlying Harlan seam workings (magenta and green) – Mark 2017](image1)

The reported pillar burst experience from the Kellioka Seam at the Manalapan No.17 Mine strongly suggests that the primary driver is excessive vertical stresses within what should otherwise be stable coal pillars. This leads to the inevitable question as to how stable coal pillars that are subjected to relatively low multi-seam vertical stress impacts by virtue of the nature of the underlying workings, can seemingly “explode” without significant warning? This is the main subject of this paper, the intention being to develop an initial hypothesis that might then be tested by others by reference to other case histories over time.

**RELEVANT PRECEDENT**

Frith, et al. (2019) described a first principles cause and effect model for the 2014 development coal burst at the Austar Mine in NSW, which in hindsight had several similar geological characteristics to
documented development bursts at the Sunnyside Mine in Utah. That model was directly linked to two geological features, namely very specific geological faulting just inbye the Austar burst site, and the presence of a planar, low friction horizontal plane within the coal seam, known as the Dosco Band, which marked the top of the burst coal section (see Figure 3).

Figure 3: Incident scene showing the left-hand side of continuous miner (NSW Department of Industry, 2015)

Without digressing into the technical detail, the three major drivers for the Austar development coal burst were assessed to be:

i. A localised increase in the major horizontal stress within the coal seam.

ii. A localised reduction to effectively zero of the minor horizontal stress within the coal seam.

iii. The presence of a low friction plane towards the top of the working section.

The effect of (i) and (ii) was taken to cause a significant and highly unusual imbalance in the horizontal stresses within the coal seam, such that the major horizontal stress was “unstable” due to a lack of confinement in the orthogonal or minor direction. The effect of (iii) was to eliminate the stabilising influence of the vertical stress, the inevitable result being a very rapid unloading of the major horizontal stress within the coal seam below the low friction plane and a resultant very high acceleration of the effected coal, which caused coal to be ejected rapidly into the mine roadway in the form of a “burst”.

The Austar incident and those reported from the Sunnyside Mine can be directly linked to a major geological faulting system, this being taken to be the local anomaly that brought about the specific ground stresses within the coal seam that theoretically allow coal bursts to occur if a suitable low friction plane also exists within the coal seam. In the case of the Manalapan No.17 Mine, no such major structures exist; hence the question posed is whether similarly unstable ground stresses can develop within the coal seam without the influence of a major geological structure?

**PRESENCE OF A LOW FRICTION PLANE**

Figure 2 demonstrates that the coal seam to stone roof contact in the area of pillar bursts at the Manalapan No.17 Mine is characterised by what appears to be a planar and smooth surface. Hence, the first requirement for a coal burst is present, the remainder of the paper focusing on horizontal stresses within the coal seam and how they might become critically unstable in this particular case.

**HORIZONTAL STRESSES WITHIN COAL**

A general model for the development of *in situ* horizontal stresses within coal measures was put forward by Nemcik, *et al.*, (2005) as follows:

\[ \sigma_H = \sigma_v(v/1-v) + E.TSF_H \]  

[1]
\[ \sigma_h = \sigma_v (v/1-v) + E \cdot \text{TSF}_h \]  
\[ \sigma_v = p.g.h \]  

where:
- \( \sigma_H \) = major horizontal stress
- \( \sigma_h \) = minor horizontal stress
- \( v \) = Poisson’s Ratio
- \( E \) = Young’s Modulus
- \( \text{TSF} \) = Tectonic Stress Factor (H = major and h = minor)
- \( \sigma_v \) = vertical stress as given by weight of overburden considerations
- \( (v/1-v) \) = numerical determination of \( K_0 \).

Cartwright (1997) examined the relationship between horizontal stress and depth using the same model as that which was later published by Nemcik, et al., (2005) according to his database of UK coal mining stress measurements. His statistical analysis applied the following equation, which is identical to [1] in its form other than the inclusion of an arbitrary constant:

\[ S_H = B_0 + B_1 [(v/(1+v)) \text{(Depth})] + B_2 \text{(Modulus)} \]  

\( B_0 \) 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 the “Tectonic Stress Factor” or TSF.

Regression analyses returned the following values for the constants, with an R-squared of 0.94:
- \( B_0 = -4.0 \text{ MPa} \)
- \( B_1 = 0.009 \text{ MPa/m} \), and
- \( B_2 = 0.78 \times 10^{-3} \)

Whilst Cartwright’s analysis indicated that Yong’s Modulus was more important than the depth for predicting the maximum horizontal stress, this would logically be expected from a stress measurement data set that is almost certainly taken from within pre-dominantly high modulus stone materials. What is intriguing in the context of this paper, which is considering horizontal stresses within low modulus coal, is that the \( B_1 \) value of 0.009 MPa/m is generally consistent with a vertical stress gradient of 0.025 MPa/m, in combination with a \( K_0 \) value in the order of 0.33 (which is linked to a Poisson’s Ratio of 0.25).

From the Cartwright (2007) analyses, it is inevitably concluded that \( K_0 \) related in situ horizontal stresses are almost certainly “alive and well” in coal measures strata, this then potentially including coal.

THE LINK BETWEEN POISSON’S RATIO AND \( K_0 \) HORIZONTAL STRESS

Poisson’s Ratio is the ratio between transverse and axial strains when a material is loaded axially, as shown in Figure 4. It is taken to be an elastic property with values ranging between 0 and 0.5.

![Figure 4: Schematic Illustration of Poisson's Ratio](image)

In terms of how Poisson’s Ratio is linked to \( K_0 \) horizontal stresses, Figure 5 shows a number of rock testing specimens that are hypothetically confined at their extremities and are all being compressed...
vertically, the resultant horizontal interaction between the samples as they all laterally expand being
the source of the induced $K_o$ horizontal stress.

With a typical value of Poisson’s Ratio for coal being in the order of 0.25, $K_o$ as given by [3], becomes
0.33, such that for every 3 MPa of vertical stress, the associated $K_o$ horizontal stress is in the order of
1 MPa.

![Figure 5](image)

Figure 5: Schematic Illustration of $K_o$ Horizontal Stress generation

![Figure 6](image)

Figure 6: Behaviour of In Situ test pillars during compression (Wagner 1974 as illustrated by
Galvin 2016)

ELASTIC LIMIT OF POISSON’S RATIO

As with all elastic material properties, Poisson’s Ratio is inevitably limited by the elastic axial strain
limit in line with Hooke’s Law. The question in the context of this study is what happens to transverse
or lateral strain relative to axial strain between the limits of an elastic Poisson’s Ratio with a value in
the order of 0.25 and the ultimate failure or maximum load-bearing ability of coal?

Fortunately, in situ testing results from the vertical compression of coal pillars in South Africa in the
1970’s provide invaluable insights into this issue, as illustrated in Figure 6.

The green shaded area of Figure 6 identifies a specific portion of the axial load-displacement (stress-
strain) curve that is within the second half (3 mm to 6 mm pillar compression) of that curve leading up
to the maximum coal or pillar strength. The critical characteristic of this area is that the measured
lateral deformation increases rapidly as compared to within the first half of the curve, whereby the axial
stress is no more than 50% of the ultimate strength and lateral deformation is in the order of 1 mm. Critically, from 3 mm to 6 mm of pillar compression, the measured lateral deformation increases from 1 mm to 12 mm, with the final condition just before coal failure being that of a vertical pillar compression of around 6 mm and a lateral deformation in the order of 12 mm. This is inconsistent with a Poisson’s Ratio of 0.25.

Based on the Wagner (1974) data, once coal or a coal pillar is compressed into the second half of its stress-strain curve, there appears to be a significant increase in the rate of lateral expansion of the coal, to the point that an elastic Poisson’s Ratio in the order of 0.25 becomes meaningless. This is worthy of further consideration in the context of the development of horizontal stresses within coal pillars under the action of high vertical stresses, both pre-mining and induced by mining.

**PRE-MINING VERTICAL STRAINS IN COAL SEAMS**

Prior to mining, the vertical strain in a coal seam is inevitably dictated by the weight of overburden that is acting on it. This is a simple and irrefutable concept.

Frith and Reed (2018), when discussing whether it is the coal pillars or the overburden that fails first prior to major pillar collapse, indicated that for pillar w/h ratios of 1 to 5, vertical strains at pillar failure were in the order of 1% according to laboratory-based coal testing data, as reported by Das (1986) for example as shown in Figure 7.

![Figure 7: Stress-Strain behaviour of coal for varying width to height (w/h) ratio (Das, 1986)](image)

Based on the green-shaded area in Figure 6, the onset of increasing lateral deformation commences at an axial strain in the order of 50% of that at ultimate coal strength, which is therefore estimated to be in the order of 0.5% (i.e. 50% of 1%).

Assuming an in situ Young’s Modulus for coal in the order of 2 GPa, 0.5% axial strain equates to an applied vertical stress in the order of 10 MPa, which is broadly equivalent to a cover depth of 400 m (i.e. 0.025 MPa/m x 400 m).

In other words, once the pre-mining vertical stress in coal exceeds around 10 MPa due to either a cover depth of 400 m or a lesser cover depth in combination with multi-seam vertical stress intensification, the upper limit of an elastic Poisson’s Ratio for coal is potentially reached. Specifically, due to the significantly increasing lateral expansion of coal above this limit, it would logically be expected that the induced horizontal stresses within the coal will become substantially higher than given by an equivalent elastic K_s analysis.

This provides a potential explanation for the source of substantially increasing major horizontal stresses within low Young’s Modulus coal seams at depths more than 400 m or in association with multi-seam vertical stress interactions at depths > in the order of 300 m.
SHEAR SLIP CONDITIONS ON HORIZONTAL PLANES OF WEAKNESS

Figure 8 contains a simple representation of the major horizontal stress in coal and the vertical stress along a horizontal pre-existing plane of weakness, the confining influence of the minor horizontal stress being ignored for the moment.

If the major horizontal stress is in the order of one-third the vertical stress, then ignoring any cohesive strength of the plane of weakness, a friction angle of only 18° (\(\tan^{-1}0.33\)) is required to prevent shear slip along the plane under the action of the major horizontal stress. Referring to Figure 9, it is clear that this is only likely achieved in association with either the presence of fine infilling material on the surface or a sheared surface, as intact, clean surfaces all have estimated friction angles > 34° irrespective of the surface condition.

![Figure 8: Vertical and horizontal stress conditions at coal to stone interface](image)

![Figure 9: Friction angles according to varying discontinuity condition (Barton, 2007)](image)
However, this changes substantially if it were the case that the major horizontal stress in the coal were at equal in magnitude to the vertical stress, which based on the previous analyses is judged to be a credible possibility in certain situations. The major horizontal stress and vertical stress being equal in magnitude requires a minimum friction angle of 45° to prevent shear slip along the plane. If the major horizontal stress in the coal were 1.5 times the vertical stress, the required friction angle then increases to 56°.

Based on Figure 9, it can be seen that smooth and slickensided planar surfaces (as shaded in red) would not provide sufficient frictional shear restraint to prevent the major horizontal stress overcoming the restraint of the vertical stress, this being entirely consistent with the coal to roof contact that is visibly evident at the Manalapan No.1 17 Mine pillar burst site in Figure 2.

**REDUCTION IN THE MINOR HORIZONTAL STRESS**

Unlike the Austar development coal burst model whereby the loss of minor horizontal stress was judged to be due to a pre-existing geological void within the coal seam linked to the development of “wing cracks” as a direct result of horizontal shear slip along a major geological fault, an increasing major horizontal stress within a coal seam due to $K_o$ effects more generally, will also act to increase the orthogonal minor horizontal stress, as $K_o$ effects do not logically result in a directional horizontal stress bias (see Figure 10).

![Figure 10: Schematic illustration of Poisson's effect under the action of vertical stress (Frith, 2002)](image)

The question therefore is how the stabilising influence of the minor horizontal stress can be overcome if there is no pre-existing void within the coal seam to allow it to be relieved over geological time prior to mining. With no major geological structures at the various pillar burst sites in the Manalapan No.17 Mine, another explanation is sought.

The only credible source for horizontal stress relief is the effect of the roadways surrounding a coal pillar, as will now be considered further by reference to the various “zones” within pillars.

![Figure 11: Zones developed within a coal pillar (Galvin, 2016)](image)

A commonly accepted zonal model for coal pillars is shown in Figure 11. It illustrates the changes in condition of the coal due to the pillar being formed and subsequently compressed vertically, the coal becoming less stable towards the outside of the pillar, with the central “elastic” core being the main source of pillar strength. The model also implies that the coal must have been in an elastic state prior...
to mining, which is why the central section or core of the pillar post-mining is defined as still being elastic.

For higher width to height (w/h) ratio pillars (> 4 to 5), coal pillars are influenced by a further phenomenon in that the central core becomes “confined” as a result of frictional restraint at contacts preventing the coal from expanding laterally as it is vertically compressed (see Figure 12). Whilst technically the core of the pillar remains “elastic”, it is only by virtue of the horizontal confinement being generated as the coal is further vertically compressed. It is this mechanism within the core of a pillar that gives rise to the ever-increasing strength of a coal pillar and the elimination of a strain-softening or collapse mode of failure as a direct function of w/h ratio (see Figure 13).

Figure 12: Diagram showing how shear resistance to lateral pillar dilation is generated on the contact surfaces of a pillar (Galvin, 2016)

Figure 13: Effect of w/h ratio on the stress-strain pillar characteristics (Galvin, 2016)

Under the model shown in Figure 11, the confined core of a high w/h ratio coal pillar is separated from mine roadways by coal material that is in varying states of failure, logically due to a lack of horizontal confinement. In other words, the coal around the perimeter of the pillar tends to provide a physical and quite substantial barrier between the mine workings and the highly stressed central pillar core.

However, if the coal itself were already compressed beyond its elastic limit prior to mining, as previously described, then in effect the confined core of the pillar would pre-exist mining and so
logically extend to the edges of the pillar immediately that a roadway is formed. This gives rise to an entirely different scenario than the model shown in Figure 11, as:

i. The coal is already in a non-elastic pre-mining state by reference to Figure 6.

ii. Elevated horizontal stresses are therefore likely acting within the coal at the edges of the pillar.

iii. The formation of the roadway removes the confinement (i.e. the coal) that allowed those elevated horizontal stresses to ever be generated prior to mining.

iv. Containment of horizontal stress in the coal at the edge of the pillar becomes entirely reliant upon the shear strength of the restraints along contact surfaces, as illustrated in Figure 12.

v. Critically, there is no immediate coal “barrier” in place to protect mine roadways from a highly stressed pillar core, which now extends to the edges of the pillar.

With the “confined core” of the pillar extending throughout the pillar to the edge of the roadways, the necessary conditions for coal to burst out into the mine workings have been established by the simple act of driving roadways, the only effective control being the cohesion and friction along horizontal contacts within the coal seam.

With the entire perimeter of a small coal pillar being in such a “burst-prone” condition, the ability of one side of a pillar to burst out between parallel roadways in a single event becomes understandable, at least conceptually. When it is also considered that the perimeter around the pillar is likely a quasi-continuous structure, the entire perimeter bursting out in response to one side becoming unstable, would also make sense.

Having established that the pre-mining condition of coal can give rise to coal burst potential at the edges of a coal pillar, it is worth considering any further influence of mining-induced vertical stress increases.

INDUCED VERTICAL AND HORIZONTAL STRESSES DUE TO PILLAR FORMATION

On the basis that roadway widths are essentially fixed, the vertical stress increase on each coal pillar due to mining increases as a direct function of reducing coal pillar size. For 24 m x 24 m roadway centres and 6 m wide roadways (as assumed for the Manalapan No.17 Mine cases), the total vertical stress acting on each coal pillar is almost double the pre-mining value.

Simply, if the coal around the perimeter of a coal pillar is not highly confined by pre-existing horizontal stress, it will inevitably tend to fracture and fail similar to that in an unconfined laboratory test under the action of increasing vertical stress due to mining, the end result being the zonal model shown in Figure 11 whereby the confined core and mine roadways are physically isolated from one another. Increasing the vertical stress on a coal pillar due to mining may drive the perimeter of the pillar from one that is initially stable, albeit containing elevated horizontal stresses, to one whereby the ratio of horizontal stress to vertical stress in the coal has the ability to result in shear failure along a horizontal contact (Figure 8), with the coal then inevitably bursting out into the roadway due to horizontal stress unloading and coal acceleration, the mechanics of which are fully explained in Frith, et al., (2019).

If this pillar burst model has credibility, it should be the case that prior to a pillar burst, minimal rib spall (“slumped” coal in Figure 11) was observed with few obvious pre-cursor warnings signs, this having been the case with the Austar incident in 2014. In relation to the pillar bursts at Manalapan No.17 Mine, Newman 2002 makes the following statements, noting his use of the term “bump” rather than “burst”:
“Prior to the first bump on December 6, 1999, mine personnel stated that there was no bumping or "working" of the roof or rib, no rib spalling, and no floor heave. Strain energy continued to build undetected until a bump occurred that damaged several pillars. The concern of mine personnel was that the bump had occurred without a series of precursor events. Mine management decided to abandon and seal the section and drive a parallel submain.”

“However, beginning two days before the second bump on September 22, 2001, there were numerous small bumps at the face”.

In other words, despite the pillars that eventually underwent a pillar burst, being formed at a cover depth in excess of 450 m and being impacted by some level of multi-seam vertical stress interaction, no rib spall prior to the first pillar burst was reported, only the occurrence of presumably minor “bumping” events at the face in the second example.

The lack of rib spall is explained by the inferred existence of high levels of horizontal confinement in the coal at the edges of the pillar when roadways are first driven. The minor bumping events that preceded the second burst at the Manalapan No.17 Mine, are taken to be indicative of marginally stable contact conditions under the action of horizontal confinement within the general area.

The same model of the confined core extending to the edge of the pillar due to the pre-existing state of the coal, also infers that more than one pillar burst event can occur if the burst coal in the surrounding roadways is subsequently removed, the reason being that this coal is acting to confine the perimeter of the remaining core of the pillar, in the same way that the “yielded”, “crushed” and “slumped” coal do in Figure 11. The tragic second major event at Crandall Canyon is entirely consistent with this scenario.

Finally in this section, it is interesting to consider that other coal industries routinely develop roadways at far greater depths under multi-seam stress interactions than those that have proven to be pillar burst prone in the USA. The mining layouts tend to incorporate either single entry drivages or wide, long pillars, the minimum width of which is commonly designed according to a one-tenth of the depth rule. In contrast, the coal pillars that burst at the Manalapan No.17 Mine, as shown in Figure 1, whilst being reported as having ARMPS Version 6.0 SF values in excess of 2 (Mark, 2017) had solid widths of around 18 m (24 m centres and assumed 6 m wide roadways), which at a cover depth in the order of 500 m is only one-thirtieth of the cover depth. The pillars were also square rather than being distinctly rectangular.

What this inevitably leads to is the hypothesis that in the same way that low w/h (< 4 to 5) coal pillars need to be designed in section (i.e. by their width and height) to remain stable under the action of vertical stress by virtue of the pillar strength, higher w/h ratios pillars that develop an internal confined core that increases pillar strength, need to be designed in plan (i.e. by their width and length) in order to ensure that if the confined core is likely to extend to the outer edge of the pillar by virtue of the level of pre-mining coal compression, it retains its stability under the action of the horizontal stresses that are subsequently generated as a direct function of the induced vertical stresses on the pillar due to mining.

**CONCLUSIONS**

A review has been conducted on two reported coal pillar bursts in the US in an attempt to identify a cause and effect model that explains why such destructive events can occur on an infrequent and seemingly random basis. This was required to further develop the Venn Diagram classification of coal bursting events following an analysis of the development coal burst event at the Austar Mine in 2014 (Frith, et al., 2019) and overburden bump causation mechanisms (Frith, et al., 2020) – see Figure 14.

The starting point for the analysis was the cause and effect model developed by Frith, et al., 2019 for the 2014 development coal burst at the Austar Mine, which is based on the coal bursting mechanism of the major horizontal stress in the coal seam becoming unstable and therefore rapidly unloading in the manner of an elastic spring. For this to occur, an elevated major horizontal stress and substantially reduced minor horizontal stress in the coal, along with a low friction plane at or towards the top of the roadway, are required for the major horizontal stress to become unstable and so unload in a violent manner. Such conditions can demonstrably come about in proximity to very specific geological faults as are known to exist at both the Austar Mine burst site and the Sunnyside Mine in Utah more generally, which also reportedly suffered with development coal bursts.
Pillar bursts where the entire perimeter of one or more coal pillars fail violently, cannot be explained by the Austar-type development burst model, however it provides a useful starting point from which to consider how and why pillar bursts may occur, the need for a low friction horizontal contact plane being common to both models.

Aged *in situ* testing data from coal pillars indicates that well before the ultimate strength of a coal pillar is reached, the rate at which the coal laterally expands under increasing vertical compression can exponentially increase so that the elastic magnitude of Poisson’s Ratio no longer applies. The logical consequence of this is that if adequate restraint exists, substantially higher horizontal stresses can be generated within the coal seam under the action of vertical stress, as compared to the coal seam being within the vertical compression range where Poisson’s Ratio applies. A general transition point has been estimated at a cover depth of around 400 m, which immediately explains the anecdotal significance of high cover depth and/or multi-seam vertical stress interactions on pillar burst potential.

If the pre-mining state of the coal contains elevated level of horizontal stress that are greater than those that would be generated by the elastic value of Poisson’s Ratio, it is argued that the traditional model of zoning within a coal pillar (as illustrated in Figure 11), does not necessarily apply, the central elastic core of a higher w/h ratio pillar potentially extending to the outside edge of the pillar. This sets up the dual conditions of (a) reduced rib spall by virtue of higher horizontal confinement within the coal at the edge of the pillar, and (b) pillar burst potential, the prevention of which is dictated by (i) the shear strength of the various horizontal contacts, (ii) limiting the magnitude of the vertical stress increases that are developed due to mining and (iii) maintaining the integrity of the minor horizontal stress within the coal.

As the induced vertical stress on the pillar due to mining is an integral part of the pillar burst model, it logically follows that the failure of any part of the confined core in one pillar by bursting, would act to increase the vertical stresses acting on adjacent pillars. Therefore, one such failure would have the potential to induce others in adjacent pillars, particular under a softer overburden loading system due to multi-seam mining effects. This is known to have occurred.

Experience from deep mining industries more generally appears to indicate that pillar bursts can potentially be averted by the use of wider and longer pillars, as compared to the more common use of small, square pillars that are often sized for place change development purposes. Wider and longer pillars achieve two things; (i) they keep parallel roadways further apart and (ii) they reduce the magnitude of mining-induced vertical stress increases on the pillar.

It is therefore hypothesised that coal pillar design under potential pillar burst conditions around the perimeter of a pillar, needs to analyse the pillars in plan, as well as in section, to ensure that the

---

**Figure 14:** Suggested classification of high energy release events in underground coal mining for four fundamental event types (Frith, *et al.*, 2020)
outside perimeter of the pillar remains stable under the action of horizontal stress within the coal. A suitable design method for pillars with w/h ratios > 4 to 5 is yet to be developed, but anecdotal evidence suggests that it is pillar width and length that are the controlling parameters for pillar bursts, in the same way that pillar width and height control the strength and stability of the pillar under vertical loading at lower w/h ratio pillars.

If nothing else, this hypothesis opens up the potential for coal to be mined without the threat of pillar bursting via the use larger pillar dimensions in plan (if necessary), rather than significant coal reserves necessarily being sterilised by maintaining the use of small pillars, thereby limiting the maximum cover depth where safe mining can be achieved.

REFERENCES


ABSTRACT: Mining subsidence has been a major hazard in most underground coal mines, particularly those where designs and practices are based on the wrong assumption of fixed, permanent and nondeteriorating coal pillars. Mining induced subsidence significantly affects mining costs where major surface structures and natural environment need to be protected. Remedial measures to manage damage caused by subsidence can often be very costly with potentially damaging impacts and irreversible consequences. Backfilling and injection of granular materials into the mining induced voids, separated beddings and cracks, as either diluted granular slurry or concrete paste, is widely used to control mine subsidence overseas. Granular grouts and slurries made of mine and power plant wastes and rejects are viable environmental backfill solutions to both ground stability and mine waste management problems. Like concrete paste, the flowing slurry can be categorised as a generally nonlinear frictional viscous cohesive (Bingham Herschel-Bulkley) fluid. The general frictional viscous, cohesive, non-Newtonian fluid model has been applied to concrete flowability problems such as L-box and slump tests. While slump test is used in shallow foundations, L-box test is used in difficult deep foundations. It is designed to measure workability and flowability of tremie pipe concrete as an indirect index measure of concrete viscosity and plastic yield. Tremie pipes are used to control concrete flow rate and minimise bleeding and dilution when placed into deep submerged excavations. Mathematical and experimental models have been developed to not only solve the flow velocity along the L-box channel length as a function of time and distance, but also simulate the flow of the backfill material and demonstrate the detailed process of filling the voids to minimise any further subsidence.

INTRODUCTION

Mining Subsidence

As a major potential hazard, mining induced subsidence significantly affects mining costs where major surface structures and natural environment need to be protected, e.g. mining under river systems, gorges, cliffs, power lines, pipelines, communication cables, major roads and bridges, and other significant surface facilities. Remedial measures to manage damage caused by subsidence can often be very costly with potentially damaging impacts and irreversible consequences. The recent mining induced subsidence events occurred at Collingwood Park of Ipswich (Figure 1) are great cases of unforeseen risk, how easily material strength can deteriorate and result in surface ground subsidence, and severe consequences in mine safety, stability, accessibility and reliability (Shen et al, 2010).

Backfilling and injection of granular materials into the mining induced voids, separated beddings and cracks, as either diluted granular slurry or concrete paste, is widely used to control mine subsidence. Granular grouts and slurries made of mine and power plant wastes and rejects are viable environmental backfill solutions to both ground stability and mine waste management problems. Like concrete paste, the flowing slurry can be categorised as a generally nonlinear frictional viscous cohesive (Bingham Herschel-Bulkley) fluid (Alehossein, 2009, Alehossein et al, 2012). However, in mining applications, to reduce ground surface subsidence and control the propagation of the overburden movement to the surface, the solid particles in the injected slurry must deposit in the bed separation gaps of the coal seam over-burden strata, e.g. in longwall mining the grout slurry is pumped into the separated beds of the rock mass from a batching plant source through pipelines connected to a central vertical borehole, which is drilled deep into the over-burden rock above the coal seam (Alehossein and Poulsen, 2010). Flow blockage can occur in the injection system, when the
slurry velocity falls below a certain critical threshold velocity. The stiffening, consolidating non-flow slurry can generally be categorised as a frictional cohesive soil (Shen, and Alehossein, 2009). In other words, a change of material phase from cohesive-viscous to cohesive-frictional will occur. Using a smaller scale model, this field injection practice has been simulated at the QCAT laboratory of CSIRO in Brisbane, Australia, to study the influence of various grout injection parameters by pumping slurries through various pipes of different sizes and diameters and for different applications (Alehossein 2009). As an important industrial application, grout injection into the inter-burden strata is used as a modern technology to control and reduce coal mine subsidence. Slurry mixes of coal mine and power plant waste materials, e.g. fly ash or any other coal wash rejects, are injected back into the inter-burden rock strata during longwall mining. To reduce subsidence and control inter-burden strata movement, the injected slurry solid particles must deposit in the opened strata bed separation gaps or cracks before crack closure. The mechanics of non-Newtonian fluids flowing between parallel disks is a classical fluid mechanics problem that has been studied by a number of researchers in the past for their specific problems of interest (Beckhaus, et al, 2011, 2012, Qin et al, 2013, Larisch et al, 2013). Tremie pipes are used to control concrete flow rate and minimise bleeding and dilution when placed into deep submerged excavations. The L-box test is designed to measure workability and flowability of tremie pipe concrete as an indirect index measure of concrete viscosity and plastic yield. The L-box model solves a dimensionless PDE in terms of the flow velocity along the L-box channel length as a function of time and distance, which is analogous to a non-homogeneous heat conduction equation. The general frictional viscous, cohesive, non-Newtonian fluid model has been applied to concrete flowability problems such as L-box and slump tests. Figure 2 shows applications of this modelled fluid in concrete L-Box tests (Left), injection into strata (Middle) and laboratory scale-down injection experiments (Right).

Figure 1: Locations of 15 drill holes designed by CSIRO distributed in both historical subsided and non-subsided areas (Shen et al, 2010)
Figure 2: Various applications of viscous slurry and paste fluids: (a) channel flow for workability and consistency testing of concrete; (b) Concrete tremie pipe flow into submerged foundations; (c) Multi-phase slurry flow in pipes and fractured rock strata for void backfilling

Pillar Failure

In the room and pillar mining method, the overburden load must be carried by the coal pillars left in the coal seam with the danger of failure due to many possible mechanisms such as coal weathering and deterioration due to the lack of confinement or erosion by flooding, or close proximity to the natural fault zones. Figure 3 shows the 3D seismic results of the amplitude distribution of seam reflections when mapping failure in the coal seam. It clearly shows that the subsidence area on the surface and failure area at the seam level are different.

Figure 3: 3D seismic results. Amplitude distribution of seam reflections for mapping the failure at the seam level. Yellow dots are the failure boundary locations at the seam level mapped from seismic sections. Thick blue curve describes the failure boundary at the seam level based on the reflection amplitude strength. Light blue dots are the boreholes drilled after the 3D seismic field work (Shen et al, 2010)

The thick blue line is the failure boundary locations at the seam level mapped from specified seismic sections. The thick blue curve describes the failure boundary at the seam level based on the reflection amplitude strength. Amplitudes are clipped to make the selection of the subsidence boundary at the seam level easier. The light blue dots are the boreholes drilled after the 3D seismic field work.
fly ash and mine slurry wastes, various backfill mixes were investigated as a mitigation solution to the prevention of further subsidence.

**LITHOLOGIES**

**Blackstone Formation**

The Blackstone Formation consists of coal, interbedded shale, siltstone and fine sandstone (Figure 4), and has been interpreted as being deposited in a fluvial environment with floodplains, meandering channels and peat forming mires (Shen et al. 2010). This environment results in stratigraphy that has a high degree of lateral variation. For example, the two seams that make up the Main Seam, the Bluff and Four Foot Seams, are separated by tens of metres of interburden elsewhere in the basin, whereas they are in contact in the study area. There are numerous clay bands (tonsteins) in the formation that have been interpreted as volcanic ash falls. Where drilled, the floor of the Main Seam (CP_C01, CP_C02, CP_O11, DME BH2) consists of siltstone and carbonaceous mudstone (Figure 3). The top of the Main Seam has been eroded by the overlying Aberdare Conglomerate in the Westfalen No. 3 area. Recent drilling suggests that this overlying unit is several metres above the roof of the Main seam throughout most of the mine area, and that the seam is largely intact (Shen et al, 2010). The immediate roof of the coal seams consists of fine grained sandstone thinly interbedded with siltstone and carbonaceous mudstone and appear to conformable with the coal in CP_C01, CP_C03 and DEM BH2. These sediments are interpreted to be part of the Blackstone Formation.

![Figure 4: Lithologies from the Blackstone Formation.](image)

**Backfilling Remediation**

Using fly ash and mine slurry wastes, various backfill mixes were investigated as a mitigation solution to the prevention of further subsidence. The freely available fly ash was a good candidate in this...
investigation. A scale-down model of the mine and pillars was developed at the CSIRO laboratory which were then filled with various fly ash slurries. These experiments proved suitability of fly ash for successful deposition, sedimentation and consolidation of the backfill and its subsidence prevention application, as shown in Figure 5. These laboratory and field experiments on mine-backfill fluids, slurries, cements, pastes and concretes proved their wide range of shear resistance and complex behaviour in response to shearing necessitating development of a general, nonlinear, cohesive, viscous, frictional, nonlinear, non-Newtonian model of shear stress versus shear strain rate, as an extension to the classical Bingham-Herschel-Bulkley fluid. The value of the shear strength function at zero shear strain rate, i.e. plastic yield and the tangent slope of the stress-strain rate curve (viscosity), at any given shear rate, are the two most important parameters of such fluids – Figure 5.

![Experiment Image]

Figure 5: Laboratory scale-down simulation of mine backfill using mixtures of fly ash being deposited in the simulated mine prototype

**FRICIONAL VISCOPLASTIC FLUID MECHANICS ANALYTICAL SOLUTIONS**

Referring to Figure 6 (left), the general constitutive equation, relating fluid shear stress to shear rate for such general nonlinear, non-Newtonian, viscous, plastic, frictional fluids, which can be applied to fresh concrete, mine backfill slurries and high frictional multiphase fluids, is as follows (Alehossein, 2009)

\[
\tau(t, x) = \mu(t, x)\left(-\frac{\partial u(t, x)}{\partial x}\right) + \eta(t, x)\left(-\frac{\partial u(t, x)}{\partial x}\right)^n + \tau_0(t, x) + \xi(t, x)p(t, x)
\]  

(1)

![Graphical Representation]

Figure 6: Various shear stress components of Equation (1) (LHS) and its applications (RHS)

In Equation (1) \( \tau \) is shear stress tensor, \( u \) is velocity vector, \( \mu \) and \( \eta \) are linear and nonlinear viscosities, \( \tau_0 \) is plastic yield, \( p \) is pipe pressure and \( \xi \) is concrete friction coefficient. The last term, involving the friction and pressure terms \( (\xi_p) \), is a frictional resistance term which can be applied only
when a pipe blockage occurs due to the concrete granular material friction and needs to be reopened by a higher pressure flow, otherwise it can be ignored. Various shear stress components are shown in the left hand side of Figure 6, where $\tau_0$ is the constant uniform plastic yield component, with no viscosity; $\mu$ is the Newtonian linear viscosity coefficient of the linear velocity gradient $y$ with a wall value $y_w$; $\eta$ is the non-linear viscosity; $\zeta$ is the friction coefficient of the fluid pressure $p$.

Right hand side of Figure 6 shows the comparisons of normalised velocity profiles for different slurries of various viscosity ($\mu$) and plasticity ($\tau$) in a pipe flow. Exact equations can be derived to relate the fluid flow rate $Q$ to the fluid pressure $p$, e.g. in a pipe or radial disc. The results are integral equations relating velocity gradient $y$ or $y_h$ (or $\psi$, or $\psi_h$ in the case of radial flow) to the flow rate $Q$ (Alehossein 2009, Alehossein, et al 2012).

Pipe Flow:

$$Q = \int A \frac{h}{3} \left( y_h - \frac{1}{h} \int_0^h x y dx \right)$$  \hspace{1cm} (2a)

Radial Flow:

$$Q = \int A \frac{h}{2} \left( \psi_h - \frac{1}{r_h} \int_0^{r_h} \frac{\psi}{r} \right)$$  \hspace{1cm} (2b)

Slurry flow may be assumed to stop in the case of a blockage ($Q \rightarrow 0$), which means the boundary values of the velocity gradients $y_h$ and $g(y_h)$ are identically zero. This is due to the effects of the cohesive frictional terms ($\tau$ and $\tau_0$) introduced in the shear stress Equation (1), which now become dominant in blocking the slurry flow. The above general theory is certainly reducible to simpler classical Newtonian and Bingham models with appropriate parameter substitutions, as demonstrated by separated components in Figure 6 (Left).

EXPERIMENTS

L-Box Test

L-box test is a relatively new laboratory and on-site testing method to check whether or not a fresh tremie concrete with a maximum coarse aggregate size (e.g. 20 mm) or less is able to flow into all spaces within a foundation excavation and through tight small spaces and openings under a certain concrete head (Beckhaus et al, 2011).

Figure 7: An L-box test gear (left) for measuring visco-plastic behavior of fresh concrete samples by pouring concrete into the vertical rectangular chimney box and let it flow into the horizontal channel box by opening a sliding gate attached to the chimney box. Flow time ($T_0$) and end heights of concrete flow profile ($y_1$, $y_2$) are measured (Beckhaus et al, 2011)

A photo of the L-box test is shown in the left hand side of Figure 7, with schematic size details on the right hand side figure. A sample of a concrete of certain weight is placed into the vertical rectangular chimney of the L-box first, then, the vertical sliding gate attached to the chimney part is opened to
allow the concrete to flow horizontally. Both the time ($T_0$), which takes the concrete to reach the other end, and the end profile coordinate, $y_2$, are measured. As an example: $L_0 = 3.94$ in [100 mm], $L = 27.56$ in [700 mm], $h = 23.62$ in [600 mm], $W = 7.87$ in [200 mm], $y_1 = 5.91$ in [150 mm], $y_2 = 2.95$ in [75 mm], time for complete flow reaching the other end $T_0$ is less than 10 seconds.

Concrete flow during pouring and flowing in channels, chutes and testing equipment for testing purposes are normally not at a steady state situation. General time-dependent 2D and 3D differential equations governing flow of concrete in rectangular channels and chutes can be developed and solved numerically, as shown in (Alehossein et al 2012). However, for the sake of understanding, it is also possible to reduce these equations to a simple 1D form, based on an assumption that there is no significant independent variation in any variable or function in the normal directions $x_2$ and $x_3$ compared to the longitudinal main flow direction $x_1$. In other words,

$$ \tau_{x_2}(t, x_1) - p_{x_1}(t, x_1) - \rho u_{x_1}(t, x_1) = 0 $$

which gives a solution in terms of Fourier coefficients

$$ u(t, x) = \sum_{n=0}^{\infty} e^{-\kappa_n^2 t} \left( A_n \cos(\kappa_n x) + B_n \sin(\kappa_n x) \right) + m x^2 $$

In the solution (4), $\kappa$ is an arbitrary constant satisfying both the differential equation and the boundary conditions, while $A_n$ and $B_n$ are Fourier coefficients to be determined from the boundary conditions. Figure 8 shows a typical result for various values of $n$ truncating the number of Fourier terms. It shows results of the Fourier analysis for the two cases of $u(0, x)$ and $u(0.5, x)$, and the increasing effects of the number of Fourier terms, namely from $n = 5, 10$ to $120$. The second line in the figure corresponds to velocity at time $t = 0.5$ for different profile points along the x line using $n = 120$. Notice that since continuity and differentiability is not a requirement at the end points of a Fourier series analysis (Alehossein et al 2012), it doesn’t converge to the numerical solution at point $x = 1$, as expected.

![Figure 8: The function $u(0, x)$ represented by a Fourier series with different number of Fourier coefficients: $n = 5, 10, 120$ (Alehossein et al, 2012)](image)

**CFD NUMERICAL SIMULATIONS**

Computational fluid dynamics (CFD) simulations of the concrete tests, e.g. slump and L-box tests, can be carried out to predict the flow-ability and stability of real fresh concretes, e.g. tremie pipe concretes placed in deep and underground pile foundations. Concrete placement in piles is a blind process, especially for deep foundation elements. To control the quality of the foundation, it is desirable to know the concrete flow performance during placement. The advance of CFD provides the possibility to simulate the whole process of pouring concrete in the deep excavation or cavity in a soil or rock mass. With the aid of graphs or videos of the results, the simulation allows engineers to virtually "see" what is happening in the inaccessible space where the concrete is flowing. Before simulating the real concrete placement process, small scale model tests need to be established to simulate the laboratory test and validate the numerical model.
Two CFD models are developed to simulate slump and L-box tests. Each model mimics the same geometry as required by the real slump and L-box tests, as described previously. The geometrical models used for CFD discretisation (mesh) are shown in Figure 9. The multiple material phase models (concrete, water, air) are used in the CFD modeling. Flow behaviour in the two tests is treated as transient. The concrete mix is treated as a non-Newtonian frictional viscoplastic fluid (2-4). For the L-box test, the concrete stays initially in the vertical section of the L-box, i.e. the horizontal section of the L-box is initially empty. The concrete starts to flow due to gravity when the gate opens or will be removed. For the slump test, the initial condition is that the test cone is fully filled with concrete. The simulation starts when the cone is lifted or removed. Figure 9 displays the process of the tremie concrete flowing along the L-box. The flow length, represented by the distance from the gate to the tip of the concrete is recorded, as shown in Figure 10. It can be seen that the tip of the concrete reached the end of the horizontal box at about $t = 8$ s, which matches the real test results. In addition to the concrete flow profiles and profile fractions, CFD method can provide many other features and information about the flow performance such as pressure, velocity and temperature at any time and any location.

Figure 9: Meshed geometries of CFD models for the slump test and the L-box test

Figure 10: CFD simulated process of the L-box test showing the fraction of concrete in the modelled domain
Figure 11 shows the CFD modelling results of the familiar concrete slump test world widely used. The initial condition is that the test slump cone is fully filled with concrete and the simulation starts when the cone is lifted or removed. In this figure, the maximum CFD drop and spread are 252 and 410mm matching well an experimental test result (Larisch et al, 2013).

![Figure 11: CFD simulation of concrete slump test at different times](image)

### CONCLUSIONS

Experimental and numerical (CFD) simulations carried out proved that both slurry backfills and modern concrete can be modelled as a non-Newtonian frictional viscoplastic Bingham-plastic fluid. The CFD simulations demonstrated that the method is capable of shedding light on the "blind process" side of concreting deep and underground foundations, as a powerful prediction tool. Quality control, optimised ingredients and best operational procedures may be achieved by repeatedly simulating the process with different conditions and configurations to ensure high quality of the foundation concrete.

On the basis of continuum equations of fluid and soil mechanics, a comprehensive, versatile, slurry shear model has been developed for transportation of granular grout, paste and fill materials used in the civil and mining industries, covering a wide range of material characteristics and behaviour, namely from the flowing fluid slurries to consolidated solid deposits in underground coal mining induced rock fractures. The theory has been specifically tailor made for grout flows through uniform pipes, discs and tremies, in order to transport material to designated injection or backfill targets. The theory can mimic both flow and blockage behaviour of the fill material. The tool can be used to predict variations of pressure and velocity and their gradients, as a function of flow rate, in the entire backfill-placement system from batching plant to the borehole cracks and foundation excavations.

The shear theory can mimic shear resistance of both: (i) a cohesive, viscous flow and (ii) a stationary, cohesive, pressure-dependent, frictional, plastic soil. The pressure dependent frictional term in the shear stress model determines the frictional resistance of the deposited fill material during a blockage. Consistent with laboratory and field experiments, the theoretical pump pressure required to open a blockage is orders of magnitude greater than the amount needed for pumping the same material when it is under a steady state flow. This explains why very high pump pressures are often needed to clean blockages compared with much lower pressures required during steady state slurry flows. Concrete flow and placement into deep foundations is normally performed under several harsh environmental conditions of tightness, inaccessibility and deep submergence. Therefore, it must be self compacting, self levelling and maintain its original quality, homogeneity and integrity all the way from the tremie pipe to the discharge point and then through the narrow paths between heavy reinforcements. Traditional slump and spread tests together with the L-box tests are used as indirect index tests to measure physical visco-plastic properties of concrete.

### REFERENCES


FURTHER INSIGHTS INTO THE MECHANICS OF MULTI-SEAM SUBSIDENCE FROM ASHTON UNDERGROUND MINE

Ken Mills¹ and Stephen Wilson²

ABSTRACT: Ashton Underground Mine (Ashton) is an underground longwall mine located northwest of Singleton in the Hunter Valley of NSW. The mine has so far extracted longwall panels in three seams with mining in a fourth seam planned and each seam progressively deeper than the last. The mining geometry in each of the seams is regular, parallel and either offset or stacked relative to the panels in the seams above. A subsidence line crossing all panels in each seam has been regularly surveyed in three dimensions since the commencement of mining. The high quality data set available from this line provides insight into the mechanics of ground behaviour in a multi-seam environment. This paper presents an update of the observations and interpretation presented in Mills and Wilson (2017) for mining in two seams with the inclusion of results from mining in a third seam.

Observations of the characteristics of multi-seam subsidence continue to indicate that although subsidence movements above multi-seam mining are more complex than single seam mining, these movements are nevertheless regular and predictable. In an offset geometry, remote from pillar and goaf edges, tilt and strain levels are similar or lower than single seam levels, despite the greater vertical subsidence, due to the general softening or reduction in shear stiffness of the overburden with each episode of subsidence. At stacked and undercut goaf edges, transient tilts and strains are significantly elevated.

Cumulative vertical subsidence after longwall mining in three seams has now reached 5.8m with incremental vertical subsidence increasing as a percentage of incremental mining height with each episode of subsidence. Latent subsidence from near stacked goaf edges is recovered when mining in the seam below. A site-specific methodology developed to forecast subsidence behaviour is allowing measured subsidence effects to be estimated reliably.

INTRODUCTION

Since the multi-seam subsidence monitoring data and interpretation from Ashton Underground Mine (Ashton) was last prepared in late 2016 and early 2017 (Mills and Wilson 2017), two additional longwall panels in the second seam of mining have been completed and three longwalls have been mined in the third seam. Observations from the monitoring of these extra panels have enhanced the previous dataset for Ashton and confirmed the contemporary understanding of multi-seam subsidence behaviour at this site.

The mining at Ashton provides a unique opportunity to study the mechanics and interactions of multi-seam mining to improve understanding for the preparation of future mining applications. The characteristics that make this site unique include:

- Longwall mining occurs in a regular, parallel layout with substantial chain pillars remaining.
- Modern, reliable mine plan records are available in all three seams mined.
- There are no areas of irregular bord and pillar mining or pillar extraction.
- There is no potential for small pillars (or ‘stooks’) to fail and contribute to risk of pillar run or pillar creep.
- Gradually increasing overburden thickness towards the west provides data for a range of panel width to depth ratios.
- Longwall panels with different starting and finishing positions and goaf edge geometries enable a range of mining scenarios to be studied.

¹ Principal Geotechnical Engineer, SCT Operations Pty Ltd. Email: kmills@sct.gs Tel: +61 2 4222 2777
² Mine Planner, SCT Operations Pty Ltd. Email: swilson@sct.gs Tel: +61 2 4222 2777
A methodology for estimating multi-seam subsidence effects at Ashton was developed based on site specific data and observations after the first two longwalls in the second seam. For vertical subsidence, this method involves the forecast of an incremental subsidence profile for a seam or seams. The forecast profile(s) is then added to the actual measurements along the subsidence profile of the overlying seam or seams to forecast the cumulative vertical subsidence. The forecasting of incremental and cumulative values of tilt and strain is based on the Holla (1991) guidelines for the Western Coalfield using K values for the constant of proportionality derived from site-specific measurements for general background, stacked goaf edges and undercut goaf edge areas.

**BACKGROUND AND SITE DESCRIPTION**

Ashton Coal Operations Pty Ltd (ACOL), owned by Yancoal Australia Ltd, operates the Ashton Underground Mine near Camberwell in the Hunter Valley of New South Wales. The mine operates via modified development consent for the Ashton Coal Project (ACP). The mining approval includes underground longwall mining in four seams. In descending order these seams are the Pikes Gully (PG), Upper Liddell (ULD), Upper Lower Liddell (ULLD) and Lower Barrett (LB).

The first longwall in the uppermost PG Seam commenced extraction in 2007. A series of eight longwall panels have been mined in the PG Seam. The longwalls in the second seam (ULD Seam) started secondary extraction in 2012 and six longwalls were completed to mid-2017. Longwalls in the third seam (ULLD Seam) started in mid-2017 and three longwall panels were completed by mid-2020.

Figure 1 is a site plan showing the outline of the longwall voids for the PG, ULD Seams and the three longwalls mined to date in the ULLD Seam superimposed onto a topographic map of the surface area. The positions of subsidence monitoring lines are also shown.

The longwalls in the ULLD Seam are substantially within the footprint of the overlying PG and ULD Seam longwalls, so that the majority of ULLD Seam longwall mining represents three seam extraction with smaller areas of two seam extraction.

The panels in each of the four seams were originally approved to be arranged in a regular, parallel, stacked (superimposed) geometry. However, the layout design has been altered to an offset (staggered) geometry to reduce the subsidence profile and surface impacts and to take advantage of the potential for reduced stress conditions during roadway development. In this offset geometry, longwall panels in the first (PG) and third (ULLD) seams are superimposed. Longwall panels in the second (ULD) and fourth (LB) seams are also superimposed. The ULD and future LB longwall panels are offset 60m to the west relative to the PG and ULLD seam longwall panels.

In the regular geometry, all longwalls panels form a void that is nominally 216m wide and all inter-panel chain pillars are 24m wide (coal rib to rib). The panels are aligned in an approximately north-south direction with the longwall face retreating from south to north. The panel mining sequence is from east to west. The naming convention for longwall panels in each seam starts with Longwall 1 in the PG Seam, Longwall 101 in the ULD Seam and Longwall 201 in the ULLD Seam.

The mining height for each seam is approximately 2.5m ±0.3m. Mining heights are limited by the seam thickness and the practical operating range of the mining equipment.

The overburden strata and seams dip moderately to steeply to the west at approximately 1 in 10. The gradient of the strata is typically greater than the gradient of the surface topography. The overburden depth to the PG Seam increases from 40m in the northeast corner of Longwall 1 to 180m in the southwest corner of Longwall 7. The interburden thicknesses are typically 35-40m for the PG to ULD seams, 20-25m for the ULD to ULLD seams and 35-45m for the ULLD to LB seams.

The surface topography above the mining area is dominated by a steeply rising ridge line adjacent to Glennies Creek in the east from which the ground slopes west toward Bowmans Creek and the Hunter River to the south.

The longwall mining area is bounded by consideration of the subsidence impacts and consequences to both the natural and built surface and sub-surface features. The main features include the New England Highway and infrastructure in the north, Glennies Creek to the east, the Hunter River to the south and to the west, a combination of Bowmans Creek, Bowmans Creek diversion channels and adjacent mining operations.
Subsidence Monitoring

A comprehensive subsidence monitoring program involving high confidence three-dimensional (3D) survey measurements on conventional monitoring lines has been in place since the start of longwall mining at the Ashton site. Aerial imagery and LIDAR surveys are also regularly captured at Ashton.

For the PG Seam longwalls, some 35 monitoring lines were installed and surveyed regularly. These subsidence monitoring lines are aligned both along the panels (longitudinal) and across the panels (transverse). The main cross panel line (XL5) extends over all the southern longwalls. Sections of this
line are surveyed for each individual panel as it is mined. The full length of this line was resurveyed at the completion of the PG Seam longwalls and again after completion of mining in the ULD Seam. Sections of XL5 have been surveyed regularly during the mining of the first three longwalls in the ULLD Seam.

A series of 12 additional longitudinal lines were established for the offset geometry in the ULD Seam. These lines are adjacent to the PG Seam lines at both the southern and northern ends of each panel. Additional 3D monitoring is conducted at other surface features or infrastructure including the poles on the 132kV powerline that traverses the surface above the southern longwall panels.

SUBSIDENCE BEHAVIOUR FOR TWO SEAMS OF MINING

Contemporary understanding of the mechanics of multi-seam subsidence at Ashton is presented in Mills and Wilson (2017) based on two seams of mining at Ashton. Observations from the monitoring of the two additional longwalls in the ULD Seam have enhanced the existing dataset for Ashton and confirmed the understanding previously presented for multi-seam mining in two seams. The key findings are summarised in this section.

The results of the ULD Seam monitoring show that subsidence behaviour falls into two categories depending on the relative geometries of the mining in the two seams. Subsidence behaviour can be categorised as general background subsidence behaviour for areas remote for pillar edges with tilts and strains of similar magnitude to those observed for single seam mining and behaviour near goaf edges where temporary or permanent stacked goaf edges are formed.

The monitoring dataset also provides significant insight into the mechanics that drive the magnitude and the distribution of subsidence movements in the multi-seam environment at the site including:

- the difference in behaviour between strata that is undisturbed by previous mining and strata that has already been subsided (disturbed or modified)
- increased incremental vertical subsidence as a percentage of the second seam mining height
- the concentrating effect of mining under overlying goaf edges on tilts and strains
- effect of mining direction on subsidence behaviour above solid goaf edges
- recovery of latent subsidence from the overlying seam.

Latent subsidence is a term referring to subsidence which did not occur during mining of the first seam owing to the support provided by nearby pillar or abutment edges.

When the second seam mines under the chain pillars and other abutment edges, the strata above the first seam chain pillar and abutment edges is disturbed and the supporting effect around the edges is lost with additional subsidence occurring at these edges. The latent subsidence increment has both vertical and horizontal components. These components affect tilt and strain levels measured at the surface.

General Subsidence Behaviour

Figure 2 shows a summary of subsidence monitoring results from XL5 Line, the main cross-panel subsidence line over all the southern panels at the completion of mining the sixth longwall (Longwall 106A) in the ULD Seam.

The vertical profiles show the effects of increasing overburden depth and a change from supercritical width caving behaviour to more critical width behaviour with a smoothing of the profile shape and reduction in the magnitude of total subsidence. The areas of latent subsidence are highlighted in the incremental profile shown in Figure 2.
Multi-seam subsidence zones across the panel width

Figure 3 shows the vertical subsidence profiles over the first longwalls in the PG and ULD seams to highlight the main zones of ground behaviour. Average or more typical magnitudes of incremental subsidence expressed as a percentage of ULD Seam mining height are also shown.

Where panels in two seams overlap in the offset geometry, mining a second seam below disturbed ground causes maximum incremental subsidence in the order of 70-83% of the second seam mining height remote from the pillar edges.

Near goaf edges in the overlying seam, maximum incremental subsidence is observed to increase as latent subsidence is recovered. Vertical subsidence as high as 92% of the second seam mining height is apparent when latent subsidence occurs, but the magnitude of this additional subsidence is not a function of the seam mining height in the lower seam, but rather a function of recovering subsidence that did not occur when the first seam was mined due to the presence of the chain pillars. Latent subsidence of approximately 300-400mm from the PG Seam edges was observed during the mining of the ULD Seam.

In general background areas, remote from pillar and goaf edges, the maximum values of tilt and strains are typically of a similar or lower magnitude to the tilt and strains measured for the first seam mined despite the greater total of vertical subsidence. This behaviour is thought to be due to a general softening or reduction in ‘shear stiffness’ of the overburden due to the multi-seam mining resulting in a difference in behaviour between strata that is undisturbed by previous mining and strata that has already been subsided (disturbed or modified).

**Strata compression above chain pillars**

Incremental vertical subsidence above the ULD Seam chain pillars is much greater than the elastic strata compression observed above PG Seam chain pillars when mining in the PG Seam. This is the result of compression of the disturbed ground above the lower seam chain pillar and the reduced stiffness of this ground from the previous episode of subsidence.

**Behaviour at stacked goaf edges**

The ULD Seam longwalls formed stacked or almost stacked edges at several locations. Consistent with previous experience, the measured tilt and strain near the stacked goaf edges are significantly elevated compared to tilt and strain observed in areas remote from goaf edges, particularly when the deeper seam undercuts the upper seam.

The direction of mining in the second seam under an existing goaf edge has a significant influence on the surface effects that develop. Mining from a goaf under solid leads to a stacked goaf edge that produces very high tilts and strains and much higher than the general background values. Mining from
solid to under a goaf produces en-masse subsidence with tilts and strains that are comparable to general background levels.

**Mining from under goaf to under solid**

At a stacked goaf edge where the lower seam is mined into solid from below an existing goaf in the upper seam, maximum tilts are observed to be approximately double the maximum tilts observed elsewhere. Horizontal strains are observed to peak at about four times the background levels measured more generally along the panel. These maxima are observed when the goaf edge in the upper seam is undercut to a distance where the caving of the goaf in the lower seam intersects the goaf edge in the upper seam.

The presence of the pre-existing fractures in the overburden from the upper seam mining acts as a preferred separation point to localise deformations from the lower seam mining. Deformations become concentrated on pre-existing fractures with the result that tilt and strain magnitudes are significantly elevated.

Figure 4 illustrates the retreat of the ULD Seam longwalls under the PG Seam goaf edge/solid coal and how the subsiding strata interacts with the overlying goaf edge as the panel retreats.

**Mining from under solid to under goaf**

Where the lower seam is mined initially as a single seam and proceeds to mine under an overlying goaf, a different subsidence behaviour is observed. A wedge of undisturbed strata is observed to subside en-masse. Tilt and strain magnitudes are of similar magnitude to single seam mining.

Figure 5 illustrates the geometries involved and shows how the disturbance caused to the ground by mining longwall panels in the two seams leaves a triangular wedge of largely undisturbed ground above the start of the PG Seam longwall. This triangle of rock subsides gradually en-masse as mining in the underlying ULD Seam progresses.

**Horizontal movements**

The magnitude, direction, and form of horizontal movements observed during mining the ULD Seam are consistent with the horizontal movement observed during mining of the PG Seam. Total horizontal
movements measured for mining in the ULD Seam are typically in the range of 20-30% of the vertical subsidence.

There is a strong similarity in the characteristics and distribution of cross-panel horizontal subsidence movements associated with each longwall panel indicating a consistent mechanism driving horizontal movements. The influences of the offset geometry and latent subsidence recovered from the PG Seam are seen as a regular pattern of incremental horizontal movements associated with mining in the ULD Seam. There is also a strong influence of strata dilation in the development of horizontal movements. This strata dilation causes a general shift in an uphill direction.

The incremental long-panel horizontal movements are characterised by movement toward the approaching longwall face, followed by movement in the reverse direction after the longwall face has passed. This behaviour is observed in single seam mining as well.

Incremental subsidence movements

Figure 6 shows the incremental vertical subsidence and incremental cross-panel horizontal movements and measured above Longwalls 101 to 106A at the time of mining in each panel. The horizontal distance is plotted relative to the tailgate or eastern ULD Seam goaf edge.

Figure 6 shows:

- the vertical subsidence profile has a regular, repeatable form, with a general smoothing and reduction in peak values with increasing overburden depth
- the maximum vertical and horizontal movements occur substantially within the footprint of the active panel
- the influence of the recovered latent subsidence from the PG Seam extends over the disturbed ground of the next panel
- movements over previous panels are generally small and insignificant for practical purposes.

SUBSIDENCE MOVEMENTS FOR THREE SEAMS OF MINING

Monitoring data from XL5 Line and the longitudinal lines at the start of Longwalls 201 and 202 provide insights into, and understanding of, the mechanics of multi-seam mining in three seams. Figure 7 shows the subsidence effects along XL5 Line after the mining of Longwalls 201-203. The incremental vertical subsidence profile, with prominent latent subsidence areas adjacent to the overlying chain pillars, is the mirror image of the incremental profile for the ULD Seam. This is due to the panel offset being in the opposite direction, relative to the panel geometry in the overlying seam.
Figure 7: Summary of the subsidence movements measured on XL5 Line to the end of LW203.
Remote from chain pillars and goaf edges in overlying seams, incremental subsidence ranges 82-88% of ULLD Seam mining height. Maximum values of tilt and strain are similar or of a lower magnitude than those measured for the PG and ULD Seams despite the increased vertical subsidence.

Close to chain pillars and goaf edges in overlying seams, incremental subsidence ranges 96-105% of the height of the seam being mined with the inclusion of latent subsidence from the overlying seam. The areas of latent subsidence adjacent to pillar edges are similar in extent and magnitude to those observed from the ULD Seam mining. The additional vertical displacement from latent subsidence is estimated at 300-500mm. Maximum tilt and strain are higher than over the centre of the panel.

After three seams of mining at Ashton, maximum cumulative subsidence has reached 5.8m. Cumulative subsidence ranges 72-77% of the combined mining heights in all three seams. Maximum incremental subsidence from the ULLD Seam mining is 2.7m representing approximately 105% of the ULLD Seam mining height with the inclusion of latent subsidence from the ULD Seam.

**Behaviour at stacked goaf edges**

Stacked goaf edges associated with mining from goaf to solid were avoided in Longwalls 201-203 to improve longwall conditions during start-up and take-off. However, Longwalls 201 and 202 created a variation of mining direction effects at stacked goaf edges not previously observed when they mined from below a single seam goaf to under goaf in two seams. Mining in the third seam progressed from solid to under a goaf edge for a second time. During the first episode of mining, a wedge-shaped block of overburden subsided en-masse as illustrated in Figure 5. This block remained relatively undisturbed by incremental subsidence effects. A second episode of mining in the third seam led to additional subsidence when incremental mining caused dilation and fracturing of the previously undisturbed strata. Figure 8 shows the mining geometry and the observed subsidence profile.

![Figure 8: Sketch illustrating the geometry and the mechanisms driven by mining direction at start of panel and vertical subsidence profile.](image-url)
The mining layout in the ULLD Seam is directly below the PG Seam goaf leading to stacked edges along both sides of the panel. The PG Seam panel edges had previously been remediated. They were then mined under by a ULD Seam longwall which is offset 60m to the west before being mined under again by the ULLD Seam longwall. Tilt and strain measured on XL5 Line at the panel edges of the PG Seam approach the levels observed at stacked goaf edge levels at other locations where only two seams are mined. Even though the panel edge had subsequently been mined under by a ULD Seam longwall without forming a stacked edge, movements continue to be focussed on the pre-existing fractures in the overburden when a stacked edge is subsequently formed by mining in a third seam. Fracturing in the overburden from the first seam of mining is reactivated and further movements are concentrated or localised at the original fractures. Surface cracks along the PG Seam panel sides opened again during mining of the ULLD Seam longwalls.

**Multi-seam subsidence zones across the panel width**

As a proportion of mining height, the values for incremental subsidence in the centre of the panel, maximum incremental subsidence with latent subsidence effects and cumulative subsidence from the third seam of mining are all generally 5-10% greater than after two seams of mining in the same location.

Figure 9 shows a comparison of the increment profiles against the cumulative profiles for the ULD and ULLD Seams with the magnitudes of incremental subsidence as a percentage of the mining height within the main zones of ground behaviour highlighted in Figure 3.

**Angle of Draw**

The conventional concept of an angle of draw varies depending on the relationship of the extracted panels. The angle of draw outside of the outermost goaf edge is the same as in single seam mining.
The angle of draw for the seam being mined extends much further when the overlying strata has been disturbed or modified by previous mining.

The angle of draw from the outer edge of multi-seam panels based on the depth to the seam of the outer panel edge is similar to single seam mining. Where panels start or finish within the boundary of overlying panels changes to the angle of draw for the upper seam(s) are imperceptible for all practical purposes. No significant change in angle of draw has been observed were multiple panels are aligned at a stacked goaf edge and additional goaf edge subsidence is measured.

Where panels start well below an established goaf, the subsidence and angle of draw at the start line is greater, consistent with a general softening of the overburden strata from the previous episode of subsidence.

Subsidence across panels is observed to extend over overlying goaf areas to the next load bearing pillar or solid coal. These low magnitude movements are a secondary subsidence effect from low-level subsidence or compression of the previously disturbed ground. While this low-level subsidence is generally insignificant, consideration of this interaction is helpful in determining assessment areas for impact and environmental consequence assessments as required by the mining approval process.

**Multi-seam incremental subsidence observations**

Figure 10 shows the incremental vertical subsidence profiles for the Longwalls 201-203. A regular pattern of behaviour is observed similar to the pattern shown in Figure 6 for the ULD Seam panels.

When compared to two seams of mining, the cross-panel incremental profiles indicate a further ‘softening’ of the overburden from the third episode of mining. This softening results in a slightly wider, steeper subsidence trough and greater subsidence as a proportion of mining height in the third seam.

![Figure 10: Incremental vertical subsidence and cross panel horizontal movements for LW201-LW203.](image)

**Horizontal movements**

Figure 10 also shows the incremental cross-panel horizontal movements associated with mining Longwalls 201-203. The magnitude, direction and form of horizontal movements observed are consistent with the horizontal movements observed during mining of the PG and ULD Seam longwalls. The influences of the offset geometry and latent subsidence areas are seen in the profile as a regular
pattern of incremental horizontal movements. The characteristics and distribution of horizontal subsidence movements indicates a consistent mechanism driving the horizontal movements.

Total horizontal subsidence movements are now 1.4m, representing 20-30% of the cumulative vertical subsidence consistent with the experience of the previous mining in the upper seams. Most of the horizontal movement occurs in an up-dip, easterly direction toward the free surface associated with the slope leading down to Glennies Creek.

Comparison of predicted and observed subsidence

Monitoring data indicates the methodology used to forecast the subsidence behaviour for the third seam of mining is providing a reasonable estimation of the measured subsidence effects for impact assessment purposes.

The maximum incremental and cumulative vertical subsidence measured on XL5 Line to date is consistent with forecast (within 1%). The results are well within the 15% allowance for natural variation expected in single seam mining and significantly less than the 20% recommended as performance indicators to set Trigger Action Response Plan (TARP) levels for compliance reporting in accordance with mining approval conditions. Observed tilt and strains, both incremental and cumulative, are all less than the maxima forecast using the Holla approach and the K values derived for general background, stacked goaf edges and undercut goaf edge areas (Mills and Wilson 2017). The main variation from predictions is that areas of maximum latent subsidence are located slightly further to the west than forecast in the moderately to steeply dipping strata.

Estimates of incremental subsidence effects for the ULLD Seam mining are derived from the differences in subsidence movements between the ULD and ULLD Seam mining. However, it is recognised the maximum latent subsidence movements, adjacent to overlying panel or pillar edges, are not necessarily captured on the longitudinal lines at the panel ends and that the calculation of incremental subsidence movements is subjective. With the offset geometry, there is scope for longitudinal lines which are not necessarily at locations of maximum subsidence to provide misleading indications of incremental subsidence as performance indicators. Performance indicators should be based on measured cumulative subsidence movements and not calculated incremental subsidence to overcome this issue.

CONCLUSIONS

The monitoring dataset from Ashton provides significant insight into the mechanics of ground behaviour that drive the magnitude and the distribution of subsidence movements in the multi-seam environment at the site.

Subsidence behaviour from the third seam of mining is consistent and predictable once the various geometry effects are recognised and considered. The patterns of incremental subsidence movements are regular and repeatable.

Ongoing softening of the overburden with each episode of subsidence and recovery of latent subsidence from previous episodes is evident as:

- incremental subsidence as a percentage of seam height is greater than for two seams of mining
- slightly wider, steeper subsidence troughs.

The methodology developed to forecast the subsidence behaviour for the later longwall panels in the second seam and for the first three longwalls in the third seam is providing a reasonable estimation of the measured subsidence effects for impact assessment purposes. The maximum incremental and cumulative vertical subsidence measured is consistent with forecast and the actual levels of incremental and cumulative tilt and strains are all less than the maxima forecast for compliance reporting.

Further refinement and adjustments to the adopted method of forecasting subsidence effects at the site is expected from ongoing monitoring to enable ACOL to continue to effectively manage the risks to health and safety from subsidence.
ACKNOWLEDGEMENTS
The authors wish to thank Yancoal Australia Ltd - Ashton Coal Operations Pty Ltd for permission to present this data.

REFERENCES
GOAF GAS DISTRIBUTION NEAR THE TAILGATE UNDER THREE GATEROAD CONDITIONS

Rao Balusu¹, Bharath Belle² and Krishna Tanguturi³

ABSTRACT: A collaborative project was undertaken to obtain an understanding of the goaf gas distribution profile near the tailgate area of the longwall goaf under 2 gateroad and 3 gateroad mining conditions for improved explosion risk management in gassy mines. Extensive computational fluid dynamics (CFD) modelling studies were conducted and calibrated using operational longwall goaf gas distribution data. The results of the simulations indicated that there is a significant difference in the spread of explosive fringe (or close to explosive range) gas distribution profiles near the tailgate side of the longwall goaf under 2 gateroad and 3 gateroad conditions, i.e. a significant increase in the spread of explosive fringe zone under 3 gateroad conditions. The results indicated that under 3 gateroad conditions, explosive fringe gas distribution extends into the middle roadway between the longwall panels thereby significantly increasing the area of explosive fringe distribution and likely relative increase in explosion risk. Results indicate that the explosive fringe area in the middle roadway can extend up to two pillars behind the longwall face position. The details and results of various modelling investigations including recommended strategies for gas control and minimisation of the spread of explosive fringe gas distribution in the longwall goaf are presented in this paper.

INTRODUCTION

Gas emissions have increased significantly in recent years in some of the Australian longwall mines due to increased gas reservoir size, high production rates and increase in mining depths. Traditionally, Australian coal mines use 2 gateroad system for longwall panels development and extraction. Extensive research work has been carried out previously to develop optimum gas and spontaneous combustion control strategies for 2 gateroad longwall panels (Balusu and Tanguturi, 2019; Balusu et al, 2017; Belle, 2015; Balusu et al, 2011). To manage high gas emissions, some mines had previously introduced 3 gateroad systems in their longwall panels to provide additional dilution capacity in longwall tailgate return and as the key solution for gas management. Another purpose of the 3 gateroad/heading philosophy was to provide continued (24*7) tailgate access for diesel equipment maintenance activities.

Although 3 gateroad system provides additional ventilation dilution capacity for tailgate gas management during longwall extraction, its effect on goaf gas distribution and explosive fringe gas distribution profiles in the longwall goaf areas is unknown. There is a perception that as the 3 gateroad system provides more ventilation capacity than 2 gateroad system for gas dilution in the longwall tailgate return, it would also reduce the explosive fringe gas distribution profile near the tailgate area in the longwall goaf to manage the explosion risk. To investigate this issue, a collaborative project was undertaken to conduct detailed computational fluid dynamics (CFD) modelling studies to obtain an understanding of the explosive fringe gas distribution profiles near the tailgate area of the longwall goaf under 2 gateroad and 3 gateroad conditions.

The CFD models were built and solved using the commercial CFD software tool ANSYS and the numerical fluid flow solver FLUENT. The main focus of the project during the initial phases was to obtain a fundamental understanding of goaf gas distribution and the extent of explosive fringe areas in longwall goaf under field site conditions using CFD modelling techniques. In the later phases, the CFD modelling studies were also used to investigate the effect of various control options and ventilation strategies to minimise the spread of explosive fringe distribution on tailgate side of the longwall goaf as well as in the middle roadway (B heading) under 3 gateroad conditions. Based on the results of these investigations, appropriate strategies have been identified for gas management and minimization of the spread of explosive fringe gas distribution in the longwall goaf under field site conditions.

---

¹ CSIRO Minerals, PO Box 883, Kenmore, QLD 4069, Australia. Email: rao.balusu@csiro.au, Tel +61 7 3327 4614
² Anglo American Metallurgical Coal, Brisbane, QLD 4000. Email: bharath.belle@angloamerican.com Tel: +61 7 3834 1405
³ CSIRO Minerals, PO Box 883, Kenmore, QLD 4069, Email: krishna.tanguturi@csiro.au, Tel +61 7 3327 4498
GOAF GAS DISTRIBUTION PATTERNS AT THE TAILGATE AREA

The CFD models were developed with the geometries of operational longwall panels at a mine site with 2 gateroad and 3 gateroad layouts covering 1.0 km length of longwall goaf. The actual floor contours of the longwall panels at the field site mine were used for development of the CFD models. The schematics of 2 gateroad and 3 gateroad layouts and ventilation systems used in CFD modelling studies are shown in Figure 1. The CFD models with 3 gateroad layout included the working longwall panel goaf as well as the adjacent sealed goaf to simulate goaf gas distribution in the middle roadway (i.e. B heading) between the adjacent longwall panels. In the 3 gateroad longwall panels, there are two tailgate headings, designated as A and B headings, with A heading for additional intake airflow for gas dilution and B heading for longwall return airflow. Velocity inlet boundary conditions were specified at the maingate (MG) intake roadways and other roadways as shown in Figure 1, such that the total ventilation quantity of 50 m³/s flows across the longwall face. At the tailgate (TG) return, an outflow boundary condition was specified in the modelling simulations. Longwall inbye airflow of 35 m³/s was from an inbye mine air cooling shaft to manage the steep geo-thermal gradient and homotropal conveyor heat in high production longwall face.

![Figure 1: Longwall ventilation systems – 2 and 3 gateroad layouts used in CFD models](image)

The working seam thickness is 2.6 m, which also represents the face height. The longwall panel width is 300 m and the roadway width on both maingate (MG) and tailgate (TG) sides of the face is 5.4 m. The goaf height up to 80 m above the working seam and the floor strata down to 10 m below the working seam is included in all the models. In these models, MG and TG cut-throughs of 5 m in width have also been incorporated, and these cut-throughs were spaced at 100 m intervals along the panel. A number of goaf gas drainage holes were also incorporated into the CFD models to replicate goaf gas drainage conditions at the field site.

In the base case simulations, longwall panel gas emissions and goaf gas drainage conditions of the field site were used. The total gas emissions into the longwall goaf were around 9,000 l/s with 98% methane (CH₄). The total goaf gas drainage rate was around 8,000 l/s, with gas concentration in different vertical goaf holes varying between 80% and 95%. In addition, the total gas emission into the adjacent sealed panel goaf was around 1,000 l/s with 800 l/s of adjacent goaf gas drainage from the sealed panel. The mine utilises closely spaced tailgate goaf holes, adjacent sealed panel goaf drainage and deep goaf gas drainage strategies as the primary controls for longwall tailgate gas management. The ventilation layout and airflows as shown in Figure 1 were used in the base-case simulations. All the mining and ventilation parameters including the goaf gas drainage conditions were same in both 3 gateroad and 2 gateroad simulations. In 2 gateroad panels, goaf gas distribution in the working longwall panel goaf only was simulated, as there was no middle roadway between the adjacent panels. However, in 3 gateroad panels two adjacent panels were included in the modelling simulations in order to incorporate the middle roadway (B heading) between the panels. In the base-case simulations, the last three cut-throughs outbye of the longwall face were not sealed off between A and B headings to allow airflow from A heading into the B heading for gas dilution in the longwall tailgate return, as per the field site conditions. In these simulations, brattice cloth seals were erected in the outbye two cut-throughs to force some air through the last cut-through, i.e. around 15% of the total tailgate intake dilution airflow, and also to allow some leakage through the other two cut-throughs. All other cut-throughs outbye of these last three cut-throughs were sealed off completely.
Figure 2: Methane distributions patterns in longwall goafs and in B heading – Base case

The results of the base-case simulations for 2 gateroad and 3 gateroad layouts are presented in Figure 2(a) and 2(b) respectively, showing methane gas distribution patterns in the working panel goaf, and gas distribution in the sealed panel goaf areas as well as in B heading for 3 gateroad layout. A close-up view of the methane concentration distribution patterns near the tailgate area of the longwall goaf under 2 gateroad and 3 gateroad layout conditions are presented in Figure 3(a) and 3(b) respectively. Methane gas distribution in B heading under 3 gateroad conditions is also shown in Figure 3(b). In the methane gas distribution figures, the red colour indicates higher gas concentration and the blue colour indicates lower methane gas concentration. Gas concentration values at some key locations are also shown in numerical values as ‘annotations’ on the figures for clarity.

The results of the simulations indicated that there is a significant difference in the spread of explosive fringe gas distribution profiles in the longwall goaf under 2 gateroad and 3 gateroad conditions, i.e. a significant increase in the spread of explosive fringe (or close to explosive range) zone in the goaf under 3 gateroad conditions. The results presented in Figure 3(b) indicate that under 3 gateroad conditions, explosive fringe (or close to explosive range) gas distribution also extends into the middle roadway between the longwall panels, thereby significantly increasing the area of explosive fringe gas distribution. Results indicate that the explosive fringe area in the middle roadway can extend up to two pillars behind the longwall face position. Beyond the two pillars, the methane gas concentration in B heading quickly rises above the explosive range, i.e. turns into methane rich region. Field gas monitoring data from B heading also showed that methane gas concentration values were in the explosive range only up to two pillars behind the face and then the gas concentration steeply rises to 50% to 60% levels.

Additional base case simulations were also carried out with slightly different gas emission rates into the working and sealed goafs and with different goaf gas drainage rates from both working and sealed longwall panels. There is only a marginal difference in the B heading gas distribution patterns in various base case simulations. A field gas monitoring programme was implemented during 3 gateroad longwall panel retreat operations at the field site to measure actual gas concentration values in B heading and its changes with face retreat. Field measurements also showed that methane concentration rises above the explosive range at two pillars behind the longwall face. Comparison of
the simulated results and field measured values show that simulated results are in close agreement with field measured data.

![Methane distributions patterns near tailgate and in B heading – Base case](image)

**Figure 3:** Methane distributions patterns near tailgate and in B heading – Base case

**B-HEADING CLOSING OPTIONS TO MINIMIZE THE SPREAD OF EXPLOSIVE FRINGE**

A number of control options involving closing/sealing off B heading at different locations were proposed to minimize the spread of explosive fringe area in B heading under 3 gateroad conditions. The modelling studies were carried out to simulate the effectiveness of various control options in minimizing the spread of explosive fringe area. To replicate sealed goaf conditions at the field site, CO2 gas was also introduced into the sealed longwall goaf in all these simulations. The details of the control strategies and the results of various simulations are presented in the following sub-sections.

**(a) B heading closed at inbye of the last cut-through (Location 1)**

It was proposed to close off B heading to control/prevent explosive gas composition in B heading. In order to investigate the effect of closing off B heading, several CFD modelling simulations were carried out by closing B heading at various locations. In the first simulation, B heading was closed off at inbye of the last ventilation cut-through location (shown as ‘Location 1’ in the following figures) to simulate its effect on the gas distribution profile in the longwall goaf and in B heading. All other conditions, such as ventilation airflow on the longwall face, goaf gas emissions and goaf gas drainage rates were kept at the same values as those in the base case simulations.

Results of the simulation with B heading closed at location 1 (inbye of the last ventilation cut-through between A and B headings) are presented in Figure 4, showing a close-up view of methane and oxygen gas distribution in the longwall goaf areas near the tailgate as well as in B heading. Results presented in Figure 4(a) indicate that methane gas concentration in some sections of B heading continue to be closer to the explosive range. Results also indicate that the methane gas concentration in B heading rises above the explosive range (i.e. into methane rich range) with the face retreat.

A comparison of the methane gas concentration distribution patterns in the longwall goaf near the tailgate area as well as in B heading with and without closing off the B heading are presented in Figure 5. Results of these simulations indicate that closing off B heading does not significantly change the methane gas distribution patterns in B heading and results in only a marginal difference in methane gas concentration profiles. Methane gas distribution in B heading at inbye of the longwall face is close to the explosive range in both cases. Analyses of the results also indicate that a higher methane gas concentration fringe behind the chock-shields moves closer to AFC tailgate drive motor area and tailgate corner when B heading is closed off. Therefore, the option of closing off B heading at location
1. i.e. at inbye of the last ventilation cut-through, is not recommended if 3 gateroad ventilation system is deployed for longwall gas management.

Figure 4: Methane distribution near tailgate – with B heading closed at Location 1

Figure 5: Methane distribution near tailgate – without and with B heading closed

(b) B heading closed at inbye of the second last cut-through (Location 2)

In this simulation, B heading was closed off at inbye of the second last cut-through location (shown as ‘Location 2’ in the following figures) to simulate its effect on the gas distribution profile in B heading. All other conditions, such as ventilation airflow on the longwall face, goaf gas emissions and goaf gas drainage rates were kept at the same values as those in the base case simulations.

Results of the simulation with B heading closed at location 2 (inbye of the second last cut-through between A and B headings) are presented in Figure 6(b), showing methane gas distribution patterns...
near the tailgate area of the longwall goaf as well as in B heading. Results indicate that methane gas concentration in some sections of B heading is still very close to the explosive range. Analyses of the results also indicate that moving the seal location in B heading towards the second last cut-through (i.e. one pillar back) resulted in the explosive range gas composition zone extension by one more pillar length towards the outbye side of the longwall panel. Therefore, the option of closing off B heading at the second last cut-through is not recommended, as this would significantly increase the explosive gas composition risk in the longwall panel.

A comparison of the close-up view of methane gas concentration distribution patterns with B heading closed at two different locations (Location 1 – inbye of last cut-through, and Location 2 – inbye of second last cut-through) are presented in Figure 6. Results of the simulations indicate that closing off B heading at any location does not significantly change methane gas distribution patterns in B heading and results in only a marginal difference in methane gas concentration profiles along the B heading roadway. Methane gas distribution in B heading at inbye of the seals location is close to the explosive range in both cases. As discussed earlier, sealing of B heading at an outbye location far from the face (i.e at location 2) is not at all recommended, as this option increases the gas explosion risk.

![Methane distribution near tailgate – with B heading closed at Locations 1 & 2](image)

**Figure 6: Methane distribution near tailgate – with B heading closed at Locations 1 & 2**

(c) **B heading closed at all cut-through locations**

In this simulation, B heading was closed off at all cut-through locations to simulate its effect on the gas distribution profile in B heading. All other conditions, such as airflow on the longwall face, goaf gas emissions and goaf gas drainage rates were kept at the same values as those in the base case simulations. Results of the simulation with B heading closed at all cut-through locations are presented in Figure 7(b), showing close-up view of methane gas distribution patterns near the tailgate area of the longwall goaf as well as in B heading. Results indicate that methane gas concentration in different sections of the B heading is close to the explosive range up to four pillars behind the longwall face. Further into the inbye goaf area, the methane gas concentration in B heading rises above the explosive range (i.e. turns into methane rich region).

A comparison of the close-up view of methane gas concentration distribution patterns with B heading sealed off at all cut-throughs and in the base case (without any seals in B heading) are presented in Figure 7. Analyses of the results indicate that sealing off B heading at all cut-through locations results in a significant change in the methane gas distribution profile along the B heading, with ‘close to explosive range gas composition’ zone length in B heading extending up to four pillars behind the longwall face. Therefore, the option of closing off B heading at all cut-throughs is not recommended, as this would significantly increase the explosive fringe gas distribution area risk in the longwall panels.
It is to be noted here that none of the options of closing off B heading at different locations offered any significant benefits in terms of reducing the spread of explosive fringe gas distribution in B heading. In fact, some of the options of closing off B heading resulted in significant increase in the spread of explosive fringe gas distribution zone. Therefore, the option of closing off B heading is not recommended, as this would significantly increase the explosive fringe gas distribution area risk and uncertainty in the longwall panels.

![Methane distribution near tailgate – without and with B heading closed at all C/T’s](image)

**Figure 7: Methane distribution near tailgate – without and with B heading closed at all C/T’s**

**VENTILATION STRATEGIES TO MINIMIZE THE SPREAD OF EXPLOSIVE FRINGE**

A number of ventilation control strategies involving closing off outbye cut-throughs at different locations were proposed to minimize the spread of explosive fringe area in B heading under 3 gateroad conditions. CFD modelling studies were carried out to simulate the effectiveness of various control options in minimizing the spread of explosive fringe area. The details of the ventilation control strategies and the results of simulations are presented in the following sub-sections.

*(a) Changes in tailgate ventilation dilution strategy – more airflow through the last cut-through*

In the base case simulations, the last three cut-throughs outbye of the longwall face between A and B headings were not closed off completely to allow airflow from A heading into B heading for gas dilution in the longwall tailgate return, as per the field site conditions. Brattice cloth seals were erected in the outbye two cut-throughs to force some air to flow through the last cut-through (around 15% of the total tailgate intake dilution airflow), and also allowing some leakage through other two cut-throughs. All other cut-throughs outbye of these last three cut-throughs were closed off completely.

To investigate the effect of increased airflow through the last cut-through on explosive fringe gas distribution in B heading, simulations were carried out with different brattice cloth resistance values in the outbye two cut-throughs. In this simulation, resistance of brattice cloth in the two outbye cut-throughs was increased to force more airflow through the last cut-through between A and B headings, with around 27% airflow in the last cut-through close to the face. Results of the simulation indicate that the methane gas distribution in B heading behind the longwall face is still in the explosive range even with 27% airflow through the last cut-through. Analyses of the results also indicate that although methane gas concentration in B heading outbye of the face decreased significantly with increased airflow through the last cut-through, it is still in the explosive range outbye of the longwall face.

*(b) Changes in tailgate ventilation dilution strategy – only one last cut-through open*
In this simulation, the second and third last cut-throughs were also closed off completely and all the tailgate intake airflow would flow from A heading to B heading through the last ventilation cut-through near the tailgate, i.e. only one last cut-through was open for airflow between A and B headings with all other cut-throughs closed off completely. All other conditions, such as airflow on the face, goaf gas emissions and goaf gas drainage rates were kept at the same values as those in the base case simulations. Results of the simulation with only the last cut-through open for dilution airflow from A heading to B headings are presented in Figure 8(b), showing close-up view of methane gas distribution near the tailgate area of the longwall goaf as well as in B heading. Results indicate that the extent of the explosive gas fringe in longwall goaf has reduced significantly with this control strategy.

A comparison of the gas concentration distribution patterns in B heading with two different ventilation practices are presented in Figure 8. In the first case, the last 3 cut-throughs between A and B headings outbye of the longwall face were not closed off completely, i.e. outbye two cut-throughs were sealed with brattice cloth only, resulting in only 15% airflow through the last cut-through near the face. In the second case, only the last cut-through was open for airflow between A and B headings with all other cut-throughs closed off completely. Results of this simulation indicate that in the first case, methane gas distribution in B heading up to two pillars outbye of the longwall face would also be in the explosive range. In the second case, methane concentration in B heading outbye of the last cut-through is below 2%, thereby significantly reducing the length of the explosive fringe zone in B heading.

![Figure 8: Comparison of methane distribution at TG – 3 outbye C/T’s open vs 1 C/T open](image)

Analyses of the results indicate that the ventilation practice of having more cut-throughs open (not closed off completely) outbye of the longwall face results in the explosive range zone in B heading extending up to four pillars, i.e. two pillars outbye of the face and two pillars behind the face, which would significantly increase the risk in operating coal mines. The results indicate that the practice of not closing off the cut-throughs outbye of the longwall face significantly increase the risk. Therefore, it is strongly recommended that mines should implement the ventilation control strategy of having only one last cut-through open for airflow between A and B headings for gas dilution. All other outbye cut-throughs between A and B tailgate headings should be closed off completely.

**CONCLUSIONS AND RECOMMENDATIONS**

The results of the CFD modelling simulations indicate that there is a significant difference in the spread of explosive fringe gas distribution profiles in the longwall goaf under 2 gateroad and 3 gateroad conditions, i.e. a significant increase in the spread of explosive fringe (or close to explosive range) zone in the goaf under 3 gateroad conditions. The results indicate that under 3 gateroad conditions, explosive fringe (or close to explosive range) gas distribution also extends into the middle
roadway (B heading) between the longwall panels, thereby significantly increasing the area of explosive fringe gas distribution. Results indicate that the explosive fringe area in the middle B heading can extend up to two pillars behind the longwall face position. The modelling results also indicate that methane gas concentration in B heading quickly rises above the explosive range (i.e. into methane gas rich range) beyond that two pillars inbye of the longwall face. Field gas monitoring data from B heading also showed that methane gas concentration values were in the explosive range only up to two pillars behind the face and then the gas concentration steeply rises to 50% to 60% levels.

The modelling simulations indicate that closing/sealing of B heading at different locations close to the face does not significantly change methane gas distribution patterns in B heading and results in only a marginal difference in methane gas concentration profiles along the B heading roadway. It is also to be noted that sealing of the B heading at all cut-through locations would result in a significant change in the methane gas distribution profile along the B heading, with 'close to explosive range gas composition' zone length in B heading extending up to four pillars behind the longwall face. Therefore, the option of closing off B heading is not recommended, as this would significantly increase the explosive fringe gas distribution area risk and uncertainty in the longwall panels.

The modelling simulations indicate that the ventilation layout with only one last cut-through open for airflow between A and B headings for gas dilution would result in a minimum length of the explosive range zone in B heading. Results indicate that the practice of not closing off all the remaining cut-throughs outbye of the longwall face for any reason/operational convenience would result in an extension of the explosive range zone in B heading. Therefore, it is strongly recommended that mines should implement the ventilation control strategy of having only one last cut-through open for airflow between A and B tailgate headings for gas dilution. All other outbye cut-throughs between A and B tailgate headings should be closed off completely.

REFERENCES


ACTIVE ROADWAY EXPLOSION BARRIER EVOLUTION IN COAL MINING

Arend Späth¹ and Bharath Belle²

ABSTRACT: Methane gas and coal dust are an ever-present source of risk during underground coal mining operation. Methane gas and coal dust can create a combustible atmosphere. Adding a high enough energy source might ignite this combustible atmosphere and lead to an explosion. The prevention of methane gas and coal dust accumulation during mechanised cutting and crushing has evolved to an extent that might be accepted as a state of safe mining. Historic events suggest that methane gas ignitions can occur despite a general preventative practice being in place. Active explosion barriers are considered a last resort once methane gas and/or coal dust explosions occur. Initially these barriers were developed to protect the continuous miner operator from those methane gas and coal dust ignitions occurring at the coal cutting face. The possibility of an explosion mitigation control being flexible enough to react independent of the scale of the explosion, sparked the evolution of the active explosion barrier technology. Over twenty years of continuous research and development, operational experience and numerous success stories, resulted in a technology being adaptable to different types of coal mining operations such as longwall and room and pillar operations. This paper reviews this evolution of the active explosion barrier technology from early prototypes to modern mobile barrier modules that are readily deployed as a safety control system. The paper furthermore outlines the requirements and techniques that influence the function of these barriers.

INTRODUCTION

Underground coal mine explosions can be addressed in two ways, through prevention or mitigation. Various mining sectors have emphasized prevention and mitigation in different ways. Figure 1 depicts the recognized approaches to underground coal mining explosion protection. Despite successful practices for the prevention of underground coal mine, explosions do still occur. Some mining countries around the world have developed and implemented mitigation systems such as passive or active explosion barriers. Explosion barrier systems do not prevent explosions but mitigate the propagation of the explosion shockwave and flame to other areas of the mine. The use of active explosion barrier systems enhances the safety of underground coal mines. Active or triggered explosion barriers will detect the arrival of the shockwave or flame front of the explosion and release inert material to suppress or extinguish the flame. The active barrier technology is readily available, and the New South Wales Department of Primary Industries’ Mine Safety Operations Division has considered active explosion barriers since 2001 (MSOD, 2001).

In 2003, the South African Department of Minerals and Energy’s Chief Inspector issued a directive making it mandatory to use active explosion barriers on mechanized cutting machines when cutting through burnt coal (Mzisa, 2003). In 2020, The South African Mine Health and Safety Inspectorate included active explosion barriers as a substitution and safety enhancement to the passive explosion barriers in their Guidelines for the Compilation of Mandatory Code of Practice for the Prevention of Flammable Gas and Coal Dust Explosion in Collieries (DME, 2020). The meaning of passive and active barrier system is related to the energy source to activate or “trigger” a barrier system to suppress an explosion. Passive barriers such as stone dust bag barriers, therefore do not require a source of energy to detect an explosion and are activated by the pressure wave that leads any explosion. Active barriers are equipment that requires a source of energy to detect an explosion. Unlike the passive barrier they are independent of the physical dimension of an explosion and react based on the logic and mechanism that has been set for this particular device. Also in 2020, the Queensland’s Resources Safety and Health Recognized Standards included active explosion barriers to mitigate the risk of underground explosions (QRSH, 2020).

¹ PhD cand., Department of Mining Engineering, University of Pretoria. Email: arend.spaeth@explospot.com
² PhD, School of Minerals and Energy Resources Engineering, University of New South Wales. Email: bharath.belle@angloamerican.com
The active explosion barriers have evolved over the years to achieve both the functional requirements (explosion suppression) and these requirements were introduced by their operational environment. As part of their evolution the active explosion barriers have become more compact, robust and feasible to operate. The active explosion barrier’s characteristic to be implemented as close as possible to the ignition source as well as their mobility for the safety and operational benefits (Spaeth et al, 2017). As an example, the NSW Department of Minerals Resources has created a guideline to support the development, implementation and assessment of underground explosion suppression systems (MSOD, 2001). The document describes the elements to be reviewed, when analysing which type of suppression barrier utilized as depicted in Figure 2. According to the guideline, the explosion suppression management system should be integrated into the mine safety system. The key components of the document include; are; design, purchasing/construction, installation, maintenance and operation. These components have been adopted in this paper as a reference to compare common explosion barriers.

**Figure 8: Underground coal mining explosion protection**

Explosion barrier means—

a) a barrier constructed, installed and maintained in compliance with a recognised standard for barriers known as explosion barriers; or
b) another barrier that achieves a level of risk that is equal to or better than the acceptable level of risk achieved by a barrier mentioned in paragraph (a).

The RS21 defines an active barrier system as "...Is a electronically triggered device or system used to contain and suppress an explosion (methane and/or coal dust explosions) and a fire (e.g. conveyor fire). The device consists of a configuration of sensors, controller(s) and canisters filled with inert suppression material. The inert suppression material is able to disperse and create a gapless barrier to contain and suppress any propagating explosion flame. An active explosion barrier disperses the inert suppression material in the opposite direction as the movement of the explosion, thereby having the highest mitigation effect. An Active Explosion Barrier is designed in such a way as to be functional in the least possible distance from any assumed ignition source..."

PREVENTION OF COAL DUST EXPLOSION USING STONE DUST APPLICATION

The primary method of preventing the propagation of a coal dust explosion is the application of stone dust in all roadways. Traditionally the quantity of stone dust to be used was dependent on a sliding scale based on the volatile matter of the coal being mined. In the late 1980s research showed that if a standard of 80% total incombustible content (TIC) was reached, the stone dust/coal dust mixture would not sustain an explosion (Phillips, 2019). In the UK, the need for stone dusting is set out in the Precautions Against Inflammable Dust Regulations, 1956 as amended. Very basically the first 30 feet from the coalface is exempt and all other roadways that need to be ventilated “shall contain not less than the minimum percentage of incombustible matter determined in accordance to these regulations”. As far as can be determined (most recent data, 2002) the UK still uses a sliding scale to determine the minimum percentage of incombustible matter according to the volatile matter content of the coal being mined is shown below in Table 1. The basis for the 85 %TIC value or the original explosion prevention test data is not readily traceable.

<table>
<thead>
<tr>
<th>Volatile Matter of coal (VM, %)</th>
<th>Minimum %TIC required</th>
</tr>
</thead>
<tbody>
<tr>
<td>Not exceeding</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>50</td>
</tr>
<tr>
<td>22</td>
<td>55</td>
</tr>
<tr>
<td>25</td>
<td>60</td>
</tr>
<tr>
<td>27</td>
<td>65</td>
</tr>
<tr>
<td>30</td>
<td>68</td>
</tr>
<tr>
<td>32</td>
<td>70</td>
</tr>
<tr>
<td>35</td>
<td>72</td>
</tr>
<tr>
<td>&gt;35</td>
<td>75</td>
</tr>
</tbody>
</table>

INDEPENDENT VALIDATION OF ACTIVE BARRIER SYSTEM PERFORMANCE

Following the initial work to determine the requirements to prevent a coal dust explosion, ways of suppressing an explosion (should it occur) were investigated such as passive and active explosion barriers. Significant amount of explosion research and development work has been carried out over the past four decades at the South African Kloppersbos Explosion Testing Facility, including stone dust bag barriers, aero-dust systems and active barrier systems under 10 m, 20 m and 200 m test galleries. By studying high speed photography and real explosion it is realized that even an explosion...
of whatever size there is a growing phase and a destruction phase as shown in Figure 3. The growing phase lasts for a few milliseconds and is dependent on the fuel that is available.

![Speed vs Distance Graph](image)

**Figure 3: Typical explosion characteristics in Kloppersbos 200 m test gallery**

Typical acceptance criteria of explosion barrier system include the following elements:

- Stop the explosion from spreading.
- Minimise the formation of toxic gases
- Eliminate pressure fronts
- Temperatures < human tolerance.

As in passive stone dust bag explosion barrier system, active barrier systems were tested extensively at the South African Kloppersbos Fires and Explosion Suppression system as early as 1998. Later the Active barrier system was deployed over the continuous miner Section at the Thermal Coal operations and further inde-pendent tests carried out in China, where it has become mandatory control for explosion prevention system on the longwall face.

The 200-m tunnel at Kloppersbos provides a means of conducting large-scale evaluations and assessments of explosion barrier performance and other requirements that cannot be done economically by other means. Figures 4 and 5 shows the unsuppressed explosion and active barrier suppressed explosion respectively at Kloppersbos. Similarly, Figure 6 shows the temperature increase trend of an unsuppressed and active barrier suppressed explosion at the continuous miner operator cabin position evaluated at Kloppersbos. The plot shows that the ExploSpot system was successful in suppressing the explosion from propagating when mono ammonium phosphate powder was used as the suppression material. Also, the temperature rise was less than 100 °C, in less than 2 seconds, the pressure rise was less than 0.5 bar with significantly reduced explosion injury to the worker.

Further independent tests done at Kloppersbos in 2005 by the CSIR and SIMTARS (SIM 05 04 02 CSIR) and the results are briefly discussed hereafter. The plot displayed in Figure 7 shows a typical successful suppression of an explosion at the 90-m position along the tunnel. In this case, the explosion was classified as strong in terms of the protocol. Both of the flame sensors at the 10 m and 110 m positions show no flame was present at that position while all sensors placed ahead of the barrier showed the presence of flame during the test, indicating positive flame suppression by the system.
In 2001, additional independent coal dust explosion tests done in Kloppersbos by the CSIR (2001). The test validations included different baseline explosions with and without coal dust:

- Baseline 1: 75 ± 1 m$^3$ methane/air mixture without coal dust
- Baseline 2: 75 ± 1 m$^3$ methane/air mixture with coal dust

In the unsuppressed explosion, the flame front reaches a distance of 180 m within 750 milliseconds, while the flame front, with the system installed at 30 m from the end of the tunnel, does not reach 50 m. The test results in the 200-m Kloppersbos tunnel were extrapolated to design the active suppression protection system for longwall mining.

For the ExploSpot system evaluation in the 200 m test tunnel incorporated both explosions to evaluate the performance of the system. For the Baseline 2 explosion, coal dust is distributed on the floor and shelves of the tunnel (for 60 m after the membrane position). This results in a methane-initiated coal dust explosion. The test sequence included the placement of the Active Barrier as indicated in Figure 8 below.

In all the tests the system was successful in suppressing flame propagation. In each case the performance of the system can be classified as “stopped on the spot”, i.e. the flame was stopped at the position at which the system was placed. The active barrier successfully suppressed propagating coal dust flames approaching the barrier at flame speeds varying from 24.4 m/s to 62.2 m/s.
The purpose of the design of a suppression barrier is to suppress an explosion as quickly and effectively as possible. Factors to take account of when designing an explosion barrier could be:

1. The assumed strength of an explosion (kPa)
2. Distance from cutting face (m)
3. Obstacles within the area e.g. ventilation ducting, ventilation control devices (VCDs)
4. General requirements of the explosion barrier. e.g. power supply

The possible strength of an explosion is of particular interest when considering passive explosion barriers. According to du Plessis and Vassard (1999) an explosion could be categorized into “weak”, “medium” “strong” or a “supplement” explosion for a specific test gallery in South Africa. The description used does not refer to the amount of damage that could be caused, but rather reflects the flame speed and pressure of an explosion. According to du Plessis et al (1995) a standard explosion would be comparable to a 36 m² volume of 9% methane/air creating a wind pressure of approx. 25 kPa. A test gallery generating a standard explosion would furthermore require 144 kg of coal dust that would be distributed on the floor for a distance of 192.5 m. A strong explosion would be considered to be similar to the standard explosion but using 25% more (192 kg opposed to 144 kg for a standard explosion) coal dust. The dynamic pressure would then increase to approx. 50 kPa. A weak explosion would be comparable to the standard explosion but using a 200 J igniter to activate the methane gas explosion. This results in an explosion of approx. 15 kPa dynamic pressure. The passive barriers commonly used are amongst others: concentrated stone dust barriers and distributed stone dust barriers, concentrated water barriers and distributed water barriers. The same research reported (amongst others) that all passive barriers have been proven effective. The basic working function of any passive barrier is based on enough wind pressure (dynamic pressure) to activate the control. An active explosion barrier in turn is stated to be independent of the strength of an explosion Spaeth et al (2017). These systems depend on detection electronics and trigger a suppression reaction based on the occurrence of an explosion. Due to the calibration of an active explosion barrier, there is no risk of an explosion being too slow or too fast. An explosion being too slow (flame speed) would mean that an active barrier is being triggered due to the detection of an explosion and then discharging the suppression agent prior the flame reaching that barrier. An explosion being too fast would, on the other hand cause, the barrier to discharge only once the explosion has passed the barrier. This would cause such a system to be inefficient. Multiple tests such as described by du Plessis and Spaeth (2014) and du Plessis, Spaeth (2001) have proven the effectiveness of active explosion suppression systems. The tests describe the effectiveness of the active explosion barrier at distances between 5 and 90 m from the explosion source. The flame speeds that were measured at the active barrier vary from 57 to 173 m/s. The maximum flame speed was however not yet determined.

The property of an active explosion barrier to be independent of the dynamic pressure of an explosion enables a mine to use the barrier even at the closest possible distance to cutting face. The closest possible position of an active barrier is at approx. 7 m mounted on a continuous miner. These active barriers are so called machine mounted systems and have been used in underground coal mining operations since 2001(Explospot, 2020) The closest possible location of a passive barrier as recommended by the Department of Minerals and Energy of South Africa is 60 m from a coal face (DME, 2020). A similar guideline is available for New South Wales in Australia (MSOD, 2001). The UK Health and Safety Executive however states the closest distance from the first passive barrier to the cutting face may be 70 m (HSE, 2014). The implementation distance from the face is also recognized as one of the greatest shortcomings of a passive barrier (Cain, 2003). A schematic of a functional implementation area is depicted in Figure 9. The implementation area directly reflects the “harmful” area of a mining operation.

![Functional area of active and passive barriers](image)

**Figure 9: Functional area of active and passive barriers (based on South African Code of Practice)**

The roadway will typically have multiple obstacles. These might be, amongst others, a conveyor or ventilation ducting. These obstacles create a dynamic environment for any explosion suppression...
system. Dynamic in this case refers to a challenge within the barrier design. An obstacle might cause a gap in the barrier development (so-called “shadow zones”) and permit a flame to escape the barrier. A good example is a conveyor. The space between the belts as well as the space between the return belt and the floor might be a location where it is difficult to ensure full coverage of the inerting or suppression powder is possible. The mine’s risk assessment might result in additional passive barriers being deployed along the conveyor. In the case of an active barrier, the suppression outlet should be extended into the shadow zones. Experiments conducted and as described by Spaeth et al (2017) show, that the active barrier must effectively close all gaps within a roadway. Figure 10 shows examples of how the different barriers can be designed to cope with obstacles within the roadway.

![Figure 10: Functional area of active and passive barriers (based on South African Code of Practise)](image)

The design of an explosion barrier should also incorporate general requirements specific to their nature. To name an example, active barriers require a continuous source of power. This source of power will depend on the environment they are located in. Current active barriers can be supplied by either an external power supply or a battery. While the external power supply will limit the barrier to be placed within the physical limits of the power source, a battery-operated barrier can be placed freely at whatever location is applicable. This requirement does not apply to passive barriers. An example of a general requirement specific to passive barriers might be the space requirement (or occupied) within the roadway. Whereas an active barrier only occupies between 1 and 3 m³ (for a 2.4 m x 7 m) roadway section, a passive barrier will occupy a much larger area. In the case of a comparable section and assuming a primary barrier is made up of 4 sub-barriers with a stone dust density of 1.2 kg/m³, one hundred (100) 6 kg bags would be required for each sub-barrier. This in turn results in an occupied space of at least 6.7 m³ per sub-barrier. According to the South African Guidelines for the Compilation of Mandatory Code of Practice the Prevention of Flammable Gas and Coal Dust Explosion in Collieries (DME, 2020) a roadway with a height of 3 – 3.5m will lose up to 0.5m vertical clearance if using for bagged barriers. A higher cut roadway will always result in bags being suspended at a height of 3.0 m above the floor. This factor is especially worth noting when considering the available space for moving machinery.

**PURCHASING / CONSTRUCTION**

Procurement is a dominant factor when evaluating any equipment to be used in an operation environment. In the case of equipment used in a mining environment it would be ideal to determine the costs based on the lifecycle of mine. The cost of a single component e.g. one stonedust bag or one active barrier cannot be generalized. The cost may vary according to country, a mine or a mining company. It should be assumed that a single passive barrier costs a fraction of one active barrier. The passive barrier comprises of e.g. stone dust and some sort of container (e.g. bag with hook). In some cases, a barrier could also include a support to fasten the barrier to the roof (e.g. water trough). An active barrier will however be a configuration of sensors, suppression canisters containing a suppression agent, piping, nozzles, controller and power supply. The suppression agent may vary according to a country’s legislation and mine’s Code of Practice. In most cases, as in South Africa,
China or Australia, Monoammonium Phosphate (more commonly known as MAP or ABC powder) will be used. Although the price of a single active barrier is much higher, the number of systems to be used within a mining operation will be much lower than in the case of a passive barrier. This assumption is based on the fact, that active barriers are reusable. An active barrier is furthermore mobile. It is also possible to move an active barrier as the cutting operation progresses. An active barrier in its design will be able to be used at a variable distance from the cutting face. As the barrier is not reliant on the strength of an explosion (weak, standard or strong) it could be initially placed at e.g. a distance of 15 m from the cutting face. As the cutting operation progresses, the distance to the face will increase. Once the distance to the cutting face becomes too great (according to a particular mine's Code of Practice or risk management system), the active barrier can be moved forward to be located at 15 m again.

**INSTALLATION**

The installation of any barrier will be based on the design performed in advance. This should result in addressing installation issues such as adequate preparation, the placement, quantity and the proper fastening of the equipment. In this process it should be ensured that adequate attention is given to faultless implementation of the design and possibilities of commissioning a barrier.

The preparation would typically address issues such as how and where to fasten a barrier. Bagged barriers would typically be suspended from the roof. Examples are the South African Guidelines for the Compilation of Mandatory Code of Practice the Prevention of Flammable Gas and Coal Dust Explosion in Collieries (DME, 2020) or the HSE “Bagged stonedust barrier” guideline, a seam height exceeding 3.5 m will require multiple layers of bags. In some cases, a structure will need to be prepared to suspend the passive barriers. Similarly, an active barrier will need to be fastened to a support structure. The support structure needs to be prepared in advance. The support structure should also allow the barrier to be relocated. Figure 11 presents a few solutions of barriers being implemented in South Africa, China and Australia. In most cases passive barriers will be suspended from the roof. Active barriers may be fastened to the ground, roof or any other structure in place that can support the load. Whereas an active barrier will be considered as one unit, passive barriers will be distributed over an area. The weight distribution would hence affect the placement. Continuing the example of a 2.4 m x 7 m roadway section and assuming the same requirements as used in chapter 2 "Design" and using South African guideline (DME, 2020), the amount of stone dust required for one passive barrier will be 1.68 t. It will however require four barriers spread over a distance of 120 m. Hence the full barrier will require 6.72 t of stone dust. The same roadway section would require 130 kg of suppression powder when using an Explospot active barrier (Spaeth et al, 2017) The active barrier will weigh 1.6 t. All connections (hydraulic and electrical) are based on the “poka yoke” principal (from the Japanese for inadvertent error prevention). Thereby ensuring unique and fault free connection and interfaces. Ensuring the design is implemented unmistakably.

Once the installation has been completed, a barrier needs to be commissioned. The commissioning procedure ensures the installation conforms to the design and barrier performance. Passive barriers will be commissioned using primary visual inspection methods. Some passive barriers such as the stone dust barrier bag have implemented visual guidance in regard to the bag and its content (Figure 11).

Once the installation has been completed, a barrier needs to be commissioned. The commissioning procedure ensures the installation conforms to the design and barrier performance. Passive barriers will be commissioned using primary visual inspection methods. Some passive barriers such as the stone dust barrier bag have implemented visual guidance in regard to the bag and its content (Figure 12).

By means of a red line the inspector will be able to determine whether the bag was roughly filled with the appropriate amount of stone dust. As the bag is transparent, the inspector will also be able to visually inspect the colour and the composition of the bags content. By pressing the bag, he will be able to establish whether the content still roughly complies with the Code of Practice in regards to clogging. Active barriers will also be commissioned by means of visual inspection. The barrier will be inspected in regards to obvious physical damage and by monitoring the controller’s display. The display will indicate any electronic or pressure error that effect the barriers performance. The controller
will monitor the health and sanity of the active barrier in real time, therefore, ensuring the immediate identification of an error (see Figure 12).

Figure 11: A. Water Trough (China), B. Active Explosion Barrier (South Africa), C. BatBag (South Africa and Australia NSW), D. Active Roadway Barrier (China)

Figure 12: Error indication and commissioning of passive and active barriers

MAINTENANCE

Maintenance is of extreme importance in regard to the proper and full functionality of an explosion barrier (MSOD, 2001). If a barrier is not maintained according the Code of Practice or manufacturer’s requirements, the risk of an explosion passing a barrier unsuppressed increases to an extent whereby it might be the same as not having a barrier at all. As part of Cain’s recommendation (Cain, 2003) the maintenance of explosion barriers should be part of any Code of Practice or Standard Operating Procedure. In practice, the maintenance of a barrier will vary according to type. Bagged barriers might need to be inspected in regard to the number of available bags per barrier. The water content of each water trough within such a barrier might need to be inspected and refilled. Amongst others, each barrier component would need to undergo a visual and under certain circumstances tough based inspection. The active barrier would similarly need to undergo a visual inspection. Here the inspection would, similarly to the commissioning, include a mechanical and an electronic inspection. The mechanical inspection would cover aspects such as “obvious physical damage” and clogging of the nozzles. The electronic inspection as part of the maintenance would include analysis of a controller’s user interface. Any sort of visual inspection, regardless of the type being active or passive) is done on-
site. Hence the inspector would need to physically go on-site to investigate the general state of a barrier. Having an electronic interface enables active barriers to communicate with surrounding personnel. Even though they might not have had adequate training to analyse a fault on an active barrier, these interfaces will use basic symbols (typical to a country, culture or other safety guidelines and symbols) that any mining staff will associate with a certain type of situation (Figure 12). Newer active barriers also provide the possibility of reporting to service dashboards such as the “Explospot Maintenance Cloud” (Spaeth and du Plessis, 2017) (Figure 13). These dashboards do not substitute routine maintenance, but rather provide information to have for more efficient and timely maintenance. Using a dashboard, the maintenance personnel would have a clear indication of any part that may be faulty. Hence providing the team e.g. with the correct spare-part prior to inspecting the faulty barrier. This in turn will lower the lead time for the barrier to be fully functional again. These kinds of support tools do however require a basic communication infrastructure. These might not always be in place at the site of an active barrier.

![Image](https://example.com/image1)

**Figure 13: Peripheral display indicating health state or operational status**

![Image](https://example.com/image2)

**Figure 14: Schematic function of a maintenance support dashboard**

The release mechanism of common active barriers is based on the discharge of pressurized container. Most legislation requires any kind of pressurized container to be serviced once every 12 months. Legislative requirements such as the service of pressurized containers, contribute to the individual barrier type maintenance plan.

**OPERATION**

The operation refers to the actual performance of a barrier in the case of an explosion. It is expected of an explosion barrier to stop and suppress an explosion according to the design set out in the above. Although the operation is often divided into “active” and “passive”, practice will rather indicate a difference in regard to the direction of an explosion barrier. Passive barriers have always been regarded to have a suppression motion in the same direction as the explosion. Active barriers, especially the machine mounted barriers, have always been considered to counter the explosion force. Therefore, the suppression motion would be considered to be towards the explosion. The active barrier is required to stop an explosion at the position of the nozzles. This performance requirement originated from the requirements set out for active barriers (du Plessis and Smith 1999). The requirement is to protect the continuous miner operator. The requirement was defined in 1999. At this time the operator’s seat was positioned at the rear of the continuous miner. The nozzles of a machine...
mounted barrier are typically located near the rear of the machine. The active barriers being able to stop an explosion on the spot could therefore also be considered to have a “countering” performance.

Other active barriers have been engineered to have a similar performance as a passive barrier. These active barriers will release the suppression agent upon detecting an explosion. The detection can refer to either heat, pressure or light. They will however not “counter” the explosion but rather release the suppression agent into the explosion. Thereby suppressing the explosion in a later inerting process. These barriers have the same obvious disadvantages as a passive barrier regarding size, placement and safety area. They do however enable the implementation of mechanisms to enhance the commissioning and maintenance aspects as discussed above.

Unlike passive barriers that cannot be reused, active barriers are designed in such a way as to allow the reuse of most of the components. This reduces the lifecycle cost of the active barrier. However, some components are subject to flameproof or intrinsic safe regulations that might not allow the reuse of these components. One manufacturer of a active “countering” barrier, has reported that an Explospot machine mounted barrier has been in use for up to six years in one of the South African mines (Explospot, 1998). During this time span multiple maintenance runs (pressure canister overhaul) have been conducted to comply to legislation. Table 2 below shows the summary of practical comparison between active and passive explosion barrier systems.

![Figure 15: Example of methodology in regard to the suppression motion](image)

**Table 2: Summary of practical comparison between active and passive explosion barrier systems**

<table>
<thead>
<tr>
<th></th>
<th>Active Barrier</th>
<th>Passive Barrier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operation (safety)</td>
<td>++</td>
<td>-</td>
</tr>
<tr>
<td>Reliability</td>
<td>++</td>
<td>-</td>
</tr>
<tr>
<td>Size</td>
<td>+</td>
<td>-</td>
</tr>
<tr>
<td>Suppression material (needed)</td>
<td>++</td>
<td>-</td>
</tr>
<tr>
<td>Failsafe installation</td>
<td>+</td>
<td>O</td>
</tr>
<tr>
<td>Cost (unit)</td>
<td>--</td>
<td>+</td>
</tr>
<tr>
<td>Cost (lifecycle)</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Maintainability</td>
<td>+</td>
<td>O</td>
</tr>
</tbody>
</table>
CONCLUSIONS

It is generally accepted that both principals of active and passive barriers have been proven to work within the specifications that they have been designed for. In regard to safety, a few differences have however been identified when reviewing the barriers in an operation environment.

1. The categorization of explosion barriers should be revisited when discussing constructive explosion mitigating controls. As to the performance, there are drastic differences between the general working directions, and hence their effectiveness in regard to the safety zone of barriers. Two working principals have been discussed in Section 6. These are “passive”, dispersing inerting material in the general direction of the explosion and “countering”, pushing the suppression material into the explosion and creating a barrier at the location of the nozzles. The change in categorization would then rather reflect the effective safety area than the means of activation. This would enable a risk management system to further scale down the high-risk area. A more risk orientated category for explosion barriers could then refer to a “countering” active barriers and passive barrier (as a characterization of the function).

2. The implementation space claim of active barriers is far less than the passive barriers. Due to the dynamic design possibilities of this type of barrier, these barriers can be placed outside the working (moving machinery) area.

3. The cost of a single passive barrier unit is far less than a single active barrier. Practical implementation however shows that the total cost within a 2000 m roadway will be greater when using passive barriers rather than using an active barrier due to its mobile capability.

4. The installation of a passive barrier is a manual and labor-intensive process. The installation is not “fool proof”, hence an installation might differ from the theoretical design of the barrier. This is not the case for active barriers, as they are built on a principal whereby the barrier is not susceptible to a wrong installation.

5. A “countering” active barrier is far more robust than a passive barrier. Due to the function of passive barrier (low/medium pressure rupture) it will not withstand even minor shock. The “countering” active barrier does per specification need to withstand strong outside forces. This however does not mean that it will withstand all influences generated by underground machinery.

6. The current and future possibilities of an active barrier to interface with an inspector, operator or maintenance team is vast. All current implementations ensure a safety orientated interface, allowing the mine to clearly incorporate an active barrier into their Code of Practice or safety procedure.

7. The design of a active barrier is easy to implement, along with its adaptation to a new environment.

REFERENCES


Department of Minerals and Energy (DME), 2020. Guidelines for the Compilation of Mandatory Code of Practice the Prevention of Flammable Gas and Coal Dust Explosion in Collieries, South Africa

Queensland Resources Safety & Health (QRSH), 2020. Recognized Standards 21, Australia


https://ro.uow.edu.au/coal/531


Health and Safety Executive (HSE), 2014. Bagged Stonedust Barriers. Guidance to Fire and Explosion. United Kingdom


Phillips, H.R., 2019, Personal Communications, South Africa.
WHY SLENDER BEAM/COLUMN BEHAVIOUR SHOULD NOT BE IGNORED FOR EFFECTIVE GROUND SUPPORT DESIGN.

Mark Colwell¹ and Russell Frith²,

ABSTRACT: As per most other earth science engineering problems, the underground coal geotechnical environment and the way in which roof and rib support interacts with the rock mass are complex issues. It is therefore generally recognised that without prudent simplification, the complexity of the problem will overwhelm all current geotechnical methods of modelling, not least for the reason that a rock mass can never be characterised to a level that allows a “non-simplified” analysis. The fact that numerical models, which are commonly purported to be a “simulation” tool and the so-called epitome of advanced geotechnical engineering, always need to be “calibrated” to a known reality is taken to be conclusive proof of this statement.

While the problem should not be oversimplified (i.e. the dominant failure mechanisms or critical data input parameters should not be ignored), without question judicious simplification is at the heart of all engineering design, to the point that it has a well-established name – “reductionism”.

This paper demonstrates that slender beam/column behaviour is the dominant instability mechanism within a coal mine roof/ribline subject to elevated horizontal/vertical stress conditions and must be representatively accounted for in any credible empirical, analytical, or numerical approach to coal mine roof/rib stability assessment and associated ground support design.

The process by which the mathematical equations associated with slender beam behaviour (including buckling due to axial loading) can be readily accommodated as a part of geotechnical assessment and design is explained. Ensuring that the mathematical modelling/equations are representative of the problem being analysed, is crucial within many branches of science such as spaceflight trajectory/analysis. As that field of science has demonstrated, if the mathematics are wrong or necessary mathematical equations/code are missing, then the model can be worthless or even dangerous leading to disastrous results, which for the coal industry essentially means compromising the safety of underground workers with inadequate ground support designs.

INTRODUCTION

The naturally unstable buckling of thin beds or columns due to axial loading (termed Euler Buckling) is well recognised observationally and from measurements (e.g. extensometry) as being one of if not the primary (and therefore dominant) behavioural mechanism associated with horizontally bedded roof instability under the action of horizontal stress and vertically cleated/jointed coal ribs under the action of vertical stress in coal mine roadways (O’Beirne et al 1987, Colwell, 2006 and Colwell and Frith, 2012).

In terms of horizontally bedded roof, the major structural feature is the bedding and/or laminae along which delamination (i.e. tensile failure leading to subsequent relative horizontal/shear movements) occurs resulting in thinner (or slender) beams, which can buckle under sufficient horizontal stress with ensuing shear failure of the rock. Figure 1 is photo taken of a roof fall cavity associated with a Queensland colliery and clearly illustrates the formation of slender beams, their thickness (in this instance) dictated by the spacing of the carbonaceous laminae.

The rock type associated with the roof fall cavity depicted in Figure 1 is 50 MPa to 80 MPa (Uniaxial Compressive Strength, UCS) sandstone with abundant carbonaceous laminae extending some 5 m above the roofline. In terms of the laboratory or sonic-derived UCS (which is measured normal to the

¹ Principal, Colwell Geotechnical Services. Email: markcolwell@bigpond.com Tel: +61 7 5499 7233
² Principal Geotechnical Engineer – Mine Advice Pty Ltd. Email: russell.frith@mineadvice.com Tel: +61 2 4088 0600
laminae) this would be considered a moderately strong to strong rock with respect to coal mine roof strata, however in terms of the approximate average 50 mm thick roof beams associated with Figure 1 their lateral load bearing capacity over a 5 m wide span is in the order of 1 MPa. Irrespective of the type of laminae (i.e. carbonaceous, micaceous, siltstone etc.), such laminated or interbedded roof rock units (with varying intensity of laminae/bedding) are extremely common to Australian collieries.

**Figure 1: Photograph of a Roof Fall Cavity Displaying the Formation of Slender Beams**

Figure 2 (from a U.S. coal mine) is a dramatic illustration of this failure mechanism (i.e. delamination, buckling with ensuing shear failure of the thin rock beams), while Figure 3 illustrates similar behaviour associated with the ribs where delamination can occur along the cleat, coal joints as well as mining-induced fractures, eventually resulting in shearing through the buckling columns, as can be clearly seen at the point of maximum lateral deflection.

**Figure 2: Coal Mine Roadway Roof displaying Buckling and Shear Failure due to Horizontal Stress (after Mark and Agioutantis, 2012)**
Despite nearly 40 years of intensive research and proving work in this subject area, there remain dissenting views that slender beam/column behaviour and buckling due to axial loading of roof strata and riblines is not a dominant behavioural mechanism (e.g. Gale, 2018 and Heritage, 2020a).

This paper specifically addresses such views and provides evidence to allow engineers to make an informed decision on which approach is more suitable to achieve their objectives. In addition this paper details the process by which the mathematical equations associated with Euler Buckling can be readily accommodated as a part of geotechnical assessment and design and how proven roof/rib support design techniques such as Analysis of Longwall Tailgate Serviceability (ALTS 2009, Colwell and Frith, 2009), Analytical Model for Coal Mine Roof Reinforcement (AMCMRR, Colwell and Frith, 2010), Analysis and Design of Rib Support (ADRS, Colwell, 2004) and Analysis and Design of Faceroad Roof Support (ADFRS, Colwell and Frith, 2012) incorporate slender beam behaviour.

BUCKLING AS A CREDIBLE BEHAVIOURAL MECHANISM

In relation to coal mine roof/rib behaviour and geotechnical evaluation/ground support design, there have been four basic approaches in relation to either modelling this environment or in the development of design tools, which in alphabetical order are:

- Analytical
- Empirical
- Numerical
- Physical

An example of a Physical Model is illustrated in Figure 4, which is the same as in Figure 126 taken from Hoek and Brown (1980) where they state, "Figure 126 illustrates the buckling of slabs in the roof and floor of an excavation in a high horizontal stress field. This type of failure was observed in model studies conducted by the Australian Coal Industry Research Laboratory (ACIRL) in an attempt to simulate the structural and stress conditions in the coalfields near Sydney, Australia."

Hoek and Brown (1980) go on to state, "In jointed or bedded rock masses, the presence of structural features parallel to the excavation surfaces will result in the formation of plates and slabs, Whatever the reason for the presence of these slabs, it takes little imagination to visualise that they are susceptible to buckling under axial stress."
Colwell (2006) in “A Study of the Mechanics of Coal Mine Rib Deformation and Rib Support as a basis for Engineering Design” indicates that up to that point in time there had been comparatively little research undertaken (worldwide) in relation to rib support design as opposed to roof support design. However based on published information, it was in Australia where the bulk of such research had been carried out over the preceding 20 years with four major studies being summarised in the following reports:


The study of O’Beirne et al (1987) highlighted two distinct failure mechanisms namely:

1. Buckling of plates, slabs or columns due to vertical closure between the roof and floor over the ribs and/or an increase in the vertical stress.

2. Existing cleat and mining induced fracture (MIF) interaction, resulting in granular and/or blocky spall.

Fabjanczyk et al (1992a) suggest that the wide variation of coal deformation mechanisms of the immediate rib does not allow for standard rib support patterns. Unlike O’Beirne et al (1987), the actual ribline deformation mechanisms are not clearly identified by Fabjanczyk et al (1992a) other than some figures suggesting that slender columns within the ribline will form under the action of vertical closure between the roof and floor over the ribs and/or an increase in the vertical stress and the impact of weak claystone bands (refer Figure 5). Fabjanczyk et al (1992a) do suggest that within a high vertical stress environment, the deformation of the rib is almost uniquely controlled by the nature of the coal and capability of the coal to generate its own confinement.

Frith and Ditton (1993) examined the relative nature of the horizontal rib displacement ($U_{\text{rib}}$) to the vertical roof displacement ($U_{\text{roof}}$) or more precisely the roof to floor convergence. They point out that if the coal rib were a homogeneous material with no fractures or structure, then prior to failure the
outward movement of the rib would simply be a result of Poisson's Effect and therefore be in the order of 0.25 times the roof to floor convergence adjacent to the ribline. However based on data associated with the monitoring sites of their project and also those of O'Beirne et al (1987), Frith and Ditton (1993) found that in the case of unstable ribs, the amount of outward rib movement can be up to several times that of the roof to floor closure.

Figure 5: Effect of Weak Bands on Coal Rib Deformation (after Fabjanczyk et al, 1992a)

A dramatic example of this behaviour is illustrated by Figure 6. This photo is taken in the tailgate (TG) of Dartbrook Colliery looking inbye to the TG intersection with the longwall face. As can be seen there is significant lateral displacement of the chain ribline (i.e. in the order of 300 to 500 mm) and yet the roof is in excellent condition and no floor heave was observed. The principal mechanism which explains this relative deformation behaviour between roof, floor and coal rib is buckling due to axial loading, subsequent to which toppling, wedge and planar (i.e. kinematic) failures can occur.

In relation to the four Australian studies previously mentioned; while there is some minor difference of opinion between the researchers in relation to the driving force behind rib degradation, all (except Fabjanczyk et al, 1992a) would appear to agree that buckling due to axial loading is a common failure mechanism and all do agree on the negative impact that weak stone bands can have on rib deterioration.

The significant and potentially detrimental impact of weak claystone bands and/or weak coal/roof & coal/floor interfaces on ribline behaviour is well documented, particularly where translation along these
weak bands is allowed. Figure 7 illustrates where initial blockside ribline displacement occurs along a weak to very weak 40 mm to 50 mm thick carbonaceous claystone band (boxed in red) located approximately 500 mm from the top of seam, with subsequent significant ribline deterioration requiring remedial support. It was found that the greater the lateral movement allowed along the claystone band, the more likely greater levels of overall ribline deterioration.

Figure 6: Dartbrook Colliery Ribline Behaviour under TG Loading Conditions

Figure 7: Effect of Weak Band on Blockside Rib Behaviour
As Colwell (2006) explains; weak bands within a seam appear to have several roles. Where present they tend to act as the ‘hinge’ or apex in relation to bulging in the riblines and/or the end point of the buckling slab. When acting as the end point (as illustrated by Figures 5 and 7) typically these weak bands modify the end-fixing condition allowing lateral movement. This increases the Effective Length \( (L_{\text{eff}}) \) of the coal plates or slabs dramatically lowering the critical load or stress for which buckling can occur. The following is provided to assist in understanding how weak bands can modify the end-fixing condition allowing lateral movement and their impact on rib stability.

Table 1 is adapted from information contained in Standards Australia AS 3600-2001: Concrete Structures and summarises buckling behaviour under various end-fixing conditions and the Theoretical K Value (i.e. the effective length factor) dependent on the end-fixing conditions. The critical stress \( (\sigma_{\text{crit}}) \) at which a column (pinned at both ends) will buckle is given by equation 1:

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

(1)

where \( E \) is Young’s Modulus of the column material, \( L \) is the height and \( d \) the column thickness.

Where the end-fixing condition of the column is pinned-pinned (refer Column c, Table 1) it will allow rotation of the end but not translation. Varying the end-fixing condition will affect the length of the column over which buckling occurs and that length is designated as the Effective Length \( (L_{\text{eff}}) \) such that:

\[
L_{\text{eff}} = K L
\]

(2)

### Table 1: K Values for Buckling Columns (after Standards Australia AS 3600-2001)

<table>
<thead>
<tr>
<th>Braced Columns</th>
<th>Unbraced Columns</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>(b)</td>
</tr>
<tr>
<td>(c)</td>
<td>(d)</td>
</tr>
<tr>
<td>(e)</td>
<td>(f)</td>
</tr>
</tbody>
</table>

Buckled shape of column is shown by dashed line.

<table>
<thead>
<tr>
<th>Theoretical K Value</th>
<th>0.5</th>
<th>0.7</th>
<th>1.0</th>
<th>1.6</th>
<th>2.0</th>
<th>2.0</th>
</tr>
</thead>
</table>

Symbols for end-fixing condition:
- Rotation fixed, translation fixed
- Rotation fixed, translation free
- Rotation free, translation fixed
- Rotation free, translation free

Unbraced columns (refer Columns d, e and f, Table 1) are those in which transverse movement of one end is not prevented. This is commonly observed underground either adjacent to a rock/coal interface (where the seam thickness is less than the development height) or adjacent to a weak low friction stone or bright coal band. This results in K values > 1, longer effective lengths and as a result a lower critical load or stress for which buckling can occur. It is the effective length, \( L_{\text{eff}} \) which is substituted into equation 1 for \( L \) to calculate the critical stress at which a column will buckle. For example, where the K value = 2, the critical stress at which movement will occur is reduced by a factor 4 (i.e. \( 1/2^3 \)).
Essentially the initial lateral movement along weak bands is simply a form of buckling, subsequent to which kinematic failures can occur.

Figure 8 is photo taken adjacent to Oaky North Colliery’s chain pillar monitoring site associated with ACARP Project C11027 (Colwell, 2004) subject to one side abutment load. It was found that the top section of the coal was somewhat fractured with a 100 mm to 200 mm ‘bulge’ in the upper portion of the ribline. The apex of the ‘bulge’ was approximately 650 mm to 750 mm from the roof adjacent to two stone bands some 30 mm thick and around 100 mm apart. However as is also illustrated by Figure 8, with the proactive use of steel mesh (installed prior to abutment loading) the ribline was under effective control with the steel mesh containing the fractured/buckled ribline.

![Figure 8: Oaky North Colliery – Chain Pillar Ribline (after Colwell, 2004)](image)

Colwell (2004) found that there was a significant reduction with respect to the extent of softening within the riblines that utilised areal support in the form of steel or plastic mesh. The major structural or mechanistic benefit of mesh to overall ribline performance is that it maintains the buckled coal plate to the ribline allowing it to behave (to a degree) like a spring, which in this instance not only provides resistance to the vertical stress but also provides an active lateral force to the inner coal plates. Essentially the most effective rib support in terms of overall roadway maintenance becomes the buckled coal rib itself.

A dramatic illustration of the above statement is that associated with a comparison of Crinum Colliery’s standard steel mesh and “no rib mesh” chain pillar monitoring sites associated with ACARP Project C11027 (Colwell, 2004) under Maingate Loading conditions (i.e. one side abutment load) as illustrated in Figure 9.

![Figure 9: Standard Chain Pillar Support adjacent to “No Rib Mesh” Trial Section – Crinum Colliery (after Colwell, 2004)](image)
Colwell (2004) found that the ribline displacement profile would readily match the type, level and effectiveness of the rib support. Figure 10 is a photo taken of Crinum Colliery’s tailgate chain pillar ribline (utilising steel bolts and mesh) just outbye of the tailgate intersection with the longwall face. The shape of the ribline displacement profile is consistent with Column a of Table 1, such that the steel bolts are modifying the end condition of the buckling ribline resulting in a K Value < 1.

Figure 10: Chain Pillar Ribline just outbye of TG Intersection – Crinum Colliery (after Colwell, 2004)

The field investigations associated with Colwell (2004 and 2006) involved the collection of information from 26 collieries with seven of those collieries also participating as instrumentation sites involving 33 monitoring locations. The extensometry and stress cell data clearly confirmed the visual observations that buckling due to axial loading was and is the dominant ribline behavioural/deformation mechanism, while recognising other far less common behavioural mechanisms can also occur under specific conditions.

THE LIMITATIONS OF NUMERICAL MODELLING AND WHY THERE IS A DISENTING VIEW

Best (1978) reported that the basic tool of the structural analyst is the mathematical theory of elasticity. He went on to state that based on the assumption of continuum behaviour this theory establishes a set of differential equations that describe the load-deflection (stress-strain) behaviour of a typical elemental region within the body. A given problem is solved by obtaining a solution for the differential equations that also satisfies the boundary conditions of the problem. The theory of elasticity provides the fundamental relationships that form the basis of numerical methods of solution.

However, the theory of elasticity is restricted to materials in which the load-deflection behaviour is linearly elastic and as rock yields and displays both linear and non-linear behaviour, the theory of plasticity (Hill, 1950) needs to be incorporated within the analyses of rock behaviour. The inclusion of plastic behaviour as well as taking into account the discontinuities within the rock mass (which introduce additional boundary conditions over those existing in a corresponding continuum) significantly increase the difficulty of realistically modelling rock behaviour using numerical methods.

Over the last 40 years these issues have presented researchers with an ever evolving challenge to select (or calibrate) the mathematical routine that best represents the expected physical behaviour of the rock mass. In contrast to ACIRL’s physical model (Figure 4); numerical modelling researchers
have been typically considering geometries (or setting up their models) which contain structural elements that, by their very nature, cannot buckle and must therefore fail in shear or the models simply do not contain mathematical routines associated with buckling (e.g. Wu, 1998).

Therefore the issue of buckling due to axial loading as a failure mechanism about the mine opening/roadway has essentially been ignored by the numerical modelling fraternity; however Gale (2018) and Heritage (2020a) take this to a whole new level. While Gale (2018) accepts that it is common to see coal ribs which are “apparently” buckled (e.g. refer Figures 3, 6, 8, 9 and 10); for Gale (2018) this is somehow caused by deeper conjugate shear failure applying a lateral force to the outer ribline to make it “look like buckling”. An extract from Gale (2018) is presented in Figure 11 to assist with the following discussion.

![Figure 11: Extract from Gale (2018)](image)

Based on his Figure 13, Gale (2018) states with respect to the roof; “only thin plies would fail, as a result of pure Euler buckling for normal stress conditions” and therefore is clearly suggesting that thin roof plies are the “exception rather than the norm” with respect to Australian underground coal mines such that Euler Buckling is no more than a “secondary consideration”. This is simply incorrect and is in direct conflict with decades (and an overwhelming level) of observational evidence from a wide range of researchers and practitioners in the field.

As Galvin (2016) points out, “In coal mines, the immediate roof and floor strata are usually bedded due to the sedimentary origin of coal deposits. Bedding planes are characterised by low to zero tensile strength normal to the bedding planes and low shear strength relative to that of intact rock. Hence, bedding planes constitute potential slippage planes and can effectively divide the roof strata into an assembly of thin rock beams.”

One of the reasons why Gale (2018) is dismissing Euler Buckling as a dominant roof failure mechanism would appear to be based on Gale and Fabjanczyk (1999), where they take no account of the impact and effect of the bedding/laminae within a roof rock/coal unit which significantly reduces the effective beam thickness from that of the unit’s lithological thickness to the spacing between the bedding/laminae (refer Figures 1, 2 and 4).
Gale and Fabjanczyk (1999) found that numerical modelling of a generalised Goonyella Middle Seam roadway roof strata indicated that the top coal (immediate roof unit) acts as an independent unit within the roof section and that the stability of this unit can be analysed using the following criteria:

I. Buckling failure of the coal top
II. Self-weight failure of the coal.
III. Overstressing of the top coal due to a combination of deflection imposed from the strata above and the in situ stress within the coal.
IV. Ability of the top coal to confine the strata above.

Gale and Fabjanczyk (1999) utilised numerical modelling to assess criteria ii to iv, while the potential for buckling failure was assessed separately on the basis of Euler failure criterion (i.e. the numerical model they used contained no mathematical code with respect to buckling and therefore the ability to simulate or assess slender beam behaviour). In relation to buckling failure Gale and Fabjanczyk (1999) state, “Buckling failure of the coal roof is likely where its thickness reduces less than a nominal 0.5 m and the stresses are low 2 MPa.” and concluded, “...that shear failure of coal is the most likely failure mechanism, with buckling failure only likely with very thin coal beams at low stress levels.”

In relation to collieries operating in the Goonyella Middle Seam, Gale and Fabjanczyk (1999) state, “the possibility exists to use low density (less than 1 bolt per square metre) roof bolting patterns within specific limitations of coal roof thickness and management of structured areas” and further indicate, “With a 2 metre thick coal roof a high level of stability can be achieved in higher stress levels (200 metres of cover)” and “A 4-bolt pattern is adequate for low stress conditions, however a 6-bolt pattern is more capable of controlling the strata under higher stress conditions.”

In subsequent years, similar recommendations based on numerical modelling studies have proved to be totally inadequate for certain mines operating in the Goonyella Middle Seam (Thomasson, 2016) and for safety reasons the use of 4 bolt patterns is clearly discouraged by the Queensland Department of Natural Resources and Mines (i.e. Mines Safety Bulletin No. 148, 23 January 2015, Version 1).

As opposed to Hoek and Brown (1980) and Galvin (2016), the preceding discussion indicates that Gale and Fabjanczyk (1999) have chosen to ignore the bedding/laminae within the coal unit when assessing the unit's propensity for Euler Buckling. This is a clear example where oversimplification of the problem (to suit one’s preferred method of analysis and its limitations) is simply unacceptable and potentially leads to disastrous results.

In relation to Australian longwall operations, there are currently 13 collieries (four of which are on care and maintenance) or approximately 50% of the industry where the roof associated with the primary bolted interval is either completely coal or contains a significant coal unit. Typically bituminous coal by its very nature is thinly bedded with bright coal, dull coal and stone bands.

Holmes (1965) suggests that any block of bituminous coal can be seen to have a well-marked banded or stratified structure. The commonest bands are composed of bright coal which readily breaks into approximately right-angled pieces with smooth brilliant surfaces. Many of the bands appear to be quite structureless, and since the material is not unlike black glass in appearance it has been called vitrain. Other bands are finely laminated and consist of shreds and films of vitrain. This type of coal material is known as clarain. These bright bands are separated by layers of a dull grey-black type coal which, being relatively hard and tough, is distinguished as durain. A fourth type of material is called fusain and consists of thin flakes or lenticles of extremely friable “mineral charcoal”, which occur at intervals through the seam. Holmes (1965) further elaborates that coal naturally splits very easily along these planes of weakness and as fusain readily crumbles to powder the broken coal becomes dusty.

In undertaking research related to the in situ strength and deformability of coal for engineering design, Medhurst (1996) conducted a detailed review of the nature and properties of coal and states, “At the megascopical scale coal can be divided into two groups, the humic (or banded) coals and the sapropelic (or massive) coals with the humic coal constituting the main economic deposits for mining. The humic coals usually show a pronounced banded appearance consisting of bright and dull laminae.”

Therefore it is not appropriate in assessing the lateral load bearing capacity of a roof coal unit to simply use the lithological thickness and ignore the bedding/laminae within the coal unit when
assessing the unit’s propensity for buckling due to axial loading. In addition (and as previously discussed), laminated roof/stone units are extremely common to Australian collieries and therefore thin roof plies are overwhelming the norm and not the exception as Gale (2018) mistakenly contends.

To further reinforce this; a review of the ADFRS database involving 26 longwall operations was undertaken. The database contains 201 roof coal/rock units associated with at least the first 5 m above the roofline. It was found that average fracture spacing (FS) was 164 mm, while 71 units had an FS < 50 mm, 139 units had an FS < 100 mm, 168 units had an FS < 200 mm and 188 units (or ≈ 94%) had FS < 500 mm and with respect to the immediate roof (i.e. roof Unit 1), 72 of 94 units (or 77%) the FS < 100 mm.

In relation to the ADRS database, which also contains 26 collieries, it was found that the average face cleat spacing was 125 mm and that 13 of the 26 collieries had a cleat spacing of < 50 mm, while for 18 of the 26 the cleat spacing < 100 mm. If mining-induced fractures are included, then once again we are dealing with the formation of thin columns within the immediate ribline (certainly the first 0.5 m to 1 m) as we are dealing with thin beams forming in the roof.

Figure 12 is a reproduction of Gale’s (2018) Figure 13, while including some noteworthy additions and changes. Firstly the maximum limit for the axes has been changed. Roadway roof and ribline stress monitoring would suggest that in relation to the y-axis the typical maximum levels of horizontal stress that a belt road roof and vertical stress that a tailgate ribline would be subject to during longwall retreat is in the order of 50 MPa. The beam/column thickness or x-axis has been changed from 1.5 m to a practical upper limit of 500 mm as previously discussed.

To simulate the roof/rock units Gale (2018) utilises a modulus (E) of 10 GPa for both a 5 m roof span (i.e. average roadway width associated with Australian collieries) and a 1 m distance between bolts. This is considered reasonable. For coal Gale (2018) utilises an E = 2 GPa (once again this is reasonable), however for some inexplicable reason a 1.5 m length. Given approximately 50% of the immediate roof associated with Australian longwall operations is coal why doesn’t Gale (2018) simulate this scenario (i.e. a 5 m roof span and E = 2 GPa) and given the average development height in Australia is approximately 3 m why is this length not also simulated for coal? Figure 12 includes these simulations to provide more representative context of the types of beam/column geometries that are present in coal mine roof and ribs.

Based on the review of the ADFRS and ADRS databases, approximately 75% of the immediate roof and rib contains material where if delamination is allowed to occur then beam/column thicknesses (of less than 100 mm) will form which, based on Gale’s (2018) own calculations, can readily buckle due to axial loading, as illustrated in Figure 12.
Now let us deal (visually, utilising previous studies and mechanistically) with Gale’s (2018) statement, “It is common to see coal ribs, which are apparently buckled, however under close inspection it is often, although not always, the case that behind the buckled zone is a conjugate shear fracture, which has dilated and allowed aside loading on the coal ply. In this case the failure process is more complex”.

**Visually:** Figure 13 is from Gale (2018) when discussing the concept of buckling failure of an isolated ply under axial loading. Figure 14 is a photo of a Crinum Colliery blockside ribline. The similarities between Figures 13 and 14 are self-evident and the only substance behind the buckled section of ribline in Figure 14 being ‘thin air’ as is behind the buckled zone illustrated in Figure 3 associated with Appin Colliery. The word-limit and space considerations associated with a technical paper, is the only reason that prevents many more similar photos from being presented.

**Previous field investigations:** As discussed earlier, all previous significant Australian studies into ribline behaviour (other than Fabjanczyk et al., 1992a) found buckling due to vertical loading to be a significant failure mechanism as opposed to what Gale (2018) suggests.

Heritage (2020a) states, “Tensile cracking can occur through the rib dilation caused from deeper shear failure pushing out the near rib – this can often appear as buckling style failure” and provides the following diagram to partially illustrate her “theory” of a stress driven rib failure mechanism.

Therefore Heritages’ (2020a) interpretation of a stress driven rib failure mechanism has the failure starting 2 m to 3 m inside the ribline and working its way out to the ribline such that it only “looks” like buckling due to axial loading. It is worth noting that this in direct contrast to what her Strata Control Operation Pty Ltd (SCT) colleagues found some 18 years earlier as illustrated in Figure 5 where failure commences in the outer rib and progresses further into the rib/pillar.

In monitoring the rib failure mechanism of buckling, O’Beirne et al (1987) found that plates or slabs formed at the ribsided are subjected to vertical loading and sometimes “pushing” from behind. They indicate that this pushing can be caused by other buckling plates (e.g. Figures 3 and 5) or simply due to overall expansion (i.e. Poisson’s effect). Therefore their finding is diametrically opposed to that of Gale (2018) and Heritage (2020a) in that the observed and monitored ribline buckling is more often

Mechanistically: In terms of civil or static structures, buckling is often described as a catastrophic failure mechanism, in that there is a sudden change to the configuration of the structural element (i.e. a column or beam either buckles or it doesn’t and this change of state can occur due to a small change in stress or plate dimension). An example of this behaviour was found at Angus Place Colliery which was one of the seven collieries that participated in ACARP Project C11027 (Colwell, 2004) as an instrumentation site. Figure 16 details the resultant chain pillar rib displacement for both the MG (following the passage of LW 26) and TG Loading Conditions with LW 26N faceline level with the instrumentation site.
Following the passage of LW 26 the total ribline displacement was approximately 50 mm, with increasing displacement occurring during the approach of LW 26N. Then with the LW 26N faceline 8 m inbye of the monitoring site there is a dramatic increase in total ribline displacement from ≈ 90 mm to 170 mm, however there is very little movement with respect to the other rib extensometer anchors. It is worth noting that the adjacent roof extensometer recorded only 5 mm of total roof displacement (TRD) and there was no observed floor heave (i.e. minimal roof to floor closure) and yet significant lateral rib displacement consistent with buckling due to axial loading.

While it is possible that σcrit was exceeded for the coal plate/slab(s) that had been formed, it is far more likely that the coal plate/slab(s) within the immediate (0.75 m) rib separated into two or more thinner slabs (similar to that illustrated in Figure 3) resulting in a sudden and rapid increase in horizontal rib displacement i.e. further buckling. If for example a coal plate/slab separates into two thinner slabs of equal thickness then based on equation 1, σcrit drops by a factor of 4.

Based on the extensometry data (as illustrated in Figure 16) what is clear is that there is no “pushing” of the outer rib from behind as Gale (2018) suggests is the primary cause. Similar ribline behaviour to that at Angus Place was found at the other ADRS instrumentation sites. Furthermore if the buckled ribline is maintained via the use of mesh, this will prevent further buckling (i.e. “domino” effect) and associated ribline deterioration which is often observed with respect to block side riblines not using mesh or steel bolts.

So why is there such disparity of opinion between Gale (2018) and most other researchers (other than his SCT colleagues e.g. Fabjanczyk et al, 1992a, Gale and Fabjanczyk, 1999, Tarrant, 2005 and now Heritage, 2020a)? The inevitable reason is simple and is to be found in the words of Heritage (2020b) when providing an update for ACARP Project C25057 where she states, “Rock failure modelling using FLAC 2D was conducted to validate the inferred failure mechanisms for the rib deformation at each site.”

In other words, a numerical model with absolutely no mathematical code associated with buckling has been used to “validate” the ribline failure mechanisms, as Gale (2018) did in his desktop study when “defining” coal mine roadway roof failure mechanisms. Therefore, in hindsight it was inevitable that buckling due to axial loading would not be identified as a failure mechanism, which a skilled design engineer would easily recognise and report. Numerical modelling is primarily a stress analysis tool and its application to roadway ground support design is a dubious extension of its purpose/use and in terms of its historical application in the Australian underground coal industry; is simply flawed.

Furthermore, utilising numerical modelling (or any model for that matter which is not set up to simulate the structural nature of the “body” under investigation); to “validate” the observed and monitored underground behaviour is at total odds and ignores/misrepresents the Scientific Method established by Francis Bacon, supported by Nicolaus Copernicus and Galileo Galilei; and essentially formalised by Isaac Newton in favour of Bacon’s empirical approach, when he outlines his four “rules of reasoning” in the Principia.

In contrast, the ACIRL physical model (Figure 4) was set up to accurately reflect or simulate the structural (i.e. laminated) nature of the roof and floor subject to horizontal stress and the interpretation was then made based on the observations. It is also important to note that the ACIRL physical model provided a clear indication of what the dominant or typical failure mechanism is with respect to coal mine roof and floor strata subjected to elevated horizontal stress conditions.

It would appear that one of the reasons that numerical modellers might have such blind devotion to their models is their (in some instances total) disregard for the empirical/mechanistic process and the enormous benefit to our understanding of strata behaviour that it has generated based on the inherent requirement (associated with the empirical/mechanistic process) to formulate a database of information specific to the problem.

Not surprisingly there is the constant “theme” associated with such numerical modelling research that all ground support design needs to be “tailored” to the site-specific mechanisms for roof/rib deformation; thereby suggesting that the roof/rib deformation mechanisms are somehow vastly different from colliery to colliery and elaborate/costly instrumentation/monitoring is required at each colliery to then be “validated” by a numerical model before “optimal” ground support design can be undertaken.
Tarrant (2005) suggests that researchers utilise numerical modelling to develop a “better understanding” of roadway behaviour. Tarrant (2005) points out that, “Use of such tools is limited by the simplifications required however when used in conjunction with field measurement and observation, the model findings can be tested and a level of confidence in the results defined.” The use of numerical modelling in the manner described by Tarrant (2005) (as well as Fabjanczyk et al 1992a, Gale and Fabjanczyk, 1999, Gale, 2018 and Heritage, 2020a) only provides a calibrated (via field measurements) model to then be used for site-specific prediction or design. With some imagination one could virtually calibrate any “model” to do that!

In relation to the research mentioned above; each project only contained between one to three field/monitoring sites. In relation to the three monitoring sites associated with ACARP Project C25057 and in comparison to previous rib related research, Heritage (2020a) states, “To provide a point of difference from the previous studies, this review focuses on measurement of rib deformation, with reference to assessment of the mechanisms of deformation.” and yet as previously described; with respect to ACARP Project C11027 (Colwell, 2004) of the 26 collieries that participated in the project seven of those collieries also participated as instrumentation sites involving 33 ribline monitoring locations incorporating roof and rib extensometry as well as stress cells.

Colwell (2004) utilised this extensive and comprehensive database of information to develop the highly successful and widely used rib support design methodology ADRS and as a basis for “A Study of the Mechanics of Coal Mine Rib Deformation and Rib Support as a basis for Engineering Design”, Colwell (2006). So there are actually three “points of difference” between ACARP Project’s C25057 and C11027 being:

- ACARP Project C25057’s lack of monitoring information as compared to ACARP Project C11027.
- ACARP Project C25057’s lack of a comprehensive Australian database.
- ACARP Project C25057’s lack of providing the Australian underground coal industry with a readily useable rib support design methodology.

Calibrating a numerical model to a limited number of sites does not provide an underground coal industry with a widely applicable and therefore accepted design tool for roadway ground support design. None of the project’s associated with Fabjanczyk et al (1992a), Gale and Fabjanczyk (1999), Tarrant (2005), Gale (2018) and Heritage (2020a) have provided industry with a readily useable ground support design methodology.

Yet the Empirical/Mechanistic models ALTS 2009, ADRS and ADFRS clearly prove that ground support design models can be developed for an entire industry and readily utilised by the minesite geotechnical engineer. As Emery, Canbulat and Zhang (2020) state, “ALTS 2009 and associated software package, has grown to be the prevalent technique for chain pillar and gateroad ground (roof and rib) support design at most operating longwall mines in Australia. This is largely because the outputs from ALTS 2009 most accurately reflect the design requirements to provide serviceable gateroads associated with longwall extraction.”

A fundamental reason for this is that slender beam/column behaviour and the associated deformation process of delamination (or de-coupling), bucking due to axial loading and ensuing shear failure (on which these models are based) is not only common to all collieries but is in fact the dominant behavioural mechanism associated with coal mine roadway roof/rib/floor deformation.

**INCORPORATING BUCKLING AS A PART OF GROUND SUPPORT DESIGN**

As the roadway is developed, the in situ stress reorientates and there is a natural tendency for the surrounding strata (i.e. roof, ribs and floor) to move towards the roadway centre. It is generally at this point (i.e. as the surrounding strata moves towards the roadway centre) that ground support (in the form of primary roof and rib bolts) is introduced into the rock mass system to reinforce the rock and coal so as to restrict this displacement to operationally acceptable levels.

Figure 17 (after Fabjanczyk et al, 1992b) illustrates a commonly held model for the development/progression of roof softening (softening being detrimental to overall roof stability) with the contents of Figure 17 being based on measured roof behaviour using sonic probe extensometry. The
The main point of note in relation to Figure 17 is that roof softening progresses higher into the roof as a series of discrete "steps" with such steps only occurring once certain levels of total roof displacement (TRD, mm) have been exceeded in the underlying roof strata.

![Figure 17: Roof Softening Progression with Displacement (after Fabjanczyk et al, 1992b)](image)

The behavioural logic behind this is that roof buckling at any given horizon in the roof can only occur if there is an underlying void into which the strata can move. Therefore for buckling of higher roof measures to take place, the underlying strata must have displaced vertically by a certain amount (i.e. only 10 mm to 20 mm TRD is required), this resulting in a void within the strata into which higher measures can move. This process is described as one of strata "de-coupling".

Conversely therefore, softening of the roof above a certain level can be prevented by limiting roof displacements (in the lower roof), thus eliminating the higher de-coupling process. Accepting that allowing roof softening to progress higher into the roof detracts from roof stability, it is self-evident that limiting the displacement of the lower or immediate roof should be a primary objective of effective strata control.

The other significant point of note with respect to Figure 17 is that the curves rapidly extend to around 2 m to 3 m above the roofline for total roof displacements of 10 mm to 20 mm and then tend to flatten off. This suggests that roof softening to 3 m can occur quite rapidly at relatively low roofline displacement levels providing the condition by which buckling of the roof layers within this zone can
occur, however an equilibrium (or load balance) is reached prior to shear failure of the buckled beams such that further vertical displacement is controlled.

Figure 17 also demonstrates that softening above 3 m becomes increasingly more difficult to propagate as disproportionately higher levels of roof displacement in the lower roof are required. Furthermore, if one assumes that the shear stress distribution and deformation is parabolic in the roof, then the maximum height of softening should be approximately 2/3 of the span (i.e. ≈ 3 m for a 5 m span) with higher softening only then occurring if the immediate 3 m roof section is allowed to displace excessively.

Consistent with the above; in the development of AMCMRR (Colwell and Frith, 2010) a 3 m roof section is utilised on the basis that the immediate 3 m roof section needs to be adequately reinforced to satisfactorily control roof deformation. In terms of engineering adequate roof stability; a fundamental basis of AMCMRR is to maintain and control the horizontal stress acting across the roof through the direct limitation of vertical roof displacement.

In using AMCMRR it is critical that the user understand that the Factor of Safety (i.e. lateral load bearing capacity/applied load over the 3m roof section) has the following general definition:

\[ \text{"It is a Factor of Safety against the onset of a process (i.e. stress driven roof deterioration), that if allowed to sufficiently propagate, could lead to a major roof fall"} \]

Typically with 10 mm to 20 mm of total roof displacement the roof is still under reasonable control (i.e. the roof achieves a load balance equilibrium), however it is once these levels are exceeded that there is a far greater likelihood for a loss of control leading to a roof fall. AMCMRR employs a design process to reinforce those slender beams that would form in the immediate 3 m roof section, via the reinforcement mechanisms of beam building (i.e. bolts and cables) and mechanical advantage associated with longer pre-tensioned cables anchored well above the 3 m roof section, so as to resist buckling and provide an adequate FOS to prevent further roof deterioration that could lead to a roof fall. The interested reader is referred to Colwell and Frith (2010 and 2012) for a complete description of how these reinforcement mechanisms and the related mathematical equations are readily employed.

To further reinforce the above discussion with respect to the critical nature of the 3 m roof section to overall roof stability further evidence is to found via ACARP Project C19008 from which the ADFRS design methodology was developed (refer Colwell and Frith, 2012). With respect to the two-pass faceroad dataset (involving 207 cases) it was found that for the 134 satisfactory cases the average TRD was 13.9 mm with an average height of softening (HOS) of 2.99 m, while for the 33 manageable cases the average TRD was 42.5 mm with an average HOS of 5.20 m and for the 40 unsatisfactory cases the average TRD was 101.7 mm with an average HOS of 5.84 m.

Colwell and Frith (2012) also found the AMCMRR 1st Pass FOS to be an excellent predictor of the overall success of the fully widened faceroad. Within ADFRS it is referred as the 1st Pass Reinforcement Index (RF\textsubscript{5m,yield}) and provides both an analytical/mechanistic and empirical basis in relation to the roof reinforcement required in respect of (in this instance) the 5 m roof section associated with the 1st pass drivage prior to widening to ensure a successful outcome.

CONCLUSIONS

From the “apple hitting Sir Isaac Newton on the head”, engineering methods have been developed in the past based on observing the real world, attempting to derive mathematical equations and algorithms that allow those observations to be replicated in terms of cause and effect, and testing predictions back in the real world. It is commonly known as The Scientific Method.

Unfortunately, in the field of coal mine strata control it is now evident that some cause and effect research is being founded in mathematical models with selective observations or measurements being used to justify the findings. This is an exact reversal of the process that has served mankind well for centuries and if allowed to proliferate must inevitably lead to a lessening rather than improvement in our fundamental understanding of the real world.

The irrefutable evidence for this warning is the recent dismissal of Euler Buckling as a dominant behavioural mechanism in coal mine roadway roof and ribs by reference to numerical modelling simulations. This goes directly against physical modelling findings, many observational reports and
around 40 years of empirical/mechanistic research studies within the Australian coal industry which are founded in the principles of slender beam/column behaviour and have returned very high statistical correlations between cause and effect.

It is a matter for each professional engineer to decide on their methods of analysis when addressing engineering problems, this paper having been written in an attempt to provide them with an evidence-based set of arguments when making their selections.

Finally, if it looks like a duck, swims like a duck, and quacks like a duck, then it probably is a duck. In terms of coal mine roof and ribline behaviour the “duck” is called buckling due to axial loading.

REFERENCES


COASTAL RESERVOIR-A TECHNOLOGY TO SUPPLY SUFFICIENT, HIGH-QUALITY AND AFFORDABLE WATER TO INDUSTRY WITH MINIMUM ENVIRONMENTAL/SOCIAL IMPACT

Shu-Qing Yang

ABSTRACT: This paper discusses how to supply sufficient water to meet industrial water demand. Australia is not running out of water, but water is running out of Australia's rivers. About 440 km$^3$/year of runoff is lost to the sea, and the total water usage is only about 5-6% of the water availability. Australia is one of the most resourceful countries in the world. All industrial water for the coastal areas can be supplied from coastal reservoirs, even the high-quality cooling water for steel making. To supply sufficient water to inland areas, the author has suggested that the Murray-Darling basin's cotton farms should be relocated to downstream areas near its coastal reservoir, thus the agricultural water demand is fully met and also the environmental flow is increased significantly. Therefore, its existing dams can be used for mining industry. Water pipelines may be needed to pump water from these dams to the mining sites, trains can also transport water bags from ports to inland areas.

INTRODUCTION

Water scarcity has always been a contentious issue in Australia, the driest inhabited continent in the world where about 70% of its land is classified as arid or semi-arid. Australia's rainfall and runoff are the lowest when compared with other continents as shown in Table 1.

<table>
<thead>
<tr>
<th>Continent</th>
<th>Area (million km$^2$)</th>
<th>Rainfall (mm)</th>
<th>Runoff (mm)</th>
<th>Runoff (km$^3$)</th>
<th>%Runoff</th>
</tr>
</thead>
<tbody>
<tr>
<td>Africa</td>
<td>30.3</td>
<td>690</td>
<td>260</td>
<td>7900</td>
<td>38</td>
</tr>
<tr>
<td>Asia</td>
<td>45</td>
<td>600</td>
<td>290</td>
<td>13000</td>
<td>48</td>
</tr>
<tr>
<td>Australia</td>
<td>7.7</td>
<td>465</td>
<td>57</td>
<td>440</td>
<td>12</td>
</tr>
<tr>
<td>Europe</td>
<td>9.8</td>
<td>640</td>
<td>250</td>
<td>2500</td>
<td>39</td>
</tr>
<tr>
<td>North America</td>
<td>20.7</td>
<td>660</td>
<td>340</td>
<td>6900</td>
<td>52</td>
</tr>
<tr>
<td>South America</td>
<td>17.8</td>
<td>1630</td>
<td>930</td>
<td>16700</td>
<td>57</td>
</tr>
</tbody>
</table>

The average annual rainfall over the whole country is 465 mm, with 87% of this total rainfall lost through evapo-transpiration and only about 12% of the rainfall or 57 mm on average enters the streams, the remainder 1% becomes groundwater. Moreover, there is wide variation in streamflow, both seasonally and annually, with the highest annual streamflow in some large rivers exceeding the annual mean by a factor 300. The wetter areas are all confined to the coastal and mountainous parts of the continent (see Figure 1).

Australia's rivers drain a total runoff volume of about 440 km$^3$ to the sea annually (Yang 2015, Yang and French 2018), estimates of this vary between 343 and 465 km$^3$. The uneven distribution of water resources in space can be seen from the data: about ¼ of the continent contributes 88% of the runoff, it being highest in Tasmania, Northwest Queensland and part of Western Australia. Tasmania accounts for less than 1% of the area of Australia but is responsible for 14.5% of total runoff. Less than 5% of the area of the continent can boast a runoff in access of 250 mm annually. Australia's water resources are highly variable (see Table 2), and this reflects the drastic variation of climatic conditions and terrain, and it is difficult for industry to use. In addition, the level of development of Australia's water resources ranges from heavily regulated rivers and ground-water resources to rivers and aquifers in almost pristine condition. Most large urban cities and dams are situated in the southern regions of Australia with Industry and irrigated agriculture principally located in the Murray Darling Basin where only 6.1% of the national surface water resources reside. Therefore, while Australia has

---

1 Assoc. Prof., School of Civil, Mining & Environmental Engineering, University of Wollongong, Australia; shuqing@uow.edu.au
significant water resources, the populations and agricultural activities are concentrated where water resources are most limited (Table 1).

Currently, Australians extract about 1 70-80 km\(^3\)/year of water and only about 18-25 km\(^3\)/year of water is used, with the remainder returned back to the environment after being used for applications such as hydropower. Table 3 shows that more than 75% of used water is related to irrigated agriculture.

Table 2: Comparison of river flows between Australia and others

<table>
<thead>
<tr>
<th>Country</th>
<th>River</th>
<th>Ratio between the maximum and the minimum annual flows</th>
</tr>
</thead>
<tbody>
<tr>
<td>Switzerland</td>
<td>Rhine</td>
<td>1.9</td>
</tr>
<tr>
<td>China</td>
<td>Yangtze</td>
<td>2.0</td>
</tr>
<tr>
<td>Sudan</td>
<td>White Nile</td>
<td>2.4</td>
</tr>
<tr>
<td>USA</td>
<td>Potomac</td>
<td>3.9</td>
</tr>
<tr>
<td>South Africa</td>
<td>Orange</td>
<td>16.9</td>
</tr>
<tr>
<td>Australia</td>
<td>Murray</td>
<td>15.5</td>
</tr>
<tr>
<td>Australia</td>
<td>Hunter</td>
<td>54.3</td>
</tr>
</tbody>
</table>

The first National Survey of Water Use in Australia was published in 1981, and reveals that the country’s annual water use was 17.8 km\(^3\), or 3500 litres per person daily. Some 74% is used for irrigation with a further 8% used for other rural purposes. The remaining 18% of water is used for urban and industrial purposes. Australia has one of the highest per capita consumptions of water in the world.

Of all the water consumed as shown in Table 3, 85% comes from surface water sources, the Murray and Darling Rivers provide almost ¾ of all water used. However, in different parts of Australia, the exploitable proportion of the runoff depends on local factors. In the northern rivers, because of their dramatic change in flow-rates, only about 20% of the runoff is available. In the Murray and Darling River, the proportion available for use is estimated at 83%. Over the entire continent, the average exploitation of runoff is about 13%, a figure that seems to suggest that Australia’s water resources are poorly utilised. However, most of the available water supplies are so far from population centres that they are uneconomic to use—highlighting the problem of uneven distribution of people and water supplies in this country and the development cost required for their use is far too expensive.
Australia is blessed as a country with abundant mineral resources. Australia earned $202 billion or 10.4% of GDP from mining while agriculture contributed 3% (about $50 billion) to GDP in 2019. Australia is number 10 in the list of countries with natural resources, comparable with USA. Australia has large reserves of coal, copper, iron ore, nickel, oil shale, and other rare metals. The country has the world’s largest gold reserves, supplying over 14% of the world’s gold demand and also 46% of the world’s uranium demand. Australia is the top producer of iron ore, lead, rutile, tantalum, uranium, zinc and Zircon. Australia was in the world’s top four exporters of black coal and sixth for brown coal in 2018. Mining in Australia has long been a significant industry and a major contributor to the Australian economy in relation to export income and employment. Historically, mining booms have also led to population growth via immigration to Australia, e.g., the Australian gold rushes in the 1850s produced 40% of the world’s gold at that time. Many different ores, gems and minerals have been mined in the past and a wide variety are still mined throughout the country.

Table 3: Comparison of Annual water availability/used in Australia between 2000-2001 and 2018-2019 in GL.

<table>
<thead>
<tr>
<th></th>
<th>2000-2001</th>
<th>2018-2019</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean annual rainfall</td>
<td>3,200,000 (352 mm/yr)</td>
<td></td>
</tr>
<tr>
<td>Mean annual runoff</td>
<td>387,184</td>
<td></td>
</tr>
<tr>
<td>Total water consumed</td>
<td>24,908</td>
<td>76,000</td>
</tr>
<tr>
<td>Agriculture</td>
<td>16,660</td>
<td></td>
</tr>
<tr>
<td>Forest &amp; fishing</td>
<td>27</td>
<td></td>
</tr>
<tr>
<td>Mining</td>
<td>401</td>
<td></td>
</tr>
<tr>
<td>Manufacturing</td>
<td>866</td>
<td></td>
</tr>
<tr>
<td>Electricity &amp; gas</td>
<td>1,688</td>
<td>51,300</td>
</tr>
<tr>
<td>Water supply, sewer &amp; drainage</td>
<td>1,794</td>
<td>1,810</td>
</tr>
<tr>
<td>Household water</td>
<td>2,182</td>
<td>1,810</td>
</tr>
<tr>
<td>others</td>
<td>3,973</td>
<td></td>
</tr>
</tbody>
</table>

Australia has mining activity in all of its states and territories as shown in Figure 3. The Minerals Council of Australia estimates that 2000 km² of Australia's land surface is directly impacted by mining and its distribution can be roughly seen from these maps, the important areas include the Goldfields, Peel and Pilbara regions of Western Australia, the Hunter Region in New South Wales, the Bowen Basin in Queensland, the Latrobe Valley in Victoria and the Murray-Darling basin. Places such as Kalgoorlie, Mount Isa, Mount Morgan, Broken Hill and Coober Pedy are known as mining towns. The major mining activities in 2018 are listed below:

Olympic Dam in South Australia produced 6% of the world’s copper, silver and uranium, the world’s largest uranium resource. The Super Pit gold mine in Western Australia has replaced a number of underground mines at Boulder.

Figure 2: Annual runoff from Australian Rivers
Australia is the world's largest exporter of coal and fourth largest producer of coal after China, USA and India. Coal is mined in every state of Australia except South Australia, and generally the coal mines are located in the coastal areas. 54% of the coal mined in Australia is exported, mostly to eastern Asia. Coal provides for about 85% of Australia's electricity production. In total, industrial water use including mining and manufacturing utilizes about 20% of all water consumed in Australia. Some cities and manufacturers need high-quality water, sometimes even better than the quality of drinking water, e.g. the cooling water for steel makers. The required salinity is only 50 ppm, lower than drinking water’s 250 ppm. Among all industrial water users, mining is a large user and this use is growing fast. Generally, the mining industry uses water in remote areas where it is often dry with low rainfall and runoff. This paper discusses how to supply sufficient, high-quality and affordable water to meet industrial needs. It is proposed that coastal reservoirs will harvest river runoff and store it in the sea to meet the coastal city’s water demand. The existing dams that currently supply water to these coastal cities can be used for the mining industry. The objective of this paper is to examine this proposal’s feasibility.

Figure 3: Australia iron ore and coal mining industry

COASTAL RESERVOIR TECHNOLOGY AND DOWNSTREAM WATER MANAGEMENT?

A coastal reservoirs is defined as a small water storage inside a large water body, separated by a barrier or barriers with some specific purpose, for instance irrigation, flood control, water supply, seawater intrusion control, and so on. (Yang and Kelly, 2015; Yang et al., 2013). As shown in Figure 4, the water inside the reservoir is different from the outside seawater in terms of chemical, physical and biological properties such as density, salinity, turbidity, nutrients, organic matter. For freshwater supply, the coastal reservoir could be simply defined as a freshwater reservoir inside seawater, with the main difference between the inside/outside waters being salinity (Yang et al. 2005).

Before the 1960s, almost all coastal reservoirs in the world were constructed for agricultural purposes due to food shortages and to solve problems like seawater intrusion prevention, land reclamation etc. Water shortages started in the 1960s when people began to migrate to coastal cities. Hong Kong was the first city in the world to experience water shortage problems caused by rapid urbanization, and the effective solution was to construct the Plover Cove coastal reservoir, where sea space is used to store rainwater. This large scale CR ended Hong Kong’s water shortage; this is also the world earliest modern coastal reservoir for a city’s water supply. Plover Cove’s construction work was completed in 1968. Its storage capacity was about 170 GL. In 1970, its 2 km long dam was increased to 28 m high, and its capacity increased to 230 GL (see Fig. 5). As mainland China has agreed to supply water to Hong Kong, this coastal reservoir is now used as a backup water source. However, Hong Kong’s experience demonstrates that it is technically feasible to store fresh water in sea space.
Shanghai is one of the world’s largest cities with a population of 24 million in 2019, situated to the south of the Yangtze River mouth. The Yangtze River is the third-longest river in the world. Shanghai is notorious for the water crisis caused by pollution from its mother river, the Huangpu. This river has been heavily polluted and international communities have long foreseen Shanghai’s water crisis. For example, in 1996 a conference organized by the UN Centre for Human Settlements (UN-Habitat) predicted that Shanghai would be one of a dozen cities with the most severe water crisis worldwide (N’Dow 1996) in the 21st century. The Shanghai Urban Master Plan in 2005 also predicted that the freshwater shortage in the city would reach 6 million m$^3$/d by 2020 (Lin et al., 2018). Surprisingly, the original idea to solve this megacity’s water problem comes from a near failed industrial project.

MINING INDUSTRY’S WATER DEMAND AND SOLUTION

Australia is a dry nation. In 2008–09, manufacturing, mining, food processing, electricity, gas supply and other industries consumed 2840 GL of water, which is about 20% of the total Australian water consumption for that year. Amongst the usage, 508 GL was used for the mining industry, compared to 582 GL of water in 1993-1994. In 2014-2015, Australia consumed over 17,000 GL of water across all sectors, and the mining industry used over 700 GL/year of water. In 2020 about 810GL-940 GL/year of water used by the mining industry is expected. It is believed that there is strong further growth for mining water as iron ore extraction and the coal seam gas industry start to bloom with exponential growth in coming decades.

There has been exponential growth in the production from most Australian mining industries, from metals to coal products since the 1950s. Coal production reached its highest level of approximately 815 Mt/year in 2008 up from 456 Mt/year in 1994. Iron ore has also grown from 129 Mt/year in 1944 to approximately 340 Mt/year in 2008. Increasing production requires large volumes of water to be used. The increasing production of coal and iron ore makes continuing access to water a critical imperative for the industry.

Different from agriculture, industry uses a very small fraction of water, but it tends to have very high economic return in dollars. Even so, Australia industry water demand is generally difficult to meet, especially for industries in the Hunter Valley or Murray-Darling River basin where few new water licences are available for industry to purchase. After the failure of Murray-Darling Basin plan in 2019, the total licensed extraction has been actually reduced for irrigation and industry. The high-security water entitlements that industry requires are infrequently traded, almost impossible to obtain from the water market. Alternatively, the lower security entitlements were purchased to meet industry needs.

As shown in Figure 3, most mining sites for coal and iron ore are located in arid or semi-arid regions where water is scarce, and agricultural water is competing with the industrial water in the Murray-Darling basin, and domestic water is also competing with the mining industry in West Australia. The mining industry in northern Australia and southern Australia is less competitive due to lower population and less agriculture. Groundwater is used in South Australia, northern Australia is wet. Mines in southern Australia are likely to experience lower water availability, more severe droughts, and full allocation of water to users. There is less chance of reduced water supply to mining in northern Australia under climate change (Prosser, 2011).

Figure 4: Conceptual coastal reservoirs to develop river flows otherwise lost to the sea
SOLUTION TO MEET MINING WATER DEMAND

The following suggestions are proposed to secure the water needs for Australian industry and mining: Coastal cities’ drinking water and industrial water are supplied from coastal reservoirs.

The Murray-Darling basin’s agriculture, especially the Darling River’s cotton farms are relocated to the river mouth and the coastal reservoir, i.e., Lake Alexandrina supplies the water for these farms. All existing dams that currently supply water to cities/agriculture should supply water to meet the nearby demand from towns, industry and mining industry. For those far away from dams/coastal reservoirs, the trains that deliver ore/coal to a port can be used to transport freshwater from the port’s coastal reservoir to the inland sites where water is needed.

The feasibility of the above strategy is investigated as follows:

In general, people like to live near the coast because of a better aesthetically pleasing living environment, access to a variety of recreational activities and more job opportunities. Consequently, 90% of Australia’s population live within 50 km of the coast, and almost all capital cities (except Canberra) are situated by the shore. As mentioned, Australia, every year uses about 20 km$^3$/year of water, only 4.5% of its runoff lost to the sea. Coastal reservoirs can supply sufficient water to meet the domestic, industrial and agricultural water demand from the coastal zones. Also, the water quality can meet the industrial requirements for electricity power plants, steel makers and mining engineering.

One of interesting examples from the 1980s occurred in Shanghai when China started its reforms and the opening-up policy for its economic development. The central government decided to build the world’s largest steel plant, the Baogang plant. Initially, the builders did not carefully check the Japanese design documents relating to the quality of cooling water. During the final stage of construction, the builders found that the specified salinity of the cooling water would need to be lower than 50 ppm, and much lower than tap water's salinity, 250 ppm. The Yangtze estuary's average salinity is much higher than this 50 ppm criterion. This problem could have potentially led to failure of this substantial investment. One of the steel engineers suggested the building of a coastal reservoir in the Yangtze estuary. He suggested that the intake gates of the reservoir will be open to take the river water during the lowest salinity period, but be closed when salinity is higher than 50 ppm. In 1985, the Baogang Reservoir was built, which has ensured the high quality of iron/steel products from this plant. Therefore, it is certain that coastal reservoirs in Australia can also supply sufficient high-quality water with salinity <50 ppm to manufactures for steel making or other purposes.

After analysing Australia rainfall and runoff data, it is concluded that Australia only uses about 20 km$^3$/year of water in total for its domestic, agricultural and industrial purposes, only 4.5% of its runoff to the sea. Australia is not running out of water, but water is running out of our river mouth during flood periods. By developing the runoff lost to the sea using coastal reservoirs, we can supply sufficient, high quality and affordable water to meet the water demands for coastal areas. Therefore, all the existing dams shown in Figure 6 can be used by inland or highland industry or agriculture.
Can we transfer water from the dams to meet mining industrial needs? The Goldfields Water Supply Scheme or the Goldfields Pipeline is a successful example implemented in Australia. It is probably the longest water supply pipeline in the world, in total it carries water over 530km from Perth to Kalgoorlie driven by eight pumping stations. The water is raised up by 411.5 m in elevation over the Darling Scarp ridge. This scheme is also probably the oldest project of a scale built over such a long distance. During the gold rush period, a 0.76 m diameter pipeline was constructed in 1903 with a capacity of 23,000 m³/day. Until 2015, this pipeline continued to supply water to more than 100,000 people, mines, farms and other businesses in the inland region of West Australia. It generates billions of dollars of annual economic return. In 2009, the American Society of Civil Engineers listed the scheme as an international Historic Civil Engineering Landmark. On 23 June 2011, Australia added the scheme to its National Heritage List. Obviously, this is a model for other places, and the dams in Figure 6 can nourish Australia inland regions, especially for its mining industry.

Currently it is almost impossible to increase water for mining and industry in the Murray-Darling basin as it has only 6% of the country’s water resources, but it produces about 2/3 of Australia’s food and fibre. In 2007, the Federal Government decided to form a special department called the Murray-Darling Basin Authority (MDBA) to find a win-win solution for environment and agriculture. In 2012, the Murray-Darling Basin Plan (MDBP) became a legal document and put into effect. The core of the plan is to cap the used water, called Sustainable Diversion Limit (SDL) which determines how much water, on average, can be used by towns, communities, farmers and industries. Its target is that, by 2019, an additional 2750 GL/year of water is returned back to the river system. Its purpose is to ensure a healthy working basin faced with climate change. In 2019-2020, the abolition of the MDBP means that a new plan should be proposed, and the author has suggested the downstream water management, i.e., to relocate all cotton farms to the downstream area near the river mouth where a coastal reservoir is located (see Figure 7).

Australia is a flat country, the seawater driven by a 1m high tide and density difference can propagate upstream some 250 km in dry years from the river mouth. In order to protect farmers’ crops, 5 barrages was installed at the lower lakes’ outlets in 1930s, and Lake Alexandrina and Lake Albert become Australia’s first coastal reservoirs. In 1960s, cotton farmers from California, USA were forced to close their business as the water was needed for drinking water. After accepted compensation, some of these farmers moved to the Murray-Darling basin. The first step was to construct dams for their irrigation as shown in Figure 7. If downstream water management was used from the 1960s, i.e., these farmers were invited to settle around the reservoirs and if they used these coastal reservoir, today we should have had no water crisis in the basin. We can let 100% of river flow run to the lower lakes first for environmental purpose first, then the water is reused for agricultural purpose, finally the
worst quality after many times’ reuse is discharged into the sea. Unfortunately today only 20 GL/year of the lake water is used, almost no agricultural activities are using the freshwater source.

Once the new MDBP is adopted, the existing dams in Figure 7a can be used for industrial and drinking purpose. Therefore, the agricultural output in the basin is increased, more than 2750 GL/yr of water is returned back to the river system, and also the industrial/drinking water is secured. We will have no difficulty to meet the mining industry’s water demand.

Figure 7: Dams and cotton farms’ distribution in the Murray-Darling Basin (left). It is suggested to relocate all cotton farms in the left to the area enclosed by red line, thus the dam water can be used for industry and drinking purpose.

Similarly, the for Hunter River basin, a coastal reservoir can be constructed at the mouth of the Hunter River to provide the water supply for coastal regions. Once Newcastle and the central coast’s water supply needs are met, the existing dams can be used for mining purposes. Grahamstown Dam and Chichester Dam in this region are large enough for coal mining operations.

Once coastal reservoirs are constructed along the coastline, coal and ore ports will have abundant supplies of water and trains that unload the ore/coal to the ports can be used to transport water bags from these ports back to the places where the water is needed. Australia has constructed a total of 36,064 km rail and for any new ore/coal development, new railways will be the 1st pre-requisite, all can be used for water bag transport in a cost effective and environment-friendly way.

CONCLUSIONS

Australia is a great and resourceful country, and also the driest inhabited continent in the world. Water scarcity has affected the country’s industry activities. The country’s data reveals that Australia is not running out of water, but water is running out of Australia’s rivers. Every year about 440 km$^3$/year of runoff is lost to the sea, and the total water usage is only about 5-6% of its water availability. If coastal reservoirs are constructed at its river mouths, the coastal water demand from domestic, industrial and agricultural sites can be fully met. More than that, the inland water demand from industry can be also met as the existing dams can be diverted from supplying water to coastal cities to inland regions. The follow conclusions can be drawn from this paper: Once coastal reservoirs are constructed, all industrial water from coastal areas can be supplied from coastal reservoirs, even the high-quality cooling water used by steel makers.

To supply sufficient water to the inland areas, the author has suggested that the Murray-Darling basin's cotton farms should be relocated to downstream area near its coastal reservoirs, thus the agricultural water is fully met and also environmental flow is increased significantly. Similarly, if Newcastle’s water supply comes from its coastal reservoir, Hunter River’s dams can supply water for its coal mining. Generally, all existing dams can be used for the mining industry in inland areas. Water pipelines may be needed to pump water from these dams to the mining sites, trains can also transport water bags from ports to inland areas.
REFERENCES


WATER TRACER TECHNOLOGIES TO DETECT SOURCES OF SEEPAGE AND PROTECT ENVIRONMENTAL ASSETS

Wendy Timms¹, Devmi Kurukulasuriya¹, Bill Howcroft¹, Ellen Moon¹ and Karina Meredith²

ABSTRACT: Water tracer technologies can help optimise water management in coal mining operations and improve outcomes from environmental studies and controls to protect sensitive assets. ACARP project C28024 (Stage 1) is demonstrating how tracer analysis of groundwater and surface water can provide information on whether systems are hydrologically disconnected, partly connected or well connected. This stage of the project is focusing on conventional tracers that are often used by other mining industries around the world (e.g. iron ore, potash) and in groundwater resource studies. Stage 2 of this project proposes to test new artificial tracers combined with suitable conventional tracers that are particularly useful for identifying seepage sources for control actions.

This paper will demonstrate and discuss the benefits and limitations of major groups of conventional tracers that are commonly measured naturally in water. These include: field parameters (e.g. electrical conductivity, temperature), major and trace ions (e.g. metals), stable isotopes of oxygen and hydrogen, industrial compounds (CFCs and SF6) and dissolved carbon isotopes (i.e. inorganic and organic forms). In addition, this paper will discuss radioisotope tracers (e.g. tritium, carbon-14 and radon-222), as robust and proven tools to help differentiate shallow and deep groundwater where there is a contrast in water residence time (groundwater ‘age’). These tracers can provide useful information on seepage, despite higher analysis costs and turn-around times for laboratory results.

Key findings from demonstration mine sites show the importance of combining physical water measurements (e.g. water levels and pumping rates) with a suitable combination of water tracers, depending on the site specific issues or study questions. For example, artificial tracers that are added to water sources are most suitable for identifying seepage and rapid flow pathways that can be a risk to underground operations. However, common artificial tracers such as added salts and dye tracers can also raise community concerns, such as producing fluorescent green creeks. Novel artificial tracers are able to overcome these risks. For example, synthetic DNA with uniquely designed fingerprints can be released at different times and locations to identify the sources of water to excavations can then be controlled.

Commensurate with the risks of the project, a combination of suitable tracer technologies of different types can increase the confidence in identifying water sources and flow rates underground. However, the costs, limitations and practical challenges of each proposed tracer should be considered in planning tracer studies. The outcomes of these ACARP projects will assist coal mining operators in deciding on the suitable combinations of tracers for different types of operational and environmental risks associated with underground mining, and show how tracer technologies can be used to check possible flow paths in conceptual and numerical models.

BACKGROUND

Water tracer technologies can help identify seepage sources for target controls and improve the outcomes for environmental studies and controls to protect sensitive assets. Underground mining assets need to be protected from flooding of the workings, whilst water resource assets such as surface waters and wetlands can be sensitive to relatively small losses of water. Identifying the source of groundwater flows to excavations, for example, from a mix of lateral inflows in a coal seam, and vertical seepage from specific overlying aquifers, is a first step in adaptive management of groundwater.

¹ School of Environmental Engineering, Deakin University, Waurn Ponds, VIC, 3216, Australia. Email: wendy.timms@deakin.edu.au Tel: +61352278692
² Australian Nuclear Science and Technology Organisation, Institute for Environmental Research, NSW 2232, Australia
Water tracers are naturally occurring hydrochemical, isotope or dissolved gases in water or soil water. These environmental tracers can be used to ‘fingerprint’ water sources and can show distinct zones of water due to hydraulic separation or partial disconnection. Importantly, these tracers are averaged over time (e.g. years, decades), and spatially at scales that can be useful to verify numerical models. Water tracers provide a missing link between current hydrogeological-geomechanical approaches and environmental conditions. The resource industry would benefit by quantifying flow and mixing that cannot be distinguished by other methods with the accuracy and confidence of multiple tracer technologies.

This paper is part of ACARP project C28024 (Stage 1) providing background information on types of water tracers, along with applications and benefits. Examples of mine locations where water tracers have been used are presented, along with steps to decide on suitable water tracers. Two water tracers studies at underground coal mines in Eastern Australia demonstrate selected water tracers for the purpose of a) evaluating potential for surface water-groundwater interactions and b) evaluating how much modern water has seeped to a deep coal seam that would naturally contain ‘old’ water that is drained for longwall operations.

Applications and benefits of water tracers in mining

Water tracers have many applications that can benefit mining at various stages from feasibility to operations and to mine closure.

Some of the applications for water tracers include the following:

1. Identifying source(s) of flow to a void
2. Evaluating hydraulic connectivity and surface water-groundwater interactions
3. Evaluating hydraulic disconnectivity - effectiveness of aquitards and hydraulic barriers
4. Aquifer recharge - age or residence time, sustainable yields
5. Aquifer interference – estimating seepage between aquifers
6. Water mixing - quantifying water mixes, discharge or baseflow in surface waters

Water tracer tools should be considered commensurate with risk, with water tracers increasingly utilised if there are operational risks of inflow to mining voids, and/or there are environmental risks to water assets (Timms et al. 2012). For example, water tracers could complement water information as part of the evaluation of proposed projects or expansions near particularly sensitive water resources and Groundwater Dependent Ecosystems (GDEs).

State of the art in water tracing tools means choosing from an increasing number of different water tracers, and the latest in appropriate tracer technology that will be useful to quantify water flows and aquifer storage. However, requirements for proponents to use specified tracers (e.g. carbon-14) may may limit useful outcomes (e.g. water-coal effects on carbon-14 may require advanced data interpretation). The Independent Expert Scientific Committee on Coal Seam Gas and Large Coal Mining Development (IESC) has provided advice to decision makers for a number of coal projects in NSW and Queensland that, rather than being prescriptive, a range of suitable environmental tracers can be considered for some projects (advices are publicaly available at www.iesc.environment.gov.au).

Some of the many benefits of using water tracers include:

- support corporate environmental responsibility and social licence to operate
- contribute to regulatory and community engagement using leading water technologies
- strengthening the science around potential environmental effects of mine water
- evidence of different water sources in mining voids, reduce risk to operations
- improve effectiveness of water treatment technologies by quantifying source types
- differentiate non-mining and mining effects on surface waters near mine sites
• constrain computer models where parameterisation and boundary conditions are often uncertain with a range of possible model outcomes
• improve evaluation of differences in empirical and model predictions and a gap between geomechanical and hydrogeological approaches
• assist with evaluating potential aquifer interference and cumulative impact
• contribute to multiple lines of evidence on the degree of hydraulic connectivity or disconnection

While there are clearly many possible benefits to using water tracers in mine water studies, multiple lines of evidence are essential. Thus water tracers can complement, not replace, other geological and hydrological information including monitoring of water levels and flows.

International and Australian mining examples of water tracers

Water tracers are increasingly utilised in mining studies locally and internationally (Figures 1-2), and across several mining sectors including iron ore, uranium, metal and coal mining. There have been several coal mining related water tracer studies in China, the USA and Australia. These selected examples of tracer studies in mining areas include feasibility studies for mines, operational mines, and sites closed and being restored, however may not be comprehensive, particularly as some water tracer studies are not publically available.

![Figure 1: International examples of mine study locations using water tracers](image)

Open cut coal mines have used hydrogeochemical tracers while other resource industry studies such as ACARP C11050 have recommended tracer and isotopes for management of mine closure. Isotopes have been used to trace biogenic nitrogen ($^{15}$N/$^{14}$N) in coal seam gas (Saghafi et al. 2012) and helium gas to evaluate connectivity above a longwall panel. A more comprehensive review of publically available reports of water tracers applied in the various mining resources sectors is currently being finalised.
Figure 2: Australian examples of mine study locations using water tracers

CONVENTIONAL TRACERS

Conventional tracers use physical aspects of the water molecule, dissolved ions or dissolved gases in water, even if influenced by human activity. In contrast, artificial tracers involve the intentional addition of chemicals or substances such as a dye or salt tracer. This stage of the project is focusing on conventional tracers. Table 1 summaries general groups of conventional water tracers, provides examples, relative costs and priority questions or applications for each mine site.

Field water quality parameters (Electrical Conductivity (EC), pH, temperature, dissolved oxygen (DO)), are widely employed in hydrogeologic investigations, are low in cost and, can be measured at the time of sampling using a calibrated, hand-held water quality meter.

Other major groups of water tracers include major and trace ions (e.g. cations including metals, anions including bicarbonate), stable isotopes of oxygen and hydrogen, industrial compounds (CFCs and SF6) and dissolved carbon isotopes (i.e. inorganic and organic forms) and radio-isotopes (e.g. 3H). The presence of young water in mines can be assessed via the sampling and analysis of the young water tracers: tritium (3H), chlorofluorocabons (CFCs) and/or sulphurhexafluoride (SF6). Tritium can be used to determine mean residence times (MRTs, the average time since recharge) up to about 150 years, while CFCs and SF6 can be used to determine MRTs up to about 70 years (Cartwright et al. 2017).

Analytical costs for investigation and advanced tracers tend to be high, laboratory turn-around times may be long, and interpretation of the data is typically requires use of lumped parameter models (LPMs) such as TracerLPM (Jurgens, Böhlke, and Eberts 2012). Radio-isotopes are robust and proven tools to help differentiate shallow aquifers and deep groundwater where there is a contrast in water residence time. These tracers can provide useful information, despite higher analysis costs and longer laboratory turn-around.

Fact sheet guides are currently in preparation for the resource industry that provide a useful summary of the major types of basic and investigation tracers, sampling and preservation requirements and interpretation. These will also consider limitations of tracers, such as assumptions for interpretation.

Standard water sampling protocols, particularly for groundwater, need to be considered when designing and implementing a water tracer sampling campaign (Sundaram et al. 2009).
Table 1: Water tracer groups and examples

<table>
<thead>
<tr>
<th>Tracer group</th>
<th>Which water samples?</th>
<th>Example water tracers</th>
<th>Application questions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic</td>
<td>all water, on site measurement</td>
<td>Field water quality parameters (e.g. EC, Temp, DO)</td>
<td>1-6</td>
</tr>
<tr>
<td>Basic</td>
<td>all waters</td>
<td>Major cations and anions (including bicarbonate)</td>
<td>1-6</td>
</tr>
<tr>
<td>Basic</td>
<td>all waters</td>
<td>Stable isotopes (e.g. $^{18}$O, $^2$H)</td>
<td>1-6</td>
</tr>
<tr>
<td>Investigation</td>
<td>all waters</td>
<td>Trace elements (e.g. Sr, rare earth elements)</td>
<td>2, 3, 5</td>
</tr>
<tr>
<td>Investigation</td>
<td>selected samples, on site measurement</td>
<td>Radon-222 ($^{222}$Rn)</td>
<td>1, 2, 6</td>
</tr>
<tr>
<td>Investigation</td>
<td>selected samples</td>
<td>Dissolved Anthropogenic gases (e.g. CFCs, SF$_6$)</td>
<td>4</td>
</tr>
<tr>
<td>Investigation</td>
<td>selected samples</td>
<td>Radio-isotopes (e.g. tritium $^3$H)</td>
<td>4</td>
</tr>
<tr>
<td>Advanced</td>
<td>selected samples</td>
<td>Radio-isotopes e.g. chlorine-36, carbon-14 (including dissolved inorganic and organic carbon)</td>
<td>4</td>
</tr>
<tr>
<td>Advanced</td>
<td>selected samples</td>
<td>Trace element isotopes (e.g. Si)</td>
<td>2, 3, 5</td>
</tr>
<tr>
<td>Other</td>
<td>surface &amp; shallow aquifers</td>
<td>Nutrients, total carbon and dissolved organic carbon, natural fluorescence, advanced dissolved gases, e-DNA (environmental DNA), dissolved noble gas isotopes (e.g. $^{39}$Ar), compound specific isotopes e.g.; algae-biomarkers</td>
<td>1-6</td>
</tr>
<tr>
<td>Artificial</td>
<td>added to waters on site if flow directions/rates known</td>
<td>Flourescent dye tracers, salts (e.g. chloride), micron scale particles, xeno-DNA (synthetic DNA markers)</td>
<td>1, 2, 3, 6</td>
</tr>
</tbody>
</table>

Steps to selecting suitable water tracers

This project has developed and tested a decision framework to assist in selecting suitable water tracers for mine sites. This framework will boost the cost-effectiveness of tracers that are applied, partly by providing reasons why some tracers that are recommended or familiar, may not be suitable, depending on site specific applications, risks and available information.

Steps that are part of deciding on water tracers for a mine site include the following:

1. What are the key risks and questions for mine water at the site?
2. What are a suitable suite of tracers, commensurate with risks to mining operation and risks to environmental assets?
3. How can these tracers complement and extend other water information that is available such as flow directions and water levels?
4. What are the costs and sampling requirements of suitable water tracers?
5. What are sampling locations, aquifer depths and reference (non-mining) sampling sites are needed?
6. Is a once-off sampling campaign sufficient, or is repeat sampling of water tracers needed?
7. Are there distinctive ‘end-members’ or variations in water tracer concentrations or activities across different waters and aquifers at the site?

8. Are the tracers mostly non-reactive, or it is possible to account for decay and/or geochemical interactions along the flow path?

9. What information and assumptions are required for interpretation of water tracer results and modelling that may be required?

10. How will water tracer data be used to improve confidence in mine water decisions and modelling of surface water and groundwater systems?

This decision framework was tested as part of this project, to determine that some tracers that were not suitable (e.g. nutrients) at a demonstration site. This process included considering whether nutrients found in surface waters and wetlands could be useful as a tracer of downwards seepage into shallow fractured rock aquifers. However, a review of available water quality data from the site revealed very low concentrations of nitrate and phosphorous. This information, combined with the fact that nutrients are reactive, and not an ideal tracer unless at high concentration with distinctive end-members, led to a decision that it was not a suitable and cost-effective tracing method in this case.

Coal contains ‘old’ carbon. When groundwaters migrate through this material, the addition of ‘dead’ carbon can occur. Therefore, it has also been found that uncertainties yet to be resolved can occur when using carbon-14 to date water from coal seams. Further research is needed to quantify the addition of ‘dead’ carbon during water-coal interactions to develop more reliable groundwater age models.

**Water tracers for evaluating hydraulic connectivity**

Mining excavations below the water table, subsidence, and withdrawal of water consequently result in high fracture porosity, permeability, altering of hydraulic connectivity, gradient and the flow network between the surface water and the subsurface aquifers. Therefore, mining has the potential to intensify the natural surface water groundwater interactions and affect the connected water resources qualitatively and quantitatively. Chemical characterization based on environmental water tracers is a common approach used to understand these interactions (Dhakate, Modi, and Rao 2018; Guo et al. 2019; Huang et al. 2017). Analysis of basic tracers such as major ions helps understand the hydrochemical and the geochemical processes the water had undergone and helps in determining the evolution of groundwater. This characterization and the change in water over time is used to identify possible contamination due to mining, sources of origin or mixing of end members in hydrogeological assessments.

Water quality assessments using major ions, trace element concentrations and their isotopes, and rare earth analysis helped identify discharge of saline groundwater from fractured streambeds, potentially caused by subsidence of a longwall mine by Morrison, Reynolds, and Wright (2019) and is vital information for the management of any plausible detrimental effects on ecological system of surface water streams.

Stable isotopic compositions ($^{18}$O, $^2$H) help delineate the recharge sources because they can represent recharging rainfall or surface water and can be used to identify evaporation. Having similar stable isotopic composition in groundwater and surface water in Ningtiaota Coalfield in China (Huang et al. 2017), revealed that some nearby rivers were mainly recharged by groundwater and weak inter aquifer hydraulic connection. Alternatively, Guo et al. (2019) identified the mine water sampled 5km away from a river to have similar major ion and $^{18}$O and $^2$H composition indicating drawing of river water at the dewatering bore of the mine.

Recharge rates and the impact on base flow rates of rivers can be estimated quantitatively by integrating the $^{18}$O and $^2$H data with groundwater level data and groundwater flow modelling. Similarly, the basic tracers can be used to estimate the spatial extents of the surface water-groundwater interactions, which is a critical component in assessing the impacts on surrounding water resources and narrowing down the boundary conditions in flow models. These critical insights will be very useful in water resource management and decision making on preventive measures such as mine water inrush control (Huang et al. 2017), buffer zones (Guo et al. 2019), or seepage barriers (Dhakate, Modi, and Rao 2018).
Radon can be further used to estimate the degree of mixing of surface water and groundwater (Stellato et al. 2013). Estimates of surface water-groundwater interaction can be determined by multiple lines of evidence including tracers that are produced in the subsurface such as radon. Their concentrations are higher in deep groundwater whereas it degasses as it reaches surface water and it helps trace upwelling of groundwater (Baskaran et al. 2009; Cook et al. 2003).

DEMONSTRATIONS OF WATER TRACERS AT COAL MINES

Two case studies are presented that demonstrate the use of water tracers for different applications at coal mines. The applications are firstly for surface water-groundwater interactions near underground coal mines, and secondly for estimating the percentage of modern water to a coal seam at a longwall mining operation.

Water tracers of potential surface water-groundwater interactions

A new environmental tracer study is in progress to help evaluate the potential for surface water-groundwater interactions near longwall coal mines (at a depth of ~300 m) in NSW. ‘The Drip’, a heritage site, is also located in a national park and is considered an iconic gorge. ‘The Drip’ has formed within sandstone from the Triassic Period, and is a 35 m high overhanging cliff beside the river where water cascades over the rocks and drips on to the river even during dry periods.

A critical issue that emerged during this study was that the water quality parameters and the basic tracers of the river provided uncertain outcomes because treated mine water is discharged to the river. Therefore, the use of chemical and isotopic tracers in the surface water are limited since their compositions are partly derived from geochemical reactions taking place in groundwater. The use of basic tracers alone were not sufficient at this site to estimate surface water–groundwater interactions. The source of water to ‘The Drip’ is reported to be a shallow aquifer, however additional evidence using multiple environmental tracers is required to verify possible sources of seepage. The degree of hydraulic connectivity of ‘The Drip’ with shallow and deep aquifers in the area are yet to be determined.

Water chemistry, major ions and stable isotopes were used to characterise the water types at the site including groundwater and surface water and the water discharging from ‘The Drip’. Chemical characterization indicates the unique composition of ‘The Drip’ sample deviating from the other water samples collected from other Triassic sedimentary aquifers (Figure 3).

Figure 3: Piper plot of chemical facies that can trace different types of waters and mixing
The variation in pH and the EC of the water discharging from ‘The Drip’ and the river samples suggested that a shallow aquifer was feeding ‘The Drip’. The hydraulic heads of the groundwater in the Triassic sandstone indicates that ‘The Drip’ could be hydraulically connected to two nearby shallow aquifers. River water had a higher $^{18}$O and $^2$H values due to evaporation whereas the groundwater was similar to rainfall with lower values. The The average stable isotope values of ‘The Drip’ suggests a hydraulic connection between ‘The Drip’ water and a shallow aquifer.

The radon concentrations in the surface water samples were below detection limit whereas groundwater indicated high concentrations. Any surface water – groundwater interactions would have been evident if the surface water samples too had relatively higher radon concentrations. These findings can be further justified by using age tracers to determine the distribution of residence times, mixing of old and young water and to calculate percentage modern water or old water in a sample.

Other tracer results that are currently being evaluated for this site include dissolved gases such as CFCs and SF$_6$. These compounds are very useful tracers since their concentrations re-equilibrate with the atmosphere faster and gains the “finger print” of the surface water for less complicated end member analysis (Cook and Dogramaci 2019). Non-reactive (ie conservative) tracers such as tritium are also being used to evaluate mixing of distinct waters and the percentage of seepage and mixing of shallow groundwater with surface water.

Water tracers to estimate percentage of modern water to a coal seam

Modern groundwater, can recharge, or seep downwareds into an aquifer (e.g. a coal seam) via a number of physical processes: 1) infiltration if and where the mined resource outcrops at the ground surface, 2) infiltration of water, including both precipitation and surface water, through ground deformation including fractures that develop as a result of underground longwall mining, or 3) downward hydraulic gradients, either naturally occurring or that have been increased as a result of mine dewatering. Generally, multiple age tracers are used to estimate residence times of the water samples (Clark and Fritz 1997).

Tritium and water balance methods were used to trace the seepage of modern water (<70 years) to a coal seam that would naturally contain old water at the Dendrobium mine in NSW. Over 1000 tritium samples have been collected since 2004 with the most recent data used for binary mixing models to evaluate the percent modern water present in the samples (HydroSimulations 2019). Three areas of the mine goaf (1, 2A and 3A) recorded 10-26% of modern water (Figure 4). Probability distributions of modern water in these areas were produced by statistical analysis and the surface water component estimated using hydrograph baseflow separation technique. The results indicated a correlation between the modern water percentage and the high-volume rain events. However, there was little correlation between tritium content and 30-day rainfall and mine inflow rates.

![Figure 4: Use of Tritium to estimate mixing fractions of different water sources, Dendrobium Mine, NSW (HydroSimulations 2019)](image)
Further investigation into why tritium indicated modern water in the coal seam is considering operational water, dual porosity effects and mixing of water of different ages (HydroSimulations 2019). The use of suitable multiple tracers could overcome such uncertainties by better distinguishing processes that occurring during water seepage and mixing in such complex groundwater systems.

CONCLUSIONS

This paper has introduced water tracers for applications in coal mining, including identifying sources of seepage for controls, and to help evaluate the potential risks of mining for environmental assets. Water tracers are often used by other mining industries around the world (e.g. iron ore, potash, uranium) and in groundwater resource studies, and there are several examples of water tracers used in coal mining operations (e.g. in China, USA, and some in Australia).

ACARP project C28024 (Stage 1) will continue work at demonstration mine sites to evaluate the benefits and limitations of multiple water tracers. The steps to selecting suitable water tracers, as a decision support tool is being developed at two underground coal mine sites, along with preliminary water mix and geochemical models. Stage 2 of this project proposes to test new artificial tracers combined with suitable previously tested tracers to improve the effectiveness of tracers and to reduce environmental risks.

ACKNOWLEDGEMENTS

We acknowledge funding provided by ACARP that enabled this work.

REFERENCES


Guo, Qiaoling, Yunsong Yang, Yaoyao Han, Jianlin Li, and Xinyi Wang. 2019. 'Assessment of surface–groundwater interactions using hydrochemical and isotopic techniques in a coalmine watershed, NW China', Environmental Earth Sciences, 78.


FILTER REQUIREMENTS FOR GRAHAM’S RATIO OXYGEN DEFICIENCY

Snezana Bajic¹, Sean Muller² and Mladen Gido³

ABSTRACT: Graham’s ratio (GR) is used to calculate the amount of carbon monoxide produced in proportion to the amount of oxygen consumed by the coal. It is a useful indicator for Coal Mines to determine the level of coal oxidation and to respond accordingly in the event of spontaneous combustion. The intensity of the coal reaction is related to the carbon monoxide produced and the oxygen consumed (oxygen deficiency). Graham’s ratio is very important as it is often used as a trigger for Trigger Action Response Plans (TARPs) for the management of spontaneous combustion. Samples with a similar composition to air may return a negative or minuscule measured oxygen deficiency unsuitable for Graham’s ratio. The same problem is identified in samples diluted with seam gas or when there are inaccuracies in other measured components when nitrogen is calculated by difference and not directly measured. The issue arises when oxygen deficiency is inadequate and insufficient, where the GR result can be overestimated and trigger a TARP level. Some mine sites introduced a filter for minimum oxygen deficiency value to avoid alarm “fatigue” for a Control Room Operator (CRO). There are cases where this minimal value is not suitable and where valid oxygen values have been filtered. This paper will present the case studies where the filter value was adjusted to suit the mine site actual real data and analysis technique.

INTRODUCTION

Mines operating in Australia are responsible for managing and monitoring their own risks. The processes used to achieve compliance with Australian legislation are complex and well established. Each mine site develops its own Health and Safety system and Mines Principal Hazard Management Plan (PHMP) in order to identify and control principal hazards, such as spontaneous combustion. The data produced by a mine site gas monitoring system is a critical component in this process. Each mine has specific conditions and should base their trigger and alarm levels on what is a “normal” condition and not what is average level observed. The operational considerations for tube bundle systems (Watkinson, Bajic, Forrester, & Ryan, 2016) and consequences of misinterpreting trends (Watkinson & Bajic, Best Practice Gas Monitoring, 2019) were assessed by Simtars in past. To enable risk identification and early response, mines establish TARPs which outline trigger points and actions to prevent any incident from escalating.

Graham’s ratio is often used as a trigger for TARPs for the management of spontaneous combustion. This emphasises the importance of accurate measurement of oxygen deficiency and the ability to successfully determine the status of an underground atmosphere. If the oxygen deficiency is inadequate and insufficient, the Graham’s ratio result can be overestimated and trigger a TARP level. If the number of these false alarms is large, it can affect Control Room Operator (CRO) fatigue and introduce another risk of missing important non-false alarms (Bajic, Muller, & Gido, 2020).

Muller, et al., (2017) explains how raw carbon monoxide concentration is not always indicative of the intensity of a heating due to dilutions or accumulation of gases. By comparing carbon monoxide generated with oxygen deficiency, a more relative measurement can be made (Graham’s ratio). This measurement is independent of air flow and various forms of the equation account for dilution effects (Cliff, etal., 1999).

In order to incorporate the initial gas readings, and to incorporate nitrogen by difference to ensure the effects of dilution are included, the following equation is applicable:

---

¹ Technical services Manager, Simtars, snezana.bajic@simars.com.au; ph +61 475 808 493
² A/Senior Analytical Chemist, Simtars, sean.muller@simars.com.au; ph +61 447 202 238
³ Control Systems Engineer, Simtars, mladen.gido@simars.com.au; ph +61 477 718 027
Equation 1:

\[
\text{Graham's Ratio} = \frac{100 \times (\text{carbon monoxide}_{\text{final}})}{(0.265 \times \text{nitrogen}_{\text{final}}) - \text{oxygen}_{\text{final}}}
\]

Note that the constant 0.265 is simply the theoretical ratio of oxygen to nitrogen in air.

Equation 1 is commonly used to calculate GR on real time sensors underground. The assumptions in this instance are that the ratio between oxygen and nitrogen in the inlet stream is the same as in fresh air, and that inlet contains no carbon monoxide. This is not applicable for inlet stream that is depleted in oxygen, enriched in carbon monoxide or enriched in nitrogen.

It is very important for a mine to establish a database for its deposit individual conditions. Using a measured fresh air value and taking dilution into account is represented by the equation 2:

Equation 2:

\[
\text{Graham's Ratio} = \frac{100 \times \left(\frac{\text{carbon monoxide}_{\text{final}} \times \text{nitrogen}_{\text{final}}}{\text{nitrogen}_{\text{initial}}} - \text{carbon monoxide}_{\text{initial}}\right)}{\left(\frac{\text{oxygen}_{\text{initial}} \times \text{nitrogen}_{\text{final}}}{\text{nitrogen}_{\text{initial}}} - \text{oxygen}_{\text{final}}\right)}
\]

Equation 2 is a common equation used to calculate Graham’s ratio for tube bundle monitoring points in underground coal mines. The measured fresh air point is typically obtained from a point on the surface at the tube bundle building, or from an intake roadway underground. More detailed explanations are presented in Muller, et al., (2017).

The final oxygen value can never exceed initial oxygen, as oxygen is not generated underground. Analysing equipment has a typical tolerance of +/- 0.2 % and therefore a small error is expected. This occasionally can lead to higher final oxygen readings than initial oxygen readings. Negative oxygen readings will produce a negative Graham’s ratio, which is impossible result.

Muller, et al., (2017) states that current practice at mine sites is to apply the minimal oxygen deficiency requirement of 0.3 %, which eliminates the majority of non-reliable data points. Graham’s ratio is previously understood to be unreliable for oxygen deficiencies below 0.3 % (Brady, 2007). Strand (1985) state that the calculation is subject to analytical limits and that oxygen deficiency of less than 0.2 % would introduce gross errors. As the technology advanced since 1985, it is now possible to investigate, with greater confidence, the threshold value for this equation.

One of the mine disasters in the Queensland Moura region occurred on 7 August 1994 at Moura No 2 Mine. On this occasion eleven miners died as a result of an explosion. Moura No 2 mine is on the eastern side of the Bowen Basin in the state of Queensland 7 km to the east of the town of Moura. The Inquiry found that the first explosion originated in the 512 Panel of the mine and resulted from a failure to recognise, and effectively treat, a heating of coal in that panel. (Windridge, Parkin, Neilson, Roxborough, & Ellicott, 1996)

The analysis of Moura 2 mine disaster data provides evidence that filters over 0.2% are not suitable for every location. An investigation performed by Muller, et al., (2017) indicated that an oxygen deficiency of less than 0.3 % may still be reliable in some situations and generate critical data for underground air monitoring.

METHODOLOGY

Pre and post explosion data from the Moura No. 2 mine disaster was used for filter demonstration purposes. As per conditions set in Muller, et al., (2017), the minimum oxygen deficiency value selected was 0.05 %, as this value appears to be the lowest and most conservative value. Data in the form of tube bundle logs were obtained from gas monitoring software. Bajic, et al., 2020 provided examples of data obtained from two underground coal mines in Australia, which had previously experienced and flagged invalid Graham’s ratio triggers in their alarm logs and had their filter threshold points were set to 0.05 %. The locations containing low oxygen deficiencies (around 0.5 % or less) were chosen for the study.
Each relevant data log was extracted to a comma separated values file (CSV) containing the following information:

- Date and time of measurement and monitoring point number (location)
- Methane, Carbon Monoxide, Oxygen and Carbon Dioxide concentrations (%)
- Carbon Monoxide Make (litres per minute)
- Graham’s ratio – calculated
- Selected fresh air point was pump room.

In addition to these gas components, the Graham’s ratio calculated from the gas monitoring software, as per industry standards, was extracted with each set of gas readings (Muller, et al., 2017). These extracted data logs were processed in order to calculate a theoretical oxygen deficiency and theoretical Graham’s ratio values based on fresh air as the initial readings for real time data, and the fresh air point for tube bundle data. For several tube bundle locations the measured initial air values were used rather than the theoretical initial values. This allowed the Graham’s ratio calculation to be replicated as accurately as possible, reproducing the actual values calculated by the mine site monitoring system before extraction. Locations processed were 512 seals and return.

The calculated Graham’s ratio value for each measurement was categorised based on the following thresholds:

- Normal data was defined as any data with corresponding theoretical Graham’s ratio calculated at 0.2 or below. This range is often used as normal conditions for spontaneous combustion management TARPs in Queensland mines (Mines Rescue Gas Detection and Emergency Preparedness 2014).
- Investigate data is defined in this testing as any data with theoretical Graham’s ratio calculated at 0.2 to 0.4. This range is often used as an ‘investigate’ trigger for spontaneous combustion management TARPs in Queensland mines (Mines Rescue Gas Detection and Emergency Preparedness, 2014).
- An invalid trigger is defined as any data with theoretical Graham’s ratio calculated at over 0.4 without a corresponding significant increase in carbon monoxide or CO make.
- A valid trigger is defined as any data where the theoretical Graham’s ratio is calculated at over 0.4 with a corresponding significant increase in raw carbon monoxide or CO make associated with the data. By definition, any Graham’s ratios over 0.4 which are not valid triggers are considered invalid.

Filtering of tube bundle and real time data sets were based on minimum oxygen deficiency set points. Overall data retention, retention of normal data, investigate data points removed, invalid data points eliminated and valid data points eliminated were evaluated for each filtered data set.

RESULTS AND DISCUSSION

Previously evaluated data was based on feedback from two selected mines. (Bajic, Muller, & Gido, 2020) The locations showed the need to adjust applied filters in order to reduce control room operator alarm fatigue. In this paper the aim was to show how a large inadequate filter, should it have existed at the mine at the time when the spontaneous combustion started, would cause the mine to have missed the spontaneous combustion event and thus reacted late.

In previously provided examples (Bajic, Muller, & Gido, 2020), if higher filters were applied (0.2 % and above) to a particular location, there would be no invalid triggers in a selected period, while a lower than 0.2% filter would retain invalid triggers. The mine investigated this instance and confirmed that the GR alarm values were invalid triggers, and there were no corresponding significant increases in CO or CO make. Furthermore, in another period, over 30 GR alarms were noted. The 0.05 % filter included these values and the mine investigated the situation. In this case there was an increase in CO and CO make, confirming valid triggers in GR ratios. If higher filters were applied in this case there would have been the possibility of valid data being lost. Applying a filter at 0.1 % would still remove 100 % of invalid data, and retain 89.20 % of valid triggers, while higher filters, 0.2 % and above, would
retain 100 % of valid triggers from this period (Figure 3). Reduction of suspected triggers “investigate” is optimised with a 0.1 % filter (85.79 %), while a 0.05 % filter only reduces 54.64 % of suspected “investigate” data points.

To provide insight into the relationship between oxygen efficiency, CO and GR, Figure 4 displays two possibilities:

- Constant CO value at 8 ppm for the entire period
- Increasing CO values from 5 ppm to 60 ppm
Figure 3: Case study, Grahams ratio (applied oxygen deficiency filter (%)) 0.05 (a), 0.1 (b), 0.2 (c), 0.25 (d) and 0.3 (e), with CO (%) (Bajic, Muller, & Gido, 2020)

Figure 4: Oxygen deficiency and GR relationship, with 8ppm CO and increasing CO values

It is clear that the relationship between GR and oxygen deficiency is exponential below 0.05 oxygen deficiency in both scenarios. Values between 0.1 and 0.3 oxygen deficiency are more conservative and will retain more data points. The relationship appears to be more linear until approximately the 0.3
oxygen deficiency point. This data indicates that the trend and filtering depends on location and situation, and should not be considered in isolation. CO and CO make values need to be considered as well. Furthermore, Figure 5 presents tube bundle data from one mine site where GR was valid and increased with increased CO. Figure 6 shows data where GR values were increased while CO remained low and stable indicating invalid GR.

When Moura No.2 data was processed, it was evident that majority of oxygen deficiency values were lower than 0.05% (Figure 7). If the mine had an oxygen deficiency of 0.3%, as it is commonly applied in mines today, the mine would not be able to see an increased valid GR (Figure 8).

In this case it is not appropriate to consider data in isolation of CO and filtered with a large (0.3%) filter. Although CO values were not high (5-15ppm) in the days leading to the explosion (Figure 9) there is an increasing trend of CO indicating oxidation.

Figure 10 presents oxygen deficiency (before and after the explosion), GR and CO (ppm). Oxygen deficiency and CO are on primary axis, while GR is on secondary axis. It is evident that data trended like this, non-filtered, would present valid triggers.
Figure 7: Moura No.2 512 top return data, before and after explosion (all data and increased oxygen deficiency resolution)

Figure 8: Moura No.2 512 top return data, GR before and after explosion
CONCLUSIONS AND RECOMMENDATIONS

The data presented in this paper clearly shows that a minimal oxygen deficiency of 0.3% leads to an indiscriminate loss of potentially valid data for atmospheres close to air. As previously stated, the optimal oxygen deficiency filter is most likely dependant on mine deposit, individual database and measurement technique (instruments). Further, it is possible that different locations may require different filter points. Data should not be considered in isolation from other indicators, such as CO and CO make. Additional filters for CO and CO make could be considered in addition to an oxygen deficiency filter for the GR ratio trigger alarm. These parameters could also be included in TARPs together with Graham’s ratio. Further testing and investigation is required for optimal alarm threshold points. Based on the presented data set, a threshold value of 0.30 % did not appear optimal.

ACKNOWLEDGMENTS

Simtars would like to acknowledge mines who agreed to provide data for this research. All data remain confidential and are used for research purposes only.

REFERENCES

Brady, D., 2007. The Influence Analytical Techniques and Uncertainties in Measurement Have on the Assessment of Underground Coal Mine Atmospheres, s.l.: QMIHSC.


DYNAMIC MODEL OF FAULT SLIP AND ITS EFFECT ON COAL BURSTS IN DEEP MINES

Jan Nemcik¹, Gaetano Venticinque², Zhenjun Shan¹ and Libin Gong³

ABSTRACT: The success of deep mining operations relies upon controlling the fractured ground. It is a documented knowledge that many coal bursts occur when mining close to the existing faults. Gradual stress relief towards excavations and other mechanisms can unload stress normal to the nearby fault plane causing it to slip. The generated seismic waves impact the mine roadway rib sides and can produce a coal burst. As part of the ACARP project, the FLAC³D dynamic numerical model was used to show how a fault slip at various locations and orientations may initiate a coal burst. This study simulates an artificial fault slip with peak velocity reaching 4m/s in 0.013 seconds and displacing 119mm in total. Seismic induced peak particle velocities in rock and its influence on coal rib stability were investigated. 89 numerical models with various fault locations and orientations at 450m depth indicated that a 4 tonne coal block can be ejected from the mine roadway rib side at speeds of up to 5m/s. The important finding is that irrespective of the fault slip magnitude, the fault geometry and the in-situ stresses enable to predict which side of the mine roadway may experience the coal burst. Instructing the mine personnel to use the other side of the roadway may improve their safety. Overall, this research produced preliminary results to prove that this method can be used to flag the coal burst dangers for certain fault locations and orientations in deeper mines irrespective of the fault slip properties that are typically difficult to predict.

INTRODUCTION

Historical data indicate that in deep mines presence of faults in close proximity to excavations affect the frequency of coal bursts. An extensive work by many researchers has attempted to correlate the fault geometry and its orientation on severity of coal bursts. For example, White and Whyatt (1999a,b) investigated the mechanism of rockbursts that took place at the Lucky Friday Mine in the USA reporting that slip movements of a steeply dipping fault towards a nearby stope resulted in an increased compressive stress near the stope, which contributed to the occurrence of rockbursts experienced. Blake and Hedley (2003), Ortlepp, (2000) and others studied peak particle velocities within rock induced by seismic waves, causing severe ground motions and damage to the rock mass. To date, no comprehensive dynamic modelling has been attempted to show the direct mechanism of how the fault slip affects both coal rib ejection magnitude and location in the mine roadway. The preliminary dynamic models of Nemcik and Venticinque (2019) attempted to show how fault slips at various orientations may affect the coal bursts.

3-DIMENSIONAL DYNAMIC ANALYSIS OF FAULT SLIPS AND THEIR INFLUENCE ON COAL RIB STABILITY

Numerous models with various fault locations and orientations were constructed adjacent to the coal mine roadway. An identical fault slip of a fixed magnitude was modelled for these geometries to enable comparison between rib ejection magnitudes and directions. To evaluate the influence of fault slip on coal rib stability, an average ejection energy at the rib side was calculated for 1m length of roadway from the kinetic momentum of an ejected 4 tonne coal block attached to the rib side. This study was aimed at producing preliminary results to prove that this method can be used to flag the coal burst dangers for certain fault orientations and excavation localities. To avoid any complications that may arise due to yield zones, elastic models were set up to carry out a number of sensitivity studies. The fault orientation, its distance from the excavation and fault slip direction were studied to estimate the severity of coal burst and side of the roadway that the coal burst may occur.

---

¹ Faculty of Engineering and Information Sciences, University of Wollongong
² SCT Operations Pty LTD, Australia
³ Researcher, Institute of Geonics, the Czech Academy of Sciences
FLAC3D Dynamic Model Setup

The constructed model was 80m wide (perpendicular to the mined roadway), 80m high (in vertical direction) and 40m thick (in excavation direction) as shown in Figure 1. Examining combinations of different fault geometries, the first 44 modelled instances were characterised with a fault plane aligned against the excavation of a 5m wide mine roadway running entirely through the centre of a 3m thick coal seam. A ‘continuous’ 2.9 m high and 1m thick coal block was attached to the rib side in the mine roadway to measure its kinetic momentum due to seismic waves. For the remaining 45 fault models a shorter mine roadway was excavated to the centre of the model and a 1m thick, 2.9m high and 2m long coal block was attached to the rib side adjacent to the roadway face. Elastic properties of the strata were chosen to enable measurements of the maximum possible kinetic energy transfer through the rock or coal seam without complications of the yielded zones. Typical sandstone rock and coal properties were assigned to the roof, floor and the coal seam as specified in Table 1.

![Figure 1 Model geometry showing one of the slipping faults and the mine roadway](image)

<table>
<thead>
<tr>
<th>Table 1 Model Strata Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strata type</td>
</tr>
<tr>
<td>Roof and Floor Sandstone</td>
</tr>
<tr>
<td>Coal Seam</td>
</tr>
</tbody>
</table>

The models were initially run until static equilibrium was achieved. A simple one-way dynamic fault slip was then artificially modelled by assigning variable slip velocities along the fault using the following decay equation:

\[ y(t) = A - \lambda \cdot \text{cos}(2\pi c t + \varphi) \]

Where \( A \) is Wave amplitude (18.4 m/s – not the actual velocity produced), \( \lambda \) is a Constant=10, \( \varphi \) is Phase=90°, \( c \) is a Constant=7 and \( t \) is Time (between 0 to 0.07s)

The equation constants were chosen to produce a maximum slip velocity \( V_p \) of 4 m/s incipient at time \( t=0.013 \) seconds and subsequently decaying to zero at approximately 0.07 seconds producing an overall maximum displacement of 119mm along the dip of the fault. Note that for the sensitivity studies the artificial fault slip was preferred as the ‘modelled slip may have produced variable results due to the model boundaries and other constrains. The \( V_p \) value adopted here from (pulse like rupture) is only an estimate based on previous Earthquake research data (Bizzari, 2012).

Assigned to the wave function in FLAC\(^3\)D dynamic model FISH subroutine, this wave command produced maximum fault slip velocity of 4 m/s 0.013 seconds after the slip began (Figure 2) with seismic waves spreading through the surrounding strata at sonic speeds.

It may be confusing to think of seismic waves as ‘extremely fast compressive or shear fronts’ travelling at several km/s. It seems instead more logical to rather interpret the movement of the rock matrix by considering the ‘rock particles’ themselves and their maximum velocities (as vectors) also known as peak particle velocities (PPV) which occur at some many orders of magnitude slower (ie several m/s) with potential to disturb the unconfined rock/coal at the boundaries. The conservation of kinetic momentum for a seismic wave can also be better understood by imagining the ‘small rock particle
collisions’ within the wave front to follow a similar momentum transfer as the balls in Newton’s famous cradle experiment.

![Fault slip velocity versus time](image)

**Figure 2** Fault slip velocity versus time

**Modelled Fault Slip at Various Fault Orientations and their Influence on Rib Stability in Mine Roadway**

Fault slips along planar faults orientated at various locations and inclinations were trialled over 44 modelled instances to quantify the values of the energy imparted on the coal rib side.

Horizontal faults parallel to the seam were modelled at locations 2m, 5m, 10m, 15m, 20m and 30m below the seam floor which was also repeated for above the seam roof. These faults were subsequently rotated from 0° to dips of 15°, 30°, 45°, and 60° through the fault rotation point which was located below and above the roadway centre depending on fault location. An additional 45 runs of the vertical fault at various distance and bearings ahead of the roadway face were also trialled.

These fault slips were also arranged such that imparting seismic velocities produced by the slip travelled in a direction towards the rib side having the attached 1 m wide coal block. The attached block attempted to emulate the yielded and low confined state of the coal mass typically found in the rib. As the fault slipped, seismic waves carried the kinetic energy (momentum) towards the rib, imparting into the block and ejecting it from the rib side. This mechanism provided a controlled means of measuring impact velocities of the seam particles generated by the seismic waves at the coal rib. Figure 3a and 3b aid to assist with visualisation of the model's geometry and how block ejection velocities were recorded. After dissipation of the seismic waves through the surrounding strata, seismic momentum accumulated within the block remain locked inside propelling it to velocities above 4 m/s.

![FLAC3D 5.00](image)

**Figure 3a** Development of velocities induced by nearby slipping fault (15° dip) 5m above the roadway at time of 0.012 seconds after fault slip began.
Calculations of coal block ejection velocities, momentum and energy were performed for each instance of the 44 modelled faults having various inclinations parallel to the mine roadway and are summarised in Table 2.

For the vertical fault models (those numbered 45 to 89) the mine roadway was excavated half way (i.e. to the centre of the model) and faults inserted in front of the roadway face. A coal block 1m thick 2.9m high and 2m long was attached to the roadway rib side adjacent to the roadway face as shown in Figure 4. After the seismic waves dissipated, the momentum locked inside the block propelled the block at typical velocities of approximately 4m/s.
Table 2 Modelled fault geometry, block ejection velocity and energy impacting on rib caused by fault slip

<table>
<thead>
<tr>
<th>Fault No</th>
<th>Strike ('')</th>
<th>Dip ('')</th>
<th>Fault Distance from seam (m)</th>
<th>Block Ejection average velocity (m/s)</th>
<th>Block Momentum (mv) (kgm/s) x10^3</th>
<th>Energy impacting the rib 0.5mv^2 (Nm) x10^3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Parallel to seam</td>
<td>0°</td>
<td>20m Roof</td>
<td>4.1</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>2</td>
<td>Parallel to seam</td>
<td>0°</td>
<td>15m Roof</td>
<td>4.3</td>
<td>17.5</td>
<td>37.5</td>
</tr>
<tr>
<td>3</td>
<td>Parallel to seam</td>
<td>0°</td>
<td>10m Roof</td>
<td>4.3</td>
<td>17.9</td>
<td>39.3</td>
</tr>
<tr>
<td>4</td>
<td>Parallel to seam</td>
<td>0°</td>
<td>5m Roof</td>
<td>4.3</td>
<td>17.5</td>
<td>37.5</td>
</tr>
<tr>
<td>5</td>
<td>Parallel to seam</td>
<td>0°</td>
<td>2m Roof</td>
<td>3.5</td>
<td>14.2</td>
<td>24.9</td>
</tr>
<tr>
<td>6</td>
<td>Parallel to seam</td>
<td>0°</td>
<td>2m Floor</td>
<td>3.9</td>
<td>15.8</td>
<td>30.9</td>
</tr>
<tr>
<td>7</td>
<td>Parallel to seam</td>
<td>0°</td>
<td>5m Floor</td>
<td>4.5</td>
<td>18.3</td>
<td>41.1</td>
</tr>
<tr>
<td>8</td>
<td>Parallel to mining</td>
<td>0°</td>
<td>10m Floor</td>
<td>4.6</td>
<td>18.7</td>
<td>43.0</td>
</tr>
<tr>
<td>9</td>
<td>Parallel to mining</td>
<td>0°</td>
<td>15m Floor</td>
<td>4.6</td>
<td>18.7</td>
<td>43.0</td>
</tr>
<tr>
<td>10</td>
<td>Parallel to mining</td>
<td>0°</td>
<td>20m Floor</td>
<td>4.5</td>
<td>18.3</td>
<td>41.1</td>
</tr>
<tr>
<td>11</td>
<td>Parallel to mining</td>
<td>15°</td>
<td>20m Roof</td>
<td>4.1</td>
<td>16.6</td>
<td>34.1</td>
</tr>
<tr>
<td>12</td>
<td>Parallel to mining</td>
<td>15°</td>
<td>15m Roof</td>
<td>4.3</td>
<td>17.5</td>
<td>37.5</td>
</tr>
<tr>
<td>13</td>
<td>Parallel to mining</td>
<td>15°</td>
<td>10m Roof</td>
<td>4.3</td>
<td>17.5</td>
<td>37.5</td>
</tr>
<tr>
<td>14</td>
<td>Parallel to mining</td>
<td>15°</td>
<td>5m Roof</td>
<td>4.4</td>
<td>17.9</td>
<td>39.3</td>
</tr>
<tr>
<td>15</td>
<td>Parallel to mining</td>
<td>15°</td>
<td>2m Roof</td>
<td>4.3</td>
<td>17.5</td>
<td>37.5</td>
</tr>
<tr>
<td>16</td>
<td>Parallel to mining</td>
<td>15°</td>
<td>2m Floor</td>
<td>4.3</td>
<td>17.5</td>
<td>37.5</td>
</tr>
<tr>
<td>17</td>
<td>Parallel to mining</td>
<td>15°</td>
<td>5m Floor</td>
<td>4.6</td>
<td>18.7</td>
<td>43.0</td>
</tr>
<tr>
<td>18</td>
<td>Parallel to mining</td>
<td>15°</td>
<td>10m Floor</td>
<td>4.4</td>
<td>17.9</td>
<td>39.3</td>
</tr>
<tr>
<td>19</td>
<td>Parallel to mining</td>
<td>15°</td>
<td>15m Floor</td>
<td>4.7</td>
<td>19.1</td>
<td>44.8</td>
</tr>
<tr>
<td>20</td>
<td>Parallel to mining</td>
<td>15°</td>
<td>20m Floor</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>21</td>
<td>Parallel to mining</td>
<td>30°</td>
<td>20m Roof</td>
<td>4.0</td>
<td>16.2</td>
<td>32.5</td>
</tr>
<tr>
<td>22</td>
<td>Parallel to mining</td>
<td>30°</td>
<td>15m Roof</td>
<td>4.5</td>
<td>18.3</td>
<td>41.1</td>
</tr>
<tr>
<td>23</td>
<td>Parallel to mining</td>
<td>30°</td>
<td>10m Roof</td>
<td>4.4</td>
<td>17.9</td>
<td>39.3</td>
</tr>
<tr>
<td>24</td>
<td>Parallel to mining</td>
<td>30°</td>
<td>5m Roof</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>25</td>
<td>Parallel to mining</td>
<td>30°</td>
<td>2m Roof</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>26</td>
<td>Parallel to mining</td>
<td>30°</td>
<td>2m Floor</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>27</td>
<td>Parallel to mining</td>
<td>30°</td>
<td>5m Floor</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>28</td>
<td>Parallel to mining</td>
<td>30°</td>
<td>10m Floor</td>
<td>4.3</td>
<td>17.5</td>
<td>37.5</td>
</tr>
<tr>
<td>29</td>
<td>Parallel to mining</td>
<td>30°</td>
<td>15m Floor</td>
<td>4.3</td>
<td>17.5</td>
<td>37.5</td>
</tr>
<tr>
<td>30</td>
<td>Parallel to mining</td>
<td>30°</td>
<td>20m Floor</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>31</td>
<td>Parallel to mining</td>
<td>45°</td>
<td>15m Roof</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>32</td>
<td>Parallel to mining</td>
<td>45°</td>
<td>10m Roof</td>
<td>4.4</td>
<td>17.9</td>
<td>39.3</td>
</tr>
<tr>
<td>33</td>
<td>Parallel to mining</td>
<td>45°</td>
<td>5m Roof</td>
<td>4.4</td>
<td>17.9</td>
<td>39.3</td>
</tr>
<tr>
<td>34</td>
<td>Parallel to mining</td>
<td>45°</td>
<td>2m Roof</td>
<td>4.4</td>
<td>17.9</td>
<td>39.3</td>
</tr>
<tr>
<td>35</td>
<td>Parallel to mining</td>
<td>45°</td>
<td>2m Floor</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>36</td>
<td>Parallel to mining</td>
<td>45°</td>
<td>5m Floor</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>37</td>
<td>Parallel to mining</td>
<td>45°</td>
<td>10m Floor</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>38</td>
<td>Parallel to mining</td>
<td>45°</td>
<td>15m Floor</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>39</td>
<td>Parallel to mining</td>
<td>60°</td>
<td>10m Roof</td>
<td>3.7</td>
<td>15.0</td>
<td>27.8</td>
</tr>
<tr>
<td>40</td>
<td>Parallel to mining</td>
<td>60°</td>
<td>5m Roof</td>
<td>3.6</td>
<td>14.6</td>
<td>26.3</td>
</tr>
<tr>
<td>41</td>
<td>Parallel to mining</td>
<td>60°</td>
<td>2m Roof</td>
<td>3.6</td>
<td>14.6</td>
<td>26.3</td>
</tr>
<tr>
<td>42</td>
<td>Parallel to mining</td>
<td>60°</td>
<td>2m Floor</td>
<td>3.0</td>
<td>12.2</td>
<td>18.3</td>
</tr>
<tr>
<td>43</td>
<td>Parallel to mining</td>
<td>60°</td>
<td>5m Floor</td>
<td>3.0</td>
<td>12.2</td>
<td>18.3</td>
</tr>
<tr>
<td>44</td>
<td>Parallel to mining</td>
<td>60°</td>
<td>10m Floor</td>
<td>2.8</td>
<td>11.4</td>
<td>15.9</td>
</tr>
</tbody>
</table>

Results from the additional vertical fault models are summarised in Table 3 at various inclinations parallel to the mine roadway.
Table 3  Modelled fault geometry, block ejection velocities and rib impact energy due to vertical fault slip ahead of the roadway face

<table>
<thead>
<tr>
<th>Fault No</th>
<th>Strike (°) (0° perpendicular to roadway)</th>
<th>Dip (°)</th>
<th>Fault Distance ahead of roadway Face (m)</th>
<th>Block Ejection average velocity (m/s)</th>
<th>Block Momentum (mV) (kgm/s)x10^3</th>
<th>Energy impacting the rib (0.5mv^2 (Nm)x10^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>45</td>
<td>0°</td>
<td>90°</td>
<td>3 m</td>
<td>4.9</td>
<td>19.9</td>
<td>48.7</td>
</tr>
<tr>
<td>46</td>
<td>0°</td>
<td>90°</td>
<td>6 m</td>
<td>4.9</td>
<td>19.9</td>
<td>48.7</td>
</tr>
<tr>
<td>47</td>
<td>0°</td>
<td>90°</td>
<td>9 m</td>
<td>4.9</td>
<td>19.9</td>
<td>48.7</td>
</tr>
<tr>
<td>48</td>
<td>0°</td>
<td>90°</td>
<td>12 m</td>
<td>5.0</td>
<td>20.3</td>
<td>50.8</td>
</tr>
<tr>
<td>49</td>
<td>0°</td>
<td>90°</td>
<td>15 m</td>
<td>5.0</td>
<td>20.3</td>
<td>50.8</td>
</tr>
<tr>
<td>50</td>
<td>15°</td>
<td>90°</td>
<td>3 m</td>
<td>4.9</td>
<td>19.9</td>
<td>48.7</td>
</tr>
<tr>
<td>51</td>
<td>15°</td>
<td>90°</td>
<td>6 m</td>
<td>4.9</td>
<td>19.9</td>
<td>48.7</td>
</tr>
<tr>
<td>52</td>
<td>15°</td>
<td>90°</td>
<td>9 m</td>
<td>4.9</td>
<td>19.9</td>
<td>48.7</td>
</tr>
<tr>
<td>53</td>
<td>15°</td>
<td>90°</td>
<td>12 m</td>
<td>5.0</td>
<td>20.3</td>
<td>50.8</td>
</tr>
<tr>
<td>54</td>
<td>15°</td>
<td>90°</td>
<td>12 m</td>
<td>5.0</td>
<td>20.3</td>
<td>50.8</td>
</tr>
<tr>
<td>55</td>
<td>30°</td>
<td>90°</td>
<td>3 m</td>
<td>4.5</td>
<td>18.3</td>
<td>41.1</td>
</tr>
<tr>
<td>56</td>
<td>30°</td>
<td>90°</td>
<td>6 m</td>
<td>4.5</td>
<td>18.3</td>
<td>41.1</td>
</tr>
<tr>
<td>57</td>
<td>30°</td>
<td>90°</td>
<td>9 m</td>
<td>4.6</td>
<td>18.7</td>
<td>43.0</td>
</tr>
<tr>
<td>58</td>
<td>30°</td>
<td>90°</td>
<td>12 m</td>
<td>4.6</td>
<td>18.7</td>
<td>43.0</td>
</tr>
<tr>
<td>59</td>
<td>30°</td>
<td>90°</td>
<td>15 m</td>
<td>4.4</td>
<td>17.9</td>
<td>39.3</td>
</tr>
<tr>
<td>60</td>
<td>45°</td>
<td>90°</td>
<td>3 m</td>
<td>3.8</td>
<td>15.4</td>
<td>29.3</td>
</tr>
<tr>
<td>61</td>
<td>45°</td>
<td>90°</td>
<td>6 m</td>
<td>3.8</td>
<td>15.4</td>
<td>29.3</td>
</tr>
<tr>
<td>62</td>
<td>45°</td>
<td>90°</td>
<td>9 m</td>
<td>3.8</td>
<td>15.4</td>
<td>29.3</td>
</tr>
<tr>
<td>63</td>
<td>45°</td>
<td>90°</td>
<td>12 m</td>
<td>3.8</td>
<td>15.4</td>
<td>29.3</td>
</tr>
<tr>
<td>64</td>
<td>45°</td>
<td>90°</td>
<td>15 m</td>
<td>3.8</td>
<td>15.4</td>
<td>29.3</td>
</tr>
<tr>
<td>65</td>
<td>60°</td>
<td>90°</td>
<td>3 m</td>
<td>2.7</td>
<td>11.0</td>
<td>14.8</td>
</tr>
<tr>
<td>66</td>
<td>60°</td>
<td>90°</td>
<td>6 m</td>
<td>2.7</td>
<td>11.0</td>
<td>14.8</td>
</tr>
<tr>
<td>67</td>
<td>60°</td>
<td>90°</td>
<td>9 m</td>
<td>2.8</td>
<td>11.4</td>
<td>15.9</td>
</tr>
<tr>
<td>68</td>
<td>60°</td>
<td>90°</td>
<td>12 m</td>
<td>2.8</td>
<td>11.4</td>
<td>15.9</td>
</tr>
<tr>
<td>69</td>
<td>60°</td>
<td>90°</td>
<td>15 m</td>
<td>2.8</td>
<td>11.4</td>
<td>15.9</td>
</tr>
<tr>
<td>70</td>
<td>-15°</td>
<td>90°</td>
<td>3 m</td>
<td>4.5</td>
<td>18.3</td>
<td>41.1</td>
</tr>
<tr>
<td>71</td>
<td>-15°</td>
<td>90°</td>
<td>6 m</td>
<td>4.6</td>
<td>18.7</td>
<td>43.0</td>
</tr>
<tr>
<td>72</td>
<td>-15°</td>
<td>90°</td>
<td>9 m</td>
<td>4.7</td>
<td>19.1</td>
<td>44.8</td>
</tr>
<tr>
<td>73</td>
<td>-15°</td>
<td>90°</td>
<td>12 m</td>
<td>4.6</td>
<td>18.7</td>
<td>43.0</td>
</tr>
<tr>
<td>74</td>
<td>-30°</td>
<td>90°</td>
<td>3 m</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>75</td>
<td>-30°</td>
<td>90°</td>
<td>6 m</td>
<td>4.2</td>
<td>17.1</td>
<td>35.8</td>
</tr>
<tr>
<td>76</td>
<td>-30°</td>
<td>90°</td>
<td>9 m</td>
<td>4.1</td>
<td>16.8</td>
<td>35.0</td>
</tr>
<tr>
<td>77</td>
<td>-30°</td>
<td>90°</td>
<td>9 m</td>
<td>4.0</td>
<td>16.2</td>
<td>32.5</td>
</tr>
<tr>
<td>78</td>
<td>-30°</td>
<td>90°</td>
<td>12 m</td>
<td>3.9</td>
<td>15.8</td>
<td>30.9</td>
</tr>
<tr>
<td>79</td>
<td>-30°</td>
<td>90°</td>
<td>15 m</td>
<td>3.8</td>
<td>15.4</td>
<td>29.3</td>
</tr>
<tr>
<td>80</td>
<td>-45°</td>
<td>90°</td>
<td>3 m</td>
<td>3.2</td>
<td>13.0</td>
<td>20.8</td>
</tr>
<tr>
<td>81</td>
<td>-45°</td>
<td>90°</td>
<td>6 m</td>
<td>3.2</td>
<td>13.0</td>
<td>20.8</td>
</tr>
<tr>
<td>82</td>
<td>-45°</td>
<td>90°</td>
<td>9 m</td>
<td>3.2</td>
<td>13.0</td>
<td>20.8</td>
</tr>
<tr>
<td>83</td>
<td>-45°</td>
<td>90°</td>
<td>12 m</td>
<td>3.2</td>
<td>13.0</td>
<td>20.8</td>
</tr>
<tr>
<td>84</td>
<td>-45°</td>
<td>90°</td>
<td>15 m</td>
<td>3.2</td>
<td>13.0</td>
<td>20.8</td>
</tr>
<tr>
<td>85</td>
<td>-60°</td>
<td>90°</td>
<td>3 m</td>
<td>2.5</td>
<td>10.2</td>
<td>12.7</td>
</tr>
<tr>
<td>86</td>
<td>-60°</td>
<td>90°</td>
<td>6 m</td>
<td>2.5</td>
<td>10.2</td>
<td>12.7</td>
</tr>
<tr>
<td>87</td>
<td>-60°</td>
<td>90°</td>
<td>9 m</td>
<td>2.5</td>
<td>10.2</td>
<td>12.7</td>
</tr>
<tr>
<td>88</td>
<td>-60°</td>
<td>90°</td>
<td>12 m</td>
<td>2.5</td>
<td>10.2</td>
<td>12.7</td>
</tr>
<tr>
<td>89</td>
<td>-60°</td>
<td>90°</td>
<td>15 m</td>
<td>2.5</td>
<td>10.2</td>
<td>12.7</td>
</tr>
</tbody>
</table>

**DISCUSSIONS**

The results summarised in Table 2 and 3 indicate that faults with the same slip characteristics at close proximity to the excavation appear to produce similar block ejection velocities. These velocities seem to be consistent with the maximum fault slip velocity \( V_p \). This is not surprising. When tracing the velocities surrounding the nearby slipping fault, the peak particle velocities that spread through either the rock or softer coal have similar maximum velocities and directions to the slipping fault. This simplifies the understanding of basic seismic wave front propagation close to the slipping faults.

Observations of the coal block ejection indicated that the block parts located closer to the slipping fault experienced dynamic impact sooner causing block rotation and uneven ejection of these blocks.
Furthermore, fault inclination produced inclined impact to the rib, further affecting block ejection trajectories, causing the block to bounce up and down.

When observing a minor refraction of seismic waves at the rock/coal seam interface (due to a slower speed of seismic waves in softer coal), PPV concentrations within the seam were observed. An additional 45 dynamic fault models were performed on seams with various stiffnesses (results not tabulated here) which indicated a magnifying effect to the rib velocities of softer seams up to 19% more depending on the fault location and inclination.

Typically, fault slips generate seismic waves that quickly spread through the surrounding strata and impact the roadway rib side. The fault slip direction however impacts only one side of the roadway where the rib ejection is most likely to occur. If the fault location is known and direction of fault slip evaluated from ground stress, then the location of the possible rib ejection can be predicted.

The ability to predict the location of seismic impact can be very valuable for safety in coal mines. When mining through certain zones of fault influence, mine personnel can be advised to keep to the safer side of the roadway to minimise the possibility of potential harm.

**CONCLUSIONS:**

To study the fault induced seismic waves and its influence on coal bursts in 3-dimensions using FLAC$^3D$, 89 dynamic model instances comprising various fault slip locations and directions were simulated. The general behaviour of fault slip was discussed. When at close proximity to excavation, the fault slip behaves as an infinite slip plane with minimum energy dissipation. The modelled fault slip failure shows that this mechanism can generate a sufficient amount of seismic energy to produce a coal burst on its own. Fault slip can typically occur as the progressive excavations towards them gradually relieve the stress normal to the fault plane. The initial rib impact appears to be approximately proportional to the maximum velocity of fault slip. The fault slips release fast seismic waves with peak particle velocities (PPV) that can exceed several m/s. These models of different fault slip locations and orientations reveal fast ejections of the detached blocks in the roadway.

The results show that the coal burst typically occurs on one side of the roadway only in response to the approaching seismic waves. Once the location of the fault zone and direction of fault slip is estimated, the mine excavation side where probable rib ejection may occur, can be predicted. The models also show that the seismic waves tend to concentrate within the coal seam producing faster coal rib ejections. Overall, this research produced preliminary results to prove that this method can be used to flag the coal burst dangers for certain fault locations and orientations in deeper mines irrespective of the fault slip properties that are typically difficult to predict.

**RECOMMENDATIONS**

Further dynamic modelling of fault slips is desirable to refine the understanding of rib burst mechanisms that may lead to safer mining through faulted strata. These modelled results were conducted to prove the concept only and were not verified with measurements in underground mines as no reliable data were found within the timeframe of this project. It is suggested that further detailed studies be undertaken in future to verify these findings.

**REFERENCES**


BENCHMARKING STUDY BY LABORATORY LOAD TRANSFER TESTING ALONG FULL RESIN ENCAPSULATED ROCK BOLTS

Sabitha Sasi¹ and Peter Craig²

ABSTRACT: The primary ground support used in Australian underground coal mines is a rebar rock bolt anchored to the rock mass using polyester resin capsules. The key objective of this research project was to benchmark different types of Australian rockbolt resin capsules, including relatively new formulations. Laboratory test methods were developed based on previous studies conducted in South Africa and subsequently at the University of Wollongong, which used 1.8 m JX bolts for installation in internally threaded steel pipes that simulated the borehole. The encapsulated bolts were cut into 80 mm long sections for push testing to obtain around 20 data points from each bolt. Different resin formulation variables were tested; these included three types of limestone fillers and two types of catalyst. The load transfer capacity of different resins were determined by evaluating the bond strength, peak strength and area under the push test curve. Significant variation in load transfer capacity was found along the length of all 1.8 m bolts tested. The load transfer capacity and its variations along the bolt length were evaluated to be a characteristic of the type of resin formulation tested. Some resin formulations produced up to 20% higher load transfer capacity with better consistency along the length of the bolt. Results from more than 300 push tests were used to assess and validate this study. The findings from this detailed research project can be used for a better understanding of the ground support performance of different Australian resin capsules.

INTRODUCTION

Ground support systems are a critical aspect in the field of underground mining. There have been continuous research and development in this field to optimise underground roadway development rates and to reduce the risk of collapsing roadways. The primary ground support technology includes a rebar rock bolt anchored in a grouting medium such as a polyester resin.

Development of resin capsules had revolutionized underground mining industry. Resin capsules can provide full-length bolt anchoring with a safer and cost-effective method. Unlike cement grout, resin technology compliments roof support by providing the benefit of chemical bonding within seconds of installation. Various formulations of resin mastic and catalyst were developed to cater to the requirements of different mining conditions. Since the last 10 years, about six different J-Lok primary resin formulations have been developed in Australia. Manufacturers typically use laboratory specimens in different shapes and sizes based on country of origin or acceptable standards, to determine the mechanical properties of resins without reflecting on the effect of bolt rib mixing the resin components or the presence of capsule film (Hillyer et al., 2013). However, there is no Australian laboratory standard for testing the mechanical performance of the resin capsule based on mixing of its component with the installation of ribbed rebar in a drilled hole (Aziz, Nemcik, Craig, & Hawker, 2014).

The amount of load that can be potentially transferred from the strata to the bolt through the resin encapsulation is determined by the mechanical interlocking between the resin, bolt and the strata. This property can only be evaluated post rockbolt installation. Underground short encapsulation pull tests are conducted periodically to verify the mechanical capacity of the rockbolt-resin system. However, this neither provides a standardised comparison between the different resins available in the Australian market nor gives adequate technical information to understand the variations in load transfer capacity of the resin along the full length of the encapsulation.

With the number of variations available in resin formulations today, it is high time to develop a standardised test method to identify the load transfer capacity of resin capsule post installation as well

¹ Mechanical Engineer, Jennmar Australia. Email: ssasi@jennmar.com.au Tel: +61 426 840 171
² National Manager-Coal, Jennmar Australia. Email: pcraig@jennmar.com.au Tel: +61 419 018 998
as its consistency along the full length of the installed rockbolt. Hence, the primary objective of this study is to benchmark the performance of different types of resin capsules available in the Australian market. Based on previous studies conducted in South Africa and subsequently at University of Wollongong, as well as considering the rockbolt installation parameters, Laboratory Short Encapsulation Push Tests (LSEPUT) were used to benchmark the different types of resins available in the Australian market.

ROCKBOLT AND RESIN CAPSULE INSTALLATION

The conventional method of coal mining rock bolt installation is by spinning a rock bolt through a resin capsule inserted into a drilled hole of diameter nearly 30% more than the bolt diameter. Spinning of the rockbolt facilitates the shredding of the resin capsule plastic film as well as promotes the mixing of the catalyst and resin mastic by the bolt ribs. The manufacturers recommended spin time is controlled by spinning through the length of the resin capsule for about 75% of the spin time and spinning the remaining 25% of the spin time with the rockbolt at the back of the drilled hole. The bolt is held stationary for tens of seconds to allow the resin to harden before applying tension, this is classified as the spin and hold method.

The recent developments in the resin capsule technologies have introduced the spin to stall resin formulations. These resin capsules are installed using spin-to-stall methods. This method negates the need for a hold time and reduces the overall installation time to improve roadway development times (Emery, Craig, Sykes, Canbulat, & Naylor, 2015).

Resin capsule comes in a sausage shaped plastic and contains the polyester resin with limestone fillers and a catalyst separated by a thin film of plastic lining. Three main types of limestone fillers in polyester resin mastic along with a combination of two different types of catalysts are used under the J-Lok Australia brand name. The range of limestone fillers include; Grit, Standard and Low Insertion Force (LIF) fillers. The two different types of catalysts used in Australian industry are water-based catalyst and oil based catalyst. Both these catalyst have the same limestone filler but have different concentrations of initiator, which is benzoyl peroxide. In order to get optimum levels of cured resin properties, the catalysts are used in different ratios with the mastic based on its concentration of benzoyl peroxide. Hence, in the case of oil-based catalyst only 7% of the capsule consists of the catalyst whereas for water-based catalyst, 20-30% of the capsule consists of the catalyst.

LABORATORY SHORT ENCAPSULATION PUSH TESTS

The original South African study was conducted on rock bolts of 20 mm diameter, which were installed through resin capsules into internally threaded steel tubes of 27 mm internal diameter and 800mm in length. The bolt-installed steel pipes were then cut to 100mm sections and the bolt was pushed through the resin to test the strength of encapsulation in each of these sections (Altounyan, 2003).

A test method was used based on this study to incorporate the Australian parameters of rockbolt installations. Laboratory Short Encapsulation Push Tests were conducted in four stages as given below:

(a) Stage 1: Rockbolt installation in internally threaded steel tube.

(b) Stage 2: Cutting of encapsulated sections into twenty sections of 80mm each.

(c) Stage 3: Push testing the rockbolt through each of the encapsulated sections.

(d) Stage 4: Result Evaluation and benchmarking of the different resins in the Australian market.

A rockbolt of 21.7 mm core diameter installed in a hole drilled with a wet drill bit of 27 – 28 mm is the Australian industry standard. Hence, 1.7 m long seamless steel pipe with internal threads of 7/8 BSPF and 5mm in thickness was used to simulate Australian underground mine boreholes. One end of the steel tube was welded close (back of the borehole) and the other end of the tube was welded on to a steel plate with a centre hole to insert the bolt. Two holes of 1mm each were drilled about 10mm from the back of the tube on its opposite sides. This provided enough gap for any air to escape but not enough to lose any resin while spinning the bolt to the back of the tube through the resin capsule. In the case of installing low insertion force (LIF) resin capsules, due to its very fine particle size, the two standard holes of 1mm each were drilled at the back of the tube and covered with a cotton cloth. This provided just enough gap for air leak but not enough for resin loss.
In order to limit the test variables to only different formulations of resin mastic and catalyst, a standard rockbolt and capsule were used for installation in the internally threaded steel pipe. The M24 JX bolt with a length of 1.8 m and core diameter of 21.7 mm was used as the standard rockbolt for installation along with different resin formulations in capsules of 24 mm diameter and 1200 mm length. The resin capsules were installed at a recommended temperature of 20 - 25 degree Celsius. All the resin capsules used were made in 50:50 dual set speed ratios.

**Installation of JX rock bolts**

The rockbolt-resin system was installed in the threaded steel tube using a Joy HFX surface drill rig at J-Lok resins testing facility. The HFX, HDR drill rig had a rotary motor with a speed range of 500 – 600 rpm. The installation method and spin time were determined as per the recommendations for the type of resin installed. Two sets of tests were conducted for each type of installation parameters.

**Preparation of fully encapsulated push test sections**

Following the bolt-resin installation, the steel pipe was cut into twenty sections of 80mm each for push test. The 80mm length ensured that each section would encapsulate three bolt ribs and hence give a standardised comparison between all the push test results. All sections were examined for full encapsulation, gloving and other factors that could affect its load transfer capacity. Before push test, these sections were machined on both ends to remove any sharp edges or burrs and to ensure that it is seated perpendicularly for push tests.

**Push Tests**

In the interest of getting consistent and quality results, only fully encapsulated sections were push tested and its results were evaluated to study the load transfer capacity for the different formulations of resins tested. All sections were examined before and after push tests for any signs of gloving, air bubbles or uncured resin encapsulation.

A steel plate with a push test spigot of 18 mm diameter at its centre and a cylindrical base plate with a seating for the steel tube and hole for the bolt displacement were custom manufactured for the push tests. In the interest of analysing the failure between the bolt and resin interface, the diameter of the push test spigot was kept smaller than the bolt core diameter of 21 mm.

The push test setup was assembled as seen in the Figure 1. The push test spigot was placed over the centre of the encapsulated bolt and then loaded at a rate of 1 mm/min up to 10 mm of displacement. An inbuilt data logging software was used to record the ‘load vs displacement’ data points for every 0.05 seconds from the push tests. The data points were extracted into excel and compiled to evaluate the load transfer capacity and its variations along the length of the bolts.

![Figure 1: Push test setup](image)

**Result evaluation**

Load vs displacement curves were generated for all the tested sections. Based on previous studies and traditional methods of result analysis different methods were considered to analyse these push test results and hence benchmark the load transfer capacity of the different types of resins. These included the average bond strength, average peak strength, average work done, and Load Transfer Index (LTI). These results were also used to evaluate the variations in load transfer capacity along the length of the bolt and its effect on the entire system performance.
Fourteen different tests were conducted and used for the purpose of this benchmarking study. These included six different variations in resin and catalyst formulations. The resin formulations were identified as per the manufacturers naming conventions, which are given in Table 1 as Resin Id.

Table 1: Resin formulations and installation methods used in the benchmarking study

<table>
<thead>
<tr>
<th>S.No</th>
<th>Test Id</th>
<th>Resin Id</th>
<th>Resin Formulations</th>
<th>Spin To Back (Secs)</th>
<th>Spin At Back (Secs)</th>
<th>Hold Time (secs)</th>
<th>Set Speed (50:50)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Test4</td>
<td>JLD</td>
<td>Standard Oil based</td>
<td>12</td>
<td>13</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>Test8</td>
<td>JLD</td>
<td>Standard Oil based</td>
<td>12</td>
<td>13</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>Test3</td>
<td>JLD5D</td>
<td>Standard Water based</td>
<td>12</td>
<td>13</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>Test7</td>
<td>JLD5D</td>
<td>Standard Water based</td>
<td>12</td>
<td>15</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>5</td>
<td>Test5</td>
<td>JG5D</td>
<td>Grit Water based</td>
<td>12</td>
<td>13</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>Test9</td>
<td>JG5D</td>
<td>Grit Water based</td>
<td>12</td>
<td>14</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>7</td>
<td>Test2</td>
<td>JGD</td>
<td>Grit Oil based</td>
<td>12</td>
<td>13</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>8</td>
<td>Test13</td>
<td>JGD</td>
<td>Grit Oil based</td>
<td>12</td>
<td>13</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>9</td>
<td>Test11</td>
<td>STSA</td>
<td>Grit Oil based</td>
<td>12</td>
<td>15</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>10</td>
<td>Test12</td>
<td>STSA</td>
<td>Grit Oil based</td>
<td>12</td>
<td>16</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>11</td>
<td>Test14</td>
<td>JLLD</td>
<td>LIF Oil based</td>
<td>12</td>
<td>13</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>12</td>
<td>Test15</td>
<td>JLLD</td>
<td>LIF Oil based</td>
<td>12</td>
<td>13</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>13</td>
<td>Test16</td>
<td>JLL5D</td>
<td>LIF Water based</td>
<td>12</td>
<td>14</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>14</td>
<td>Test17</td>
<td>JLL5D</td>
<td>LIF Water based</td>
<td>12</td>
<td>13</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

TEST METHOD VALIDATION

To test the test method, resin liquid was pumped through a static mixer to fill a threaded pipe and bolt pushed into the mixture. This installation eliminated effects of the plastic film and rebar mixing. As shown in Figure 2, a JX bolt with static mixer gave relatively consistent results along the length of the pipe compared to a bolt mixing through a resin capsule. This test proved the selection of 80mm long sectioning was appropriate for the JX bolt rib spacing.

Figure 2: Resin mixed through a static mixer and without resin capsule plastic (left); Resin mixed by ribbed rebar rotated through the resin capsule (right)
RESULTS AND DISCUSSIONS

Results from about 300 push tests were compiled to evaluate and benchmark the different types of J-Lok resins manufactured in the Australian market. These results were also used to study the effects of ribbed rebar mixing the contents of the different resin capsule designs by evaluating the variations in load transfer capacity along the length of an encapsulated bolt.

As seen in Figure 3, six main variations in the type of load vs displacement curves were derived from the push test data and these were quite inconsistent throughout the length of the bolt.

Bond strength taken when the gradient reaches 20kN/mm is a standard parameter generally used in previous papers to study the load transfer capacity of resins (Hillyer, The Influence of Installation Method and Resin Properties on Rock Bolt Performance in Underground Coal Mines, 2012). This is a key parameter specified by the British standards for determining the load transfer capacity from a short encapsulation pull test (British Standards Institution, 2007). However, as seen from the graphs in Figure 3, the bond strength was sensitive to the type of push test graph generated and hence its variation along the bolt length was quite significant and inconsistent. These trends variation of bond strength along the bolt length was identified by Hillyer from his studies as well. Because of the many different load-displacement curves, the selection of a single data point such as bond strength from each curve was thought not to represent the overall performance of each section. Thus, additional methods were investigated to benchmark the load transfer capacity of different resins.

Unlike bond strength, parameters that were independent of the rate of change of load transfer capacity were recorded to benchmark the mechanical performance of the different resins. Hence, the peak strength along with the area under the curve that represents the work done was evaluated. Average peak strength predicted the maximum capacity of a resin while average work done summed the optimum load capacity of the resin along with its residual strength. The average work done data was evaluated to be in agreement with the average peak load and hence it verified the validity of the benchmarking study. Table 2 and Figure 4 benchmarks the load transfer capacity for different resin types available in the Australian market.
It can be analysed from the Figure 4 that, independent of the catalyst, the grit based resin mastic resulted in the highest strength capacity whereas standard fillers resulted in the lowest strength capacity. Additionally, it was noted that oil based catalyst resulted in 30% to 40% more load transfer capacity compared to water based catalyst in case of the same resin mastic formulations.

In addition to evaluating the average load transfer capacity of the different types of resins, its variations along the length of the bolt were also evaluated to understand the effects of bolt rib mixing the resin capsule components. This is depicted in Figure 5 and Figure 6, as variations in peak strength, bond strength and work done for the different resins. It was verified that the variations along the length of the bolt was primarily an indicative of the type of resin formulations tested.

Table 2: Comparison of load transfer parameters for different types of resins

<table>
<thead>
<tr>
<th>Resin Capsule ID</th>
<th>Average Peak Strength (kN)</th>
<th>Average Bond Strength (kN)</th>
<th>Average Work Done (J)</th>
</tr>
</thead>
<tbody>
<tr>
<td>JL5D</td>
<td>90</td>
<td>46</td>
<td>677</td>
</tr>
<tr>
<td>JLL5D</td>
<td>99</td>
<td>56</td>
<td>747</td>
</tr>
<tr>
<td>JG5D</td>
<td>114</td>
<td>52</td>
<td>863</td>
</tr>
<tr>
<td>JLD</td>
<td>124</td>
<td>56</td>
<td>809</td>
</tr>
<tr>
<td>JLLD</td>
<td>125</td>
<td>58</td>
<td>910</td>
</tr>
<tr>
<td>STSA</td>
<td>149</td>
<td>66</td>
<td>1070</td>
</tr>
<tr>
<td>JGD</td>
<td>165</td>
<td>83</td>
<td>1143</td>
</tr>
</tbody>
</table>

Since bond strength identifies the point along the push test where the loading rate drops below 20kN/mm, it had been seen from the push tests graphs in Figure 3 that the type of load-displacement curve directly affects the bond strength value of each push test. The variations in the push test curves and hence the bond strength is analysed to be the result of the inconsistencies in the mixing of the resin mastic with the catalyst by spinning the bolt through the resin capsule. This had been verified from the test method validation method were a resin and catalyst were mixed through a Static Mixer.
with No Plastic Film (SMNP). As seen in Figure 5, this has resulted in minimal variations of the bond strength.

Figure 5: Box plot on bond strength distribution of different J-Lok resins

Figure 6: Box plots on peak strength and work done distribution of different J-Lok resins

From Figure 5 and Figure 6, it was noted that the average bond strength ranking of the different types of resins were mostly in alignment with the peak strength and work done ranking. Due to significant variations in bond strength, the average bond strength of a resin encapsulation was insufficient to benchmark the different types of resins. However, peak strength and work done resulted in minimal variations in its results, which gave much accurate data for benchmarking.
The variations in bond strength were quite significant irrespective of the type of resin formulation whereas the variations in peak strength and work done were more dependent on the type of resin formulation tested. To verify the correlation between the resin formulation and consistency in mixing along the bolt length, the Load Transfer Index (LTI) of each of the push test sections were evaluated based on previous studies (Thomas, 2012). LTI provides the ratio of peak load by displacement at that peak load. The data points from each tests were compiled on a scatter plot to evaluate the range of variations in LTI along the length of the bolt. Each resin formulation had a signature scatter plot of LTI data points. Higher peak strength at lower displacement gives an optimal or ideal resin performance.

![Figure 7: LTI Scatter plots - Oil based resins (left); water based resins (right)](image)

It was verified from the LTI scatter plots and the box plots that the variations in load transfer capacity and its mean value were dependent on the type of resin capsule formulation that was installed. Primarily it was noted that though the water based catalyst had lower average strength capacity compared to oil based catalyst, it had better consistency in its load transfer variations along the bolt length. Though oil based catalyst resulted in higher variations in its load transfer capacities, grit fillers in oil-based catalyst promoted better consistency in its results.

From the above sets of results, it was analysed that grit fillers promoted better mixing of the catalyst and resin mastic. This resulted in better consistency in its load transfer capacity along the bolt length, independent of the catalyst type. Whereas standard and LIF fillers consisted of finer limestone particle size and resulted in less consistent mixing of resin mastic with the catalyst, hence the consistency in its load transfer capacity was more dependent on the type of catalyst. The smaller sized fillers induced gloving towards the top 200 mm of encapsulation in the case of standard and LIF fillers, which resulted in reduced load transfer capacities of these sections. The effects of different catalysts were also evaluated from the above results. Though oil based catalyst resulted in higher load transfer capacities, the larger catalyst to mastic ratio of a water-based catalyst resulted in more consistency along the bolt length, irrespective of the type of resin filler.

**CONCLUSIONS**

The J-Lok benchmarking project has developed a test method based on previous studies and research to benchmark the mechanical performance of all the J-Lok resin formulations available in Australia. The results from this project has taken into account the effects of rockbolt mixing through the resin capsule on the load transfer capacity of different resin formulations and hence it gives a more realistic benchmarking study compared to traditional laboratory mixed resin strength tests. The bond strength of a resin encapsulation was primarily dependent on the rate of load transfer with displacement. This was evaluated to be significantly inconsistent along the bolt length in the case of a bolt rib mixing the resin capsule components. This is validated by the consistent bond strength obtained by a static mixer and no plastic film.
Different types of resins have different load transfer capacities, which were primarily influenced by the resin capsule components (limestone fillers and catalyst). Oil based catalyst had higher load transfer capacity compared to water based catalyst. In addition, independent of the catalyst, grit fillers resulted in the highest resin performance whereas standard fillers resulted in lower resin performance.

In the case of bond strength, variations along the bolt length were independent of the resin type. Whereas in the case of peak load, work done and LTI, the variations of the load transfer capacity along the bolt length was dependent on the type of resin tested. It was analysed that higher catalyst to mastic ratio volumes in the water-based catalyst resulted in better consistency of load transfer capacity compared to significant variations in most resins with oil-based catalyst. Additionally, the coarser particles of grit fillers were observed to promote better mixing and shredding of the resin capsule. This had resulted in more consistent load transfer capacity along the bolt length even in the case of an oil-based catalyst. Hence, grit based resin mastic with an oil-based catalyst, having higher catalyst to mastic ratio is expected to result in optimal resin performance. Further tests and trials on similar formulations are required to verify the optimal resin formulations.

REFERENCES


British Standards Institution, 2007. BS7861-1: Strata reinforcement support system components used in coal mines, pp. 36 - 42.

Emery, J, Canbulat, I, Craig, P, Naylor, J, Sykes, A, 2015. Development and implementation of the “spin to stall” resin bolting system at Anglo Americans Australian underground coal operations, In Proceedings of the 34th International Conference on Ground Control in Mining, WV.


EFFECT OF PRETENSION ON THE MECHANICAL BEHAVIOUR OF BOLTED ROCK

Mahdi Saadat¹ and Abbas Taheri²

ABSTRACT: A stepwise pull-and-shear test (SPST) scheme that numerically analyses the mechanical behaviour of bolted rock joints subjected to simultaneous pull-shear loading. The SPST scheme allows us to identify the optimum pretension stress magnitude at which the bolting system exhibits its ultimate shear capacity. The micro-mechanical properties of grout and bolt-grout interface were calibrated against the laboratory data. The micro-mechanical parameters of rock were calibrated against the laboratory data of coal and shale, and the micro-mechanical properties of rock joint interface were identified by reproducing the laboratory behaviour of coal-shale interface under the direct shear test. Then, the SPST scheme was employed to study the effect of pretension stress magnitude on the macroscopic behaviour of bolted coal. The numerical results revealed that at yield pretension stress magnitude (pull-out test) the rock bolting system could exhibit its ultimate shear performance.

INTRODUCTION

The natural discontinuities form unstable rock blocks, which control the safety and stability of underground coal mine structures. Any damage due to roof fall can hinder coal production and results in severe penalties being imposed on coal production companies. Therefore, reinforcement of unstable blocks is essential in providing a safe environment for mine personnel and promote sustainable coal production.

One of the most widely used reinforcement elements in coal mining is fully grouted rock bolts which are cost-effective and easy to install due to advancements in the bolting technology (He et al. 2018; Jin-feng and Peng-hao 2019). Fully grouted rock bolts form a self-supporting reinforcement system in rock mass through reinforcing unstable rock blocks and improve the shear resistance of bolted rock joints (Ma et al. 2017). The bolt-grout interface contributes to controlling the load transfer capacity of fully grouted rock bolts and the mechanical interlocking between the grout material, and rock bolt ribs enhance the axial strength of the bolting system (Li et al. 2019). The load transfer mechanism of fully grouted rock bolts can be identified using pull-out testing (Jin-feng and Peng-hao 2019). However, the field observations revealed that the failure of bolted rock joints occurs due to combined pull-out and shear forces (Li 2010). Therefore, understanding the failure mechanism and shear performance of fully grouted rock bolts under such mixed loading conditions is crucial for designing safe and stable support system in underground coal mining. Figure 1 illustrates the behaviour of a bolted rock joint in an underground coal mine subjected to combined pull-shear loads. Both pull-out and shear forces contribute to the failure of rock bolts. Figure 2 shows a failed rock bolt which was possibly broken due to combined pull-shear loads (Li 2010).

(Saadat and Taheri 2019b) proposed a SPST scheme to analyse the mechanical behaviour of fully grouted rock bolts subjected to combined pull-shear loads. The SPST scheme employs a discrete element method (DEM) framework augmented with a new cohesive contact model (Saadat and Taheri 2019c). The proposed SPST scheme is able to determine the mechanism involved in enhancing the shear strength of bolted rock joints and identify the pretension stress magnitude at which the rock bolting system exhibits its ultimate mechanical performance. This paper examines the application of the proposed SPST scheme in determining the optimum pretension stress magnitude at which the maximum shear resistance of a bolted coal specimen can be achieved.

¹ School of Civil, Environmental, and Mining Engineering, The University of Adelaide. Email: mahdi.saadat@adelaide.edu.au
² School of Civil, Environmental, and Mining Engineering, The University of Adelaide. Email: abbas.taheri@adelaide.edu.au
Figure 1: The reinforcement of coal layers in underground coal mining (longwall mining): (a) cross-section of the longwall mine at gob-side entry (modified from Zhu et al. (2018)), (b) mechanical behaviour of bolted rock joint subjected to combined pull-shear load (modified from Indraratna and Haque (2000)).

Figure 2: A failed rebar bolt subjected to both pull and shear loads (Li 2010)

COHESIVE CONTACT MODEL IN DEM

A DEM-based cohesive contact model that can be used for simulating geomaterials (e.g. rock, soil, and grout) and the interface between two materials (e.g. bolt-grout interface) (Saadat and Taheri 2019b). proposed model to facilitate the calibration procedure of the micromechanical parameters. In addition, a simplified contact model allows us to reduce the computational time which leads to performing of faster simulations. The details of model formulation and constitutive relationships can be found in the previous research (Saadat and Taheri 2019a, 2019c). Figure 3 illustrates the mechanical behaviour of cohesive DEM contacts in mode I (tension) and mode (II). The DEM model is called cohesive contact model (CCM) when it is used as a material model and the micro-properties of the model contained a subscript of CCM (e.g. C0CCM). When the model is employed as an interface model, which is called it cohesive smooth-joint model (CSJM) and the micro-properties of the model contained a subscript of CSJM (e.g. C0CSJM). it is clear that under the tensile model, the contact exhibits an elastic stage before reaching its maximum strength, which is shown in the graph by the value of initial cohesion (C0) divided by friction ratio (μ) of the DEM contact. Then, the contact experiences a gradual softening stage that is illustrated in Figure 3 as an exponential decay function. Similarly, under shear loading, the contact experience an initial elastic stage, a peak (C0), and finally a gradual softening stage. Notice that the maximum strength of the DEM contact during a shear failure is represented by the initial cohesion. Smooth-joint model (SJM) was used to reproduce the mechanical behaviour of rock joint. The details of SJM constitutive relationships can be found in Bahaaddini et al. (2013).
Figure 3: The behaviour of DEM-based cohesive contact model (Saadat and Taheri 2019c) in (a) tension and (b) shear.

MODEL CALIBRATION

The micro-properties of the proposed cohesive model have to be calibrated before employing the model for further parametric study of the bolted rock joints. In DEM analysis, calibration of micro-properties is the essential procedure since a unique set of DEM micro-properties can be regarded as a synthetic rock specimen representing a physical counterpart. The typical methodology for calibrating the micro-properties in PFC-DEM approach is to use the experimental observations of uniaxial compressive strength test and reproduce a similar macroscopic response by altering the micro-properties of the contact model (Bahaaddini et al. 2013; Saadat and Taheri 2019b). The experimental results obtained by Li et al. (2015) on coal and shale were used in the present work. Two different numerical specimens were generated with the dimension of 100 (mm)*50 (mm) and loaded uniaxially to reproduce the macroscopic response of the physical specimens. The details of the calibration procedure of the uniaxial compressive strength test can be found in the previous research of (Saadat and Taheri 2019b). Table 1 shows the micro-mechanical properties of the proposed model after completing the calibration procedure for both coal and shale. The macroscopic response of the numerical simulations, as well as their physical counterparts, are given in Table 2. You can see from Table 2 that the mechanical, macroscopic properties of the synthetic specimens are very close to the experimental specimens, thus the micro-properties in Table 1 represent the synthetic coal and shale specimens and can be used for further analysis of bolted rock joints.

Table 1: The micro-properties of the numerical coal and shale specimens

<table>
<thead>
<tr>
<th>Micro-property</th>
<th>Symbol</th>
<th>Rock type</th>
<th>Coal</th>
<th>Shale</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus (GPa)</td>
<td>E&lt;sub&gt;CCM&lt;/sub&gt;</td>
<td>1.2</td>
<td>4.68</td>
<td></td>
</tr>
<tr>
<td>Normal to shear stiffness ratio</td>
<td>k&lt;sub&gt;CCM&lt;/sub&gt;</td>
<td>1.25</td>
<td>1.55</td>
<td></td>
</tr>
<tr>
<td>Initial cohesion (MPa)</td>
<td>C&lt;sub&gt;0,CCM&lt;/sub&gt;</td>
<td>2.2</td>
<td>32.5</td>
<td></td>
</tr>
<tr>
<td>Friction coefficient</td>
<td>μ&lt;sub&gt;CCM&lt;/sub&gt;</td>
<td>0.58</td>
<td>0.55</td>
<td></td>
</tr>
<tr>
<td>Dilation coefficient</td>
<td>β&lt;sub&gt;CCM&lt;/sub&gt;</td>
<td>0.25</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>Softening parameter (1/m)</td>
<td>κ&lt;sub&gt;CCM&lt;/sub&gt;</td>
<td>5.2e6</td>
<td>18.5e6</td>
<td></td>
</tr>
</tbody>
</table>

Table 2: The comparison between macroscopic properties of numerical specimens and experimental counterparts

<table>
<thead>
<tr>
<th>Macroscopic property</th>
<th>Numerical values</th>
<th>Experimental values (Li et al. 2015)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus (GPa)</td>
<td>1.1</td>
<td>1.08</td>
</tr>
<tr>
<td>Uniaxial compressive strength (MPa)</td>
<td>6.5</td>
<td>6.38</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.28</td>
<td>0.32</td>
</tr>
</tbody>
</table>

Notice that the calibration of the coal-shale interface characteristics was needed before performing a direct shear test on the bolted specimen. This is because a set of interface micro-properties are required to accurately mimic the shear behaviour of the coal-shale interface as well as its asperity damage. Therefore, the direct shear test results of coal-shale interface obtained by Li et al. (2015) were used in order to identify the micro-properties of the DEM interface that represents the rock joint in numerical simulations. The details of the direct shear test setup in PFC-DEM can be found in the
previous rock bolt investigation (Saadat and Taheri 2019b). The direct shear test was conducted under constant normal load (CNL) condition with a normal stress magnitude of 2 MPa. The smooth-joint model was assigned on the rock joint contacts that represent coal-shale interface, without infill material, in the physical specimen. The micro-properties of the SJM are normal, and shear stiffness and friction ratio (a micro-properties which controls the friction between DEM particles) which are identified as 150 GPa, 65 GPa, and 0.85, respectively. The complete shear stress-shear displacement of the numerical simulation and its experimental counterpart is illustrated in Figure 4. that the calibrated model successfully reproduced the macroscopic shear response of the coal-shale interface that is presented by close agreement between shear stress-displacement curves of numerical and experimental specimens (Figure 4a).

Figure 4b illustrates the asperity damage of the numerical model at different stages of shear loading. During elastic stage (point a) very few micro-cracks appeared in the specimen which is due to early bond-break in critical asperities. At peak shear stress (point b), the accumulation of micro-cracks is observed in the numerical specimen which is due to progressive bond failure of the cohesive contacts. In DEM simulations, the coalesce of micro-cracks form macroscopic fractures which demonstrate the damage propagation in the specimen. After reaching the peak shear stress, the numerical model exhibited a gradual softening response which is due to the incorporation of the softening parameter in the constitutive relationships of the contact model. This allows the contacts to release certain fracture energy which is not achievable in conventional contact models such as parallel bond model (PBM) or flat-joint model (FJM). After softening response, the specimen exhibits a residual behaviour which is the result of significant asperity degradation (points c and d).

Figure 4: The results of numerical and experimental direct shear tests: (a) shear stress-displacement curves, (b) asperity damage at different stages of shearing in the numerical specimen. Experimental results are from Li et al. (2015).

The mechanical properties of the grout material and bolt-grout interface are calibrated against the laboratory data obtained by Shang et al. (2018) and the details of the calibration procedure, as well as the values of micro-properties of CCM and CSJM, are given in Saadat and Taheri (2019b).

**NUMERICAL ANALYSIS OF BOLTED COAL**

Figure 5 illustrates the schematic view of the proposed SPST scheme (Saadat and Taheri 2019b) that is followed in this study to conduct combined pull-shear load test on a fully grouted rock bolt. Three major elements are involved in the SPST test setup: rock joint, rock bolt, and grout. The proposed SPST scheme involves two main steps:

1. Performing a pull-out test on the fully grouted rock bolt and achieving the axial stress-strain curve and damage response of grout material as well bolt-grout interface. At this step, the mechanical behaviour of fully grouted rock bolt is monitored stored at seven different points (i.e. points m, n, o, p, q, r, s in Figure 6) in various loading stages of axial stress-strain curve including linear elastic, pre-
hardening, peak, gradual softening, and residual stages. These seven points represent different pretension stress magnitudes during pull-out testing of the fully grouted rock bolt.

(2) Restoring the numerical files saved in the previous stage in order to conduct direct shear tests. At this stage, seven different direct shear tests are conducted on the bolted rock joints each of which has a different pretension stress magnitude. The macroscopic shear stress-displacement of the bolted rock joints is monitored during the shearing procedure, and the maximum value of peak shear strength is identified. This presents the point at which the fully grouted rock bolt exhibits its ultimate shear performance under combined pull-and-shear loading.

Figure 5: DEM test setup for conducting combined pull-shear load experiment based on SPST scheme.

Figure 6 illustrates the results of the pull-out test. Figure 6a shows the axial stress-displacement response of the fully grouted rock bolt. Figure 6b illustrates the magnitude of normal stress induced on the rock joint interface during the pull-out test. You can see that the stress-displacement curve consists of four different stages (I-IV). The first stage is a linear elastic stage (I) at which the bolting system perform elastically. The stress response of the system at two different points (points “m” and “n”) was monitored, which are equivalent to the pretension stress magnitude at the elastic stage. The stress-displacement curve then exhibits a non-linear response from point “o” to “p” resulting in a reduction in the axial stiffness of the bolting system (stage II). You can see that the rate of increase in the normal stress magnitude showed a significant drop during this stage. From point “p” to “r” the fully grouted rock bolt reproduced a gradual softening behaviour which was due to softening response of grout material and bolt-grout interface (stage III). Finally, from point “r” to point “s” the stress-displacement curve reached a plateau which was due to frictional response of the broken grout (stage IV). You can see that from point “o” the rate of increase in the magnitude of normal stress significantly reduced and this continued until the end of the pull-out procedure (Figure 6b).

Figure 7 illustrates the results of the numerical direct shear tests performed on the bolted rock joints. The rock blocks consist of coal and shale with their micro-mechanical properties were identified by conducting a UCS test (section 3). The micro-mechanical properties of grout and bolt-grout interface were calibrated against laboratory data (Saadat and Taheri 2019b). Figure 7a shows the shear stress-displacement graphs, and Figure 7b illustrates the damage response of the numerical specimens at the end of the shearing procedure. The pretension stress magnitudes that were stored in the previous step (Figure 6) are now restored and the direct shear tests were performed in order to observe at which pretension stress magnitude the rock bolting system exhibits its ultimate performance. You can see that specimen “o” exhibited the highest possible resistance against shearing which means that at the pretension stress magnitude prior to the peak axial strength (point “o” in Figure 6) the bolting system can produce its ultimate performance. It is interesting that although the magnitude of induced normal stress was higher during peak and post-peak stages of the pull-out process, the other pretension stresses from point “p” to “s” failed to return a higher peak shear stress than point “o” that represents the peak stress point during the pull-out test. This may be attributed to the fact that the
compressive forces on the rock joint interface grew rapidly during post-peak (Figure 6) that encouraged the rock contacts to come closer to their yielding limits, and consequently the specimen with post-peak pretension stress magnitudes exhibited lower shear resistance with a more severe asperity degradation when compared to specimen at point “o”. The results of this numerical analysis show that the combined pull-shear load significantly affect the shear performance of fully grouted rock bolts and the SPST scheme is found to be an effective methodology in assessing the performance of rock bolting systems.

Figure 6: The results of the numerical pull-out test using he SPST scheme (Saadat and Taheri 2019b): (a) axial stress-displacement curve during the pull-out procedure, (b) induced normal stress on the rock joint interface versus axial displacement of the rock fully grouted rock bolt

Figure 7: The results of combined pull-shear load tests using the SPST scheme: (a) shear stress-displacement curves, (b) failure of the specimen after completing the shearing process.
CONCLUSIONS
This paper presents the application of the SPST scheme in analysing the behaviour of bolted coal specimen subjected to different pretension stress magnitude. The SPST scheme enables us to conduct combined pull-shear loading tests on bolted coal specimens that is beneficial in identifying the hidden mechanisms involved in the failure of bolting systems in underground coal mining. It was observed that the fully grouted rock bolts experienced four different stages during the pull-out procedure, and the monitoring results revealed that the pull-out load (axial force applied on the fully grouted rock bolt) induces normal stress on the rock joint interface that shows an increasing trend during the pull-out procedure. With the failure of grout material and bolt-grout interface, the increasing trend slowed down but still showed an upward trend which was due to a transition from cohesive softening to the frictional softening response of grout material. The numerical analysis also revealed that the rock bolting system delivered its ultimate shear performance at yield pretension stress magnitude. The proposed SPST scheme has provided an efficient numerical framework that can be employed by mining engineers for carrying out realistic experiments (i.e. combined pull-shear loading test) to achieve new insights about the failure mechanism of rock bolting system. This promotes a reliable design outcome and increases the safety of mining operations.

REFERENCES
Saadat M, Taheri A. (2019c) A numerical approach to investigate the effects of rock texture on the damage and crack propagation of a pre-cracked granite. Computers and Geotechnics 111: 89-111. doi: https://doi.org/10.1016/j.compgeo.2019.03.009
INTRODUCTION TO NEW METHODS OF STATIC AND DYNAMIC PULL TESTING OF ROCK BOLTS AND CABLE BOLTS

Sina Anzanpour¹, Naj Aziz², Jan Nemcik³, Ali Mirzaghorbanali⁴, Jordan Wallace⁵, Travis Marshall⁵ and Saman Khaleghparast¹

ABSTRACT: Despite the decades where application of rock bolts and tendons have been the main supporting system for ground in both hard and soft rock formations in mines and tunnels, there remains significant uncertainty about the behaviour and performance of these easy-to-use technologies. The importance of effective support is paramount with regard to ground seismicity and dynamic loading cause by outburst, rock burst and rock blasting activities. The evaluation and assessment of the axial loading and shear behaviour of rock bolts enables the design of a credible methodology for effective and sound ground reinforcement. This paper deals mainly with the development of a pull testing rig that is used for the testing of cable bolts and tendons under both static and dynamic conditions. The dynamic test was carried out by the drop hammer impact test. It was found that a 30 % greater pulling force is needed for the pull testing of cables statically in comparison with the dynamic impact test, this being in agreement with past numerical studies.

INTRODUCTION

Safe mining and tunnelling relies on a sound and economically viable supporting system for effective ground control. Hence, the design and installation of the supporting system should carefully tip toe on the verge of safety and cost efficiency. This has led to continuous studies in support system technologies in underground space structures during the last few decades. Among the common supporting systems (from old-fashioned wooden frames to the masonry arch, to the steel arch and to the most modern concrete segments) the combination of anchor bolts (rock bolts and cable bolts and), wire mesh and shotcrete has been one of the popular supporting systems available due to their following advantages:

- Not constrained by underground excavation shape
- Swift installation after excavation
- Does not occupy operational area (optimum excavation profile)
- Provides active support and safer strata control by pretension force, and
- Economical in comparison to other modern systems

Several types of rock bolts, FRPs and cable bolts are marketed in Australia and are used in different ground conditions. Solid rock bolts are used mainly for primary support while tendons are used for both primary and secondary supports. Some of the basic and more frequent types of rock bolts and tendons are classified in Table 1.

Over the past several decades a number of studies have covered the performance of rock bolts and tendons under different loading conditions. Tendons are mainly subjected to axial or shear load or the combination of both; however, available experimental studies and numerical modellings are addressed separately. In spite of studies undertaken and widely reported as to the behaviour of tendons under static loading condition, only a handful were reported on the dynamic testing of tendons, particularly under axial conditions. These include the works of Habenicht’s (1965), Otuonye (1988), Ortlepp and Stacey, (1994) and Tennent, et al., (1995). These studies were based mainly on field measurements of controlled blasting using geophones and strain gauges.

As one of the first experimental studies in 1977, Veesaert studied both the static and dynamic pull-out resistance of anchors buried in dry conditions and with the emphasis being placed on the profile of the

¹ PhD student, School of Civil, Mining and Environmental Engineering, University of Wollongong
² Professor, Naj Aziz, Email: naj@uow.edu.au Tel: +61 2 42221 3449
³ Dr./Honorary Senior Fellow, University of Wollongong. Email: jnemcik@uow.edu.au Tel: +61 2 4221 4492
⁴ School of Civil Engineering and Surveying, University of Southern Queensland, email: ali.mirzaghorbanali@usq.edu.au
⁵ Technical officer, School of Civil, Mining and Environmental Engineering, University of Wollongong
uplifted sand subjected to static and dynamic loadings of embedded half anchors. Their study revealed that both the static and dynamic loading of the anchors does not significantly affect the failure of the surface profile, even though, the depth of embedment can alter the geometry of failure. Veesaert’s tests showed a greater pull-out resistance with the dynamic test compared with the static test and the reasons provided were convincible, based on the theory of inertial forces and increased shear resistance under dynamic motions (Veesaert 1977).

<table>
<thead>
<tr>
<th>Table 1 Common Australian rock bolts and tendons</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Rock bolts</strong></td>
</tr>
<tr>
<td><img src="image" alt="Rock bolt" /></td>
</tr>
<tr>
<td>Cross section</td>
</tr>
<tr>
<td>Solid steel</td>
</tr>
<tr>
<td>Hollow</td>
</tr>
<tr>
<td>FRP</td>
</tr>
</tbody>
</table>

Fundamental studies by the end of the twentieth century are numerous and some notables are worth mentioning; these include the works of Orlepp and Stacey, (2000); Kabwe and Wang (2015), Thenevin et al. (2017); Wang and Cai (2017), Aziz, et al (2016, 2019, 2020); Khaleghparast, et al., (2020), S Chen, Saydam, and Hagan (2018) and Wang et al. (2018). In the most recent laboratory development studies, Thenevin (2017) and Chen et al. (2017) introduced new laboratory equipment for static pull-out testing. Thenevin, et al., (2017) designed a double embedment pull-out test rig to study the effect of various parameters including the confining pressure, embedment length and roughness of the borehole (Figure 1). In this study, three types of rock bolt, smooth, ribbed rebar and Fibre Reinforced polymer (FRP) bolt were used. Additionally, three other types of 23 mm diameter seven-strand Reflex cable, 16.2 mm diameter mini-cage cable bolt and a special cable for Polish coal mines were tested. The unique design of the hydraulic confinement pressure of up to 25 MPa and using rock samples instead of concrete samples added to the value of this research. The hydraulic confinement system was capable of controlling either confinement or stiffness of the medium during the test. Even though their results with these rock bolts would follow a reasonable trend and satisfied the theory of the pull test, however, experiments carried out on cable bolts, including mini-caged cables were not successful, as major cracks in the rock samples did not allow for the investigation of the effect of confinement.

Also Chen (2016) and Hagan, et al. (2017) studied the performance of tendons under static pull-out loading conditions. In this study, a modified version of the double embedment pull-out test (Figure 2) was used and evaluated the effect of several parameters including media strength, cable type, the confinement stiffness and embedment length.

The abovementioned studies have been carried out on static pull-out loading of tendons only, however, the dynamic response of tendons under the same loading condition remains to be determined. Accordingly, this study describes a new pull-out rig which is capable of pull testing rock bolts and tendons under both static and dynamic test conditions.
TESTING MECHANISM

Generally, two methods are used for the pull testing of both rock bolts and wired tendons; single embedment and double embedment (Aziz, 2004; Cao et al. 2013; Ma et al. 2016). Double embedment is the common pull-out test method in which the whole length of the tested cable is encapsulated in the double embedment within different encapsulation types and lengths (Thomas 2012, Aziz, et al., 2015). In this particular study, tendons are encapsulated by cementitious grout in concrete media, which is closely simulating the real field conditions for bolting. 40 MPa cylindrical concrete samples (300 mm in diameter and 450 mm in height) are confined externally by 30 mm thick half-cylinder steel.
clamps. The two halves are bolted to each other to clamp and confine the concrete firmly to prevent possible radial cracks in the concrete samples. A tendon is inserted in a 35 mm rifled borehole in the centre of the concrete sample and the gap between the tendon and concrete is filled by cementitious grout.

Unlike previously presented methods, the new design has been based on push-to-pull loading of the tendon out of the grouted concrete sample. Figure 3 shows the main elements of the new pull-out test rig. The static or dynamic load can be applied downward on the loading plate. The applied load is transferred to a vertical confining plate by four connected load transferring shafts. In addition to lateral confinement the concrete sample is held vertically firm by 30 mm vertical confining plates. The vertical confining load is also adjustable by using bolts and confining rods up to 60 N.m. The top extruded end of the tendon is covered by an anti-rotation confining tube (100 mm long), the chamfered edges of the tube are firmly gripped by the anti-rotation plate. Eventually, the hollow load cell records the axial load carried by the barrel and wedge.

![Figure 3. Simplified schematic drawing of new static and dynamic pull-out test rig](image)

The rig has been designed to tolerate a 1000 kN static load. Two different loading rigs are used for loading in either static or dynamic modes. A hydraulic sixty-tonne compression load frame shown in Figure 4a applies the static load with a minimum rate of loading being between 0.5 mm/min and 25 mm/min. For dynamic loading the load can be applied by a free falling impact drop hammer as shown in Figure 4b. The hammer weights 600 kg and the drop height is 3.5 m. In other words, roughly 20 kJ of potential energy can be applied in a fraction of a second at an approximate speed of 8 m/s.

Possible modes of failure in this study include:

- Deboning of cable and grout: According to the design, this is the most probable failure mode.
- Cable failure outside of the concrete: if the bond between the grout and cable inside the concrete sample remains sound, then failure occurs between the anti-rotation grout and concrete sample.
- Cable failure inside the barrel and wedge: the main applied load on the cable is carried by the barrel and wedge. This may result in stress concentration on the B&W and possible failure of the cable or B&W as part of the whole supporting system.
Concrete cylinder casting

The circular concrete blocks are cast in 300 mm diameter Formatube cardboard cylinders. Two 300 mm and one 450 mm cardboard lengths are cut and assembled in a specially prepared wooden frame for the concrete pour as shown in Figure 5. During the casting of the concrete and production of the central hole for cable installation, a conduit wrapped with 8 mm PVC tube is held vertically along the mould to precast a rifled hole through the centre of the concrete blocks. Once the concrete was poured it was left to set and harden, the steel conduit as well as the PVC tube are removed in similar fashion as reported by Aziz et al. 2017 in ACARP project report C24012.

After the concrete pour, samples are cured for 28 days in a purpose-built water tank to reach their maximum strength. In the next step, each tendon is encapsulated vertically in the cast center hole. As soon as the grout sets, radial clamps (two half) and axial confinement plates are placed around the concrete cylinder and bolted together. Flexible rubber mats are inserted between the top and bottom steel plates and the concrete sample to minimise the effect of concrete surface roughness.

RESULTS AND DISCUSSION

The first set of preliminary static and dynamic tests were carried out using MW9 Megabolt cables. The purpose of the test was for calibration and justification of the designed rig. Figure 6 shows test specifications and results of the static test.

Figure 4: (a) Pull test rig ready for static test by 60-ton hydraulic jack and (b) Pull test rig ready for dynamic test by impact drop hammer

Figure 5. Concrete sample preparation

University of Wollongong, University of Southern Queensland, February 2021
The static test results revealed the contribution of each of the cable and B&W in simultaneous load distribution. This can be found from the gentle slope of the graph in the first 10 mm. Normally the first 5 mm of the loading graph presents the fully elastic strength of the grout against the pull-out test. This is while in existence of B&W, flexible deformation of the wedge inside the barrel leads to the smoother force distribution on the system. The second test evaluates the behaviour of the same cable under impact dynamic loading. A drop hammer applies an equivalent energy of 14.7 kJ by free falling from a height of 2.5 m. Test specifications and results have been summed up in Figure 7. As shown in the graph of impact load versus time in the mentioned figure, there are three stages of loading. The first peak load (1) is the inertia effect, which is the inertia transferring force from the moving object to the stationary object. The second stage (2) is known as the load bearing of the tendon, when the sample is moving downward along with the impacting hammer, which creates a low-frequency oscillation in the load-time curve. According to the conservation of momentum, the momentum before collision equals the momentum after collision. The third peak (3) is the pull-out load after post peak loading.

CONCLUSIONS

A universal new testing rig has been developed for pull testing of rock bolts and tendons under static and dynamic conditions. The availability of this rig together with the double shear testing facilities, available in the same laboratory would permit a better understanding of the effectiveness of the rock bolts and flexible tendons in underground and surface structures undergoing seismic activities. The rig is a versatile tool which can be used for testing the various ground support reinforcement tools of both rock bolts and cable bolts, thus enabling the selection of reinforcement units to suite the prevailing ground conditions particularly in locations where the effect of rock bursts and gas outburst may occur. Preliminary comparison of static and dynamic test results revealed that the dynamic pulling force is 30% lower than that of the force spent in pulling out the cable statically because of the absence of time related frictional force needed to pull out the cable; however, further experiments are required to verify this statement. The rig is a useful tool together with double shear rigs under one roof for testing and selecting the correct tendons effectively for given ground conditions, particularly when the ground is likely to be subjected to seismic activities, thus mitigating the effect or influence of rock burst.

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Grout</th>
<th>cable</th>
<th>Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength (MPa)</td>
<td>Age (days)</td>
<td>Water/Cement ratio</td>
<td>Age (days)</td>
</tr>
<tr>
<td>40</td>
<td>28</td>
<td>0.4</td>
<td>14</td>
</tr>
</tbody>
</table>

![Figure 6: Specification and result of static test](image)

<table>
<thead>
<tr>
<th>Test Outputs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max applied load (kN)</td>
</tr>
<tr>
<td>Max load in cell (kN)</td>
</tr>
<tr>
<td>Max displacement (mm)</td>
</tr>
<tr>
<td>Absorbed energy (kJ)</td>
</tr>
<tr>
<td>Unit load (kN/mm)</td>
</tr>
<tr>
<td>Unit energy absorption (kJ/mm)</td>
</tr>
</tbody>
</table>
Concrete Grout Cable Test

<table>
<thead>
<tr>
<th>Strength (MPa)</th>
<th>Age (days)</th>
<th>Water/cement ratio</th>
<th>Age (days)</th>
<th>Strength (MPa)</th>
<th>Type</th>
<th>Embedment length (mm)</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>28</td>
<td>0.4</td>
<td>14</td>
<td>55</td>
<td>MW9</td>
<td>plain</td>
<td>300</td>
</tr>
</tbody>
</table>

Test Outputs

<table>
<thead>
<tr>
<th>Max rig load (kN)</th>
<th>Max load cell (kN)</th>
<th>Max displacement (mm)</th>
<th>Applied energy (kJ)</th>
<th>Unit load (kN/mm)</th>
<th>Unit energy absorption (kJ/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>-</td>
<td>124</td>
<td>14.7</td>
<td>3.25</td>
<td>0.119</td>
</tr>
</tbody>
</table>

Figure 7: Specification and result of dynamic test

REFERENCES


NUMERICAL APPROACH TOWARDS DYNAMIC DOUBLE SHEAR TESTING OF TENDONS USING LS-DYNA

Saman Khaleghparast¹, Alex Remennikov² and Naj Aziz³

ABSTRACT: Underground support systems using tendons has been one of the significant achievements in Civil and Mining engineering endeavors in facing the challenges of ground control. However, shear failure of rock bolts is a concern in underground excavations particularly with respect to seismic events. The understanding of the performance of rock bolts under dynamic loading condition requires a great deal of research. A series of tests were undertaken utilising a drop hammer mass of 592 kg dropped from a maximum height of 2 m onto grouted rock bolts encapsulated in concrete blocks in the double shear box to investigate the performance of rock bolts under dynamic shear load. Load cells, a displacement laser and high-speed camera were used to monitor the tests. Results from the data analyses are presented in the form of displacement, hammer mass drop velocity, force variation with time for all components involved in each test. A numerical simulation using ANSYS/LS-DYNA was used to simulate the behavior of 18 mm conventional rock bolts under high impact velocity loading conditions. The numerical simulation model was found to be in good agreement with the experimental results.

INTRODUCTION

Characterization of the strength of rock bolts and cable bolts for underground mining applications is generally based on tensile and shear strength. These two properties are determined by static testing, however in recent years, many tests have been reported with respect to the dynamically testing of rock bolts (Player, 2004, Plouffe, et al., 2008). The current thinking of ground reinforcement in underground mining has moved away from the emphasis being solely on ground support. The increased rate of mining development, production and mine operator safety are continuing to gain equal importance with ground support and its resilience in adverse mining environments. The prospect of ground seismicity and rock bursts requires special attention be focused to support infrastructure installation effectiveness, both in metal and coal mining. The Beaconsfield gold mine collapse, in Tasmania, triggered by seismic activity and pressure bursts at Austar coal mine, due to high levels of stress contribution caused by the presence of disturbed structural geology in the region with fault zones and shear zones as reported by Galvin and Hebblewhite (2016) are stark reminders of the challenges that mines are faced with in adverse conditions. This necessitates the need for credible research on ground support under these adverse conditions under both static and dynamic testing. The need for effective research on ground support credibility is of equal importance to the collapse of the ground due to gas outbursts, which are more common in coal mines worldwide and are well documented (http://miningst.com/category/coal-mine-outburst/). It is obvious that dynamic testing of tendons appears to focus on axial tensile testing and no reporting has been made on dynamic testing in shear. Tendon shear strength characteristics are important when shear deformation occurs across joints and shear zones, which are the weakest zones of the ground structure and that normally yield readily to rock burst or any other seismic activity. This paper describes the method of dynamic shear testing of tendons using double shear apparatus and compares the findings with the static method.

METHODOLOGY

The methodology used in this study will utilise testing of tendons by the double shear test rig, known as MK-I. The procedure of the testing was previously described in detail and it can be found from Khaleghparast, et al (2020).

¹ PhD candidate, Saman Khaleghparast. Email: sk329@uowmail.edu.au M: +61 (0) 423 812 516
² Professor, Alex Remennikov. Email: alexrem@uow.edu.au
³ Professor, Naj Aziz, Email: naj@uow.edu.au Tel: +61 2 42221 3449
The high-capacity impact machine

The drop hammer test method simulates a high energy impact load condition similar in amplitude and velocity to a rock burst event. Therefore, in this study, the drop hammer was employed to examine the impact and shear performance of conventional rock bolts and cable bolts. Figure 1 shows the schematic and general view of the drop hammer load impact rig. The core of the test rig is the free-fall hammer having a 592 Kg weight that can be dropped from a maximum height of 4.0 m, or equivalent to the drop velocity of up to 8.85 m/s. A 1200 kN dynamic load cell (Type Interface Model 1200) attached to the hammer measures the force applied to the medium at the time of the impact. The drop hammer test method simulates a high energy impact load condition similar in amplitude and velocity to a rock burst event. As the hammer falls, a laser gate triggers the load cell which allows data to be recorded by a data acquisition system. The collected data is transferred to a computer where the result can be analysed /processed. A high speed camera “Fastec trouble-shooter” was utilised to capture the high-energy impact between the drop hammer and the MK-I shear box with high accuracy. This allows an accurate analysis of the displacement of the central block during the shear load drop over the period of time. The shearing displacement of the central block was also monitored through utilising a laser placed underneath the central block. The MK-I set up box was seated on a U shape beam. The beam was placed between two base plates, which were anchored to the ground. The outer frames of the shear box were then clamped tightly to the base platform to avoid rotation of the blocks during drop loading impact.

![Drop hammer test and MK-I set up](image)

To ensure that the load from the drop hammer is distributed evenly over the central block and also to prevent damaging to the box, a 30 mm thick plate was placed on top of the central block and covered with 3 mm thick plastic rubber. The potential height for each specimen was calculated based on the absorbed energy in the static test calculated from the area under the Load-displacement curve.

**NUMERICAL MODELLING**

The numerical analysis of the dynamic behaviour of rock bolts in a jointed rock mass, when subjected to extreme loading, can be studied using non-linear finite element software such as ABAQUS and LS-DYNA. In this study, ANASYS/LS-DYNA non-linear finite element software was considered. The advantage of using LS-DYNA is the efficient computational capability and the availability of a comprehensive material library. The current version of LS-DYNA contains more than 270 material models, of which 100 are constitutive models controlled by ten equations of state to cover a broad spectrum of materials (LSTC 2016, Bohara et al. 2019). LS-DYNA is the most widely used explicit analysis program, capable of simulating the response of material to short duration dynamic loading. Its many elements contact formulations, material models, and other controls can be used to simulate complex models with control over all the details of the problem. LS-DYNA can perform simulations of mechanisms involving joints and articulations subjected to impacts whether from drops or collisions. A three-dimensional (3D) FE model was generated using Strand7 software. Strand7 is a finite element analysis software that gives the user unparalleled functionality in a single application (Strand7 2010). Strand7 allows the user to have automatic mesh generation tools, working directly from the geometry.
It allows users to automatically generate 4-node and 10-node tetrahedral solid elements from 3D solid models. For this study, automatic surface mesh generation was used to create a high-quality FE mesh. The created mesh was composed of linear, quadratic triangular, and quadrilateral elements (3 node triangular elements and 4 node quadrilateral elements). After creating the geometry in Strand7, the file was exported to LS-PrePost for the dynamic analysis, shown in Figure 2. To represent the drop hammer, the 50 mm steel plate, the dynamic load cell as well as the tup, eight-node solid hexahedron elements with single-point integration were used. To reduce the computational time and make the simulations more efficient, one quarter of the entire body of the 3D model was considered due to the existing symmetry planes in the model.

**Table 1: Results of static and dynamic tests carried out on 18 mm ribbed rock bolt**

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Tension type</th>
<th>Pretension (kN)</th>
<th>Borehole Ø (mm)</th>
<th>Drop height (m)</th>
<th>Concrete strength (MPa)</th>
<th>Internal confinement</th>
<th>Static load (kN)</th>
<th>Dynamic load (kN)</th>
<th>70% static load (kN)</th>
<th>Effective friction (%)</th>
<th>Absorbed Energy (kJ)</th>
<th>Static</th>
<th>Dynamic</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>18 mm ribbed bolt</td>
<td>30</td>
<td>24</td>
<td>2</td>
<td>40</td>
<td>Yes</td>
<td>324</td>
<td>230</td>
<td>227</td>
<td>71</td>
<td>11.3</td>
<td>7.9</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>18 mm ribbed bolt</td>
<td>50</td>
<td>24</td>
<td>1</td>
<td>40</td>
<td>Yes</td>
<td>324</td>
<td>227</td>
<td>227</td>
<td>70</td>
<td>10.3</td>
<td>7.1</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>18 mm ribbed bolt</td>
<td>30</td>
<td>24</td>
<td>2</td>
<td>40</td>
<td>Yes</td>
<td>324</td>
<td>236</td>
<td>227</td>
<td>73</td>
<td>11.3</td>
<td>8.1</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>18 mm ribbed bolt</td>
<td>50</td>
<td>24</td>
<td>2</td>
<td>40</td>
<td>Yes</td>
<td>342</td>
<td>200</td>
<td>219.5</td>
<td>62</td>
<td>13.1</td>
<td>6.6</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>18 mm ribbed bolt</td>
<td>30</td>
<td>46</td>
<td>2</td>
<td>20</td>
<td>NO</td>
<td>331</td>
<td>160</td>
<td>232</td>
<td>59</td>
<td>13.5</td>
<td>4.7</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>18 mm ribbed bolt</td>
<td>50</td>
<td>24</td>
<td>2</td>
<td>60</td>
<td>NO</td>
<td>304</td>
<td>200</td>
<td>213</td>
<td>70</td>
<td>15.7</td>
<td>8.3</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>18 mm ribbed bolt</td>
<td>30</td>
<td>32</td>
<td>2</td>
<td>20</td>
<td>NO</td>
<td>331</td>
<td>190</td>
<td>232</td>
<td>69</td>
<td>13.5</td>
<td>11.1</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>18 mm ribbed bolt</td>
<td>50</td>
<td>32</td>
<td>2.5</td>
<td>60</td>
<td>NO</td>
<td>304</td>
<td>184</td>
<td>313</td>
<td>65</td>
<td>15.7</td>
<td>9.9</td>
<td></td>
</tr>
</tbody>
</table>

*Figure 2: 3D model of MK-I DS box geometry as well as drop hammer compartments in LS-PrePost environment*

The material models used in the FEM analysis was chosen as follows. The Continuous_Surface_Cap_Model (CSCM, *MAT_159) was used for concrete and grout and the Mat_Piecewise_Linear_Plasticity model (*MAT_024) was used for the rock bolt. The Plastic_Kinematic model (*MAT_003) was used to simulate the steel confinement wrapped around the concrete blocks. The RIGID model (*MAT_020) was used to simulate the drop hammer and the ELASTIC model (*MAT_001) was used to simulate the load cell as well as the impact plate, respectively.
Overview of the CSCM model (*MAT_159)

CSCM model (*MAT_159) was originally proposed by Murray (2007). It is a developmental version of the concrete model that the developer has successfully employed and progressively enhanced since 1990 on defense contracts to analyse the dynamic loading of RC structures. This model has been implemented into the dynamic FE code, LS-DYNA, beginning with version 971. This model is in keyword format as *MAT_CSCM for CONTINUOUS_SURFACE_CAP_MODEL. Murray (2007) developed this material model to predict the dynamic behaviour, both elastic and plastic deformation or the failure of concrete material, which was used in roadside safety structures when involved in a collision with a motor vehicle. The user manual for LS-DYNA concrete material model 159 and Evaluation of LS-DYNA concrete material model 159 (FHWA-HRT-05-063) are the two documents that completely document this material model. This model includes initialisation routines that provide the user with the default input parameters for normal strength concrete. These initialisation routines set the required strengths, stiffness, hardening, softening, and rate effecting parameters as a function of concrete strength, maximum aggregate size, and the units.

Yield surface

The CSCM model consists of a smooth failure surface and uses damage mechanics to simulate strain softening and modulus degradation in both tensile and compression regimes as well as viscoelasticity for strain rate effects. It is a three-invariant extension of the *MAT_GEOLOGIC_CAP_Model (MAT_025). This model considers plastic flow and damage accumulations as a separate process based on the effective stress concept and the hypothesis of strain equivalence in continuum damage mechanics (CDM). The assumption for this model is that the shear stresses control the plastic flow, which may cause permanent deformation without causing degradation of elastic moduli, and the damage is assumed to result in the progressive degradation of the moduli. To model the plastic volume change, an elliptical cap surface was added to the model. This feature, besides concrete, is capable of modelling geo-materials including soils and rocks (Murray 2007). The model is a combination of a yield surface of a shear failure surface $F_f(l_1)$ and a cap surface $F_c(l_1, \kappa)$, with a continuous and smooth connection between the two as shown in Figure 3.

![Figure 3: Graph of a multiplicative formulation of the shear and cap surfaces (adapted from (Murray 2007))]({})

Damage formulation

Concrete shows strength reduction in the tensile and low-to-moderate compressive regimes. In this model, the softening or strength reduction is modeled via a damage formulation. Murray (2007) stated that without the damage formulation, the cap model predicts perfectly plastic behaviour for experimental simulation such as direct pull, unconfined compression, triaxial compression, and tri-axial extension, which is not realistic behaviour for concrete at lower confinement and in tension. The damage stress function can be written according to the effective stress concept in CDM:

$$\sigma = (1 - d)\overline{\sigma}$$

where the scalar damage variable $d$ ($0 \leq d \leq 1$) grows from zero (undamaged material) to unity (completely damaged material with effective area reduced to zero). $(1 - d)$ is a reduction factor related
to the amount of damage at a material point. The accumulation of damage is based on two distinct formulations, which are known as brittle damage and ductile damage.

**Overview of the Piecewise Linear Plasticity Model (MAT_024)**

The PIECEWISE_LINEAR_PLASTICITY (*MAT_024) is an elasto-plastic material with an arbitrary stress-strain curve. In this model, a failure can be defined in two different ways including minimum step size or a plastic strain. This model is suitable and recommended by LSTC (2016) for solid elements. If the elastic strains of the material are finite before yielding, the material model treats the elastic strains using a hyper-elastic formulation. In this model the log interpolation keyword option is available, allowing the model to interpolate the strain rates in a table defined in LCSS with logarithmic interpolation. This model requires 28 parameters among which six parameters including density, young's modulus, Poisson's ratio, effective plastic true strain at failure, initial yield stress, and tangent modulus are required to simulate the model appropriately.

Stress-strain behaviour may be controlled by a bilinear stress-strain curve by defining the tangent modulus, shown as ETAN in the card. However, LSTC (2016) recommends calculating effective stress as a function of effective plastic strain. Figure 5 illustrates a curve of effective stress as a function of effective plastic strain.

![Figure 4: Strain softening and modulus reduction simulation using CSCM material model in LS-DYNA adapted from (Murray 2007)](image)

Figure 4: Strain softening and modulus reduction simulation using CSCM material model in LS-DYNA adapted from (Murray 2007)

![Figure 5: Effective stress as a function of the plastic strain curve for MAT_024 adopted from (LSTC 2016)](image)

Figure 5: Effective stress as a function of the plastic strain curve for MAT_024 adopted from (LSTC 2016)

Accordingly, the true stress vs the true strain relationship was calculated for this model. It is recommended to input a smoothened stress-strain curve utilising a minimal number of points (LS-DYNA Support). The reason is that the experimental results always include some degree of error and tend to be somewhat noisy and erratic, which may create confusion within the model. Therefore, the smoother the stress-strain curve, the better the outcomes. Also, it is stated that the plastic strain in the defining curve should be the residual true strain after unloading elastically and true stress should be
used directly for stress values. Therefore, as an input, the curve shown in Figure 6 is defined as an input for the MAT24 model.

Figure 6: True stress versus plastic strain curve as an input in LS-DYNA

**Mechanical response of MK-I in LS-DYNA**

A 592 kg weight hammer was set to be dropped on the middle section of MK-I to create a dynamic double shear load in the system. In this regard, an initial impact velocity was set by choosing the drop hammer elements, measuring 6.3 m/s equal to 11.6 kJ of potential energy. Once the hammer dropped on the middle section, the whole section including the impactor, and the middle section including the rock bolt began to accelerate, travelling downward. The mechanical response of the whole system was found to be the same as what was expected from the laboratory experiments. In the model, once the impactor hits the box, the velocity of the impactor decreases whilst the velocity of the specimen increases.

Figure 7 illustrates the energy transformation from the kinetic energy of the impactor to the internal energy of the MK-I box predicted by the LS-DYNA. As it can be seen, once the impact occurred, the kinetic energy starts to decrease whilst the internal energy of the system begins to increase. The duration of this transformation of energy is 10 ms. Meaning that the kinetic energy of the impactor decreased to zero in 10 ms whilst the internal energy of the system increased significantly and reached 8.4 kJ. The hourglass energy is also presented in Figure 7, where it is calculated to be less than 7% of the total energy. It was found that the amount of energy absorbed by the system is not the same as the total input energy, meaning some amount of energy was dissipated during the process of dynamic double shearing. Total energy was determined to be 11.6 kJ and only 8.4 kJ was absorbed by the system.

Figure 7: Total energy, kinetic energy versus absorbed energy, and hourglass energy in the system
To understand in detail what and how much energy is dissipated or how much energy is absorbed by the system a thorough investigation will be conducted on individual components of the system including the rock bolt, grout, concrete blocks, and steel casing confinement in the following.

Table 2 represents the calculated velocity and energy absorption of the MK-I apparatus in experimental and numerical tests.

Table 2: Velocity and energy absorption by MK-I in numerical and experimental results

<table>
<thead>
<tr>
<th>Test No</th>
<th>Velocity (m/s)</th>
<th>Absorbed energy (kJ)</th>
<th>Absorbed energy/total input energy (%)</th>
<th>Part</th>
<th>Velocity (m/s)</th>
<th>Absorbed energy (kJ)</th>
<th>Part energy/total absorbed energy (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>4.38</td>
<td>7.1</td>
<td>68</td>
<td>MK-I</td>
<td>5.75</td>
<td>0.15</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.35</td>
<td>2.0</td>
<td>17</td>
</tr>
<tr>
<td>3</td>
<td>5.23</td>
<td>8.1</td>
<td>72</td>
<td></td>
<td>6.20</td>
<td>1.5</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.2</td>
<td>0.4</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.2</td>
<td>3.0</td>
<td>26</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0</td>
<td>0.3</td>
<td>2.6</td>
</tr>
<tr>
<td>4</td>
<td>4.75</td>
<td>6.6</td>
<td>80</td>
<td></td>
<td>0</td>
<td>1.05</td>
<td>9</td>
</tr>
<tr>
<td>Average</td>
<td>4.78</td>
<td>7.3</td>
<td>67%</td>
<td></td>
<td>4.94</td>
<td>8.4</td>
<td>72%</td>
</tr>
</tbody>
</table>

Looking at the numerical results, the energy as well as the velocity of each element measured and recorded in Table 2. The velocity of concrete and steel confinement is much the same and measured as 6.20 and 6.35 m/s respectively. On the other hand, the velocity of the rock bolt and grout is recorded as being the same at 3.2 m/s. However, the amount of energy that the rock bolt can absorb in comparison with grout is significantly different. The contribution of the rock bolt in absorbing energy is 26 % of the internal energy in the system, whilst grout only absorbed 3.5 % of the internal energy. It is understandable that the rock bolt plays a pivotal role in the system and it can simply define the stiffness of the system. The role of grout, however in the dynamic double shear testing is not as important as it is in static loading conditions. In low strain loading conditions, the grout acts as a strong bonding element between concrete and the rock bolt. However, since the grout has a high compressive strength of almost 65 MPa for this study, it can absorb less energy, because of its brittle nature. Therefore, it is understandable that as soon as energy travels through the material, the grout will crust quickly. The steel confinement and concrete block in the middle section, on the other hand, absorb a great deal of energy from the system, 20 and 13 %, respectively.

Comparing the results from an experiment with a numerical model based on energy absorption can confirm that the mechanical response of the MK-I in numerical results is in good agreement with the experimental results. In both situations, almost 70 % of the total input energy is absorbed by the system. Furthermore, the velocity of the system is in good agreement with the calculated velocity of the system driven from experiments. It can be observed that the average calculated velocity from the tests of the MK-I is 4.78 m/s and the velocity of the system in the numerical model is measured as 4.94 m/s. This proves that the system’s response in the numerical model is very similar to the response of the physical system.

Figure 8 demonstrates the impact load versus time histories of the double shear test in the experiment and numerical simulation.
Figure 8: Impact load versus time in the experimental results as well as numerical simulation

It is thus clear that the impact load versus time from the numerical modeling is in good agreement with experimental results. The impact load versus time has three-stages of loading. The first stage is the inertia effect which is quite high when compared with the experiment. The second stage is when the model is moving downward along with the impactor which creates a low-frequency oscillation in the load-time curve. Nevertheless, the impact load in the numerical simulation is almost doubled as it is in the experiment. This could be due to the usage of a thick rubber pad between the impactor and impact plate in the real experiments. The rubber apparently could reduce the impact load by reducing the effect of inertial force at the moment of impact. However, a great deal of effort has been put in to include a rubber pad in the model to examine the response of the system. However, designing a 3 to 5 mm thick rubber pad as a hyper-elastic material in such a huge energy impact was concluded to be impossible, due to the inability of the software to simulate the situation. Therefore, the effect of the rubber pad between the impactor and impact plate was neglected and the impact was performed directly on the steel enclosure. Still, by excluding the rubber, the model was recognised as a reliable tool to study the response of rock bolt under impact load.

The development of energy along the 18 mm Jennmar rock bolt

When a bar is loaded laterally, it is deformed into a curve, and the resulting stresses and strains are directly related to the deflected curve, which is affected by the surrounding materials (Jalalifar, 2006). Figure 9 demonstrates the stress development in the rock bolt at joint faces at the point of impact. Once the load is exerted on the specimen at t=0.6 ms, a part of the energy is immediately transferred to the rock bolt at the joint faces. The figure illustrates half of the model.

Figure 9: General view of axial stress development in half of the rock bolt at t=0.6 ms

As can be seen from the graph, the developed stress at the time of impact on the rock bolt at t= 0.6 ms is higher than the yield strength of the rock bolt. The yield stress of the rock bolt was measured as 306 MPa and the maximum stress imposed on the rock bolt in the tension and compression zone is 318 and 403 MPa, respectively. The tension was shown by positive values and compression was marked by negative values. The high amount of stress at the time of impact can be interpreted in a
way that the rock bolt starts deforming plastically, immediately after the impact. This prevents the development of axial stress in the rock bolt as a significant amount of energy in the form of concentrated energy is developed in the vicinity of the shear joints. The concentrated energy in the vicinity of the joint gradually increases, starting with plastic deformation and ends with a rupture at 10 ms. From the theory, it is known that the shear stress in the vicinity of joints is the highest and reduces from the shear face towards the bolt end. On the other hand, the bending moment (hinge point) in the shear face is zero. The maximum effective Von Mises stress is 872 MPa which can create 224 kN of shear load at 8 ms with a shear displacement of 44 mm, as is shown in Figure 10.

The shear load is developed at the shear zones under the initial impact velocity of 6.3 m/s. The built-up shear force can rupture the 18 mm Jennmar ribbed rock bolt as the load exceeds its ultimate tensile strength. Table 3 shows the numerical and experimental results under impact loading conditions. The average shear load from four experimental results was measured as 223 kN and the deflection of the bolt was determined as 42 mm. From the numerical results, the peak shear load was measured as 224 kN with a shear displacement of 44 mm.

![Figure 10: Dynamic shear load vs time as well as dynamic shear displacement vs time predicted by LS-DYNA](image)

<table>
<thead>
<tr>
<th>Sample properties</th>
<th>Experimental results</th>
<th>Numerical results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test No</td>
<td>Borehole $\varnothing$ (mm)</td>
<td>Concrete strength (MPa)</td>
</tr>
<tr>
<td>1</td>
<td>24</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>24</td>
<td>40</td>
</tr>
<tr>
<td>3</td>
<td>24</td>
<td>40</td>
</tr>
<tr>
<td>4</td>
<td>24</td>
<td>40</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

As can be seen, both the shear load and the deformation captured from the numerical results are in relatively good agreement with the experimental results. Therefore, it is safe to say that the numerical model can predict the shear load as well as shear displacement under impact loading conditions.

**Friction effect**

Calculation of the frictional energy was undertaken through activation of FRCENG to 1.0 in the menu of control_contact. By activating this option, the frictional energy between all elements that have been defined by the Contact algorithms is calculated. The results are recorded by activating an option SLEOUT. On the other hand, the total sliding energy is also calculated through the GLSTAT command. Some 2.35 kJ of the total energy was wasted as sliding energy to overcome the friction between each component, meaning that 20% of the total energy is wasted through friction between each part. These contacts are between confinement and concrete, grout and concrete, grout and rock bolt, middle confinement and side confinement, and middle concrete to side concrete. Figure 11 shows the energy absorption between each part due to friction.
Figure 11: Dissipation of energy to overcome the friction between each element

As can be seen from the graph, 80% of the wasted energy is lost through overcoming the friction between the shear faces, which is sliding between mid and side confinement as well as concrete. The frictional energy in the shear joint was recorded as 1.9 kJ which is almost 20% of the input energy.

Figure 12 shows the ratio of energy dissipated due to the erosion effect. Eroded energy is the energy associated with deleted elements (internal energy) and deleted nodes (kinetic energy) due to a negative influence on the calculation. Approximately 8% of the energy dissipated in the system is due to erosion. This 8% of energy loss was previously interpreted as energy loss due to vibration, heat, noise, and cracking in concrete blocks, without quantifying this amount.

Figure 12: Energy dissipation in the system due to erosion

CONCLUSIONS

Dynamic response of conventional 18 mm ribbed rock bolt was investigated numerically using LS-DYNA under impact loading conditions. The mechanical response of the MK-I double shear box under high-velocity impact loads was assessed using 40 MPa concrete blocks. It was found that the model could predict the dynamic shear load as well as dynamic shear displacement of the 18 mm rock bolt under impact load. The following conclusions can be drawn:

- The impact load in the numerical model was almost double that of the experimental results. This was interpreted as the effect of using a 3 mm rubber pad between the impactor and impact plate during the impact experiments.

- The contribution of the rock bolt in absorbing energy was the highest among each component of the specimen, absorbing 26% of the internal energy before steel casing confinement and concrete with 20 and 13%, respectively. The rock bolt can define the stiffness of the MK-I double shear box in dynamic double shear testing.
70% of the total input energy is absorbed by the MK-I double shear apparatus. This percentage was in good agreement with the experimental results.

Only 20% of the total input energy went to overcome the friction between joints, which was the friction between the steel confinement and concrete faces. Furthermore, only 8% of the energy was wasted due to the effect of erosion activated in the numerical model.

REFERENCES


Strand 7, 2010. Introduction to the Strand7 finite element analysis system.

Villaescusa E, Thompson, A and Player J R, 2005. Dynamic testing of ground support systems, Phase I, MERIWA project No M3492, 10s9 p.
**ABSTRACT:** Pretension plays a critical role in providing active support to control roof sagging and bedding plane slippage in underground coal mines. However, the optimal amount of pretension for cable bolts to achieve effective ground control remains unknown. Applying excessive pretension on cable bolts may cause cable bolt failure. On the other hand, insufficient pretension is ineffective in preventing large roadway deformation and roof fall incidents. This paper investigates the pretension effect on cable bolts at various geological conditions for underground coal mines. A numerical model based on a coal mine in the Western Coalfield, NSW has been developed by considering the variance of in-situ stress, bedding plane and claystone properties. Model results focused on analysing the pretension effect on the cable bolt force distribution and bond failure along the cable bolts. The overall bond failure along the cable bolts at different geological conditions was also analysed with different pretensions. The neutral point, where relative shear movement between the cable bolt and rock mass is zero, is considered as a key position along the cable bolt to describe its loading behaviour. The results suggest that inappropriate high pretension would promote bond failure. This paper is expected to provide a theoretical guidance for applying optimal pretension in the coal mining industry.

**INTRODUCTION**

Fully grouted cable bolts are commonly used as one of the most popular ground support methods in underground coal mines. Fully grouted cable bolts consist of a long metal wire and grout cement which is an interface between rock mass and cable bolts, which is widely used to reduce roof sagging and bedding plane slippage. However, ground failure occasionally occurs due to cable bolt failure in the past few decades, threatening support efficiency and underground coal mining safety (Li et al., 2017; Thenevin et al., 2017).

In order to provide better support efficiency, pretension is generally applied on the rock bolts - a similar support system to cable bolt. Without pretension, rock bolt provides passive support since it cannot generate force until roof starts to deform. Pretension mobilizes axial tensile force inside rock bolt, enabling the whole support system to perform proactively. The benefits of pretension on bedding planes and roof control were widely confirmed by many researchers. Frith and Thomas (1998) stated that pretension on rock bolt has a positive effect on restricting bedding plane aperture, especially 0.5-0.8m above excavations. Peng (1998) reported that pre-tensioned rock bolts provide additional vertical support above excavation, resulting in higher shearing resistance on bedding planes. Gao and Kang (2008) proved that active rock bolt is beneficial for roof stability, by increasing vertical stress above excavation. However, excessive pretension can also cause adverse effect on ground control. Kang (2016) stated that stress induced by pretension concentrates at the end of rock bolt, which may lead to rock bolt failure.

However, no systematic study on the pretension effect was conducted that can guide the pretension applications at actual mining conditions. The magnitude of pretension can vary significantly in field applications. In the US, 11 kN (about 1.1 tonne-force) pretension is a common benchmark, whereas under complex geological conditions, pretension can be four times higher (Tadolini and Ulrich, 1986; Dolinar and Bhatt, 2000). Yet, in Chinese deep coal mines, high strength and high pre-tensioned rock bolts are applied at large deformation roadways (Kang, 2014). Compared with traditionally low-pre-tensioned rock bolts (20-30 kN), pretension can reach 300kN. Field data analysis suggested that...
roadway roof and rib deformation are effectively controlled by rock bolts with high pretension being applied (Kang, 2014).

As a supplement to rock bolts, cable bolts are generally untensioned in underground coal mines (Moosavi, Bawden and Hyett, 2002). The investigation of pretension effect on cable bolts is inadequate and no instructions are available to guide suitable pretension values to be applied on cable bolts at different geological conditions. Site experience suggested that excessive pretension may cause cable bolt failure, whereas insufficient pretension cannot control excavation roof and laminated bedding planes effectively. In this paper, a systematic investigation about pretension effect on cable bolts at different geological and in-situ stress conditions is conducted. A series of numerical models are developed based on the setup of Mine A in the Western Coalfield, NSW, Australia, where cable bolt failure due to complex roof conditions has been reported as a severe issue for the last few decades.

METHODOLOGY

Site introduction

Mine A is located in the Western Coalfield, New South Wales, Australia. The challenging ground conditions for Mine A include roof failure and uncontrollable deformation. The main hazards at the Western Coalfield are weak bedding planes and water weakened claystone above the excavation (Lu, 2001; Hebblewhite and Lu, 2004). By strata RQD, the bedding planes are classified into weak or very weak fractures, and their properties are difficult to be obtained by lab testing. Also, the claystone above the roof strata is responsible for the unstable roof conditions as well. The intact and dry claystone is relatively stiff and competent, but after being exposed to water would significantly deteriorate its mechanical strength. Therefore, the bedding plane and claystone properties are considered as key variables that may largely affect roof behaviour in this numerical study.

Model development

The commercial software UDEC developed by ITASCA is adopted in this study, and the software is based on distinct element method (DEM). A 2D model has been developed to simulate the cross-section of a developing roadway excavation, as shown in Figure 1. The model is 40 m long and 25 m high. A rectangular roadway excavation is 3.4 m long and 4.8 m high at the model centre. Three main components are illustrated in the model, which are cable bolts, rock mass and bedding planes. Two cable bolts are installed from the roadway excavation roof to 8.2 m into deep rock mass, 0.75 m symmetrically to model centre, with 2 m spacing. Above the roof, lithology is separated into coal, claystone, sandstone and siltstone by laminated bedding planes. For further reference, eight laminated bedding planes are numbered from I to VIII as shown in Figure 1. The vertical displacement at the model bottom and horizontal displacement at side boundaries are restricted. The 'in-situ' command and a constant stress on the top boundary are used in the UDEC model to apply in-situ stress. The input properties for cable bolt, grout material, rock mass and bedding planes are shown in Table 1 to 4.

Scenarios considered

In this research, different geological conditions are simulated as different scenarios to understand the performance of pretension on cable bolts in actual mining conditions. Firstly, in-situ stress is divided into two categories as the low-stress state (1 MPa vertical stress and 2 MPa horizontal stress) and high-stress state (9 MPa vertical stress and 18 MPa horizontal stress). The high-stress state (H) is based on the field measurement result from the Mine A, representing the actual site condition, while the low-stress state (L) is a hypothetical case at very shallow mining depth. Base case (BC) is firstly defined with typical bedding plane stiffness and dry claystone properties based on the data in Table 1 to 4. Apart from the BC, four different scenarios are defined as low joint modulus (LJ), high joint modulus (HJ), low rock modulus (LR), and low UCS (LU). For future reference, all scenarios are named with three capital letters. Generally, the first letter indicates the stress state (H: high stress; L: low stress). Except the BC, the second letter illustrates the degree of change (L: low material property; H: high material property) and the third letter shows the controlled parameters (J: bedding planes; U: UCS; R: rock modulus), as summarised in Table 5. The LJ, LR, and LU scenarios apply 50% of the corresponding discontinuities modulus, rock modulus, and UCS, respectively. The HJ scenario has strengthened bedding plane stiffness, which is 50% higher than the BC. The change
of those parameters is within a reasonable range as found in the previously introduced literature showing in Table 1 to 4. The pretension of cable bolts is also a critical variable, ranging from 0 kN to 350 kN in all five scenarios.

Figure 1: The UDEC model geometry used in this research.

Table 1: Input cable bolt mechanical properties (Mega bolt, 2020).

<table>
<thead>
<tr>
<th>Cross-sectional area (cm²)</th>
<th>Density (kg/m³)</th>
<th>Borehole diameter (mm)</th>
<th>Elastic modulus (GPa)</th>
<th>Tensile Capacity (kN)</th>
<th>Pretension (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.80</td>
<td>2000</td>
<td>22</td>
<td>200</td>
<td>700</td>
<td>0-350</td>
</tr>
</tbody>
</table>

Table 2: Input grout material mechanical properties.

<table>
<thead>
<tr>
<th>Yield Capacity (kN)</th>
<th>K_bond (N/m/m)</th>
<th>S_bond (N/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>83</td>
<td>4.016x10⁷</td>
<td>5.54x10⁵</td>
</tr>
</tbody>
</table>

Table 3: Input rock mass mechanical properties (Seedsman and Mallett, 1987, 1987; Lu, 2001; Hebblewhite and Lu, 2004; Zhang, 2014; Le et al., 2019).

<table>
<thead>
<tr>
<th>Lithology</th>
<th>Density (Kg/m³)</th>
<th>Bulk modulus (GPa)</th>
<th>Shear modulus (GPa)</th>
<th>Friction angle (°)</th>
<th>Cohesion (MPa)</th>
<th>Tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Claystone</td>
<td>2693</td>
<td>6.5</td>
<td>4.3</td>
<td>32</td>
<td>11.0</td>
<td>0.5</td>
</tr>
<tr>
<td>Coal</td>
<td>1500</td>
<td>5</td>
<td>1.1</td>
<td>22</td>
<td>4.0</td>
<td>3</td>
</tr>
<tr>
<td>Sandstone</td>
<td>2527</td>
<td>3.3</td>
<td>2.5</td>
<td>46</td>
<td>5.0</td>
<td>5.2</td>
</tr>
<tr>
<td>Siltstone</td>
<td>2400</td>
<td>5.0</td>
<td>2.3</td>
<td>38</td>
<td>4.5</td>
<td>5</td>
</tr>
</tbody>
</table>
Table 4: Input bedding plane mechanical properties (Thin, Pine and Trueman, 1993; Nemcik, Indraratna and Gale, 2000; Vakili, 2009; Zhang, 2014; Bastola and Chugh, 2015; Zhang et al., 2015; Sainsbury and Sainsbury, 2017; Kang et al., 2018; Le, 2018; Le et al., 2019; Mo, 2019).

<table>
<thead>
<tr>
<th>Bedding plane stiffness</th>
<th>Normal stiffness (GPa/m)</th>
<th>Shear stiffness (GPa/m)</th>
<th>Tensile strength (MPa)</th>
<th>Friction angle (°)</th>
<th>Cohesion (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>20</td>
<td>2</td>
<td>0</td>
<td>30</td>
<td>0</td>
</tr>
<tr>
<td>Typical</td>
<td>10</td>
<td>1</td>
<td>0</td>
<td>30</td>
<td>0</td>
</tr>
<tr>
<td>Low</td>
<td>5</td>
<td>0.5</td>
<td>0</td>
<td>30</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 5: Various in-situ stress and geological scenarios considered in this research

<table>
<thead>
<tr>
<th>Model name</th>
<th>Full name</th>
</tr>
</thead>
<tbody>
<tr>
<td>HBC</td>
<td>High in-situ stress base case</td>
</tr>
<tr>
<td>HLJ</td>
<td>High in-situ stress low joint modulus</td>
</tr>
<tr>
<td>HHJ</td>
<td>High in-situ stress high joint modulus</td>
</tr>
<tr>
<td>HLR</td>
<td>High in-situ stress low rock modulus</td>
</tr>
<tr>
<td>HLU</td>
<td>High in-situ stress low rock UCS</td>
</tr>
<tr>
<td>LBC</td>
<td>Low in-situ stress base case</td>
</tr>
<tr>
<td>LLJ</td>
<td>Low in-situ stress low joint modulus</td>
</tr>
<tr>
<td>LHJ</td>
<td>Low in-situ stress high joint modulus</td>
</tr>
<tr>
<td>LLR</td>
<td>Low in-situ stress low rock modulus</td>
</tr>
<tr>
<td>LLU</td>
<td>Low in-situ stress low rock UCS</td>
</tr>
</tbody>
</table>

RESULTS

Stress distribution along cable bolt and grout

Figure 2 illustrates the cable axial force and grout shear force for the LBC and HBC scenarios with 0 kN, 50 kN, 200 kN, and 350 kN pretensions. As expected, higher pretension can result in higher cable axial force. Compared with the 0 kN pretension LBC scenario, whose maximum cable axial force is 25 kN, the 350 kN pre-tensioned LBC scenario generates 547.5kN axial force. The result is consistent with the previous finding reported by Jalalifar, et al., (2006), who stated that by applying high pretension tensile force would increase along the cable bolt. The position of the maximum cable axial force varies between 2 m to 4 m from the cable bolt plate (collar) into rock mass and axial force reduces to zero at the two ends of the cable bolt. A clear trend is that the maximum axial force presents at deeper positions for higher pretensions (2.5 m from the plate for 0kN pretension LBC scenario and 4.0 m from the plate for 350 kN pretension LBC scenario).

As the coupling interface between the cable bolt and rock mass, grout shear force determines how much load can be transferred from the support system to country rock. As shown in Figure 2, the maximum grout shear force is also proportional to the pretension, from only 3 kN for 0 kN pretension scenario to 36.22 kN under 350 KN pretension. The maximum grout shear force is located near the collar of the cable bolt, resonating with previous analytical and field investigation results proposed by Li (2009) and Li and Stillborg (1999), where the similar phenomenon is observed on rock bolts. From the modelling results, grout shear force reaches the maximum negative value near the plate. Then it increases along the cable bolt as moving upwards and reaches zero approximately in the middle of the cable bolt. Then the shear force inverses to positive and increases to its maximum positive value near the rear end of the cable bolt. The absolute value of shear force at the rear end (near borehole bottom) is lower than that at the shallow end (near the plate). Overall, pretension results in more tensile force along the cable bolt and shear force in the grout, providing more proactive support to the country rock.
Pretension induced bond failure

For fully bonded cable bolts, bond failure is one of the most common reasons causing cable bolt failure (Li et al., 2017). Once grout shear force exceeds its shear strength, grout material would lose cohesion and rock mass are no longer coupled with the cable bolt in the grout failure zone. Bond failure is found in 350 kN pretension HBC scenario as shown in Figure 2. Bond failure initiates from the plate to 1.41 m deep into the country rock along the cable bolt. This bond failure phenomenon is comprehensible because grout shear force is more likely to be concentrated close to the excavation boundary. Li (2009) and Li and Stillborg (1999) also found a similar phenomenon on rock bolts, based on field observations and analytical models. In addition, grout material here is defined as a brittle material, which will completely lose its cohesion soon after failure and lead to cable bolt decoupling from the country rock. Cable bolts within the grout failure zone will no longer sustain/transfer any load from country rock, and therefore the cable axial force and grout shear force are zero in that zone.

Figure 3 is presented to illustrate the relationship between bond failure and pretension, showing bond failure propagation in the HBC scenario with 250 kN, 275 kN, 300 kN and 350 kN pretensions, respectively. As mentioned above in Figure 2, bond failure starts from the excavation boundary and progresses into deep rock mass. No bond failure is observed from the rear end of the cable bolt. In addition, bond failure is presented in a continuous manner along the cable bolt, where the failure of local grout elements will induce more debonding of adjacent elements. It is worth noting that bond failure length is dependent on pretension. As the pretension increases from 275 kN to 350 kN, more bond failure initiates at the excavation boundary and propagates to the deeper level of country rock. The debonding is only 0.35 m above the excavation roof with 275 kN pretension and then rises to 0.70 m and 1.31 m above roof with 300 kN and 350 kN pretensions, respectively.

Figure 4 presents the bond failure generation along the cable bolt for all scenarios. No bond failure is observed in low in-situ stress scenarios. Another interesting observation is that, although the debonding zone is continuous along the cable bolt, the grout failure propagation is in sections, which is segmented by multiple bedding planes above the excavation (see Figure 4). The bond failure length for all five models is mainly constrained by the position of bedding planes, e.g. Bedding plane I at 0.35m, Bedding plane II at 1.32 m, Bedding plane III at 1.14 m and Bedding plane IV at 2.77 m, above the excavation roof. Grout bonding within each section can either sustain shear loading altogether or fail entirely. This explains that the bond failure length is roughly equal to the distance between the excavation boundary and bedding planes. All debonding length increases with pretension, revealing pretension can result in and accelerate debonding as discussed in Figure 3 before. The scenario with low bedding plane stiffness is more sensitive to pretension induced bond failure. For example, in the HLJ scenario, bond failure begins to initiate at 100 kN pretension, while in the HBC scenario, bond failure will not emerge until 275 kN pretension is applied.
Figure 3: Bond failure initiation and propagation along cable bolt for the HBC scenario under 250 kN, 275 kN, 300 kN and 350 kN pretensions (from the left to right).

Figure 4: The variation of bond failure length along the cable bolt by increasing pretension from 0 kN to 350 kN.

Pretension effect on bond failure

Model results in Figures 2, 3 and 4 support the argument that over-pretensioned cable bolt may induce bond failure more easily, which will be discussed more in this section. To better explain the relationship between pretension and debonding, Figure 5 is constructed illustrating the cable bolt at...
different pretension conditions: (a) Insufficient pretension applied (b) Optimal pretension applied and (c) Excessive pretension applied. As discussed in Figure 2, cable axial force and grout shear force generated along the cable bolt increase by the application of pretension. As shown in Figure 5.a, without sufficient support, g Bedding plane I opening may happen at relatively low pretension conditions (0 kN and 50 kN). Therefore, insufficient pretension is unfavourable for roadway ground support. Cable bolt at optimal pretension conditions is shown in Figure 5.b. Compared with insufficient pre-tensioned scenarios, higher force is initiated along the cable bolt and more active support can be transferred into surrounding rock mass. The Bedding plane I opening and roof sagging is thus restricted, which results in less roof deformation and better ground control. However, excessive pretension leads to bond failure, which can cause cable bolt failure and ultimately unstable ground, which is discussed in Figure 2 and 3.

The geotechnical hazards that are related to cable bolt bond failure includes roof sagging, bedding plane reopening and massive stress concentration. Bond failure will be initiated, if the grout shear force exceeds the grout shear strength. The shear force in grout interface is dependent on the grout shear stiffness and the relative displacement between the cable and rock (noted as Δdy in Figure 5.c). Since grout shear stiffness is a constant (K_{bond}) value in the numerical model, the grout shear force is totally dependent on Δdy in this study. Pretension, as axial cable force, constrains the vertical cable expansion, which is equivalent to increase the cable bolt stiffness. However, the influence of pretension on the country rock is limited to the borehole area and the overall roof strata will deform as the combination effect of in-situ stress and gravity. Therefore, a higher pretension induces higher shear relative displacement between the rock mass and cable bolt, resulting in higher shear force on the grout interface, shown as Figure 5.c, and thus more likely to induce bond failure. In order words, excessive pretension increases the cable bolt stiffness and reduces its flexibility to accommodate the large deformation of country rock, and this amplifies shear force loading on grout material and leads to bond failure eventually.

![Figure 5: A cable bolt at different pretension conditions: (a) insufficient pretension applied; (b) optimal pretension applied, and (c) excessive pretension applied.](image)

**Pretension effect on cable bolt force distribution**

In Figure 2 and 3, pretension effect on the cable bolt and grout material is summarised as: Pretension increases the axial and shear force along the cable bolt. To further explore the pretension effect on cable bolt force distribution, the neutral point theory proposed by Freeman (Freeman, 1978) based on field observations has been applied here. In this theory, the neutral point is defined at the position on the bolt system where the grout shear force is zero, the pick-up length is defined as the distance from the near end of bolt to neutral point, and the anchor length is the distance from the neutral point to the rear end of bolt in the rock mass. Figure 6 illustrates the cable axial force and grout shear force along the cable bolts and the neutral points on the cable bolts are highlighted. The neutral point can be
considered as a critical position, where the relative movement between cable bolts and rock mass is zero (grout shear force is zero). At two different sides of neutral point, the relative movement directions between cable bolt and rock mass are opposite. The rock mass in the pick-up length is supported by the cable bolt, while the rock mass in the anchor length drags the cable bolt upwards, pulling the rock mass in the pick-up length via the cable bolt. Thus, the neutral point defines the 'polarity' of the cable bolt and rock mass, whether the rock mass is unstable, requiring support from the cable bolt, or stable enough to provide additional support to other relatively unstable rock mass.

Figure 6: Axial and grout shear force distribution on the cable bolt and the neutral point movement for the HBC scenario with 0 kN, 50 kN, 200 kN and 350 kN pretensions.
Extensive studies stated that the position of the neutral point is related to the excavation geometry, rock bolt length and grout decoupling (Yu and Xian, 1983; Indraratna and Kaiser, 1990; Hyett, Moosavi and Bawden, 1996). However, results in this research show that pretension also causes the neutral point position change, moving upwards into the deep rock mass if pretension increases. From Figure 6, an upwards movement for the neutral point is observed. The neutral point is 2.49 m above the excavation under 0 kN pretension while increases to 4.49 m above the cable bolt collar when the pretension reaches 350 kN. The pick-up length increases illustrating as pretension applied, more rock mass is under the support of cable bolt, whereas anchor length decreases which may affect the support provided by the cable bolt.

CONCLUSIONS

The research aims to investigate the pretension effect on the performance of cable bolts in underground coal mines. Based on a coal mine in Western Coalfield, NSW, a numerical model is developed, considering different in-situ stress, bedding plane stiffness, claystone stiffness and UCS. The results show that pretension benefits the mechanical performance of cable bolts in underground coal mines. The cable axial and grout shear force increase with pretension, which means the cable bolt can provide more support to rock mass. Bond failure, as one of the most common cable bolt failure, is observed along cable bolt under high in-situ stress. Cable bolt within grout failure zone cannot sustain any force since it loses its cohesion with rock mass. The effect of bond failure is unfavourable for ground support. Bond failure is more likely to generate at high in-situ stress and weak bedding planes conditions if high pretension is applied. The reason is due to the relative shear displacement between the cable bolt and rock mass. Neutral points, where the grout shear force is zero, are defined to analyse force distribution along the cable bolt. The results show both pretension and bond failure triggers neutral point moving upwards, representing force redistribution along the cable bolt.

REFERENCES


ANGLED SHEAR TESTING OF 15.2 MM SEVEN WIRE CABLE BOLT

Naj Aziz¹, Sina Anzanpour¹, Saman Khaleghparat¹, Ashkan Rastegarmanesh², Ali Mirzaghorbanali², Alex Remmenikov¹, Jan Nemcik¹, Joung Oh³ and Guangyao Si³

ABSTRACT: This paper focuses on the experimental study of shear testing of 15.2 mm, 25 t capacity seven wire cables at zero, 30 and 45 degree angles using two different shear testing facilities at the University of Wollongong (UOW) and the University of Southern Queensland (USQ) in Toowoomba. A circular double shear rig MKIV was used for testing cable perpendicular to the sheared joint faces (zero angle of orientation), while testing the cable at 30 and 45 degrees was carried out using a larger size rectangular shaped rig. This study was part of the tri-universities funded ACARP project C27040 awarded jointly to the University of New South Wales, University of Wollongong and University of Southern Queensland. The objective of the experimental testing programme was to provide the essential information for the development of numerical models that included not only the technical parameters but also the behavioural outcomes from various tests with respect to the angles of testing and their effect on the nature of cable failure, be it pure shear, tensile shear or shear tensile, cable pretension and the credibility of the effectiveness of the Barrel and Wedge (B&W) anchorage system were evaluated. Laboratory facilities at both USQ and UOW were used in the study. It was found that increased angle of shear contributes to increased stiffness of the cable in shear with other parameters being equal.

INTRODUCTION

Shear testing of rock bolts and cable bolts is normally undertaken in the laboratory environment because of the difficult challenges of undertaking such study in-situ particularly in underground situations. Research into shear on cable bolts is relatively new in comparison with rock bolts. Research on cable bolting began in earnest in Canada in the 1960's and involved the use of discarded mine winder cables for ground reinforcement, particularly in metal mines. The research soon spread to North America and Europe, particularly in Sweden in the early 1970's, with the work of Bjurstrom (1974), Ludvig (1983) and Stillborg (1984). Stillborg (1984) PhD thesis from Lulea, Sweden, on shear studies used 15.2 mm and 38 mm diameter cable bolts subjected to single shear testing in 40 MPa concrete at different angles of orientation. Stillborg found that by testing a 15.2 mm cables at a 45° orientation to the shear plane the cable gave a more effective resistance to shear than cables oriented at 90° to the shear plane.

However, research on cable bolt shearing started in earnest in Australia with the work of Fuller and Cox (1978) and the design procedure for rock mass reinforcement based on the concept that reinforcement should limit displacements at structurally weak regions of the rock mass. Their theoretical approach included consideration of bolt orientation at different angles. Dight (1982) reported on shear testing of steel wire strands in the direct shear machine, initially developed by Williams (1980) but later modified to handle cable shearing under constant normal load conditions. The tested cable was first grouted in a plastic tube and the plastic sleeved cable was then grouted in a 65 mm steel tube using cementitious based grout. Shearing was carried out perpendicular to the axis of the cable.

There are two main methods of testing cable bolts for shear; (1) the British standard BS 7861-2 (BSI 2009) single shear test method and the new Megabolt Integrated Single Shear Rig Test method (MISSRT), reported by Mackenzie and King (2015) and Aziz et al (2017); (2) Double shear methods of rectangular and circular shapes, and are reported by Craig and Aziz (2010); Aziz, et al., (2015, 2016, 2019; Khaleghparast, et al (2020). Since then, the double shear system has been developed and diversified to include shearing in confined and unconfined state conditions. The system has also been

¹School of Civil, Mining and Environmental Engineering, University of Wollongong, email: naj@uow.edu.au
²School of Civil Engineering and Surveying, University of Southern Queensland, email: ali.mirzaghorbanali@usq.edu.au
³School of Mineral and Energy Resources Engineering, University of New South Wales, email: g.si@unsw.edu.au
used for shear testing of tendons and rock bolts at different angles of orientation with respect to the direction of shearing (Bjurstrom, 1974; Dight, 1982; Grasselli, 2005; Li, et al., 2018; Aziz, et al., 2015, 2016, 2017, 2019). Accordingly, this paper describes the adoption of the double shear testing method that was used initially by Grasselli to characterise the shearing behaviour of 15.2 mm cable bolts under different angles / orientation to the direction of shearing.

EXPERIMENTAL PROCEDURE

Two methods for shear testing cables were used at different angles of inclinations in this study. The choice of the method was based on the angle of the cable inclination with respect to sheared joint faces.

**Double shear testing of cable perpendicular to the joint face**

This test involved using the MK IV Double Shear Box, known as the Naj Aziz Double Shear Box. The rig is a cylindrical version of the double shear testing devices developed over the years at the University of Wollongong. In this model, efforts were made to eliminate the effect of friction between the middle block and side blocks by using the Lateral Truss System (LTS) as reported previous by Aziz, et al., (2019). In the study only one cable will be subjected to shearing perpendicular to the sheared joint faces. Further discussion on test results and analysis will be reported later in the paper. Figure 1 shows a typical view of the double shear rig post testing.

**Double shear testing of cables at inclined Angle**

The second method of testing involved the testing of cables at 30 and 45 degree inclinations. This new arrangement consisted of shear testing two 15.2 mm cables in larger size rectangular concrete blocks as shown in Figure 2, which is similar to the testing of solid rock bolts as reported by Grasselli in 2005.
Figure 3 shows the schematic drawing of the proposed angled set up for shear testing of cables at 30° and 45° respectively to joint faces in concrete blocks. The testing facilities of both The University of Wollongong and the university of Southern Queensland were utilised to undertake the ACARP funded project C57040 on “the development of modelling approach to better understand the effect of cable bolt performance on roof failure mechanisms in varying rock mass conditions”, a tri-university project between UNSW, SQU and UOW. This paper covers the experimental study of the shear testing of 15.2 mm, 25 t capacity and seven wire cable bolts which formed a part of similar tests on larger capacity bolts used in Australian mines.

![Figure 3: General view of the initially proposed assembled double shear testing concrete with the angle of cable installation at 30 and 45 degrees to joint faces](image)

**Concrete blocks construction**

*Double shear circular concrete blocks casting:* The circular concrete blocks are cast in 300 mm diameter Formatube cardboard cylinders. Two 300 mm and one 450 mm cardboard lengths are cut and assembled in a specially prepared wooden frame for a concrete pour as shown in Aziz, et al., (2019 p150). During casting of the concrete and the production of the rifled central hole for cable installation, a steel bar wrapped with 8 mm PVC tube is held vertically along the mould to precast a rifled hole through the centre of the concrete blocks. Once the concrete was poured it was left to set and harden, the steel conduit as well as the PVC tube are removed in a similar fashion as reported by Aziz et al (2017) in ACARP project report C24012. After a 28-day curing time, the concrete blocks are removed from the cardboard and mounted on the double shear circular frames for shear loading. Figure 4 shows the process of concrete casting and central hole creation.

![Figure 4: Preparation of the cylindrical concrete blocks and casting of rifled holes in concrete block using PVC flexible tubes wrapped around the central steel rod](image)
Concrete blocks construction for 30 and 40-degree angle shearing: Three water resistant, 15 mm thick marine plywood frame moulds, were constructed for casting concrete blocks for double shear testing at both 30° and 45° angles of bolt inclination. Figure 5 shows a typical set of moulds used for the preparation of blocks for the 45-degree orientation.

![Figure 5: a) structure of reinforced casting mould for 45° cable bolt inclination; (b) Concrete cast in the mould](image)

Each mould was fitted with a 12 mm steel bar cage to provide additional reinforcement to the cast concrete block. The top outer edges of the side blocks were chamfered to provide a smooth surface area for mounting the bolt collar plates bearing the load cell held in place with a Barrel and Wedge (B&W) assembly for subsequent tensioning, using a cable bolt tensioner Blue-Heeler ram. A 20 mm vertical hole was drilled on top of each side block, close to the chamfered ends, to access the inclined cast hole for grout injection and cable encapsulation. Side planks were also holed in the lower sides of the mould for the installation of bolts across both outer and centre blocks. Finally, a threaded steel bolt was fixed on top of each mould with its longer side being bent to allow for its firm casting in concrete to subsequently allow for the lifting and manoeuvring of the concrete block. These anchored steel bolts were also used for holding three blocks together in a rigid position during cable tensioning and inter-laboratory transportation.

To cast angled holes in the concrete blocks, two steel conduits, of appropriate diameter, were wrapped with an 8 mm PVC tube and inserted diagonally in the mould holes in a similar fashion to that shown in Figures 4 and 5b. Two cables were mounted in each set of the cast moulds at different lateral locations so that the space between them was maintained constant at 100 mm, thus leaving the cable anchored at 150 mm from concrete sides (Figure 3). The location of the cast holes in the concrete blocks varied depending on the bolt inclination / orientation with respect to the sheared joint faces. Depending on the inclination of the hole, the lower end of the installed cable, with its B&W, would protrude from beneath the central concrete block, in the case where the hole inclination was 45-degree. For 30 degree encapsulated inclination, the cable end with its B&W will end up housed on the side socket of the centre block. A 3D printer was used to print plastic cups for the purpose of creating side socket holes in the cast.

**Assembling and grouting**

The following steps were followed in the preparation and assembling of the concrete blocks:

- Mounting of self-adhesive packers at the mouth of the side holes between the joint planes to minimise or eliminate grout leakages from the concrete blocks.
- Mounting two frictionless 100 mm wide Teflon strips vertically on the joints contact face in the double shear assembly to reduce the effect of friction during the shearing process.
- Drilling small 20 mm diameter holes close to the chamfered side of the side blocks and close to the mouth of the hole for grout pouring to encapsulate the cable in the hole.
- Mounting three concrete blocks on wooden blocks and butting them together. These blocks were then held together and laterally reinforced by three sets of Lateral Truss Systems (LTS). Each set of the reinforcement truss system consisted of a 9 mm thick, 100 mm x 60 mm
rectangular steel tube butted on either side to 10 mm thick 90 mm x 150mm open steel channels. The real aim of the LTS was to counteract the forces generated by cables being pretensioned and the subsequent double shear loading of the central block. This applied counter force would resist the applied pretension forces in squeezing the central block thus minimising the influence of contact friction between joint faces.

- Also, the LTS, shown in Figure 6, (a) was to prevent individual blocks from twisting laterally and (b) to prevent the central block from lifting upwards. The later point (b) proved difficult to prevent at high pretension loads; however, the central block uplift during cable pretensioning up to 50 kN was possible with the addition of a 10 mm thick and 150 mm wide open steel channel (green) placed over the three blocks during cables pretensioning and grouting stage.

- Inserting the bolt in each hole to be fitted on the lower side with a small load bearing plate, then secured by a B&W. The cable was held firmly from the cable top-end in order to hold the B&W in place at the lower end. This protruding topside cable end was then fitted with a steel plate and a 75 t capacity load cell to be followed with a B&W. The whole system was then gently pretensioned. The same procedure was used in the installation of the second cable on the opposite side of the three blocks. During shearing each cable was then pretensioned separately using two identical Blue Heeler tensioners simultaneously to avoid blocks twisting.

- Injecting grout (Stratabinder HS grout) from the top short holes to grout cable bolts down in to each instrumented cable bolt hole. Any leakage from the packers was checked and the cable tightening level was maintained for the duration of the grout hardening.

**Figure 6: (a) Schematic drawing of assembled DS box with constraining mechanisms and (b) readily assembled rig of the large double shear box**

**TESTING PROCESS**

Table 1 shows the results of the shear testing of the 15.2 mm diameter cable bolt under different test environments. A total of six tests were carried out on the cable. One test was made using the standard MK-IV rig, one test was at a 30 -angle and the other four tests were carried out on cable bolts with an orientation of 45-degree.

**Double shear testing of 7 wire 15.2 mm cable bolt at 0 – degree angle**

Figure 7 shows the circular double shear rig for testing cable at zero-degree orientation and Figure 8 shows the load-displacement profile of the tested cable, together with the axial load generated during the shearing process. The shear load per sheared joint plane was 157.0 kN (314 kN = 157 kN per sheared side). The axial load at peak load was 53 kN. Further comparative analysis of this test with other angled shear cables will be addressed in the discussion section.
Figure 7: Circular Double shear MK-IV testing rigs for testing 15.2 mm cable bolt at 0° of inclination

Figure 8: Load displacement profile of the tested 15.2 mm cable bolt cable, together with the axial load generated during the shearing process. Tests were made in the MK IV double testing rig

Double shear testing of 15.2 mm cable bolts at 30° inclination

Figure 9 shows the schematic drawing of the cable installation at 30-degree angle of installation in DS blocks, which is further elaborated as shown earlier in Figure 3. The 30° inclination arrangement is different from 45°, where B&W anchors do not reach the bottom of the central block.

Figure 9: Installation of a cable in two blocks of the three block assembly. Only one side is shown. Note Side socket for B&W housing and Teflon strips
The peak shear load at cable failure of 508 kN occurred at a shear displacement of 30 mm. During the early stage of the system loading, the compression machine malfunctioned for the first few minutes. The fluctuation in axial load-displacement during this early stage of shearing, shown particularly in the axial load-displacement profiles shown in Figure 10 was stabilised and the testing continued uninterrupted until reaching the cable peak load of 508 kN. Figure 10B shows the view of the pattern of cable strand wire failures.

Figure 10: (A) Load - displacement profiles of testing 15.2 mm Cable bolt at 30°. (B) Pattern of strand wires failure

Double shear testing of 7 wires 15.2 mm cable bolt at 45-degree angle

Four tests were carried out at a 45° inclination as shown in Table 1. The first test was carried out in the 60 t capacity compression loading frame at the UOW laboratory. Figure 11 shows the assembled 40 MPa concrete blocks. Both cables were pretensioned at 20 kN. The lower level for the initial cable pretension load was necessary because the pretensioned force of the cable bolts was not in line with the counteracting opposite load generated by the LTS force, thus pulling the central block upwards slightly and causing it to shift. The lateral truss system consisted of three unit sets of 9 mm thick closed box rectangular steel channels (Blue colour side straps) bolted on each end to 9 mm thick and 150 mm wide open steel channels.

Once pretensioned the double shear block set up assembly, weighing around 1.4 t was mounted on the compression machine loading plates with its outer concrete blocks resting on H-shaped steel channels, leaving the middle block free to be sheared down. During the initial phase of the first load application on to the central block, the 300 mm², 25 mm thick load steel bearing plate placed beneath the ram over the central concrete block caused a slight crushing of its top surface (Figure 11a). This was soon overcome by pouring a layer of plaster of Paris gypsum solution over the crushed surface, trowelled smooth, followed by placing a 30 mm thick steel plate large enough to cover the whole surface area of the middle concrete (Figure 11b), thus allowing the applied load to be distributed evenly on the top side of the centre block. Information retrieved during the loading process from both the vertical ram and from load cells on the angled bolts were processed and displayed on a PC monitor screen. The loading rate was maintained at 1 mm per minute. Figure 12 shows the load-displacement graphs of the applied load and axial forces generated for both bolts. The axial load variation may have been caused by the initial crushing of the centre block at surface.

Also, the increased overall shear load spent in overcoming friction between joint faces because of the fact that the outer concrete blocks were not restrained from lateral or sideways movement during shearing as they were not bolted to the side open channel steel plates of the LTS. This may have
caused the concrete joint faces to come into forced contact with each other during the shearing process. This will be further evaluated later in the discussion section.

Figure 11: 45 mm diameter cables encapsulated at 45 degree angle being ready for shear testing

![Figure 11](image1.jpg)

**Figure 12: Load - displacement of 15.2 mm, 7 wire cable**

![Figure 12](image2.jpg)

**Table 1: Shear test results of 15.2 mm cables**

<table>
<thead>
<tr>
<th>Test</th>
<th>Angle (degree)</th>
<th>Test date</th>
<th>Cable axial pretension(kN)</th>
<th>Shear load (kN)</th>
<th>Shear displacement (mm)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>20/08/2019</td>
<td>50</td>
<td>314 (157/side)</td>
<td>52</td>
<td>One cable used for double shear test, hence the peak load pre joint face = 157 kN</td>
</tr>
<tr>
<td>2</td>
<td>30</td>
<td>17/07/2019</td>
<td>50</td>
<td>508</td>
<td>25</td>
<td>Cable wires failure mostly in tension with cone and cup ends</td>
</tr>
<tr>
<td>3</td>
<td>45</td>
<td>3/9/2018</td>
<td>50</td>
<td>572</td>
<td>21</td>
<td>Centre block top surface damaged during early shear loading stage. The first test in the angle shear programme did not involve anchoring/bolting outer concrete blocks to the LTS</td>
</tr>
<tr>
<td>4</td>
<td>45</td>
<td>09/05/2019</td>
<td>0</td>
<td>581</td>
<td>39</td>
<td>Grout leakage during cable encapsulation and loss of pretension load during transportation from Wollongong to Toowoomba, Qld</td>
</tr>
<tr>
<td>5</td>
<td>45</td>
<td>09/09/2020</td>
<td>50</td>
<td>501</td>
<td>28</td>
<td>Cable wire failure mostly in tension</td>
</tr>
<tr>
<td>6</td>
<td>45</td>
<td>20/09/2020</td>
<td>40</td>
<td>485</td>
<td>24</td>
<td>Cable wire failure mostly in tension</td>
</tr>
</tbody>
</table>
DISCUSSION

Figure 13 shows the load-displacement graphs of all 15.2 mm cable bolts tested at angles zero degree, 30° and 45°. The effect of the cable’s inclination and load transfer mechanisms in all six tests are clearly evident. Only one test out of the six 15.2 mm cable type tested (four at 45°, one at 30° and one at zero degree) was carried out at the USQ Toowoomba laboratory, others were carried out at the UOW laboratory. Test results revealed that the higher the bolt angle of inclination / orientation to the sheared joint plane the greater the ultimate failure load of the cable bolt. In other words, the ultimate breaking load of the cable bolt increases with the increase in the angle of inclination / orientation. This was expected, as the greater the angle of inclination with respect to the direction of shearing force, the less likely all cable wires fail in shear, as the applied shearing force tends to pull the cable wires more axially rather than in shear. With the exception of the first test in which the top of the middle concrete block was damaged slightly, none of the blocks were found to end up cracked or split. In many cases both cables snapped fully with excessive vertical movement of the central block as shown in Figure 14. Once the cable was snapped the loading process was terminated. It should be stated that during the double shear testing of a cable, where the tested cable orientation was perpendicular to the applied shear load or the tested cable axis was laid horizontally (0° orientation), the failure load per side was 157.0 kN (15.7 t).

The shear failure load per cable in the first test at 45-degree inclination was in the order of 286 kN (total load for both sides 572 kN). This level of spent force was greater than the tensile failure load of the cable of 250 kN (25 t). This increase in loading force is likely to be attributed to; (a) the inefficiency of the two 100 mm wide single sheet Teflon strips instead of double sheets, sandwiched between joint faces; (b) that the lateral truss reaction force was not in line with the applied encapsulated cables force axis against the vertical shearing of the centre block, and (c) the additional force needed to overcome the butted joints face friction. However, this additional lateral friction force between the concrete sides may not reach fully the estimated 30% of the applied shear force, based on the Mohr-Coulomb Fourier series mathematical model as report by Aziz, et al., (2016) and by dynamic testing reported by Khaleghparast, et al., (2020), but will be of a significant amount proportional to the size of Teflon strips.

**Figure 13: Load-displacement of profiles of testing 15.2 mm diameter cable bolts at different orientations (angles)**

It is worth noting that similar tests were reported in Stillborg’s PhD thesis where shear testing of 15.2 mm cable bolts at a 45-degree inclination in a single shear test and without sheared joint faces coming in contact with each other were undertaken. The average shear value reported by Stillborg was 206
kN (20.6 t). The sheared cable was encapsulated in 600 mm$^3$ granite rock (UCS of 234 MPa) embedded in a 60 MPa concrete (Stillborg, 1984).

The failure pattern of various failed wires in the 15.2 mm cable testing was mostly in tensile/shear as shown in Figure 15. At 45-degree of shear testing in Toowoomba all wires were snapped from both cables. All wire failures in both cable strands are clearly from the result of the excessive level of centre block downward travel during the shearing process, as shown in Figure 14. The pattern of observed broken wires was typically in tensile shear as shown in Figure 15.

Figure 14: Centre block shear displacement in which booth cable have failed symmetrically. Note the snapped and smeared leaked grout between concrete joint faces

Test 4
Test 5
Test 6

Figure 15: Various snapped view of 15.2 mm cables tested for shear at 45 degree inclination

Comparing all six test results at different orientations, shown in Figure 13, the following was inferred:

- The shear displacement, at peak shear load failure, increases with reduced angle of orientation,
- In shear testing of the cable at a zero angle of inclination, the shearing was carried out on a single cable, not two as would be the case with angled shearing. The shear load displacement is higher because of the nature of double shear testing with short side concrete blocks, and the fact that the cable will undergo elongation at each hinge point in the vicinity of both joint planes. Recent research by Aziz, et al (2020) has found that the level of displacement is less with internally reinforced concrete medium and with fully secured ends.
• The blue load-displacement graph (4) shown in Figure 13 of the 15.2 mm diameter cable bolts tested in Toowoomba (SQU) at 45° orientation indicated that the first of the two peak loads was due to the forces required to separate cemented joint faces from each other, caused by the leaked grout from the seal packers sandwiched between the sides of three blocks during cable grouting process as shown in Figure 14 in which the leaked grout smeared across joint faces. The third peak shear load is attributed to the snapping of the cable at 571.6 kN.

• The brown graph (3) is the first test carried out at the UOW at 45-degree inclination. It has a peak failure load of 572 kN, which is almost identical to that obtained and shown in the blue graph. This near equal shear failure load demonstrates the consistency of the cable shear value at 45°.

• The purple graph (3) is shear values at 30° cable orientation carried out at the UOW,

• The orange graph (4) is obtained from the standard double shear testing using the Naj Aziz circular double shear rig with the direction of shearing perpendicular to the cable axis ‘zero angle’ as shown in Figure 7.

• The red (5) and green (6) graphs are additional graphs at 45 degree tested lastly at the UOW.

• In general, the increased angle of orientation at 30-degree and 45-degree -caused cable bolts to fail in tensile shear as shown in Figure 15, which is evident from the snapped failed cable strand wires.

• The peak shear load failure of the majority of 15.2 mm cables at 45-degree inclination has demonstrated the consistency and reliability of the test results which adds to the credibility of the experimental study undertaken.

CONCLUSIONS

The behaviour 15.2 mm diameter cable bolts subjected to double shear loading at different angles are examined and evaluated. Comparison of test results demonstrated the influence of the cable installation angle with respect to the direction of shearing. Cable tensioning was difficult to evaluate with the existing LTS arrangement because of the unforeseen consequences of the concrete blocks upward movement during excessive pretension loading and the whole concrete block system failure. Accordingly, the pretension loads were kept to around 50 kN.

From the study it was found that the higher the bolt angle of inclination to the sheared joint plane the greater was the ultimate failure load of the cable bolt. This was expected, as the greater the angle of inclination with respect to the direction of shearing force, the less likely all cable wires fail in shear.

ACKNOWLEDGEMENTS

The experimental work was carried out at both the University of Wollongong and the University of Southern Queensland -Toowoomba campus, because of the availability of different loading machines and technical support. The technical support provided by both institutions were very valuable and the authors record their appreciation for the support given by various technical staff in both institutions as listed in the report.

The inkind support of Jennmar Australia, Megabolt and Minova Australia is very much appreciated. All three companies have been very generous in providing material as well as technical support whenever needed.

This paper was part of the tri-university ACARP funded project C57040 on "the effect of cable bolt performance on roof failure mechanisms in varying rock mass conditions", awarded to UNSW, USQ and UOW. Both Brian McCowan and Roger Byrne were keenly instrumental in promoting this research study to a successful end for the benefit of the effective mine ground control and operator safety.

REFERENCES


Stillborg, B., 1984. Experimental investigation of steel cables for rock reinforcement in hard rock, PhD thesis Lulea University Sweden,
RESPIRABLE COAL DUST AND SILICA EXPOSURE STANDARDS IN COAL MINING: SCIENCE OR BLACK MAGIC?

Nikky LaBranche¹, David Cliff², Kelly Johnstone³ and Carmel Bofinger⁴

ABSTRACT: The re-identification of mine dust lung diseases in coal mine workers has prompted much work to be done to improve exposure monitoring and health surveillance in Australia. It is now recognised to be inadequate to talk about respirable dust in general terms, because size, shape and chemical content can affect the adverse consequences of excessive exposure. The Minerals Industry Safety and Health Centre within the Sustainable Minerals Institute undertook a gap analysis to identify needs for further research into respirable dust exposure monitoring and control. One such project is characterising the dust present in different mining atmospheres to understand the contribution of the chemical components, particle sizes and shape to the incidence of mine dust lung diseases in Australian mine workers.

Exposure standards were first set in British Coal and have subsequently been adopted by the US and Australia. This paper starts with a discussion of the British Pneumoconiosis Field Research data collection methodology and assumptions on which those initial standards were based. From there the discussion moves into the application of these standards to the US and Australia, the history of revisions to the exposure standard and limitations in sampling equipment and radiological diagnosis of disease.

INTRODUCTION

This paper will discuss the history of the Workplace Exposure Standards (WES) for Respirable Coal Dust (RCD) and Respirable Crystalline Silica (RCS) in coal mining. The role of silica in the diagnosis of mine dust lung diseases, the limitations of radiological diagnosis and difference in monitoring equipment are also addressed. British Coal set initial standards with the British Pneumoconiosis Field Research (BPFR), which were then modified by the United States. Australian Standards are often based on US standards. Safe Work Australia recommended the same limits for RCS in 2019 that the National Institute for Occupational Safety and Health (NIOSH) and the American Conference of Governmental Industrial Hygienists (ACGIH) had proposed in 1974 and 1983, respectively (NIOSH, 1974, ACGIH, 2010). Safe Work Australia recommended the same limits for RCD in 2019 that NIOSH and ACGIH proposed in 1995 and 1996, respectively (ACGIH, 2001, NIOSH, 1995b).

While the exposure limit for RCD and RCS have gradually decreased, there continue to be incidences of mine dust lung disease (MDLD) occurring globally. For example, the number of miners being diagnosed with MDLDs is increasing in Central Appalachia in the USA (Blackley et al., 2018). Is simply continuing to lower the exposure standard enough? Or is there more to the exposure than meets the eye?

COAL MINE DUST COMPOSITION

Coal mine dust may contain a complex mixture of over 50 different elements and their oxides (IARC Working Group on the Evaluation of Carcinogenic Risk to Humans, 1997). Dust characteristics can vary widely between and even within mines and what is present in the respirable fraction may differ from the overall seam composition (Sarver et al., 2019). In a systematic review of the relationship

---
¹ Research Manager-OHS, Minerals Industry Safety and Health Centre, Sustainable Minerals Institute, The University of Queensland, Email: n.labranche@uq.edu.au, Tel: +61 4 0761 0108
² Professional Research Fellow, Minerals Industry Safety and Health Centre, Sustainable Minerals Institute, The University of Queensland, Email: d.cliff@uq.edu.au, Tel: +61 7 3346 4086
³ Senior Lecturer, School of Earth and Environmental Science, The University of Queensland, Email: k.johnstone2@uq.edu.au, Tel: +61 7 3346 7816
⁴ Principal Research Fellow, Minerals Industry Safety and Health Centre, Sustainable Minerals Institute, The University of Queensland, Email: c.bofinger@uq.edu.au, Tel: +61 7 3346 4082
between ‘pure coal’ (non-quartz) and interstitial lung disease, Beer et al., (2016) did not find any studies addressing the pure carbon part of coal dust.

Bennett et al., (1979) found a progressive and five-fold increase in the incidence of Coal Workers Pneumoconiosis (CWP) from UK collieries mining low rank coal to those mining high rank coal (Bennett et al., 1979). Beer et al., identified nine papers which evaluated the ‘pure coal effect’ which supported an independent effect of non-quartz coal dust on the development of Interstitial Lung Diseases. However, further evidence is needed to prove this theory due to methodological limitations of the existing evidence (Beer et al., 2016). Beer concluded “While the association between coal mine dust exposure and lung disease has been investigated for decades it is still not clear what components of the coal dust are actually responsible for disease development.” (Beer et al., 2016).

Correlations have been made between disease and the surrounding rock. However, minimal research has been carried out to measure the health effects of various concentrations of the other components of dust even though they may be more prevalent than coal dust in the respirable fraction. These other components may include calcite, muscovite, calcium silicate, kaolinite, apatite, chlorite, orthoclase, plagioclase, and amphiboles.

A study by Stocks (1962) of disease among coal miners in the UK made a connection between the surrounding soil and rock exposure and stomach cancer “...where mortality from stomach cancer is peculiarly high, farmers, quarry workers in slate and igneous rock and coal miners all showed pronounced excess in age adjusted death rates from stomach cancer compared with men in other occupations, and this suggested that direct contact with soil in areas with high mortality may be a factor of importance. It may be therefore that the notably large mortality excess in the South Wales miners is connected with the kind of rock and soil rather than, or in addition to, the kind of coal.” (Stocks, 1962). This study is included in the ACGIH document setting the WES for RCD (ACGIH, 2001).

**HISTORY OF OCCUPATIONAL DISEASE IN MINING**

Occupational diseases, including mine dust lung disease, have been recognised in mining for centuries. Agricola wrote in his posthumously published De Re Metallica circa 1556: “It remains for me to speak of the ailments and accidents of miners, and the methods by which they can guard against these, for we should always devote more care to maintaining our health, that we may freely perform our bodily functions, than to make profits. Of the illnesses, some affect the joints, others attack the lungs, some the eyes, and finally some are fatal to men.” (Agricola, 1556). The next few sections will step through the history of the RCD WES and to some extent the RCS WES.

**Early Identification in the UK**

The setting of a WES for coal dust began in the UK. In 1831, Dr James Craufurd Gregory first described black pigmentation and disease in the lungs of a deceased coal miner. He linked this to pulmonary accumulation of coal mine dust. Gregory hypothesised that the black material seen at autopsy in the collier’s lungs was inhaled coal dust and this was confirmed by chemical analysis carried out by Professor Sir Robert Christison (Donaldson et al., 2017).

Gregory suggested that coal dust was the cause of the disease and warned physicians in mining areas to be vigilant for the disease. This first description of what came to be known as ‘coal worker’s pneumoconiosis’ sparked a remarkable intellectual effort by physicians in Scotland, culminating in a large body of published work that led to the first understandings of this disease and its link to coal-blackened lungs (Donaldson et al., 2017). Pneumoconiosis in British coal miners had been identified before 1930, but relied upon miners voluntarily coming forward for medical examination (Fay and Rae, 1959).

**British Pneumoconiosis Field Research**

The Pneumoconiosis Research Unit (PRU) was established in 1945 and by 1952 researchers had determined that coal workers’ pneumoconiosis could be divided into simple pneumoconiosis and complicated pneumoconiosis (or Progressive Massive Fibrosis (PMF)). The dust alone was considered to be the cause of simple pneumoconiosis while complicated pneumoconiosis was thought to be caused by the addition of an infection, probably tuberculous onto lungs already affected by coal dust.
The Pneumoconiosis Field Research Unit Interim Standards Study (ISS) was established in 1952 due to lingering uncertainties of the true prevalence of pneumoconiosis throughout the UK as well as the attack and progression rates of the disease under various conditions. In this study, the National Coal Board (NCB) aimed to find a ‘safe’ exposure standard in terms of dust quantity and quality plus the relationship between the disease and respiratory disability.

Detailed research was performed across the UK including both medical studies and environmental studies, which were then assembled and analysed together. In the medical studies, chest radiographs were taken of all miners who volunteered to participate at 24 test collieries.

To measure the dust in the environment a thermal precipitator was used to collect area samples of 1-5 micron diameter dust in the vicinity of the worker. The particles were then counted by hand using a microscope. It later became evident that this method significantly underestimated the count due to particle overlap and the data had to be adjusted after scientific experiments were carried out to estimate the required correction (Attfield and Kuempel, 2003). It was not possible to take an exposure sample of each worker due to the size of equipment and the fact there were 35,000 men participating in the study.

The population was divided into work groups, commonly referred to as Similar Exposure Groups (SEGs) in Australia. Samples were collected from each working group which reduced the stratification to 1,500 SEGs. A representative from each group was selected at random to have their working area monitored over a shift. Over 60,000 samples were collected across 14,000 shifts by September 1958 (Fay and Rae, 1959). Collieries included in this study were specifically chosen as examples of all of the major variations in mining conditions in Britain.

Occupational hygienists at mines assessed the dust exposures of all SEGs. Here dust concentrations were measured gravimetrically by placing samplers close to the participating men throughout their working shifts. From these samples a gravimetric area concentration was determined for all occupational groups and five year inter-medical-survey periods. These gravimetric area samples are in contrast to the particle counts performed by the standard thermal precipitator used in the first phase of the research. A series of side by side instrument comparisons were taken to relate the particle count to the gravimetric mass, and the earlier measurements were re-expressed in equivalent gravimetric terms (Hurley et al., 1987).

No direct monitoring (referred to as personal or occupational monitoring) of the workers exposure to dust concentrations took place either before the research began or subsequent to the study at non-research collieries. For those men not at research collieries, six categories of coal mining activity were used to assume the average concentrations to which they were exposed. It was assumed that dust concentrations in other mines in the region were similar to those at the research pit in the corresponding period. It was also assumed that concentrations before the research were similar to those experienced during the first 10 years of monitoring.

With the data collected and the numerous assumptions previously discussed, a series of curves were developed. They estimated the probabilities of radiological changes (CWP and or PMF) over a mines working tenure, for various combinations of cumulative dust exposure, age and carbon content of the coal, using logistic regression methods.

Jacobsen et al.’s 1971 curve, the heavy dashed line in Figure , modelled the probability of developing Category 2 simple CWP or greater over a 35 year working tenure to average dust concentrations. This curve was thought to give the incidence of Category 2 simple CWP or greater that would result from a certain level of dust exposure in a population of workers (Jacobsen et al., 1971). The Jacobsen 1971 curve was extrapolated using data from the 10-year ISS, which indicated that a miner would never be at high risk of developing PMF at an average exposure of 2mg/m³ or below, over their working life. Following this model, the focus of CWP control lay primarily in the simple reduction of respirable coalmine dust exposure levels. This curve supported the results of other research at that time, which indicated that PMF was very unlikely to develop from cases below Category 2 CWP (Cochrane et al., 1961, McLintock et al., 1970). However, it was later identified that contrary to Cochrane’s findings, PMF could develop from Category 0 or 1 CWP (Hurley et al., 1987, Hurley and Maclaren, 1988, Maclaren et al., 1989, Shennan et al., 1981).
The Hurley and MacLaren (1987) curve, the dash dot line labelled as “this report” in Figure 1 indicates the percentage of miners predicted to have Category 2 or greater simple CWP after 35 years of work at 1740 hours per year of mining at dust levels ranging from 1-8 mg/m\(^3\) at 86.2% carbon (Hurley and MacLaren, 1987). Dust concentrations were monitored close to the men throughout the working shift. Side-by-side comparisons were conducted to convert particle counts to gravimetric units and dust concentrations at other mines were assumed to be similar to those at the research pits where the measurements were taken (Hurley and MacLaren, 1987).

The Jacobsen et al., (1971) and Hurley and MacLaren (1987) curves shown in Figure 1 were an average over a number of collieries. When the data is broken out into the individual mines as seen with the solid lines from Hurley et al., (1982), the slope of curve for the individual mines themselves varied and the average does not represent all cases. The results were based on the same set of data, however the Hurley and MacLaren data includes percent carbon as a predictor variable. Hurley et al., (1982) shows the comparison of the mean dust concentrations at 10 British coalmines to the probability of developing CWP 2/1+ over 35 years. He found that for eight of the 10 collieries, there was minimal difference in risk associated with working in similar dust conditions for the same length of time. However, for the other two collieries, the results differed sharply from this pattern. It was thought that mineralogical characteristics of the coal may be influencing the dust-pneumoconiosis relationship. The coal rank indices did not explain the extreme variation, nor did the Quartz exposure explain the difference as Colliery Q had a quartz content of 6.4% while Colliery T was 5.0% (Hurley et al., 1982, Hurley and MacLaren, 1987).

It was clear to Hurley et al., (1982) that the risks are far higher or lower than the average values at some collieries. This implies that the “probability estimates will not necessarily reflect the risks to coalminers generally unless the dust concentrations and the sources of unexplained colliery-related variability occur in a pattern broadly similar to that observed in this study (Hurley et al., 1982).

While coals may be similar, mining methods may present important differences in the dust exposures. In 1992, Attfield stated that “because the various methods may give rise to different particle-size distributions and involve cutting into roof, floor, or dirt band rock to differing degrees, the resulting dust clouds may vary considerably in type and composition and thus may have differing fibrogenic potential.” (Attfield, 1992). In summary, the British system was based on particle counts a number of
research collieries which were later correlated to gravimetric sample. It was at first thought that PMF only resulted from Category 2+ CWP, which was not possible at an exposure of 2mg/m³ or below over a working life.

US Workplace Exposure Standard History

The United States Mine Safety and Health Administration (MSHA) changed the coal dust exposure limit from 3.0 to 2.0 mg/m³ in 1972. And then on 1 August 2016, the concentration limit of respirable coal dust reduced from 2.0 mg/m³ to 1.5 mg/m³. The 2 mg/m³ exposure standard, which came into effect in the US in December 1972, was based on the UK data from the Pneumoconiosis Field Research study. Attfield (1992) commented on the validity of extrapolating the results from previous studies of British mines to the US situation stating it may not be possible “given that such an evaluation would require knowledge that is now unavailable (such as that particle-size distributions or composition for mines that are now closed)” (Attfield, 1992). He also noted that the British studies were based on x-ray readings from international classification standards for pneumoconiosis that were no longer current (Attfield, 1992).

The US has well documented regional variations in the prevalence of coal worker’s pneumoconiosis. In Figure 2, Graph A shows the prevalence of CWP for all of the US, while Graph B breaks out central Appalachia into its own category. In this instance, central Appalachia is defined as Kentucky, Virginia and West Virginia. As can be seen, the rates of CWP have been increasing in central Appalachia. The third graph shows the US without the effects of the central Appalachia region where the increase is only slight for this cohort. The CWP prevalence is four times higher for Central Appalachia underground miners than it is for long tenured underground miners elsewhere in the US. One in 20 long tenured miners in central Appalachia has CWP that has progressed to PMF. This is even with the 1.5 mg/m³ exposure standard and the use of continuous personal dust monitoring technology underground (Blackley et al., 2018).

![Figure 2: Prevalence of coal workers pneumoconiosis in the US by region](image)

NIOSH recommended a WES of 1.0 mg/m³ in 1995 (equivalent to 0.9 mg/m³ measured according to ISO/CEN/ACGIH criteria) (NIOSH, 1995a). There was pushback from industry and MSHA ultimately adopted a WES for RCD of 1.5 mg/m³. The US measures quartz content as opposed to RCS. These samples are collected gravimetrically and a reduction factor is applied to the RCD limit measured via real-time monitoring, if necessary. In summary, the US adopted the UK exposure standard without redoing the epidemiology to account for the different coal geology and mining conditions, but then revised the standard downward given the high incidence of CWP.

Australian Workplace Exposure Standard History

The two major coal mining states, Queensland (QLD) and New South Wales (NSW) both adopted the 3.0 mg/m³ standard from the US. Australian coal mining legislation is state based. Each state therefore has their own legislation and systems for setting and monitoring the RCD WES. In 2004, the pump flow rate changed to 2.2 L/min in order for the cyclone elutriators to better conform to the ISO
curve. At that point, NSW lowered the WES to 2.5 mg/m^3. QLD remained at 3.0 mg/m^3 with shift adjustment.

Queensland reduced the RCD WES from 3.0 mg/m^3 to 2.5 mg/m^3 on 1 November 2018 and then applied a further reduction to 1.5 mg/m^3, which commenced 1 September 2020 (Queensland Government, 2018). Queensland also reduced the WES for RCS to 0.5 mg/m^3 on 1 September 2020 (Queensland Government, 2018). In QLD, only about a third of the MDL cases being diagnosed are CWP and a growing number are silicosis and COPD (Queensland Government, 2020). NSW reduced the RCS WES from 1.0 mg/m^3 to 0.5 mg/m^3 on 1 July 2020. The reduction of the RCD exposure standard from 2.5 mg/m^3 to 1.5 mg/m^3 will take place on 1 February 2021.

Silicosis in Coal Miners

The exposure limit for silica is independent of the limit for respirable dust and much of the literature used to set the ACGIH TLV-TWA is for non-coal mining applications (ACGIH, 2010). The ACGIH recommends a WES of 0.025 mg/m^3 to protect against silicosis and lung cancer. There were no studies referenced confirming a protective effect at 0.025, rather studies were cited indicating that 0.05 mg/m^3 “would probably not be sufficiently protective of workers’ health” (ACGIH, 2010). This recommendation comes from the findings of several epidemiological studies that a WES of 0.05 mg/m^3 has not shown a change in longevity or lung function even though a percentage were found to have 1/0 or 1/1 ILO profusion rating (ACGIH, 2010). The risk of silicosis and lung cancer was found to significantly increase at levels greater than 0.06 and 0.65 mg/m^3 (Graham et al., 2004, Steenland and Sanderson, 2001). There is also evidence that Silicosis can progress even after miners leave the industry and the exposure to silica dust has ceased (Hnizdo and Sluis-Cremer, 1993).

For the UK data, in the Hurley et al., (1982) study, silica did not explain the variation in predicted incidence of CWP by colliery. However, there was evidence that some miners show unusual radiological changes when exposed to coal mine dust with a relatively high quartz content (Hurley et al., 1982). Hurley hypothesised that a slight overall quartz effect may remain hidden as a miners estimated lifetime exposure to quartz is less accurate than his corresponding mixed dust exposure estimate (Hurley et al., 1982).

The US has also found similar confounding factors between coal dust and silica. The comparison between countries did not take into account the differences in methodology used to estimate quartz levels (Attfield, 1992). Since 1980, the prevalence of r-type opacities (associated with silica) have increased sixfold among underground coal miners in Central Appalachia, while remaining static for the rest of the US. A 2016 case study in the US found that miners with rapidly progressive pneumoconiosis had lung pathology consistent with accelerated silicosis, mixed dust pneumoconiosis and these miners were exposed to silica and silicate minerals contained in respirable coal dust during their mining careers. Jobs associated with higher exposures to silica more frequently have severe and rapidly progressing disease than jobs associated with low silica exposure (Hall et al., 2019).

Residence time of the dust in the lungs may also be a factor for disease progression (Hurley et al., 1982, Graham et al., 2004). The ACGIH TLV documentation cites the 2004 work of Graham et al., on granite miners that found that when retirees whose RCS workplace exposure concentrations average 0.06 mg/m^3 were studied, the risk of silicosis was significantly greater (7.1% versus 1.2%) when compared to employees examined at or before retirement. (Graham et al., 2004).

Hnizdo and colleagues (1993) investigated a cohort where 313 South African miners developed 1/1 silicosis at an average age of 55.9 years. They found that in 57% of the silicotics studied, the radiological signs developed on average 7.4 years after mining exposure ceased. This means the risk of silicosis was strongly dose-dependent while the latency period was largely independent of the cumulative dust exposure (Hnizdo and Sluis-Cremer, 1993).

**RADIOLOGICAL DIAGNOSIS OF CWP AND OTHER MDLDS**

Diagnosis of CWP has historically been based on chest radiographs, also known as x-rays, which add an additional set of confounding factors to the estimation of the exposure standard. The historically reported CWP levels may be an underrepresentation of actual disease prevalence for several reasons. In the UK, ISS miners needed to have been working at the colliery for 10 years (Hurley et al., 1982). It
is likely that miners may be self-selecting themselves out of the cohort if they start to experience symptoms before this threshold.

It is unknown what the false negative rate was for CWP in chest radiographs, so there may be cases that exist that were not diagnosed by x-ray. Most surveillance programs stop when a miner retires or leaves the industry. The few studies that have been performed on retired miners have found that MDLs, especially silicosis, can continue to develop after exposure has stopped. Baseline prevalence also increases with age. While age was included in some UK analysis, the model chosen does not allow for this variable (Attfield, 1992).

Interpretation of x-rays also differed between the UK and US. US readers tended to report more abnormalities that past UK readers. Based on this trend, the CWP estimates for the US would be greater than those of the UK for the same expected exposure curve. The classification standards for x-rays changed over time and were different during the ISS than they were for more recent US studies (Attfield, 1992).

Comparisons of X-rays to CT scans show that x-rays tend to significantly underestimate disease. In a study by Remy-Jardin 1990, 48 patients were diagnosed as Category 0 by radiograph, but CT revealed that only 36 were in fact Category 0, while seven were Category 1 and five were Category 2. For the 65 patients diagnosed at Category 1 by radiograph, 31 patients were Category 0, 29 maintained their Category 1 status and five were Category 2 (Remy-Jardin et al., 1990).

Hnizdo compared x-ray diagnosis of silicosis to autopsies in 984 miners and found that where silicosis was positively identified in the autopsy it was not diagnosed by the most accurate x-ray reader in 75%, 54% and 26% of cases of slight, moderate and marked silicosis cases, respectively. The 326 cases diagnosed/confirmed at autopsy averaged of 63% of silicotics not diagnosed as positive using x-ray analysis alone with a 1/1 cut-off point (ACGIH, 2010, Hnizdo and Sluis-Cremer, 1993).

Past studies have predominately only looked at cumulative dust in coal mines and not addressed the pure carbon part or other components of coal dust, with a small exception for bioavailable iron (Huang et al., 2005). In many cases, the exposure to silica dust was not treated separately to the exposure to coal dust. There are most likely other contaminants within the coal seam that are leading to differences in prevalence between mines and seams, thus necessitating further work to identify the components.

**MONITORING EQUIPMENT**

The UK, USA and Australia use different monitoring equipment to measure respirable dust. Cyclone elutriators have biases in sampling and different models may have different biases. These biases may be due to both the sampler design and manufacture as well as the different particle size distributions being sampled. Sampling performed with different samplers may not be directly comparable. For instance, the US uses Dorr-Oliver cyclones with a 1.7 L/min flowrate while Australia uses Higgins-Dewell type cyclones at 2.2 L/min. A direct comparison of dust cannot be made between the two without side by side testing as different coal seams have different particle size distributions and therefore different sampling bias. Further error in sampling can be introduced based on pump pulsation.

The efficiency of a cyclone elutriator is affected by the airflow draw from the pump and flow pulsations may invalidate sample collection. Sampling requirements in BS EN ISO 13137:2013 Workplace atmospheres- Pumps for personal sampling of chemical and biological agents - Requirements and test methods requires that pump pulsation not exceed 10% of the flowrate (ISO (International Organisation for Standardization), 2013). Lee tested 13 widely available sampling pumps and found that ~80% of the pump models tested generated pulse magnitudes ≥10% with a wide variety of pulse shapes (Lee et al., 2014). Cornelissen (2008) tested pumps from three different manufacturers and found that all the pumps tested failed to maintain ≤ 10% pump pulsation. The flowrate also exceeded the allowable limits of flowrate variation in 77% of the readings (Cornelissen, 2008).

The respirable fraction of coal mine dust and the components within it have differing particle size distributions. Two mines in different seams may measure the same gravimetric mass. However, the number of particles present and the surface area of those particles may vary considerably. Cyclone elutriators measure aerodynamic equivalent diameter and not simply particle diameter. Particles with the same size and shape may have different densities due to different chemical compositions.
SHIFT ADJUSTMENT

The workplace exposure standards for RCD and RCS are based on 8-hour days, five days a week. Most Australian mines work longer shifts, for instance 12 hours a day, seven days on and seven days off. To account for these longer shifts, the WES needs to be adjusted.

The Brief and Scala model for adjusting WESs for non-traditional work weeks, is one of the more conservative models and is recommended by Safe Work Australia. It reduces exposure standards proportionally for increased exposure and reduced recovery time. The Brief and Scala model was developed for the petroleum industry and has not been validated for dust exposures. The adjustment process has no consideration for the agents’ activity on the body, the process by which the body removes the chemical or the biological half-life of the chemical (AIOH (Australian Institute of Occupational Hygienists), 2016, Teirnan and VanZanten, 1998).

The Occupational Safety and Health Administration (OSHA), Quebec and Pharmacokinetic models are based on the residence time for the particles in the lungs and assume that a long biological half-life for a substance is 1000 hours. This assumption needs to be analysed and verified for coal dust as measurements of the half-life of coal dust yield significantly longer timeframes. Morrow (1982) cited several mammal studies measuring half-lives of coal including, Heppleston et al., (1971), which found rats with chronic high level exposure had 380 and 430 day half-lives for high and low rank coals and Stober et al., (1967), which found a 4.9 year biological half-life in a post-mortem investigation of miners. Morrow (1982) found coal retention half-lives to be 767 and 637 days for anthracite and high rank bituminous coals in dogs lungs (Morrow et al., 1982).

OTHER LIMITS OF DATA

The epidemiological data only considers papers from English-speaking countries. Historic study populations consisted solely of males. There is therefore no information about the particular risks of female susceptibility or any potential impact of ethnicity. Smoking was only taken into account in a few studies and it is suggested that smoking is a true risk factor as well as an effect modifier. Most studies were not adjusted for smoking status and may, in general, overestimate the effect of coal dust (Beer et al., 2016).

CONCLUSIONS

In conclusion, there are a number of factors to be considered when setting exposure standards. A WES cannot simply be copied across from one country to another without consideration of the differences in monitoring methodology, mining conditions, coal geology and health surveillance. Australia uses different sampling equipment than the US at a different flow rate, which has a different bias to the ISO sampling curve. The cyclone elutriators are measuring aerodynamic equivalent diameter and not strictly particle diameter, which means particles of similar sizes but different densities may be treated differently by the cyclone. Australia should perform its own studies to verify that overseas WESs are fit for purpose in the Australian mining environment including shift lengths.

The UK and US standards are based on averages of dust concentrations for all mines. The data shows disease prevalence can vary significantly in different mines and regions. Consideration should be given to these differences in Australia. More research is needed into the chemical composition and particle size distribution of these coals and what is causing these differences in disease prevalence. Future WESs may need to include mine or region specific limits based on these factors. In Queensland, CWP accounts for only a third of MDLD cases. The WES for RCD also needs to consider not just CWP, but the host of other MDLDs that are being diagnosed. This may include the need to understand the contributions of the inhalable and submicron fractions of dust in addition to the respirable fraction.

REFERENCES


ISO (INTERNATIONAL ORGANISATION FOR STANDARDIZATION) 2013. BS EN ISO 13137:2013 Workplace atmospheres - Pumps for personal sampling of chemical and biological agents - Requirements and test methods.


STOCKS, P. 1962. On The Death Rates From Cancer Of The Stomach And Respiratory Diseases In 1949-53 Among Coal Miners And Other Male Residents In Counties Of England And Wales.

LABORATORY AND FIELD TESTING OF SURFACTANTS USED TO MEET NEW WORKPLACE EXPOSURE STANDARDS FOR RESPIRABLE DUST IN COAL MINES

Neil Alston¹, Ping Chang², Zidong Zhao³ and Apurna Ghosh⁴

ABSTRACT: Respirable coal dust is generated during the mechanised mining process and is one of the main occupational health hazards in coal mines. Due to its fine characteristics, it can affect the performance of mining equipment and cause adverse health effects in mine workers. There is a recent focus upon improved dust control measures in Australian mines. This is primarily due to a resurgence in reported cases of coal mine lung dust disease in both NSW and QLD coal mines. New more stringent workplace exposure standards (WES) are currently being introduced for both respirable coal and silica dusts. Considerable research has been carried out in developing coal dust control technologies, and water suppression with added surfactant is one high-level control measure available to coal mines. This paper will review recent dust monitoring results, available control measures and then analyse recent laboratory and field test results for a commercially available surfactant known as DUST KING. These test results will greatly assist mines to consider surfactants as an effective control measure to improve the occupational health of mineworkers.

BACKGROUND AND MOTIVATION

The mechanised mining process used to extract coal in mines generates fine dust particles during the rock breaking process. These dust particles are raised into the workplace and in close proximity to mine workers operating mining equipment causing a health hazard that must be managed to avoid adverse health effects. Prolonged exposure to dust is one of the main occupational health hazards in coal mines. Respirable dust can penetrate into the lungs and can cause a range of dust diseases collectively referred to as coal mine lung dust disease. With the improvement of coal dust controls in coal mines, the prevalence of coal mine lung dust disease has decreased over the last few last decades (Laney and Attfield, 2010). However, new cases are reported in Australia (Cliff et al, 2018), also in China and USA (Blackley et al, 2016, Xu et al, 2017) leading to heightened awareness of the dust hazard in coal mines. In addition to coal dust, a large amount of crystalline silica dust is generated during the mining process, especially for stone cutting (Jokonya, 2014). There were 15 new silicosis cases reported in QLD mining, resources and quarrying industries for the 2019-20 period (Queensland Health, 2020).

Two key factors have led to a resurgence in coal mine lung dust disease in Australia:

1. Improved diagnostic and reporting strategies, and
2. Dust control measures have not kept pace with increased dust make from larger, higher productivity mechanised mining equipment.

To protect miners from adverse health effects, the WES for respirable coal dust has reduced by 40% from 2.5 to 1.5 mg/m³ TWA-8h and respirable crystalline silica dust by 50% from 0.1 to 0.05 mg/m³ TWA-8h (SWA, 2019). In NSW, implementation was 1 July 2020 for crystalline silica and is 1 February 2021 for respirable coal dust whilst in QLD, implementation for both dusts was 1 September 2020.

Recent dust monitoring results were presented at the NSW Standing Dust Committee Forum (Coal Services, 2020). Figure 1 shows the number of actual dust exceedances in NSW for both surface and...
underground coal mines in 2019 (in light and dark blue) overlain by the number of exceedances under the proposed WES (in red and grey). It is predicted respirable quartz exceedances will significantly increase by around 3-fold unless new or improved control measures are considered and introduced to address this critical health issue. In QLD, when the new limits are applied to Q1 to Q3-2020 monitoring data for underground coal mines it shows a 5-fold increase in single sample exceedances for respirable coal and near 4-fold for respirable silica (Djukic, 2020). Industry must immediately take action to arrest these predicted dust exceedances.

The hierarchy of controls to manage hazards in the mining industry is widely accepted and is a feature of most guidance material on the topic of dust suppression in mines. Some leading types of engineering controls for dust suppression in underground coal mines are:

- automation involving total removal of workers from dusty environments,
- water infusion to increase the moisture content of in-situ coal,
- high pressure water and atomising sprays to optimise water spray dust capture using very fine water droplets,
- foams used to blanket broken coal and dust, and
- surfactants used to increase the wettability of coal.

Surfactants, an abbreviation for surface acting agents, are a simple and proven cost-effective control measure to reduce water tension thereby improving the wetting capability of water droplets and increasing coal dust capture efficiency (Xu et al, 2017). However, the effectiveness of surfactants warrants further investigation, so mines have a clearer understanding of the underlying science when assessing this control measure for their operations. Other benefits include improved control measure design, optimisation and to screen and develop new surfactants (Meng et al, 2019). Prior studies have shown a wide variance of coal dust suppression improvement using surfactants compared to water only ranging from 0 to 93% (Chandler et al, 1990, Kilau et al, 1996, Kost et al, 1980, Meets and Neethling, 1987 and Tien and Kim, 1997). This study will focus on the use of surfactants in the Australian coal mining context.

LABORATORY TEST PROGRAM

A laboratory test program was scoped and undertaken by WA School of Mines: Minerals, Energy and Chemical Engineering, Curtin University, Kalgoorlie (Chang et al 2020a). The laboratory tests involved three dust types and three surfactants. The dust types were a sub-bituminous thermal coal sourced from Collie WA, a bituminous coking coal sourced from the Bulli Seam South Coast, NSW and a high silica content Hawkesbury Sandstone sourced from a Sydney NSW tunnel project. The three surfactant types were DUST KING, DUST KING A and DUST KING B. DUST KING is a commercially available anionic/non-ionic blend surfactant used for dust suppression in mines, quarries and tunnels (Quarry Mining and Construction Equipment, 2020). Its main use is to improve the dust capture efficiency of water sprays but can also be used on roadways and stockpiles. For water sprays, DUST
KING is simply injected into the existing water system with a pneumatic (sidewinder) pump at a 1:3000 mix ratio (Figure 2). DUST KING A and DUST KING B are variants of the standard DUST KING product selected to test certain constituents of the standard product. The aim was to better understand how the standard product worked and to identify if variants worked better with different dust types.

Static tests, including sink tests and surface tension tests, were firstly conducted to evaluate the effect of doses of each surfactant on the dust suppression efficiency. Wind tunnel tests were then conducted to evaluate the dust suppression efficiencies of the surfactant solutions during dynamic progress with a shorter contact time. The latter test method is considered a more reliable indicator of surfactant performance in the field.

Sample Preparation and Test Methods

All dust samples were prepared following the procedures of ASTM (ASTM, 2013). A jaw crusher was used first to crush raw samples to fine particles. Then the samples were dried in an oven at 35 °C until the weight change less than 0.1% per hour. After that, the samples were sieved to a size range of 0 – 38 μm. Surfactant solutions were prepared in deionized water by using a magnetic bar. For each surfactant, DUST KING, DUST KING A and DUST KING B, five different concentrations of solutions were prepared, includes 0% (pure water), 1:1000, 1:2000, 1:3000, 1:4000, 1:5000.

Sink test is one of the most direct and reliable methods to investigate the effect of different surfactant solutions on the dust wettability (Chen et al, 2019). In this test, 0.5 g dust particles were placed on the surface of 80 ml surfactant solution. Sink time was recorded as the duration of the particles disappear from the surface of the surfactant solution. The test was considered to fail when the sink time was greater than 0.5 hour. All the sink tests were run for three replicates, and the average data was used as the final sink time for each test.

Usually, the surface tension of the surfactant solution is one of the main factors that determine the sink time. Thus, the surface tension for each surfactant solution was also conducted. The surface tension of each surfactant solution was tested by using Analite surface tension meter, model 12141. The surface tension tests were run three times for each solution, and the highest data was used as the surface tension for each solution.

A schematic diagram of the wind tunnel is shown in Figure 3. The dimensions of this wind tunnel are 0.5 m (H) × 0.5 m (W) × 4.50 m (L). The cyclone dust collector acted as an exhaust fan and provided an air velocity of 0.68 m/s in the wind tunnel. The water was sprayed by a nozzle (CPB1322 TEEJETBODY and D12-45HSS DISC&CORE, Spraying Systems Co. Pty. Ltd.) with a flow rate of 4.97 l/min. The dust was injected by the dust generator to wind tunnel first until the dust concentration reached a stable state. After that, the pump was turned on to spray water or surfactant solutions. Dust concentrations were measured by a TSI DustTrack™ II handheld aerosol monitor before and after turning on the pump. The dust suppression efficiency was calculated by the following equation:

\[
\eta = \frac{C_{\text{before}} - C_{\text{after}}}{C_{\text{before}}} \times 100\%
\]  

(1)
where \( C_{\text{before}} \) and \( C_{\text{after}} \) are the dust concentrations before and after applying the water spray, respectively.

Figure 3: Schematic diagram of the wind tunnel: (1) dust generation part, (2) spray part, (3) dust measurement part, (4) dust collection part (Chang et al, 2020b).

Static Test Results

The surface tensions for three surfactant solutions under different concentrations are shown in Figure 4. As can be seen, the surface tension for pure water is around 65 mN/m. All three surfactants could decrease the surface tension dramatically. The surface tension is dropped from 64.6 mN/m to 45 mN/m by adding 1:5000 DUST KING A, and the surface tension is further reduced to 34.8 mN/m with the surfactant concentration increased to 1:1000. DUST KING B and DUST KING show similar performance in surface tension reduction, both superior to DUST KING A. Sink test results are shown in Figure 5.

Figure 4: Surface tension of three surfactants with various concentrations

DUST KING A fails to wet both coal types completely in 0.5 hour. For DUST KING, the sink time decreases dramatically with the increase of solution concentration, which drops from around 1500 s with 1:5000 concentration to about 330 s with 1:1000 concentration. DUST KING B gives the best performance amongst three surfactant solutions at all of the five concentrations. However, for the crystalline silica, different trends are observed. All three surfactants wetted the crystalline silica in a
short period. Even for pure water, the average sink time is around 150 s. Overall, from the static tests, DUST KING B gives the best performance amongst three tested surfactants.

![Sink time graphs](image)

**Figure 5:** Sink time of Premier thermal coal (top LHS), Bulli coking coal (top RHS) and crystalline silica (bottom) with three surfactants and various concentrations

**Dynamic Test Results**

The mean dust suppression efficiencies of surfactant solutions for three dust types are presented in Figure 6. As expected, water gives the lowest suppression efficiencies, and DUST KING B gives the highest efficiencies for all three dust types. Consistent with the results of static tests, the surfactant solution with shorter sink time results in a higher suppression efficiency. Suppression efficiency of Bulli coking coal and crystalline silica were similar but greater than Premier thermal coal. The results show different types of coal particles affect suppression efficiency. Overall, the results obtained by wind tunnel tests have a good agreement with the results of static tests. DUST KING B has the best performance in dust suppression.

**FIELD TEST PROGRAM**

Field tests have been undertaken in NSW South Coast and Hunter Valley underground coal mines to validate the laboratory test data. This involved tests in both development (DEV) and longwall (LW) production panels. Coal type was Bulli Seam coking for South Coast mines and a semi-soft coking for the Hunter Valley mine. All mines were mining thin seams in the order of 2.0m thickness and routinely cutting stone in the floor, roof or interseam bands. Stone types ranged from sandstone, mudstone or siltstone, all containing high levels of crystalline silica. DUST KING was tested at South Coast mines whilst DUST KING B was tested at the Hunter Valley mine.

For South Coast mines, body worn gravimetric sampling was undertaken in accordance with AS 2985 Workplace atmospheres – Method for sampling and gravimetric determination of respirable dust (2.2 L/min flow rate). Whilst for the Hunter Valley mine, co-located stationary gravimetric samplers and Nanozen 9000 Dust Count real-time aerosol monitors were used with a respirable dust size selection inlet in accordance with ISO 7708:1995 Air quality – particle size fraction definitions for health-related sampling (1.0 L/min flow rate). The field test results are shown in Figures 7 and 10.
Figure 6: Mean suppression efficiency of Premier thermal coal (top LHS), Bulli coking coal (top RHS) and crystalline silica (bottom) with three surfactants.

Figure 7: Field test results for DUST KING in DEV and LW panels at South Coast Mines.

Field Test Results

DUST KING shows significant reductions in dust readings of between 35 and 51% (mean 43%) for respirable coal dust and 62% for respirable silica dust. DUST KING B also shows significant reductions in dust readings of between 38 and 67% (mean 57%) for respirable dust and 33 to 67% (mean 50%) for respirable silica dust. The accuracy of silica field test data is +/- 0.01 mg/m$^3$ which makes comparisons between surfactant products difficult to quantify with reasonable certainty at low dust loads. However, the laboratory results undertaken at higher dust loads (above 50mg/m$^3$) clearly showed DUST KING B was superior for both dust types.
The real-time data is very useful for the assessment of dust control effectiveness. Due to the scale of the graph the visual impact of the surfactant is less discernible however the large number of 5-second average short interval measurements provides a greater level detail and a better understanding of how the control measure impacts upon dynamic face operations. Furthermore, the real-time monitor had a lower detection limit of +/- 0.001 mg/m$^3$ and comparisons between water only and surfactant added are more reliable as these are made within shift versus separate days for the body worn gravimetric sampling method.

Box plots show a reduced interquartile range for DUST KING B compared to water. That is, the dust readings are more consistent and the control measure effectiveness is more reliable.

Figure 8: Field test results for DUST KING B in LW panel at Hunter Valley Mine

Figure 9: Real time data plot of results for DUST KING B in LW general purpose water for Chock 1 and BSL locations at Hunter Valley Mine
CONCLUSIONS

This study independently evaluated the suppression efficiency of three surfactants in the laboratory and two surfactants in the field with a range of coal mine dusts. Both the static and dynamic laboratory tests showed that DUST KING B gives the best performance amongst three tested surfactants. Field tests validated the laboratory tests, with DUST KING B superior to DUST KING. The reduction in dust concentrations measured using the surfactants in the field trials ranged from 35 to 67% (mean 49%) across all dust types. This reduction is in line with recent WES changes for respirable coal and crystalline silica dusts of 40% and 50% respectively. The study is unique as it uses field tests to validate laboratory tests for a range of surfactants and Australian coal mine dust types, thereby providing operators with confidence in surfactant performance when applied in a dynamic mining environment. The results are also relevant to tunnel, civil and metalliferous mine operators where crystalline silica is the primary dust type and the same WES of 0.05 mg/m$^3$ also applies to these industries.

ACKNOWLEDGMENTS

This research project is supported by Quarry Mining & Construction Equipment PTY LTD and Curtin University (CON-SE-WAS-PC-62599-2).

REFERENCES

Boyne L, 2020. 2019 Airborne Dust Results, Standing Dust Committee Regional Forums, Coal Services, February and March.
Coal Services 2020. Standing Dust Committee Regional Forums, February and March.
Djukic, F 2020. Workplace exposure standards – respirable dust, Mine Managers Association of Australia, Northern Region Seminar and Webinar, Mackay, 13 November.


BEYOND BLASTING: EVALUATION OF A DISC-BASED ROCK CUTTING SYSTEM FOR SURFACE COAL MINING

Isaac Dzakpata¹, Dihon Tadic², Amin Mousavi³ and Mehmet Kizil⁴

ABSTRACT: Continuous rock cutting technology is steadily advancing, with the development of undercutting-disc systems for hard rock applications progressing to the commercial prototype stage. Building on research supported by ACARP that investigates the potential for this technology to be applied to surface coal mining for overburden removal, an economic evaluation is presented to assess potential costs and benefits.

A case study deposit was utilised to examine various mining fleets based on a boom-style cutting machine, including comparison with a conventional drill/blast operation. Several material transport systems were considered: haul trucks, conveyors, and hybrid systems incorporating de-coupled tucks with hopper-style transfer stations. A key finding is that the break-and-load Operating Cost (Opex) for the cutting systems is higher in all cases for this deposit scenario than the baseline case (drill/blast/shovel), by 50% or more. The results also indicate that the haulage Opex for the cutting systems varies considerably, depending on the transport method. Direct truck haulage of cut material has higher Opex than the baseline system, due to the larger fleet requirements and increased loading duration, whereas a conveyor transport system provides a more cost-effective haulage solution compared to the baseline.

The analysis further suggests that whilst the capital and operating costs on a per unit basis ($/bcm TMM) may be higher for a cutting system, when compared to traditional mining methods however, there are significant benefits that are unique to a cutting system that can indeed result in a better overall economic outcome, depending on the specific nature of the deposit and various operational constraints and opportunities. Such benefits include direct value offered by the potential for steeper pit slopes and increased coal recovery, as well as indirect yet tangible value through elimination of blasting-related impacts (e.g. fumes, noise and vibration), process simplification, automation potential, improved safety and lower emissions.

INTRODUCTION

Overburden removal in surface coal mines typically involves blasting to fragment the material for subsequent handling via shovels, excavators and draglines (Dzakpata, Tadic and Quidim, 2019). Blasting is relatively cost-efficient compared to mechanical fragmentation methods however, evolving rock cutting technologies are beginning to close this gap when broader aspects of a mining operation are considered (Karekal, 2013). Whilst explosives are likely to remain superior for the foreseeable future in terms of bulk rock fragmentation, there are often many benefits of a continuous rock cutting system that bring new opportunities and efficiencies to a mining operation (Mitani et al., 1987; Hood et al., 2005).

This paper explores an undercutting disc method of rock excavation, and its potential for application to surface coal mining. The envisaged cutting system for coal overburden is based on hard rock cutting technology being developed by Komatsu as DynaCut. Specifically, this work focussed on the economic aspects of the technology, utilising a case-study deposit and comparison with traditional mining processes. Critically, this evaluation extends beyond the basic concept of fragmentation cost per bcm, to identify and explore the potential benefits relating to factors including increased selectivity, continuity, simplicity and electrification. This evaluation is part of a broader body of work that aims to clarify the potential for mechanical rock cutting in surface coal mining, and identify key aspects that require careful and comprehensive assessment on a case-by-case basis, to ultimately determine can this technology bring value to my operation?

¹ Dr Isaac Dzakpata, Mining3. Email: idzakpata@mining3.com Tel: +61 7 3346 5602
² Dihon Tadic. Independent Consultant. Email: dihontadic@hotmail.com Tel: +61 7 3365 5640
³ Dr Amin Mousavi, Mining3/CSIRO. Email: amousavi@mining3.com Tel: +61 7 3365 5640
⁴ Assoc. Prof. Mehmet Kizil, University of Queensland. Email: m.kizil@uq.edu.au Tel: +61 7 3365 4499
CASE STUDY DEPOSIT AND METHODOLOGY

A Central-Queensland coal deposit was selected as a case study to examine the application of a continuous mining system. This coal seam varies in thickness from about 5.0 m to 8.7 m, with a dip of approximately 5 degrees. A typical stratigraphic column indicates adjacent bands of siltstone and claystone up to 3 m in thickness with Uniaxial Compressive Strength (UCS) of up to 40 MPa. The principal overburden material is sandstone, with UCS of up to 90 MPa.

As discussed by Tadic, Powell and Lever (2015), mechanical rock cutting systems typically have inflection points for cost (rapidly increasing) and efficiency (rapidly decreasing) with increasing rock UCS strength of 40-50 MPa however, the type of undercutting disc system assessed herein allows effective cutting of rock of much higher strengths - even beyond UCS of 200 MPa. This makes such a technology suitable for even the toughest of coal measure rocks, from a standpoint of not requiring intervention should high-strength zones or intrusions be encountered in otherwise manageable lower-strength overburden.

The approach for this analysis involved determining a suitable pit and mining sequence using a conventional (drill/blast/load/haul) mining process, and then evaluating excavation of the same pit (i.e. uncovering the same coal block) with a rock cutting system. For simplicity and to minimise extrapolation of indicative performance and costs, a cutting machine of a similar style to current prototypes was considered; a track-mounted, single-boom, single-cutter machine. Several material transport options for the cutting system were considered and evaluated. Equipment fleets for each mining scenario were identified, with cost estimates developed as summarised in Table 1. The cost estimates presented herein will certainly not represent costs at all operations, however, for the purposes of this analysis, it is the consistency across the various systems and relative costs that are more relevant and insightful than specific values estimated for individual machines/ processes/ items.

Table 1: Summary of cost-sourcing for mining equipment fleets presented in this analysis

<table>
<thead>
<tr>
<th></th>
<th>Conventional Equipment</th>
<th>Cutting Machine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capital Costs CapEx</td>
<td>CostMine (2019/20) estimates used for shovels, trucks, etc.</td>
<td>No cost guidance available; the authors have estimated costs based on approximate machine mass and $/tonne CapEx costs for roadheader machines of similar size.</td>
</tr>
<tr>
<td>Operating Costs OpEx</td>
<td>CostMine (2019/20) estimates used with adjustments to labour rates as per industry guidance. Variance (for cost range estimation) based on industry guidance.</td>
<td>The authors have estimated operating costs based on CostMine (2019/20) estimates for roadheader machines of similar size, with adjustments to labour rates as per industry guidance, adjustments to energy costs as per estimated power requirements, and adjustments to wear part costs based on presumed cutter life and part costs (also estimated by the authors).</td>
</tr>
</tbody>
</table>

* Estimates should not be considered as guidance for potential future machine costs

Baseline mining system

A baseline mining scenario was developed for the deposit, utilising a drill-and-blast mining process, shovel loading and truck haulage. An optimisation technique was developed to identify a solution for the Ultimate Pit Limit (UPL), defining the final shell of the mining pit over the life of the mine, block extraction sequencing, and volumes of coal and waste to be mined.

The defined mining area included about ~12 million tonnes of coal and ~47.5 million bcm of waste rock. Figure 1 shows (a) Ultimate Pit Limit (UPL) of West-East section of the mine and (b) annualised Life of Mine (LOM) material movement schedule and discounted cumulative value. The mine life was specified as 11 years to provide about 5.5 million bcm of total material mined per year (coal and overburden) with an EX3600 excavator. In this scenario, one excavator and four 220-tonne trucks would be sufficient to achieve the production target.
SUMMARY OF POTENTIAL CONTINUOUS MINING SYSTEMS

Several mining system configurations based on the single-boom, single-cutter style of cutting machine were considered and evaluated, with the key difference being the mode of material transport (Figure 2).

Figure 2: Continuous mining systems considered for economic analysis and comparison

PERFORMANCE AND COST MODELLING

This section summarises the recommended equipment fleets, and the estimated capital and operating costs for the various mining systems. The transport requirements are based on the average haul...
distance (return) of approximately 9 km. The total operating cost estimates ($/bcm) include maintenance and labour, and are shown across three key categories: 1) Break and Load - all drill-and-blast costs and shovel costs (conventional mining) or cutting machine costs (continuous cutting); 2) Ancillary Equipment - includes supporting equipment (e.g. dozers, graders) in the pit and at the waste dump; 3) Material Handling/Transport - haul trucks, conveyors, hoppers as relevant to each mining system.

**Baseline mining system**

The recommended fleet for the baseline mining scenario is illustrated in Table 2. The baseline system Opex estimates indicate a cost for Break and Load of between $1.95/bcm and $2.32/bcm. Haulage costs (four 221 t haul trucks) are approximately $1.67/bcm, whilst ancillary equipment totals $0.99/bcm and includes two-wheel dozers, one grader and one water cart. The total estimated Opex for this system, for the case study mining scenario, is about $4.62/bcm to $4.99/bcm. Capital Cost (Capex) estimates for each of the three categories are also shown in the table. In this case, these are approximately $10.5 M, $7.0 M and $16.7 M for the Break and Load, Ancillary and Transport categories respectively.

**Continuous mining systems**

Performance modelling of various potential scales of cutting machines determined that three machines with cutter discs of 1250 mm in diameter (denoted OB-1250) would be well-matched to the case study mining scenario, having a combined production capacity of approximately 6 million bcm per year. The various continuous mining systems presented here therefore incorporated a fleet of three of these OB-1250 machines, which are effectively an up-scaled version of the existing prototype hard rock machines that have a cutter disc around 700 mm in diameter.

**Direct truck loading**

For the case study deposit, it was determined that a direct loading system with three OB-1250 machines would require six 180-tonne capacity trucks in total (two per machine). The fleet (Table 2) includes ancillary equipment of two dozers, one grader and one water cart. The total operating cost (including maintenance and labour) for the mining operation in this case would be about $6.50/bcm - $7.71/bcm. The break and load cost, representing the operating cost of the three cutting machines, is estimated to be about $3.10/bcm - $4.31/bcm, with total fleet production of about 969 bcm per operating hour.

**Conveyor transport**

Transportation of cut material via conveyor has a key advantage of matching a continuous transport system with the effectively-continuous cutting nature of the cutting machine. As three cutting machines are required, the targeted system configuration incorporates three on-bench flexible/extendable conveyor units, each feeding a single trunk conveyor that transports all of the material from the pit to a stacker/spreader unit at the dump location.

A summary of the selected fleet for the case study deposit, applying cutting machines with conveyor transport, is shown in Table 2. In this case, the ancillary equipment requirement is lower than that of a truck-based transport system, comprising one dozer and one grader. The total operating cost (including maintenance and labour) for the mining operation in this case would be about $4.42/bcm - $5.52/bcm. The break and load cost is about $2.82/bcm - $3.92/bcm, with total fleet production of about 1065 bcm per operating hour. The productivity and operating costs vary slightly (about 10%) compared to the above truck-based system, because there are no spotting delays in the conveyor loading process.

**Indirect truck loading via hopper transfer station**

This system style includes multiple configurations that aim to allow truck haulage that is decoupled from the actual cutting machine, by introducing an intermediate hopper/transfer station. The first option is to utilise a mobile hopper system that follows the cutting machine and provides surge capacity so that trucks can be loaded without the cutting machine pausing for spotting, or indeed waiting for trucks to arrive. A summary of the selected fleet for this system is shown in Table 2.
The total Opex in this case would be about $7.20/bcm - $8.30/bcm. The break and load cost is about $2.82/bcm - $3.92/bcm, with total fleet production of about 1065 bcm per operating hour; the same as for the conveyor-based system.

A second option involves a fixed or semi-mobile (i.e. relocatable) in-pit hopper system for loading trucks. This option allows the hopper/transfer station to be larger in capacity and simpler in construction when compared to the prior mobile hopper system option. In this analysis, on-bench conveyor units have been selected to transport the material from the cutting machines to the hopper. Table 2 shows a summary of the selected fleet.

Table 2: Fleet recommendations for the case study deposit (baseline Drill & Blast plus cutting system scenarios), showing equipment and associated Opex (includes maintenance and labour); total Capex estimates for each category are also indicated

<table>
<thead>
<tr>
<th>Fleet / Cost Items Summary</th>
<th>Baseline System: Shovel &amp; Trucks</th>
<th>Cutting System 1: Cutting Machines &amp; Trucks</th>
<th>Cutting System 2: Cutting Machines &amp; Conveyors</th>
<th>Cutting System 3: Mobile Hoppers</th>
<th>4: Conveyors &amp; Fixed Hopper loading Trucks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>OpEx $/bcm</td>
<td>OpEx $/bcm</td>
<td>OpEx $/bcm</td>
<td>OpEx $/bcm</td>
<td>OpEx $/bcm</td>
</tr>
<tr>
<td>Break &amp; Load:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Blasthole drill x 1</td>
<td>Y 0.30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Truck: Shot loader x 1</td>
<td>Y 0.13</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other D&amp;B / ANFO costs</td>
<td>Y 0.52 - 0.69</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydraulic Excavator EX3600 x 1</td>
<td>Y 1.01 - 1.21</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mobile Cutting Machines; OB-1250 x 3</td>
<td>Y 3.10 - 4.31</td>
<td>Y 2.82 - 3.92</td>
<td>Y 2.82 - 3.92</td>
<td>Y 2.82 - 3.92</td>
<td>Y 2.82 - 3.92</td>
</tr>
<tr>
<td><strong>OpEx Sub Total ($/bcm)</strong>:</td>
<td>1.95 - 2.32</td>
<td>3.10 - 4.31</td>
<td>2.82 - 3.92</td>
<td>2.82 - 3.92</td>
<td>2.82 - 3.92</td>
</tr>
<tr>
<td><strong>CapEx estimate ($)</strong></td>
<td>~$10.5M</td>
<td>~$28.1M</td>
<td>~$28.1M</td>
<td>~$28.1M</td>
<td>~$28.1M</td>
</tr>
<tr>
<td>Ancillary Equipment:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wheel Dozer x 1 for prep of face/ bench/ haul road areas</td>
<td>Y 0.32</td>
<td>Y 0.38</td>
<td>Y 0.34</td>
<td>Y 0.34</td>
<td>Y 0.34</td>
</tr>
<tr>
<td>Wheel Dozer x 1 for profiling of waste dump/ roads</td>
<td>Y 0.32</td>
<td>Y 0.38</td>
<td>Y 0.34</td>
<td>Y 0.34</td>
<td>Y 0.34</td>
</tr>
<tr>
<td>Grader x 1 for bench/ haul road prep/ drainage cutting</td>
<td>Y 0.15</td>
<td>Y 0.17</td>
<td>Y 0.16</td>
<td>Y 0.16</td>
<td>Y 0.16</td>
</tr>
<tr>
<td>Water Cart x 1; road dust suppression</td>
<td>Y 0.22</td>
<td>Y 0.26</td>
<td>Y 0.24</td>
<td>Y 0.24</td>
<td></td>
</tr>
<tr>
<td><strong>OpEx Sub Total ($/bcm)</strong>:</td>
<td>0.99</td>
<td>1.19</td>
<td>0.50</td>
<td>1.08</td>
<td>1.08</td>
</tr>
<tr>
<td><strong>CapEx estimate ($)</strong></td>
<td>~$7.0M</td>
<td>~$7.0M</td>
<td>~$3.4M</td>
<td>~$7.0M</td>
<td>~$7.0M</td>
</tr>
<tr>
<td>Material Handling/Transport:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rear-Dump Truck; 2211t x 4</td>
<td>Y 1.67</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rear-Dump Trucks; 1811 x 6</td>
<td>Y 2.22</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rear-Dump Trucks; 240t x 3</td>
<td>Y 1.48</td>
<td>Y 1.48</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bench Conveyors; transport from Cutting Machine; 1500 tph x 3</td>
<td>Y 0.36</td>
<td>Y 0.36</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trunk Conveyor, transport out of pit; 4536 tph x 1</td>
<td>Y 0.16</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stacker/spreader, distribution on waste dump; 4500 tph x 1</td>
<td>Y 0.59</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mobile Hopper-Feeders 1500 tph x 3; receive from Cutting Machine</td>
<td>Y 1.82</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fixed Hopper-Feeder 4500 tph x 1; single unit to receive material from bench conveyors &amp; load trucks</td>
<td>Y 0.74</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>OpEx Sub Total ($/bcm)</strong>:</td>
<td>1.67</td>
<td>2.22</td>
<td>1.10</td>
<td>3.30</td>
<td>2.58</td>
</tr>
<tr>
<td><strong>CapEx estimate ($)</strong></td>
<td>~$16.7M</td>
<td>~$19.7M</td>
<td>~$15.0M</td>
<td>~$32.6M</td>
<td>~$24.0M</td>
</tr>
<tr>
<td><strong>Estimated System Productivity: (bcm per operating hour)</strong></td>
<td>1159</td>
<td>969</td>
<td>1065</td>
<td>1065</td>
<td>1065</td>
</tr>
</tbody>
</table>
The total Opex (including maintenance and labour) in this case would be about $6.48/bcm - $7.58/bcm. The break and load cost is about $2.82/bcm - $3.92/bcm, with total fleet production of about 1065 bcm per operating hour.

Conveyor transport

Transportation of cut material via conveyor has a key advantage of matching a continuous transport system with the effectively-continuous cutting nature of the cutting machine. As three cutting machines are required, the targeted system configuration incorporates three on-bench flexible/extendable conveyor units, each feeding a single trunk conveyor that transports all of the material from the pit to a stacker/spreader unit at the dump location.

A summary of the selected fleet for the case study deposit, applying cutting machines with conveyor transport, is shown in Table 2. In this case, the ancillary equipment requirement is lower than that of a truck-based transport system, comprising one dozer and one grader. The total Opex (including maintenance and labour) for the mining operation in this case would be about $4.42/bcm - $5.52/bcm. The break and load cost is about $2.82/bcm - $3.92/bcm, with total fleet production of about 1065 bcm per operating hour. The productivity and operating costs vary slightly (about 10%) compared to the above truck-based system, because there are no spotting delays in the conveyor loading process.

Indirect truck loading via hopper transfer station

This system style includes multiple configurations that aim to allow truck haulage that is decoupled from the actual cutting machine, by introducing an intermediate hopper/transfer station. The first option is to utilise a mobile hopper system that follows the cutting machine and provides surge capacity so that trucks can be loaded without the cutting machine pausing for spotting, or indeed waiting for trucks to arrive. A summary of the selected fleet for this system is shown in Table 2.

The total operating cost in this case would be about $7.20/bcm - $8.30/bcm. The break and load cost is about $2.82/bcm - $3.92/bcm, with total fleet production of about 1065 bcm per operating hour; the same as for the conveyor-based system.

A second option involves a fixed or semi-mobile (i.e. relocatable) in-pit hopper system for loading trucks. This option allows the hopper / transfer station to be larger in capacity and simpler in construction when compared to the prior mobile hopper system option. In this analysis, on-bench conveyor units have been selected to transport the material from the cutting machines to the hopper. Table 2 shows a summary of the selected fleet.

The total operating cost (including maintenance and labour) in this case would be about $6.48/bcm - $7.58/bcm. The break and load cost is about $2.82/bcm - $3.92/bcm, with total fleet production of about 1065 bcm per operating hour.

DISCUSSION OF RESULTS

The estimated Opex for the various mining systems, applied to the case study deposit, are summarised in Figure 3. Estimates for total Capex, for the entire fleets in the above scenarios, are shown in Figure 4. The Capex is presented as a total figure ($M) as well as per bcm of material mined, representing the capital recovery cost component associated with each of the mining operations.

Key observations/findings from analysis and comparison of these systems and their associated estimated cost profiles include:

1) The break and load Opex for the cutting systems is higher in all cases for this deposit scenario than the baseline case (drill/blast/shovel), by 50% or more. The low estimate of baseline break and load Opex is about $1.95/bcm, whilst the best cutting system break and load cost is about $2.82/bcm (for the conveyor-based and hopper-based systems).

2) Ancillary equipment Opex is similar for each case, except for the cutting-conveyor system that requires less equipment and personnel, and therefore has a notably lower Opex than the other systems for this component.

3) The cutting system with the lowest total Opex ($4.42/bcm - $5.52/bcm) is the cutting-conveyor system, due to its relatively low Opex for transport and ancillary equipment.
4) The cutting-conveyor system ($4.42/bcm - $5.52/bcm) may indeed be competitive with the baseline system ($4.62/bcm - $4.99/bcm), in terms of total Opex; however, the complete cutting-conveyor system has a total Capex that is about $12 M or 35% higher (approximately $46 M vs $34 M).

5) For a truck-only haulage system, the costs of haulage for a cutting system and a conventional truck-shovel system converge with increasing haul distance.

6) A different configuration and scale of cutting machine may indeed prove more cost-competitive with the traditional process of drill/blasting/load.

![Figure 3: Operating cost summary for the baseline and cutting system scenarios applied to the case study deposit; Baseline = 1 x EX3600 shovel and 4 x 220 t trucks, Cutting systems = 3 x OB-1250 mobile cutting machines with various haulage/transport systems; High and Low Opex estimates shown ($/bcm) include maintenance and labour](image)

![Figure 4: Capital cost estimate summary for the baseline and cutting system scenarios applied to the case study deposit; Total fleet Capex (LHS), Capital recovery on a $/bcm of total material mined basis (RHS)](image)

**Sensitivity: Performance and Opex**

Figure 5 illustrates the influence of key variables on the performance (net cutting rate) of the cutting machine, based on an improvement of each of these variables by 10% from their nominal values. This shows that the most influential parameters on machine performance variance are the cutter diameter,
depth-of-cut (DOC) and the cutter velocity, and that parameters such as the time between cuts or the cutter change-out time have a substantially lower contribution to the variance of overall performance.

**Figure 5:** Illustration of key parameter variance effects on cutting machine performance (Net Cutting Rate, NCR): Impact is based on 10% improvement of individual parameters (adjusted individually, not collectively)

Key factors contributing to the Opex range estimates for the cutting machine system are shown in Figure 6. This specific example is for the OB-1000 machine (1000 mm diameter cutter); the OB-1250 has a very similar profile. This identifies that there are several key performance and cost factors that can be targeted to reduce Opex, and that even if the full improvements shown cannot be regularly or consistently achieved, even modest improvements can significantly reduce operating costs. Interestingly, reduction of the cost of each cutter by 25% from the nominal estimated cost would deliver an overall Opex saving of almost 5%. Modest improvements in cutter life, as well as depth-of-cut and cutter velocity, from their estimated nominal levels, would also deliver marked overall Opex savings.

**Figure 6:** Opex for OB-1000 cutting machine; Low/Nominal/High Opex cases shown, including breakdown of contributing factors for the Opex range estimate; (Opex estimates ($/bcm) include maintenance and labour)
Case study deposit mining outcomes

There are two critical elements associated with the mining outcomes from applying a cutting system to the case study deposit:

1) Reduction in total overburden excavated - due to a cutting method enabling steeper pit slope angles than a blasted excavation process

2) Increased coal recovery - cutting the overburden to the top-of-coal can significantly reduce the blast-related coal losses, which typically range from about 5% to 20% of in-situ coal (Esen, 2017; Chiapetta, and Postupack, 1995; Goswami and Brent, 2015; Kanchibotla and Scott, 2000)

The baseline mining case (drill/blast) assumed a stable pit slope angle of 45 degrees. An increase in this stable pit slope angle to say 55 degrees, by cutting the rock mass instead of blasting, results in over 2 M bcm less overburden removed over the life-of-mine, whilst uncovering the same coal seam area. This would equate to a 4.3% reduction in total material movement if achievable.

The effects of reduced coal loss offered by a mechanical cutting system are also substantial. Even with a conservative estimate of a reduction in coal loss during mining from say 10% to 3%, the resulting impact for the case study would be an additional 850 kt of coal recovered (11.83 Mt vs 10.98 Mt). This additional coal recovery represents additional revenue of approximately $80 M (AUD) based on a coal price of USD70/t. Even when considering the lowest Opex estimate for the lowest-cost cutting system (cutting + conveyors; $4.42/bcm), and based on the average strip ratio of this case study deposit, this additional amount of coal would otherwise require over 3 M bcm of overburden to be mined to uncover, at a cost of about $15 M. After the additional costs associated with processing and selling this extra coal (e.g. washing, freight, royalties, etc.), the added value to the operation is approaching $50 M. These outcomes are summarised in Figure 7.

Figure 7: Summary of outcomes for the case study deposit: Conventional mining vs cutting-conveyor system; illustration of pit slope and coal loss effects on mining volumes and coal recovery

A comparison of the cutting-conveyor system and the baseline system for the case study example is illustrated in Figure 8. This shows the variation in key mining outcomes, over the life-of-mine, for the high and low cost cutting estimates relative to the low cost estimate for the baseline operation. It can
be seen that in this case, the benefits of mining less overburden and realising higher coal recovery with the cutting system outweighed the higher Opex and Capex ($/bcm) of this system compared to the baseline. In this example, even the high cost estimate for the cutting system delivered a similar total profit to the baseline operation.

Figure 8: Case study mining outcome example: Life-of-mine cutting-conveyor system outcomes vs Low-cost baseline system (drill/blast/shovel/trucks); Includes capital costs

Clearly, the potential benefits of a cutting system for mining this case study deposit are indeed substantial, and although the underlying drivers of these (steeper pit slopes and reduced coal loss in this instance) are yet to be comprehensively quantified, the estimates and examples explored herein indicate that the key value offered by this technology is related to its ability to considerably improve/change mining practices, rather than simply compete with traditional processes on a like-for-like basis (i.e. bulk excavation rates and cost). This highlights the importance of carefully assessing the specific characteristics of a particular operation when considering the viability of applying a cutting system, so that the additional benefits and value can be appropriately weighed against the Capex and Opex differences.

ADDITIONAL BENEFITS AND HIGH-VALUE OPPORTUNITIES

It is evident that simply replacing the drill/blast/load process with a cut and load process of equal capacity will involve significantly higher Capex and Opex on a $/bcm basis, at least for the type and size of machine assessed herein i.e. a single-boom, single-cutter machine. However, due consideration must also be given to the many benefits introduced by such a system, which are simply not captured in this basic analyses of relative costs. These can offer varying levels of very tangible value to mine operators, particularly those with specific challenges relating to safety, unique environments, quality control, etc.

Key benefits

The quality/properties of a blasted muck pile play a critical role in the performance of the loading unit, and therefore the performance of the entire load and haul operation. Inherent variation, errors and issues in the early stages - even related to seemingly minor aspects like drilling accuracy and explosives quality - have cascading effects that are detrimental to the overall system performance and cost profile. In the stepwise drill/blast/load process, where the performance of each stage relies heavily on the implementation and outcomes of preceding stages, and there is added complexity of natural materials (rock mass variability and uncertainty), elimination and simplification of processes can introduce substantial benefits to cost and performance outcomes, as well as simplifying planning and introducing more certainty and consistency to operations.
Key benefits of introducing a continuous cutting system that replaces drill/blast/load include those presented in Figure 9. Many of these benefits have direct implications for improved safety of personnel (notably the sheer reduction of pit-based activities and equipment), whilst some directly influence costs and profitability (e.g. reduced coal loss and dilution, narrow seam recovery). Critically, the significant simplification of activities offered by a continuous cutting system provides a substantial reduction in operational risk and uncertainty, especially due to the removal of multiple activities that are inherently inconsistent, and commonly present operational complications and issues that degrade overall productivity and efficiency.

There are also potential risks or downsides associated with shifting to a continuous mining system; which vary depending on the system scale and configuration. For example, reliance on a single trunk conveyor for material transport introduces risk of complete transport stoppage should this conveyor go offline due to fault/maintenance/repair. Also mining agility may be compromised due to the relocation of equipment (e.g. bench conveyors) being relatively complex in comparison to conventional mining fleets.

![Common issues and key requirements](Image)

**Figure 9:** Summary of key additional benefits of continuous cutting vs conventional mining for surface coal mining application
Evaluating the opportunities: examples

In addition to the potential advantages relating to stable pit slopes and reduced coal loss presented above, key examples of high-value opportunities offered by a mechanical cutting system include:

*Recovery of otherwise-uneconomic narrow seams* - Consider the case study deposit with an additional narrow seam above the main coal seam, being say 1m in thickness with a 2m interburden between the seams. A traditional mining operation may not recover such a narrow seam due to the complexity and costs associated with the blasting and extraction processes required to separate it from its surrounding strata effectively. However, a continuous cutting system allows much finer control of excavation horizons and unit thicknesses to extract and separate relatively narrow coal-interburden sequences in the course of its typical operation.

In this case, the additional coal recovered due to the selective extraction of this narrow seam, would be over 1.35 Mt over the life-of-mine, even after allowing for 20% coal loss in this instance due to the very thin nature of the extracted unit. This extra coal represents additional revenue of about $128 M (AUD) at USD70/t (Figure 10).

![Figure 10: Example of potential value of thin seam extraction enabled by highly selective continuous cutting](image)

It is recognised that extracting a relatively narrow coal seam with a cutting system introduces operational inefficiencies (e.g. additional machine movement per volume mined), however, even if the OpEx for extracting the narrow seam separately is assumed to be 30%, 50% or even 100% more than the typical estimated costs, the value of extra coal recovered massively outweighs this marginal cost plus the costs associated with coal preparation and sale).

*Improved resource utilisation with vastly reduced proximity issues* - Mechanical cutting can provide the ability to mine closer to lease boundaries in locations where blasting issues can severely constrain proximity of operations to neighbouring properties. With blasting impacts like dust, fumes, noise and vibration heavily constraining various operations, and indeed affecting their licence-to-operate in some cases, a continuous cutting system with minimal noise and dust - eliminating all of the blasting-specific impacts - provides great opportunity for many mines that face these environmental challenges. In some cases, it may unlock part of a resource that is otherwise unable to be mined due to these proximity/neighbour issues.

Consider the case study deposit with a restriction applied to the shallow end of the pit. In this case, traditional mining activities are not permitted due to blasting impacts on neighbouring properties. Assuming that the pit is shortened by 200m for a drill-and-blast operation, but is otherwise fully mineable with a continuous cutting system, the impact on costs and coal recovery were estimated.

The exclusion zone reduces the coal accessed/recovered through the traditional mining process by about 2.60 Mt; this is about 21% less than that uncovered in the full pit zone (12.20 Mt total) without this restriction. With the cutting system allowing access to this excluded zone (i.e. the full pit area), this additional coal is mined at a considerably reduced strip ratio, due to the relative shallow cover at this
side of the pit; the strip ratio is about 0.9 bcm/t, compared to the average (total pit) strip ratio of 3.9 bcm/t. Figure 11 summarises this scenario and outcomes.

![Pit limit example: Cutting (+200m)](image)

- Total coal not accessible due to exclusion zone for drill/blasting operation = 2.60 Mt (~1.7 Mbcm)
- Additional coal recovered with cutting to full pit limit = 2.52 Mt (assumes 3% coal loss)
- Strip ratio for extra coal = 0.9 bcm/t

- Boundary proximity impact example = ~$269M more coal recovered for an additional mining cost of about $18M - $22M (cost of mining additional 4 Mbcm i.e. ~2.3 Mbcm overburden + ~1.7 Mbcm coal)

**Figure 11:** Example of potential proximity relaxation enabled by continuous cutting

**Reduced carbon footprint / emissions** - Mechanical cutting provides opportunity to substantially reduce emissions through elimination of process steps and equipment (e.g. no blasting, reduction of diesel machine fleets), and accelerated transition to electrified equipment. This is particularly the case for a continuous cutting system that utilises an electrified machine in combination with conveyors for material transport, in which case the key remaining diesel equipment is the limited ancillary machines (e.g. dozers and graders) predominantly for bench/pit/road maintenance.

Estimates presented by Goswami and Brent (2015), Ener (2015) and LowCVP (2020) were used to compile a summary of greenhouse gas (GHG) emissions for a typical Australian surface coal mining operation, for each tonne of coal mined (Table 3). In this comparison, the continuous cutting system incorporates the single-cutter style of machine with a conveyor network for material transport, as per cutting system 2 outlined above.

**Table 3:** Summary of principal greenhouse gas (GHG) emissions from surface coal mining operations: Typical conventional operation vs continuous cutting with conveyor transport

<table>
<thead>
<tr>
<th>Emissions per tonne of coal mined</th>
<th>Conventional Coal Mining Operation</th>
<th>Cutting Machine + Conveyors*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Note</td>
<td>kg CO₂-e</td>
<td>kg CO₂-e</td>
</tr>
<tr>
<td>Combustion of diesel fuel</td>
<td>~5 litres per tonne of coal mined</td>
<td>13.0</td>
</tr>
<tr>
<td>Production and distribution of diesel fuel</td>
<td>Upstream: ~0.6kg CO₂-e per litre</td>
<td>3.0</td>
</tr>
<tr>
<td>Explosives detonation</td>
<td>~2 kg per tonne of coal mined</td>
<td>0.4</td>
</tr>
<tr>
<td>Explosives manufacture</td>
<td>Upstream: ~1-4kg CO₂-e per kg</td>
<td>2.0 - 8.0</td>
</tr>
<tr>
<td>Generation of electricity consumed</td>
<td>~15 kWh per tonne of coal mined</td>
<td>16.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seam gas release upon exposure and disruption</td>
<td>Independent of mining method</td>
<td>45.0</td>
</tr>
<tr>
<td>Total principal emissions:</td>
<td>79.4 - 85.4</td>
<td>62.8</td>
</tr>
<tr>
<td>Total emissions excluding seam gas release:</td>
<td>34.4 - 40.4 kg CO₂-e</td>
<td>17.8</td>
</tr>
</tbody>
</table>

* These CO₂-e figures were derived from the cutting-conveyor scenario for the case study deposit, but adjusted (increased) for a strip ratio of 6 bcm/t to allow more suitable comparison with the referenced figures for a typical surface coal mining operation.
Emissions for a typical conventional mining operation total about 34.4kg – 40.4kg CO$_2$-e per tonne of coal mined, excluding the seam gas release component, which occurs regardless of the mining method. If a continuous cutting operation with conveyor transport is considered by comparison (as an example of the ultimate reduction in emissions through electrification of mining and haulage equipment), an indicative emissions profile suggests a total of about 17.8kg CO$_2$-e per tonne of coal mined (excluding the seam gas release component). This illustrates potential for circa 50% reduction in GHG emissions offered by this style of mining system compared to typical mining methods utilising drill-blast and extensive diesel equipment. Increasing the proportion of renewable energy in the mix of electricity generation would of course add even further benefit.

This emissions analysis excludes the impact of a lower strip ratio from steeper pit slopes (less overburden excavated per tonne of coal mined) and the impact of improved coal recovery (more coal product per mined volume), which act to further reduce (by perhaps 5-10%) total emissions per tonne of coal mined/recovered.

This may have significant economic ramifications in the case of a carbon price (or equivalent). For example, based on the above emissions estimates, for every $10/t CO$_2$-e a cutting-conveyor system may reduce this “penalty” by around $2 M for every 10 Mt of coal mined.

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER RESEARCH

This performance and cost assessment, focusing on a case study deposit scenario and various continuous mining systems based on single-boom single-cutter machines, indicates that whilst the Opex and Capex of these cutting systems is typically higher than that of conventional mining, there are configurations - such as cutting machines with conveyor transport - that are likely very competitive on an Opex basis. Critically, there are also various benefits offered by continuous cutting that must be considered alongside the basic metrics of Opex and Capex, which can substantially improve the impacts and overall cost profile/profitability of a cutting system - even to the point where a cutting system with higher Opex and Capex than a conventional operation can actually deliver a better overall economic outcome.

Specific factors such as haul distances and production requirements have a significant impact on equipment selection and matching, and can considerably affect the associated costs, even on a $/bcm basis. These cost analyses should therefore be considered as a framework from which to conduct evaluation and comparison for different operational scenarios, rather than provision of specific or consistent costs for the various components and systems. Economic modelling of potential cutting systems to-date is not exhaustive and does not fully integrate all of the associated benefits (and compromises) that a cutting system may provide. It is recommended that more comprehensive analysis is conducted, particularly to give further consideration to the potential value offered by the identified high-value opportunities for specific mining operations. Also, the potential for automation of cutting systems likely brings substantial efficiencies and should be examined in detail to establish potential operational and costs benefits.

ACKNOWLEDGMENTS

The authors gratefully acknowledged for their assistance and support with this project:

- **Komatsu Mining Corp**: Research Partner
- **ACARP**: Research sponsor and team of Industry Monitors

REFERENCES


Goswami, T. and Brent, G. 2015, Blasting approaches to increase mine productivity and reduce greenhouse gas emissions in surface coal mining, 11th International Symposium on Rock Fragmentation by Blasting, Sydney, Australia.


DETERMINING ROCK ELASTIC PARAMETERS USING A NEW TRUE-TRIAXIAL-BASED TECHNIQUE

Robert Purser¹, Mutaz El-Amin Mohmoud², Mehdi Serati³ and Zhongwei Chen⁴

ABSTRACT: Rock elastic parameters (e.g., elastic modulus and Poisson’s ratio) are the most critical design features in geotechnical engineering since intact and rock masses generally respond in an elastic fashion up to the yield point when under complex three-dimensional stress-strain states. For instance, when a low-porosity rock is subjected to a uniaxial loading condition, it responds elastically up to the Crack Initiation (CI) point which is defined as the onset of stress-induced damage after the closure of pre-existing cracks. The CI stress level can then be used as an estimate for designing a rock structure against in-situ spalling if the elastic properties of the material are known. However, measurement of elastic parameters for rocks could readily become very challenging when adopting conventional methods that heavily involve strain gauging of rock samples. This study proposes and investigates a new technique based on step-loading of a rock sample under true-triaxial (polyaxial) loading conditions to measure rock elastic parameters without strain-gauging. The results obtained with the new test method are first compared with the traditional techniques, and future work is suggested to validate the method further with anisotropic and orthotropic solids.

INTRODUCTION

The study of stress-strain behaviour of rock is of great importance when designing safe excavations, but requires an in-depth understanding of the force, stress, and strain states to which the rock is exposed (Wittke, 2014; Brady and Brown, 2006). However, unlike metals, rocks are relatively more brittle and fracture in nature without significant plastic deformation prior to failure (Masoumi et al., 2017; Bahaaddini et al., 2019; Serati and Williams, 2019). That is, the energy received by a rock sample up to the yield point could be as high as 85%-95% of the total amount of energy received by the material up to the rupture (failure) point. Hence, rock elastic parameters (elastic modulus E and Poisson’s ratio ν in particular) play a far more important role in the response of rock masses to underground excavations and are typically derived from the elastic zone of the stress-strain curve captured through Brazilian tensile strength (BTS) or Uniaxial Compressive Strength (UCS) testing of rock samples (Małkowski and Ostrowski, 2017).

If any plane in the material is a plane of elastic symmetry, then the material is said to be isotropic, which means that the number of independent elastic constants used to describe its elastic constitutive behaviour is reduced to two parameters only, i.e. E and ν (Ding et al., 2006). Depending on the symmetry and hence the number of elastic constants, rock can be classified into four key categories; namely (i) perfectly isotropic, (ii) transversely isotropic, (iii) orthotropic, (iv) and anisotropic as presented in Table 1 (Wittke, 2014).

If any plane in the material is a plane of elastic symmetry, then the material is said to be isotropic, which means that the number of independent elastic constants used to describe its elastic constitutive behaviour is reduced to two parameters only, i.e. E and ν (Ding et al., 2006). Depending on the symmetry and hence the number of elastic constants, rock can be classified into four key categories; namely (i) perfectly isotropic, (ii) transversely isotropic, (iii) orthotropic, (iv) and anisotropic as presented in Table 1 (Wittke, 2014). It can be seen that the elastic moduli vary from 21 to 5 when the symmetry condition changes from general anisotropic to transversely isotropic. Nevertheless, intact rocks with planar grain structures often exhibit significantly lower E perpendicular to the planar structures (Wittke, 2014). The elastic behaviour of such rocks is described by transverse isotropy which increases the number of elastic constants in the compliance and elasticity matrix. The more elastic moduli that are required to describe the elastic behaviour, the more arrays of strain gauges are needed and the more cumbersome the procedure would be.

Rock elastic parameters are typically measured using the Brazilian Tensile Strength (BTS) and the Uniaxial Compressive Strength (UCS) tests through strain gauging of rock samples (Jianhong et al., 2009; Acar et al., 2014). However, given rock structures are almost exclusively exposed to a three-dimensional (3D) stress condition in practice, determining rock elastic behaviour on a 3D-scale is expected to present rock in-situ conditions more reliably. The true triaxial test (TTT) method was first introduced in the rock mechanics literature as a rock testing method after the successful measurement of rock samples during underground excavations and other rock projects.

¹ Undergraduate Student, The University of Queensland. Email: r.purser@uq.net.au Tel: +61 7 3365 3742
² PhD Student, The University of Queensland. Email: m.mahmoud@uq.net.au Tel: +61 7 3365 3742
³ Lecturer, The University of Queensland. Email: m.serati@uq.edu.au Tel: +61 7 3365 3911
⁴ Senior Lecturer, The University of Queensland. Email: zhongwei.chen@uq.edu.au Tel: +61 7 3365 3742
implementation of the Mogi-type true triaxial system in 1970 (Mogi, 1970). However, despite numerous studies on rock behaviour under true triaxial conditions being conducted since then, limited reports are available for the determination of the elastic parameters of rock and rock-like materials under general TTT stress states (Vida et al., 2020; Tiwari and Rao, 2004; Oku et al., 2007). More recently, step-compression testing was introduced by Chen et al., (2020) to determine the directional mechanical properties of some coal samples. This study aims to further quantify symmetrically the directional mechanical properties of isotropic granite and sandstone samples using a polyaxial testing rig recently commissioned at the Geotechnical Engineering Centre (GEC) of the School of Civil Engineering at the University of Queensland. To validate the results, further tests were also conducted under BTS and UCS loading conditions as benchmark points.

Table 1: Symmetry and elastic constants of different rock types (Wittke, 2014)

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Number of Elastic Constants</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>General anisotropic rock</td>
<td>21</td>
<td></td>
</tr>
<tr>
<td>Orthotropic rock</td>
<td>3 Young’s moduli, 3 Poisson’s ratios and 3 shear moduli</td>
<td></td>
</tr>
<tr>
<td>Transversely isotropic rock</td>
<td>2 Young’s moduli, 2 Poisson’s ratios and 1 shear modulus</td>
<td></td>
</tr>
<tr>
<td>Perfectly isotropic rock</td>
<td>1 Young’s modulus and 1 Poisson’s ratio</td>
<td></td>
</tr>
</tbody>
</table>

**EXPERIMENTAL DESIGN**

Granite and sandstone bulk samples were extracted from a quarry in Queensland, Australia, and then were cut and cored using the cutting and coring machine available at the GEC to the desired dimensions. Samples were then ground to right circular cylinders, discs and cubes in order to maintain smooth surfaces with the required perpendicularly, as shown in Figure 1.

- Brazilian disc samples (54 mm diameter and 27 mm thickness)
- UCS cylinder samples (40 mm diameter and 100 mm height)
- True triaxial cubes (50 mm cubic samples)
The BTS test is widely accepted to determine rock indirect tensile strength (Serati and Williams, 2015; Serati et al., 2017; Serati et al., 2018) where the ISRM suggests using an arc to apply a distributed load that has an angle ranging between 10-15° as shown in Figure 2. To measure the elastic modulus (E) and Poisson’s ratio (v) at the centre of the Brazilian disc with strain gauges, Hondros (1959) proposed the following formulations:

\[
v = \frac{3\varepsilon_\theta + \varepsilon_r}{3\varepsilon_r + \varepsilon_\theta}
\]

(1)

\[
E = \frac{2P(1-v^2)}{\pi D t (\varepsilon_{\theta y} + v \varepsilon_{r y})} = -\frac{6P(1-v^2)}{\pi D t (\varepsilon_{r y} + v \varepsilon_{\theta y})}
\]

(2)

Where \( \varepsilon_\theta \) is the tangential strain at a point situated at \((r, \theta)\) (in the polar coordinate system), \( \varepsilon_r \) is the radial strain at a point situated at \((r, \theta)\), \( P \) is that load applied on a width \(a\) of the loaded section of the rim acting on a circular element with a thickness \(t\) and diameter \(D\), \(2\alpha\) is the angle from the origin to a loaded section of the rim, \( \varepsilon_{\theta y} \) and \( \varepsilon_{r y} \) are the tangential and radial strain at a point situated on the OY-axis, as shown in Figure 2; respectively. Outside the centre of the Brazilian disc, the radial \( \sigma_r \) and tangential \( \sigma_\theta \) stresses at any direction and orientation of strain gauges can be calculated through a Fourier series (Hondros, 1959). Therefore, the elastic modulus and Poisson’s ratio at any location of the strain gauge around the disc can be calculated as follows:

\[
E = \frac{\sigma_\theta - v \sigma_r}{\varepsilon_\theta} \quad \text{or} \quad E = \frac{\sigma_r - v \sigma_\theta}{\varepsilon_r}
\]

(3)

\[
v = \frac{\sigma_r \varepsilon_\theta - \sigma_\theta \varepsilon_r}{\sigma_\theta \varepsilon_\theta - \sigma_r \varepsilon_r}
\]

(4)
To measure the localized strain at different locations on the Brazilian disc, 5 mm two-element rosette strain gauges (vertical and horizontal element) stacked with a circular backing were chosen so that the granite and sandstone grain size would not affect the results. These strain gauges were used because they were readily available. The strain gauges were then arranged so that the BTS discs had each eight strain gauges in different locations on the surface of the disc (see also Figure 3a). An image processing software (Phantom Camera Control, PCC) was then used to measure the exact distance and angles of the strain gauges, and hence to determine the localized stress and strain precisely as illustrated in Figure 3b.

![Image](image1.png)

(a) (b)

**Figure 3: BTS setup of (a) strain gauge pattern (b) Phantom software instant measurement**

The UCS is the most common, conventional, and simple test to determine the strength of rock samples. The samples were prepared as per the guidelines of the ISRM suggested methods for determining the uniaxial compressive strength and deformability of rock materials (1979). The strain gauges of a rosette type which contain vertical and horizontal elements were arranged so that the UCS sample had four in total equally spaced in the middle of the cylinder where the elastic parameters can be measured directly. The elastic modulus is simply that applied vertical stress ($\sigma_v$) which causes a vertical strain ($\varepsilon$) captured by the vertical element of the rosette type strain gauge. Whereas, the Poisson's ratio is simply the ratio between the horizontal to vertical strain measured by both elements of the strain gauges.

The available true-triaxial test frame at UQ GEC was further used in this study which is a multi-function apparatus capable of performing 3D orthogonal stress loading, permeability test, hydraulic fracking, and thermo-mechanical testing of solids under an elevated temperature of up to 100 °C (Figure 4). The system can apply up to 340 MPa on a 50 mm cubic sample and is configured to adapt three different cubical sizes of 50, 100, and 200 mm (Serati et al., 2020; Mutaz et al., 2020).

![Image](image2.png)

**Figure 4: TTT at the University of Queensland (Mutaz et al., 2019)**
Granite and sandstone are generally isotropic rocks where their compliance/stiffness matrix has only two elastic constants (one elastic modulus and one Poisson’s ratio). In this work, to determine the elastic parameters of both materials, a step-loading technique is maintained through the TTT by adopting the stress-path condition as presented under Table 2 where the stress increased by 40 and 10 MPa at each step for granite and sandstone, respectively.

Table 2: TTT step-loading path

<table>
<thead>
<tr>
<th>Loading Step</th>
<th>Granite</th>
<th>Sandstone</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \sigma_x (MPa) )</td>
<td>( \sigma_y (MPa) )</td>
</tr>
<tr>
<td>1</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>2</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
<td>60</td>
</tr>
<tr>
<td>4</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>5</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>6</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>7</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>8</td>
<td>100</td>
<td>140</td>
</tr>
<tr>
<td>9</td>
<td>100</td>
<td>140</td>
</tr>
<tr>
<td>10</td>
<td>140</td>
<td>140</td>
</tr>
</tbody>
</table>

The elastic parameters are linked to the stress-strain relationship under the true-triaxial condition in a compliance matrix through the following formulations (Chen et al., 2020):

\[
\begin{align*}
\varepsilon_x &= \frac{1}{E_x} \sigma_x - \frac{u_{yx}}{E_y} \sigma_y - \frac{u_{zx}}{E_z} \sigma_z \\
\varepsilon_y &= \frac{1}{E_y} \sigma_y - \frac{u_{xy}}{E_x} \sigma_x - \frac{u_{zy}}{E_z} \sigma_z \\
\varepsilon_z &= \frac{1}{E_z} \sigma_z - \frac{u_{xz}}{E_x} \sigma_x - \frac{u_{yz}}{E_y} \sigma_y
\end{align*}
\]

According to Chen et al. (2020), the elastic parameters can be measured individually in each direction of loading (X, Y or Z) at each increment of loading of the true triaxial step-loading through the sequence in the following equations:

\[
\begin{align*}
E_x &= \frac{\Delta \sigma_x}{\Delta \varepsilon_x} \quad \text{and} \quad u_{xy} = \frac{\Delta \varepsilon_y}{\Delta \sigma_x} \times E_x \quad \text{and} \quad u_{xz} = \frac{\Delta \varepsilon_z}{\Delta \sigma_x} \times E_x \\
E_y &= \frac{\Delta \sigma_y}{\Delta \varepsilon_y} \quad \text{and} \quad u_{yx} = \frac{\Delta \varepsilon_x}{\Delta \sigma_y} \times E_y \quad \text{and} \quad u_{yz} = \frac{\Delta \varepsilon_z}{\Delta \sigma_y} \times E_y \\
E_z &= \frac{\Delta \sigma_z}{\Delta \varepsilon_z} \quad \text{and} \quad u_{xy} = \frac{\Delta \varepsilon_y}{\Delta \sigma_z} \times E_z \quad \text{and} \quad u_{xz} = \frac{\Delta \varepsilon_x}{\Delta \sigma_z} \times E_z
\end{align*}
\]

RESULTS AND ANALYSIS

In order to measure the indirect tensile strength (\( \sigma_t \)) and the unconfined compressive strength (\( \sigma_c \)) of the tested granite and sandstone, BTS and UCS tests were carried out accordingly. The BTS test was carried out by using an Instron 4505 load frame with a maximum capacity of 100 kN, loading rates of 100 N/sec, and a 25% stress sensitivity. The test results are presented in Table 3. The tensile strength of granite was in the range of 8.50 - 9.70 MPa while it is 3.50 - 3.70 MPa for sandstone. Due to the expected higher strength of granite, the UCS tests were carried out on an an MTS loading frame with a maximum capacity of 1000 kN, loading rates of 120 N/sec, and a 25% stress sensitivity. The UCS of granite ranged between 151 – 160 MPa whereas it is 40 – 49 MPa for sandstone (Table 3). All measurements were made at atmospheric pressure and room temperature.

Since cracks usually initiate at around 40% of the maximum applied load in a UCS test, 40% of the tensile and compressive strength was designed for the Brazilian and UCS test on the specimens with strain gauges for the sake of determining their elastic properties (see Figures 3 and 5). Hondros’ equations 3 and 4 were implemented to determine E and \( \nu \) of the Brazilian test samples based on the arrangement of the strain gauges as explained above. The elastic modulus and Poisson’s ratio of the
UCS samples were further measured directly from the readings of the vertical strain gauges ($\varepsilon_{V,SG}$) and horizontal strain gauges ($\varepsilon_{H,SG}$) through the following direct equations:

$$E = \frac{\sigma_V}{\varepsilon_{V,SG}}$$

$$\nu = -\frac{\varepsilon_{H,SG}}{\varepsilon_{V,SG}}$$

Table 3: Tensile strength and Unconfined strength of the BTS and UCS samples

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Granite (MPa)</th>
<th>Sandstone (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_t$</td>
<td>$\sigma_c$</td>
</tr>
<tr>
<td>1</td>
<td>9.56</td>
<td>3.51</td>
</tr>
<tr>
<td>2</td>
<td>9.70</td>
<td>3.77</td>
</tr>
<tr>
<td>3</td>
<td>8.53</td>
<td>3.72</td>
</tr>
</tbody>
</table>

Table 4: Average elastic parameters of the granite samples using BTS and UCS tests

<table>
<thead>
<tr>
<th>Brazilian Tensile Test</th>
<th>Uniaxial Compression Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cycle</td>
<td>$E$ (GPa)</td>
</tr>
<tr>
<td>------</td>
<td>----------</td>
</tr>
<tr>
<td>1</td>
<td>51.04</td>
</tr>
<tr>
<td>2</td>
<td>53.04</td>
</tr>
<tr>
<td>3</td>
<td>54.94</td>
</tr>
<tr>
<td>4</td>
<td>56.23</td>
</tr>
<tr>
<td>Average</td>
<td>53.81</td>
</tr>
<tr>
<td>SD</td>
<td>2.27</td>
</tr>
<tr>
<td>CV</td>
<td>4.21%</td>
</tr>
</tbody>
</table>

Figure 5: Elastic parameter measurements through strain gauges (a) Brazilian test (b) UCS

Four cycles of loading were carried out for each set of BTS and UCS tests, and the average elastic parameters were calculated together with the standard deviation (SD) and coefficient of variation (CV) as listed in Tables 4 and 5. It can be deduced from both tables that, within each set of cycles, the elastic parameter values are very close with low CVs. The average elastic modulus of granite was measured as 53.8 GPa and 47.6 GPa from the Brazilian and UCS test; respectively. The difference could be potentially due to the different grain distribution across the loading axis in each test. Nevertheless, in both tests, the same Poisson’s ratio of 0.40 was confirmed for granite. However, very similar values for the elastic modulus of 22 GPa and Poisson’s ratio of around 0.20 with the Brazilian and UCS tests were confirmed for sandstone.
Table 5: Average elastic parameters of the sandstone samples for BTS and UCS tests

<table>
<thead>
<tr>
<th>Cycle</th>
<th>E (GPa)</th>
<th>ν</th>
<th>Cycle</th>
<th>E (GPa)</th>
<th>ν</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>22.83</td>
<td>0.27</td>
<td>1</td>
<td>21.65</td>
<td>0.21</td>
</tr>
<tr>
<td>2</td>
<td>22.77</td>
<td>0.26</td>
<td>2</td>
<td>23.00</td>
<td>0.20</td>
</tr>
<tr>
<td>3</td>
<td>21.93</td>
<td>0.27</td>
<td>3</td>
<td>23.07</td>
<td>0.20</td>
</tr>
<tr>
<td>4</td>
<td>22.52</td>
<td>0.27</td>
<td>4</td>
<td>23.18</td>
<td>0.20</td>
</tr>
<tr>
<td>Average</td>
<td>22.39</td>
<td>0.27</td>
<td>Average</td>
<td>22.73</td>
<td>0.20</td>
</tr>
<tr>
<td>SD</td>
<td>0.48</td>
<td>0.005</td>
<td>SD</td>
<td>0.72</td>
<td>0.007</td>
</tr>
<tr>
<td>CV</td>
<td>2.14%</td>
<td>1.87%</td>
<td>CV</td>
<td>3.17%</td>
<td>3.45%</td>
</tr>
</tbody>
</table>

The true triaxial loading condition is used to mimic the in-situ condition when the underground rock is under a truly polyaxial loading condition. Smoothed cubic granite and sandstone samples of 50 mm were placed into the true triaxial loading cell as shown in Figure 6 by maintaining a seating load of 8 kN to ensure a proper sample contact and avoid sample eccentricity at the initial stage of loading. Then, the step-loading condition, as listed in Table 2 above, was applied by maintaining an increment of 40 MPa for granite and 10 MPa for sandstone. Figure 7 presents the step-loading sequence for granite samples as an example.

![Figure 6: Granite and sandstone cubes placed inside the true triaxial cell](image)

![Figure 7: Step-loading conditions for granite samples](image)

To measure the elastic parameters in the direction of major (Z), intermediate (Y), and minor (X) principal stresses, equations 6 to 8 were implemented. To measure the actual sample deformations, the cell deformation in each direction of X, Y, and Z were deducted from the machine’s LVDTs readings. Table 6 summarizes the elastic modulus and Poisson’s ratio of the tested granite and sandstone. It is obvious that there is a high standard deviation between the elastic modulus and Poisson’s ratio values of granite for X, Y, and Z-axis compared to sandstone, and this could be
attributed to the inhomogeneity distribution of grain particles in granite compared with that in sandstone. This inhomogeneity has a pronounced effect on the determination of E and \( v \), especially in the Z-direction. The average elastic modulus and Poisson’s ratio of granite in all three axes is 60 GPa and 0.45; respectively. Nevertheless, the average value is relatively close to that estimated by both Brazilian and UCS tests with a relative difference of 10-20%. The average elastic modulus and Poisson’s ratio of sandstone in all axes is 16 GPa and 0.13; respectively. Compared to the elastic parameters determined by the Brazilian and UCS tests, a relative difference of 30% was reported. That is, the TTT can be used to determine the elastic parameters (i.e. elastic modulus and Poisson’s ratio) with the confidence of up to 90%.

Table 6: Average elastic parameters under step-loading of the TTT

<table>
<thead>
<tr>
<th>Granite</th>
<th>Sandstone</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axis</td>
<td>E (GPa)</td>
</tr>
<tr>
<td>X</td>
<td>44.89</td>
</tr>
<tr>
<td>Y</td>
<td>55.92</td>
</tr>
<tr>
<td>Z</td>
<td>80.28</td>
</tr>
<tr>
<td>Average</td>
<td>60.36</td>
</tr>
<tr>
<td>SD</td>
<td>18.11</td>
</tr>
</tbody>
</table>

CONCLUSIONS

This study aims to present the results of a systematic approach to measure rock elastic parameters under true triaxial loading conditions without strain gauging. Sandstone and granite were chosen as isotropic rocks and were prepared and tested according to standard recommendations. First, Hondros’ equation was implemented to determine the elastic modulus (E) and Poisson’s ratio (\( v \)) of Brazilian samples with the selected rock types. Then, UCS specimens were prepared and tested according to the ISRM recommendations to re-measure the selected elastic parameters under unconfined compressive stress conditions. Finally, a newly developed step-loading technique was adopted by increasing the major, intermediate, and minor principal stresses gradually at equal intervals.

Since the BTS and UCS results are quite similar, the results indicate that the new true-triaxial-based method’s results are in quite good agreement with the benchmark test results of UCS and BTS. However, the new method also eliminates the need for strain gauge preparation and calibration, which could be highly favoured where a polyaxial testing machine is available. An endeavour towards validating the new step-loading method for anisotropic and transversely isotropic material is further underway.

REFERENCES


AXIAL BEHAVIOUR OF ROCK BOLTS–PART (A)
EXPERIMENTAL STUDY

Hadi Nourizadeh¹, Sally Williams¹, Ali Mirzaghorbanali¹, Kevin McDougall¹, Naj Aziz² and Mehdi Serati³

ABSTRACT: Several experiments were carried out to investigate the effect of curing time and water to grout ratio on the ultimate load capacity of fully grouted rock bolts subject to pulling out loads. For this purpose, various samples were cast with water to grout ratios of 30%, 36% and 40%. Pull-out tests were conducted on samples with different curing times, ranging from 7 to 28 days. Results indicate that the peak value of the pull out load increased as curing time increased. In contrast, increasing water to grout ratio weakens the pulling out resistance of fully grouted rock bolts.

INTRODUCTION

Fully-grouted rock bolts are widely used to strengthen and support rock walls and tunnels in mining and civil industries (Li, 2017). These reinforcing systems can be designed as temporary or permanent reinforcement to mitigate the risks associated with tunnelling, namely rock falls and structural collapse (Cao, 2012). A typical grouted rock bolt consists of internal and external fixtures such as a ribbed steel bar installed and encapsulated by cementitious grout in a drilled hole. Once grout cures, chemical adhesion, friction and mechanical interlocking provide the bond strength between the grout and bolt (Cao et al., 2016). System debonding may occur at the grout, bolt-grout interface, grout-rock interface or surrounding rocks (Li and Stillborg, 1999). However, failure at the bolt-grout interface is the most common mode according to previous experimental and in-situ studies. Rock bolt systems develop forces in response to rock deformation and displacement. Different stresses are acting on rock bolt systems; these include tension, shear, compression and rotation. Figure 1 illustrates six different loading types that can occur on a rock bolt system depending on the geometric properties of joints and the bolt’s spatial position relative to the joint. In this figure, rotation was not taken into account, which creates extra complexities (Thompson et al., 2012).

As illustrated in Figure 2, tensile loads within a rock bolt system occur when a discontinuity dilation displaces a rock mass. Tension is produced between stable and unstable rock regions as shown in Figure 3, thus creating shear loads between the rock-grout interface. The tensile strength of steel, the

---

¹ School of Civil Engineering and Surveying, University of Southern Queensland, email: hadi.nourizadeh@usq.edu.au
² School of Civil, Mining and Environmental Engineering, University of Wollongong, email: naj@uow.edu.au
³ School of Civil Engineering, University of Queensland, email: mehdi.serati@uq.edu.au
bond strength of the bolt-grout interface and the bond strength of the grout-rock interface in combination hold the unstable region in place.

![Figure 2: Loading of reinforcement caused by block translations (Windsor, 1997)](image)

Figure 2: Loading of reinforcement caused by block translations (Windsor, 1997)

Failure of rock bolt systems can occur in five modes as shown in Figure 4 (Hutchinson and Diederichs, 1996). Mode A represents breakage of a bolt shaft; this form of failure is rare because it requires the bond strength between the grout and bolt to be greater than the bolt's tensile strength. Many studies confirmed that the failure of fully grouted rock bolts occurs at the bolt grout interface as shown in mode B (Chen et al., 2020, Thenevin et al., 2017, Paul Hagan, 2014, Cao, 2012). This type of failure is due to the small contact area between the bolt and grout. However, if the rock strength is relatively weak or if the borehole diameter is relatively small, the rock-grout interface failure is more likely to occur as shown in mode D. Modes C and E depict failure at the grout-rock interface and the surrounded rock respectively (Cao, 2012, Paul Hagan, 2014, Hutchinson and Diederichs, 1996).

![Figure 3: Load transfer concept (Thompson et al., 2012)](image)

Figure 3: Load transfer concept (Thompson et al., 2012)

![Figure 4: Different failure modes of cable bolts (Hutchinson and Diederichs, 1996)](image)

Figure 4: Different failure modes of cable bolts (Hutchinson and Diederichs, 1996)

Grout acts as a medium to transfer the initiated stresses from rock masses to bars. The load-bearing capacity of this reinforcing system depends on numerous factors. The most effective ones are the bolt tensile strength, bolt surface profile, bolt length and diameter, hole diameter, annulus thickness and grout mechanical properties.

There are different types of grout and each has its unique mechanical characteristics. Grout cohesion, shear strength and angle of friction play an essential role in rock bolt systems. Up to now, a large
number of experimental studies have been conducted to investigate the effect of grout mechanical properties on the efficiency of fully-encapsulated rock bolts. Hyett et al. (1995) performed a number of tests on fully-grouted strand cable bolts indicating the sensitivity of the bond strength to the grout properties. Kilic, et al. (2002) conducted numerous pull-out tests on steel bars, encapsulated into basalt rock to study the effects of grout’s mechanical properties on the bearing capacity. In this study, besides mechanical properties, bolt length, bolt diameter, bonding area, water to grout ratio and curing time were also investigated. According to the test results, an increase in the bolt diameter and bolt length results in a linearly increase in the ultimate pull-out load. On the contrary, an increase in the water to cement ratio results in a decrease in the ultimate pull-out load. The bond strength of fully encapsulated rock bolts is basically frictional and depends on the shear properties of the grout and bolt-grout or grout-rock interfaces such as cohesion, frictional angle and interface roughness. The results of different studies show that Uniaxial Compressive Strength (UCS) and shear strength of grouts play vital roles in the determination of rock bolts and cable bolts bearing capacity (Hyett et al., 1992, Benmokrane et al., 1995, Miller and Ward, 1998, Rao Karanam and Dasyapu, 2005, Bin et al., 2012, Teymen and Kilic, 2018, Zhang et al., 2020). Kilic et al. (2002) observed that the bond strength increases logarithmically with an increase in UCS and shear strength of grouts. In another research study, Teymen and Kilic (2018) investigated the axial stress distribution of rock bolts by applying strain gauges on bolts. The bolts were installed in high strength rock media. Mortar with different mechanical properties was prepared using admixtures with various water to cement ratios. It was concluded that grouts’ mechanical characteristics significantly affect shear and axial stress distributions, and consequently the ultimate bearing capacity. Aziz et al. (2017) carried out numerous experiments on commercial grout products (Stratabinder and BU-100) investigating the effects of water to cement ratio and curing time on grout mechanical properties. The results show that an increase in water to grout ratio from 28% to 42% causes a 43% reduction in UCS of Stratabinder samples. In this study, the effect of water to grout ratio is also noted on grouts’ shear strength. Kim et al. (2019) evaluated the performance of fully-encapsulated rock bolts using cementitious grout and resin. It was concluded that the grouted bolts’ ultimate bearing capacity is a function of water to grout ratio and curing time. Li (2017) illustrated that water to grout ratio is a crucial factor in determination of critical length. For instance, for a sample with water to cement ratio of 0.4, the critical embedment length is 32 cm. This length increases to 36 cm for a water to cement ratio of 0.50. Experiments also indicated a correlation between water to cement ratio and grout UCS as shown in Figure 5. Chang et al. (2017) conducted a series of tests indicating that grouts’ UCS influences the interface bond strength. They showed that when UCS of grout increases from 30 to 60 MPa, the interface bond strength increases from 3.8 to 11.5 MPa (Figure 6).

This paper is the first of two-paper publication in this proceeding. In Part (A), the effects of water to grout ratio and curing time on the axial behaviour of rock bolts were studied by conducting more than 36 pull-out tests. In the subsequent paper (Part B), FLAC 2D was incorporated to simulate numerically the pulling out resistance of fully grouted rock bolts subject to confinement stresses, and by considering various rock bolts surface roughness.
SAMPLE PREPARATION

Short encapsulation was employed to study debonding mechanisms of rock bolts, ensuring uniform distribution of shear stress along the bolt-grout interface (Benmokrane et al., 1995, Blanco Martín et al., 2011). For this purpose, grout samples were prepared with water to grout ratios of 30%, 36%, and 40%, and samples were left to cure for 7, 14, 21 and 28 days. The mixing process of batches (grout and water) followed the instruction recommended by the manufacturer, the Minova Stratabinder. Pull-out loading test was repeated three times on each curing-time and water-grout ratio (36 pull-out tests in total) to obtain accurate and reliable results, and then the average values were taken into account to interpret the results. Firstly, bars with 18 mm in diameter and 500 mm in length were installed in the centre of steel pipes, and then the annulus area between the pipe wall and bar was filled using the mixed grout. Steel pipes with an internal diameter of 50 mm and thickness of 2 mm were used to simulate the surrounding rock and confining material. Pipes were 100 mm in length (equal to the bar encapsulation length) without any internal thread. Samples were left in the laboratory to cure, based on the desired curing times as shown in Figure 7.

Figure 6: Relation between critical bond strength and grout compressive strength (Chang et al., 2017)

Figure 7: Samples prepared for the pull-out test
RESULTS ANALYSIS

Pull-out tests were conducted by setting the servo-controlled MTS 100 kN tensile machine to a strain rate of 1 mm per minute (Figure 8). Samples were threaded into an attachment and subsequently gripped from both the top and bottom by the machine's jaws. Sample arrangement in the testing machine is shown in Figure 8. The load-displacement interaction was monitored on integrated computer software connected to the pull-out machine. The test was repeated weekly on samples cured in 7, 14, 21 and 28 days.

Results of pull-out tests show that the curing time and water to grout ratio have definitive influences on the bearing capacity of encapsulated rock bolts, albeit some unusual behaviours can be ascribed to the grout mixing procedures (Figures 9 to 11). Figure 12 is a comparative chart showing the ultimate pulling-out loads for the samples with different water to grout ratios and curing times. It is evident that under each curing time condition, the peak values belong to the samples with a 30% of water to grout ratio. It is inferred from the test results that the ultimate bearing load of samples for each curing time decreases with an increase in the water content. For example, samples with 40% of water content fail with lower pulling load (35.6 kN) when compared to the samples prepared with 30% of water content (45.4 kN). Moreover, a general increasing trend in ultimate pulling out resistance is observed with an increase in the curing time. For instance, samples with 36% of water to grout ratio cured for 7 days failed at 33.2 kN whereas samples cured for 28 days failed at 39.95 kN. As mentioned above, in some cases there are strange behaviours observed in the pulling-out results. Figure 9 shows the ultimate load for samples with a 14 day curing time which is slightly higher than those with a 21 and 28 day curing time. This might be associated with the preparation and curing process. The observations also demonstrate that variables, including water to grout ratio and curing time have noticeable effects on the system's rigidity against shear displacement. According to the results, samples with a water to grout ratio of 30% and a 28 day curing time possess the highest shear stiffness (6.94 kN/mm), however, the lowest belongs to samples with a water to grout ratio of 40% and a curing time of 7 days (3.82 KN/mm). Figure 13 represents an interactive three-dimensional plot between water to grout ratio, curing time and shear stiffness for the grouted bolts with a 100 mm of encapsulation. Figure 14 shows the ultimate bearing loads subjected to the samples and the associated displacement at the maximum load point. From this graph, it is concluded that the highest magnitude of energy is stored in samples with water to grout ratio of 30% in comparison to other tests.

Photos of samples taken after the pull-out test were collected showing tangential and radial cracks (Figure 15). Tangential cracks initiated from the bolt and extended to the steel pipe whereas radial cracks formed a ring around the bolt shaft. All samples showed both types of cracks to various extents. Nevertheless, no correlation between grout properties and the crack type was noted.
Figure 9: Load-Displacement curves for samples with water to grout ratio of 30%

Figure 10: Load-displacement curves for samples with water to grout ratio of 36%

Figure 11: Load-displacement curves for samples with water to grout ratio of 40%
Figure 12: Ultimate bearing capacity based on curing time and water to grout ratio (w:g)

Figure 13: Effects of curing time and water to grout ratio (w:g) on shear stiffness of fully grouted rock bolts

Figure 14: Ultimate loads and relevant displacement
CONCLUSION AND STUDY LIMITATIONS

In this study, experiments were carried out to evaluate the effects of the grout preparation process such as water to grout ratio and curing time on grouted rock bolts' axial bearing capacity. These variables are considered as the main influential factors on the mechanical properties of grouts. The following main conclusions are drawn from this research study:

- The proportion of water in the grout mixture plays an influential role in the ultimate axial bearing capacity of rock bolting systems regardless of other factors. The same effect was observed with respect to the curing time. However, the experimental data showed unusual behaviours in a few cases.

- In all tests, the positive effect of decreasing the water to grout ratio on the ultimate load is evident. As the water to grout ratio decreases from 40% to 30%, the axial bearing capacity increases by 20% on average for each curing time period.

- The maximum pulling-out load (48 KN) was measured for the samples cured for 28 days with a 30% of water to grout ratio.

- In terms of failure mechanisms, tangential and radial cracks were observed in all samples regardless of water content and curing time.

- This study was carried out to compare the effects of water to grout ratio and curing time on axial behaviour of rock bolts and may not represent field conditions. 2 mm of steel pipe used as the confinement wall (Figure 8) may cause lateral dilation, thus, affecting the ultimate pulling out capacity. It is recommended to carry out similar research study with confinement tube of 9 mm or more thickness, avoiding lateral displacement during experiment.

ACKNOWLEDGMENTS

Authors' would like to cordially acknowledge Minova Australia’s in-kind support in various stages of this research study. Grateful thanks also shall be extended to Mr Robert Hawker for his valuable advices and support.
REFERENCE


AXIAL BEHAVIOUR OF ROCK BOLTS–PART (B)  
NUMERICAL STUDY

Hadi Nourizadeh¹, Ali Mirzaghurbanali¹, Kevin McDougall¹, Naj Aziz² and Mehdi Serati³

ABSTRACT: Axial load behaviour of rock bolts was studied using FLAC commercial software in a two-dimensional framework. The numerical model was calibrated using experimental pull out data. Effects of confinement stresses and rock bolt surface roughness on the axial load behaviour were investigated incorporating the calibrated numerical code. Results indicated that the pull out resistance increases with an increase in confinement stresses and surface roughness.

INTRODUCTION

Fully-encapsulated rock bolts are embedded in the borehole using either resin or cementitious grout. This increase the overall stiffness by generating resistances against axial and shear forces. Sliding blocky rocks or movement of bedding planes induce stress in the grout. The induced stresses are then transferred to the reinforcing element, thus, creating interaction between the bolt and surrounding medium. Eventually, this process produces tensile forces in the bolt, preventing further movement of separated blocks and bedding planes.

LITERATURE REVIEW

In fully grouted rock bolts, failure may occur in various modes including rock-grout interface, grout-bolt interface, rock bolt and surface plate. Nemcik et al. (2014) reported that the most common failure mode is the bolt-grout interface. Relaxation in the confinement stress reduces rock bolt anchorage capacity, particularly at the rock-grout interface. Hyett et al. (1992) carried out split-pipe tests using PVC, aluminium and steel pipes to investigate the effects of confinement stress on the bond capacity of grouted cable bolts. They concluded that the axial bearing capacity of cable bolts increases with an increase in confinement stress. In addition, they reported that the failure mechanisms changed as the confinement stress increased. In another study, Hyett et al. (1995) performed several pull-out tests on encapsulated cable bolts incorporating the modified Hoek cell to simulate confinement stresses. It was noted that confining stress affects the ultimate bearing capacity of encapsulated cable bolts. Blanco Martin et al. (2011) carried out a series of pull-out tests to examine the influence of several factors such as confining stress and the bolt's surface profile. The results revealed that confining stress possesses noticeable effects on the anchoring capacity. It was observed that the radial fractures are more pronounced in low values of confining stress. In another study, Nie et al. (2019) reported that the highest bond and residual strength are achieved with high confining stress.

Hawkes and Evans (1951) carried out pull-out tests showing the distribution of shear (bond) stress along the bolt. They concluded that the load distribution follows an exponential function and the peak takes place before any decoupling occurs. Farmer (1975) conducted theoretical and experimental research on the shear stress distribution along resin encapsulated reinforcement elements and concluded that the mobilised shear resistance is an influential factor in the bond resistance. Li and Stillborg (1999) developed an analytical model for fully encapsulated rock bolts by assuming the peak shear stress occurring a short distance from the loading point, diminishing exponentially to the free end (Figure 1). According to the piecewise function proposed by Li and Stillborg (1999), the shear stress distribution is divided into four sections along fully-grouted rock bolts. These sections include entirely decoupled (A), partially decoupled with a constant bond strength (B), partially decoupled with linearly increasing bond strength (C) and compatible deformation with no decoupling (D) (Figure 2). The proposed model includes some assumptions which may limit the model practicality. Developing a reliable mathematical model to simulate rock bolt bond-stress behaviour is a cumbersome task due to problem complexities. Several tri-linear bond-slip models were presented to simulate the interfacial

¹ School of Civil Engineering and Surveying, University of Southern Queensland, email: hadi.nourizadeh@usq.edu.au
² School of Civil, Mining and Environmental Engineering, University of Wollongong, email: naj@uow.edu.au
³ School of Civil Engineering, University of Queensland, email: mehdi.serati@uq.edu.au
debonding of steel bolting systems (Benmokrane et al., 1995, Ren et al., 2010, Blanco Martín et al., 2011). Benmokrane et al. (1995) proposed a bond-slip model for the interfacial constitutive behaviour of three linear sections (I, II, III) as shown in Figure 3. Ren et al. (2010) represented the full range behaviour of fully grouted rock bolts to define the shear stress and bond-slip relation along the interface, axial load in the bolt and load-displacement relationship by applying the tri-linear bond-slip model. The proposed full-range constitutive model consists of five distinguished stages, namely, elastic, elastic-softening, elastic-softening-debonding, softening-debonding and debonding. Also, closed-form solutions were proposed to calculate the axial load distribution, interfacial shear stress distribution, load-displacement relationship and bond-slip relationship. This model is not straightforward to apply in practice due to its complexity. Blanco Martín et al. (2011) modified Ren’s model to predict the full-range mechanical behaviour of rock bolts. In the presented approach, the input parameters are bolt elastic modulus, bolt radius, displacement of the free end and the constitutive law governing the bolt-grout interface. In this model, shear stress is a function of displacement and tangential interfacial stiffness. Ma, et al. (2013) developed a non-linear bond-slip model (Figure 4) by applying the slip function presented by Zhou et al. (2010).

![Figure 1: Shear stress along a full-encapsulated bolt subjected to a pull-out load before decoupling occurs (Li and Stillborg, 1999)](image1)

![Figure 2: Distribution of shear stress along a full-encapsulated bolt subjected to a pull-out load after decoupling (Li and Stillborg, 1999)](image2)

![Figure 3: Idealised bond-slip relationship at the bolt-grout interface (Benmokrane, et al., 1995)](image3)
The interaction between the components in the rock bolting systems subjected to a tensile force is complicated. Therefore the conventional approaches may not be capable of studying this behaviour in details. On the other hand, computer simulations are powerful tools dealing with complexity in Engineering problems. Rao Karanam and Dasyapu (2005) conducted extensive experimental pull-out and push-out tests to investigate fully grouted rock bolt performance with variations in the bolts diameter and length and water to cement ratio. According to the collected experimental data, detailed numerical analysis using the ALGOR computational package was carried out to analyse displacement, stress and strain distribution along the interface. Ghadimi et al. (2014) developed a three-dimensional numerical code based on finite element method to investigate the behaviour of rock bolts in jointed rocks and to validate the proposed analytical solutions. Nemcik, et al., (2014) developed a numerical model for fully grouted rock bolts loaded in tension by applying a non-linear bond-slip relationship using FLAC software. Nie, et al. (2014) established several rock bolt models by assigning elastic and linear strain-hardening behaviour. Chen, et al. (2018) proposed a numerical model using the commercial code of FLAC2D to determine the maximum induced confinement stress in pull-out tests of different cable bolts. Results indicated that modified cable bolts produce more confinement stress within the rock samples, making them suitable for high in situ stress conditions. Nie, et al. (2019) used two-dimensional Discontinuous Deformation Analysis to investigate rock bolt behaviour under various conditions. The bond-slip curves obtained from the numerical models follow trends that can be translated to the tri-linear bond-slip model. Parametric studies showed that the ultimate bond strength and residual strength increase as confining stress increases. Yu, et al. (2019) evaluated the behaviour of fully grouted rock bolts by conducting experimental and numerical analysis. The numerical models simulated the failure process under various embedment length and indicated that microcracks occur initially at the loading end expanding toward the free end as the axial load increases. Moreover, the influence of critical embedment length and the friction coefficient on the residual axial stress were analysed. Che, et al., (2020) investigated the behaviour of ribbed bolts installed in soft rocks from micro-scale to macro-scale using the discontinuous element method. The influence of embedment length and confining stress on the ultimate load as well as failure mode were analysed.

As far as can be determined, there are limited numerical simulations investigating the effects of confinement stress and surface roughness on axial load behaviour of fully grouted rock bolts, which are the subject of this paper. This is a companion paper, which complements the part (A) paper, discussing experimental results.

**SIMULATION OF ROCK BOLT AXIAL BEHAVIOUR IN FLAC**

FLAC software was developed by the ITASCA Consulting Group, Inc. based on explicit finite difference concepts and represents several structural reinforcing elements such as rock bolts and cable bolts (Itasca, 2000). FLAC 2D is a robust two-dimension simulation software to assess the
mechanical behaviour of rock bolts. In FLAC, rock bolts are modelled based on the pile element, comprising axial and bending behaviour. The pile element is connected to the grid in the normal and shear directions by spring coupling concepts. The reinforcement element is divided into several segments with the length of $L$ connected at nodal points. The segments are assumed to behave linearly elastic, yielding in the axial direction once the tensile or compressive stress reaches the user-defined tensile or compressive strengths. The shear behaviour of the bolt/grid (grout) interface is defined by the spring-slider system at the nodal points. This behaviour is described by the coupling spring shear stiffness ($cs_{s\text{stiff}}$) in the software once the nodal points move relative to the grid (Equation 1) (Figure 5a).

$$\frac{F_s}{L} = cs_{s\text{stiff}}(u_p - u_m)$$  \hspace{1cm} (1)$$

where, $F_s$ is shear force developing in the shear coupling spring; $L$ is the contributing length of the bolt element; $cs_{s\text{stiff}}$ is the coupling spring shear stiffness; $u_p$ is the axial displacement of the bolt and $u_m$ is the axial displacement of the grid (grout or rock).

The maximum induced shear force due to pull-out load along the bolt-grid is determined by the interface's cohesive strength and frictional resistance along the interface. The following equation determines the maximum shear force (Figure 5b):

$$\frac{F_{s\text{max}}}{L} = cs_{s\text{coh}} + \sigma'_c \times \tan(cs_{sfric}) \times P$$  \hspace{1cm} (2)$$

where $cs_{s\text{coh}}$ is the cohesive strength of the coupling shear spring, $\sigma'_c$ is the mean effective confining stress normal to the bolt element, $cs_{sfric}$ is the friction angle of coupling shear spring and $P$ is the perimeter of the bolt. User-defined tables, $cs_{s\text{ctable}}$ and $cs_{s\text{ftable}}$, are available in FLAC to prescribe the softening behaviour, which is a function of shear displacement for shear coupling spring cohesion and friction angle properties. The tables mentioned above defines a number of tables relating cohesion and friction angle of shear coupling spring to relative shear displacement.

The anchorage capacity and load transfer mechanism in fully grouted rock bolts are affected by the mechanical properties of grouts, bar specifications and in-situ conditions. In this study, the results of pull-out experiments (part (A)) were employed to numerically study the fully grouted rock bolt axial load transfer mechanisms subject to confinement stresses. Within the literature, it is proposed that short encapsulation can be employed to experimentally study the debonding mechanisms. This is aimed to ensure uniform distribution of shear stress along the bolt-grout interface (Blanco Martín et al., 2011, Benmokrane et al., 1995, Tepfers, 1979). For this reason, only 100 mm of the bolt was encapsulated using Stratabinder HS, a cementitious grout produced by Minova Australia. The pull-out test was conducted by setting the servo-controlled MTS 100 kN tensile machine at the strain rate of 1mm per minute, and over the tests, the load-displacement interaction was monitored on integrated computer software connected to the machine. The obtained load-displacement relationships from the pull-out tests are converted to bond-displacement using the following equation:

$$\tau = \frac{P}{\pi d_b L}$$  \hspace{1cm} (3)$$

where, $P$ is the subjected pull load, $d_b$ is the bolt diameter and $L$ is the embedment length. This conversion is valid until the embedment length ($L$) is short enough, and the shear stress is uniformly distributed along the entire bolt.
NUMERICAL MODELLING

Finite element method using FLAC2D was applied with the primary aim of examining the load and displacement developed along the encapsulated bolts. The secondary objective is to examine the effects of confining stress and friction angle on the axial behaviour of fully grouted rock bolts. In order to verify the capability of FLAC2D in simulating the behaviour of fully grouted rock bolt in pulling out conditions, the model was compared and validated with the results of pull-out experiments. After calibration, confining stress ranging from 0 to 7.5 MPa, and bolt-grout interface friction angle ranging between 30-45 degrees were applied. Normal displacement at the top was constrained to simulate testing conditions. Nevertheless, the bolt was free of movement along the vertical axis.

The relevant properties used in the model are the cross-sectional area of 2.44 E-4 m$^2$, Elastic modulus of 200E9 Pa, 5.65E-2 in perimeter, cs_scoh of 4.5E5 N/m and cs_sstiff of 6.89E9 N/m/m. Rock properties were assigned a bulk modulus of 5.00E9 Pa, shear modulus of 3.2 E9 Pa and density of 2000 Kg/m$^3$. In the simulation process, it was assumed that the confining stresses acting on the bolts are isotropic.

Figure 6 illustrates a comparison between the load-displacement curve of the numerical simulation and that of experimental data with a grout curing time of 28 days and water to grout ratio of 36%. The numerical simulation agrees well with the experimental data, capturing the elastic region, peak value and residual conditions. Axial load distribution along the bolt is illustrated in Figure 7. It is noted that the bolt axial load gradually decreases from the loading-end to the free-end. The numerical study also demonstrates that the bond strength increases with confining stress. According to the simulation (Figure 8), the ultimate pulling force increases from 43.55 kN to 48.44 kN when 2.5 MPa confinement stress is exerted to the lateral boundaries. However, the incremental impact of confinement decreases after reaching 5 MPa, as the ultimate pull-out load reaches 55.6 kN when the confinement stress is 7.5 MPa (i.e. lower increment compared to the previous case). Frictional strength also indicates a jump when confining stress increases in the model. Figure 9 depicts the effect of the friction angle (i.e. bolt surface roughness) on the anchorage capacity. As shown, increasing the friction angle from 30 to 45-degree causes the ultimate anchorage capacity to increase from 46.2 kN to 50.6 kN.

Figure 5: a) Shear force against displacement b) Shear strength criterion in FLAC (Itasca, 2000)

Figure 6: Comparison between numerical model and experimental data
Figure 7: Axial load distribution along the bolt

Figure 8: Effects of confinement on the bolt axial behaviour

Figure 9: Effects of surface roughness on bolt axial behaviour
CONCLUSIONS

In this study, FLAC 2D was incorporated to evaluate the effects of confining stress and bolt surface roughness on axial load behaviour of grouted rock bolts. The following main conclusions are drawn from this investigation:

- The numerical model is in close agreement with the experimental data, thus, indicating the capabilities of FLAC 2D in simulating rock bolt axial behaviour.
- The numerical models also showed that the confinement and surface roughness influence the bonding resistance of fully grouted rock bolts subject to axial loading. According to the simulations, the ultimate bearing capacity and residual strength increase with an increase in confining stress and friction angle.
- This study was carried out under the assumption of isotropic confinement conditions. In the field, horizontal stresses may differ from each other. Therefore, it is suggested to carry out further research studies by considering anisotropic confinement conditions.

REFERENCE


FRACTURE PROPAGATION MODE OF COAL UNDER INDIRECT TENSILE STRESSES

Mehdi Serati¹, Hamid Roshan², Ali Mirzaghborbanali³, Mutaz El-Amin Mahmoud⁴ and Thejaswe Valluru⁵

ABSTRACT: This work presents the results of an investigation on the study of fracture pattern in coal under induced tensile stresses using image processing techniques. Several high-speed recordings captured at 5 kHz and above were examined, and three distinct fracture propagation modes were identified. As the coal tensile strength increases, the fracture behaviour was observed to be a spalling-like rupture in the form of a dominant tensile crack accompanied with multiple large secondary shear fractures that break the test specimen apart into several pieces. Combined localised tensile and shear cracks at intact rock bridges within a coal Brazilian sample (with pre-existing defects) was also observed, indicating a very different failure pattern from that in standard recommendations.

INTRODUCTION

Rock burst and spalling is a spontaneous and violent rock failure that occurs (within moments or seconds) commonly in high-stress deep mines in stiff and competent rocks. It produces flakes with sharp edges, flat cutting pieces, and large fragments. According to the literature, initiation and propagation of unwanted tensile cracks is responsible for such catastrophic and disastrous rock failures (Diederichs and Martin, 2010). But, rockburst is only one of the many cases in geomechanics where the rock breakage is predominantly governed by tensile (brittle) cracking. Other examples in which rock tensile fracturing plays a critical role include: (i) hydraulic fracturing, (ii) shallow tunnels where horizontal stresses are much larger than the vertical stress components, (iii) deep circular excavations where the horizontal stresses are less than one-third of the far-field vertical (gravitational) stresses - according to the Kirsch theory (Hudson and Harrison, 1997; Serati, et al., 2020), (iv) weak rock types such as lightly cemented sandstones and clay-rich rocks such as shales when lightly confined (Kaiser and Kim, 2008), and (v) in large open pit slopes and high mountain (Stead, et al., 2007). A proper estimation of rock tensile strength is therefore critical in a wide range of rock engineering applications. The coal longwall caving, in particular, requires the knowledge of rock tensile activity and its location which in turn assists in managing longwall geomechanics and associated issues in caving control.

The rock mechanics literature is almost exclusively replete with both direct and indirect testing methods to estimate rock tensile strength. Some of the widely accepted testing standards include the direct tensile test (ISRM, 1978), ring test (Serati and Williams, 2015), truncated Brazilian test (Serati, et al., 2017), Hydraulic fracturing test, block and double punch tests, pull-off test (Cacciari and Futai, 2018), the Point load strength test (Serati, et al., 2018), bullet-shaped tensile testing (Serati, et al., 2015), the flattened Brazilian disc test (Perras and Diederichs, 2014), and testing of a sphere under concentrated loads (Chau and Wei, 1999). While the direct tensile test provides the most reliable rock strength value in tension, samples often fail outside the centre at grip points in practice (i.e. premature anchorage failures). The test is also relatively more expensive compared to other alternative methods, and unwanted induced bending stresses often interfere with the test results.

The Brazilian test method was presented first in September 1943, at the 5th meeting of the Brazilian Association for Technical Rules, and has become very popular since then, mainly due to its ease of preparation and interpretation (Perras and Diederichs, 2014; Masoumi, et al., 2017). In this inexpensive test, a disc is loaded between two points across its diameter until it fails under uniformly distributed induced tensile stresses at the vicinity of the sample centre. Knowing the sample geometry (e.g. radius $r$ and thickness $t$) and the load at failure ($F$), the material tensile strength ($\sigma_t$) can be

---

¹ Lecturer, The University of Queensland. Email: m.serati@uq.edu.au Tel: +61 7 3365 3911
² Senior Lecturer, University of New South Wales. Email: H.Roshan@unsw.edu.au Tel: +61 2 9385 5535
³ Senior Lecturer, University of Southern Queensland. Email: Ali.Mirzaghborbanali@usq.edu.au Tel: +61 7 4631 2919
² PhD Student, The University of Queensland. Email: m.mahmoud@uq.net.au Tel: +61 7 3365 3742
⁵ Student, The University of Queensland. Email: t.valluru@uqconnect.edu.au
calculated using Equation 1. However, according to Griffith’s strength criterion, it should be noted that the test is only valid if the sample breaks into two equal halves due to the formation of a single crack initiated at the specimen centre, where the induced tensile stress is the maximum.

\[
\sigma_t = \frac{F}{\pi r t}
\]

(1)

To study the fracture mode of coal under induced tensile stresses, the Brazilian test was performed in this research work and coal samples of different strength values were tested using high-speed photography techniques.

**EXPERIMENTAL SETUP**

A total of 15 different coal samples were prepared and tested according to the American Society for Testing and Materials (ASTM) or International Society for Rock Mechanics (ISRM) standards (ISRM, 1978; ASTM, 2016). For the ASTM tests, samples were tested in direct contact with load frame platens under a continuously increasing compressive load until failure. To record the fracture propagation pattern, a Phantom v2012 camera was utilised. The camera is capable of recording at up to 1,000,000 frames per second (fps) at reduced resolution or 22.5 kHz at a full resolution of 1280 x 800 pixels (Phantom, 2020). Our previous investigations (Serati and Williams, 2015; Serati, et al., 2018; Xu, et al., 2018; Bahaaddini, et al., 2019) suggest that such capabilities make the camera system a very suitable gear for monitoring cracking processes in brittle solids. To identify the type of fracture pattern, each video recording was then carefully examined frame by frame using Phantom Camera Control (PCC) image processing software to identify the frame position in which the first macro crack became visible and then the last frame in which the specimen was fully disintegrated. In some cases, the edge Sobel vertical (i.e. Sobel operator), the edge Prewitt vertical or Laplacian filters were used to ease the monitoring of tensile crack propagation pattern (Phantom, 2020). For each test, the load at failure, frame number at which the first crack was identified (and its corresponding force and deformations), and the sample’s tensile strength; were recorded. All tests were conducted at 450 N/sec.

It is well understood that the presence of water in water-sensitive soft rocks can soften its textures, loosening its structure and increasing its deformability leading to an overall reduction in the material’s strength (Joseph et al., 2009; Brady and Brown, 2004). After the completion of all Brazilian tests, it was therefore decided to further measure the moisture content for each tested coal sample by calculating the ratio of the bulk mass of the tested specimen to its dry mass after being dried in an oven for at least 24 hrs. However, due to the flammability of coal, the oven temperature was kept at 60 °C. In addition, bulk, saturated, dry weight density and porosity (n) were also measured using Equations 2 and 3.

\[
n = \frac{100 V_v}{V} \%
\]

(2)

\[
V_v = \frac{M_{sat} - M_s}{\rho_w}
\]

(3)

In the above equations, \(V_v\) is the pore volume measured as a ratio of saturated mass (\(M_{sat}\)) subtracted by solid (dry) mass to the water density (\(\rho_w\)). To measure the rock porosity, the coal samples were saturated for 24 hrs by using a vacuum chamber and then dried in an oven for another 24 hrs (Paul WJ, 2002; Kacy M, 2013). Table 1 and Figure 1 summarise the obtained results, but for consistency, only the test performed by the ASTM method are presented and discussed in this report.

After careful examination of all high-speed recordings, three distinct fracture types were identified (see also Figure 1):

**Type I:** in samples exhibiting Type I fracture mode, a single straight tensile crack is initiated at the middle of the sample and propagates radially in both directions towards the outside of the specimen along the axis of diametral compressive loading. This is the accepted fracture pattern for standard Brazilian tests based on Griffith fracture criterion, where a single crack splits the sample into two halves (see Figure 1a).
Table 1: Summary of the test results

<table>
<thead>
<tr>
<th>Sample</th>
<th>Moisture content [%]</th>
<th>Dry density [kN/m$^3$]</th>
<th>Porosity [%]</th>
<th>Tensile Strength [MPa]</th>
<th>Observed Fracture Pattern (see also Fig. 1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-01</td>
<td>0.7</td>
<td>10.29</td>
<td>2.16</td>
<td>2.89</td>
<td>Multiple cracking</td>
</tr>
<tr>
<td>S-02</td>
<td>1.5</td>
<td>11.90</td>
<td>11.53</td>
<td>1.35</td>
<td>Multiple cracking</td>
</tr>
<tr>
<td>S-03</td>
<td>1.0</td>
<td>12.33</td>
<td>7.33</td>
<td>0.83</td>
<td>A single straight crack</td>
</tr>
<tr>
<td>S-04</td>
<td>3.2</td>
<td>12.08</td>
<td>12.20</td>
<td>0.47</td>
<td>Cracking through rock bridges</td>
</tr>
<tr>
<td>S-05</td>
<td>1.7</td>
<td>12.21</td>
<td>8.37</td>
<td>0.52</td>
<td>A single straight crack</td>
</tr>
<tr>
<td>S-06</td>
<td>0.9</td>
<td>11.51</td>
<td>4.82</td>
<td>0.17</td>
<td>Cracking through rock bridges</td>
</tr>
<tr>
<td>S-07</td>
<td>0.8</td>
<td>11.91</td>
<td>7.13</td>
<td>1.87</td>
<td>Multiple cracking</td>
</tr>
<tr>
<td>S-08</td>
<td>1.5</td>
<td>12.11</td>
<td>9.84</td>
<td>0.38</td>
<td>Cracking through rock bridges</td>
</tr>
<tr>
<td>S-09</td>
<td>1.8</td>
<td>11.81</td>
<td>11.45</td>
<td>0.28</td>
<td>Cracking through rock bridges</td>
</tr>
</tbody>
</table>

Type II: in this fracturing mode, the central tensile crack propagates outwards in either direction to the edges of the specimen along the axis of diametral loading. But, unlike Type 1 above, when the tensile fracture has fully propagated to the outside of the sample boundary, moderate-sized inverse shear conical plugs are formed in the vicinity of contact points. The shear fractures then propagate back towards the centre of the sample (see also Figure 1b). However, it was observed that the secondary induced shear cracks do not necessarily propagate fully through the sample diameter and either terminate inside the intact material or join the central tensile crack. From the results in Table 1 and Figure 1 combined, it can be deduced that Type II cracking mode is mostly observed in specimens with the highest tensile strength. That is, the more brittle and the stronger the coal sample in tension, the higher the possibility of observing multiple cracking in the coal Brazilian testing; hence deviating from the standard recommendations.

Type III: this fracture growth involves an interesting pattern of smaller-scale tensile and/or shear cracks happening simultaneously at local points through intact rock bridges (i.e. non-persistent pre-existing flaws) inside the Brazilian test domain. Given that rock bridges are distributed randomly in different directions with various scales, the final rupture surface was reported with different directions in each sample; instead of a single straight central crack (see also Fig. 1c, d, e, f). Figure 2 illustrates the time-lapses of a typical Type III fracture pattern in which highlighted areas represent points where rock bridge failures were first identified at each stress level.

Figure 1: Three distinct fracture formation patterns observed during Brazilian testing of coal at 3,500 fps including: (a) formation of a single straight crack at the sample centre, (b) multiple cracking, and (c, d, e, f) cracking through intact rock bridges.
RESULTS AND DISCUSSIONS

As shown in Figure 2, if the fracture mode is governed by breaking through rock bridges in tension (or shear), visible macro cracks are always formed prior to the sample reaching its peak strength in the Brazilian test. The force of failure then provides an overestimation of the sample’s tensile strength if used and plugged into Equation 1. Alternative testing techniques should then be adopted instead, or an adjustment factor is to be applied to the test results to compensate for the overestimation of material's tensile strength. It is therefore concluded that extreme precautions should be taken when the Brazilian test is utilised for testing of coal samples with pre-existing defects; otherwise, the fracturing process will be governed by the rupture of rock bridges thus making the test invalid and erroneous. The same applies when testing high strength brittle coal specimens in which multiple cracking becomes the dominate rupture mode, hence making the test results invalid. The key question raised form this investigation is that how a proper adjustment factor can be introduced for Brazilian testing of highly fractured or very brittle coal samples to make the test results reliable for design purposes. To answer this key question, further research and more tests are planned and are underway, which will be the subject for future studies.

Figure 2: High-speed time-lapses of Type III fracture pattern captured at 2,600 fps
CONCLUSIONS

Using high speed image processing techniques, different coal types with various strength properties were tested under Brazilian indirect loading conditions to study fracture propagation mode of coal under induced tensile stresses. Three distinct cracking patterns were observed, including: (i) a single central tensile crack, (ii) multiple cracking, and (iii) progressive failure of intact rock bridges that were often initiated well below the sample’s ultimate Brazilian strength. From the results, it is suggested that extreme precautions should be taken when the Brazilian test is utilised for testing a highly fractured or a brittle coal with relatively high tensile strength for which the standard recommendations are difficult to be followed entirely. That is, for coal samples in which the Brazilian fracture mode is not a valid single crack, it is seems a significant leap to utilise such results to deduce the coal tensile strength correctly.

REFERENCES


Paul, WJ, 2002. Formation Evaluation. MSc Course Notes, University of Aberdeen, UK.


A CASE OF LONGWALL COAL MINING PRODUCTIVITY & SAFETY OPTIMISATION

Sahar Ardehali¹, Naj Aziz², Habib Alehossein³, Matthew Bowerman⁴ and Matt Robbins⁵

ABSTRACT: This paper presents a study, the design and implementation of the Longwall Remote Operations centre that incorporates all aspects of automation for Longwall and coal clearance systems.

Developing the existing Longwall technology strategy and the design of coal transferring conveyors to smart systems incorporating emerging technologies such as artificial intelligence, automation and business strategy development.

The design phase of the project is structured to bring together the overall findings of the diagnose phase, business strategic direction, value case and options analysis to develop an overall project that covers people, technology, systems and process for the automated control of the Longwall. As part of this process a draft implementation plan that includes risk and change management, capability transfer and a roadmap for optimisation is included.

The site engagement team cooperates closely with the analysis team on collecting real data and analysing data in-depth by considering safety regulation and analysis of the business value case for incorporating any current and proposed automation technologies. The achieved designs introduced the opportunity to incorporate in the future of the mining industry as one of the main pillars of Australia’s economy. A key objective is to remove people, as far as practically possible, from potentially hazardous operational areas through applying remote operations and control concepts.

There is a strong value case for this change. In a Longwall operation this change reduced exposure to underground operational hazards by some 50,000 hours per year, add 3 hours per day to operations, and increase cutting rate by 150 tonnes per hour.

The strategy and design apply to limited mines and their successful outcomes provides a wide approach and an open discussion platform for the current mining industry and make us propose a new strategy for coal clearance methodology.

This paper proposes a practical strategy to revolutionise the conveyor mining industry via implementing Innovation, Technology and Digitalisation and open LEAN management techniques, automation and artificial intelligence based on the real data outcomes.

BACKGROUND

In each century miners tried to apply different kinds of optimisation techniques on the mines process and have achieved more success in the open-pit mines. Due to the complexity of an underground mine, optimisation in processes like transportation to the ground have been limited and less extensive than for open-pit mines. (Alford, C., Brazil, M. and Lee, D.H., 2007)

Mining operation research for practical projects helps to enhance the application of mining-specific and generic optimisation techniques in the mining industry. This is where LEAN management and strategy technology become highlighted in the journey of extraction and transferring resources.

The main operating processes in underground coal mining are cutting and transporting. Around the1950s modern mechanised Longwall mining became widespread. Initial practical design of automated Longwall cutting machine was in 1984. Since then automation and health-and-safety technology for Longwall operations have been developing. Longwall reliability, productivity and cost effectiveness have also been improving. (Peng, S.S., 2019)

---

¹ Miss Sahar Ardehali, Aurecon-USQ. Email: Sahar.Ardehali@aurecongroup.com Tel: +61 4 1545 1770
² Prof. Naj Aziz, Honorary Professorial Fellow, Email: naj@uow.edu.au Tel: +61 2 4221 3449
³ Dr. Habib Alehossein, USQ. Email: Habib.Alehossein@usq.edu.au Tel: +61 4 1788 6992
⁴ Mr Matthew Bowerman, Aurecon. Email: Matthew.Bowerman@aurecongroup.com Tel: +61 4 0996 3230
⁵ Mr Matt Robbins, Aurecon. Email: Matt.Robbins@aurecongroup.com Tel: +61 4 1792 5713
This paper can be a starting guideline to understand and implement the new concepts in a mining workplace and move toward a better future in Australia with proposed new technology strategic innovation. The focus of this new strategy will be on the remote operations of the Longwalls. Fast communication technology was used in stage 1 as a remote control from Remote Operational Centre (ROC). On stage 3 of this practical project fully remote control will be utilised from the site control room and solve any real-time operational problems.

ROAD MAP FOR LONGWALL OPERATIONS

Future smart mining is facing an ever-growing need for resources and this demand is exponentially increasing all around the world. In this path, education development and three key goals will be required to meet future trends:

- Innovation;
- Technology; and
- Digitalisation.

Removing people from potentially hazardous operational areas, as far as practically possible, was a key objective. Instead, remote operations and control concepts or automation from ROC was proposed for Longwall system.

This developed strategy already has been applied successfully on one of the underground hard coking coal mines and accordingly resulted in a strong value case for its remote-control change and its efficient conveying process in 2020. The mining operation has more than 600 employees, and the mine life is estimated at more than 15 years. The size of each development Pillar of this mine is 120m long and 60m wide. Each Panel has 45 pillars (each 4.5km). Coal seam thickness or cut height is 3.5m with roadway width of 5.4m. The Longwall detail is:

- Longwall shearer cuts coal using bi-directional method;
- The Longwall panel width is 300m with 1m advance per shear. Extraction height 4.2m to 4.7m. (Mining Data Solutions, 2001).

In the late 1800s and early 1900s, mines were changing from carrying the ore to the surface by different type of conveyors instead of any manual skip or self-dumping bucket. After World War 2 the coal mining industry became one of the hot industrial topics and this industry started to develop since then. In the 1970s this industry had achieved various technical developments. In the early 1980s, remote connection and automatic control of mining equipment such as shearer-loaders and frame support started to develop. (Nishimatsu, Y., 1992)

Longwall operations have made coal mining safer but still carry risks for Longwall cutter operator and other operators in the area. Another critical risk that threatens operators’ health is black lung disease. Especially in underground coal mines, dust may cause pneumoconiosis in coal workers. This disease is a chronic, irreversible lung disease. (Halldin, C.N., ..., 2015) (Barczak, T.M., 1992)

The method of Longwall mining is a highly productive and smooth underground operation. The main part of the Longwall machine is a drum shearer whose direct role is cutting the coal. It is important to keep the mine production at a desired level. The position of the drum cutter, and accordingly the roll angle, plays a very important role in cutting and remaining in the coal seam.

The main role of a Longwall operator is measuring and adjusting the degrees of the roll of the body of a Longwall shearing machine. In order to achieve remote supervision of the roll angle to control drum cutting positions via roll angle needs signal detecting and camera monitoring systems. (Ramsden Jr, J.W., American Mining Electronics Inc, 1993).

SCOPE

Controlling Longwall operation remotely from ROC resulted in the following benefits as detailed in Table 2:

- Exposure to underground operational hazards (exposure hours per week) reduced by 17%;
- Operating hours per week increased by 7%; and
• Cutting production rate (tonnes per hour) increased by 7%.

Remote Longwall operation control has required substantial advancements in automation. However, the first step which has been implemented was developing operation to semi-automated process. Future improvement in the remote supervision, including further automation incorporating best practice and lessons learnt from Phase 01, will further reduce risk and moving the operation to a completely automated process as outlined in Figure 1.

- Fully autonomous in-seam positioning and creep control through LIDAR and 3D analytical modelling
- Automatic camera control by exception – coal flow monitoring, bretby, blockages etc.
- Remote mining as a standard way of working
- 3D interactive cut planning
- Remote Operation Trials
- Camera band detection user interface
- Trial Seam recognition
- Complete auto creep management
- Improved anti collision and sensing technology
- Complete UG LIDAR trial
- Trial of acoustic monitoring for seam recognition
- BSL positioning using ultrasonic
- Automatic Creep management “Auto Steer”
- Image recognition for exception management
- Automatic seam steering
- Trial LIDAR based steering and control
- Extendable boot-end and automatic alignment
- Real-time 3D visualization
- State of the art Operations and Control Room and Sustainable Remote Operations

**Figure 1: Longwall automation roadmap includes scope of work.**

The implementation timeline for individual activities has been scheduled in Figure 2. As the project schedule confirms, the fully automated operation will be implemented in 2022. The Longwall automated design is cost effective and has minimal interruption on day-to-day operations. The project schedule milestones were set to compare work to date performance with the past.

- Remote operation trials
- Camera band detection user interface
- Trial Seam recognition
- Complete auto creep management
- Improved anti collision and sensing technology
- Complete UG LIDAR trial
- Trial of acoustic monitoring for seam recognition
- BSL positioning using ultrasonic
- Fully autonomous in-seam positioning and creep control through LIDAR and 3D analytical modelling
- Automatic camera control by exception – coal flow monitoring, bretby, blockages etc.
- Remote mining as a standard way of working
- 3D interactive cut planning

**Figure 2: Longwall automation implementation schedule**

A new role as remote operational control operator was required in the ROC. The role was filled with a suitable candidate, who passed the 2 weeks training successfully.

The scope of work is optimising the coal cutting and transport from the coalface without human presence with minimal human interaction (Figure 3) and develop the implementation plan towards remote operations. The aim is to automate the operations and control of the Longwall from the site control room.
OPERATING MODEL

There were some technical challenges in moving from in-situ control (face control) of the Longwall operations, to the site based remote control, and there will be more challenges from the level of equipment automation between now and 2022 with Longwall becoming a fully automated operation.

For assuring that all challenges have been considered, an operating model was proposed, Figure 4.

To stay on the line of efficiency, the proposed operating model monitors and focuses on how the organisation operates and delivers value. This operating module helps for better understanding and management of the operations. A Target Operating Model for remote operations control was proposed and presented in the Scope of the work.

Operating model has been proposed based on the business expectation and LEAN management requirement for the Longwall cutting process. Project leading practice, industry learnings, and operating experiences informed development of the operation plan. Accordingly work management was developed to achieve the purpose process of planning, scheduling and execution.

Operating model was planned, managed and executed on a real site with site useful data. Mine operation was managed via received updated site data. The data was analysed by designers and technology strategy consultants. Where the data revealed any process point gap, experience learned and apply the efficiency on the process where the gap initiated.
OUTCOME

The outcome of this design was a practical strategy implemented by local engineers for Phase 01. This practical strategy revolutionised the Australian conveyor mining industry via implementing Innovation, Technology and Digitalisation and open LEAN management techniques, automation and artificial intelligence based on the real data outcomes.

The Longwall operation is being optimised gradually, in 3 phases. At Phase 01 Longwall operation remote control was implemented in ROC. This process will be enhanced more in Phase 02 towards reliable automated operation.

In Phase 03 the Longwall operation will be fully automated incorporating the following enhancements:

- Fully automation in-seam positioning and creep control through 3D analytical modelling;
- Automatic Camera control for coal flow monitoring and blockage;
- 3D interactive cut planning;
- Remote control of the Longwall from site control room by a trained operator.

Phase 01 of this project implemented successfully. A key success was removing people, as far as practically possible, from potentially hazardous operational areas.

Phase 03 of the project (fully automated remote control) will be implemented in 2022.

The purpose of the three individual phases are as below, refer to Figure 5:

Phase 1: Trial incremental introduction of remote operation functions;
Phase 2: Committed, steady state remote operation and increasing automation;
Phase 3: Business as usual remote operation & automation with ongoing further optimisation. Change management will be reassessed.

Figure 5: Longwall automation 3 Phases

By terminating Phase 01 currently Longwall primary control moved from Underground face control-UG to ROC remote operations control brings down the operator's life risk tremendously.

The following Target Operating Model for Remote Operations Control design had been planned and constructed successfully.

The execution of the change plan in Phase 03 has been planned to move, the operational centre from main gate to a control room. This fully automated remote control will be implemented in 2022.

A clear and consistent approach to project management is a key success factor. A weekly project status update was arranged, and the project sponsor and any other nominated points of contact attended. This covers progress against plan, work achieved the previous week, issues encountered and actions to resolve, and work forecast for the next week.

Transitioning to remote longwall operations reduces exposure to hazards, increases operational hours and increases cutting rate and rate consistency.

The entire project is supported by an implementation plan and change management plan.
**Figure 6: Target Operating Model (TOM) for Remote Operations, aligned with proposed Operation Module**

Target Operating Model achievements are:

**Business Expectation:** (TOM: Vision and goals; Governance)
Specify the ROC operational goals as a new vision and goals to support the Business expectation. Determine governance and assurance processes, providing clear accountabilities and pathways for decision making.

**Operational Planning:** (TOM: Services; Organisation and people; Technology and facilities)
The target is planning and providing client’s expectation services with enhanced technology. In this path, employees need to receive operating culture training with co-commitments for success and deliver the services.

Determine the technology and workplace requirements for delivering the services.

**Work Management:** (TOM: Processes and execution)
Manage the operating processes to execute the services.

**Feedback (TOM: Analytics and improvement)**
Create processes to analyse and measure performance and apply a solution on the process where the gap initiated. Feedback of analysed data helps innovate on how services to be delivered.

Reviewing the Target Operating Modules will be used to review of work to date and make the process to fit-for-purpose operating model. Also, the review of Target Operating Modules helps to identify and assess the process’s risks.

**PRODUCTION AND COMMERCIAL TERMS**
At the end of the project Phase 01 in 2020, not only the Longwall Remote Operational Control from main gate design became successful, but also there were significant progress on the Production and Operation hours.

According Table 1 and Table2, the cost analysing divided into the following factors:

Direct ROC improvement only- (remote control from face to the main gate)
• Production improved by 15 tonnes per hour
• Operation hours improved 2 Hours per week with less waste hours
• Exposure hours reduction was -381 hours per week

Automation improvement-
• Production improved by 170 tonnes per hour
• Operation hours improved 5.2 Hours per week
• Exposure hours reduction was -1 hours per week

### Table 1: Production and exposure hour benefits

<table>
<thead>
<tr>
<th>Item</th>
<th>Production Rate (T/hr)</th>
<th>Operating hours (hrs/week)</th>
<th>Exposure hours reduction (hrs/week)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baseline Actual</td>
<td>2500</td>
<td>100</td>
<td>2208</td>
</tr>
<tr>
<td>Improvements attributed to ROC</td>
<td>185</td>
<td>7.2</td>
<td>-382</td>
</tr>
<tr>
<td>Direct ROC improvements</td>
<td>15</td>
<td>2.0</td>
<td>-381</td>
</tr>
<tr>
<td>Automation improvements</td>
<td>170</td>
<td>5.2</td>
<td>-1</td>
</tr>
<tr>
<td>Future baseline</td>
<td>2685</td>
<td>107.2</td>
<td>1826</td>
</tr>
<tr>
<td>ROC Improvements (%)</td>
<td>7%</td>
<td>7%</td>
<td>17.3%</td>
</tr>
</tbody>
</table>

### Table 2: % Improvement in the ROC Project

<table>
<thead>
<tr>
<th>Item</th>
<th>Improvement %</th>
<th>Improvement</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposure Hour Reduction</td>
<td>17%</td>
<td>-382</td>
<td>Hrs / week</td>
</tr>
<tr>
<td>Production Rate Improvement</td>
<td>7.4%</td>
<td>185</td>
<td>T / Hr</td>
</tr>
<tr>
<td>Operating Hour Improvement</td>
<td>7.15%</td>
<td>7.2</td>
<td>Hrs / week</td>
</tr>
<tr>
<td>Improvement due to rate</td>
<td></td>
<td>(185<em>100</em>40) = 740,200</td>
<td>T/year</td>
</tr>
<tr>
<td>Improvement due to hrs</td>
<td></td>
<td>(2500<em>7.2</em>40) = 715,000</td>
<td>T/year</td>
</tr>
<tr>
<td>Total Increased Improvement</td>
<td>15.5%</td>
<td>0</td>
<td>740,200+715,000=1,455,20</td>
</tr>
</tbody>
</table>

The actual production rate was reported 2500 tonnes Per Hour with 100 hours per week operation hours. This production rate increased by 185 tonnes per hour.

The operation hours enhanced by 7.2 hours per week. Future actual production rate will be 2685 tonnes per hour and future actual operation hours will be 107 hours per week. 77% utilisation of a year with 52 weeks will be 40 weeks.

**CONCLUSION**

This project outcomes confirm the success of the remote control of Longwall from the ROC as follows:

1. The improvements in productivity achieved through remote operation / automation;
2. The commercial realisation business value improves;
3. The system and the way that detect the coal is more improved than the overseas Longwall versions;
4. Health and Safety of the system improved compared to the old automated Longwall operation;
5. Automation development undertaken using local engineering expertise with Australian regulation and approving process, as overseas technology can’t be implemented and maintained easily in Australia, especially after COVID19 (Australian regulation is firmer than overseas). Regulations and risk appetites vary between countries; and

6. Introducing a new skill to the mine industry. Although, there is no need for Longwall cutter operator at the coal face there are opportunities to reemploy and retrain operators for control room and automation engineering maintenance.

7. Personnel costs within the budget has been managed.

This project faced some challenges as expected of a project of this kind.

The main challenges that were overcome were:

- Remote Operations Centre (ROC) operator competency level selection who already has face experience and control experience;
- Training ROC operator to follow client’s standards and make the new role compatible with the Enterprise Agreement;
- Stability of communications between surface and underground via redundant comms system;
- High quality of communications between surface and underground with low background noise via a strong filtered standard coverage;
- Placement, coverage, maintain and redundancy function of cameras;
- Resolving the experienced issues after implementing the design;
- Accurate simulation of scenarios prior to implementation; and
- Training the clients at the end of the project and handover.

At Phase 01 & 02 Longwall primary control moved from face to main gate in Intelligent control unit. This process needs to be enhanced more on Phase 03 to be fully automated and for operator to remotely control the Longwall from site control room.

Designers and strategy development teams closely work with the client and follow their standard policies while developing in different phases to ensure risks are managed appropriately with improved productivity.

This Longwall coal mining case study has been successfully commissioned. This execution can be applied to any underground Ore Longwall mining to achieve optimised cutting with high productivity and reduced risks.

REFERENCES


Mining Data Solutions, 2001, https://miningdataonline.com/property/


NUMERICAL MODELLING ON THE STABILITY OF AN UNDERGROUND RESEARCH FACILITY EXCAVATION IN CZECH REPUBLIC

Libin Gong¹, Petr Waclawik¹, Kamil Soucek¹, Martin Vavro¹, Jan Nemcik¹,², Sahendra Ram¹ and Radovan Kukutsch¹

ABSTRACT: A new Underground Research Facility (URF) is planned to be constructed in the Rožná Mine, Czech Republic. It is essential to evaluate the feasibility of the current excavation design before the start of the excavation. Numerical simulation using the software FLAC3D was adopted to investigate the stability or the strength-stress ratio of the URF. Laboratory tensile strength and uniaxial compressive strength tests on the rock specimens at various inclinations of the weak planes combined with the Geological Strength Index (GSI) system provide reliable deformation and strength input parameters for the explicit continuum model. The well-known modified Hoek-Brown constitutive model was employed for the numerical evaluation of the URF stability. In situ three-dimensional stress tensors were measured from the field and applied to the model boundaries. Displacement/convergence and distribution patterns of the stress state and strength-stress ratio were analysed. It was found that the current URF layout is on the safe side and the total displacement is minor and acceptable. Locations with potentially low strength-stress ratios were mainly around the roadway-chamber intersections. The current study can provide a valuable reference to similar numerical works since such large-scale numerical simulations are rare, due to the complexity and the time-consuming nature of the study.

INTRODUCTION

Deep Geological Repositories (DGRs) have been constructed all over the world for storing toxic or radioactive waste, which has been widely accepted to be the safest way of long-term isolation and containment of such waste materials (Feiveson et al., 2011). A large number of Underground Research Facilities (URFs) have also been built in many countries to investigate the geotechnical and environmental issues of such DGRs and to study their isolation ability and performance (Apted, 2019; Zita Bukovská et al., 2019; Delay et al., 2014; Laverov et al., 2016; NEA-OECD, 2013; Ota et al., 2007; Wang et al., 2018). Recently, the second URF in the Czech Republic is planned to be excavated in the Rožná Mine area (hereinafter referred to as Bukov II), after the successful construction of the Bukov URF between 2013 and 2016 (Zita Bukovská et al., 2019; Kamil Souček et al., 2017).

The URF Bukov II is designed to be located at 550 m underground in the metamorphic sequences of rocks. Five groups of laboratory chambers and two ventilation roadways are to be excavated along a main roadway maintained from the original uranium mining. Before the construction commencement, it is essential to evaluate the feasibility and stability of the designed roadway and chamber layouts. The complex 3-Dimensional (3D) geometry configuration of the caverns, the excavation sequence, in situ stress distributions and the existence of foliation in the metamorphic rock masses are all important parameters dominating the URF stability. Due to complexity of design, stability analysis or the estimation of the stress-stress ratio for the cavern groups has to be conducted through the numerical simulation method.

The Itasca program Fast Lagrangian Analysis of Continua in 3 Dimensions (FLAC3D) has been widely adopted to numerically model and analyse the stability of engineering projects ranging from underground excavations to rock slopes on the ground surface (Napa-García et al., 2019; Renani and Martin, 2020). FLAC3D is therefore employed in this paper to analyse the feasibility and stability of the designed URF Bukov II.

¹ Dr./Researcher, Institute of Geonics, the Czech Academy of Sciences.
² Corresponding author, Email: libin.gong@ugn.cas.cz
2 Dr./Honorary Senior Fellow, University of Wollongong. Email: jnemcik@uow.edu.au Tel: +61 2 4221 4492
GEOLOGICAL BACKGROUND

Lithology

The region of interest is formed by a highly metamorphosed volcano-sedimentary rock sequence at the north-eastern edge of the Strážek Moldanubicum of the Bohemian Massif (Zita Bukovská et al., 2019; Ptáček et al., 2013; “Rožná Uranium Deposit: Model of Late Variscan and Post Variscan Mineralizations (in Czech),” 2005; Kamil Souček et al., 2017; Vavro et al., 2015). The dominant rock types are paragneisses, migmatites and amphibolites with minor intercalation of calc-silicate rocks, marbles, granulites, granites/pegmatites, and peridotites. Both basic lithologies i.e. metapelites (paragneisses) and metabasites (amphibolites) are affected by different degrees of migmatization and are usually connected by gradual mutual transitions. The subjected area of Bukov II URF is formed mainly (in about 90% of the area) by: (1) medium- to coarse-grained migmatites, and (2) fine- to medium-grained, slightly- to medium-migmatized biotite- to biotite-amphibole paragneisses with transition to biotite amphibolites (see Figure 1). These two lithology types are very close to each other in terms of geomechanical properties. For simplicity, we consider the rock masses in this region as quasi-homogeneous and to consist of one general rock type with similar mechanical properties.

Geological structures

As shown in Figure 1 the whole rock sequence in the area of interest is trending relatively monotonously in the directions NW-SE to NNW-SSE. The dominant system of metamorphic fabric is represented by metamorphic foliations, in general gently to moderately dipping towards SW.

The first-order tectonic structure (zone R-1) occurs in the immediate vicinity of the survey area. This fault has a direction of N-S to NNW-SSE and a general dip of 45-55° to W, representing a cataclastic to mylonitized zone with a thickness of about 5-15 m and a strike length of up to 15 km (“Rožná Uranium Deposit: Model of Late Variscan and Post Variscan Mineralizations (in Czech),” 2005). In terms of mineralization, zone R-1 is one of the two main ore-bearing structures of the Rožná uranium deposit. The second-order tectonic structures (e.g. zone R-17) are spatially and probably also genetically associated with zone R-1.

The number and orientation of the regional joint systems are estimated by analogy with structural measurements performed at the GS12 geotechnical station (Bukovská et al., 2020). The station is located on a traffic roadway situated on the north-western edge of the Bukov II excavation area (see Figure 1).

Four main joint systems were captured by structural-geological mapping performed in a section about 10 meters long in the GS12 area. The dominant joint system is spatially identical to metamorphic foliation, with a general dip direction to the SW and a dip of 50° - 70°. The second system is represented by cracks with a medium inclination (approximately 30° ~ 50°) to the NW ~ NNW. The other two systems are formed by steep to vertical (70° ~ 90°) cracks with inclinations to the NW or sometimes the E ~ ENE (Figure 2). The existence of all fracture systems described above was confirmed well by optical (OPTV) and acoustic (HiRAT) television recordings from a sub-horizontal (BGS12-H) borehole and a vertically-down (BGS12-VD) borehole at the GS12 geotechnical station (Figure 3). In addition, a system of steep to vertical structures, especially secondarily filled cracks and small fissures, was also identified in the BGS12-H borehole (Figure 3).

Evaluation of geomechanical properties of rock masses

The input mechanical properties of the rock mass for the numerical model were determined based on available geological survey and laboratory testing data (Z Bukovská et al., 2020; K. Souček et al., 2018). Laboratory testing was conducted in three different directions to their metamorphic foliation planes to capture the property anisotropy. Testing results are summarised in Table 1.

Based on the testing results, the Hoek-Brown failure envelope was determined for these samples using the RocData toolkit (Rocksience Inc., 2017). The Hoek-Brown parameters for three different loading directions were then averaged to obtain the general failure envelopes for the quasi-homogeneous lithological unit. An example is illustrated in Figure 4. The Hoek-Brown constant m for intact rock was selected following the recommended tables in RocData. Other important parameters were estimated by the following equations:
\[ E_b = E_i \left[ 0.02 + \frac{1}{1 + e^\left(\frac{60 - \text{GSI}}{11}\right)} \right] \] (1)

\[ m_b = m_i e^{\frac{GSI - 100}{28}} \] (2)

\[ s = e^{\frac{GSI - 100}{9}} \] (3)

\[ a = \frac{1}{2} + \frac{1}{6} \left( e^{\frac{GSI - 100}{15}} - e^{-\frac{28}{3}} \right) \] (4)

\[ \sigma^t = -\frac{s \sigma_{ci}}{m_b} \] (5)

where: \( E_i \) and \( E_b \) are deformation moduli for intact rock and rock mass, respectively; \( m_i \) and \( m_b \) are Hoek-Brown constants for intact rock and rock mass, respectively; \( s \) and \( a \) are Hoek-Brown parameters; \( \sigma^t \) is the rock mass tensile strength; \( \sigma_{ci} \) is the Uniaxial Compressive Strength (UCS) of Intact Rock.

Figure 1: Geological condition in the area of interest (Patocka and Jaros, 2020; Ptáček et al., 2013)

To extrapolate the averaged Hoek-Brown parameters from intact rock samples to field-scale values, the Geological Strength Index (GSI) of the rock mass was employed (Hoek et al., n.d.). Structural characteristics including persistence, roughness, undulation, and opening and filling of rock mass discontinuities were previously documented on the uncovered opening walls within the Bukov URF.
The GSI was then calculated using two different methods:

\[
GSI = 1.5 \int \text{Cond}_{89} + \frac{RQD}{2} \tag{6}
\]

or

\[
GSI = \frac{52J_r}{J_a+J_r} + \frac{RQD}{2} \tag{7}
\]

where, \( RQD \) is the Rock Quality Designation index value, \( J\text{Cond}_{89} \) is the Joint Condition rating according to (Bieniawski, 1989), and the joint roughness number \( J_r \) and joint alteration number \( J_a \) are parameters in the \( Q \)-system (Barton et al., 1974).

Figure 2: Contour diagram of joint poles on the GS12 profile

Figure 3: Stereograms of interpreted cracks (presented by projection of gradient lines) in boreholes BGS12-VD (left) and BGS12-H (right). The colour differentiation reflects the openness or closure of cracks, filling existence; orientation of structures is in the format of dip direction (zero value is displayed in the centre of the network, and 90˚ lies on the circumference of the network circle).

The estimated \( RQD \) value for the rock mass of the whole Rožná uranium mine area is mostly between 60 - 65%, peaking up to 70 to 80% (Z Bukovská et al., 2020; Zita Bukovská et al., 2019; K. Souček et al., 2018; Kamil Souček et al., 2017; Vavro et al., 2015). However, Patocka and Jaros (2020) report an average \( RQD \) value of up to about 86% in the area of interest. We have chosen the \( RQD \) value of 60% for the calculation of GSI out of concern for safety. The values of other parameters (\( J\text{Cond}_{89}, J_r \) and \( J_a \)) were determined from a distribution analysis of the discontinuity factors (e.g., persistence, roughness, etc.) expressed by a weighted average. Table 2 shows the determined geological input values for the calculation of GSI under the most probable occurrences of \( RQD \) values at Bukov URF. It is clear that at the value of \( RQD = 60\% \), GSI varies in the range of 63-67. Hence, a value of 65 was chosen for estimating the filed-scale geomechanical properties of the above-mentioned intact rock samples. The
obtained Hoek-Brown parameters for the rock mass were further averaged to describe the quasi-homogeneous rock mass for the numerical modelling. In this way, both the anisotropic rock fabric (foliation) and the geological structure are taken into account. The failure envelope representing the macro-scale rock mass properties corresponding to the sample V22 (see Table 1) was also shown in Figure 4.

### Table 1: Average values of physical-mechanical properties of selected intact rock samples

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>sample T3</th>
<th>sample V22-R3</th>
<th>sample V22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lithology</td>
<td>- Biotite-amphibole gneiss</td>
<td>Biotite migmatite</td>
<td>Sillimanite-biotite migmatite</td>
<td></td>
</tr>
<tr>
<td>Bulk density, $\rho$</td>
<td>kg/m$^3$</td>
<td>2836</td>
<td>2742</td>
<td>2635</td>
</tr>
<tr>
<td>UCS, $\sigma_{ii}$ - direction K</td>
<td>MPa</td>
<td>165</td>
<td>165</td>
<td>160</td>
</tr>
<tr>
<td>UCS, $\sigma_{ii}$ - direction P</td>
<td>MPa</td>
<td>195</td>
<td>126</td>
<td>111</td>
</tr>
<tr>
<td>UCS, $\sigma_{ii}$ - direction S</td>
<td>MPa</td>
<td>147</td>
<td>74</td>
<td>82</td>
</tr>
<tr>
<td>Young’s modulus, $E$ (K)</td>
<td>GPa</td>
<td>45</td>
<td>41</td>
<td>41</td>
</tr>
<tr>
<td>Young’s modulus, $E$ (P)</td>
<td>GPa</td>
<td>56</td>
<td>54</td>
<td>44</td>
</tr>
<tr>
<td>Young’s modulus, $E$ (S)</td>
<td>GPa</td>
<td>49</td>
<td>29</td>
<td>32</td>
</tr>
<tr>
<td>Poisson ratio, $\nu$ (K)</td>
<td>-</td>
<td>0.15</td>
<td>0.17</td>
<td>0.19</td>
</tr>
<tr>
<td>Poisson ratio, $\nu$ (P)</td>
<td>-</td>
<td>0.17</td>
<td>0.16</td>
<td>0.17</td>
</tr>
<tr>
<td>Poisson ratio, $\nu$ (S)</td>
<td>-</td>
<td>0.14</td>
<td>0.12</td>
<td>0.10</td>
</tr>
<tr>
<td>Tensile strength, $\sigma_{ii}$ (K)</td>
<td>MPa</td>
<td>12.0</td>
<td>7.8</td>
<td>6.2</td>
</tr>
<tr>
<td>Tensile strength, $\sigma_{ii}$ (P)</td>
<td>MPa</td>
<td>12.8</td>
<td>12.7</td>
<td>11.1</td>
</tr>
<tr>
<td>Tensile strength, $\sigma_{ii}$ (S)</td>
<td>MPa</td>
<td>10.9</td>
<td>10.4</td>
<td>8.1</td>
</tr>
</tbody>
</table>

Note: UCS - uniaxial compressive strength; K - direction of loading perpendicular to foliation plane; P - direction of loading parallel to foliation plane; S - direction of loading diagonal (approximately 45°) to metamorphic foliation plane.

### Table 2: Estimated quality parameters of Bukov URF rock mass for the calculation of GSI

#### BZ-XIIJ access gallery (mapped approx. 220 m, evaluated approx. 650 discontinuities)

<table>
<thead>
<tr>
<th>$J_{\text{Cond}}$</th>
<th>ROD (%)</th>
<th>GSI (from Eq. 2.6)</th>
<th>GSI (from Eq. 2.7)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24.7</td>
<td>60</td>
<td>67.1</td>
<td>64.2</td>
</tr>
<tr>
<td>2.30</td>
<td>70</td>
<td>72.1</td>
<td>69.2</td>
</tr>
<tr>
<td>1.21</td>
<td>80</td>
<td>77.1</td>
<td>74.2</td>
</tr>
</tbody>
</table>

#### Main laboratory tunnel BZ-XII (mapped approx. 90 m, evaluated approx. 273 discontinuities)

<table>
<thead>
<tr>
<th>$J_{\text{Cond}}$</th>
<th>ROD (%)</th>
<th>GSI (from Eq. 2.6)</th>
<th>GSI (from Eq. 2.7)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24.5</td>
<td>60</td>
<td>66.8</td>
<td>62.7</td>
</tr>
<tr>
<td>2.13</td>
<td>70</td>
<td>71.8</td>
<td>67.7</td>
</tr>
<tr>
<td>1.26</td>
<td>80</td>
<td>76.8</td>
<td>72.7</td>
</tr>
</tbody>
</table>

### Stress state of rock mass

One of the typical features of the rock mass in the Bukov region is relatively significant anisotropy, verified both physically (e.g. ultrasonic wave velocity, thermal conductivity and specific heat capacity) and mechanically (e.g. splitting tensile strength, uniaxial compressive strength) (Z Bukovská et al., 2020; Zita Bukovská et al., 2019; K. Souček et al., 2018; Vavro et al., 2015). The high degree of textural anisotropy of rocks, as well as the relatively significant disturbance of the rock mass by ductile and brittle tectonics, result in the in situ stress field of the rock mass in this region as highly anisotropic.

The in situ stress state within the rock mass was determined based on the knowledge obtained in the Rožná mine area, e.g. from the URF Bukov I project (K. Souček et al., 2018) or the Deep Horizons project (Z Bukovská et al., 2020). The following in situ stress measurement methods was adopted to interpret the stress state of the rock mass:

- Hydro-fracturing,
- CCBO and CCBM method,
- Mathematical back-calculation based on the roadway convergence measurement.

From the measured results it can be stated that the rock mass is relatively complicated in terms of the orientation and magnitude of the stress field. The local variability of the interpreted results is manifested mainly in the orientation. The global evaluation of the whole group of current and
previously performed measurements shows that the orientation of the main component of horizontal stress ($S_H$) is in the direction of NW-SE to N-S, but the directions NNE - SSW are not exceptional either. In the current work, the maximum horizontal stress orientation was simplified as N – S. As for the magnitudes of horizontal stress components, the value of maximum horizontal stress reaches 1~3 times the assumed vertical stress for a given depth and the horizontal stress ratio ($\sigma_H/\sigma_V$) generally reached a ratio of 1.5~2 : 1 (Z Bukovská et al., 2020).

<table>
<thead>
<tr>
<th>Hoek Brown Classification</th>
<th>Input parameters for:</th>
<th>Intact Rock</th>
<th>Rock Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact UCS</td>
<td>137.187 MPa</td>
<td>137.19 MPa</td>
<td></td>
</tr>
<tr>
<td>GSI</td>
<td>100</td>
<td>65</td>
<td></td>
</tr>
<tr>
<td>$m_h$</td>
<td>15.082</td>
<td>15.08</td>
<td></td>
</tr>
<tr>
<td>Disturbance factor</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Intact modulus</td>
<td>40000 MPa</td>
<td>40000 MPa</td>
<td></td>
</tr>
</tbody>
</table>

**Hoek Brown Criterion**

$\sigma_H = 15.082$  
$\sigma_V = 1$  
$\sigma = 0.5$  
$\sigma = 0.502$

**Failure Envelope Range**

- Application: Custom
- $\sigma_{max} = 30$ MPa

**Mohr Coulomb Fit**

- Cohesion: 24.657 MPa
- Friction angle: 48.094 deg

<table>
<thead>
<tr>
<th>Rock Mass Parameters</th>
<th>Tensile strength</th>
<th>Uniaxial compressive strength</th>
<th>Global strength</th>
<th>Modulus of deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>9.096 MPa</td>
<td>137.187 MPa</td>
<td>130.151 MPa</td>
<td>39773.157 MPa</td>
</tr>
<tr>
<td></td>
<td>0.65 MPa</td>
<td>19.477 MPa</td>
<td>40.301 MPa</td>
<td>25268.776 MPa</td>
</tr>
</tbody>
</table>

**Figure 4: Example for generating the average Hoek-Brown failure envelopes based on three testing results with different loading directions (sample V22).**

In accordance with the measured in situ stress values, a horizontal stress was applied to the vertical boundaries of the numerical model in the second Hoek-Brown model with the measured horizontal stress. As part of the simplification of the conditions, the maximum horizontal stress ($\sigma_H$) was applied in the direction of the models Y axis, i.e. in the N-S direction. Other stress components follow the approximate ratio $\sigma_V : \sigma_H : \sigma_V = 1 : 2 : 1 = 14.55 : 28 : 15$ (MPa), with $\sigma_V$ being located in the model Z-axis, $\sigma_H$ in the Y-axis and $\sigma_V$ in the X-axis.

**NUMERICAL SIMULATION**

**Model Generation and Boundary Conditions**

The commercial program Fast Lagrangian Analysis of Continua in 3 Dimensions (FLAC3D) (Itasca Consulting Group 2012) was adopted to analyse the stability of the planned underground research facility.

The model dimension was determined carefully to eliminate the boundary effect. The top and bottom boundary of the model were set as ten excavation height away from the excavation periphery (Itasca Consulting Group 2012), i.e. the model has a height of 84 m. To select the appropriate lateral
boundary dimension, a series of parallel models (Figure 5) with different values of width/length were generated. All the designed roadways and laboratory chambers were excavated and run to equilibrium. Figure 6 shows the change of maximum displacement magnitude of the models with varying lateral boundary dimensions. It is clear that the lateral boundary effect on the model displacement is minor. The variation of the maximum displacement magnitude is within 1 mm when the lateral distance between the boundaries and the excavation region increased from 50 m to 300 m. Considering the calculation efficiency, $L = 50$ m was selected and a corresponding dimension of $224 \times 336 \times 84$ m was finally determined for the 3D model. A total of 583,233 zones are contained in the model, with the zone volume ranging from $8.5 \text{ cm}^3$ to $130 \text{ m}^3$.

Figure 5: Plane view of the model geometry with different lateral boundary dimensions
The model is fixed on the bottom boundary while free on the top and side boundaries. The vertical stress $\sigma_v$ is applied on the top surface, where

$$\sigma_v = -\rho g H$$  \(8\)

where $\rho = 2700$ kg/m$^3$ is the density of rock, $g$ is gravity, and $H$ is depth of the excavation given as 550 m. Two different scenarios in terms of horizontal stresses were studied. In the first case, the coefficient of horizontal stresses is only determined by the Poisson ratio ($\nu$) of rock, and hence

$$\sigma_H = \sigma_h = \frac{\sigma_v \nu}{1-\nu}$$  \(9\)

where $\sigma_H$ and $\sigma_h$ are the maximum and minimum horizontal stress, respectively, and $\nu$ is given as 0.2. In the second case, the horizontal stresses obtained from in-situ measurement were applied. The maximum horizontal stress $\sigma_H = -28$ MPa pointing north which coincides with the positive y-direction of the current model. The minimum horizontal stress $\sigma_h = -15$ MPa is applied in the x-direction. Figure 7 demonstrates the boundary condition and the roadway/chamber configuration of the model.

**Figure 6:** Lateral boundary effect on the maximum displacement magnitude after a whole-stage excavation

**Figure 7:** Model boundary conditions with different horizontal stresses and excavation sequences of the designed underground research facility roadways and chambers

**Modelling Procedure**

The modelled roadways and chambers were excavated step by step. As shown in Figure 7(right), the excavation includes ten sequences. Firstly, the main roadways remained from previous mining works were excavated. The roadway branch in the furthest southern part (RoadwayL8) was then excavated, followed by the next branch the northern vicinity (RoadwayL7), and a ventilation roadway (Ventilation7-8) connecting these two branches was developed afterwards. Next, the excavation of the third and fourth branches (RoadwayL6 and RoadwayL5, respectively) were conducted, following the
second ventilation roadway (Ventilation5-6) later on. After that, the last group of roadways and chambers (RoadwayL4) were excavated sequentially, as depicted in Figure 7(right). The model was run to equilibrium after each sequence. Both velocity and displacement of the whole model were reset to zero after the main roadway excavation, so that the net increments resulted from the new excavations can be directly observed.

Constitutive Models and Mechanical Properties

The well-known Modified Hoek-Brown (MH-B) model was employed to control the rock mass behaviour in this project. Compared with the original Hoek-Brown model, the modified version includes a tensile yield criterion and also allows the user to specify a dilation angle, which performs better at low confinement or under tensile-stress conditions. Model properties for the MH-B model are summarised in Table 3.

Table 3: Rock mechanical properties for the MH-B model

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
<td>65</td>
</tr>
<tr>
<td>Deformation Modulus, $E_b$ (GPa)</td>
<td>28</td>
</tr>
<tr>
<td>Poisson’s Ratio, $\nu$</td>
<td>0.2</td>
</tr>
<tr>
<td>Elastic Bulk Modulus, $K$ (GPa)</td>
<td>15.56</td>
</tr>
<tr>
<td>Elastic Shear Modulus, $G$ (GPa)</td>
<td>11.67</td>
</tr>
<tr>
<td>Density, $\rho$ (kg/m$^3$)</td>
<td>2700</td>
</tr>
<tr>
<td>$\sigma_{ci}$ (MPa)</td>
<td>147.86</td>
</tr>
<tr>
<td>$\sigma'$ (MPa)</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Modelling Results and discussion

The stability of the URF Bukov II was analysed in terms of displacement/roadway convergence, stress distribution and strength-stress ratio. Eight cross sections were applied to generate 2D contour plots of these parameters (Figure 8), including three vertical cutting planes nearly perpendicular to the X-axis, four more vertical planes roughly perpendicular to the Y-axis, as well as a horizontal plane 2.5 m above the floor level of the excavations. In addition, a series of monitoring points with an interval of approximately 1 m were set up along each roadway axis to analyse the displacement convergence.

![Figure 8: Plane-view demonstration of the locations of the cross sections](image-url)
Displacement and convergence

Figure 9 shows the changing trend of the zone-based maximum displacement during the step-by-step excavation. The maximum displacement magnitude increased almost linearly from 3.7 mm to 5.1 mm during excavation.

Figure 9: The change of maximum displacement magnitude in the model during incremental excavation stages

Figure 10 demonstrates the vertical and horizontal convergence of each roadway respectively at different mining stages. Roadways perpendicular to the highest in situ stress component (i.e. Roadway L8, L7, L6, L5, L4) generally have a higher horizontal convergence (4.5 ~ 6.3 mm) than vertical convergence. The values of vertical convergence in these roadways are in the range of about 2.5 ~ 5 mm, peaking at 6 mm in L6 at the positions intersecting with the symmetrically-designed laboratory chambers (see Figures 7-8). In roadways where the chambers have an alternating (asymmetrical) arrangement, the vertical convergences peak at about 5 mm. It seems that alternating the layout of the laboratory chambers can more evenly distribute the load in the roadway roof/floor and lower the convergence values. This could have a positive effect on excavation stability, especially in locally weakened rock mass conditions.

The influence of the maximum horizontal stress on the convergence development is evident from the ventilation channels which are roughly parallel with the Y-axis. Horizontal convergences in these channels are only in the magnitude of approx. 0 ~ 0.5 mm compared to approx. 4.5 ~ 6.3 mm in roadways L8, L7, L6, L5, and L4. In contrast, the values of vertical convergences in the ventilation channels are at a comparable level (2.0 ~ 3.3 mm) with the roadways L4 ~ L8 (2.7 ~ 5 mm), and slightly lower due to a smaller cross section.

The calculated values of convergence from the 3D numerical modelling are in the same order of magnitude compared with the real measured values at the previously excavated URF Bukov I. For example, at the access roadway BZ-XIIJ with a profile of about 9.2 m2, steady horizontal convergence reached approx. 2 mm. On the larger profile of the excavated mining work BZ1-XII (approx. 14.5 m2), horizontal convergences were measured at a level of approximately 4 to 6 mm, reaching peak values of approximately 9 to 12 mm affected by the presence of a local tectonic fault. Vertical convergences showed values of about 2 ~ 4 mm (Souček et al. 2018).

Stress distribution

As mentioned before, suitable cross sections were used to analysis the stress distribution inside the 3D model. Figure 11 show the case of the contour distribution on the horizontal cutting plane at the final stage of the excavation in terms of horizontal stress in the y-direction and vertical stress, respectively. The maximum stress concentration ratio and corresponding localization on each cutting planes are summarised in Table 4. Note that the maximum stress concentration ratio represents the ratio of the existed maximum stress in the cutting plane to the value of the original (in situ) far-field stress. The ratio of the maximum stress concentration in the evaluated sections mostly ranges from 1.09 to 2.0, peaking at 2.5 in the area of roadway-chamber intersections, and 2.3 in the area of the
ventilation channel ribs. Stress concentrations were again largely identified in the ribs, roof and floor of the mine workings.

Figure 10: Vertical and horizontal convergence of the roadways
Figure 10 (continue): Vertical and horizontal convergence of the roadways
Table 4: Values of the ratio of the maximum stress concentration in individual sections

<table>
<thead>
<tr>
<th>Cutting planes</th>
<th>Maximum ratio of stress concentration and its localization</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ZZ stress</td>
<td>YY stress</td>
</tr>
<tr>
<td></td>
<td>Maximum stress concentration ratio</td>
<td>Corresponding location</td>
</tr>
<tr>
<td>1-1'</td>
<td>1.53</td>
<td>The ribs of the mining work</td>
</tr>
<tr>
<td>2-2'</td>
<td>1.7</td>
<td>The ribs of the mining work</td>
</tr>
<tr>
<td>3-3'</td>
<td>1.69</td>
<td>The ribs of the mining work</td>
</tr>
<tr>
<td>A-A'</td>
<td>2.3</td>
<td>The ribs of the mining work</td>
</tr>
<tr>
<td>B-B'</td>
<td>1.99</td>
<td>The ribs of the mining work</td>
</tr>
<tr>
<td>C-C'</td>
<td>1.89</td>
<td>The ribs of the mining work</td>
</tr>
<tr>
<td>D-D'</td>
<td>1.96</td>
<td>The ribs of the mining work</td>
</tr>
<tr>
<td>E-E'</td>
<td>2.15</td>
<td>The ribs of the mining work</td>
</tr>
<tr>
<td>Horizontal plane</td>
<td>2.5</td>
<td>Roadway-chamber intersections</td>
</tr>
</tbody>
</table>

Figure 11: Contour of YY-stress and ZZ-stress on the horizontal cutting plane at the final stage of the URF excavation

Figures 12-17 shows the distribution of the horizontal (YY) and vertical (ZZ) components of the stress field in close vicinity to the mining works. It can be seen that the vertical stress relief in the roof/floor area is obvious, in some cases even with the occurrence of tensile stress. The concentration of the horizontal stress component in the YY direction is also evident, which could lead to structural overburden in the case of a sub-horizontal or slightly inclined fracture system especially with biotite-weakened metamorphic foliation. Such overburden should be enhanced by using appropriate support systems. Details from the horizontal section (see Figure 17) show that the centre of the rock pillars between the laboratory chambers will be in a slightly concentrated state with respect to vertical stress (concentration ratio = 1.3). Towards their edge, the stress concentration ratio increases up to a value of about 2. If an in situ stress level is required in the rock pillars for some field experiments in the future, the pillar width should increase to an appropriate value and be verified using numerical simulation.

Strength-stress ratio

The maximum stress concentration ratios do not exactly represent a danger for rock failure in the vicinity of the designed mine workings. The strength-stress ratio (SR) was also analysed in the following discussion. The value of SR indicates the actual rock failure potential by comparing the stress state to the specified rock failure criterion. An SR lower than 1 indicates that the rock mass is at failure state and loses its stability.

Figure 18 shows the evolution of the minimum SR in the model during excavation. The minimum SR decreased gradually from 3.33 in the first excavation stage and remains almost stable at 2.1 after the 6th stage.
Figure 12: ZZ- and YY-Stress distribution: detail of plane 1-1' - roadway L8 (see Figure 8)

Figure 13: ZZ- and YY-stress distribution: detail of plane CC' - intersection area of opposite ZK6b and ZK6e with L6 (see Figure 8)

Figure 14: Stress distribution in directions ZZ and YY: detail of cross section perpendicular to plane CC' - intersection area of opposite arranged ZK6b and ZK6e with L6 (see Figure 8)
Figure 15: Stress distribution in directions ZZ and YY: detail of plane $BB'$ - intersection area of alternately arranged ZK7b, ZK7d and ZK7e with L7 (see Figure 8)

Figure 16: Stress distribution in directions ZZ and YY: detail of cross section perpendicular to plane $BB'$ - Chamber ZK7b (see Figure 8)

Figure 17: Stress distribution in ZZ and YY directions: detail of horizontal cross section in area L5, Zk5e and ventilation channel L5-L6 (see Figure 8)
Figure 18: The change of minimum strength-stress ratio in the model during incremental excavation stages

The distribution of SR in the whole model following the final excavation stage was statically analysed and plotted as per the Histogram shown in Figure 19. The minimum zone-based average values of the SR are all higher than 1.5, and more than 97% of the zones possess a ratio higher than 3.16.

Figure 19: Histogram of the zone-based strength-stress ratios in the whole model

Figure 20 shows the case of the contour distribution on the 3-3’ cutting plane at the final stage of the excavation in terms of SR. The changing ranges of SR, the minimum value, and the location corresponding to the minimum SR on each cutting planes are summarised in Table 5. Several typical closer views of the SR contour plots can be seen in Figures 21-24. The minimum values of SR were mainly located in the roadway roof and floor (Figure 21), the roadway-chamber intersection (Figure 22(left) and 23(left)) and chamber ribs (Figure 24). In the horizontal section, the values of the minimum SR are located in the corner area of the mine working intersections (see Figure 24).

Figure 20: Contour of strength-stress ratio on 3-3’ cutting plane at the final excavation stage
Table 5: strength-stress ratio characteristics on each cross section

<table>
<thead>
<tr>
<th>Cutting planes</th>
<th>Range of SR and its minimum value, including its location</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range of SR</td>
</tr>
<tr>
<td>1-1’</td>
<td>1.52 – 9.98</td>
</tr>
<tr>
<td>2-2’</td>
<td>1.44 – 9.68</td>
</tr>
<tr>
<td>3-3’</td>
<td>1.40 – 9.40</td>
</tr>
<tr>
<td>A-A’</td>
<td>1.60 – 9.28</td>
</tr>
<tr>
<td>B-B’</td>
<td>1.41 – 7.72</td>
</tr>
<tr>
<td>C-C’</td>
<td>1.42 – 7.35</td>
</tr>
<tr>
<td>D-D’</td>
<td>1.72 – 9.99</td>
</tr>
<tr>
<td>E-E’</td>
<td>1.58 – 10.0</td>
</tr>
<tr>
<td>Horizontal plane</td>
<td>1.58 – 10.0</td>
</tr>
</tbody>
</table>

Figure 21: SR distribution on plane 1-1’ - Roadway L8 (see Figure 8)

Figure 22: SR distribution of the intersection area of opposite arranged ZK6b and ZK6e with L6. Left: cross section on plane CC’; right: cross section perpendicular to plane CC’ (see Figure 8).
SUMMARY AND CONCLUSIONS

The feasibility and stability of the planned URF Bukov II excavation in the Rožná mine area was investigated by large-scale 3D numerical modelling using the explicit Lagrangian finite-volume program FLAC3D. The modified Hoek-Brown constitutive model was adopted to control the rock mass behaviour. Input geotechnical properties of the rock mass were estimated by combining field geological mapping, laboratory experiments and theoretical calculations, considering the relatively poorer geological conditions. Parameters including displacement/convergence, stress distribution and strength-stress ratio were used for the stability analysis. Based on the modelling results, we can conclude that the designed mining sequence is feasible and the mine workings are stable exhibiting the usual geological and structural conditions. No significant excavation-induced rock mass disturbance is expected. However, it is necessary to pay attention to the intersection areas of roadways and chambers in the roof, shoulder and corner where lower strength-stress ratio values were found in the range of about 1.3 - 1.5. In poor geological conditions in such areas, it may be necessary to use bolt reinforcement. Future detailed geotechnical monitoring in the field is essential to refine the numerical modelling, and contribute to the design and construction of the next-stage national nuclear waste repository.
ACKNOWLEDGEMENT

The authors thank the Radioactive Waste Repository Authority of the Czech Republic for permission to publish this work. This article was supported by a project for the long-term conceptual development of research organizations (RVO: 68145535).

REFERENCES


APPLICATION OF MONTE CARLO SIMULATION TO QUANTIFY UNCERTAINTIES OF FIRST WEIGHTING INTERVAL ESTIMATION

Sadjad Mohammadi¹, Mohammad Ataei², Ali Mirzaghorbanali³ and Naj Aziz⁴

ABSTRACT: This paper aims to assess and predict first weighting interval in block 3 of Parvadeh IV, Tabas, Iran by using empirical and analytical models incorporating Monte Carlo Simulation. For this purpose, a database was established in a probabilistic manner to use with the Rock Quality Index (RQI) and the Central Institute of Mining and Fuel Research Index (CMRI) and analytical model. Due to similarities of the geo-mining environment with other mines in the Parvadeh district, data was collected from other mines and combined with those of block 3. Results of Monte Carlo Simulations showed that the first weighting interval varies from 15 to 20 m with a 50% probability. In addition, findings indicated that the probability of the first weighting interval being between 16 and 22 m is 90%.

INTRODUCTION

Based on strata mechanics theory, the first weighting event in longwall mining involves deflection, separation, fracturing and collapse of rock beams. During this process maximum induced stresses will be applied to the face and support system. Accordingly, prediction of the first weighting interval is imperative in assessment of the maximum induced stress during extraction, and subsequently, progressive caving of the immediate roof strata (Mohammadi, et al., 2019a). In the literature, there are several empirical, analytical and numerical models to predict the first weighting interval, however, the main issue in the evaluation of caving behaviour and prediction of caving spans, particularly in the basic design phase is the lack of enough and accurate data. In addition, there are many uncertainties in relevant parameters values. Therefore, planning and designing need to be carried out by taking a possible range for the parameters. Probabilistic simulations can be used to overcome this challenge, which is the subject of this paper.

The selected case study is block 3 of Parvadeh IV in Tabas, Iran, which is a new longwall project to extract a thin coal seam. This mine is planned to produce 750,000 tonnes of coking coal by fully mechanized longwall mining as a national project. This paper is aimed to the model weighting interval providing a reliable insight into understanding the caving potential for geotechnical engineers and designers of underground coal mines. For this purpose, the first weighting interval was calculated probabilistically incorporating empirical and analytical models, based on available data, from the mine’s geo-environment and Monte Carlo simulation (MCS) technique.

Several empirical (qualitative and quantitative), analytical (based on plate and beam theory) and numerical (continuum and discontinuum) models have been proposed in the literature to predict weighting intervals (Mohammadi, et al., 2019b). For this study, two quantitative empirical models and one analytical formula were selected, based on the available data for the statistical simulation using Monte Carlo technique (Table 1).

MCS determines mean and standard deviation of random variables function by performing repeated computations using randomly selected points estimated for component variables (Ang and Tang, 1975, 1984; Khalokakaie, et al., 2000). Using this method, the probability distributions of dependent random variables are calculated and different statistical moments are estimated. MCS is based on the below four steps:

- Step 1: determination of probability distribution for each input variable based on field data,
- Step 2: random sampling from a defined probability distribution,

¹ Ph.D., Shahrood University of Technology, Iran. Email: sadiadmohammadi@yahoo.com
² Professor, Shahrood University of Technology, Iran. Email: ataei@shahroodut.ac.ir
³ Senior lecturer, University of Southern Queensland, Australia. Email: ali.mirzaghorbanali@usq.edu.au
⁴ Professor, University of Wollongong, Australia. Email: naj@uow.edu.au
Step 3: using selected value for input parameters (step 2) to calculate output, and
Step 4: repeating steps 2 and 3 to generate a stable probability distribution for the output.

**Table 1: empirical and analytical models to estimate first weighting interval**

<table>
<thead>
<tr>
<th>Type</th>
<th>Model</th>
<th>Formula</th>
<th>Parameters</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Empirical</td>
<td>Rock Quality Index (RQI)</td>
<td>$L = 4.47(0.0064\sigma_c^{1.7}K_cK_t)^{0.4}$</td>
<td>$L$: first weighting interval (m); $\sigma_c$: UCS of roof rock (kg/cm$^2$); $K_c$: in situ strength coefficient (0.33 for sandstone, 0.42 for mudstone, and 0.5 for claystone or siltstone); $K_t$: creep coefficient (0.7 for sandstone and 0.6 for mudstone, clay stone or siltstone); $t$: strata thickness (m); $n$: a constant depending upon the RQD as: $1$: $RQD &lt; 33.3$; $n$: $33.3 &lt; RQD &lt; 66.6$; $2$: $66.6 &lt; RQD$</td>
<td>(Pawlowicz, 1967; Bilinski and Konopko, 1973)</td>
</tr>
<tr>
<td>CMRI (Indian model)</td>
<td>$L = 0.72 \left( \frac{(RQD - 26.75)^{0.3}(t_s^{0.5})}{5} \right)$</td>
<td>$L$: first weighting interval (m); $\sigma_c$: UCS of immediate roof (kg/cm$^2$); $t_s$: strata thickness (m); $n$: a constant depending upon the RQD as: $1$: $RQD &lt; 33.3$; $n$: $33.3 &lt; RQD &lt; 66.6$; $2$: $66.6 &lt; RQD$</td>
<td>(Sarkar, 1998; Singh, 2015)</td>
<td></td>
</tr>
<tr>
<td>Analytical</td>
<td>Beam theory</td>
<td>$L = \frac{2\sigma_c t}{E t}$</td>
<td>$L$: first weighting interval (m); $\sigma_c$: tensile strength of immediate roof (Pa); $t$: strata thickness (m); $\gamma_c$: effective unit weight of rock (N/m$^3$) as: $\gamma_c = \frac{E_i t_i}{\sum E_i t_i}$; $E$: Young’s modulus of the $i$th rock layer (GPa); $\gamma_c$: unit weight of the $i$th rock layer (N/m$^3$); $t_i$: thickness of the $i$th roof layer (m)</td>
<td>(Obert and Duvall, 1967)</td>
</tr>
</tbody>
</table>

**CASE STUDY**

The Parvadeh coalfield with an area of about 1200 km$^2$ and geological reserve of 1.1 billion tonnes is the largest coking coal deposit in Iran, located in the South Khorasan province (about 75 km from Southern Tabas). It is divided into five regions called Parvadeh I to IV and East Parvadeh. Parvadeh IV (PIV) is one of the largest sub-regions with an area of 110 km$^2$. Parvadeh IV is divided into two Northern and Southern sections by the Zenoghan fault. The Northern section comprises blocks 1 to 9 and the Southern section includes blocks 10 to 14. This case study was carried out on block 3 in the north section of Parvadeh IV with an area of 8.3 km$^2$ (highlighted area in Figure 1).

There are two minable coal seams in this mine (called C1 and B2) in which C1 is the major minable coal seam with the average thickness of 1 m. The first phase of this mine is aimed to extract 750 thousand tonnes of coal annually using the mechanized longwall retreat mining method. Based upon data of 57 boreholes in 10 sections, the overburden depth of C1 seam is less than 200 m with the average of 125 m. The immediate roof of the C1 seam is siltstone and sandy siltstone. The roof and joint sets are mostly dry (CMC, 2018). Since this project is in the basic design phase, there is no adequate geotechnical and geomechanical data. Therefore, data from various mines in the Parvadeh coalfield was collected and integrated with those of block 3 to establish the statistical database (Table 2).
Figure 1: The location of case study in Parvadeh Coal field

Table 2: Geomechanical properties of block 3 roof in Parvadeh IV

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Min.</th>
<th>Max.</th>
<th>Mean</th>
<th>S.D</th>
<th>Probability distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_c$ (MPa)</td>
<td>26.11</td>
<td>37.89</td>
<td>32</td>
<td>1.96</td>
<td>Normal</td>
</tr>
<tr>
<td>$\sigma_t$ (kg/cm$^2$)</td>
<td>266.43</td>
<td>386.63</td>
<td>326.53</td>
<td>20.03</td>
<td>Normal</td>
</tr>
<tr>
<td>$\sigma'$ (MPa)</td>
<td>1.82</td>
<td>3.18</td>
<td>2.50</td>
<td>0.23</td>
<td>Normal</td>
</tr>
<tr>
<td>$\gamma$ (N/m$^3$)</td>
<td>23030</td>
<td>26754</td>
<td>24892</td>
<td>620.67</td>
<td>Normal</td>
</tr>
<tr>
<td>$\gamma$ (ton/m$^3$)</td>
<td>2.35</td>
<td>2.73</td>
<td>2.54</td>
<td>0.06</td>
<td>Normal</td>
</tr>
<tr>
<td>RQD (%)</td>
<td>35</td>
<td>53</td>
<td>44</td>
<td>3</td>
<td>Normal</td>
</tr>
</tbody>
</table>

Equations (1) and (2) were used to calculate the immediate roof thickness as per Peng (2019):

$$h_{im} = \frac{H - d}{K - 1}$$  \hspace{1cm} (1)

$$d = c.H$$  \hspace{1cm} (2)

where $h_{im}$ is the immediate roof height (m), $H$ is the extraction height (m), $d$ is the sagging of the lowest un-caved strata, $K$ is the bulking factor of the immediate roof and $c$ is the ratio of the actual
strata sagging before caving to the mining height. In the Parvadeh coal filed, \( c \) and \( K \) were determined to be 0.50 and 1.25, respectively (Hosseini et al., 2014). Subsequently, by assuming the mining height to be equal to 1 m (CMC, 2018), the immediate roof height is calculated as 2 m.

RESULTS

*Crystal Ball* commercial software was used to simulate empirical and analytical models (Table 1) applying MCS. Iteration steps of simulation were set to 50,000 cycles to ensure achieving reliable output distributions.

Figure 2 shows the Probability Density Function (PDF) and Cumulative Density Function (CDF) of a simulated Rock Quality Index (RQI) model using MCS. It was noted that the immediate roof type is siltstone, thus, the coefficients of \( K_1 \), \( K_2 \) and \( K_3 \) were assumed to be 0.5, 0.6 and 0.6 respectively. Table 3 presents the statistical analysis of RQI based on the Mont Carlo technique.
Table 3: RQI statistical analysis

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Min.</th>
<th>Max.</th>
<th>Mean.</th>
<th>S.D.</th>
<th>95% confidence interval</th>
<th>Best fitted distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value (m)</td>
<td>11.92</td>
<td>18.01</td>
<td>15.28</td>
<td>0.64</td>
<td>14.01-16.51</td>
<td>Normal</td>
</tr>
</tbody>
</table>

Figure 3 shows PDF and CDF of the first weighting interval estimated by the Central Institute of Mining and Fuel Research index (CMRI) incorporating MCS. Table 4 presents statistical analysis of the first weighting interval based on CMRI and MCS.

Figure 3: Statistical analysis of CMRI using MCS

Table 4: CMRI statistical analysis

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Min.</th>
<th>Max.</th>
<th>Mean.</th>
<th>S.D.</th>
<th>95% confidence interval</th>
<th>Best fitted distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value (m)</td>
<td>8.50</td>
<td>26.77</td>
<td>19.13</td>
<td>1.98</td>
<td>15.09-22.88</td>
<td>Normal</td>
</tr>
</tbody>
</table>
By using analytical formula for estimating the first weighting interval, PDF and CDF were calculated based on the MCS as shown in Figure 4. Statistical analysis of the first weighting interval, based on MCS, is given in Table 5.

![Figure 4: Analytical model simulation using MCS](image)

**a. Probability density function**

![Figure 4: cumulative density function](image)

**b. cumulative density function**

**Figure 4: Analytical model simulation using MCS**

**Table 5: Statistical analysis using MCS and analytical model**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Min.</th>
<th>Max.</th>
<th>Mean.</th>
<th>S.D.</th>
<th>95% confidence interval</th>
<th>Best fitted distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value (m)</td>
<td>15.90</td>
<td>23.55</td>
<td>20.02</td>
<td>0.95</td>
<td>18.11-21.85</td>
<td>Normal</td>
</tr>
</tbody>
</table>
DISCUSSION

CDF of all models are depicted in Figure 5 to provide clear insights into the results for the sake of comprehensive comparison.

Figure 5 illustrates that the results of CMRI and the analytical model are approximately close to each other whereas the values of the RQI model are less. It is inferred from the above figure that the maximum expected first weighting interval estimated by RQI, CMRI and analytical models are 18, 27, and 24 m, respectively. In addition, the lower bound of the first weighting interval is 12, 9, and 16 m, estimated by RQI, CMRI and analytical models, respectively. Figure 5 shows that with 50% of probability, the first weighting interval will be between 15 and 20 m. The findings also indicate that the first weighting interval will be between 16 and 22 m with probability of 90%.

![Figure 5: CDFs of simulated first weighting interval](image)

The results indicate that the first weighting interval in block 3 of Parvadeh IV will be more than that of Parvadeh 1 (Hosseini et al., 2014; Mohammadi et al., 2018). According to previous projects in the Parvadeh district, there is reasonable agreement between the cavability class of the immediate roof and the predicted first weighting interval which represents reliability of the results.

CONCLUSIONS

The following main conclusions are drawn from this study:

- The results of probabilistic simulation of RQI showed that the first weighting interval varies with 95% probability in the range of 14 to 16.5 m. This model estimates the minimum and maximum possible first weighting interval as 12 and 18 m with an average of 15 m, respectively,

- Monte Carlo simulation of the CMRI showed that the first weighting interval varies in the range of 15 to 23 m with a probability of 95%. The minimum, maximum and average possible first weighting intervals estimated by the model are 8.5, 27, and 19 m, respectively,

- Probabilistic simulation of the analytical model based on beam theory indicated that the range of the first weighting interval with 95% of probability is between 18 to 22 m. This simulation showed that the minimum and maximum values of the weighting interval using this model will be 16 and 24 m, respectively. In addition, the average value of 20 m is expected,
The results showed that the outputs of CMRI and the analytical model agree well with each other whereas the results of RQI are less than both, and

It was concluded that the first weighting interval will be between 16 to 22 m with 90% of probability based on three models results.

REFERENCES

IMPROVING ROCK MECHANICAL PROPERTIES ESTIMATION USING MACHINE LEARNING

Ruizhi Zhong¹, Matt Tsang², Gift Makusha³, Ben Yang⁴ and Zhongwei Chen⁵,*

ABSTRACT: Rock mechanical properties (e.g., uniaxial compressive strength or UCS, Young’s modulus, and Poisson’s ratio) are important input parameters for geotechnical assessment and excavation designs. Two common methods used to obtain these parameters are laboratory testing and geophysical logging. The former delivers probably the most reliable results, but can be costly and time-consuming and for a lot of the time it is challenging to source sufficient samples. Alternative ways to better predict rock mechanical properties are needed.

In this case study, the XGBoost machine learning algorithm was applied to correlate laboratory and geophysical logging data with the three mechanical properties of UCS, Young’s modulus, and Poisson’s ratio. The proposed machine learning approach better predicted UCS values with a smaller Mean Absolute Error (MAE) and Root Mean Square Error (RMSE) and a larger $R^2$. Similarly, better results were obtained for the Young’s modulus prediction using the XGBoost algorithm. However, poor correlations existed between the inputs of geophysical and Poisson’s ratio, most likely due to the uncertainties associated with the acquisition of Poisson’s ratio data and the nature of this parameter. This study concluded that a machine learning approach has the potential to predict rock mechanical properties more reliably than the conventional methods, and further study is underway to have more quantitative and detailed analysis with more data inputs and other machine learning models.

INTRODUCTION

Rock mechanical properties are key inputs for many aspects of engineering design and analysis, from large slope stability to local excavation stability analysis, and from analytical analysis to numerical modelling. For underground coal mining operations, rock mechanical properties are critical for assessing the roof stability, strata movement, and the stability of Surface-to-In-Seam (SIS) gas drainage boreholes. These properties are generally obtained by direct laboratory testing of core samples or by in-situ measurements such as sonic logging. The former delivers probably the most reliable results, but can be costly and time-consuming. For highly fractured and/or very weak rocks sourcing core specimens can be difficult or impossible. Measurements of rock mechanical properties involves a number of standard tests (e.g., uniaxial compressive strength, point load test, triaxial and shear strength tests). It is impractical to gain sufficient core samples with a similar texture to conduct these standard tests because of the variability of the rock cores, which constantly vary with respect to lithotype, fracture intensity and microstructure, bedding, water content, state of stress in the rock mass, pore pressure, time, and compaction. All of these factors contribute to the intrinsic variability of rock strength, which is inherited to the sedimentation process. Therefore, the reliable prediction of these properties requires the inclusion of the factors.

The Uniaxial Compressive Strength (UCS) is probably the single most important parameter for geotechnical analysis, and in rock mechanics UCS is a key parameter to indirectly derive other fundamental properties, such as cohesion, friction angle, and tensile strength. The current industry standard practice is to use sonic logs to identify UCS values at depths of cover via a statistically averaged mine-specific function with the Sonic Transit Time (STT) or P-wave velocity (Oyler et al., 2010). As sonic velocity logs are relatively inexpensive and easy to obtain during exploration, the technique has provided Australian underground coal mines with an abundance of rock strength data for use in all aspects of ground control designs (Oyler et al., 2010), which is of particular importance for the lithological units where obtaining intact cores is impractical, or only a limited number of cores are available. However, the correlation lies on the assumption that the sensitivity of rock mechanical

¹ Postdoctoral Research Fellow, The University of Queensland. Email: r.zhong@uq.edu.au
² Geotechnical Specialist: Modelling, Anglo American Met Coal. Matt.Tsang@angloamerican.com
³ Manager Geotechnical, Anglo American Met Coal. Email: gift.makusha@angloamerican.com
⁴ Graduate Geotechnical Eng, Anglo American Met Coal. Email: Ben.Yang@angloamerican.com
⁵ Senior Lecturer, The University of Queensland. Email: zhongwei.chen@uq.edu.au, *Corresponding Author
properties is solely correlated to the sonic velocity, with the exclusion of a number of other influencing factors, such as (i) rock porosity, (ii) rock mass composition, in particular, the organic, shale and quartz contents, (iii) large difference in strata velocities causing cycle skipping, and (iv) attenuation across joints or fractures causing cycle skipping (Hatherly et al., 2005; Hatherly et al., 2001; Oyler et al., 2010). The above factors are routinely acquired through geophysical logs for exploration boreholes. They offer a great amount of information for lithology assessment and strata characterisation. Therefore, theoretically, a better correlation may be gained if some of the above factors, if not all, were explicitly considered in the correlation fitting.

Although the potential value of geophysical logs has been increasingly realised, the direct use of geophysical data for geotechnical assessment has yet to become an industry standard practice. Nevertheless, there has been significant progress in this area, such as the development of the Geophysical Strata Rating (GSR) (Hatherly et al., 2008), which aims to provide a measure of rock quality through the combined analysis of three sets of geophysical logging data: sonic log, density log, and natural gamma ray log (Zhou and Guo, 2020). McNally conducted a comprehensive study aiming to predict geotechnical rock properties from sonic and neutron logs, and concluded that sonic and neutron logs tend to overestimate porosity in clay-rich coal measures rock (McNally, 1990). Zhou et al. (2005) later applied rock strength from geophysical logs using the Radial Basis Function (RBF) and Self Organizing Maps (SOM) methods to improve the work of McNally (1990). There are a significant amount of other studies related to the utilisation of geophysical data for various other purposes, such as, using full-wave sonic logs to determine the Young's, shear and bulk moduli, as well as porosity and Poisson's ratio (Feng and Jimenez, 2014; Karacan, 2009a; Karacan 2009b), and characterising rock fracture features (e.g., spacing, length, and aperture) using image logs (Li et al., 2018; Özkaya, 2003).

Over the last decade, machine learning techniques have rapidly evolved and have contributed to many areas of mining design and safety assessment. A few wireline-log-data based studies have been conducted using machine learning to classify the lithology of conventional and unconventional hydrocarbon reservoirs, such as employing artificial neural network (ANN) (Wang and Carr, 2012; Wang and Zhang, 2008), support vector machines (SVM) (Al-Anazi and Gates, 2010) or other advanced machine learning algorithms (Xie, et al., 2018) to identify the lithology, including the identification of thin coal seams conducted by the authors (Zhong, et al., 2020a; Zhong, et al., 2020b). These wireline-log-data based studies have shown good accuracy in the range of 70-100%. In addition, there have been a few attempts to apply machine learning for estimating rock mechanical properties from logging data (Gong, et al., 2019), core analysis data (Mahmoud, et al., 2020), or drilling data (Igor, et al., 2017).

This work aims to apply a machine learning approach to utilise the significant amount of rock laboratory measurements and geophysical data in the hope of improving the prediction of rock mechanical properties. We first present correlations between rock properties, which serve as the basis for applying a machine learning algorithm to estimate other rock properties. Then we show the workflow of the project and introduce the machine learning algorithm (i.e., XGBoost). To evaluate the model performance, R², Mean Absolute Error (MAE), and Root Mean Square Error (RMSE) are used as the evaluation metrics. Subsequently, the geophysical data and its processing method are described, followed by the summary and discussion of machine learning results. Finally, concluding remarks are presented, and other possible applications are suggested.

METHODOLOGY

In this paper, a series of laboratory data and the corresponding geophysical logs (i.e. sonic, density, and gamma-ray logs) for the same depth are combined and used to assess the effectiveness of a machine learning model in improving rock mechanical properties estimation. The collected data are relevant to open cut and underground operations in the Bowen Basin, Australia.

The data analysis process is sequenced in five steps, as shown in Figure 1. The data pre-processing (Step 1) is to filter the collected laboratory and field geophysical data to ensure each data value makes physical sense, such as the range of coal and rock density. Based on the processed data, the second step is to directly correlate each individual geophysical logging parameter (i.e., sonic, density and gamma ray) with the UCS (as the output) using direct curve fitting. For Step 3, the machine learning algorithm (XGBoost) is applied to predict UCS using a combination of the three geophysical logging parameters. The final two steps focus on predicting Elastic Modulus and Poisson's Ratio using the
same approach as Step 3. Quantification of the importance of each parameter to the predicted parameter values for Steps 3-5 will be obtained at the end.

Figure 1: Diagram of the sequence of the data analysis and machine learning.

Figure 2 shows the schematic of the predictor generation. Conventionally, curve fitting methods (e.g., linear function, exponential function, etc.) are used to generate the predictor (i.e., method A in Figure 2). In this paper, an additional three types of correlations are also applied to find the best fitting: logarithmic, polynomial, and power. These types of correlations are commonly used in Excel to find the correlation between parameters. To obtain better correlations, XGBoost is used in this study as the machine learning algorithm. The function of XGBoost is the same as the curve fitting method to find a correlation between parameters. Compared to other algorithms, XGBoost generally has a stable and better performance. XGBoost was initially developed in 2014 and has gained much popularity in many data science competitions due to its state-of-the-art accuracy on many problems (Chen and Guestrin, 2016). It is an optimized distributed gradient boosting library designed to be highly efficient, flexible, and portable. It implements under the framework of gradient boosting. Boosting refers to any ensemble method that can combine several weak learners into a strong learner. Gradient boosting works by sequentially training a new predictor to correct the residual errors made by the previous predictor. Previously, the XGBoost and other machine learning algorithms were used to identify coal section (Zhong et al., 2020b) and to generate a pseudo density log (Zhong, et al., 2020a), which delivered very promising results in both applications.

Note that the curve fitting method and XGBoost use all the data to generate the predictor and then output the results.

Figure 2: Schematic of the predictor generation for both conventional direct curve-fitting and XGBoost approach.

Evaluation metrics

In this paper, we use the $R^2$ as the evaluation metric. $R^2$ is also called the coefficient of determination, which quantifies the variance of an independent variable from the other independent
variable(s). \( R^2 \) is the square of \( r \), the Pearson correlation coefficient (or Pearson’s \( r \)), which measures the linear correlation between two variables \( X \) and \( Y \). For a dataset with samples of \((x_i, y_i)\), \( r \) and \( R^2 \) have the following expression

\[
r = \frac{\sum_{i=1}^{n} (x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\sum_{i=1}^{n} (x_i - \bar{x})^2 \sum_{i=1}^{n} (y_i - \bar{y})^2}}
\]

\[
R^2 = r^2
\]

where \( \bar{x} \) is the average of \( x_i \); \( \bar{y} \) is the average of \( y_i \); \( n \) is the sample number. Additionally, we also use mean absolute error (MAE) and root mean square error (RMSE) to quantify the results, which have the following expression:

\[
MAE = \frac{1}{n} \sum_{i=1}^{n} |y_i - \hat{y}_i|
\]

\[
RMSE = \sqrt{\frac{1}{n} \sum_{i=1}^{n} (y_i - \hat{y}_i)^2}
\]

where \( \hat{y}_i \) is the \( i \)th sample output from machine learning. MAE measures the average of all absolute errors, and RMSE can detect the error variance. Before we feed data into XGBoost, some data processing steps are conducted. Samples with missing or zero values are removed. Furthermore, some abnormal data points with a density less than 1.0 g/cc or greater than 4.0 g/cc are removed. After processing, Table 1 shows the sample number for each task.

<table>
<thead>
<tr>
<th>Output</th>
<th>Sample number</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS</td>
<td>4950</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>277</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>1953</td>
</tr>
</tbody>
</table>

### Results and Discussions

**Results of Uniaxial Compressive Strength**

Figure 3 shows the results of UCS using direct curve fitting with each single input (i.e., STT, density, or gamma ray). The blue dashed lines are curve fitting using the linear correlation, and red dashed lines are cases with the highest \( R^2 \) from all four selected correlations. Both STT and density show a good correlation with UCS, but a low correction is observed for gamma ray as the \( R^2 \) is only 0.0411. The STT delivers the strongest correlation with the \( R^2 \) being 0.4118.

![Figure 3: Results of curve fitting using single input to generate UCS. Blue dashed lines are linear correlation and red dashed lines are the best correlation. (a) Input is STT; (b) Input is density, and (c) Input is gamma ray.](image)

All three parameters were then used as inputs for machine learning fitting. The 3D results are shown in Figure 4. The red points are actual UCS, and blue points are UCS generated from machine learning. Compared to Figure 3, the fit is better, and the blue points are well-matched with red points. We
would like to mention that additional attempt was also conducted to apply XGBoost to each individual parameter and the combination of two parameters (i.e., STT+ Density, STT+ Gamma ray, and Density+ Gamma ray). A gradual improvement of predicted results was observed with the increase in the number of input parameters, but due to the page limit the associated results are not presented here. They are available upon request.

Note that in Figure 3(a) the samples with an STT smaller than 200 μsec/m (i.e., P-wave velocity greater than 5.0 km/s) are more discrete than samples with an STT larger than 200 μsec/m. Thus, if only samples with STT greater than 200 μsec/m are considered, we can expect better results. After removing the samples with STT smaller than 200 μsec/m, the sample number is 4884 (i.e., 66 samples are filtered out). Figure 5 shows the curve fitting results for the filtered STT samples. Compared to Figure 3, a better correlation is obtained for the STT vs. UCS as the $R^2$ increases from 0.4118 to 0.4828. For the density or gamma ray vs. UCS, the correlation is almost the same.

Figure 4: 3D view of results of machine learning using three inputs (i.e., STT, density, and gamma ray) for UCS prediction.

Figure 5 shows the results of filtered samples using the XGBoost. There is an improved match between the actual UCS and predicted UCS by XGBoost. The improvement is largely attributed to the exclusion of STT values smaller than 200 μsec/m. It is arguable that the predicted results may be further improved as well by slightly increase the density range to around 1.3 g/cc, but due to the limited data available at this density range, we decided to only exclude the data points with density values no greater than 1.0 g/cc. The overall trend of the results is very similar to Figure 4.

Figure 5: Results of curve fitting using single input to generate UCS. Only samples with STT larger than 200 μsec/m are considered. Blue dashed lines are linear correlation and red dashed lines are the best correlation. (a) Input is STT; (b) Input is density, and (c) Input is gamma ray.

The comparison of the corresponding performance to both direct curve-fitting and machine learning is summarised in Table 2. The MAE for the direct curve-fitting method ranges from 14.844 MPa to 16.94 MPa, and $R^2$ varies from 0.041 to 0.412. When XGBoost is used, we observed a better performance with a MAE of 10.893 MPa and $R^2$ of 0.578. Compared to the best case using the curve fitting (i.e., STT with a MAE of 14.844 MPa and $R^2$ of 0.412), the MAE is decreased by 3.951 MPa. The reduction of error percentage is 26.61% (3.951 MPa/14.844 MPa), indicating that machine learning delivers a better performance.
If samples with an STT greater than 200 μsec/m are removed, as explained for Figure 5, the performance of curve fitting using STT is significantly improved: the MAE decreases from 14.844 MPa to 12.161 MPa. However, the performance of machine learning is only slightly enhanced, in which the MAE decreases from 10.893 MPa to 10.783 MPa. Nevertheless, this still indicates further data processing (or filtering) can improve model performance.

Finally, the feature importance based on the XGBoost algorithm is also extracted and ranked. It is found that STT is the most important feature, accounting for 42.84%, followed by the density (31.25%) and Gamma ray (25.91%). The feature importance of STT can also be seen through curve fitting results in Table 2. The results of direct curve fitting using STT shows the best performance giving a MAE of 14.844 MPa and $R^2$ of 0.412. The feature importance of density and gamma-ray is also consistent with the results of curve fitting in Table 2.

| Table 2: Performance of curve fitting and machine learning for UCS prediction. |
|-----------------------------|---------|--------|---------|
| Input                       | MAE (MPa) | RMSE (MPa) | $R^2$ Score |
| Curve fitting               |          |        |          |
| STT                         | 14.844   | 21.749 | 0.412    |
| Density                     | 16.475   | 46.611 | 0.3      |
| Gamma                       | 16.94    | 21.619 | 0.041    |
| Curve fitting (STT>200 μsec/m) |        |        |          |
| STT                         | 12.161   | 16.313 | 0.483    |
| Density                     | 16.811   | 60.458 | 0.3      |
| Gamma                       | 16.784   | 21.314 | 0.043    |
| XGBoost – three inputs      | STT+Density+Gamma | 10.893 | 14.325 | 0.578 |
| XGBoost – three inputs (STT>200 μsec/m) | STT+Density+Gamma | 10.783 | 14.15  | 0.578 |

Figure 6: 3D view of results of machine learning using three inputs (i.e., STT, density, and gamma ray) for UCS prediction. Only samples with STT larger than 200 μsec/m are considered.

Results of Young’s modulus

Figure 7 shows the results of curve fitting for every single input (STT, density, or gamma ray) for Young’s modulus (E) prediction. As explained in Table 1, the sample number for Young’s modulus modelling is limited (only 277 samples). STT has a good correlation with E with a $R^2$ of 0.2557. The density also has a noticeable correlation with $R^2$ of 0.1333. However, gamma ray does not show a meaningful correlation with Young’s modulus as the $R^2$ is close to 0. Thus, if we use simple curve fitting, the best option is to use STT or density to predict E. We would like to point out that in the literature there have been some good correlations of E with STT or density observed from hard rocks, but our results do not show a similar trend, likely due to the limited number of data points available.

Figure 8 shows the 3D view of the machine learning results using all three geophysical logging data sets as the inputs for Young’s modulus fitting. The figure illustrates that results predicted by XGBoost match well with the measured Young’s modulus, delivering better results than from the direct curve fitting.

The goodness of the predicted results of Young’s modulus from both curve fitting and machine learning is summarised in Table 3. The direct curve fitting using density as the input gives the best performance with an MAE of 3.907 GPa, RMSE of 6.285 GPa, and $R^2$ of 0.1333. Although using STT delivers larger $R^2$, the MAE and RMSE are smaller. The results of UCS using the XGBoost gives an MAE of 2.918 GPa, RMSE of 3.915, and $R^2$ of 0.779. This means that, for the tested data set, the
machine learning approach better predicts $E$ values, which approximately decreases the error by 1GPa.

![Figure 7](image1.png)

**Figure 7:** Results of curve fitting using single input to generate Young's modulus. Blue dashed lines are linear correlation and red dashed lines are the best correlation. (a) Input is STT; (b) Input is density, and (c) Input is gamma ray.

Unlike for UCS, the density shows the most important feature from XGBoost (43.22%). STT is the second important feature (32.85%), followed by gamma ray of 23.92%. The possible reason of change of feature importance is the small sample size. In other words, the feature importance is only valid for this dataset.

![Figure 8](image2.png)

**Figure 8:** 3D view of the machine learning results using three inputs (STT, density, and gamma-ray) for Young's modulus prediction.

**Table 3: Performance of curve fitting and machine learning for Young's modulus prediction.**

<table>
<thead>
<tr>
<th>Input</th>
<th>Mean absolute error (GPa)</th>
<th>Root mean absolute error (GPa)</th>
<th>R² Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curve fitting</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>STT</td>
<td>6.583</td>
<td>9.434</td>
<td>0.2557</td>
</tr>
<tr>
<td>density</td>
<td>3.907</td>
<td>6.285</td>
<td>0.1333</td>
</tr>
<tr>
<td>Gamma</td>
<td>6.238</td>
<td>8.309</td>
<td>0.0182</td>
</tr>
<tr>
<td>XGBoost – three inputs</td>
<td>2.918</td>
<td>3.915</td>
<td>0.779</td>
</tr>
</tbody>
</table>

**Results of Poisson's ratio**

Figure 9 shows the results of curve fitting using single input (i.e., STT, density, or gamma ray) for Poisson's ratio prediction. $R^2$ for all input parameters are smaller than 0.02, which indicates these input parameters have almost no correlation with Poisson's ratio. Thus, machine learning is not used because results generated by machine learning will be meaningless and only represent statistical results.

A possible reason for weak correlations is Poisson's ratio has poor correlation with these three inputs in nature. Another possibility is uncertainties associated with the measurement of Poisson’s ratio in the laboratory. In the raw data, there is a considerable proportion of Poisson's ratio falling into the range of 0.4 to 0.5 (or even above), which is much higher than for typical sedimentary rocks. In the literature, there is a lack of conclusive correlation between Poisson's ratio and any geophysical parameters, though a reduction in Poisson's ratio with Young's modulus is occasionally observed. Theoretically, this can be explained using the equation (5), which illustrates how to calculate dynamic Poisson’s ratio based on sonic velocity.

$$v = \frac{V_P^2 - 2V_S^2}{2 (V_P^2 - V_S^2)}$$

where $E_d = \text{Dynamic Young's Modulus}$
\(V_p, V_s = \text{Compressional/ Shear wave velocities}\)
\(\nu = \text{Dynamic Poisson’s ratio coefficient}\)

From the equation, it can be seen that the value of dynamic Poisson’s ratio was affected by both P and S wave velocities (two dimensional). This is different from UCS and Young’s modulus, which are largely determined by P-wave velocity (or STT) alone.

**Figure 9:** Results of curve fitting using a single input to generate Poisson’s ratio. Blue dashed lines are linear correlation and red dashed lines are the best correlation. (a) Input is STT, (b) Input is density, and (c) Input is gamma ray.

**CONCLUSIONS**

In this paper, we attempted to use the XGBoost algorithm, a machine learning technique, to estimate three rock mechanical properties (UCS, Young’s modulus, and Poisson’s ratio) by integrating three sets of geophysical log data (Sonic, Gamma ray, and Density logs). Main findings from this study are:

1. The results of UCS from the combined data set of STT, density, and gamma-ray using the XGBoost deliver an MAE of 10.873 MPa, RMSE of 14.294 MPa, and \(R^2\) of 0.58, which produces better prediction results than the common practice of direct curve. The most important feature among three inputs is STT.

2. A similar improved trend was observed for the prediction of Young’s modulus. The results of Young’s modulus from the combined data set using the XGBoost deliver an MAE of 2.918 MPa, RMSE of 3.915 MPa, and \(R^2\) of 0.779. The density is the most important feature for Young’s modulus prediction, but no significant difference in the coefficient value of each important feature is observed between UCS and Young’s modulus prediction.

3. No noticeable correlations between inputs parameters (STT, density, and gamma ray) and Poisson’s ratio was observed for the studied data set, which is consistent with the existing literature.

The results from this works demonstrate that the machine learning technique is able to better predict the rock mechanical properties by integrating the geophysical log data, which is of particular importance for estimating rock properties for the boreholes with limited laboratory data available. The use of more reliable parameter values will help reduce the uncertainties associated with the subsequent geotechnical design and assessment. Further studies are underway to understand the differences in results between different machine learning models.

**REFERENCES**


Karacan, C.O., 2009a. Reservoir rock properties of coal measure strata of the Lower Monongahela Group, Greene County (Southwestern Pennsylvania), from methane control and production perspectives, International Journal of Coal Geology, 78, 47-64.
Mott, P.H., Roland, C.M. 2013. Limits to Poisson’s ratio in isotropic materials - general result for arbitrary deformation, Physica Scripta, 87, 055404.
CHARACTERIZATION OF THE FRACTURE MODE IN ASPHALT AT VARYING TEMPERATURES

Mehdi Serati¹, Thejaswe Valluru² and Ian Van Wijk³

ABSTRACT: Cracking is a primary mode of distress in asphalt pavements that is generally caused due to repeated traffic loadings, exposure to temperature fluctuations, aging or reflection of cracks in underlying layers. Such cracking can readily lead to higher maintenance and rehabilitation costs for pavement infrastructure, hence negatively affecting the economy both directly and indirectly. To prevent excessive cracking, it is important to understand the cracking characteristics of asphalt mixtures for implementation in road, airport and port pavements. This study aims to investigate the effect of loading and temperature on the cracking behaviour of asphalt using the Indirect Tensile Test (IDT), along with high-speed photography analysis techniques. The results indicate that cracking can occur prior to the asphalt reaching its peak strength in the IDT test. Furthermore, it was observed that increasing temperature can cause a decrease in the peak strength of the asphalt samples and change its fracturing behaviour as well.

INTRODUCTION

The road industry is a vital part of the Australian economy with contributions of over $200 billion in economic value, along with half a million Australians relying on roads for their full-time employment (Roads Australia, 2020). With such high reliance on road networks, the construction and road maintenance are crucial as any premature failures could result in large costs that can negatively affect the social prosperity and economic progress. The main structural elements of a road include pavement and subgrade, which are subjected to repeated mechanical impacts of vehicles and daily changing climatic factors. These mechanical impacts and temperature fluctuations are the most critical factors that govern the overall performance of asphalt mixes as reflected in its strength and fatigue performance (Teltayev and Suppes, 2018).

Depending on the site location, temperatures can vary drastically during a year (e.g. the average temperatures in Australia in a year can change from 3 °C to 35 °C between warm days in summer and cool evenings in winter), and these changes can aggravate the damages caused to the asphalt due to temperature fluctuations over time (Aussie Specialist, 2020; Teltayev and Suppes, 2018). However, while many studies have investigated the ultimate fracture strength in asphalt pavements at different temperatures using methods such as Semi-Circular Bend (SCB) and Three-Point Bend tests (Zhou & Newcomb, 2016), there have not been many applications of high-speed photogrammetry in the study of the dominant fracturing/cracking pattern (i.e. the crack initiation and propagation) in asphalt mixes at varying temperatures.

Further, it is understood that for low porosity rock-like geomaterials, the stress required to initiate microcracks (known colloquially as the Crack Initiation point) is considerably less than the material's peak strength (Nicksiar & Martin, 2012). While this has not been thoroughly verified with cracking in asphalt pavements, the authors observed in a recent study that macrocracks could also be identified in asphalt samples well before the sample's peak strength even if the temperature remains unchanged during the test (Serati, et al., 2020). This recent study of the authors was, however, conducted with limited test samples and more investigations were deemed necessary to confirm the initial observations.

With the above objectives in mind, the current work aims to validate whether cracks do indeed appear before an asphalt sample reaches its peak strength during the IDT test. If tensile cracking could occur prior to the maximum load in the IDT method, alternative testing techniques should then be used instead, or an adjustment factor to be applied to compensate for the overestimation of asphalt tensile strength in the IDT method. In addition, this study investigates the effect(s) of varying temperature on both the strength and cracking/fracture pattern of some selected asphalt mixtures.

¹ Lecturer, The University of Queensland. Email: m.serati@uq.edu.au Tel: +61 7 3365 3911
² Student, The University of Queensland. Email: t.valluru@uqconnect.edu.au
³ Adjunct Professor, The University of Queensland. Email: l.vanwijk@uq.edu.au Tel: +61 4 0239 9948
METHODOLOGY

Many linear elastic fracture mechanics-based (LEFM) models assume an ideal isotropic and homogenous material when describing crack propagation in a solid. These models also suggest a continuous extension of pre-existing flaws as a method of crack propagation (Serati, et al., 2015; Masoumi, et al., 2017; Roshan, et al., 2018; Serati, et al., 2020). However, asphalt is a heterogeneous and viscoelastic composite material consisting of aggregate, binder, air voids and microstructure with complex geometries. The available LEFM theoretical models are therefore generally inadequate to understand crack propagation behaviour in asphalt mixes. Direct laboratory testing is thus considered as the most accurate and convenient approach for the study of asphalt fracture characterisation, provided the testing equipment is available. Several testing methods have been developed and are widely used, namely the semi-circular bend (SCB), disk-shaped compact tension (DCT), uniaxial thermal stress and strain test (UTSST), Texas overlay test (OT), and the indirect tension (IDT) test (Zhou and Newcomb, 2016; Serati, et al., 2017). The IDT test, in particular, was designed to characterise the static and dynamic cracking performance of asphalt pavements and is classified as a test with excellent ease of sample preparation and good reproducibility (Christensen and Bonaquist, 2004). A simple IDT test configuration is also shown in Figure 1. The IDT test was therefore chosen as the preferred testing method in this study.

![Figure 1: Schematic of the IDT testing configuration (Zhou & Newcomb, 2016)](image)

A total of 15 field-cored asphalt briquettes were used that were prepared using standard laboratory compaction. The samples comprised of 14 mm aggregate with a modified binder (A15E) compacted to 50 blows Marshall compaction. Samples were sprayed with white spray paint (on the face) to provide contrast to the natural dark colour of the asphalt as this contrast aids the recognition of cracking propagation during analysis. In order to test the temperature effect, there were three designated temperature ranges for testing:

- 5 °C for lower bound temperature analysis;
- Room temperature at 24 °C; and
- 35 °C for upper bound temperature analysis.

These temperatures were carefully selected to replicate the temperature ranges observed in Australian climates throughout the year. To develop each target temperature within the samples, five asphalt samples were placed in a refrigerator set to 5 °C whereas four samples were placed in an oven set at 35 °C; each sample for more than 72 hours before testing. Samples were then tested immediately (within less than two minutes) after being taken out of the refrigerator/oven to ensure the samples did not transition back into the room temperature and neither cools nor heats up during testing. A Phantom v2012 ultra-high-speed camera was also used to monitor the cracking pattern in tested samples. The camera is capable of capturing images at up to 1,000,000 frames per second (fps) at reduced resolution, and up to 22 kHz at a full resolution of 1280 x 800 pixels (Phantom, 2020). Such capabilities make the camera system a suitable gear for monitoring the cracking process of brittle solids (Serati and Williams, 2015; Serati, et al., 2018; Bahaaddini, et al., 2019). Two sets of displacement sensors were selected to capture horizontal and vertical deformations during loading. The vertical displacement was measured using the load frame signal (after being calibrated to account for the machine deformation) whereas a Linear Variable Differential Transformer (LVDT) sensor (detailed below) was used and connected directly to the samples to record the horizontal expansion.

- Burster 8712-50 Linear Transducer LVDT (Burster, 2020)
• Measurement range is 50 mm
• Linearity is ±0.1%

In order to synchronize the load and displacement signals with the high-speed camera, a high-resolution National Instruments Data Acquisition (NI USB-6221 DAQ) unit was further utilized. An Infrared Light Circuit Unit was designed and connected to the DAQ system to perform preliminary calibration tests to ensure various sources of signals collected during each test (also summarised in Table 1) are in proper synchronization with the high-speed camera recordings. The schematic of the DAQ and the test setups are illustrated in Figure 2 and 3.

<table>
<thead>
<tr>
<th>Data Source</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load</td>
<td>Instron 4505 load frame</td>
</tr>
<tr>
<td>Vertical displacement</td>
<td>Instron 4505 load frame</td>
</tr>
<tr>
<td>Horizontal displacement</td>
<td>Linear Variable Differential Transformer (LVDT) sensor</td>
</tr>
<tr>
<td>Video capture</td>
<td>Phantom v2012 high-speed camera</td>
</tr>
</tbody>
</table>

**Table 1: Data acquisition sources**

**RESULTS**

It should be noted that since displacement measurements using the load frame sensors provides a combined deformation measure of the machine (metal platens) and the asphalt samples, the strain of the metal platens should always be subtracted by means of a preliminary calibration. Therefore, the vertical displacement from the LVDT was compared to that of the readings from the load frame first. Several control tests were run at different displacement rates to provide reference/benchmark points. After the data was acquired from the LVDT and the load frame, the two sets of data were then graphed to find the relationship between the LVDT and the load frame readings. Interestingly, it was found that the load frame readings are in all cases up to 27% higher than the LVDT recordings. A correction factor was then used and applied to the load frame displacement readings to obtain the sample vertical deformation directly. The sample horizontal deformation was also measured using the Burster LVDT transducer, as explained above (see also Figure 3).
Most of the tests were captured at 4,500 fps and a few at higher frequencies of 10,000 fps and above. Camera recordings were then carefully analysed using an image processing software (Phantom Camera Control, PCC; Serati, et al., 2017) to determine the very first frame at which a macro crack was observed on the sample face. For each test, the load at failure, frame number at which the first crack was identified (and its corresponding force and deformations from the load and LVDTs signals), and the sample's tensile strength; were recorded. In some cases, image sharpening techniques using the Laplacian filter was further applied to identify the initial macro crack(s). Table 2 and Figure 4 summarize the results in which areas highlighted by yellow circles represent points on the sample where fresh macro cracks were spotted first at each stress level. All tests were conducted at 120 mm/min.

### Table 2: Summary of test results for crack appearances

<table>
<thead>
<tr>
<th>Sample</th>
<th>Diameter (mm)</th>
<th>Thickness (mm)</th>
<th>Temperature (°C)</th>
<th>Tensile strength (MPa)</th>
<th>Maximum load at failure (kN)</th>
<th>Load when the first macrocrack was observed (kN)</th>
<th>Percentage of max load (%)</th>
<th>Cracking pattern</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC14H-01</td>
<td>98.06</td>
<td>61.49</td>
<td>35</td>
<td>0.66</td>
<td>6.25</td>
<td>5.86</td>
<td>93.76</td>
<td>Tensile</td>
</tr>
<tr>
<td>AC14H-02</td>
<td>97.74</td>
<td>61.28</td>
<td>35</td>
<td>0.63</td>
<td>5.92</td>
<td>5.01</td>
<td>84.63</td>
<td>Tensile</td>
</tr>
<tr>
<td>AC14H-03</td>
<td>98.46</td>
<td>61.34</td>
<td>35</td>
<td>0.67</td>
<td>6.39</td>
<td>1.77</td>
<td>27.70</td>
<td>Tensile</td>
</tr>
<tr>
<td>AC14H-04</td>
<td>97.66</td>
<td>61.40</td>
<td>35</td>
<td>0.65</td>
<td>6.11</td>
<td>5.79</td>
<td>94.76</td>
<td>Tensile</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.65</strong></td>
<td><strong>6.17</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Coefficient of Variation (COV)</strong></td>
<td><strong>2.47%</strong></td>
<td><strong>2.80%</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AC14H-06</td>
<td>98.61</td>
<td>62.93</td>
<td>5</td>
<td>2.57</td>
<td>25.05</td>
<td>17.65</td>
<td>70.46</td>
<td>Shear</td>
</tr>
<tr>
<td>AC14H-07</td>
<td>98.58</td>
<td>62.11</td>
<td>5</td>
<td>2.91</td>
<td>28.02</td>
<td>26.40</td>
<td>94.22</td>
<td>Tensile</td>
</tr>
<tr>
<td>AC14H-08</td>
<td>98.26</td>
<td>61.74</td>
<td>5</td>
<td>1.33</td>
<td>36.76</td>
<td>36.52</td>
<td>93.95</td>
<td>Shear</td>
</tr>
<tr>
<td>AC14H-09</td>
<td>98.34</td>
<td>62.16</td>
<td>5</td>
<td>3.47</td>
<td>33.34</td>
<td>27.39</td>
<td>82.15</td>
<td>Shear</td>
</tr>
<tr>
<td>AC14H-10</td>
<td>97.32</td>
<td>62.30</td>
<td>5</td>
<td>2.34</td>
<td>22.30</td>
<td>22.20</td>
<td>99.55</td>
<td>Tensile</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>2.53</strong></td>
<td><strong>29.09</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Coefficient of Variation (COV)</strong></td>
<td><strong>28.04%</strong></td>
<td><strong>18.21%</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AC14H-11</td>
<td>98.73</td>
<td>65.35</td>
<td>Room</td>
<td>1.77</td>
<td>17.93</td>
<td>15.92</td>
<td>88.79</td>
<td>Tensile</td>
</tr>
<tr>
<td>AC14H-12</td>
<td>98.22</td>
<td>62.34</td>
<td>Room</td>
<td>2.19</td>
<td>21.07</td>
<td>19.26</td>
<td>91.41</td>
<td>Tensile</td>
</tr>
<tr>
<td>AC14H-13</td>
<td>98.32</td>
<td>62.12</td>
<td>Room</td>
<td>2.12</td>
<td>20.26</td>
<td>19.84</td>
<td>48.33</td>
<td>Tensile</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>2.03</strong></td>
<td><strong>19.79</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Coefficient of Variation (COV)</strong></td>
<td><strong>9.10%</strong></td>
<td><strong>6.79%</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### DISCUSSION

When a low-porosity rock is subjected to a uniaxial loading condition, it responds elastically up to the Crack Initiation (CI) point which is defined as the onset of stress-induced damage after the closure of pre-existing cracks. The CI level is approximately one-third of the unconfined compressive strength, but the actual damage becomes only visible after the coalescence of microcracks initiated and extended past the CI stress threshold (Nicksiar and Martin, 2012). Similarly, when testing asphalt under the IDT load configuration, the seven (7) stages below in Table 3 are typically identified, but it is believed that macroscopically visible cracks can only be observed once the peak load is reached (Zhou et al., 2017).

In contrast, several of the obtained results in this study indicated otherwise. That is, the cracks started appearing at various stages prior to the material's maximum strength. Examples of snapshots from high-speed recordings are shown in Figure 4 that demonstrate the state of the sample at the respective points of the load curve. A good potential explanation for this observation could be the transition from tensile to shear cracks happening simultaneously at local points inside a failed IDT asphalt sample. It should be noted that this observation isn’t valid during the indirect tensile testing of a rock specimen where the rupture mechanism is solely governed by the initiation and propagation of tensile cracks. An endeavour towards validating this hypothesis using more tests and further numerical modelling is underway. In addition, another objective of this study was to observe the effect of temperature on the behaviour of asphalt under the IDT test. The results show that there is a statistically significant difference between the asphalt strength as the temperature of the sample changes. For example, the cooler the sample, the higher the average peak strength observed. This
can be seen more clearly in Table 2 where the cooler samples at 5 °C exhibit almost a 5-times higher peak strength, on average, compared to the samples tested at 35 °C (see also Figure 4).

**Table 3: Seven stages of the IDT test’s load-displacement curve (Zhou, 2017)**

<table>
<thead>
<tr>
<th>Segment</th>
<th>Stage</th>
<th>Load range and characteristic</th>
<th>Specimen status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-peak load</td>
<td>1</td>
<td>0 – 1/3 peak load</td>
<td>No visible crack</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1/3 – 2/3 peak load</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>2/3 – peak load</td>
<td></td>
</tr>
<tr>
<td>Peak load</td>
<td>4</td>
<td>Peak load point</td>
<td>Macro-crack starts to appear</td>
</tr>
<tr>
<td>Post-peak load</td>
<td>5</td>
<td>Peak load – 2/3 peak load</td>
<td>Macro-crack starting to be visible</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>2/3 – 1/3 peak load</td>
<td>Crack propagating quickly and more visible</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>1/3 peak load – 0 load</td>
<td>Specimen separation into 2 or more pieces</td>
</tr>
</tbody>
</table>

![Graphs](image)

**Figure 4: Summary of the test results conducted at varying temperatures**

As the temperature increases, the failure in rock often transitions from brittle to a more ductile rupture mechanism, particularly in the post-peak region (Hudson and Harrison, 1997; Syagala, et al., 2014). This behaviour can also clearly be seen in the high-speed recordings obtained with the asphalt samples as shown in Figure 4. The cooler sample at 5 °C exhibits a steeper increase and decrease in the load when compared to the sample tested at higher temperatures. However, when looking at the results in Table 2, there is also a higher variability (i.e. a larger Coefficient of Variation) in the maximum load experienced by the cooler samples. An explanation might be that lower temperatures can result in un-even damage to the asphalt microstructure matrix. More interestingly, some of the cooler samples exhibit a distinct shear cracking/fracturing behaviour in comparison to the single-tensile cracking (which is expected in the IDT test) observed with the samples tested at room and 35 °C temperatures (see Figure 5). Only a limited amount of information was found available in the literature to explain this observation, and further study is underway by the authors’ team on what governing parameter(s) could change the cracking pattern/mode in asphalt pavements at low temperatures.
CONCLUSION

Crack initiation and propagation in the IDT indirect tensile strength test under varying temperatures was investigated in this study using high-speed photogrammetry. From the results, two key observations can be made: (i) visible macro cracking in asphalt samples prior to the peak strength was confirmed which could be related to the transition of tensile to shear cracks along the grain boundaries, (ii) temperature variation not only influences the strength of asphalt pavement but also the dominant cracking/fracturing mode.

REFERENCES


Syagala, A, Bukowska, M, and Janoszek, T. 2014. High temperature vs geomechanical parameters of selected rocks – The present state of research


Zhou, F, and Newcomb, D. 2016. Experimental design for field validation of laboratory tests to assess cracking resistance of asphalt mixtures, in *National Cooperative Highway Research Program*

EFFECT OF SURFACE PROFILE ON AXIAL LOAD TRANSFER MECHANISM OF TENDONS

Ashkan Rastegarmanesh¹, Joel Misa², Ali Mirzaghorbanali³, Naj Aziz⁴ and Kevin McDougall³

ABSTRACT: Cable bolts as one of the most common means of ground support in surface and underground projects are attracting more attention due to their advantages over more expensive and cumbersome support elements. Thus, newer cables with new configurations and surface properties are emerging on a regular basis. Conventionally, the performance of cable bolts is studied in two main categories of axial and shear load transfer mechanisms. This paper focuses on the former by developing and proposing an experimental plan to cast light on the effect of surface profile on axial performance of cable bolts. Consequently, 15 representative surface profiles were designed and machined on metal base plates using a CNC machine. Later, these plates were used as moulds for casting cylindrical samples of Stratabinder cementitious grout. Three curing times of one day, 7 days, and 28 days were studied under static compression loading. It was observed that the surface roughness of the samples plays a major role in failure as it inhibits free movement of grout metal interface and inflicts end effect as well as introducing stress concentration point. These two drastically reduce the performance of grout and cause tensile crack growth.

INTRODUCTION

Tendons are extensively used in civil and mining projects across Australia to support underground structures (Bajwa et al., 2017). In highly stressed ground conditions, tendons become the dominant form of ground support. Tendons are a type of rock reinforcement used to stabilise the rock mass during tunnel excavations (Figure ). Tendons transfer the load of the unstable rock mass to the much stronger interior rock mass through the grout. This transfer occurs as a direct result of cohesion and friction between the bolts and the grout. The grout then transferred the loads to the surrounding rocks (Hutchinson et al., 1996).

Grouts are blended cementitious powder when mix with water would act as a high strength bonding agent. It is easily pumpable on the hole. The tendons are normally installed in a patterned spaced borehole to provide reinforcement and support for the wall and roof of an underground excavation floor (Figure 2).

The tendon-grout interface’s normal load behaviour and failure mechanism are of great significance for load transfer capacity and design of the rock bolting and cable bolting system (Zhang et al., 2020). Improvement of the tendons’ surface geometry significantly improves the tendons’ load transfer efficiency and anchorage capacity. This research is intended to investigate the effects of the tendon surface profile and curing time of grout on normal load transfer mechanisms of rock reinforcing elements.

The experiment will replicate the normal load transfer between the rock formation and tendon/grout. A circular metal mould with different surface asperity will be cast with grout. After curing time, the specimen will be subjected to a continuous load until its structural integrity is compromised.

LITERATURE REVIEW

The underground primary support system aims to maintain the underground excavation open and safe for their required designed life span. It is achieved through the correct design and installation procedures of internal and external support and reinforcement system. The term tendons are used here in a generic sense and cover cable bolts and rock bolts as reinforcing elements. Tendons are either primarily frictional or fully coupled devices. Tendons prevent separation and slip along planes of...
weakness in the rock mass. The design life of the tendons can range between 50 to 100 years. In Australia, the trend is to specify a 50 or 100 design life (Pells, et al., 1999).

Figure 1: Typical Bolts Details

Figure 2: Typical Stress/Load Imposed on Tendon

Tendons can either be mechanically or manually installed in rock mass and tensioned to ensure all of the components are maintained in contact with and that a positive force is applied to the rock mass. Where the bolts are required to carry a significant load, it is generally recommended that a tension of approximately 70% of the capacity of the bolt be installed initially (Hoek, 2006). The load is transferred to or from the rock mass along their entire length unless debonded sections occur due to design or installation error. This transfer occurs as a direct result of friction between the tendons and the encapsulating grout. The load is then transferred to the surrounding rock mass (Hutchinson et al., 1996).

Hagan, et al., in 2014, released a paper on the load transfer mechanism of fully grouted cable bolts under laboratory tests. The author outlines the developments and evolution in understanding the load transfer mechanism of fully grouted cables under axial loadings. Three basic mechanisms provide the shear resistance; chemical adhesion, mechanical interlock, and friction. However, the influence of chemical bonding is only temporary. Hence in most cases, the mechanical interlock and friction dominate the effect of load transfer. The mechanical interlock is the relative movement between the cable bolt and cement grout, while friction occurs between the cable/grout interface preventing the cable from slipping. Modified cable bolt designs, such as the bulb on the strand, give a much higher load transfer capacity compared to conventional cables. Grout plays an integral part in keeping the tendons from failure due to corrosive environments. The grout must act as protection against corrosion. Longevity and sensitivity are an important consideration when cable bolts are exposed to corrosive environments, blasting, and changes in local stress and confinement (Hutchinson et al., 1996).

Also, grout serves to transfer load between the rock and the tendons. The essential properties of the grout to allow it to carry out its functions are strength and stiffness. These properties are functions of water to cement ratio (W/C), grout composition, and elapsed time since placement. For Portland cement grout Hyett, et al., (1992) found a decrease in both 28-day uniaxial compressive strength and deformation modulus with increasing water/cement ratio. Their results show that the properties of grouts with water/cement ratios of 0.35 to 0.4 are significantly better than those with ratios above 0.5. Majoor, et al., (2017) has studied the effect of the water-grout ratio. In their studies, the shear strength and UCS decrease as the water-grout ratio increases. The shear strength decreases by 27%, while the UCS decreases by 43%.

University of Wollongong,  University of Southern Queensland,  February 2021  370
In 2016, Mirza, et al., conducted an extensive study the mechanical properties of two different grouts at various curing times using, the Jennmar Bottom-Up (BU100) and Orica Stratabinder HS. The experiment used a 50 mm cube mould with a mixing ratio of 5 and 7 litres/bag by weight of grout to water. All their samples failed in shear planes during compression tests. The named study was followed by Majoor, et al., in 2017 study of effect of the water to cement ratio on UCS and shear strength. They also performed triaxial testing to analyse the impact of confining pressures and obtain cohesion and internal friction angle values. The paper concluded that the water to grout ratio is a significant factor in both the uniaxial compressive strength and the shear strength of grout.

In 2018, Mirzaghorbanali et al., conducted an extensive experiment on Orica Stratabinder HS using two different sample moulds, 70 mm cube mould, and 100 mm diameter x 200 mm high cylindrical mould. The samples were cured for 1, 7, 14, and 21 days and tested for UCS. The authors found out that the cube sample's UCS is higher than the cylindrical sample and the curing time has a significant effect on the samples' strength.

The bonding strength between the grout and rock and the grout and steel also plays a major role because they constitute the system’s weakest points (Potvin et al., 1989). The surface profile of the tendons affects the stiffness of the bond strength between the tendon and grout. In Craig et al., (2012) study, the comparative laboratory testing determined that the nutcaged cables and indented wire cables provide stiffer and higher capacity bond strength than plain cable. The tendons' load transfer capacity is influenced by the shear strengths developed between the rock-grout and the grout-tendons interface interaction. In laboratory testing conducted by Aziz et al., (2006:2008), the grout-rock interface failure rarely occurs. The bonding strength between the bolt-grout interface dominates the effect of rock bolting.

Mechanical interlock is a critical component in load transfer capacity in rock bolting systems. The tendon profile configuration is the rib height, rib spacing, rib face angle, and rib cross-section shape. Cao et al., (2013) theorise that when an axial load is applied to cause the rock bolt's failure, the parallel shear failure, and dilational slip are the two major failure modes. Adding the Coulomb’s shear failure criterion and the rock bolt’s geometric profile to the equation, a universal upper limit of slipping angle can be calculated as the grout internal friction angle’s complementary angle a more accurate result is achieved.

In 2006, Aziz, et al., experimented on the bolt surface configurations and load transfer mechanisms. Their experiment involves a laboratory test and numerical analysis. Using two types of bolts and two types of encapsulating steel sleeves for the laboratory pull and push test and ANSYS3D for the numerical simulation. They have concluded that the resin/bolt surface’s load failure depends on the profile height and the spacing. The difference in grout material was studied by Pullan et al., (2018) and found both cement and resin grouts are both effective in load transfer. Their results indicated significant differences in the cable bolt’s performance between being grouted in strong and weak material and little difference in the stiffness between all test scenarios.

In Zhang, et al., (2020) study result, there is a correlation between the bolt profile and grout mixture on the shear behaviour and failure mode under the constant normal load conditions. The results demonstrated the bolt-grout interface’s shear behaviour is related to the profile, diameter, length of the rock bolts, and the grouting materials’ mechanical properties and the boundary conditions. Failure in cable/grout interface was also observed by Bajwa et al. (2017), experiment whereby the plain strand cable profile offers a minimum amount of friction between the grout and cable compared to a modified cable bolt. This paper tries to replicate the axial load transfer mechanisms between the grout and tendons surface asperity under the tangential stress imposed on grout in situ.

**METHODOLOGY**

**Specimen preparation**

The two primary materials used in this experiment are aluminium mould and Minova Stratabinder HS grout. Minova Stratabinder HS is a high strength, anchoring grout. It is a blended cementitious grout. Supplied ready to use, and when mixed with water, it is free-flowing. Aluminium was chosen because it can easily be fabricated. Three-dimensional (3D) models of the sample moulds were produced using AutoCAD software. The specimen models are
then fabricated with a Computer Numerical Control (CNC) machine. There are a total of 15 tendon moulds with different asperities.

The tendon moulds have a base of 10 mm thick and a diameter of 63.5 mm (Figure 3). Figure 4 and Table 1 illustrate the mould’s detail. It is worth mentioning that these patterns are not equivalent of the cables and bolts in the industry but rather were chosen as a representative of a smooth to an exaggerated rough surface condition. The other component of the specimen mould is a 65 mm diameter by 160 mm high PVC pipe. It forms the shape of the specimen. Figure 5 shows the set of PVC pipes used in the experiment. A heat gun shown in Figure 6 is used to widen the PVC pipe slightly so the tendon mould can be fitted inside the PVC pipe and form a tight fit to stop the seepage of the grout. The mixer shown in Figure 7 was to mix the grout mixture.

![Tendon Mould](image1)

**Figure 3: Tendon Mould**

![Tendon Mould Typical Cross Section](image2)

**Figure 4: Tendon Mould Typical Cross Section**

![PVC Pipe](image3) ![Heat Gun](image4) ![Mixer](image5)

**Figure 5: PVC Pipe  Figure 6: Heat Gun  Figure 7: Mixer**
Table 1 illustrates the moulds’ detail. It is worth mentioning that these patterns are not equivalent of the cables and bolts in the industry but rather were chosen as a representative of a smooth to an exaggerated rough surface condition.

Table 1: Surface Asperity

<table>
<thead>
<tr>
<th>Mould name</th>
<th>A (width spacing mm)</th>
<th>B (height mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flat surface</td>
<td></td>
</tr>
<tr>
<td>0 x 0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 x 2.5</td>
<td>6</td>
<td>2.5</td>
</tr>
<tr>
<td>6 x 5</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>6 x 7.5</td>
<td>6</td>
<td>7.5</td>
</tr>
<tr>
<td>10 x 2.5</td>
<td>10</td>
<td>2.5</td>
</tr>
<tr>
<td>10 x 5</td>
<td>10</td>
<td>5</td>
</tr>
<tr>
<td>10 x 7.5</td>
<td>10</td>
<td>7.5</td>
</tr>
<tr>
<td>10 x 10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>15 x 2.5</td>
<td>15</td>
<td>2.5</td>
</tr>
<tr>
<td>15 x 5</td>
<td>15</td>
<td>5</td>
</tr>
<tr>
<td>15 x 7.5</td>
<td>15</td>
<td>7.5</td>
</tr>
<tr>
<td>15 x 10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>20 x 2.5</td>
<td>20</td>
<td>2.5</td>
</tr>
<tr>
<td>20 x 5</td>
<td>20</td>
<td>5</td>
</tr>
<tr>
<td>20 x 7.5</td>
<td>20</td>
<td>7.5</td>
</tr>
</tbody>
</table>

Procedure in specimen preparation

1. The PVC pipe was cut to length, then heated using a heat gun to widen slightly, and then the tendon mould was inserted inside. The height to diameter ratio used is 2.5, according to ISRM recommendation.

2. A release agent (shuttering oil) was applied to the internal surface of the PVC pipe to facilitate the removal of the specimens; Making sure no oil is spilled on to the moulds. After applying oil on the pipe, the pipe is tip over, so the oil will not drip down the tendon mould as shown in Figure 8.

3. A grout mix is then prepared according to Minova’s technical datasheet. Below is the calculation of the required grout mixture for the experiment

Table 2 Grout mix design (divided by three for easier mixing)

<table>
<thead>
<tr>
<th>Water</th>
<th>Grout</th>
<th>Mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 L</td>
<td>20 kg</td>
<td>12.5 L</td>
</tr>
<tr>
<td>1.22 L</td>
<td>4.05 kg</td>
<td>2.53 L</td>
</tr>
</tbody>
</table>

\[
\text{Volume of grout mixture} = 15 \times \pi \times \frac{63.5^2}{4} \times 160 = 7.6 \times 10^6 \text{ mm}^3 \text{ or } 7.6 \text{ L}
\]

The breakdown of the 7.6 L grout mixture

\[
\text{Grout weight} = \frac{7.6 \times 20}{12.5} = 12.16 \text{ kg of grout}
\]

\[
\text{Litre of water} = \frac{7.6 \times 6}{12.5} = 3.65 \text{ L of water}
\]

For easy handling, the grout mixture was mixed in three batches (1.22L:4.05Kg)
1. The grout mixture was mixed, then poured into the specimen mould.

2. The grout mixture was tamped to make sure the grout mixture filled up the tendon mould’s crevices. The PVC side was tapped slightly to release the entrapped air, then the top surface of the specimen mould was flattened to create an even surface. An even surface would distribute the load evenly to the specimen.

3. Once all 15 specimens have been cast, they were placed in the curing room for proper hydration and strength development. The specimens were cured at 1-day, 7-day, and 28-days strength.

4. A hack saw was used to remove the specimen from the PVC mould. This way, the integrity of the tendon-grout binding was preserved. The bond between the tendon and grout is sensitive due to the nature of the aluminium.

5. The specimens were marked using a marking pen, then the tested.

6. After testing, the tendon moulds were cleaned and prepared for the next curing time. The experiment was repeated three times for different curing periods.

![specimen Mould Tip Over (marked)](image)

Figure 8: specimen Mould Tip Over (marked)

**Specimen testing**

The specimens were tested using the SANS Universal Testing Machine (UTM) to determine the Uniaxial Compressive Strength (UCS) of the grout confined by the tendon mould at the bottom. The specimen was centrally mounted on the testing loading plate, and on top to remove the end friction effect. The load rated was maintained at at 1 m/min with the load being uniformly loaded until failure. Figure 9 and Figure 10 show the SANS UTM set up.

![SANS machine](image)

![SANS data logger and control module](image)

Figure 9: SANS machine

Figure 10: SANS data logger and control module
RESULT ANALYSIS

Experimental observations

The following notes were observed:

1. The specimen breaks before the grout reaches its peak UCS (Figure 11, Figure 12 and Figure 13).
2. The crack opening in the top part is wider. Figure 14, Figure 15 and Figure 16 are the 6x7.5 specimen at 1-day, 7-day & 28-day strength. As shown by the specimen, some icicle shape fragments are formed. The photos above imply that there is constrained from the bottom.
3. The line of failure is in a vertical direction or tensile mode. No shear mode failure was observed. Figure 17, Figure 18 and Figure 19 show the 10x2.5 specimen at 1-day, 7-day, and 28-day strength. As demonstrated by the specimen, all failures are tensile. Once the load is applied, the top portion of the specimen is free to move (expand), but mould constrains the specimen's bottom part creating this type of failure.
4. The initial cracks are parallel to the groove of the mould. Figure 20, Figure 21 and Figure 22 show the 10x7.5 specimen at 1-day, 7-day, and 28-day strength. As demonstrated by the specimen above, the cracks are parallel or align with the tendon mould's crevices or one can assume they initiate from the tip of the ridges due to stress concentration.

The UCS of the specimens were computed using SANS UTM’s data output. The results are then tabulated and graphed. It was observed on the graph that the curved created was not a continuous curve path. There are intermittent drops in load applied as the specimen reaches its peak normal load or maximum UCS. Figure 23, Figure 24, Figure 25 and Figure 26 illustrate the results for 6 mm wide moulds in 1 day curing time period. The rest of the graphs all have the similar behaviour but with different failure load and displacement.

The peak normal load for specimen 10x2.5 in a 1-day strength test is 3.33 MPa, 8.95 MPa in a 7-day strength test, and 14.95 MPa in a 28-day strength test. The test result is below the strength in the technical data sheet of Minova Stratabinder HS of 50 MPa for 1-day, 78 MPa for 7-days, and 100 MPa for 28-days. The difference in results is attributed to, stress concentration and the end effect.

The poison effect of the normal stress on the top of the surface of the specimen causes the specimen to expand freely. Simultaneously, the mould restricted the same movement at the bottom part of the sample. The third factor was the stress concentration on the tip of the ridges on the aluminium moulds.
which provided a place of onset of cracks. These three factors lend hand to create the failure pattern observed in these experiments (Figure 28).

Figure 14: 6x7.5 specimen at 1-day strength
Figure 15: 6x7.5 specimen at 7-day strength
Figure 16: 6x7.5 specimen at 28-day strength

Figure 17: 10x2.5 specimen at 1-day strength
Figure 18: 10x2.5 specimen at 7-day strength
Figure 19: 10x2.5 specimen at 28-day strength

Figure 20: 10x7.5 specimen at 1-day strength
Figure 21: 10x7.5 specimen at 7-day strength
Figure 22: 10x7.5 specimen at 28-day strength

Figure 27 demonstrated the effect of the curing time on the specimen as the specimen is left to cure for a more extended period, the specimen peak normal load increases. The results indicated that the
Height of asperity has a consistent effect on the specimen’s strength compared to the asperity width. The higher the asperity, the higher the UCS of the specimen. The narrower the asperity width combined with lower asperity height, the lower specimen’s UCS. As demonstrated in Figure 29, on a 1-day test and 28-day test, the specimen’s UCS is higher when the height is 7.5mm. Reading the 7-day test result does not conform with earlier statements. To eliminate this phenomenon a more comprehensive experiment is required.

Figure 23: 6 mm wide 1 day results

Figure 24: 10 mm wide 1 day results

Figure 25: 15 mm wide 1 day results

Figure 26: 20 mm wide 1 day results

Figure 27: Test Results Maximum Stress of specimen
In terms of the asperity's width, the results are inconclusive as there is no definite pattern that has emerged from the graphs. Although the narrow 6 mm wide asperity has the lowest UCS in the 1-day and 28-day test, it is not the case in the 7-day test. The 15x5 has the highest UCS in the 7-day and 28-day test (Figure 30). It is expected that the 20x5 specimen would have the highest UCS. It is recommended to conduct a more comprehensive experiment to determine the effect of the height.
The secant modulus was computed using the first highest stress before the drop of stress and divide that by the strain. The results are tabulated and compared. In Figure 31, the effect of the curing time on the secant modulus is mixed. The experiment produced a contradictory result. Three of the specimens have conformed to the notion that the longer the curing time, the higher the secant modulus. The majority of the specimens have their highest secant modulus either in the 1-day test or 7-day test. The experiment result is inconclusive that the stiffness of the specimen increases as the curing time is longer.

The height and width of the surface asperity have an impact on the stiffness of the specimen. The calculated result produced is inconclusive. Figure 32 shows the comparison of the specimen with the same height of asperity with different width. The graph illustrated that on a 1-day test, the 15x2.5 specimen has the highest stiffness while on a 7-day test 6x2.5 specimen has the highest stiffness, but on a 28-day test, the 15x2.5 specimen has the highest stiffness. Further studies are recommended to determine the effect of the asperity on the stiffness of the tendon-grout specimen.
Figure 32: Wide specimen Comparison

Overall, the results produced conflicting results, especially with the 7-day test result producing some of the specimen's highest uniaxial compressive strength. Figure 33 summarises the test results across the three-day test. The 10x7.5 specimen in the 28-day test has the highest uniaxial compressive strength of 19.28 MPa, and the 6x2.5 specimen in the 28-day test has the lowest uniaxial compressive strength of 4.19 MPa. It was anticipated that the flat surface would have the highest uniaxial compressive strength, but the result did not reflect this.

CONCLUSIONS

The following conclusions are inferred from the study:

1. The tendon's normal load transfer mechanisms are influenced by the curing time and surface profile of the rock reinforcing elements. In general, the peak normal load increases with the increase in grout curing time. The uniaxial compressive strength of the specimen is higher at 28-day curing time. However, half of the results contradicted the expected outcome and suggested to carry out further studies.

2. The surface asperity also influences the stiffness and peak values of the normal load. In general, the peak normal load is lower as the height of asperity with the same width of asperity is lower. In the experiment, the 6x2.5 specimen has the lowest uniaxial compressive strength of 4.19 MPa, and the 10x7.5 specimen has the highest uniaxial compressive strength of 19.28 MPa. The study is inconclusive on the specific width and height of asperity. In theory, the
wider the asperity, the less restriction the grout to move, and the experiment did not prove this theory. In the comparison, the smaller the gap and lower the asperity’s height, the lower the uniaxial compressive strength based on the 28-day curing time. Overall, the effect of the asperity on the grout’s UCS is significant.

3. Comparing the results obtained to Mirzaghorbanali’s 2017 work on cube mould and cylindrical mould, the maximum UCS obtained is 442% below the cube mould UCS and 373% below the cylindrical mould. It is noted in the named study experiment is an unconfined test whereby both ends of the samples are free to move or expand. In contrast, this experiment has one end confined by the tendon’s mould asperity. That restricts the bottom end from expanding or moving, also causing a tensile plane mode of failure.

4. The stiffness of the tendon-grout specimen is inconclusive in this experiment. The surface asperity has a definite impact on the stiffness of the specimen. Still, it is inconclusive to determine which portion of the asperity impacts whether the height or width has the most impact. The curing time also affects the stiffness, although the experiment did not produce the desired outcome. Some specimens demonstrated the desired effect of the curing time, such as 0x0, 10x5, 10x7.5. The longer the specimen is cured, the stiffer it gets.

5. Aluminium does not reflect the tendon’s material. Furthermore, it proved to be more susceptible to losing its bond to the grout prior to the tests. Lastly, due to soft nature of aluminium, cleaning of the moulds is not easy. Yet, a CNC machine takes a considerately more time to machine a steel mould which may make the aluminium compromise worth it,

6. Lubricating the PVC pipes may cause the oil to be washed down by the grout to the metal mould in the bottom. This will cause a thin layer of oil to be form between the contact of mould and grout which will weaken the bond. Once the oil is applied in the inside face of the PVC pipe, it is highly recommended to tip over the specimen’s mould. Also, the amount of the oil should be extremely limited or perhaps completely removed from the experiment.

7. Due to the small size of the samples the usual grout mixing precautions should be applied. For instance, the samples are significantly sensitive to the condition of the grout (fresh bags or badly sealed bags), water content in the mix, proper even mixing, tapping and disturbing the sample for air content reduction and grout penetration into the crevices, avoiding disturbance to the samples specially in the first 10 hours of casting, etc.

8. It is recommended to do a test without the top cap to inflict a top-end effect and compare the previous study results. In the field, both the top and bottom part of a grout element are constrained; one by the bond to the rock and the other by a bond to the cable. This will call to a experiment where end effect is inflicted on both sides of the sample, contrary to this study where the top surface was free for lateral expansion. Also, this can be even further extended by providing confinement to the sample.

ACKNOWLEDGEMENT

Authors’ would like to acknowledge the ongoing in-kind support of MINOVA, in particular Mr Robert Hawker, for this research study.

REFERENCES


MINING EQUIPMENT HUMAN FACTORS DESIGN FOR WORKFORCE DIVERSITY

Danellie Lynas¹, Gary Dennis² and Robin Burgess-Limerick³

ABSTRACT: The minerals industry is a complex system in which procedures, equipment and people need to interact safely and efficiently to achieve operational requirements. The sector is also an attractive employment option, creating an environment of significant workforce diversity within equipment operators and maintainers. While a variety of standards and guidance materials exist to assist designers to provide equipment that accommodates workplace diversity, designers face significant challenges in applying this information, and may unnecessarily restrict the range of potential employees who can operate and maintain this equipment, and in turn create elevated injury risk. Understanding how equipment design impacts safe and comfortable operation and maintenance will provide additional assistance to designers. Information gathered at seven Australian surface coal mines was used to undertake a comprehensive review of the limitations of current equipment designs with regard to accommodating diversity in physical characteristics required to perform operational and maintenance tasks on site. A control framework approach to equipment design for diversity was developed where two required operating states were defined, (i) earth-moving equipment can be safely and comfortably operated by people of a maximum range of anthropometric diversity; and (ii) earth-moving equipment can be safely and comfortably maintained by people of a maximum range of anthropometric diversity. The general business case for increasing workforce diversity in mining is well established. Improving earth-moving equipment design is required to remove significant anthropometric and other work demand impediments to supporting this diversity. This paper highlights the inadequacy of currently available guidance material to equip designers to understand and address these challenges.

INTRODUCTION

Workforce diversity is often assumed to mean gender distribution however, a range of additional worker attributes is encompassed, including physical and cognitive abilities, cultural background, language and communication styles. While the minerals industry is seen as an attractive employment option by many, within the industry a number of challenges arise including a diversity of company cultures reflected in different procedures, rules and practices at mines; a variety of national laws, regulations and guidelines; many different equipment manufacturers and suppliers; differences in the mining environment, and significantly, the diversity within the workforce employed across mine sites.

A project funded by the Australian Coal Association Research Program (ACARP) was undertaken to identify and describe design issues with current mining equipment which created a barrier to diversity within the workforce and to communicate the results of these investigations to equipment designers and mine sites. The information gained during the project was used to populate an Earth Moving Equipment Safety Round Table (EMESRT) control framework for equipment design for diversity. EMESRT is a global initiative involving major mining companies which engages with key mining industry Original Equipment Manufacturers (OEMs) to advance the design of equipment to improve safety operability and maintainability beyond standards (www.emesrt.org).

Seven surface coal mines in Queensland and NSW were visited during the project. Utilising a participative ergonomics approach equipment operators and maintainers were actively involved in on-site focus groups. Information was gathered to demonstrate the anthropometric issues arising from equipment designed for the mining sector, and where the design of this equipment may place unnecessary employment restrictions, and potentially create elevated risks of injury for those who

1 Research Fellow, Sustainable Minerals Institute, University of Queensland. Email: d.lynas@mishc.uq.edu.au
Tel: +61 417 791 138

2 Industry Fellow, Sustainable Minerals Institute, University of Queensland. Email: g.dennis@ergoenterprises.com.au
Tel: +61 7 5668 3422

3 Professor of Human Factors, Sustainable Minerals Industry, University of Queensland. Email: r.burgess-limerick@uq.edu.au
Tel: +61 401 714 511
currently undertake tasks associated with operating and maintaining the equipment. This information
was supplemented with previously documented assessments of manual tasks associated with earth-
moving equipment maintenance, including attempts to reduce manual tasks risks undertaken by a
range of mine sites. Two required operating states were defined: (i) earth moving equipment can be
safely and comfortably operated by people of a maximum range of anthropometric diversity, and (ii)
earth moving equipment can be safely and comfortably maintained by people of a maximum range of
anthropometric diversity. Twelve credible failure modes were identified for equipment operation and
two credible failure modes were identified for maintenance tasks.

CHALLENGES POSED BY EXISTING DESIGN GUIDELINES

A variety of standards and guidance materials currently exist to assist equipment designers
accommodate workplace diversity. This material may include guidance on visibility; noise
measurements; whole body vibration assessments; ergonomics and human factors; controls and
displays; manual tasks risk analyses; and audits against relevant standards and Mining Design
Guidelines (MDGs). However, designers of equipment for mining operations face significant
challenges in applying this information. It is therefore important to understand how the design of
mining equipment restricts the range of potential employees who can safely and comfortably operate
and maintain the equipment to provide additional assistance to equipment designers.

The main reference material is found within ISO standards, however, the ISO standards process is
formal and change often lags behind technology development. In some cases this may result in weak
or ineffective standards that need to be written in general terms to accommodate future technology
developments. In this way the general nature of some standards may only establish minimum
requirements and not be overly helpful to equipment designers, in particular when designing for
diversity and inclusivity within the workforce. In contrast, Mining Design Guidelines (MDGs) as
produced by the New South Wales regulator are able to respond more quickly to changes in
technology. Standards do however have a safety and productivity focus, and therefore provide design
consistency between equipment manufacturers. Standards also provide a basis for auditing purposes,
and assist in mining regulation, compliance, and the development and application of safety
management systems. For example, ISO12100 (2010) “Safety of machinery - General principles for
design - Risk assessment and risk reduction”, provides a general framework approach to equipment
design, including basic terminology, principles and a method for achieving safety in the design of
machinery. More specifically, ISO 9241-210 (2010) proposes a human-centred design approach to
design that has substantial economic and social benefits for users, employers and suppliers. In
summary, it provides a set of principles based on explicit understanding of users, tasks and
environments; users are involved throughout design and development; the design is driven and refined
by user-centered evaluation; the process is iterative; the design addresses the whole user experience;
and the design team includes multidisciplinary skills and perspectives (Horberry, et al 2018).

ISO3411 “Earth-moving machinery - Physical dimensions of operators and minimum operator space
envelope” provides data approximating the 5th, 50th and 95th percentile of the “earth-moving
machinery operator population”, for numerous static dimensions relevant to the design of earthmoving
equipment (eg. Figure 1) and these data are utilised in ISO6682 to provide recommendations
regarding the location of controls (eg. Figure 2). There are considerable limitations to the use of such
data. The standard notes for example, that: “In some areas of the world, more the 5% of the operators
have leg lengths less than the value given for the smallest operators” and suggests that “special
adjustments may be provided”, without specifying the nature to these adjustments. This note highlights
an example of one area in which current equipment designs limit the diversity of the potential
workforce by providing insufficient seat adjustment to accommodate short leg lengths. It is noteworthy
that the limitation will disproportionately affect potential female employees.

Attempting to utilise the data provided in ISO3411 (or other sources of static anthropometric data)
creates further problems for equipment designers, in that the reference percentiles offered is
problematic for design purposes. There is no “5th percentile operator” or “95th percentile operator”.
Rather, individuals vary along each dimension (Robinette and Hudson, 2006) and although
dimensions have some degrees of correlation, when multiple dimensions are considered, the range of
individuals which fall within a given range on all dimensions reduces substantially. For example, only
about 82% of individuals in a population will fall within the 5th to 95th percentile ranges for both height
and weight. The more dimensions are considered, the smaller the range of people who actually “fit”
the description. Looking at this issue in another way, Figure 3 provides a representation taken from whole body scans of two people with the same sitting height; and also illustrates a hypothetical person generated case using all 95th% male dimensions.

Figure 1: Example of static anthropometric data currently available with ISO3411 “Earth-moving machinery - Physical dimensions of operators and minimum operator space envelope”.

Figure 2: ISO6682: Earth-moving machinery - Zones of comfort and reach for controls
BS 6912-19:1996, ISO 11112:1995 (part 19) provides specifications for dimensions and requirements for operator’s seat. This International Standard specifies the dimensions, requirements and adjustment ranges for operator seats on earth-moving machinery as defined in ISO 6165. Additionally, it provides dimensions for armrests when fitted on these machines. Again, whilst providing nominal values of dimensions regarding seat features, their mutual locations and adjustments are established on the basis of ergonomic requirements. Taking into consideration operator sizes it is referenced according to ISO 3411, and considers from the 5th percentile through the 95th percentile. Seat dimensions and adjustments, if provided, are referenced to the Seat Index Point (SIP) determined in accordance with ISO 5353. Confusion arises as the standard advises “dimensions and adjustments other than those specified in this International Standard may be used only if they provide better accommodation for the operator”.

Figure 3: (left) Two individuals with the same sitting height, and (right) a hypothetical 95% male.

Seat design, condition and adjustment is known to influence operator whole-body vibration exposure levels (Lewis and Johnson 2012; Paddan and Griffin, 2002), meaning seat installation needs to suit both the vehicle and its operating environment. Performance differences across seats have shown significant health implications for drivers and equipment operators (Blood et al, 2010). Seating remains a design problem, with “standard fitment” often stipulating mass range from 80-150 kg, and limited fore-aft adjustability, which clearly provides a restriction on the diversity of operators who could be employed. This highlights another example of an area in which current equipment designs limit the diversity of the potential workforce by providing insufficient seat adjustment to accommodate in particular smaller operators. It is noteworthy that these limitations disproportionately effect potential female employees, typically those smaller in stature. The potential implication of providing a seat that is not matched to operator mass was illustrated in ACARP project 26016 (Continuous Monitoring of Whole-body Vibration Jolts and Jars Associated with Operating Earth Moving Equipment at Surface Coal Mines). During the project data was collected from a haul-truck being driven at a central Queensland surface coal mine where accelerometers placed in the seat of the truck, and on the floor under the seat. The data collected at the operator-seat interface was frequency weighted according to ISO2631.1 to provide an assessment of the whole-body vibration exposure of the driver relative to the Health Guidance Caution Zone (HGCZ) provide by the standard. Comparing the magnitude of the accelerations collected on the floor under the seat with these data provides an assessment of the effectiveness of the seat in attenuating biologically relevant accelerations. A 2 hour 13 minute measurement yielded a ratio of seat to floor acceleration amplitude of 0.63 when the data are expressed as RMS, and 0.67 expressed as VDV (a measure more sensitive to high amplitude shocks) which indicates that the seat is effectively attenuating the relevant vibration frequencies and as a consequence the vertical whole-body vibration exposure assessment of 0.45 m.s² RMS lies below the ISO2631.1 HGCZ for an 8 hour daily exposure, and just within the HGCZ when expressed as VDV. Another 2 hour 26 minute measurement taken from the same truck driving the same circuit earlier on the same day indicates that during this period the seat has provided less effective attenuation. When the accelerations are expressed as VDV the seat appears to be amplifying the floor accelerations. The difference between the two measurement periods is the driver. It is likely in the second example that the driver was relatively light and either did not, or could not, adjust the suspension of the seat to match their mass. The resulting whole-body vibration levels are likely to be associated with detrimental health effects across multiple body systems if exposure is prolonged.
ISO 5006: “Earth moving machinery-operators field of view” provides a test method and performance criteria to address the operator’s visibility in such a manner that the operator can see around the machine to enable proper, effective and safe operation that can be quantified in objective engineering terms. While the test method uses two lights placed at the location of the operator’s eyes, it does not include all aspects of the operator’s visibility. However, it does provide information to assist in determining the acceptability of visibility from the machine.

Mine Design Guideline MDG15 (2002) “Mobile and transportable plant for use at mines and petroleum sites “ was developed by the NSW regulator with the aim of improving an unacceptable rate of injury to people operating and maintaining mobile plant; fires on mobile plant; and unplanned movement of mobile plant. Whilst not a mandatory compliance document, it includes advice from a number of A/NZ and ISO Standards to inform safe mining equipment design features such as provision of safe access and egress via ladders and stairs, walkways and handrails; and location of controls within the zones of comfort and reach of intended users. It does not provide specific design information regarding diversity within the population of equipment operators and maintainers. It does however, in a very general statement, indicate a person competent in ergonomics should provide an assessment of the equipment, which should take into consideration the intended use of the equipment and the operating environment, and consider “all relevant ergonomic matters relating to human factors”.

Additionally, whilst not specific to the design of mining equipment, ANSI Z590.3 (2011) “Prevention through Design Guidelines for Addressing Occupational Hazards and Risks in Design and Redesign Process” provides guidance on the avoidance, elimination, reduction or control of occupational safety and health hazards and risks in design and redesign processes. Overall, these standards and guidelines have limited utility for designers in that they can only consider a limited range of typical tasks, such as those undertaken while sitting in a driver’s seat, rather than the complete range of tasks associated with the operation and maintenance of equipment. As well as differing in static dimensions, potential employees differ in terms of dynamic capabilities such as strength, flexibility and reach distances. Taking all task components into account requires a task-based assessment to be undertaken during the design process, such as that provided by the Earth Moving Equipment Safety RoundTable (EMESRT) Design Evaluation for Equipment Procurement (EDEEP) process (Burgess-Limerick, et al., 2012). The most common manifestation of the failure to accommodate diversity is the design of equipment such that hazardous tasks are required to operate and maintain the equipment.

A CONTROL FRAMEWORK APPROACH TO EQUIPMENT DESIGN FOR DIVERSITY

Six central Queensland surface coal mines (two different mining companies) and one New South Wales surface coal mine participated in the study. The focus groups identified equipment, tasks and situations in which the range of potential operator or maintainer characteristics were not accommodated, including both situations which had been resolved by design changes made onsite, situations which were currently managed through administrative controls, as well as any situations for which no solution had been identified. Extensive time was spent in the workshop and in-field with equipment maintainers who identified a number of tools and platforms purpose built to assist with maintenance tasks. Both equipment operators and maintainers identified similar issues across sites and across companies. Working within individual site requirements, video footage and still images were captured to illustrate the design issues identified during the focus groups. Manual task analysis was undertaken using Ergoanalyst an injury management software system that utilities participative ergonomics to identify and assess manual task risks and potential control implementation to reduce risk and maximise productivity.

The information gathered during the project was used to gain an improved understanding of the limitations of current equipment designs with respect to accommodating diversity in operator and maintainer physical characteristics (static anthropometric variability); and with equipment operation and maintenance tasks which require combinations of high exertion, awkward or static postures, repeated similar movements and long duration which do not accommodate potential variability in operator and maintainer strength, flexibility and reach distances (dynamic anthropometry). Additional information was sourced from previously documented assessments of manual tasks associated with earth-moving equipment maintenance, including attempts to reduce manual tasks risks undertaken by a range of mine sites. The information obtained was used to construct and populate an EMERST Control Framework for equipment design for diversity.
Two required operating states were defined:

1. Earth-moving equipment can be safely and comfortably operated by people of a maximum range of anthropometric diversity.

2. Earth-moving equipment can be safely and comfortably maintained by people of a maximum range of anthropometric diversity.

A range of credible failure modes were identified including:

1. Small operators have difficulty reaching isolation points, fire suppression and emergency stop.
2. Small operators find access systems initial step height uncomfortable.
3. Height and weight of refuelling hose and attachments makes refuelling difficult.
4. Location of displays requires excessive neck extension and/or shoulder extension.
5. Controls difficult or uncomfortable to operate for smaller operators.
7. Seat suspension cannot be adjusted sufficiently to suit the mass of small operators.
8. Seat height cannot be adjusted to suit leg length of small operators.
9. Routine maintenance or inspection tasks performed by operators require excessive reach.
10. Seat belt height cannot be adjusted to be comfortable for small operators.
11. Mirrors do not provide the field of view required by small operators.
12. Truck handrail impedes vision for smaller truck drivers.

**DISCUSSION AND CONCLUSION**

The consistency observed across focus groups and workshop observations undertaken confirms the concerns regarding the current design of mining equipment which prompted the initial ACARP project were justified in that aspects of earth-moving equipment designs may unnecessarily restrict the range of potential employees who can operate and maintain the equipment, and in turn create elevated risks of injury for those who undertake tasks associated with operating and maintaining the equipment. The observations also confirm the concerns are not limited to one particular mine operator, mine site or original equipment manufacturer.

During the focus group discussions most operational concerns were raised by female members of the workforce, with concerns related to anthropometric issues associated with seating; visibility whilst operating haul trucks; inability to reach isolation points, and procedures regarding in-pit refuelling. A number of female operators commented they felt most mining equipment was “designed for a 6 foot male”, however shorter stature male operators reported somewhat similar operating concerns. Smaller stature female operators reported difficulty with in-cab seat adjustments that suited their weight; provided comfortable foot pedal, dashboard control and switch reach; and allowed adequate operational visibility. Female operators reported shoulder, neck and chest discomfort from the seatbelt sash component. A female operator (55 kg / 157 cm) reported habitually operating a wheel dozer with the seat air cushion mechanism completely deflated. In this position she reported her foot often slipped off the pedals, however this set up was her preferred operating positions she believed removing as much air as possible from the seat provided a less jarring ride. To the contrary, removing the air completely negated seat attenuation to reduce exposure to excessive whole-body vibration levels, placing her at significant risk of musculoskeletal injury and other associated tissue damage associated with excessive exposures.

Maintenance tasks associated with heavy earth moving equipment are a frequent and significant source of exposure to musculoskeletal injury risks. In general across mine sites, the majority of the maintainer workforce is male dominated, however, an increasing number of female maintainers both as apprentices and fully qualified maintenance personnel are joining the workforce. Typically, design inadequacies associated with maintenance tasks related to poor access, inadequate provision of lifting points, inappropriate tooling, and the need for excessive manual forces to undertaken and complete maintenance tasks. Female maintainers face additional challenges in undertaking routine maintenance tasks, with a female apprentice approximately 3-4 months into her apprenticeship commenting “she wasn’t yet strong enough but would get stronger”. Comment was made that the torque of many bolts...
required excessive exertion to undo them, with it often not possible to get a rattle gun into a confined working space to assist with the task. On observation, awkward postures were often needed to complete tasks such as accessing oil service points which are generally located higher up on equipment. Consistent problems identified by male maintainers included narrow flame rails (e.g. on 709C trucks) which caught their tool belts. Absence of lifting points to assist hose change outs on haul trucks, lack of appropriately sized working platforms and lack of specific tools were also highlighted as significant impediments to safe task completion. Maintainers frequently commented "everything is big and heavy, we need proper lifting gear and tools which we don’t have". Maintainers also considered adoption of regular maintenance scheduling rather than waiting until damaged components such as hoses needed replacement would reduce the frequency of working in awkward postures.

Recommendations arising from this research included the following: (i) EMESRT promotion of earth-moving equipment design improvements to reduce barriers to workplace diversity through communication of the findings of this project to Original Equipment Manufacturers, and standards committees. The equipment design limitations identified in this research frequently led to the performance of manual tasks associated with equipment operation, and especially maintenance, that involve high exertion and/or awkward postures. Frequent or prolonged performance of such tasks increases the risk of musculoskeletal disorders. A combination of task redesign and administrative controls should be employed to reduce the identified risks. Harnessing the expertise of the workers who undertake the tasks through a participatory ergonomics process has potential to both ensure that the solutions proposed are optimal, and will be accepted by workers. Training in ergonomics principles, team work and problem solving is likely to be required; as well as the provision of tools for the efficient analysis of manual task risks and for the development and documentation of proposed and implemented changes. If this can be achieved, the evidence is that such a program will reduce injury risks (Burgess-Limerick, 2018) and such approaches are recommended by resource industry regulators. It is also recommended that:(ii) mine operator implement participatory ergonomics programs that assess hazardous manual tasks associated with equipment operation and maintenance, including a combination of design and administrative controls to reduce risks as far as reasonable practical.

Many of the challenges documented during the focus groups and workshop observations may have been arisen because original equipment designers and manufacturers do not see or understand the conditions under which maintenance tasks in particular are performed, however, and more importantly the standards and guidance material available to designers does not adequately equip them to understand how to address these challenges. While the general business case for increasing workforce diversity in mining is well established, and improving earth-moving equipment design can remove significant anthropometric and other work demand impediments to establishing a more diverse mining workforce, it is clear both practical on-the-ground improvement of current operational practice and improvements in equipment design is required, particularly for maintenance tasks.

ACKNOWLEDGEMENTS

This research was an initiative of the Earth Moving Equipment Safety Round Table (EMESRT) a collaboration of mining companies with a long history of influencing the design of mining equipment through providing Original Equipment Manufacturers with a consolidated view of the experiences of mining companies. Funding was received from The Australian Coal Association Research Program (ACARP) for this project (C2804).

REFERENCES


# INDEX TO AUTHORS

<table>
<thead>
<tr>
<th>Authors</th>
<th>Page Numbers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alehossein, Habib</td>
<td>91, 317</td>
</tr>
<tr>
<td>Alston, Neil</td>
<td>261</td>
</tr>
<tr>
<td>Anzanpour, Sina</td>
<td>210, 239</td>
</tr>
<tr>
<td>Ardehali, Sahar</td>
<td>317</td>
</tr>
<tr>
<td>Ataei, Mohammad</td>
<td>345</td>
</tr>
<tr>
<td>Aziz, Naj</td>
<td>210, 218, 229, 239, 294, 303, 317, 345, 369</td>
</tr>
<tr>
<td>Bajic, Snezana</td>
<td>177</td>
</tr>
<tr>
<td>Balusu, Rao</td>
<td>115</td>
</tr>
<tr>
<td>Belle, Bharath</td>
<td>115, 124</td>
</tr>
<tr>
<td>Bernal, John</td>
<td>6</td>
</tr>
<tr>
<td>Bofinger, Carmel</td>
<td>251</td>
</tr>
<tr>
<td>Bowes, Robert</td>
<td>317</td>
</tr>
<tr>
<td>Burgess-Limerick, Robin</td>
<td>383</td>
</tr>
<tr>
<td>Canbulat, Ismet</td>
<td>26</td>
</tr>
<tr>
<td>Chang, Ping</td>
<td>261</td>
</tr>
<tr>
<td>Chen, Zhongwei</td>
<td>42, 285, 353</td>
</tr>
<tr>
<td>Cliff, David</td>
<td>251</td>
</tr>
<tr>
<td>Colwell, Mark</td>
<td>138</td>
</tr>
<tr>
<td>Craig, Peter</td>
<td>194</td>
</tr>
<tr>
<td>Dennis, Gary</td>
<td>383</td>
</tr>
<tr>
<td>Duan, Joey</td>
<td>17</td>
</tr>
<tr>
<td>Dzakpata, Isaac</td>
<td>270</td>
</tr>
<tr>
<td>Emery, Jason</td>
<td>26</td>
</tr>
<tr>
<td>Frith, Russell</td>
<td>78, 138</td>
</tr>
<tr>
<td>Ghosh, Apurna</td>
<td>261</td>
</tr>
<tr>
<td>Gido, Miaden</td>
<td>177</td>
</tr>
<tr>
<td>Gong, Libin</td>
<td>186, 325</td>
</tr>
<tr>
<td>Holt, Peter</td>
<td>63</td>
</tr>
<tr>
<td>Howcroft, Bill</td>
<td>167</td>
</tr>
<tr>
<td>Johnstone, Kelly</td>
<td>251</td>
</tr>
<tr>
<td>Kalubadangaje, Dulara</td>
<td>63</td>
</tr>
<tr>
<td>Khaleghparast, Saman</td>
<td>210, 218</td>
</tr>
<tr>
<td>Khaleghparast, Saman</td>
<td>239</td>
</tr>
<tr>
<td>Kizil, Mehmet</td>
<td>270</td>
</tr>
<tr>
<td>Kizil, Mehmet Siddik</td>
<td>42</td>
</tr>
<tr>
<td>Klenowski, George</td>
<td>6</td>
</tr>
<tr>
<td>Kukutsch, Radovan</td>
<td>325</td>
</tr>
<tr>
<td>Kurukulasuriya, Devmi</td>
<td>167</td>
</tr>
<tr>
<td>LaBranche, Nikky</td>
<td>251</td>
</tr>
<tr>
<td>Lee, Chris</td>
<td>50</td>
</tr>
<tr>
<td>Li, Xu</td>
<td>229</td>
</tr>
<tr>
<td>Luo, Xun</td>
<td>17</td>
</tr>
<tr>
<td>Lynas, Danellie</td>
<td>383</td>
</tr>
<tr>
<td>Macfarlane, Ian</td>
<td>1</td>
</tr>
<tr>
<td>MacGregor, Stuart</td>
<td>50</td>
</tr>
<tr>
<td>Name</td>
<td>Page Numbers</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>--------------</td>
</tr>
<tr>
<td>Mahmoud, Mutaz El-Amin</td>
<td>311</td>
</tr>
<tr>
<td>Makusha, Gift</td>
<td></td>
</tr>
<tr>
<td>Marshall, Travis</td>
<td>210</td>
</tr>
<tr>
<td>McDougall, Kevin</td>
<td>203</td>
</tr>
<tr>
<td>Meredith, Karina</td>
<td>115</td>
</tr>
<tr>
<td>Mills, Ken</td>
<td>167</td>
</tr>
<tr>
<td>Mirzaghorbanali, Ali</td>
<td>239, 303, 369</td>
</tr>
<tr>
<td>Misa, Joel</td>
<td>101</td>
</tr>
<tr>
<td>Mohammadi, Sadjad</td>
<td>345</td>
</tr>
<tr>
<td>Mohmoud, Mutaz El-Amin</td>
<td>285</td>
</tr>
<tr>
<td>Moon, Ellen</td>
<td>167</td>
</tr>
<tr>
<td>Morton, Claire</td>
<td>42, 71</td>
</tr>
<tr>
<td>Mousavi, Amin</td>
<td>270</td>
</tr>
<tr>
<td>Muller, Sean</td>
<td>177</td>
</tr>
<tr>
<td>Nemcik, Jan</td>
<td>186, 210, 239</td>
</tr>
<tr>
<td>Nourizadeh, Hadi</td>
<td>294, 303</td>
</tr>
<tr>
<td>Oh, Joung</td>
<td>229, 239</td>
</tr>
<tr>
<td>Owen, Nathan</td>
<td>71</td>
</tr>
<tr>
<td>Pattinson, Dinghy</td>
<td>50</td>
</tr>
<tr>
<td>Purser, Robert</td>
<td>285</td>
</tr>
<tr>
<td>Qin, Zongyi</td>
<td>91</td>
</tr>
<tr>
<td>Ram, Sahendra</td>
<td>325</td>
</tr>
<tr>
<td>Rastegarmanesh, Ashkan</td>
<td>239, 369</td>
</tr>
<tr>
<td>Remennikov, Alex</td>
<td>63, 218</td>
</tr>
<tr>
<td>Remmenikov, Alex</td>
<td>239</td>
</tr>
<tr>
<td>Ren, Ting</td>
<td>63</td>
</tr>
<tr>
<td>Robbins, Matt</td>
<td>317</td>
</tr>
<tr>
<td>Roshan, Hamid</td>
<td>311</td>
</tr>
<tr>
<td>Saadat, Mahdi</td>
<td>203</td>
</tr>
<tr>
<td>Sasi, Sabitha</td>
<td>194</td>
</tr>
<tr>
<td>Serati, Mehdi</td>
<td>285, 294, 303</td>
</tr>
<tr>
<td>Shan, Zhenjun</td>
<td>186</td>
</tr>
<tr>
<td>Shen, Baotang</td>
<td>17, 91</td>
</tr>
<tr>
<td>Si, Guangyao</td>
<td>229, 239</td>
</tr>
<tr>
<td>Soucek, Kamil</td>
<td>325</td>
</tr>
<tr>
<td>Spáth, Arend</td>
<td>124</td>
</tr>
<tr>
<td>Tadic, Dihon</td>
<td>270</td>
</tr>
<tr>
<td>Taheri, Abbas</td>
<td>203</td>
</tr>
<tr>
<td>Tanguturi, Krishna</td>
<td>115</td>
</tr>
<tr>
<td>Timms, Wendy</td>
<td>167</td>
</tr>
<tr>
<td>Tsang, Matt</td>
<td>353</td>
</tr>
<tr>
<td>Valluru, Thejaswee</td>
<td>311, 362</td>
</tr>
<tr>
<td>Vavro, Martin</td>
<td>325</td>
</tr>
<tr>
<td>Venticinque, Gaetano</td>
<td>186</td>
</tr>
<tr>
<td>Waclawik, Petr</td>
<td>325</td>
</tr>
<tr>
<td>Wallace, Jordan</td>
<td>210</td>
</tr>
<tr>
<td>Watson, Samantha</td>
<td>71</td>
</tr>
<tr>
<td>Wij, Ian Van</td>
<td>362</td>
</tr>
<tr>
<td>Williams, Sally</td>
<td>294</td>
</tr>
<tr>
<td>Wilson, Stephen</td>
<td>101</td>
</tr>
</tbody>
</table>

University of Wollongong, University of Southern Queensland, February 2021
<table>
<thead>
<tr>
<th>Author</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Xiang, Zizhuo</td>
<td>229</td>
</tr>
<tr>
<td>Yang, Ben</td>
<td>353</td>
</tr>
<tr>
<td>Yang, Shu-Qing</td>
<td>158</td>
</tr>
<tr>
<td>Yang, Xiaohan</td>
<td>63</td>
</tr>
<tr>
<td>Zhang, Chengguo</td>
<td>26</td>
</tr>
<tr>
<td>Zhao, Zidong</td>
<td>261</td>
</tr>
<tr>
<td>Zhong, Ruizhi</td>
<td>353</td>
</tr>
</tbody>
</table>