“Rock Engineering in Difficult Conditions” is synonymous of almost any engineering activity involving rocks and rock masses.
Welcome Message

It gives us great pleasure to welcome you to RockEng09 — 3rd Canada-US Rock Mechanics Symposium & 20th Canadian Rock Mechanics Symposium. First held in 1962, the Canadian Rock Mechanics Symposium series was re-launched with the 1st Canada-U.S. Rock Mechanics Symposium, held in Vancouver in 2007. The Vancouver meeting, with over 200 papers and attracted an international audience of 450 delegates, has made significant contributions towards raising awareness on the importance of rock mechanics and rock engineering for the mining industry worldwide. RockEng09 will try to improve the already high quality standards and be the perfect setting to generate and communicate solutions to today’s rock engineering problems, with particular emphasis to the Canadian mining industry.

RockEng09 brings together nationally and internationally recognized experts from industry, government and academia to discuss the advances made in rock mechanics and rock engineering for mining, civil and petroleum applications. The RockEng09 runs over 2 ½ days, and it is complemented by six workshops and short courses offered prior to and after the conference. Those workshops and short courses, tailored for a broad and diverse audience, target specific problems in rock engineering and provide an additional learning opportunity.

RockEng09 is organized as a unique, immersive learning experience. The symposium will expose graduate and undergraduate students, young and senior industry engineers, academics and government representatives to the latest rock engineering challenges and solutions. We encourage you to share in the symposium deliberations and hope that you will take advantage of the numerous opportunities to discuss the various aspects of rock engineering with your peers.

We would like to thank the keynote speakers and authors for their exceptional contributions, and express our gratitude to CIM National Office staff for their hard work in preparing the symposium, and our appreciation to our sponsoring organizations.

I hope you will find RockEng09 meaningful and rewarding, and also find time to take in the beautiful sights and enjoy your stay in Toronto.

Giovanni Grasselli
Chair, RockEng09

Mark Diederichs
Co-Chair, RockEng09
Organizing Committee

Giovanni Grasselli, Chair
University of Toronto

Mark Diedrichs, Co-chair
Queen's University

Murray Grabinsky
University of Toronto

Luc Beauchamp
MASHA

Ming Cai
MIRARCO, Laurentian University

Erik Eberhardt
University of British Columbia

Joe Carvalho
Golder Associates Ltd.

Erik Westman
Virginia Tech

Denis O'Donnell
Vale Inco

Tom Lam
Nuclear Waste Management Organization

Chris Hawkes
University of Saskatchewan

Jon Sykes
University of Waterloo

Kaiwen Xia
University of Toronto
General Information

Registration
The registration desk is located in the lobby of the 200 level of the Metro Toronto Convention Centre. Delegates must register before accessing the technical sessions, purchasing tickets for social functions, or participating in any phase of the conference. The deadline for all refunds was April 27, 2009; no refunds will be issued onsite. The conference badge must be worn at all times (including social events).
Opening hours:
Saturday, May 9 13:00-17:00
Sunday, May 10 8:00-21:00
Monday, May 11 7:30-18:00
Tuesday, May 12 7:30-18:00
Wednesday, May 13 7:30-12:00

Conference Information Booth
Located in the lobby of the 200 Level at the Metro Toronto Convention Centre, the Conference Information Booth will be open during registration hours.

CIM Awards Gala Ticket Exchange
Purchased tickets for the Awards Gala must be exchanged for seat reservation tickets at the Awards Gala Ticket Exchange Booth (located at the registration desk) before 17:00 on Monday.

Presenters’ Preparation Room
Room 201B is open from Sunday, May 10, to Wednesday, May 13. The room has all the necessary audio-visual equipment available for reviewing presentations, which allows presenters to become better acquainted with the equipment.

Breakfast for Session Chairs and Presenters
Breakfast meetings will be held on Tuesday and Wednesday, May 12 and 13, from 7:00 to 8:30 in Room 202 for all authors presenting during that day, to go over the schedule and details with the session chairs and the technical program chair.

CIM Exhibition, Exhibit Halls AB
The CIM Exhibition Showguide is included in the delegates’ bags. The CIM Exhibition, showcasing exhibitor products and services, will be open as follows:
Sunday, May 10 18:00-21:00
Monday, May 11 10:00-18:00
Tuesday, May 12 10:00-18:00

Exhibition Cyber Centre
Delegates are welcome to use the Cyber Centre located in the Exhibition, open during the exhibition hours.

Mining in Society and Career Fair, Exhibit Hall C
The Mining in Society show and the CIM Career Fair will be held at the following times:
Sunday, May 10 9:00-16:00
Monday, May 11 10:00-16:00
Tuesday, May 12 10:00-16:00

Poster Session, Room 202
Monday, May 11 17:40-20:00 Informal poster presentation & mingling around reception.
# Symposium Schedule

## Saturday, May 9
- **8:00-17:00** Workshops<br>University of Toronto
- **13:00-17:00** Registration<br>Metro Toronto Convention<br>Centre Lobby, 200 Level
- **18:30-23:00** Welcome Event: “Taste of Toronto”<br>Royal York Hotel,<br>Canadian Room

## Sunday, May 10
- **8:00-21:00** Registration<br>Metro Toronto Convention<br>Centre Lobby, 200 Level
- **8:00-21:00** Conference Information Booth<br>Metro Toronto Convention<br>Centre, Lobby, 200 Level
- **8:00-17:00** Workshops<br>University of Toronto
- **9:00-16:00** Mining in Society/Career Fair<br>Exhibit Hall C
- **14:00-16:00** CIM Coal and Industrial Minerals Reception and Annual General Meeting<br>Room 104A
- **15:00-21:00** Presenters’ Preparation Room<br>Room 201B
- **18:00-21:00** Opening Reception of the Exhibition<br>Exhibit Halls AB

## Monday, May 11
- **7:30-10:30** Guest Hospitality Room with Breakfast<br>InterContinental Hotel,<br>Caledon/Oakville Rooms
- **7:30-18:00** Registration<br>Metro Toronto Convention<br>Centre Lobby, 200 Level
- **7:30-18:00** Conference Information Booth<br>Metro Toronto Convention<br>Centre Lobby, 200 Level
- **8:00-17:00** Presenters’ Preparation Room<br>Room 201B
- **9:00-11:30** CIM Plenary Session<br>Room 104<br>Departure from the InterContinental Hotel,<br>Caledon/Oakville Rooms
- **9:00-16:00** Wine! Niagara Wine Tours<br>Exhibit Halls AB
- **10:00-16:00** Mining in Society/Career Fair<br>Exhibit Hall C
- **10:00-18:00** CIM Exhibition<br>Exhibit Halls AB
- **12:00-14:00** Lunch<br>in the Exhibition<br>Exhibit Halls AB
- **14:00-16:00** Student Information Session<br>Room 104A
- **14:00-17:40** Rock Engineering Plenary Session<br>Room 203<br>Room 202
- **15:50-16:10** Networking Break<br>Room 202
- **16:30-18:00** Networking Cocktail Reception in the Exhibition <br>Exhibit Halls AB
- **17:40-20:00** Rock Engineering Mingling Around Poster Reception<br>Room 202
- **18:00-19:00** Awards Gala Reception<br>Foyer of the Constitution Hall, 100 Level
- **19:00-23:00** CIM Awards Gala<br>Constitution Hall, 100 Level
Tuesday, May 12
7:00-8:30 Breakfast for Tuesday’s Presenters and Session Chairs Room 202
7:30-10:30 Guest Hospitality Room with Breakfast InterContinental Hotel, Caledon/Oakville Rooms
7:30-18:00 Registration Metro Toronto Convention Centre Lobby, 200 Level
7:30-18:00 Conference Information Booth Metro Toronto Convention Centre Lobby, 200 Level
8:00-17:00 Presenters’ Preparation Room Room 201B
8:40-17:20 Rock Engineering Technical Sessions Rooms 203ABCD
9:00-17:00 Dine! A Toronto Culinary Experience Departure from the Inter Continental Hotel, Caledon/Oakville Rooms
10:00-10:20 Networking Break Room 202
10:00-16:00 CIM Exhibition Exhibit Hall C
12:00-14:00 Rock Engineering Luncheon Room 202
15:20-15:40 Networking Break Room 202
16:30-18:00 Networking Cocktail Reception in the Exhibition Exhibit Halls AB
16:30-18:00 CIM VIP Reception (on invitation only) Room 104B
20:00-00:00 P&H Reception and Dance Liberty Grand

Wednesday, May 13
7:00-8:30 Breakfast for Wednesday’s Presenters and Session Chairs Room 202
7:30-10:30 Guest Hospitality Room with Breakfast InterContinental Hotel, Caledon/Oakville Rooms
7:30-12:00 Registration Metro Toronto Convention Centre Lobby, 200 Level
7:30-12:00 Conference Information Booth Metro Toronto Convention Centre Lobby, 200 Level
8:00-15:40 Presenters’ Preparation Room Room 201B
8:30-15:40 Rock Engineering Technical Sessions Rooms 203ABCD
9:00-10:30 Clothesline! Guest Speaker InterContinental Hotel, Caledon/Oakville Rooms
10:00-10:20 Networking Break Room 202
12:00-14:00 Rock Engineering Luncheon Room 202
15:40-16:00 Networking Break Room 202
18:30-22:30 Baseball Game Rogers Centre
18:30-23:00 Rock Engineering Closing Gala Dinner Hart House, University of Toronto

Thursday, May 14
6:30 Field Trip Departure InterContinental Hotel
9:00-18:00 Workshops University of Toronto

Friday, May 15
9:00-18:00 Workshops University of Toronto
Workshops

Pre-Symposium

1 day - Saturday, May 9
Dynamic Test of Rocks Using Split Hopkinson Bar Facility
Organised by Dr. Xia, U. Toronto,
Fee: $300 including networking breaks, luncheon and course notes

2 days - Saturday and Sunday, May 9 and 10
Rock fracture characterization and Networking modeling in 3D
Organised by Dr. Kulatilake, U. Arizona
Fee: $755 including networking breaks, luncheons and course notes

Instrumentations
Organised by Dr. Bawden, U Toronto
Fee: $500 [Student Fee: $350] including networking breaks, luncheons and course notes

1 day - Sunday, May 10
Two-Dimensional Finite Element Modelling of Slopes and Underground Excavations in Blocky Rock Masses
Organised by RocScience Inc.
Fee: $350 including networking breaks, luncheon and course notes

Post-Symposium

1 day - Thursday, May 14
Lidar, photogrammetry and remote sensing technologies in Rock Engineering
Organised by Drs. Grasselli, U. Toronto, & Diederichs, Queens U.
Fee: $400 [Student Fee: $350] including networking breaks, luncheon and course notes

1 day - Friday, May 15
Combined Finite-Discrete Element Method (FEM-DEM) for Modeling Damage and Fracture in Rock
Organised by Drs. Grasselli, U. Toronto, & Munjiza, QMUL
Fee: $900 [Student Fee: $350] including networking breaks, luncheon and course notes
Field Trip

One field trip is planned for Thursday, May 14, offering participants first-hand insight into tunnelling in Ontario. The tour includes transportation and lunch. Departure will be from the InterContinental Hotel.

Niagara Tunnel, Niagara Falls, Ontario
(07:00 to 17:00, Thursday, May 14)
Cost: $150

Specific protective equipment to bring
CSA approved safety boots

Description: The world’s largest hard rock tunnel boring machine (TBM) is currently driving a 14.4 metre diameter tunnel under the City of Niagara Falls. By the time it finishes the 10.4 kilometre underground grind from the Niagara River to the Sir Adam Beck hydro-electric complex, it will chew up 1.6 million cubic metres of rock.
Social and Guest Programs

*Guest Hospitality Room
The Guest Hospitality Room, located in the Caledon/Oakville Rooms at the InterContinental Hotel, is open from 7:30 to 10:30, from Monday to Wednesday. A continental breakfast will be served daily at 7:30. All guest program tours leave from the Guest Hospitality Room.

Saturday, May 9
Welcome Event: “Taste of Toronto”
Time: Cocktail at 18:30; tasting starts at 19:00
Place: Royal York Hotel, Canadian Room
Price: $100

Sunday, May 10
CIM Coal and Industrial Minerals Reception and Annual General Meeting
Time: 14:00-16:00
Place: Room 104A
Open to all participants

CIM Exhibition Opening Reception
Time: 18:00-21:00
Place: Lounge of Exhibit Halls AB
Open to all participants

Monday, May 11
*Wine! Niagara Wine Tours
Time: 9:00-16:00
Place: Bus departure from the InterContinental Hotel. Group will meet in the Caledon/Oakville Rooms before departure.
Price: $130

Lunch in the Exhibition
Time: 12:00-14:00
Place: Exhibit Halls AB
Included in the registration fee
Additional tickets are available at the registration desk for $20.

Networking Cocktail Reception in the Exhibition
Time: 16:30-18:00
Place: Exhibit Halls AB
Included in the registration fee

Rock Engineering Mingling Around Poster Reception
Time: 17:40-20:00
Place: Room 202
Included in the registration fee

CIM Awards Gala
Time: 18:00 Reception; Dinner at 19:00
Place: Metro Toronto Convention Centre, Constitution Hall, 100 Level
Price: $150
Tuesday, May 12
*Dine! A Toronto Culinary Experience
Time: 9:00-17:00
Place: Bus departure from the InterContinental Hotel. Group will meet at the Caledon/Oakville Rooms before departure.
Price: $75

Rock Engineering Luncheon
Time: 12:00-14:00
Place: Room 202
Included in the registration fee

Networking Cocktail Reception in the Exhibition
Time: 16:30-18:00
Place: Exhibit Halls AB
Included in the registration fee

CIM VIP Reception
Time: 16:30-18:00
Place: Room 104B
By invitation only

P&H Reception and Dance
Time: 20:00-00:00
Place: Liberty Grand; shuttle bus will be provided from 19:30 to 00:30
Included in the registration fee
The conference badge must be worn to access this event.

Wednesday, May 13
*Clothesline - Guest Speaker
Time: 9:00-10:30
Place: InterContinental Hotel, Caledon/Oakville Rooms
Included in the guest registration fee

Rock Engineering Luncheon
Time: 12:00-14:00
Place: Room 202
Included in the registration fee

Baseline Studies - Baseball Game
Time: 18:30-22:30
Place: Windows @ Rogers Centre (entrance located next to Radisson Hotel Lobby)
Price: $90; includes entrance ticket to view game and stadium fare buffet

Rock Engineering Closing Gala Dinner
Time: 18:30-23:00
Place: Hearth House, University of Toronto
Included in the registration fee
Additional tickets are available at the registration desk for $150.
Mining in Society and CIM Career Fair

The Mining in Society exhibition and Career Fair will be held at the following times in the Exhibit Hall C:
Sunday, May 10 09:00-16:00
Monday, May 11 10:00-16:00
Tuesday, May 12 10:00-16:00

Mining in Society Pavilions

**Exploration Pavilion**
- Atlas Copco
- Golder Associates
- PDAC Mining Matters
- Dynamic Earth

**Products and Fabrication Pavilion**
- CIM
- Victaulic
- PDAC Mining Matters

**Mining and Processing Pavilion**
- Tormont
- Lincoln Strategic Initiatives
- CMP Toronto
- Sheritt
- Coal Association of Canada
- PDAC Mining Matters

**Sustainability Pavilion**
- MAC
- Vale Inco
- NRCAn
- Xstrata
- PDAC Mining Matters

**Education Pavilion**
- MiHR
- Victaulic
- OMA
- Norcat
- University of Toronto
- Laurentian University
- MIRARCO
- Northwest Community College / School of Exploration and Mining
- PDAC Mining Matters

**New Frontiers Pavilion**
- Norcat
- Penguin ASI

**Career Fair Exhibitors**
- Agrium Partnership
- Cameco Corporation
- Canadian Natural Resources Ltd. - Horizon Oil Sands Project
- De Beers Canada Inc.
- Goldcorp
- Imperial Oil Limited
- North American Construction Group
- P&H MinePro Services Canada Ltd.
- Rio Tinto
- Shell Canada
- The Mosaic Company
- Xstrata
TECHNICAL PROGRAM
<table>
<thead>
<tr>
<th>Time</th>
<th>Session</th>
<th>Title</th>
<th>Speaker/Institution</th>
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<tbody>
<tr>
<td><strong>MONDAY May-11</strong></td>
<td></td>
<td><strong>14:00-14:15</strong> WELCOME TO ROCKENG09</td>
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<tr>
<td>14:15</td>
<td>1</td>
<td>PLENARY KEYNOTE 1 - Challenges facing ground control engineers in a hostile mining environment</td>
<td>William Bawden</td>
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<tr>
<td>15:00</td>
<td>2</td>
<td>PLENARY KEYNOTE 2 - Designing underground excavations for a nuclear waste repository: Old Issues and New Approaches</td>
<td>Derek Martin</td>
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<tr>
<td>16:00</td>
<td>3</td>
<td>PLENARY KEYNOTE 3 - The numerical analyses toolbox in geomechanics - the present and the future</td>
<td>Joe Carvalho</td>
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<td>14:27</td>
<td>4</td>
<td>Geomechanics Strategies For Rockburst Management At Vale Incos Creighton Mine</td>
<td>Malek Deep Mining</td>
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<tr>
<td><strong>SESSION 3</strong></td>
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<td>8:40-10:00 Fracture Mechanics Examples Of Applications In Rock Engineering</td>
<td>Backers Numerical modeling of continuum-discontinuum behaviour</td>
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<td>09:24</td>
<td>5</td>
<td>Rock Mechanics Approach For The Recovery Of The Zone G Crown Pillar At The Raglan Katinniq Mine</td>
<td>Andrieux Use of LIDAR and digital photogrammetry in rock engineering</td>
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<td>09:34</td>
<td>6</td>
<td>Measurement Of Surface Roughness Of Rock Discontinuities</td>
<td>Poropat Use of LIDAR and digital photogrammetry in rock engineering</td>
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<td>09:48</td>
<td>7</td>
<td>Remote Sensing Discontinuity Characterization At The Palabora Open Pit, S Africa</td>
<td>Sturzenegger Use of LIDAR and digital photogrammetry in rock engineering</td>
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<tr>
<td>10:05</td>
<td>8</td>
<td>Geotechnical Applications Of 3-Dimensional Laser Scanning In Tunnelling</td>
<td>Fekete Use of LIDAR and digital photogrammetry in rock engineering</td>
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<tr>
<td><strong>SESSION 4</strong></td>
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<td>10:20-12:00 Influence Of Anisotropic Rock Mass Characteristics On Tunnel Stability</td>
<td>Bewick Conventional and TBM tunneling</td>
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<td>11:00</td>
<td>9</td>
<td>Update Of The Conditions In The Donkin-Morien Tunnels</td>
<td>Seedsman Conventional and TBM tunneling</td>
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<td>11:15</td>
<td>10</td>
<td>The Pavoncelli Tunnel Case Study</td>
<td>Barla Conventional and TBM tunneling</td>
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<td>11:30</td>
<td>11</td>
<td>Analytical Solution For A Tunnel Excavated In A Porous Elasto-Plastic Material Considering The Effects Of Seepage Forces</td>
<td>Barbosa Conventional and TBM tunneling</td>
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<td><strong>SESSION 5</strong></td>
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<td>10:20-12:00 Validation Of Csat For Characteriztion Of Internal Macropores</td>
<td>Hudyma Geophysics in rock engineering</td>
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<td>11:00</td>
<td>12</td>
<td>Application Of 3d X-Ray Ct Scanning Techniques To Evaluate</td>
<td>Nasseri Behrestaghi Geophysics in rock engineering</td>
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<td>11:15</td>
<td>13</td>
<td>Rock Mechanics And Subsurface Imaging At Dusel, Homestake Mine</td>
<td>Van Beek Geophysics in rock engineering</td>
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<td>11:30</td>
<td>14</td>
<td>Multiparameter Petrophysical Characterization Of An Orebody: An Exploration Case History</td>
<td>Bongajum Geophysics in rock engineering</td>
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<td><strong>SESSION 6</strong></td>
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<td>10:20-12:00 Numerical Expansion Analyses Of The Strategic Petroleum Reserve In Bayou Choctaw Salt Dome, Usa</td>
<td>Park Reservoir geomechanics</td>
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<td>11:00</td>
<td>15</td>
<td>A New Analytical Model To Simulate Sagd Process Considering Geomechanics</td>
<td>Azad Reservoir geomechanics</td>
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<td>11:15</td>
<td>16</td>
<td>Poroelastic Modelling Of Production And Injection-Induced Stress Changes In A Pinnacle Reef Using Semi-Analytical And Numerical Methods</td>
<td>Hawkes Reservoir geomechanics</td>
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<td><strong>SESSION 7</strong></td>
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<td>14:00-15:20 In Situ Fracturing Mechanics Stress Measurements To Improve Underground Quarry Stability Analyses</td>
<td>Ferrero Surface Construction</td>
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<td>14:30</td>
<td>17</td>
<td>Calcium Leaching Of Rock Concrete Interface Using Dem</td>
<td>Buzzi Surface Construction</td>
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<td>14:45</td>
<td>18</td>
<td>Deriner Hydropower Scheme Geotechnical Issues Related To The Dam And Hepp Construction</td>
<td>Cekerevak Surface Construction</td>
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<td>14:45</td>
<td>19</td>
<td>Study On Feasibility Of Columnar Joint Basalt As A High-Arch Dam Foundation</td>
<td>Wei Surface Construction</td>
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<tr>
<td><strong>SESSION 8</strong></td>
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<td>14:00-15:20 Numerical Modelling Of A Brazilian Disc Test Of Layered Rocks Using The Combined Finite-Discrete Element Method</td>
<td>Khajeh Mahabadi Numerical modeling of continuum-discontinuum behaviour</td>
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<td>14:30</td>
<td>20</td>
<td>Shear strength reduction with the Barton-Bandis Joint Yield Criterion</td>
<td>Hammah Numerical modeling of continuum-discontinuum behaviour</td>
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<td>14:45</td>
<td>21</td>
<td>Thm Processes In A Fluid-Saturated Poroelastic Medium: Comparison Of Analytical Results And Computational Approaches</td>
<td>Selvadurai Deep underground nuclear waste repositories</td>
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<td>14:45</td>
<td>22</td>
<td>Bonded-Particle Modeling Of Tunnel Sealing Experiment</td>
<td>Wanne Deep underground nuclear waste repositories</td>
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<td>14:45</td>
<td>23</td>
<td>Reasoned Argument Why Large-Scale Fracturing Will Not Be Induced By A Deep Geological Repository</td>
<td>Read Deep underground nuclear waste repositories</td>
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<td><strong>SESSION 9</strong></td>
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<td>10:20-12:00 An Improved Definition Of Rock Quality Designation</td>
<td>Li Rock mass characterization and Site investigation</td>
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<td>11:00</td>
<td>24</td>
<td>Constitutive model for small rock joint samples in the lab and large rock joint surfaces in the field</td>
<td>Barbosa Rock mass characterization and Site investigation</td>
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<tr>
<td>11:15</td>
<td>25</td>
<td>Complex Networks on a Rock Joint</td>
<td>Ghaffari Rock mass characterization and Site investigation</td>
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<td><strong>SESSION 10</strong></td>
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<td>14:00-15:20 Instrumentation Of A Graphite Zone In The #3 Shaft At Brunswick Mine</td>
<td>Turichshev Innovation in ground support and instrumentation</td>
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<td>14:30</td>
<td>26</td>
<td>Testing And Analysis Of Welded Steel Wire Mesh Used For Surface Rock Support</td>
<td>Thompson Innovation in ground support and instrumentation</td>
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<tr>
<td>14:45</td>
<td>27</td>
<td>Dynamic Testing Of Friction Stabilisers</td>
<td>Player Innovation in ground support and instrumentation</td>
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<tr>
<td><strong>SESSION 11</strong></td>
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<td>14:00-15:20 Overview Of Ontario Power Generations Proposed L&amp;llw Deep Geologic Repository Bruce Site, Tiverton, Ontario</td>
<td>Jensen Deep underground nuclear waste repositories</td>
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<tr>
<td>14:30</td>
<td>28</td>
<td>The Role Of Rock Engineering In Developing A Deep Geological Repository In Sedimentary Rock</td>
<td>Read Deep underground nuclear waste repositories</td>
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<td>Session 13</td>
<td>13 4019</td>
<td>Investigating Fault-Slip Mechanisms In Swaradity Active Structures</td>
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<td>8:40-10:00</td>
<td>13 4026</td>
<td>Research To Reality: Application Of Mining-Induced Microseismic Hazard Maps</td>
<td>Valak</td>
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<td>8:40-10:00</td>
<td>13 4028</td>
<td>Re-Entry Protocols For Swaradity Active Mines</td>
<td>Vahidi</td>
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<td>45</td>
<td>13 4275</td>
<td>Basic Characteristics of Wenchuan Earthquake And Its Geological Hazard Effects</td>
<td>Wang</td>
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<tr>
<td>Session 14</td>
<td>14 4033</td>
<td>Stress Measurements At Great Depth At Craig-Onaping Mines, Sudbury, Canada</td>
<td>Violosa</td>
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<tr>
<td>8:40-10:00</td>
<td>14 4034</td>
<td>Statistical Multi-Scale Method Of Mechanics Parameter Prediction For Rock Mass With Random Quadrilateral Distribution*</td>
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<td>8:40-10:00</td>
<td>14 4035</td>
<td>Geomechanical Model Of An Alpine Rock Mass</td>
<td>Apauni</td>
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<td>9X</td>
<td>14 4109</td>
<td>A New Tool For Field Characterization Of Joint Surfaces For Engineering Design</td>
<td>Mine</td>
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<td>Session 15</td>
<td>15 3999</td>
<td>Structural And Mechanical Characterization Of The Rock Mass Formations Of Santa Barbara Open Pit Mine (Tuscany, Italy)</td>
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<td>8:40-10:00</td>
<td>15 4034</td>
<td>Statistical Multi-Scale Method Of Mechanics Parameter Prediction For Rock Mass With Random Quadrilateral Distribution*</td>
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<td>Geomechanical Model Of An Alpine Rock Mass</td>
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<td>8:40-10:00</td>
<td>15 4036</td>
<td>Use Of 3D Digital Systems For The Characterization Of Rock Joint Roughness In-Situ And In-The Laboratory</td>
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3rd Canada-US Rock Mechanics Symposium &
20th Canadian Rock Mechanics Symposium

ROCKENG09
ROCK ENGINEERING IN DIFFICULT CONDITIONS

9 | 15 May 2009
Toronto, ON, Canada

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Editors M. DIEDERICHS & G. GRASSELLI
EXTENDED ABSTRACTS

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Mike Yao
Chief Ground Control Engineer, Mine Technical Services, Vale Inco, Sudbury, Canada
D. Reddy Chinnasane
Ground Control Specialist, Copper Cliff North Mine, Vale Inco, Sudbury, Canada
Dunn Harding
Ground Control Engineer, Mine Technical Services, Vale Inco, Sudbury, Canada

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Y.X. Zhao & Y.D. Jiang
State Key Lab of Coal Resources and Safe Mining, China University of Mining and Technology, Beijing, P.R. China

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D.A. Beck, M. J. Pfitzner, S.M. Arndt and B. Fillery
Beck Arndt Engineering Pty Ltd, Sydney, Australia

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Geomechanics Strategies For Rockburst Management At Vale Inco Creighton Mine
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Mitigation Plans for Mining in Highly Burst-Prone Ground Conditions at Vale Inco Copper Cliff North Mine

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ABSTRACT: While mining the approved stope sequence in the 100/900 ore bodies at the North Mine in Sudbury Basin of Vale Inco Ltd and following a crown blast in one of the stopes in 100 orebody in a sill pillar area, several major seismic events, with the highest magnitude of 3.8 Mn (Nuttli), occurred in both the Trap Dyke, which is located between 100 and 900 orebodies, and in the sill pillar itself. This paper describes the mechanism of these events and the methodology of designing highly yielding ground support for rehabilitation areas with a view to withstand future seismic impact. A risk-based systematic approach was developed to determine where enhanced support is required in any other areas to be exposed by future seismic activities.

Numerical modelling undertaken for the low part of the 100 ore body reveals that a future sill pillar, having similar geometry to the one in the upper, will be subjected to even higher mine-induced stress conditions. This paper discusses the mitigation plans to control seismicity in this highly burst-prone ground conditions, which include: modifying mine plans to eliminate the narrow sill pillar, an introduction of high yielding ground support, such as de-bonded cable bolts, Modified Cone Bolts and FS-46 friction sets etc, and installation of field instrumentation as well as the utilization of de-stressing techniques in both development drifts and around stopes, which has been successfully implemented at other mines of Vale Inco’s Sudbury basin.
There are a number of major geological structures present at the Copper Cliff North Mine. At the southern end of the North Mine, the Creighton Fault displaces the quartz diorite dyke approximately 700 meters. The Creighton fault strikes east-west, and dips steeply towards north. The 900 orebody cross fault strikes east-west and dips 55 degrees north and it does not appear to displace the quartz diorite dyke. The Number 2 Mine cross fault strikes northeast, and dips 55 degrees northwest. The Number 2 Mine fault displaces the quartz diorite dyke 70 meters to the west. The Number 1 cross fault displaces the quartz diorite dyke 65 meters west. This fault strikes east-west and dips 40 degrees north. Apart from the above major faults, two dykes namely Quartz Diabase Dyke (Trap Dyke) and Olivine Diabase Dyke are present in the close proximity of 100/900 ob. The Trap Dykes are located between 100 and 900 ob striking east-west and dipping steeply towards north.

Historical data revealed that 900 cross fault was very seismically active and caused significant seismic events/rock bursts in and around 100/900 ob in the past. Recent seismic activity in the middle and lower 100/900 ob revealed that even trap dykes are known sources of major seismic activity.

After any major seismic events, mine operators face a few critical questions, which need immediate attention. Questions include: what types of enhanced support is required for areas, which suffered damage in order to withstand future seismic risks? Considering the occurrence of these events, which areas are likely to be burst prone in the future thus enhanced support being required? This paper describes a simple risk rating system to determine where enhanced support is required. A methodical approach is also presented to establish what type of enhanced support is required for both rehabilitation areas as well as for future potentially burst prone areas. In addition to implementation of enhanced support, mine design and planning strategies to control seismic risks are also discussed in this paper, including eliminating diminishing pillars, de-stress practice in both development and stopes, alteration to mining method as well as the introduction of field instrumentation to monitor ground movement and ground support effectiveness. All these mitigation plans are implemented to ensure that the remaining 100 and 900 orebody can be mined safely and efficiently.
This paper briefly describes the characteristics and induced factors of coal mining bumps based on the investigation of recent bump accidents that have occurred in China. According to the theory of non-equilibrium thermodynamics and dissipate structure, the process of strain energy accumulation and dissipation in the ‘Coal-Surrounding Rock’ system (CSRS) is discussed during the preparation of coal bumps. In addition, a series of experiments are conducted to analyze the relationship between bump-prone property and micro-structural characteristics of coal. The process of coal bumps induced by propagation of fractures and deterioration of coal mass properties are also analyzed systematically.

Coal bump is defined as a sudden release of the geologic strain energy that can expel large amounts of coal and rock into the face area, resulting in fatalities and injuries to underground workers (Fig. 1). This has been recognized as a sudden catastrophic failure of coal and causes serious problems to underground coal mining worldwide in the past 100 years. In the past ten years, coal bump incidents have increased with rapid development of coal mining in China. Statistics showed that bump accidents caused hundreds of fatalities and injuries in the period from 1997 to 2008 in underground coal mines. Coal bumps have already been one of the most dangerous damage occurrences to underground mining safety in China. The other three destructive damage events are rock fall, coal and gas outburst and water inrush. So understanding the mechanism of coal bumps becomes more and more urgent.

Characteristics of coal mining bumps. (a) Heavy equipment thrown by coal bump, Tangshan coal mine. (b) U-shaped steel support destroyed by bump, Laohutai coal mine.
Though the prime factors play a more important role in triggering coal bumps, many bumps are induced by multivariable coupling factors. In view of assessing the influence of a combination of different factors, the information from four Chinese coal mines were studied, including the Zhao-gezhuang, Laohutai, Xinzhouyao and Yaoqiao mines. The results indicate that the important variables should include the effects of bump liable coal, mining depth, geological structures, coal pillar as well as earthquakes or blasting tremors. So the multivariable coupling factors can be defined as:

- **Energy release ratio** - includes the effects of depth, coal properties, and geological structures.
- **Disturbed ratio** - includes the effects of mining method, and blasting or earthquakes tremors.
- **Coal Pillar stability index** - includes geometry of the coal pillars, and the surrounding rocks conditions.

The Figure below illustrates the process of the system from its original stable state to a new stable state triggered by free energy release.

![Figure illustrating energy release and stability](image)

Different stages of free energy stored in the ‘coal-surrounding rock’ system.

Coal bumps can be characterized as the process of unstable energy release with time and non-uniform in space, which is associated with yielding that occurs with progressive mining. Many variables can affect the bump prone conditions. This paper briefly described the characteristics and induced factors of coal mining bumps based on the investigation of recent bump accidents that occurred in China and investigated the thermodynamic process in the nucleation of coal bumps. The following conclusions have been drawn regarding the micro-structural features of bump prone coal and their potential application to better understanding coal bump mechanisms:

1. The far-field region away from excavations in the coal seam was stable and saw little effect from the mining during the roadway tunneling process. The stability of the coal mass adjacent to the excavations can be determined by the stability of internal thermodynamic process. The factors which affect the thermodynamic process in coal are mainly stress gradient, plastic transformation, microstructures and macerals in the coal.

2. A bump liability indices \( \xi = (L_a - L_c)/L_c \) was proposed to determine the bump potential of coal. It was found that the bigger the \( \xi \) value, the more dangerous and liable to bump in the coal seam. The macerals analysis revealed that the coal, composed of more vitrinites and inertinites, had more potential to bump because of the micro-hardness and micro-brittleness. The value of \( |R_{max} - R_{min}| \) had some relationship with bump liability: the smaller the value of \( |R_{max} - R_{min}| \), the less potential of coal bumps. The results also indicated that the microstructure features can aid to determine the bump liability and the historical stratigraphic evolution, which can be recorded by the microstructures in the coal.

3. It was also proved that energy dissipation in bump preparation process affects the mechanical properties of coal significantly. The influence on bump liability can be described quantitatively by the \( \xi \) value.
Estimating rock mass properties and seismic response using higher order, discontinuous, Finite Element models

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The load-deformation response of discontinuous rock under static and dynamic loading conditions has been simulated using Explicit Finite Element models. The intent of the analysis was to investigate the effects of specimen size and confining stress on strength, dilation and comminution.

The simulations allow the development of homogenised constitutive material properties for discontinuous rock masses using laboratory scale measurements and representative Discrete Fracture Networks (DFNs). A procedure for this is presented which includes a comparison of measured seismic response in a mine to the Dissipated Plastic Energy (DPE) that is released in the simulated rock masses. The models also show how confinement and scale affect the stress-strain and DPE response of the simulated rock specimens, reproducing a number of known rock phenomena that are often poorly captured in geotechnical modelling.

A case study is presented showing a satisfactory match between the model-derived, homogenised material properties and values achieved by calibration of a mine-scale model where many thousands of seismic, displacement and damage measurements were available.

Some procedures for rock mass scale property estimation using 3D, discontinuous FE models and some potential applications have been tested. The results show that it is feasible to re-create realistic rock mass phenomena, including confinement dependence of the residual response, seismic behavior and comminution.

More work is required to better understand some of the applications of this technology, but it is probable that the techniques could eventually offer an analytical option for estimating rock mass scale material properties that improves on current empirical tools and serves as an adjunct to quantitative calibration.
An example of a DFN model of skarn

DFN based on structural mapping

Fragmented, comminuted specimen after simulation

Higher order element mesh for homogenization studies
At Creighton Mine, magnitude events are a regular occurrence due to depth and major seismically-active structures. Over the last ten years, major advances have been made in research and development to develop strategies for the management of these major events. Today, the number of rockbursts has been drastically reduced at the mine and damage due to seismic events, when they occur, is often minimal. This paper describes the evolution and implementation of appropriate geomechanics strategies, designs, and practical ground control measures undertaken for the management of seismicity at Vale Inco’s Creighton Mine. A description of the research and development approach, implementation, and ground control strategy is provided for placement of enhanced support systems to manage the consequences of magnitude events at Creighton Mine. The approach ensures the safety of workers, stability of mine infrastructure, continuous access to ore and minimal downtime after large events.seismicity occurs close to mining activity, mostly due to sudden mining induced stresses. A small fraction of the recorded seismicity, and significant events, occur tens to hundreds of meters away from mining, as a result of fault-slip along major seismically active shear zones (e.g., Plum Shear, Footwall Shear). These events are generally of large magnitude, pose substantial damage potential and can occur minutes, days or even weeks after production blasts.

The geology of Creighton Mine is complex whereby several discrete geologic structures intersect the major rock units. As well, this mine has been in operation for over a century and the extraction has reached maturity, literally leaving a significant void underground from surface to 7810 level. The combination of the complex geology with discrete geological structures, a large mined-out volume and the increasing mining depth is challenging in terms of designing and mining in a way that effectively manages the inevitable seismicity.

To date, through strategic planning, in combination with research, Creighton Mine has successfully managed the difficult mining conditions. This is evident from the reduction in the number magnitude events occurring each year, in the limited amount of damage that has occurred to the mine infrastructure, and in the continued safety and protection of equipment and underground personnel.

Scientific visualization techniques in the Laurentian University’s Virtual Reality Laboratory have enabled a better understand of the behavior of seismic and microseismic activities in the complex mining environment. This understanding has made it possible to develop seismic hazard maps as an aid in strategically identifying locations where enhanced support systems are required and in locating the future mine infrastructure. This, combined with sound ground control practices, has enabled Creighton Mine to effectively manage the occurrence and consequences of seismic events.
Late-stage faults, locally termed shears, at a depth of 7200 feet (2200 m).

ParaviewGeo visualization of a Seismic Hazard Map on section and projected on drifts, showing integrated geology and stope information.

Seismic Excavation Hazard Maps at Creighton Mine will be an integral part of the geotechnical review process for both strategic and tactical approaches for mine design as the mine progresses to greater depths.
SESSION 2  NUMERICAL MODELLING OF CONTINUUM-DISCONTINUUM BEHAVIOUR I

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Constitutive Model for Numerical Modelling of Highly Stressed Heterogeneous Massive Rocks at Excavation Boundaries
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3894
Fracture Mechanics Numerical Modeling – Potential and Examples of Applications in Rock Engineering
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3983
Discrete Element Modeling Of The Influence Of Void Size And Distribution On The Mechanical Behavior Of Rock
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4060
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A numerical modelling approach was developed to explicitly simulate geomechanical characteristics of intact rock: mineralogy, grain size and fabric. The approach involved creating a representative constitutive model (Figure 1) for each of three common rock-forming minerals: mica, quartz and feldspar. The constitutive models developed are valid within the low confinement realm of excavation boundaries, where tensile fracture processes dominate. The mineral types were assigned to numerical elements, which were associated with each other through an algorithm created in a finite difference model, FLAC 2D (Itasca 2007), to simulate real crystal geometries and orientations (Figure 2). The numerical models were used in a parametric investigation of the geomechanical characteristics and compared with published observations of the rock yielding process in laboratory testing. This approach has allowed the explicit grain-scale investigation of the impact of geomechanical characteristics on rock yielding at low confinement, leading to an improved mechanistic understanding of excavation-scale rock yielding processes at excavation boundaries.

The mineral-specific behaviour described in literature was also observed in the 2-D FLAC UCS and Brazilian models. During damage accumulation the micas, being soft in shear can:
- induce failure in the surrounding stiffer, stronger, feldspar grains
- offset tensile fractures propagating through feldspar since mica does not propagate fractures well at a large angle to the cleavage
- halt tensile fractures propagating through feldspar at the softer and weaker grain boundary.
Eventually a single macrofracture will develop under uniaxial loading, and the fracture path will include all mineral types.

Figure 1: Schematic of strength property (cohesion or friction coefficient, m) and resulting axial strength as it changes with strain, where \( \varepsilon_E \) and \( \varepsilon_P \) are accumulated elastic and plastic strain, respectively. Peak strength parameters are used until the element fails and begins to accumulate plastic strain, at which friction increases instantaneously, followed by two levels of intermediate strength parameters (a,b), as a function of increasing plastic strain, until the residual strength parameters are reached.
Peak values in the biaxial numerical model tests may actually be more representative of the crack initiation threshold, rather than true laboratory peak strength. This phenomenon is particularly important for biaxial model test results since the impact of confining stress is not correctly modelled, as demonstrated by the lower slope angles of modelled test results (Figure 3). With respect to UCS testing the confining stress is not an issue as the test is undertaken in the unconfined zone of Figure 1, and the behaviour of the model test can be taken as an analogue to the laboratory test. Issues arising from 2-dimensional versus 3-dimensional samples for UCS testing are independent of this phenomenon. Tensile failure in Brazilian tests in FLAC is less dependent on fracture accumulation and coalescence than the UCS failure, since the tensile stress is generated at the centre of the sample and leads to tensile failure in the material of least resistance within the zone of tensile stress.

Figure 3: (left) Photo of Stanstead granodiorite showing isotropic nature of the material. Grey=quartz, white=feldspar, predominantly plagioclase, black=mica, predominantly biotite. (right) Image of modelled Stanstead granodiorite in FLAC. Colours relate to mineral type as follows: turquoise = feldspar, green = quartz, red = mica, yellow = grain boundaries.

Figure 3: Peak strength envelopes for Stanstead lab test data (courtesy of J. Archibald), Stanstead model data with 15% dilation and Stanstead model data with 15% dilation and instantaneous friction increase.

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In classical rock mechanics rock is viewed as a continuous flawless material. Failure or strength of rock is described by rather empirical criteria, which depend on the phenomenological description of mostly laboratory data. The parameters to the strength criteria are determined in laboratory geotechnical testing, where the response is characterized only globally, i.e., based on measurements at the boundary that are assumed representative of the overall (homogeneous, continuous) sample response. Thus, only in case of a perfectly homogeneous material undergoing perfectly uniform deformation, a constitutive behavior may be expected. However, the hypothesis of continuity does not hold for rock. Rock material is a discontinuous combination of solid matter, pores, cracks and fractures.

All analyses of the structural breakdown of rock clearly show that a continuous approach cannot reflect the mechanics of breakdown correctly. The existence of a crack in an otherwise solid homogeneous body reduces the strength of the structure considerably. Any load acting on the body is magnified several times at the tip of such a discontinuity and when the stress concentration at the tip of the crack reaches a critical level, it propagates. When rock is loaded, a swarm of pre-existing discontinuities redistributes the stresses locally and individual cracks may start propagating. When stressed further, isolated reactivated microcracks may coalesce and form a larger fracture.

Linear fracture mechanics provides the tools to estimate the stress and displacement fields around the tip of a crack. Any loading of a fracture will result in an alteration of the stresses at the fracture tip, which may be described by the stress intensity factor $K_I$, which is a function of the crack length describing the shape and grade of stress concentration at the crack tip. The resistance of rock to the propagation of fractures is described in terms of fracture toughness. The fracture toughness $K_{IC}$ is the limit of local stress increase, i.e. stress intensity, due to an existing fracture at onset of critical extension.

Therefore, discontinuities, i.e. cracks, pores, fractures or joints, are an important feature of rock and rock mass and control not only the hydraulic properties of rock by well-connected fracture networks, they also govern the mechanical behavior. When the stresses in a rock mass are altered, be it e.g. by the depletion of a reservoir, sequestration of liquid or gas, increase of pressure by EOR operations, or the introduction of a new excavation, the pre-existing discontinuities may grow. This may create new pathways for fluid flow or, if pre-existing fractures are propagated and coalesce to form larger structures that intersect with an excavation or another free surface, the structures may lose integrity and fail.

Due to the complex nature of rock mass behavior and related fracturing, a numerical modeling approach to the task is favorable. In this contribution the capability of using fracture mechanics based numerical simulations to rock mechanics applications is tested on a variety of examples. A selection of rock fracture mechanics modeling codes is outlined and the perspectives and future potentials are discussed.

Several numerical approaches have been applied to rock mechanics challenges in general. For a review of the different methods and a comprehensive reference base refer to Jing and Hudson (2002) and in particular Jing (2003). For explicit fracture propagation analysis of geomaterials only few codes are available; see Bäckström et al. (2008) also. In this paper only selected approaches based on different platforms (particle flow code PFC, boundary element method BEM,
finite element method FEM, and Extended or Generalized finite element method XFEM) are discussed in general; completeness of arguments and methods is not assumed.

Selected examples show the potential and limitations of today’s fracture mechanics modeling. In the contribution an example of simulating shear fracture propagation in the Punch-Through Shear with Confining Pressure PTS/CP Experiment using DIGS, modeling Class II behavior in uniaxial compression experiments and the estimation of the in-situ stress field using Fracod2d are highlighted, showing that fracture mechanics based numerical simulations are one step closer towards a physical based model of rock. As pointed out, fractures in rock may govern the mechanical and hydro-mechanical behavior of rock. Therefore, it is a logical and consequent move to mirror the processes involved in the formation of fractures by means of material properties. Instead of applying bulk properties for rock mass, which always requires proper tuning of any conventional model, a fracture mechanics approach demands mechanical properties for intact rock, which can be determined in the laboratory for readily available rheological models, and properties describing fracture generation and behavior. The latter are all physically based and can mostly be determined in the laboratory or field.

As was shown in the examples, the propagation of fractures and generation of fracture networks can be simulated in two dimensions already today. The next step towards a comprehensive fracture mechanics solution for rock is the extension to three spatial dimensions. This is not realized convincingly yet. The mathematical constraints for the extension are not well defined and the numerical tools for this task need to be developed. There are some efforts in this direction undertaken currently.

Rock and rock mass display discontinuities, i.e. cracks, fractures, joints or faults, on different scales. The discontinuous nature of rock is reflected by the DIGS code through an imprinted grid, which fails in the given example to reflect the mesoscopic breakdown process in conjunction with the energy demand. Fracod2d uses an equivalent macro-failure constitutive law; hence the granular nature is not addressed and restricts the code to addressing of the fracturing processes in the meter scale.

As discussed by Napier and Backers (2006) and as manifested in the basic theory for the extension of fractures, the longer a fracture, the smaller the energy required for its extension. This implies for rock or rock mass, that in principle the largest fracture influenced by a change of stress (or any other boundary condition lowering the energy demand for fracture extension) will propagate until the energy is consumed or further fracture propagation is geometrical impossible. In return, this implies that new discontinuities will not be created/initiated, but discontinuities readily available will be activated only. How to introduce realistic sets of discontinuities into such numerical models is not sufficiently solved today without exceeding the numerical capabilities, as the dimensions of discontinuities span several orders of magnitudes. Here the available statistical models have to be reviewed and adapted for the needs of fracture mechanics modeling, bearing the chance to be able to realize scale insensitive models. The most promising approach is the XFEM in combination with adaptive hp-approaches.

Based on the combination of readily available models and rock fracture mechanics there is a large potential for different geomechanical applications. In geothermal projects such an approach could help analyze the risk for a thermo-/hydraulic- shortcut or optimize the drilling operations in the reservoir, where underbalanced drilling is favorable in some cases. In the radioactive waste disposal industries, the fracture mechanics approach will help and already has helped understanding the risk for the creation of potential pathways for radionuclide transport and optimize the layout of underground repositories from a long term safety and performance assessment point of view. Slope stability analyses will be able to focus on fracture interaction rather than single discontinuity analysis and hence improve the reliability of the predictions. In the reservoir mechanics applications the factors influencing borehole instabilities or sand production might be identified with such analyses. Hydraulic stimulation campaigns could be analyzed with the aim of an optimized connection of the existing fracture network to the wellbore.
The effect of porosity on mechanical behavior of rock has been studied in the past using both experimental and numerical techniques. One important aspect of this study that has not received much attention is the role of void size and distribution on the mechanical behavior of rock. In this study, the CA2 computer program that is a hybrid discrete-finite element program for two-dimensional simulation of geomaterials is used. The rock is modeled as a bonded particle system. To obtain numerical samples with different porosity, two different sets of cylinders are generated. The first set, macro void cylinders, is made of cylinders that represent the macro pore spaces. The second set of cylinders, grain cylinders, is to model the rock grains. The grain cylinders are generated by assuming a uniform random distribution for their radii while the void cylinders are assumed to have a constant radius. Both sets of cylinders are randomly placed in a confined domain defined by surrounding finite elements. The cylinders are then inflated to their final radii and the equations of motion together with linear contact laws for interaction of cylinders are solved to adjust the location of cylinders and to distribute the micro void spaces more uniformly. At this stage, the cylinders are not bonded at the contact points and are friction free but the void cylinders are held fix in their position to control the macro void positions and distribution within the numerical sample. After sample preparation, the initial stresses, contact forces, and velocity vectors are initialized to zero, normal and shear bonds and friction are introduced at the contact points, and the macro-void cylinders are deleted from the numerical sample. The generated samples are tested numerically to obtain Young’s modulus, Poisson’s ratio, and uniaxial strength values. It is shown that with increase in pore space, the modulus and uniaxial strength reduce while Poisson’s ratio can increase.

To investigate the effect of macro void distribution on the elastic modulus and uniaxial strength, fifty numerical specimens with the porosity of 16% and macro void radius of 2.5 mm and fifty specimens with the porosity of 16.8% and macro void radius of 0.5 mm were generated. For each numerical specimen, a uniform random number generator was used to find the x and y coordinates of the center of each macro void cylinder. Figures 1 and 2 show the histograms for the Young’s modulus and uniaxial strength of specimens with large and small macro pore sizes, respectively. The mean and coefficient of variation values for the Young’s modulus and uniaxial strength of specimens with large and small macro void sizes are (32.5 GPa, 11%; 73.1 MPa, 17.5%) and (25.5 GPa, 4.2%; 43.0 MPa, 13.1%), respectively. Note that specimens with larger macro void size are both stiffer and stronger but they show greater variation around mean values of Young’s modulus and uniaxial strength. This is consistent with the fact that for the same porosity value but larger macro pore sizes, the sample has a greater chance of being non-homogeneous in the distribution of the macro pore locations. This finding suggests that samples with larger pore size must be greater in dimension if a Representative Volume Element (RVE) is to be obtained.
The results of the paper can be summarized as follows:

- From numerical specimens with the same porosity of about 16%, specimens with larger macro pore size have larger mean values of Young’s modulus and uniaxial compressive strength.
- Random change of the location of the macro-pores in the specimen causes variation in specimen stiffness and strength. These variations are greater for specimens with larger macro-pore radius. This suggests that for specimens with larger macro-pore size, the dimensions of a Representative Volume Element should be larger.
- The range of crack initiation stress for specimens with larger macro-pore size is wider. On average, for specimens with the same dimensions and the same porosity of about 16%, the micro-cracks start at earlier stages of loading (with respect to the uniaxial strength) in a specimen that has macro-voids of larger size.
- Failure pattern of specimens with the same porosity is affected by both the macro pore size and distribution.
- The numerical model shows similar scatter in uniaxial strength values to that from some published physical tests. The scatter of physical elastic modulus values is greater than that of the numerical model.
- For rocks that the majority of micro-cracks are developed at the grain boundaries, i.e. for rocks with strong grains, in addition to porosity, the dimensionless parameter of macro pore size to grain size ratio can affect the mechanical behavior of rock.

Histograms of: (a) Young’s modulus, (b) uniaxial strength of specimens with a porosity of %16 and a macro void radius of 2.5 mm.

Histograms of: (a) Young’s modulus, (b) uniaxial strength of specimens with a porosity of %16.8 and a macro void radius of 0.5 mm.
Modeling of rock fracture flow using the Lattice Boltzmann Method on graphics hardware

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Bioremediation has been accepted as a treatment technique for groundwater contamination in subsurface soils and shows promise for contaminated fractured rock environments. Biological growth in fractured rock is expected to occur predominantly as biofilms attach to the fracture surfaces. Biofilms in rock fractures are subject to a complex system of forces and other phenomenon due to the dynamics of the bulk fluid in which they grow. In this paper, through the applications of computational fluid dynamics (CFD) to rock fractures, where the boundaries are rough and the flow is complex, a precise analysis was conducted of the interaction of a fluid flow and the rock fracture. Specifically, hydraulic parameters and velocity profiles of an actual rock fracture were calculated and compared to a fracture of equivalent aperture. From the analysis it is clear that it is important to use more complex models such as the Lattice Boltzmann Method used in this paper to describe fracture flow.

In this paper, flow through a single rock fracture is analysed through the development of a Lattice Boltzmann (LB) fluid model. Laboratory scale and in-situ testing is expensive and not always possible. Modeling the system allows the study of rock fractures and their role in bioremediation to be studied at a potentially faster pace. Although there is still a need for experimental evidence before any generalizations can be made, models allow the intelligent selection of potential experimental systems by trial and error on the computer, not in-situ.

Lattice Boltzmann Methods are types of numerical methods for solving CFD problems. Other types of CFD start with the Navier-Stokes equations, which govern the macroscopic movement of fluids, then discretize to get a solution to a system of PDEs. In the LBM model the microscopic interaction of particles on a grid and the averaging of those interactions emerge into the macroscopic continuum of a fluid. These interactions include two main steps: streaming and collision. The streaming step is a translation of particles from one node on the grid to the next. The collision step conserves momentum by redirection of particles which ‘collide’ or occupy the same node.

LBM are essentially explicit finite difference approximations of the Boltzmann equation and using a Chapman-Enskog expansion, the Navier-Stokes equations for incompressible flow can be recovered. The LBM are typically 1st order accurate in time and 2nd order accurate in space depending on the implementation of the collision term.
The Figure below compares the results from the two separate simulations. The left hand side consists of a rock fracture along the base of the model with a no-slip smooth top boundary, constant gradient outlet and parabolic inlet boundaries. The right hand side models flow through parallel plates spaced at an equivalent aperture calculated using geometric mean of the fracture data. It can be seen that the actual rock fracture compresses the velocity profile much more than that of the equivalent fracture. It is the peaks of the rock fracture that significantly change the velocity distribution, leading to an apparently smaller equivalent aperture than that found by the geometric mean which corresponds to the Cubic Law aperture. The flow distribution is clearly different from that predicted by simple parallel plates and there are areas of recirculation downstream of each fracture constriction. How this would affect a biofilm or perhaps nutrient concentration is poorly understood and the subject of future research. Since this is a complex phenomenon, it would be difficult to create a single variable that could adjust for such effects. Rather, it is important that a given system be simulated with a model of equal complexity such as the presented LBM model.

![Figure 5. On the left: Flow through a fracture. On the right: Flow through parallel plates with the geometric mean aperture equivalent to the actual aperture on the left.](image)

Performance of the LBM on the GPU is much faster, roughly an order of magnitude, than a comparable LB model running on a CPU, consistent with the findings of Tolke (2008a). It is the hope of this paper to lend insight into the various computer architectures that are available to engineers for high performance computing. It is possible for any researcher to now harness tremendous computing power. While the GPU is used for general computation in this research it is also used for real-time visualization. The model developed for this paper is well suited for simulating laminar flows through simple systems like parallel plates, and more complex system such as rock fractures. It can model internal flow dynamics that are lost to other types of flow approximation like the Cubic Law because it takes into account the complex boundaries that arise in rock fractures. Even in laminar flow, recirculation occurs, creating potentially interesting phenomenon for the interaction of biofilms in those areas leading to exiting future research into this area.
SESSION 3 USE OF LIDAR AND DIGITAL PHOTOGRAMMETRY IN ROCK ENGINEERING I

3965
Rock Mechanics Approach For The Recovery Of The Zone G Crown Pillar At The Raglan Katinniq Mine
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3976
Measurement of Surface Roughness of Rock Discontinuities
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3984
Long-Range Terrestrial Digital Photogrammetry For Discontinuity Characterization At Palabora Open-Pit Mine
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3987
GEOTECHNICAL APPLICATIONS OF LIDAR SCANNING IN TUNNELLING
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The extraction of the Katinniq Zone G at the Xstrata Nickel Raglan Mine in the Nunavik region in northern Quebec had evolved by the spring of 2008 to the point where the recovery of the crown pillar between the open pit and underground workings had to be planned. The geomechanics aspects of this recovery, planned with blastholes drilled from the bottom of the pit, were investigated numerically with a large-scale three-dimensional non-linear model. In particular, the following issues were examined: 1) the response of the crown pillar to successive blasts fired in it, as well as that of the back of Cut 1525 from where mucking was to be carried-out underground, and including the behaviour of the successive brows created by consecutive blasts in the crown pillar; 2) the behaviour of the rock mass around the main draw point on the east side of Cut 1525; and, 3) the stability of the hanging wall that was going to be created and progressively enlarged with mining.

Two types of numerical analyses were conducted. Firstly, zoned analyses were completed, whereby the rock mass was considered continuous. This approach focused on the behaviour of the rock matrix in response to the stress changes and deformations caused by mining. The zoned analyses did not highlight any major problem for the rock mass properties considered, excavation geometry examined, three faults taken into account and various zones of low RQD material incorporated in the model. In other words, no excessive deformation and/or stress changes that would have caused failure around the voids were identified with the mining approach, extraction sequence and ground support systems proposed for the recovery of the crown pillar above Zone G.

Following these zoned analyses, a series of discrete jointed un-zoned simulations were completed, whereby the discrete blocks created by the various joint sets were allowed to move along, or rotate around, their interfaces, as well as detach and free-fall into the voids. These kinematic analyses focused on the potential for the rock mass to unravel along the joint sets and major faults when large spans are developed in it. This unravelling was expected to be the most likely mode of failure given the low confining stresses acting upon the rock mass near surface. Since the likelihood of this failure mechanism was confirmed by this second round of simulations, subsequent work focused on trying to quantify the chance of it materialising. This paper focuses on this particular aspect of the work.

Two series of discrete models were constructed and used for this aspect of the project: one for the analysis of the crown pillar (and particularly the back above Cut 1525 and the brows of the excavation), and another for the analysis of the hanging wall. Both considered only limited spans along with simplified geometries, and, as mentioned, were meant to investigate the potential for kinematic structurally-controlled gravity-driven unravelling to occur with the joint set...
configuration associated with each exposed surface. The limited spans and simplified geometries were needed to keep the model run times reasonable.

The first step of the process was to derive the geometrical characteristics of the various joint sets (including their variability) in the areas of interest, i.e., in the ore material for the crown pillar model, and in the encasing Peridotite waste for the hanging wall model. This was done in a number of stages. Firstly, stereographic pairs of photographs were taken underground in both horizons in Zone G along various orientations. Detailed line mapping was then done “virtually” in the office with the ShapeMetriX3D package on 17 photographic pairs, which provided values of dip, dip direction, spacing and length for 244 joints. The determination of the joint sets (in terms of dip and dip direction) was then completed from the geometrical joint data obtained with ShapeMetriX3D. The standard deviation values around the average geometrical characteristics of the joint sets were then derived.

For each series of analyses (for the crown pillar and hanging wall simulations), a succession of unique models were created by randomly generating explicit joint patterns, based upon the average and standard deviation values of the dip, dip direction, spacing and persistence of the various sets in each geological horizon. Because standard deviations were considered, a slightly different jointing pattern was effectively constructed each time a model was run, the exact same jointing pattern never being obtained twice. This variability allowed the creation of random block geometries that are much more realistic and less prone to artificial uncontrolled unraveling than when regularly-shaped blocks are generated, as is the case when only average (i.e., constant) values of dip, dip direction and spacing are considered. The variability of the persistence of each set was also taken into account, which resulted in the creation of realistic intact rock bridges throughout the rock mass. The algorithm used to generate the joint sets in 3DEC is described in the paper. Because of the variability in the input file, a number of simulations must be completed in order to capture the range of possible behaviours. Ideally, a large number of runs should be completed to derive some statistical confidence in the results, as is done with Monte Carlo simulations, for example. In this particular case, this was not feasible. Instead, only three (3) runs were completed for each surface of interest, to broadly assess the variability of the results and whether or not more runs were subsequently needed.

When the stability of excavations is primarily controlled by unravelling along the local joint sets and large-scale faults, as opposed to stress-induced failure within the rock mass matrix, jointed analyses with rigid blocks are well-suited to examine, in a stochastic manner, the likelihood of such failure on various surfaces, as well as its possible extent. With this approach it would have ultimately been possible to estimate the probability of occurrence of problematic ground falls in various areas – for this purpose, additional runs would have been required – probably at least thirty – in order to perform statistical Monte Carlo analyses on the results.

Since modelling also provides the type and extent of failure to expect, should it materialise, it generally makes it possible to estimate the cost of such failures, which is generally in the form of production delays (or losses) and rehabilitation work (which, besides direct expenses, also causes additional delays). Multiplying this cost by the probability of failure estimated with the model would give the risk associated with the mining approach being considered. Risk is indeed best quantified in terms of dollars, in the form of a cost to be subtracted from the base case financial performance. If the risk associated with a particular mining approach is deemed too high, one option is to assess the effect on stability of additional ground support and weigh it against the cost of its implementation, to evaluate if it improves the expected risk-factored financial performance of the design. This methodology allows engineers to present risk in a manner that can be used by management in the decision making process.
Measurement of Surface Roughness of Rock Discontinuities

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1 INTRODUCTION

The behaviour of objects in contact is controlled by the geometry of the contact surfaces and the material properties of the objects. The ‘geometry’ of a surface can be considered to be comprised of two distinct components; one that may be referred to as the shape in terms such as waviness or curvature and a random component sometimes referred to as ‘unevenness’. At fine scales roughness influences the shear strength of a discontinuity while at large scales roughness, or more precisely waviness, affects both the direction in which shearing occurs and the dilation of the discontinuity during relative motion of the surfaces, assuming that the components of the rock mass do not fail.

Methods of measuring roughness in the field without physical contact with the rock surface have become available including laser ranging and 3D imaging using photogrammetry. These measurement have inherent noise and a significant issue that arises from the measurement of surface geometry is the effect that measurement noise has on the characterisation of roughness. The presence of noise in measurements adds an additional component of random variation that will increase the apparent roughness as estimated from the raw data.

Remote measurement of surface roughness has been demonstrated (Grasselli et al 2002, and others) and this work has described some methods of processing these data to deal with sensor noise. This paper addresses aspects of the measurement of surface coordinates used to estimate roughness in field situations at ranges that generally preclude the use of laboratory equipment and thus utilise laser ranging or photogrammetry techniques.

2 ESTIMATION OF THE JOINT ROUGHNESS COEFFICIENT

‘Roughness’ is effectively a description of the geometry of a surface and the roughness of the surface of a discontinuity can be parameterised by measuring the deviation of the surface from ‘smooth’ or ‘flat’. Estimates of roughness attempt to describe, in a single parameter, the surface topography of a discontinuity in a rock mass.

Roughness is an inferred parameter and the measurements that are used to characterise roughness must suit the models used to parameterise it. The measurement of the shape of a surface profile may be applied through the relationship between the peak friction angle, the compressive strength of a discontinuity (JCS) and the joint roughness coefficient (JRC) where:

\[
\phi_{\text{peak}} = JRC \cdot \log_{10} \left( \frac{JCS}{\sigma_n} \right) + \phi_{\text{residual}}
\]

The JRC proposed by Maertz is based on an empirical correlation of the form:
where $c$ is an empirical constant of the order of 400 and $R_p$ is the roughness profile index which is defined as the ratio of the true profile length to the length of the projection of the profile on the plane that is the plane of best fit to the 3D points.

Whereas the JRC proposed by Tse and Cruden (1979) is the root mean square (RMS) estimate of the local slopes of the profile defined over the intervals between measured data points. It is expressed as:

$$JRC = c \cdot (R_p - 1)$$

It is suggested that an extension of the Tse and Cruden JRC to two dimensions may be considered.

3 MEASUREMENT BANDWIDTH AND NOISE

The discussion presented highlights the issue of the scale at which measurements of roughness are to be made. The spacing of the measurement rods of the coordinates of points on the surface of a rock joint defines the smallest feature that can be measured and thus the scale at which roughness can be measured.

4 MEASUREMENT OF ROUGHNESS / MEASUREMENT USING 3D POINT CLOUDS

The paper discusses remote measurement of surface roughness using scanning laser ranging systems and photogrammetry and provides data from a calibration plane mapped using a scanning laser system. The data acquired validated the performance of the system as specified by the manufacturer and provides a baseline for assign the utility of laser data for measuring surface roughness.

5 REDUCTION IN NOISE BY DATA PROCESSING

It has been shown that, without smoothing, the estimation of the JRC using either the Maertz or Tse-Cruden formulae with linear profile data can produce quite large variations in the estimated JRC. The results reported by and the laser data presented here demonstrate that noise reduction or removal is critical for the estimation of roughness if the sensor noise is significant relative to the scale of the surface features being measured.

6 CONCLUSION

Estimation of roughness requires acquisition of detailed spatial data that defines the geometry of a surface combined with careful application of the data to infer roughness. Published data and the results reported here indicate controlled experiments conducted on the same surfaces with different sensors are required to compare the effectiveness of these sensor systems for the measurement of roughness. In addition the effects of various data smoothing process must be quantified and their suitability for use in the estimation of roughness validated.

7 REFERENCES


Terrestrial digital photogrammetry (TDP) for discontinuity characterization has been applied predominantly at close-range, on individual benches or road cuts. Mapping large open pit rock slopes at medium- to long-range represents a logical and necessary extension to this work. Large rock slopes could represent entire mountain cliffs or large open-pit slopes from several benches to entire pit walls. Using TDP on large rock slopes can address obvious issues related to accessibility and rock fall hazard. In addition, it allows accurate measurement of the properties of large structures, potentially affecting slope stability.

The current paper summarizes the challenges and presents preliminary results on the application of long-range TDP for discontinuity characterization at the Palabora open-pit mine, South Africa. Photographs were taken across the pit diameter from a distance of 1600m. Complete sets of photographs, covering the entire pit, were obtained using \( f = 20 \text{mm, 55mm, 100mm and 200mm lenses} \) (Fig. 1) and partial coverage of the walls were achieved using a \( f = 50 \text{mm and 400mm lens} \). Multi-scale discontinuity characterization is being achieved using these stereomodels and the results at selected locations will be presented.

Figure 1. \( f = 20 \text{mm} \) stereomodel of the entire open pit, providing a general view of the mine. The diameter of the pit is approximately 1600m.

The long-range photogrammetric survey undertaken represents an extension of common close-range TDP applications. This paper documents the results of our work and details of the methodology. The experience gained in addressing several practical issues will be beneficial for future applications of the technique. During the fieldwork and subsequent stereomodel generation, practical difficulties encountered included registration of stereomodels with a limited number of control points, due to the lack of optimal and accessible targets; lighting issues and non-optimal weather conditions necessitating careful planning of fieldwork according to the position.
of the sun; management of large sets of photographs, providing difficulties to achieve resection and successful bundle adjustment.

Concerning discontinuity characterization, the following observations are noted based on the preliminary results of this research. Characterization at the pit wall scale using the f=55mm lens has proven useful, since it has enabled mapping to be conducted relatively quickly at reasonable detail, providing an initial view of the structure. Clearly defined, persistent structures have been mapped at this resolution and compare well to structures mapped on other focal length stereomodels. The intention is to extend the use of f=400mm imagery to map key areas in an attempt to improve the detail of the structure captured.

A current limitation of the photogrammetry-specific software for f = 400m mapping is the number of models that can be stored on screen at any one time. On high resolution stereomodels, this increases the potential for censoring of persistent planes, making it harder to establish a true trace length. Some initial work using the Maptek Vulcan code has been carried out to overcome this issue, the use of this code allowing more images to be loaded on-screen at any one time while the operator continues delineating persistent planar structures. (This limitation is considered transient and related to hardware/software issues, which will undoubtedly be alleviated in the near future.)

The results of this research show that long-range TDP is able to accurately quantify discontinuity orientation, persistence and intensity. These three parameters are the main input required for the generation of discrete fracture network (DFN) models. In this preliminary application at the Palabora site, it is shown that discontinuity persistence and fracture intensity are highly dependent on the photogrammetric observation scale (Fig. 2).

Work to date emphasizes that it is critical that geologists and geological engineers decide at the planning stage on appropriate resolutions for the specific purpose of their application. If only large structures need to be characterized, low ground resolution stereomodels may be sufficient, while if a more detailed characterization of rock mass fracturing is required, higher ground resolution must be achieved.

Figure 2. Equivalent trace length distributions of discontinuities mapped on the north wall on both f=100mm and f=400mm stereomodels.
Geotechnical applications of lidar scanning in tunnelling

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1 INTRODUCTION

Safety, long term stability and quality control in modern tunnelling operations all demand the acquisition of geotechnical information as well as excavation and support data as the tunnel face progresses. Operational constraints in tunnel construction, however, demand the implementation of fast and effective technology to achieve this goal. The use of high definition 3-dimensional laser scanning (lidar) in a tunnel environment demonstrates great potential for this.

2 GEOTECHNICAL APPLICATIONS IN DRILL AND BLAST TUNNELS

Researchers from Queen's University, Kingston, have demonstrated the practical employment of portable lidar within the normal excavation cycle at a drill and blast tunnelling operation near Oslo, Norway. In addition, the scanning other unlined tunnels sites near Oslo was conducted for further geotechnical investigation. The work presented is the product of a collaborative project involving Queen’s University, Kingston and the Norwegian Geotechnical Institute (Oslo). The use of these tunnel lidar data sets shows much potential for geotechnical applications, both for operational/contractual information as well as geo-structural data. A summary of the applications discussed in the paper as well as the parties to most directly benefit is given below:

<table>
<thead>
<tr>
<th>Who's interested</th>
<th>Contractor/Tunnel engineer</th>
<th>Geological engineer/Geologist</th>
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<td>Application</td>
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<td>As-built tunnel model</td>
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<td>Structural discontinuity geometry</td>
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<td>Contoured shotcrete thickness</td>
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<td>As-built bolt spacing</td>
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<td>Surface characterization</td>
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<tr>
<td>Potential leakage mapping</td>
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2.1 Data collection in a operational drill and blast setting

The authors used a stationary tripod setup (Figure 1) for tunnel investigations.

Figure 1. Lidar system with tripod set-up at 10 m diameter Sandvika tunnel near Oslo, Norway.
During this demonstration phase, three headings (two 10 m diameter and one 15 m diameter) were scanned at multiple face positions. Set-up, scanning and take down could be completed in 5 minutes for each round. In this way, scanning did not disrupt excavation workflow. Scans were performed with the Leica Geosystems HDS6000, a phase-based scanner with a maximum range of 79 m, a 360° horizontal field of view and a 310° vertical field of view.

2.2 Advantages of geotechnical analysis of lidar data

The primary advantage of lidar scanning is the ability to obtain permanent digital rockmass and support installment documentation that can be interpreted in office and reinterpreted by other specialists later if required. The alignment of scans allows for simultaneous interpretation of rock that was once exposed and rock/lining that is currently exposed. An example of three aligned scans with visible rock structure is found below (Fig. 2). This alignment of subsequent rounds can be used to create more extensive rockmass models, enhancing the ability to identify critical discontinuity features. The results of structural discontinuity analysis with lidar data were found to be very comparable to the results of traditional mapping, except that they provide a larger quantity of measurements and include random features that are not noted otherwise. The alignment of successive scans also allows for the characterization of linear features in the face as the excavation advances.

Figure 2. Alignment of three 5 m rounds scanned in an advancing 10 m diameter drill and blast tunnel.

Figure 3. Structural data (joint orientations) obtained from an aligned lidar scan.

The authors find that lidar scanning in operational tunnel environments shows great potential as a tool for onsite contractors and supervising engineers. A lot of information that is otherwise lost as the excavation advances can be retrieved and used to improve the safety, stability as well as the precision and rate of advancement. Various successful examples of tunnel lidar data interpretation are presented in the paper.
SESSION 4  CONVENTIONAL AND TBM TUNNELLING

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Hatch Energy, Niagara Falls, ON., Canada
Dr. Mark S. Diederichs, PhD., PEng.
Queen's University, Kingston, ON., Canada

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Golder Associates Ltd., Sudbury, ON, Canada
P.K. Kaiser, P.Eng., Ph.D.
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Seedsman Geotechnics Pty Ltd, Mt Kembla, Australia

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Analytical Solution For A Deep Tunnel Excavated In A Porous Elasto-Plastic Material Considering The Effects Of Seepage Forces
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EngSolutions, Inc., Ft. Lauderdale, Florida, USA
Tunnelling in horizontally laminated ground

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Parallel laminations within a rock mass create anisotropic ground conditions, whether the laminations are caused by sedimentary bedding, joints or a tectonic fabric. Anisotropy is both modulus and stress dependent (Chappell 1990) and therefore not all rock masses with parallel structure or fabric will exhibit the same degree of anisotropic behavior.

The current Niagara Tunnel Project, in the Canadian city of Niagara Falls, Ontario, has descended down through the entire stratigraphy of the Niagara Escarpment, passed underneath the St. David’s Buried Gorge and is now making its ascent back to surface. This project provides the back drop for a larger study of rock mass heterogeneity and the role of anisotropic ground response to large scale excavations. Further information on this project can be found in Perras and Diederichs (2007).

The state of practice in excavation design is to model the rock mass as an isotropic material, with the exception of specific structurally controlled instability, reducing the intact strength parameters to account for heterogeneities within the rock mass. Using both UDEC, by Itasca, and Phase2, by Rocscience, software models were constructed with horizontal laminations with spacing ranging from 0.16 to 16m for a 16m diameter tunnel with 150m of ground cover and a stress ratio of $K_0 = 3$. The laminations properties were fixed as cohesion = 0.14 MPa, tension = 0.3 MPa, friction = 25°, normal stiffness = 25,000 MPa/m and shear stiffness = 2500 MPa/m.

Significant differences in the yield zone size and shape and crown deflections were found to exist between the non-jointed and the jointed models. UDEC and Phase2 gave similar yield zone sizes, as seen in figure 1a, for the jointed models. The distinct element model (UDEC) allows block detachment and new contact creation resulting in larger crown deflections then the finite element model (Phase2).

The crown deflection behaviour, as the lamination thickness decreases, varies from the non-jointed model and from the trend predicted by the laminated voussoir model presented by Diederichs and Kaiser (1999). As the lamination thickness decreased five distinct excavation behavior zones were found to exist as seen in the thickness vs. crown deflection graph in figure 1b. These five zones are controlled by the stress flowing around the excavation and the anisotropic ground movement. The snap through limit for the voussoir model does not apply to the jointed models because of the circular excavation geometry. The voussoir model was intended for a rectangular opening.

By modeling the laminations directly an anisotropic rock mass was created which has a distinct impact on the size and shape of the yield zone and the magnitude of the crown deflections over the state of practice of modeling the rock mass isotropically. These distinct behavioral differences will have an impact on the design process for excavations in anisotropic rock masses. Further study is necessary to determine the range of properties for which this behavior applies.

ACKNOWLEDGEMENTS

The authors would like to express their appreciation to Ontario Power Generation and members of Hatch Mott MacDonald and Hatch Acres for the permission to publish the information used in this paper and for their input in reviewing the content.
Figure 1. Variation in yield zone size and shape between non-jointed (Phase2) models and jointed (UDEC & Phase2) models (a) and crown deflections (Phase2) (b) for decreasing lamination thickness

References
Influence of Rock Mass Anisotropy on Tunnel Stability

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1 INTRODUCTION

Stress induced, gravity-assisted failure processes typically dominate long before stress levels approach the strength of the intact rock (at approximately $\sigma_{\text{max}} > 0.4\text{UCS}$ where $\sigma_{\text{max}} = 3\sigma_1 - \sigma_3$ and UCS is the Unconfined Compressive Strength of the intact rock (Martin et al. 1999)). Under these conditions, discontinuities typically become clamped and the failure process becomes brittle. This brittle failure process is dominated by new stress induced fractures growing sub-parallel to the excavation boundary.

While it is possible to identify areas with the potential for brittle failure, for example, using the general assumption that brittle failure will occur in areas at right angles to the major principal stress (Hoek et al., 1995), when the threshold $\sigma_{\text{max}} > 0.4\text{UCS}$ is reached in areas around the tunnel boundary, or elastic modelling and the Hoek-Brown brittle parameters proposed by Martin et al., (1999) (i.e. $m \approx 0$ and $s \approx 0.11$), amongst others, recent observations discussed by Everitt & Lajtai (2004), and Kaiser (2005; 2006) suggest that brittle failure processes are enhanced when structural features such as joints, weakness zones, bedding planes or foliations are preferentially orientated around the excavation boundary creating stress raisers which in turn facilitate the stress-driven rock mass disintegration process (e.g., as illustrated by Everitt & Lajtai (2004) on Figure 1b).

Such weaknesses induce stress heterogeneities that thus promote tensile-spalling failure processes (Diederichs 2000). The observations suggest that stress induced damage can occur “prematurely” depending on the orientation of micro and macro structures in situations where the known stress conditions should not yet initiate brittle failure processes. In general, when structural features (micro and/or macro), especially foliations, are present in a rock mass, the strength is effectively reduced, largely due to tensile strength heterogeneity in areas depending on the orientation of the structural features with respect to the excavation boundary. As a result, stress induced damage could occur in areas not normally considered due to the influence of anisotropy (Figure 1).

2 CONCLUSIONS

The influence of structural orientation was assessed for a 10 m diameter circular tunnel. The results compare well to observations in the field which suggest that:

- The location and extent of stress induced damage is not only a function of stress magnitude relative to intact rock strength and stress orientation but is also significantly affected by the orientation of structures and weakness planes such as systematic foliation near the excavation boundary.
- The presence of unfavorably oriented systematic structure around the boundary of an excavation can lead to stress induced breakouts to occur in areas that would not be predicted using current stress induced failure assessments.
A span to spacing ratio of >10 is required for systematic structural layering to influence stress driven rock mass degradation processes.

Based on the parametric analyses, the influence of structural orientation around the boundary of the tunnel was determined to typically influence the rock within a 100° window, 50° on either side of a direction normal to the parallel structure orientation (i.e. β angles 0° to 50°). This data was then used to propose an Orientation Reduction Factor (ORF) similar to the Stress Reduction Factor (SRF) after Barton et al. (1974). The ORF concept was tested by Bewick (2008) on three case examples found in the literature (results not shown here) and found to produce reasonable results.

Figure 1: a) URL test tunnel reproduced after Martin et al., (1999) showing stress as dominating control on location of stress induced damage. b) Interpretation by Everitt & Lajtai (2004) showing foliation orientation as dominating control on location of stress induced damage. c) & d) breakouts observed in the tunnel back in Leventina-Gneiss along foliation planes with a k_o=0.7 where failure should have been in the walls if stress levels were sufficient (Kaiser, 2006).

3 ACKNOWLEDGMENTS

The main Author would like to thank: MIRARCO; the mining group at Golder Associates Ltd.; and Peter Kaiser for his time and patience. NSERC also deserves mention for the financial support provided through Dr. Kaiser.
An update of conditions in the Donkin-Morien tunnels

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In the early 1980s, 2 by 7.5m diameter TBM tunnels were driven from the Donkin-Morien peninsula in Nova Scotia to access the Harbour Seam under the North Atlantic Ocean. The ground support utilised the steel rings used as thrust rings by the TBM. The rock loading onto the rings was estimated by the application of finite element analyses utilising a presumed stress field and Hoek Brown strength parameters derived from widely spaced exploration drill holes. The presumed worst case loading condition resulted in a maximum load on the steel sets equivalent to 3.0m of loosened rock. The tunnel excavation was successful and the tunnels were in a serviceable condition in 1989 when the mining project was abandoned. The tunnels were allowed to flood in 1992.

The tunnels were used as a case study for the development of the concepts of brittle rock failure. Significantly, the reported loosening zones were higher than predicted by the standard Hoek Brown parameters that had been used in the design. Furthermore, the reports indicated a reduction in the strength of one of the key units associated with the worst-case loading condition.

In 2007, the tunnels were pumped out as part of a feasibility study into the reintroduction of longwall mining into the coalfield. Two major falls were encountered near coal seams. In other areas the arches in the crown have severely deformed, and there has been extensive collapse of the sides above the spring line.

There is a good match between observations of the collapse zones in the roof and sides of the tunnel and simple elastic analyses using the concepts of brittle failure together with consideration of transverse anisotropy. Overloading of the steel rings is predicted for material with a UCS less than 30 MPa, a spalling limit of less than 5, and a shear modulus less than 250 MPa.

This simple back-analysis required the spalling limit for these relatively low strength rocks to be less than about 5. Similar values have been required when applying the brittle criterion to coal measure rocks in Australian longwall mines.
Sketch of the fall at the Emery Sea

Typical condition of the tunnels in type V mudstones.

Development of concentric fractures, (b) extreme example of corrosion.
Pavoncelli tunnel case study

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1 BACKGROUND, SCOPE AND OBJECTIVES

The paper deals with the seismic design of tunnels. The case study of the Pavoncelli tunnel, which belongs to the Acquedotto Pugliese and crosses the Apennines through a highly seismic region in South of Italy, is considered. The construction of the Pavoncelli tunnel (10 km long) began in 1904 and was completed in 1915. Among the severe difficulties incurred during construction, the presence of the highly squeezing/swelling Varicolori Clay-shales is to be mentioned, as they caused large convergences and high stresses in the lining. Based on what is known (Viggiani, 2001), the tunnel originally had a horse-shoe cross section, while a circular cross section was adopted only rarely when crossing the Varicolori Clay-shales. Due to swelling, the tunnel experienced, immediately after completion, heavy damages with longitudinal cracks and invert heave. Due to this, starting from 1922, the tunnel needed be reinforced by placing a stronger lining. The tunnel experienced severe damages also during the 1980 Irpino-Lucano earthquake, which were localised in the areas where swelling of the local Clay-shales Formation occurred after construction. A by-pass tunnel is being designed and will be constructed close to the existing one. Therefore, the interest is to gain useful information on the tunnel behaviour during an earthquake by back analysing the existing tunnel behaviour. This will allow to improve the current design, also accounting for the swelling phenomenon that may occur during and after construction.

2 NUMERICAL ANALYSES, RESULTS AND CONCLUSIONS

The tunnel cross section of interest in this paper is that adopted after its reconstruction in 1931, at chainage 4550 m, located at 400 m depth, where an horse-shoe shaped tunnel with lining thickness between 1.15÷1.60 m at the crown and 0.6-1.0 m at the invert is present. This section was excavated in Varicolori Clay-shales and is located in the area where severe damages occurred in the lining, during the 1980 Irpino-Lucano earthquake: mostly cross fractures and invert heave.

The numerical analyses were conducted with the Finite Difference Method (FDM) in plane strain conditions with the aim to reproduce the main phenomena experienced by the tunnel during construction, during its service life, in long term conditions (including swelling of the ground), and due the earthquake.

The analyses are conducted with constant gravity and in effective stress conditions. The initial stress state for the tunnel cross section of interest is defined as follows: vertical total stress = 6.7 MPa, horizontal total stress = 5.4 MPa, pore pressure = 200 kPa. The geotechnical parameters of the Varicolori Clay-shales are assumed on the basis of laboratory tests results given in Cotecchia et al. (1992). The tunnel masonry lining is assumed to follow a linear elastic behaviour. A no slip condition is assumed between the lining and the ground.

For the swelling behaviour, reference is made to the method described in Barla (2008), where the swelling potential of the ground is directly related to its tendency to develop negative excess pore pressure during excavation by defining a functional relationship between the volumetric strain (\(\varepsilon_{vol}\)) and the excess pore pressure (\(\Delta u\)): \(\varepsilon_{vol} = f(\Delta u)\). The Storno surface record of the 23 November 1980 Irpino-Lucano earthquake is considered. The record of the horizontal acceleration is applied to the bottom of the FDM grid, reduced by 50%. This is considered acceptable as the model is based on bedrock.
The analysis was carried out considering the following steps:
1) Assessment of the initial state of stress in the ground by applying gravity loading and initial stress conditions.
2) Excavation of the tunnel full section with a relaxation factor of 0.9 in undrained conditions.
3) Placement of the lining and full relaxation in undrained conditions.
4) Simulation of the swelling process.
5) Earthquake simulation by running the seismic analysis in undrained conditions.

The analyses allowed to reproduce the lining response during the stages 3, 4 and 5 above (i.e. after excavation, after swelling, after earthquake) in a reasonable way, depicting stress increments in the lining during swelling and dynamic shaking.

Higher hoop stresses occur at the springline and at the crown where the lining is thicker. To evaluate the damaged sections in the lining, the stress state at the crown, invert and springlines is compared to the strength domain in Figure 1.

Figure 1. Strength domain and stress state in the lining and photo of typical damages occurred.

Three different strength domains are plotted, each one corresponding to a different lining thickness. Each dot in the diagram represents the stress state (axial force and bending moment) for a single element of the lining. It is clearly shown that the lining is within the elastic domain during the first four stages, getting near to the failure envelope after swelling, particularly at the invert. During the dynamic analysis the stress components cause the lining to go beyond the elastic domain at the invert. The critical sections are at the invert and at the lower portion of the springlines, highly stressed already after construction and subsequent swelling of the Varicolore Clay-shales Formation. The overall behaviour appears to be in good agreement with field observations after the 23rd November 1980 Irpino-Lucano earthquake, as shown in the same Figure 1. The information gained so far is of interest to update the seismic design of the by-pass tunnel to be constructed close to the existing one in the near future.

3 REFERENCES

Analytical solution for a deep tunnel excavated in a porous elasto-plastic material considering the effects of seepage forces

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1 INTRODUCTION

Tunneling in water bearing ground affects the hydraulic equilibrium of the surrounding ground leading to seepage into the tunnel. The seepage forces generated in the ground by the movement of water towards the tunnel can have a significant effect on the behavior of the opening. Groundwater inflow can cause severe instability problems, including total ground collapse and flooding into the tunnel.

Numerous case histories of tunnel failures during construction, at sections with high water pressures, have been reported. Failure often consists of a sudden “mud inrush” flowing violently into the opening, dragging equipment, facilities and sometimes even workers, and completely invading long stretches of tunnel. Even in less striking cases, seepage forces can have a strong effect on the ground support requirements and on the structural requirements of the support system. Nevertheless, no adequate analytical solutions that consider the effect of seepage forces are currently available for analyzing the inelastic ground-support interaction to dimension tunnel support elements, and for properly assessing cases of “flowing ground”.

In this paper a proper elasto-plastic solution is presented, for a deep cylindrical tunnel excavated in a Mohr-Coulomb perfectly plastic material under the water table, considering seepage forces. In addition to establishing the radius of the plastic zone, radial convergence, the stress and displacement fields around the tunnel, and identifying the condition for development of flowing ground condition at the tunnel face, a procedure for hydromechanic ground-support interaction is proposed.

2 METHOD OF ANALYSIS

2.1 Basic Assumptions

The main assumptions in the method of analysis are as follows:

- Tunnel is circular
- The tunnel depth is much greater than the tunnel radius. Hence, changes in the values of in-situ effective stresses and water pressures with depth are not significant for such a deep tunnel.
- In situ effective stress field is hydrostatic (i.e. equal stresses in all directions)
- Surrounding ground is isotropic and homogeneous. Failure is not controlled by major structural discontinuities.
- Ground obeys the principle of effective stress.
- Ground is a porous permeable media.
- The type of analysis is drained and in terms of effective stresses.
- Support response is elastic-brittle-perfectly plastic.
- Support is modeled as a uniform effective pressure acting around the entire tunnel wall.

The method of analysis is applicable to tunneling in water bearing ground, subjected to high water pressures, including weak rocks (such as friable rocks), highly fractured rocks, crushed rocks (such as found in fault zones), as well as general soil-like materials.
2.2 Elasto-Plastic Solution

Based on fundamental relations of equilibrium, compatibility and continuity, the effective stress, water pressures and wall displacements around the tunnel, are determined for in terms of the effective support pressure exerted by the lining and the water pressure at the tunnel wall-lining interface.

![Graph showing effective stresses, water pressures and displacements around a tunnel.]

Figure 1. Effective stresses, water pressures and displacements around a tunnel.

3 HYDROMECHANICAL GROUND-SUPPORT INTERACTION

A ground-interaction analysis is used to determine the pressures acting on the lining, including the hydraulic pore water pressure $P_{wi}$ and the mechanical effective pressure $P'_{i}$. The analysis is carried out for a time $t$, at which the tunnel support is installed. The support system considered includes a concrete lining and (or) rockbolts.

![Graphs showing hydraulic and mechanic ground-support interactions.]

Figure 2. (a) Hydraulic ground-support interaction (b) mechanic ground-support interaction

Equilibrium is achieved if the combined support characteristic curve intersects the ground response curve before a ‘flowing ground’ condition is developed and either of these curves has progressed too far.
SESSION 5  GEOPHYSICS IN ROCK ENGINEERING

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Nick Hudyma
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F. Rezanezhad
Cold Regions Research Centre, Wilfrid Laurier University, Waterloo, Canada
S.H. Cho,
Department of Mineral Resources & Energy Engin., Chonbuk Nat.Univ., south Korea

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J.K. Van Beek and W.M. Roggenthen
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M. Magliocco and S.D. Glaser
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School of Geography & Earth Sciences, McMaster University, Toronto, Canada
M. H. B. Nasseri & D. S. Collins
Lassonde Institute, University of Toronto, Toronto, Canada

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GroundProbe
Dave Bates, P.G.
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Characterizing internal macropores using cross-specimen acoustic tomography: initial two dimensional results

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1 INTRODUCTION

The engineering properties of a geologic material are greatly affected by the presence of macropores. Previous research using analog brittle specimens, numerical modeling and rock specimens has demonstrated that the size, location, and proximity of macropores influence both the strength and stiffness of the specimens. These studies have shown that at the same level of macroporosity there can be large differences in strength and stiffness. As such, determining the size, shape, and location of macropores prior to laboratory testing is important for evaluating the laboratory test results.

There are a number of common imaging techniques to characterize the internal structure(s) of geological specimens. Many of these techniques have been developed for the medical industry and require expensive equipment and highly skilled personnel for operation and interpretation. The goal of this work is to develop a simple yet robust non-destructive method to characterize macropores within laboratory specimens prior to destructive testing. The technique that is being developed has been named cross-specimen acoustic tomography (CSAT). This method is a specific application of elastic wave tomography that uses high frequency signals as the source for waves. Highlights of the work to date are presented below.

2 METHODOLOGY

2.1 Specimens

A set of three cylindrical plaster of Paris specimens 15.2 cm in diameter and 30.5 cm in length were produced. One of the specimens was solid plaster and the other two specimens contained Styrofoam spheres which represented the macropores. One of the specimens contained a single 5.1 cm diameter Styrofoam sphere, and the other contained two 7.6 cm diameter spheres separated by a vertical distance of approximately 5.1 cm. A cylindrical coordinate system was placed on the specimens to provide reference points for the data collection.

2.2 Data collection and processing

A set of eight piezoelectric sensors that are able to send and receive high frequency (f=1.26 kHz) acoustic signals were used. The sensors are placed equidistant from each other in a predetermined horizontal plane around the cylindrical specimen. One of the sensors sends the high frequency acoustic wave that travels through the specimen and is captured by the remaining seven sensors. Each sensor acts as both transmitter and receiver. The sensors are then rotated horizontally 22.5° (one-half turn) clockwise and the data collection is repeated. The set of sensors is then moved to a different horizontal plane for additional data collection.

Once all of the data has been collected, it is put into the proper form and it processed through a tomographic inversion. The inversion process produces a two dimensional velocity model or tomogram of the horizontal plane.
3 RESULTS

3.1 Specimen containing a single macropore

The figure shown below contains 5 tomograms of the specimen containing a single macropore. The locations of the macropore are determined visually by locating zones of relatively low velocities within the specimen. Based on testing of the solid specimen, the macropore should be located within a low velocity zone at velocities less than 2.18 m/ms (86 in/ms). Those velocities correspond to a color scale of light green tending to blue.

![Tomograms of the specimen](image)

Figure 1. Two dimensional tomograms of the one macropore specimen. The dashed lines in the tomograms indicate the location of the macropore.

4 CONCLUSIONS

This preliminary study using cross-specimen acoustic tomography (CSAT) successfully demonstrated that acoustic wave tomography can be an effective non-destructive technique for determining the location of macropores. Additional investigations are currently underway to improve the technique and fully develop it into a true three dimensional imaging technique. Improvements will focus on additional techniques for inverting the travel time data such as curved raypath approximations, wavefront approximations, and the application of velocity damping functions.
Application of 3D X-ray CT scanning techniques to evaluate fracture damage zone in anisotropic granitic rock

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Abstract

Application of linear elastic fracture mechanics to understand the stress and strain at the vicinity of a crack tip continuous to be a very active and challenging field of research (Irwin 1948, Barenblatt 1962, Hoagland et al., 1973). It is postulated that a “process zone” is generated in the vicinity of a propagating macrocrack tip and the notions of critical stress intensity factor, $K_{IC}$ or effective fracture energy are a cornerstone of fracture mechanics. The damage zone which is the final form of fracture process zone (FPZ) is used to describe the accumulated deformation surrounding the main crack as a whole. (Chester and Logan, 1986). The geometry of the process zone of a tensile crack was first calculated by Evans et al. (1977). The width of FPZ is considered a material property and it was found to increase with increasing grain size (Ouchterloney, 1980). No attempt has been made so far to investigate the effect of preferably oriented pre-existing microstructural fabrics on the surface area of fracture path and the dimension/structure of damaged zone formed under mode I in granite. In this study we used 3-D X-ray micro CT scanning techniques to correlate between the physical properties in a damaged zone such as test cracks passage area, crack porosity (cracked volume/total volume), crack density (number of cracks per mm$^2$), and contribution of individual mineral grains for two different paths, perpendicular (case 1) and parallel (case 2) to pre-existing preferably oriented microstructural fabric and its correlation with measured respective fracture toughness in Barre granite. An attempt is made to relate the structural differences observed within the damage zones for these two cases to the nature of interaction between the test crack and the pre-existing microstructural fabric’s orientation.

X-ray computed tomography was performed on a small volume ($3.8 \times 7 \times 6.3$ mm$^3$) of Barre granite containing the test crack and associated induced fractures for both situations when the test crack was forced to propagate parallel (case 1) and perpendicular (case 2) to the pre-existing micro-fabric orientation. Micro CT imaging provides detailed information on microscopic rock structure, and is capable of evaluating the 3D induced fracture geometry with high resolution. The spatial resolution of the CT system was sufficiently high, at 7 μm resolution, and hence the high attenuation contrast between induced fractures and rock material allows direct imaging of individual fractures and networks. X-ray CT scanning images were binarized to “solid” (rock materials) and void space (induced fractures associated with the test crack) by means of a neighborhood-based standard deviation thresholding algorithm. The 3D fractures space surface area and volume were determined by a cluster labeling algorithm and verified by a “3D Objects Counter” algorithm in ImageJ. This algorithm provide an accurate segmentation into binary images that allows for the frequency (i.e. number of fractures) measurements as well as some morphology characteristics (volume and surface area of inter-particle fractures, the centre of mass and the centre of intensity) for each separable fracture.

The damage zone for the case 1 is characterized with almost twice newly generated crack surface area than for case 2. The measured induced crack porosity for the case 1 is more than ten times that of case 2 (Table 1). This conclusion further justifies the reason for the fracture toughness in case 1 being almost twice of that of case 2. It is also concluded that the nature of interaction between the test crack and pre-
existing microstructural fabric’s orientation dictates the type and structure of the branched and the subsidiary parallel cracks in the damages zone in case 1 and 2 respectively. As a result of such an interaction many long branches originate from the main crack, making the damage zone more complex and wider in size in case 1 scenario. Whereas, the damage zone in case 2 scenario is characterized with subsidiary cracks that run more or less parallel to the main crack and are shorter in length. These observations may be applied to the field scale of various size in which the stress induced tensile forces promote fracture propagating quasi-statically. Rift zones, sites for dyke intrusion and supporting roof of a mine under tension are good examples. Depending on the nature of interaction of propagating tensile fractures with pre-existing oriented sets of geological structures, a rougher crack profile and more damaged rock masses can be formed. This will eventually affect the post strength, post-frictional and hydro-geological and flow regime properties of resultant rock mass. The present study, linked with observations in field scale will help in better understanding of fracture propagation/interaction and its application to enhance the natural stability of underground openings, control of rock fragmentation and damage in blasting, prediction of transport properties with divers flow regimes in rocks and enhancement of oil reservoirs during hydraulic fracturing.

Table 1: Summary of 3-D analysis of micro CT images obtained from the test crack and the damage zones for both cases. Measured fracture toughness for both cases is included.

<table>
<thead>
<tr>
<th>Propagation With Respect to Micro-fabric Orientation</th>
<th>Induced crack porosity, damage zone only</th>
<th>Number of cracks, damage zone only</th>
<th>Volumetric crack density, in damage zone only</th>
<th>Total crack passage area test crack &amp; damage zone</th>
<th>KIc</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1: Perpendicular</td>
<td>0.014</td>
<td>223</td>
<td>1.31</td>
<td>28.32</td>
<td>1.9</td>
</tr>
<tr>
<td>Case 2: Parallel</td>
<td>0.001</td>
<td>24</td>
<td>0.14</td>
<td>14.65</td>
<td>1.1</td>
</tr>
</tbody>
</table>

Fig 1. 3-D micro CT image on the left side shows the propagation of the test crack perpendicular to pre-existing micro-fabric orientation (case 1) and the 3-D image on the right shows parallel propagation (case 2) of test crack with respect to micro-fabric orientation.
Rock mechanics and subsurface imaging at DUSEL, Homestake mine

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Operating from 1876 to 2001 in the northern Black Hills of South Dakota, the Homestake gold mine (Fig. 1a) extracted over 41,000,000 oz of gold from depths reaching 2.4 km. In its 125 year history 560 km of drift and several hundred stopes were opened (Caddey et al., 1991). The closure in 2001 provided an opportunity to use the mine as a laboratory and has led to the conversion of the mine into a dedicated underground science and engineering laboratory. The Sanford Laboratory at Homestake currently provides the infrastructure for scientific experiments in the fields of astrophysics, geology, geophysics, microbiology, and hydrology. The development of this laboratory will involve the construction of very large cavities, in some cases requiring spans of 20 m at 4850 ft and 7400 ft below the surface. In addition, extremely large cavities with spans of 50 m and heights of 50 m are also being considered at the 4850 ft level. These excavations will provide the location for large arrays of photomultiplier tubes used to detect the interaction of neutrinos with the liquids filling the cavities.

Our team of geologists and engineers is carrying out the first scientific project awarded to the site – Towards a Transparent Earth. Taking advantage of existing, open, boreholes we are installing a seismic observatory consisting of an array of instruments sensitive over the frequency bandwidth of 0.2 - 1 KHz. Down-hole 3D accelerometer packages, platform mounted dual-axis bubble-type tiltmeters, and hydrostatic water level systems have been installed on the 2000 ft level of the mine. This instrumentation is connected to a data acquisition system that records data and allows adjustments to be made both locally and in real-time over internet connections. The currently installed tilt sensors allow investigation of the solid earth deformation as a response to tidal potential. The accelerometers are being used to locate and characterize events associated with mine dewatering and rehabilitation.

The 3D accelerometer packages have recorded several hundred events since installation in April, 2008 (Fig. 1b). Initial evaluation of data frequency content from a bubble-type tiltmeter displays high signal to noise ratios in the semi-diurnal band with decreasing resolution in the lower frequencies due to discontinuous and short recording intervals (maximum 48 days continuous data acquisition), instrument drift, and thermal variations at the recording site (Fig. 1c). Future seismic installations will concentrate on deeper levels of the laboratory and will assist in monitoring the deformation effects of constructing the large cavities associated with the physics experiments. Results of data frequency analysis relate primary signal constituents in the semi-diurnal range to some values detailed by Wahr (1995). Though longer period tidal constituents are not currently evident in the data, longer monitoring time of both the bubble-type tiltmeters and the hydrostatic water level system will likely result in a higher signal to noise ratio in the extremely low frequency range.
Figure 1. a) Location of the Sanford Lab at Homestake within the Black Hills of South Dakota. b) Seismic data from the tri-axial seismometer on the 2000 foot level (610 m) below surface, Homestake/Sanford laboratory, recorded on 12 December 2008 at 2:52 pm local (18:52 UTC) showing an event with radial distance of approximately 1200 m. y(0) is approximately vertical. c) Tidal constituents from the diurnal, 1.16e-5 Hz, and semi-diurnal, 2.24e-5 Hz, from FFT of de-trended windowed data collected from one axis of the resistive-type bubble tiltmeter.

References

Multiparameter petrophysical characterization of an orebody: an exploration case history

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ABSTRACT
We present results from petrophysical studies to characterize a Zn-Pb-Ag deposit in Nash Creek, New Brunswick. The measurements are part of an integrated study that includes geological and geophysical data acquired on the property. In order to successfully characterize the orebody, an understanding of the relevant physical parameters and their change with vertical and lateral variations of the target lithology is necessary. The quality of the estimated 3D variation in the physical properties is supported by the presence of several boreholes that randomly span the survey region. Density was measured from cores of thirty-two boreholes across the area. Seventeen of these boreholes constitute two intersecting borehole profiles, which is ideal for locally modeling the 3D distribution of the ore deposit. High densities (>3.2g/cm³) correlate strongly with Zn-rich mineralization zones (low resistivity). Compression and shear wave velocity measurements made from representative core samples of the regional geology range from ~2.91 to ~6.41 km/s for P-waves and from ~1.41 to ~4.20 km/s for S-waves. The lower bounds of these velocity values correspond to regions with a high degree of alteration. Moreover, porosity estimates from rocks on the property suggest that porosities range from 0.35% to 11.78%. Induced polarization (IP) measurements confirm that mineralized samples show a significant chargeability and resistivity contrast with respect to the host rock. Thus, chargeability and resistivity information can be combined to adequately identify conductors associated with mineralization in electrical survey data.
Information on these physical parameters and their 3D statistical distribution have serious implications on various aspects of mineral exploration that include: resource estimation, choice of geophysical data acquisition and interpretation, and information for preliminary rock stability assessment.

REFERENCES

INTRODUCTION
The management of risks associated with slope instability is essential in the safe and economic operation of open cut mines. Over 80 Slope Stability Radar (SSR) systems have been installed at major mines around the world to manage those risks. The SSR technology is now considered best-practice by the global mining industry for managing unstable and production-critical slopes. The SSR uses radar to remotely scan a rock slope, continuously measuring movement of the face. The technology can detect and alert users of wall movements with sub-millimeter precision. Radar waves adequately penetrate through rain, dust and smoke giving reliable and real-time measurements, 24 hours a day. SSR systems have detected and recorded warning movements in over 200 rock falls and slope failures, ranging from small wedge failures of a few tonnes to dump failures of up to thirty million tonnes which has allowed productivity, planning and safety gains in many operations.

Framework for Slope Hazard Management using the SSR

One of the most successful opportunities to reduce the consequence of a slope failure is to ensure that no personnel or mining equipment is present at the location of a slope failure when collapse finally occurs. This has been the major benefit of the SSR system to the different mining operations around the world.
The SSR is a state-of-the-art development for monitoring slope movement in open pit mines. It offers unprecedented sub-millimeter precision and broad area coverage of wall movements through rain, dust and smoke. The real-time display of the movement of mine walls has allowed continuous management of the risk of slope instability at a mine operations level. There are two key roles where mines are now using the slope stability radar:

1. Safety Critical Monitoring: The radar is used during mining production as a primary monitoring tool of a designated unstable slope.
2. Campaign Monitoring: The radar is moved around the mine in a repeatable manner to compare movements at each site over an extended time, and determine problematic areas. Campaign monitoring in this manner is often used in metalliferous mines until determination of developing failure is observed.

In the case studies discussed in this paper, the SSR was utilized in both roles. The SSR technology has enabled a radical change in the management of risks in open cut mining operations, which has resulted in a rapid take-up of the technology throughout the world to date. Our first SSR system was deployed in Canada in January 2008 and we currently have 5 systems deployed in Canada. GroundProbe is committed and looks forward to our continued success in providing real-time slope hazard management services to the Canadian Mining industry.
SESSION 6 RESERVOIR GEOMECHANICS

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Numerical Expansion Analyses Of The Strategic Petroleum Reserve In Bayou Choctaw Salt Dome, USA
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Numerical Expansion Analyses of the Strategic Petroleum Reserve in Bayou Choctaw Salt Dome, USA

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1 NEED

The Strategic Petroleum Reserve (SPR) currently stores over 700 million barrels (MMB) of crude oil at four sites located along the Gulf Coast. The capacity of the existing 62 caverns is 727 MMB. The U.S. Department of Energy (DOE) decided to increase the size of the reserve to 1 billion barrels. While this will require the development of a new site, existing sites will be enlarged. At Bayou Choctaw (BC), the current storage capacity of 76 MMB will be expanded to 109 MMB through the leaching of two new 11.5 MMB caverns and acquisition of one existing 10 MMB cavern.

Fifteen active and nine abandoned caverns exist currently at BC, with a total cavern volume of some 164 MMB. The DOE has a plan to leach two additional caverns and convert one extant cavern within the BC salt dome [URNS, 2006]. Cavern 102, a former Union Texas Petroleum cavern, is potentially available for conversion to a SPR cavern. This simulation attempts to investigate the structural integrity of the three expansion caverns and their interaction with other caverns in the dome. The impacts of the expansion by three caverns on underground creep closure, surface subsidence, infrastructure, and well integrity are quantified.

2 DESCRIPTION

The cavern shapes and locations vary widely. Since the three caverns planned for the expansion, the six existing SPR caverns, and seventeen other caverns have structural interactions, a model including all caverns in the dome was used to investigate the SPR structural behavior.

Two new caverns, A and M, and the conversion of Cavern 102 were added into the BC salt dome. The shapes of Caverns A and M are modeled as bifsurtum while Cavern 102 is modeled as a cylinder. Four material blocks are used in the model for the overburden, caprock, salt dome, and surrounding rocks. Design criteria for DOE caverns is governed by a requirements document [DOE, 2001]. The analysis results show that the locations of the two newly proposed expansion caverns are acceptable, and all three expansion caverns can be safely constructed and operated.

*Existing caverns within Bayou Choctaw salt dome*
References


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Using rock physics for constructing synthetic sonic logs

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ABSTRACT: Knowledge of velocity-depth trends in shales is important for determining background velocity, interpreting seismic data and for predicting abnormal pressures for drilling purposes. However, the existing log data on shale formations is often unavailable or unreliable. Hence, we have used both empirical and effective medium theory based rock physics models for constructing synthetic sonic logs for mudstone sequences at North Sea wells. Our approach is to use resistivity or porosity as the independent variables. The RMS errors for the resistivity based Faust (1953) and Hubert (2008) models are 3.2-11.6 % and 4.2-11.7 %, respectively. The RMS errors are 4.4-7.9% and 4.4-12.7% for the porosity-based Wyllie (1951) and Holt & Fjær (2003) models, respectively. In terms of model complexity it is the Faust (1951) model that provides the best fit to the measured sonic data since it has the fewest number of fitting parameters.

The location of the three wells at the North Sea (NPD 2009).

In this study we have investigated the predictive power of purely empirical and effective medium theories for generating synthetic sonic logs for three North Sea wells. The Faust model
provides the best fit to the actual sonic log for the three lithostatigraphic groups for well 6507/2-1. It is also the least complex model with only one fitted parameter $\gamma$.

We have demonstrated that both empirical and effective medium theories can be used for computing synthetic sonic logs for mudstone sequences at North Sea wells. The RMS errors for the resistivity based Faust (1951) and Hubert (2008) models are 3.2-11.6% and 4.2-11.7%, respectively. The RMS errors for the porosity based Wyllie (1951) and Holt & Fjær (2003) are 4.4-7.9% and 4.4-12.7%, respectively. In terms of model complexity it is the Faust (1953) model that provides the best fit to the measured sonic data since it has the fewest number of fitting parameters.

Figure 3. The modeled velocities with the Faust (1953), Hubert (2008), Wyllie (1956) and Holt & Fjær (2003) models for well 6507/2-1.

1 REFERENCES

Compaction localization in high porosity sandstones with various degrees of heterogeneity: insight from X-ray computed tomography

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High resolution X-ray computed tomography imaging (with voxel size 48 μm) was conducted on various sandstone samples with different initial degrees of heterogeneity. All samples were imaged intact, and some after having developed localized compaction features in conventional triaxial experiments at high confining pressure, i.e. in the shear enhanced compaction domain (Wong et al., 1997).

Our analysis of heterogeneity from X-ray CT data is based on the use of the coefficient of variation \( \delta = \sigma / \mu \) (\( \sigma \) being the standard deviation and \( \mu \) the average value). The coefficient of variation was shown by Otani et al. (2005) to be a convenient indicator of material heterogeneity in CT images of experimentally deformed sand piles. The authors reported differences in \( \delta \) before and after their experiments, and interpreted these differences as the effect of grain crushing resulting in more even material distribution within the voxels, hence in lower \( \delta \) values. The same concept was used by Louis et al. (2006) to image contrasts in \( \delta \) values throughout a volume of rock that had developed compaction bands in triaxial laboratory testing, knowing that the bands that had formed could be identified by visual inspection but not on the X-ray CT images. This study resulted in a radically enhanced image of the discrete compacted zones within the CT-scan series, whereby more deformed areas showed relatively low values of coefficient of variation due to grain crushing and pore collapse, as compared to nominally undeformed areas.

In the present work, we extended the use of the coefficient of variation to undeformed rock samples in order to tentatively relate initial heterogeneity to compaction localization. As already been suggested by several experimental studies (Klein et al., 2001; Baud et al., 2004) and theoretical simulations (Katsman et al., 2005; Wang et al., 2008), mechanical heterogeneity may constitute a first order controlling parameter, together with porosity, for the development of localized compaction features such as compaction bands. The coefficient of variation was calculated over sliding blocks of 3*3*3 neighboring voxels in all available CT volumes of undeformed sandstones. In order to observe the effect of the image resolution on this local value of the coefficient of variation, we also performed the same calculation for stepwise decrease of the resolution, which was achieved through successive reductions in image size. Figure 1a presents the results of such calculations for all the sandstones studied. In every case, \( \delta \) increases and reaches a peak value, before the increase in voxel size progressively homogenizes the volume considered. This characteristic pattern, where the voxel size at \( \delta_{\text{max}} \) is likely related to the dominant spatial frequency, and the \( \delta_{\text{max}} \) itself carries information on the pore size distribution and mineralogical composition, provides a mean to compare the different sandstones. We chose to extract \( \delta_{\text{max}} \) for every rock to use as an indicator of rock heterogeneity and plot it against the measured porosity in order to provide a potentially predictive diagram for the type of compaction observed eventually in the deformed samples. Such plot is showed in Figure 1b, and the type of compaction actually observed is specified after the name of each sandstone. ‘CB’ stands for compaction bands and corresponds to the formation of an array of well defined compaction
bands, ‘diffuse CB’ denotes the presence of zones of preferred compaction with ill-defined boundaries (Baud et al., 2004), and ‘no CB’ denotes the absence of localized compaction features (homogeneous compaction). Figure 1b suggests that discrete compaction features may form when the two conditions of high porosity and low δ value are met.

Figure 1. a. Average local δ value as a function of voxel size in the sandstones studied. b. Type of compaction as a function of porosity and heterogeneity. Compaction bands develop in more homogeneous and more porous samples.

Our results suggest that 3D X-ray CT data can readily provide a local indicator of heterogeneity that is likely to play a significant role in the formation and propagation of compaction bands in porous sandstones. The coefficient of variation as second controlling factor after porosity may explain why, for instance, rocks with comparable porosity exhibit distinct compaction patterns. In situations where porosity was not measured beforehand or can not be obtained, the same X-ray CT data, through proper X-ray attenuation/density calibration, may be used to retrieve that porosity as well, making this tool potentially powerful for the prediction of localized compaction.

References:


INTRODUCTION

The fundamental concept of SAGD is to heat the oil sand by steam and to reduce the viscosity of oil. This concept, however, is the principal of any thermal recovery process in petroleum engineering. SAGD is a continuous process of oil production for heavy oil reservoirs.

SAGD consists of two boreholes that are drilled horizontally one on the top of the other. Both boreholes are drilled close to each other and close to the bottom of the oil sand layer and steam is injected through the top borehole. An initial startup phase is completed, typically after several months of steam circulation in both the upper and lower horizontal wells, when a clear communication between two boreholes is identified. The upper well is converted to steam injection and the lower well is converted to a production well. While steam is injected into the formation, a zone all around the injector will be affected by steam and heat transfer mechanism causes the oil to flow and causes the steam chamber to grow.

Once the steam chamber grows vertically and reaches the upper barrier, lateral growth becomes the dominant mechanism for steam chamber growth. Butler model that was originally proposed by Butler et al. (1981) originally proposed a model (the “Butler” model) which provided an elegant analytical solution for SAGD when the steam chamber has reached the caprock at the top of the oil sand layer. Experimental modeling has been also carried out to prove the feasibility of SAGD such as Chung and Butler (1988). The theory for the Butler model, however, does not consider the effect of geomechanics. Geomechanics describes the process in which stress-induced deformations and mechanical failures change the initial properties of the reservoir. Considering geomechanics in modeling can define why permeability and porosity increases. This is also true for the many subsequent models developed based on the original Butler theory such as Reis (1992 and 1993), Akin (2005), Liang (2005) and Nukhaev et al. (2006).

All the models above have used small-scale lab test data or numerical flow simulators to check the accuracy and to validate the model. The problem with flow simulators is clear; they solve hydro-thermal equations and geomechanics is ignored. Therefore, those models that have been validated with flow simulators can be used as a replacement of a flow (not full physics) simulator. But why those models that have been verified with lab tests are not strong enough to capture full physics? By other words, why these models could not predict SAGD process in a real reservoir? An important reason lies on lab test preparation. Lab tests conducted to verify the analytical models were not configured to include geomechanical effects and so, can not be used to infer the relevance of geomechanical processes on SAGD physics. The simulated oil sand that is synthetically reproduced in laboratory does not have the natural interlocked structure reported by Dusseault and Morgenstern (1978). Also, surcharge effects and test rate are not included in the experimental models. Therefore, the geomechanical effect does not exist in both model and data set and so flow models alone can successfully match lab tests.

The importance of considering geomechanics in modeling and designing SAGD has been confirmed in many researches (e.g. Chalaturnyk, 1996, Li, 2006, and Collins, 2007). A very common solution to consider the effect of geomechanics for modeling a SAGD problem is numerical modeling. Coupling methodologies are usually suggested. In general, these iteratively coupled or sequentially coupled reservoir-geomechanical models have very long run times.
which make them currently unsuitable for inclusion in closed-loop reservoir management workflows for SAGD projects. Ideally, a fast model with a low level of complexity that is able to capture the important or most relevant geomechanical physics and its impact on the SAGD process would valuable. In this paper the first stage of including geomechanics in an analytical Butler-type SAGD model (a version of the Reis model) is described which is briefly summarized below.

2 GEOMECHANICAL COUPLING; AN ANALYTICAL MODEL

For coupling geomechanics and flow simulation, as shown in Figure 1, we have assumed that there exists a geomechanical shear zone which is moving in front of the steam chamber. The idea comes from the fact that permeability changes do not necessarily occur inside the steam chamber. Comparing the results of some limited studies such as Chalaturnyk (1996) and Collins (2007) supports that permeability changes due to geomechanics is not limited to the area inside or very close to the area with the steam temperature. For simplicity, the shear zone is assumed to be linear and effective stresses on the triangular regions (SC and SZ in Figure 1) are bounded by a surcharge force (overburden stress) from the top and the lateral pressure on the sides. Symmetry of the zones about the wells creates a zero displacement boundary condition along the left boundary of the steam chamber (SC). Equilibrium within the model is assumed to satisfy a limit equilibrium condition, a common approach to the solution of geotechnical problems (e.g. Duncan, 1996).

3 GEOMECHANICAL COUPLING; AN ANALYTICAL MODEL

In this paper a modified version of Reis model was proposed to simulate the drainage process occurring in SAGD. Modifications were applied to modify two concerns; (i) the physical inaccuracy of the mathematical solution and (ii) heterogeneity of permeability. The modified model was compared with two lab tests results and a validated numerical modeling. In both cases results showed an improvement in fitting the data especially in actual size data. It was also showed that the energy balance that firstly proposed by Reis has been effectively improved in the modified model.

The proposed geomechanical modeling (coupled with the drainage model) which is the main goal of this study was tested against the available numerical data. The results confirm an excellent agreement more for cumulative oil production and less for injected steam.

Regardless of the limited available data for supporting the validity of the model, the proposed methodology presented in this paper has shown that the model may capture sufficient reservoir-geomechanical processes inherent in the SAGD process to warrant the use of the model in smart field or closed-loop reservoir management studies of the SAGD process.

Figure 1. Moving shear zone in front of steam chamber
Poroelastic Modelling of Production and Injection-Induced Stress Changes in a Pinnacle Reef

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1 POROELASTIC INDUCED STRESS CHANGE MODELLING

During fluid production from hydrocarbon reservoirs, and fluid injection for enhanced oil recovery, greenhouse gas sequestration or waste disposal, stress changes are induced within and surrounding the reservoir. To ensure that production or injection can be maintained in a safe and effective manner, it is necessary to assess the effect of these stress changes on the hydraulic integrity of the rocks that bound the reservoir. For example, if shear or tensile fractures are induced, or if existing faults or fractures are re-opened or reactivated, these features are likely to serve as fluid leakage paths. The full-length version of this paper presents a summary of closed-form and semi-analytical models that are appropriate for geomechanical modelling of induced fracturing and fault/fracture reactivation potential in a reservoir that has undergone a historical pore pressure reduction due to fluid withdrawal, followed by pore pressure increase due to fluid injection. The application of these solutions is illustrated for a pinnacle reef with properties representative of the reefs present in the Zama oil field of northwest Alberta, Canada, which is currently an acid gas injection site.

2 CASE STUDY: ACID GAS INJECTION IN THE ZAMA OIL FIELD

The Zama oil field contains more than 400 pinnacle reefs of the Middle Devonian Keg River Formation. In this work, the effects of geomechanical processes on the hydraulic integrity of a single, representative pinnacle reef was investigated. An idealized reservoir geometry was assumed, consisting of an axisymmetric ellipsoid 90 m high and 320 m wide, with a mid-point depth of 1500 m. The reservoir was modelled as a porous dolomite unit, overlain and surrounded by low-porosity anhydrites of the Muskeg Formation. Previously published values were used for rock mechanical properties and in-situ stresses. An initial pore pressure of 14.5 MPa was used, and the consequences of pore changes of $\pm 10$ MPa were modelled.

A summary of the modelling results is provided in Figures 1 and 2. These results show that, for the scenarios considered, the potential to induce shear fracturing is not significant at any point within the reservoir or the surrounding rock during both fluid production and injection. Similarly, fault or fracture reactivation is not predicted for the reservoir or any of the points that were analyzed in the surrounding rock. However, the results do suggest that optimally-oriented faults or fractures (i.e., dipping roughly $60^\circ$ in a direction sub-parallel to the minimum horizontal stress azimuth), if present at points in the sideburden, may have been in a near-critical state during historical production operations (Figure 1b). As such, this work suggests that this site is favourable for containment of acid gas, but further illustrates a means of using geomechanical models to focus geological characterization efforts on identifying the presence or absence of specific features that are most critical to the security of acid gas injection.
Figure 1. Effective stress state after pressure depletion of 10 MPa: (a) within the reservoir; (b) in the sideburden aligned with minimum horizontal stress; (c) in the sideburden aligned with maximum horizontal stress; and (d) in the caprock. The dashed circle represents the original stress state, in which the maximum horizontal stress and the vertical stress magnitudes are equal. \( H \) denotes the maximum horizontal stress, \( h \) denotes the minimum horizontal stress, and \( V \) denotes the vertical stress.

Figure 2. Effective stress state after a pressure increase of 10 MPa: (a) within the reservoir; (b) in the sideburden aligned with minimum horizontal stress; (c) in the sideburden aligned with maximum horizontal stress; and (d) in the caprock. The dashed circle represents the original stress state, in which the maximum horizontal stress and the vertical stress magnitudes are equal. \( H \) denotes the maximum horizontal stress, \( h \) denotes the minimum horizontal stress, and \( V \) denotes the vertical stress.
SESSION 7  SURFACE CONSTRUCTION

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In Situ Fracturing Mechanics Stress Measurements To Improve Underground Quarry Stability Analyses
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DEM Study Of The Mechanical Behavior Of A Leached Interface Upon Shearing
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Deriner Hydropower Scheme – Geotechnical Issues And The Particular Case Of The Spillway Tunnels Design And Construction
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Study On Feasibility Of Columnar Jointed Basalt As A High-Arch Dam Foundation
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In situ fracturing mechanics stress measurements to improve underground quarry stability analyses

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EXTENDED ABSTRACT

Stability condition of underground excavations have to be assessed to guarantee safe operations within the quarrying working areas. This objective requires a reliable assessment of the rock mass natural state of stress and of the stress redistribution induced by the excavations. This goal is often complicated by difficult geological conditions induced by tectonic stresses, complex topographies and large and irregular excavation geometries. Consequently, the acting state of stress needs to be measured by specific in situ measurement devices. This work refers on an in situ measurements campaign carried out by CSIRO tests in an underground quarry located in the Carrara marble basin. The difficult path carried out for measurement interpretation is described in this paper. In particular, the numerical modeling performed by using the Boundary Element Method (BEM) first, to study the influence of both surface topography and excavation geometry and then the Distinct Element Method (DEM) to evaluate the influence of rock discontinuities in the measurement zone are illustrated in this work. Parametrical analysis of the rock mass mechanical features is also described in order to calibrate the numerical models and to analyze the experimental results.

The work has been carried out through several phases, which are:

- characterization of the rock mass through geostuctural surveys: this allowed us to classify the rock mass and determine the mechanical and physical parameters necessary for modeling;
- on site measurement of stress levels through the application of overcoring techniques;
- numerical analyses for the simulation of excavation phases, using the Examine3D (BEM) engineering analysis code.
- comparison between the measured stress levels and those numerically calculated and subsequent calibration of the BEM model;
- Numerical model of the forecasted excavation based on the calibrated (BEM) model.

This studied area is located in two adjacent underground quarries with a complex excavation geometry. The geomechanical survey was carried out following the ISRM recommendations (1978) in the chamber that borders the old access gallery on the west side. Laboratory tests were carried on intact rock specimen and natural rock discontinuities. The on site tests were carried out by using standard CSIRO cells (Dunnicliff J., 1993). In order to assess the on site state of stress of the rock mass surrounding the quarries using the overcoring technique, five horizontal borehole where drilled and nine tests were carried out inside those boreholes at various depths. Duncan et al. (1980) proposed a relationship between the measured strains and the stress field acting on an isotropic media; such relation was applied to the data obtained from the CSIRO cells.
The purpose of 3D BEM modelling using the Examine code is to provide a preliminary assessment of the mechanical behaviour in the exploited rock mass due to the evolution of the excavation phases for the three experimental room and pillar panels.

The objective of 3D DEM (Cundall P.A., 1971) numerical modelling, using 3DEC code Itasca Consulting Group, 1988), is to assess the mechanical behaviour of the experimental room and pillar panel, by taking the blocky structure of the rock mass into account. Successively, using the statistical option of the Resoblok (Héliot D., 1988) code, a 3D model reproducing the whole rock mass involved in the excavation was generated. After the model was built, in order to verify its geometrical reliability, several section containing the surveyed scanlines were created. The creation of a three dimensional model of the quarry, allowed us to simulate stress levels with an assessment of parameters involved, which led us to our first conclusions regarding the stability of the excavation tunnel walls. This calibration procedure is possible by following a back analysis based on on-site stress level measurements through the application of CSIRO testing method. The distance between the stress values measured on-site and those calculated numerically during an initial phase of simulation showed the fundamental importance of the “calibration” phase by including also the $k_0$ assessment. The model thus obtained could be a fair approximation of the real situation and laid the basis for the simulation of further phases of excavation.

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DEM study of the mechanical behavior of a leached interface upon shearing

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Leaching of concrete is a well known phenomenon which takes place as soon as concrete is exposed to water for a significant time. Accelerated leaching processes, such as ammonium nitrate leaching or electric leaching, have been developed in order to obtain enough data in an acceptable time. Wide knowledge has been acquired on the influence of leaching on the mechanical properties and porosity of bulk concrete and mortar (Carde and Francois, 1999). However, little data is available on the influence of leaching on the mechanical behaviour of interfaces. Buzzi et al. (2008) have recently performed a series of hydro-mechanical shear tests on rock-leached concrete interfaces. Their results suggest that leaching the interface tends to reduce its mechanical strength and to turn the behaviour from contractant/dilatant into fully contractant. The mechanical behaviour of a leached concrete interface has been investigated using the particle flow code PFC3D. The distinct element method has been chosen because it allows one to simulate the evolution of the macro porosity during the leaching process by randomly deleting a fraction of the particles. Moreover, the progressive degradation of the interface asperities during shearing can also be captured. These two features are of major significance to reproduce the mechanical behaviour of leached interface.

![Figure 1: Validation of the leaching implementation: Ratio of Young's modulus over initial Young's modulus.](image)

The increase of macro porosity is achieved by knowing the chemical composition of the five leached zones identified by Bernard et al. (2008) and the relative position of the different dissolution fronts. A volumetric fraction of particles corresponding to the amount of species physically dissolved is removed from the numerical specimen. Then,
cylindrical specimens are tested in unconfined compression. Numerical results, shown in Figure 1, are in good agreement with the experimental results.

Different leaching depths (LD) have been applied to a simplified saw-tooth interface and several shear tests have been performed in order to qualitatively characterize the mechanical response of both the intact and leached interfaces. As for the bulk material, a progressive loss of mechanical strength is observed for the interface when the thickness of the leached zone increases (Figure 2 (a)). Logically, the failure criterion of the interface is also reduced. The qualitative change in behaviour discussed by Buzzi et al. (2008) has also been observed during numerical tests (Figure 2 (b)). The interface initially displays a contractant/dilatant behaviour, which turns into a fully contractant response with leaching. This can be explained by the local loss of shear strength within the tooth preventing both surfaces from riding each other and thus dilating.

Figure 2: (a) Evolution of shear stress versus tangential displacement. (b) Evolution of normal displacement versus tangential displacement. Shear test under constant normal stress of 4MPa. Leaching depth ranges from 0 mm to 10 mm.

Further work is to be done on the more realistic joint morphology and on coupling hydraulic and mechanical behaviour.

REFERENCES


Deriner Hydropower Scheme – Geotechnical Issues and the Particular Case of the Spillway Tunnels Design and Construction

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EXTENDED ABSTRACT

The paper addresses some geotechnical issues related to the construction of the Deriner dam and HEPP. Design considerations and rock stabilization measures are presented for the excavations which took place in a highly fractured and weathered rock mass prevailing on site. Particularly, the shallow excavations for the overflow spillways tunnels are discussed.

The Deriner hydropower scheme, which is currently under construction, is located in the north-eastern Black sea region of Turkey. It encompasses a 249 m high, double curvature arch dam with crest length of 720 m with all classic appurtenant structures and an underground powerhouse complex, which has been excavated on the right bank of the Çoruh River at a depth of approximately 100 m. The powerhouse has a width of 20 m, length of 126 m and a height of 45 m and will have four vertical Francis units with an installed production capacity of 670 MW. Two gated spillway tunnels have been designed to evacuate a 2250 m$^3$/s flow. In addition, 8 orifice spillways have been designed to ensure the evacuation of a maximum flood of 10’110 m$^3$/s.

The rock mass at the Deriner site is mainly composed of granodiorite intruded by diabase dykes, which are generally very suitable for the foundation of an arch dam. Nevertheless, the upper layer of the rock is decompressed and heavily jointed, which means that it has to be excavated so as to ensure sound rock for the dam foundations. Based on the results of the unconfined compression tests, the following Unconfined Compressive Strength (UCS) values have been recommended as design values for dry rock: left bank: $\sigma_{ci} = 120$ MPa and right bank: $\sigma_{ci} = 80$ MPa.

In order to evacuate the floodwater safely, two gated spillway tunnels have been designed. The overflow spillway tunnels start with a transition zone over which the tunnel shape (horseshoe) and size (12 m width and 18 m height) is decreasing to a circular section 10 m of diameter. The first part of the tunnel is in curvature with a radius of 180 m and a central angle of 62°. The overflow spillway transition tunnel on the right bank is shallow over a distance of about 50 m. At the beginning, overburden is about 20 m and the pillar roughly 15 m wide with a huge excavation section. Thus, a special attention has been given to the excavation of this shallow tunnel. In addition to that, on the right bank, a persistent low strength C joint set (dip direction/dip angle 250/45) has been encountered that dips into the excavation at a highly unfavourable angle. The fault was responsible for the failure of the slope below the overflow spillway platform and above the power intake area where an anchored wall was constructed. In order to achieve a minimum short term factor of safety of 1.2, 200 pre-stressed anchors have been installed in the slope above the overflow spillway and the power intake.

The design of the tunnel excavation and the rock support is based on the classical rock classifications (GSI, RMR and Q) followed by sophisticated Finite Element analyses and a structural...
discontinuity analysis. A finite element analysis has been carried out to confirm that the overflow spillway tunnel can be excavated with a necessary structural safety of the tunnel temporary support. A plane strain analysis has been carried out, considering a vast region in the horizontal direction extending from −800 m to +500 m from the tunnel axis. It must be noted that the temporary support is introduced in calculations after 30% of stress relaxation. Thus, the remaining 70% of deformation of a certain sub-step plus of course each following sub-steps introduce forces into the temporary support. The C-fault mentioned earlier has been included in the analysis using contact elements.

In addition to that, a rock wedge stability analysis has been carried out. The method consists in a geometrical analysis allowing to define systematically the most critical combination of joint sets leading to falling or sliding of the removable wedges with respect to the excavation surfaces, such as the roof, walls, edges or corners of the tunnel.

Based on the above presented analyses, the rock support has been assumed as follows:
- INP 220 steel ribs, spacing 1.0 m,
- shotcrete 12 cm thick, which is then increased to get a temporary vault 35 cm thick,
- in the walls, the shotcrete is 12 cm thick, and reinforced with wire mesh.
- systematic bolting spaced 1.5 x 1.5 m (2.25 m²/pce), L=5 m. Due to presence of the C-fault (Figure 1) on the valley side of the tunnel length of the rockbolts has been increased to 9 m and spacing 1 x 1 m.

For successful overflow excavation works, it was of fundamental importance that blasting and excavation are very carefully executed in order to prevent from damage of the surrounding rock mass. To minimize over-break, the direction of the blast-holes was carefully selected and the borehole deviation and the charging operation were controlled. Since the tunnel is highly inclined, the excavation started with benches of 1 m high and then heading with a round of 3 m. For the purpose of the tunnel excavation, a special winch on rails has been developed, allowing transport of the excavated material.

It should be noted that the excavation of the overflow spillway has been successfully carried out without any stability problem in spite of a quite complex geometry of the tunnel and quite difficult geological conditions.

![Figure 1. Rock support of the overflow spillway on the right bank.](image)

ACKNOWLEDGEMENT

The authors are grateful to the General Directorate of State Hydraulic Works (DSI), Ministry of Environment and Forestry of Turkey for allowing publication of this paper.
Study on Feasibility of Columnar jointed basalt as a high-arch dam foundation

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ABSTRACT: Columnar jointed basalt is a fractured rock, with a lot of small spacing structural planes and poor integrity of the rock mass. On the basis of the comprehensive study of the columnar shape of Columnar joints, rugosity of the fracture surface and the chemical composition of basalt, columnar joints of Baihetan dam area were formed as a result of the cooling and shrinkage effect of magma. Columnar joints basalt are mainly chlorite, kaolinite, epidote, tremolite chemical reactions and the content of columnar joints are mainly chlorite by slice identification and chemical analysis. Columnar joints basalt showed high uniaxial tension test, low coefficient of friction and high cohesion by indoor physical and mechanical test. Columnar joints rock mass showed high strength and large deformation by field shear test and deformation test. Columnar joints basalt can be used directly as a foundation of dam as long as we follow the rules and reduce disturbance strictly.
The implicit microfracture developed in rock mass thin section

Implicit microfracture of plagioclase phenocryst of phenocryst basalts

The study comes to a conclusion as follows:

1. Columnar joints at Baihetan dam site were formed as a result of the cooling and shrinkage effect of magma.

2. Columnar jointed basalt easy microfissure, submicrofissure, and it is filled with quartz and local tremolite vein, columnar joints basalt are mainly chlorite, kaolinite, epidote, tremolite chemical reactions and the content of columnar joints are mainly chlorite.

3. The results from indoor physics experiment suggest that columnar joints basalt showed high uniaxial tension test, low coefficient of friction and high cohesion.

4. The results from large-sized experiment, deformation experiment, and indoor “small rock mass” triaxial experiment suggest that columnar jointed rock mass showed higher tension and formation index, which can be directly used as dam foundation if enforced with strict discipline of operation rules and reduced disturbance.
SESSION 8  NUMERICAL MODELLING OF CONTINUUM-DISCONTINUUM BEHAVIOUR II

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Numerical Modelling Of A Brazilian Disc Test Of Layered Rocks Using The Combined Finite-Discrete Element Method
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4360
Estimation Of Rock Block Strength
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Variation of Failure Mechanisms of Slopes in Jointed Rock Masses with Changing Scale
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The combined finite-discrete element method (FEM/DEM) is a numerical technique capable of dealing with mechanics of discontinuum. In this study, a modified FEM/DEM research code has been used to study the behaviour of layered rock samples under standard Brazilian tests. The effect of layering and the direction of loading on the samples behaviour are studied through various layering and loading configurations. A parametric study including the effect of loading rate on the tensile strength of samples has been carried out. This paper demonstrates the suitability of the numerical approach to explicitly model rock deformation and failure.

The combined finite-discrete element method (FEM/DEM) is a numerical technique capable of dealing with mechanics of discontinuum. In this study, a modified FEM/DEM research code has been used to study the behaviour of layered rock samples under standard Brazilian tests. The effect of layering and the direction of loading on the samples behaviour are studied. The results show a promising potential for the use of FEM/DEM for such studies.

Schematic of a Brazilian Disc for indirect measurement of tensile strength.
Fracture propagation at different time steps for the homogeneous model (Model A) after (a) 2.1 ms, (b) 2.16 ms, (c) 2.25 ms, and (d) 3.6 ms. Colours represent vertical stresses ($\sigma_y$).

Load – displacement curve for the homogeneous rock (Model A). Note that the platens were not in contact when the test started and moved 0.05 mm to reach the disc.

The applicability of the combined finite-discrete element method to model laboratory scale experiments was shown through the simulations of Brazilian disc tests. Both a homogeneous rock and a layered rock representing schistosity were used. The simulation results suggest that the presence of layers or bedding planes plays a major role in mechanical behaviour of the models. Also, direction of loading with respect to those planes of weakness is of great importance.

The main failure mechanism for the homogeneous disc and those layered and inclined at 0° and 90° was tensile splitting. In contrast, the layered disc aligned at 60° showed mixed tensile splitting and shear failure. This is due to the weak joint interface which is approximately aligned along the direction of large shear stresses near the loading platens.

The simulated tensile strength values were generally higher than the input. Several reasons including loading rate and mesh sensitivity were given for this issue. However, further research is needed to overcome this problem.
1 INTRODUCTION

In the present paper, numerical simulations based on a finite/discrete element approach are used to study deformation and failure process during standard rock mechanics. FEM/DEM modeling has been used to explicitly model the transition from continuum to discontinuum. The transition is simulated through a crack nucleation and propagation that obeys to the Griffith’s failure criterion. If the fracture criterion within the intact rock (represented by FEM) is met, a new crack (represented by DEM) is initiated. Re-meshing allows the fracture process through the FEM mesh to be tracked and visualized, thus contact properties can be assigned to pre-existing fractures and newly generated fractures. In this study, compressive tests of rock samples are simulated. In this study, compressive tests of rock samples are simulated. The applicability of ELFEN to properly and efficiently simulate typical laboratory rock mechanics tests, with reference to the rock fracture process, is described in detail. In addition, an indirect validation of the extensional strain criterion proposed by Stacey (1981) is given.

The rock mechanics tests modeled and described in the following are:
- Brazilian test;
- Uniaxial compression test;
- Triaxial compression test in a homogeneous rock.
Results obtained in the simulation of the Brazilian test

Force and vertical displacement relation in unconfined and confined compression tests simulations

In order to model the fracture behaviour of rock under loading, the numerical code ELFEN has been used successfully. Experimentation with rock parameter selection was required to achieve good correspondence. Typical rock mechanics tests have been simulated by showing that the fracture process in rock is represented remarkably well. A notable feature of ELFEN is that no a priori assumptions need be made about where and how fractures will be initiated and develop thus leading to failure. Fracturing can occur spontaneously and exhibit a variety of mechanisms when certain local stress conditions are met.

The entire fracturing process is shown to be simulated during testing, including initiation, propagation and coalescence of fractures. In particular, the analysis of the axial compression strain versus the lateral extension strain diagrams during testing in uniaxial and triaxial compression shows that the Stacey’s extension criterion is apparently confirmed.
Estimation of rock block strength

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The intact strength of rock blocks larger than standard core sizes (i.e. 50 mm diameter) is often of interest in rock engineering, particularly in underground mines where insufficient natural or blasted fragmentation may lead to large rock blocks and associated difficulties with handling and comminution. It is well established that the intact strength of rock decreases with increasing scale. Hoek & Brown (1980) developed an empirical scale effect relation for intact strength on the basis of laboratory testing conducted by a number of different researchers on homogenous hard rock samples (i.e. samples lacking significant microfracturing or alteration). Their relation takes the form:

\[
\frac{\sigma_c}{\sigma_{c,50}} = \left( \frac{d}{50} \right)^{-0.18}
\]

where \( \sigma_{c,50} \) is the uniaxial compressive strength of a cylindrical specimen with diameter \( d = 50 \) mm, and \( \sigma_c \) is the uniaxial compressive strength of a specimen with an arbitrary diameter, \( d \) (10–200 mm). Yoshinaka et al. (2008) proposed the use of equivalent length, \( d_e = V^{1/3} \), in place of diameter and suggested that the exponent, \( k \), is variable and dependent on the presence of microflaws (i.e. pores, open cracks, veins, etc.):

\[
\frac{\sigma_c}{\sigma_{c,0}} = \left( \frac{d_e}{d_{e,0}} \right)^{-k}
\]

Their analysis of test results on a wide range of rock types, strengths, sample shapes and sample sizes suggest that \( k \) ranges from about 0.1 to 0.3 for homogeneous hard rock, and from about 0.3 to 0.9 for weathered and/or extensively microflawed rock.

Laubscher & Jakubec (2001) suggest that the strength of a large homogenous rock block is 80% of the mean intact strength obtained from testing standard (i.e. 50 mm diameter) core samples. This corresponds well with the asymptote (at large block sizes) for \( k = 0.1 \). For rock blocks containing veins and open fractures, an overall scaling factor of approximately 50% is suggested for rocks with a very high flaw density and/or very weak flaw strength; this corresponds well with the asymptote (at large block sizes) for \( k = 0.3 \). Overall, the Laubscher & Jakubec (2001) relations result in rock block strengths that are slightly higher than what Yoshinaka et al. (2008) suggest may be possible in heavily weathered and/or microfractured rock.

A study of rock block strength was conducted for the quartzite lithology present at the Bingham Canyon Mine, a porphyry copper-gold-molybdenum deposit located in the Oquirrh Mountains, just 30 km southwest of Salt Lake City, Utah in the USA. The purpose of the study was to estimate the strength of rock blocks that might be produced during cave mining of the rock mass beneath the existing pit. The quartzite contains microdefects, which are defined as healed hairline fractures less than 1mm in thickness. Of the quartzite that has been logged for microdefects, 7% of has either no microdefects or a minor microdefect intensity (spacing >10cm), 39% has a moderate microdefect intensity (spacing 1 cm to 10 cm) and 54% has a heavy microdefect intensity (spacing <1 cm).

Although the number of samples tested at the larger scales was limited (three 140 mm diameter samples; one 240 mm diameter sample), a relatively strong scale effect is suggested when compared to the strengths obtained from testing on standard-sized cores. The overall trend appears reasonably consistent with the scale effect suggested by Yoshinaka et al. (2008) for weathered and/or heavily microflawed rocks (Figure 1). A somewhat higher rock block strength was estimated using the relations of Laubscher and Jakubec (2001). The study would benefit from testing on a greater number of large-scale samples.
A series of Synthetic Rock Mass (SRM) tests were also conducted to generically assess the impacts of flaw strength on exponent $k$ in the scale effect relation (Equation 2). The SRM approach was originally developed to model large-scale jointed rock masses (Pierce et al. 2007; Mas Ivars et al. 2007) and involves the embedment of discrete planar fractures into a bonded-particle model of the intact rock, developed in this case using the Particle Flow Code (PFC3D). An SRM model representing a 1-m cube of intact rock with an embedded network of cohesive planar fractures was constructed for the study. The fractures were persistent, of random orientation and of a density that resulted in an average linear fracture frequency of 20/m. The effect of scale on intact strength was examined by sub-dividing the 1-m-cube sample into 500 mm, 250 mm, 125 mm and 62.5 mm wide samples. For each of these sample sizes, a number of tests were conducted for each of four different relative vein strengths (100%, 50%, 33% and 25% of host strength).

The SRM samples exhibited an overall power-law trend of decreasing strength (and decreasing variability) with increasing sample size under simulated UCS testing. Values for $k$ of 0.04, 0.15 and 0.22 were found from power law fits to the relative vein strength trends for 50%, 33% and 25% of host strength, respectively. By combining these with the average $k$ value of 0.2 suggested by Yoshinaka et al. (2008) for homogenous rocks (Figure 1), $k$ values for the complete scale effect relation of 0.24 to 0.42 are indicated for rocks with a vein frequency of 20/m and vein strengths ranging from 25% to 50% of the host strength. Larger values of $k$ (i.e. approaching the maximum value of 0.9 suggested by Yoshinaka et al.) might be expected for higher vein frequencies and/or lower vein strengths. These results demonstrate that Synthetic Rock Mass (SRM) modeling techniques can be used to relate the value of the $k$ exponent in the scale effect relation (Equation 2) to the strength of persistent veins. By extending the technique to examine a wider range of microflaw types (e.g. cracks, pores, veins) and properties (e.g. strength, density, persistence) it may be possible to refine empirical relations for rock block strength estimation.

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1 INTRODUCTION - SCALE EFFECTS AND THE STABILITY OF SLOPES IN ROCK MASSES

It has been long recognized in rock mechanics that discontinuities (geological structures) significantly influence the response of rock masses to loadings and excavation (Goodman et al, 1968, Manfredini et al, 1975, Cundall et al, 1975, Bandis et al, 1983). It has also been long observed from slope and other failures that this influence is not the same at different scales (excavations sizes). Generally, at smaller scales discontinuities exert greater influence on behaviour than do intact rock properties. In small slopes, failure mechanisms such as planar wedges, which are controlled by joints, are common. As the scale increases more complex mechanisms such as step-path failures and rotational shear failure, which combine failure along discontinuities with shearing through intact rock bridges, begin to occur. These complex mechanisms can follow overall curved paths that can be similar to those encountered in soils. Toppling and columnar flexural bending or buckling are other failure mechanisms that can occur with increasing slope scale.

At intermediate and large scales, anticipation or prediction of the stability of rock slopes and the manner in which they can fail can be very difficult. This is because at such scales stability is affected by the strength and deformation properties of both intact rock and joints, the geometry and distribution of joints throughout a rock mass, and stress and groundwater conditions.

Of the numerical methods used for the stress analysis today, the family of Discrete Element Methods (DEMs) and Discontinuous Deformation Analysis (DDA) have been considered to be the most well-suited to the problems of blocky rock masses. Recently however, it has been demonstrated that the Finite Element Method (FEM) with explicit representation of discontinuities with joint elements is a credible alternative (Hammah et al, 2009, 2008 and 2007). This paper examines the ability of the FEM to capture the variation of factor of safety and failure mechanism of blocky rock mass slopes with scale. The paper will show that such modelling can help engineers to understand the behaviour of blocky rock slopes at different scales much better, and to more accurately predict failure mechanisms and factors of safety.
2 APPLICATION OF THE FINITE ELEMENT METHOD (FEM) TO PROBLEMS OF BLOCKY ROCK MASSES

Due to the widespread availability of powerful desktop and laptop computers, the FEM with explicit modelling of the behaviour of individual joints can be used for practical engineering in blocky rock masses. This has also been facilitated by the development of techniques for generating networks of discrete fractures, and the development of the Shear Strength Reduction (SSR) method. SSR analysis (Dawson et al, 1999, Griffiths and Lane, 1999, Matsui and San, 1992) allows factors of safety of slopes to be calculated with numerical methods. Studies have confirmed the accuracy of the FEM-SSR technique in general and for the variety of failure mechanisms encountered in rock slope engineering (Dawson et al, 1999, Griffiths and Lane, 1999, Hammah et al, 2005 and 2007).

Although FEM-based SSR analysis is an alternative to conventional limit equilibrium methods in many cases, its ability to readily combine slip along joints with failure through intact material offers several advantages in the modelling of blocky rock mass problems. The method can model the broad range of behaviours of slopes at different scales, from wedge sliding to toppling and rotational failures (Hammah et al, 2007). As well it can easily handle cases in which fractures intersect in a manner such that discrete blocks may not necessarily be formed, i.e. cases in which joints may terminate within intact rock and not only at intersections with other joints (Hammah et al, 2008). Perhaps, the greatest benefit of FEM-based SSR analysis is that it can automatically determine the broad variety of failure mechanisms with no prior assumptions regarding the type, shape or location of these mechanisms. These advantages will be demonstrated on simple slope examples to be described next.

Figure 1a: Joint network pattern comprising two sets of infinitely long joints used in Example I

Figure 1b: Joint network pattern comprising two sets of finite joints used in Example II

Figure 1c: Joint network pattern comprising Voronoi polygons used in Example III
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Radial Flow Permeability Testing of Indiana Limestone

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ABSTRACT:
The study of fluid flow through porous media and soils is particularly important when dealing with groundwater movement in surficial geomaterials that can be initiated by hydraulic gradients developed during construction activity, groundwater extraction, impounding of reservoirs and groundwater recharge during adverse climatic events. In a geotechnical context, rocks are regarded as virtually impermeable materials, with comparatively greater resistance to flow of water through their pore structure. The permeability of rocks is important when deep geologic formations are to be used in geoenvironmental activities for disposal of hazardous and toxic materials, since the movement or transport of the hazardous materials during their eventual release will be largely governed by the permeability characteristics of the geologic formation. The estimation of the permeability characteristic of geologic formations is a non-trivial exercise since the measure of permeability can be influenced by the scale at which the measurement is made. The intact permeability of geomaterials is regarded as an important property that influences the efficiency and reliability of geoenvironmental solutions for deep geologic disposal. The permeability of geomaterials such as concrete and cementitious grouts are also important to many geoenvironmental and infrastructure applications where durability of structural materials in general and cements used for sealing deep boreholes and other access shafts employed in waste management endeavours is influenced by the permeability characteristics of these materials.

The in situ measurement of permeability characteristics of intact rocks has been in existence since construction of concrete dams founded on rock. The laboratory measurement of the permeability of relatively low permeability geomaterials, including rocks and concrete, has been the subject of extensive research over the past four decades; the testing methods include both steady state flow and pressure transient techniques conducted on cylindrical cores of rock. Ideally, the simplest technique for determining the permeability of intact rock involves the attainment of steady state flow conditions in a sample, which is both easy to perform and easy to analyse theoretically, so that the permeability of the material can be conveniently estimated. The primary advantage of a steady state flow test is that the permeability can be estimated from knowledge of the hydraulic gradients applied to the sample, the established flow rate and the dimensions of the flow domain. The primary advantage of the transient test is that it can be performed relatively quickly compared to the steady state test. The choice of the most appropriate type of test ultimately depends on the type of material being examined; transient tests are advocated for rocks such as granite (also cement grout) that have relatively low permeability and the steady state tests have been used for determining the permeability of materials with a comparatively high permeability, including sandstone and Indiana Limestone, which was used in this research.

This paper presents the results of a series of laboratory bench-scale tests that have been conducted to determine the permeability characteristics of Indiana Limestone. The paper describes the experimental facilities, the test procedures and the results of the steady state flow experiments that are used to estimate the permeability of the rock. This research program uses a radial flow steady state test to determine the permeability of Indiana Limestone. The radial flow permeability test is not a straightforward test for determining the permeability of geologic materials since the experimental configurations should assure that the seals enabling the application of a constant radial flow are effective and do not allow leakage that would give erroneous estimations of the permeability. The paper discusses the experimental procedures and presents the results of radial flow tests performed on cylindrical samples of Indiana Limestone measuring 100 mm in diameter, 200 mm in length and containing a cylindrical cavity of diameter 23 mm. The plane ends and cylindrical surfaces of each sample were ground to a smooth finish and, since the flow is initiated from the central cavity, the inner cavity was kept free of debris from the drilling activity. The upper and lower plane faces of the samples were epoxy coated to a thickness of 1
mm. Finally, a groove was machined at the mid-plane of the sample to facilitate the introduction of a flat crack normal to the axis of the cylinder, which will be investigated in future experiments.

A steady flow was attained by the application of a constant flow rate to the internal sealed cavity of the sample. The sealing of the central cavity was achieved through the use of O-ring seals located at the base and the upper epoxied surfaces of the sample. The limestone sample was kept in a water reservoir and an outlet provided to maintain a constant hydraulic potential on the outer cylindrical surface of the boundary. The reservoir containing the limestone sample was placed on a hydraulic jack, which allows movement of the experimental set-up so that the central cavity can be sealed for subsequent pressurization. Maintaining constant cavity pressures over a 4 hour period is considered an adequate seal. The central cavity pressure and the axial load applied to the sample were recorded using a TracerDAQ data acquisition system.

A constant flow rate was applied to the saturated sample, and the water pressure recorded at constant time intervals using the data acquisition system. This data was used to estimate the permeability of the tested sample. Altogether six samples were tested in the radial flow configuration to determine the intact permeability. As expected, there were variations in the results both within the sample group and within the separate samples, with a slightly more noticeable variation between results on the separate samples. This emphasizes the importance of defining an acceptable range when assigning a value for the permeability of naturally occurring geological materials.

Acknowledgements
The work described in this paper was supported by a NSERC Discovery Grant awarded to A.P.S. Selvadurai. The authors are grateful to the Technical Staff of the Department, Mr. John Bartczak and Mr. Marek Przykorski, in particular, for assistance with various aspects of the research.
THM Processes in a Fluid-Saturated Poroelastic Geomaterial: Comparison of Analytical Results and Computational Estimates

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ABSTRACT: The theory of poroelasticity, extended to include thermal effects, provides a useful model for the study of the quasi-static response of fluid-saturated geomaterials subjected to heating. The basic model has been used quite extensively for modeling thermo-hydro-mechanical responses of rock encountered in nuclear waste management and geothermal energy extraction endeavors. The practical application of these theories invariably requires recourse to computational approaches where the governing partial differential equations are solved using Galerkin finite element techniques along with a suitable time-integration technique that assures stability of the solution. In recent years a number of multi-physics codes and general purpose finite element codes have been advocated for the study of such THM responses. The unconditional accuracy of these computational approaches can be assessed only by recourse to comparisons with known analytical solutions. This paper examines the capabilities of two computational codes in predicting the thermo-poroelastic response in a column of heated geomaterial subjected to heat diffusion and pore pressure dissipation through the upper surface, and to surface tractions.

In thermo-poroelasticity, temperature changes can cause deformations of the pore water and the solid phase, which can lead to changes in pore pressure and effective stress. An increase in the pore pressure, in particular, can cause damage to the solid skeleton, and several previous investigations have dealt with the thermo-hydro-mechanical behavior of fractured and intact geomaterials proposed for storing heat-emitting nuclear fuel waste.

This paper demonstrates the use of the computational multi-physics code COMSOL™ for solving a thermo-hydro-mechanical problem in geomechanics. COMSOL allows the user to enter, as an input, the governing partial differential equations. For the purpose of validating the COMSOL software, we use an analytical solution for a one-dimensional problem of a fluid-saturated geomaterial, initially at a uniform temperature and fluid pressure, which undergoes heat diffusion and pore pressure dissipation due to the reduction of the surface temperature and pressure to zero. In addition, the geomaterial column is subjected to a normal traction at the surface. This research also provides an inter-code validation of the accuracy of COMSOL by comparing the results obtained for the one-dimensional problem, using the general purpose finite element code ABAQUS™.

A fully saturated poroelastic medium subjected to external mechanical loading and heating is considered. This medium consists of two phases - the porous solid and the liquid occupying the pore space. The porous solid is assumed to be isotropic, linearly elastic, and locally non-deformable (i.e. rigid grain material). It is worth noting that when drainage is allowed, the fluid pressure in the geomaterial will dissipate with time. Thus, the mechanical properties $K_D$, $G_D$ (bulk modulus, shear modulus) thermal expansion coefficient of the drained geomaterial will be equivalent to those for a porous geomaterial skeleton with empty pores. However, even when pressure is zero, the liquid is technically present in the porous fully saturated geomaterial. Therefore, the temperature field must be obtained as a solution of the heat transfer (conduction) equation for a porous medium with liquid filled pores.

The analytical solution for the one-dimensional problem was used to validate performance of the multi-physics code COMSOL. The solution was also verified using the ABAQUS finite element program. We also define a sequence of auxiliary problems: In each auxiliary problem, the temperature is applied gradually within a short period of time, starting from a zero value and reaching a maximum value $T_0$. 
When the user solves the auxiliary problems with the help of ABAQUS, the step GEOSTATIC is not needed, since the initial temperature and all initial fields are indeed zero. In fact, this step will give an incorrect solution, in which for example, the displacement is zero at time $t = 0$ for the case of incompressible fluid. Also, it is noted that ABAQUS requires the input of the void ratio $e$ instead of porosity. When solving the problem with COMSOL the user has to modify certain dialog boxes; all necessary details are given in the full paper.

The THM problem for a one-dimensional column of height 10 m was considered; the initial elevated temperature was 100°C within the column and, in addition, was acted upon by the non-zero pore pressure that builds up due to the absence of fluid drainage across the boundaries. The pressure and the temperature at the upper surface are then reduced to zero and the constant compressive stress of 10 MPa is applied at the upper surface. Results are presented for the temperature distribution through the depth of the geomaterial column at 1, 100, and 365 days after initiation of heat diffusion; the computational results obtained using the ABAQUS and COMSOL codes were in good agreement with the analytical results.

With time, the pressure dissipates inside the geomaterial with the maximum pressure attained at time $t = 0$, for the case of an incompressible fluid. The initial fluid pressure in the absence of temperature change is equal to 10 MPa (which is consistent with the undrained response of a saturated geomaterial) but with a temperature increase, the pressure reaches the value 36.3 MPa. The positive value of the initial pressure caused by temperature change alone ($(36.3\text{MPa} - 10\text{MPa})=26.3$ MPa) can be explained in the following way: Thermal expansion of the solid phase and the assumed zero thermal expansion of the fluid would lead to pore fluid tension, and thus negative pressures would develop, if the pores were free to deform in the lateral direction. However, due to constraint on lateral displacements, the resulting fluid pressure is positive, i.e., the fluid is still in compression.

The effect of heating on the deformation of a fluid-saturated porous medium was examined. The solution for the one-dimensional problem of a geomaterial column initially at a uniform temperature and pore fluid pressure subjected to subsequent heat dissipation is obtained analytically in the form of a power series expansion. In addition, the authors demonstrate the applicability of two commercial finite element codes COMSOL and ABAQUS to solve thermo-hydro-mechanical problems. The computational results for a one-dimensional problem are validated through comparison with an analytical solution and satisfactory agreement is observed.

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1 INTRODUCTION

The study of changes in the physical properties of rocks as a function of thermal cracking is a subject of widespread interest since it applies to human-induced applications such as the optimization of geothermal recovery and the safe design of used nuclear fuel depositories. It is important to see how the perturbation of the environmental stress state caused by human activities may alter the rock structure, thus leading to changes in mechanical or transport properties of the host rock. Such alterations may have undesirable consequences on the integrity of the rock mass.

The presented work was part of a research project conducted at the University of Toronto to validate bonded-particle models for thermo-mechanical problems. Validation was performed with the help of case studies in which the simulation responses were compared to the laboratory- and field-scale experimental data. The full study is presented in Wanne (2009). Particle Flow Code in two dimensions (PFC2D, Itasca. 2004) was chosen for the research because of its several benefits. The code provides a natural heterogeneity of numerical specimens due to the random particle-packing scheme. In addition, the code exhibits dynamic material behavior where seismic waves propagate at a velocity dependent on the material properties.

Tunnel sealing experiment (TSX) was conducted between 1997 and 2004 in the Canadian Underground Research Laboratory located in Manitoba. In the experiment, a tunnel was excavated parallel to the maximum principal stress direction, filled with sand and sealed with bulkheads at both ends. It was then pressurized and heated by injecting hot water to the center of the chamber. The experiment was monitored with AE/MS systems over the period of seven years (Haycox, Collins & Pettitt. 2004). Seismic activity was the highest during the excavation phase, and 95% of the events occurred within 1.4 meters from the chamber walls. The seismic activity around the tunnel indicated that time-dependent microcracking occurred for years following the excavation. A clear relationship was observed between the increase of the temperature and the increase in the microseismic events. During the heating phase, the events did not exceed beyond the initial excavation disturbance zone. It was interpreted that there was an increase in crack densities in the areas of previous damage. Moreover, the observations clearly indicated that during the heating phase, there was less cracking activity at the floor region compared to the roof.

2 SIMULATION OF TSX

The main interest in the simulation of the TSX was the damage formed during the heating phase. The large-scale PFC2D models replicated a central plane-strain section of the TSX chamber. PFC2D specimens were created based on the material genesis procedure by Potyondy & Cundall (2004). The specimens were of a rectangular shape, with a width of 28 meters and a height of 21 meters. The number of PFC2D particles in a specimen was about 19000. The properties of the Lac du Bonnet granite were based on Read et al. (1997) and Martin (1993). During the study several modeling cases were executed with varying input parameter values.
The excavation of the opening was performed by removing the bonds between the particles located in the chamber area. The particles themselves remained. The filling material was thus modeled by loose PFC2D particles. The particles settled under the force of gravity, and a small gap formed between the filling and the chamber roof. The numerical specimens were cycled to equilibrium, a process during which any microcracking was recorded. The state of equilibrium represented the TSX chamber after the excavation and filling stages.

Heating was simulated by raising the temperature of the particles in the chamber to a target value of 85°C. Since the filling material had settled under the gravity and created a small gap between the roof and the filling, the roof region was initially not directly heated because the heat could only conduct between those particles that were in direct contact. Therefore, another heating mechanism, in which the chamber perimeter particles were regarded as part of the heating range, was created. In order to simplify the simulations, pressurization of the chamber by fluid was omitted in the modeling sequences.

Thermal simulation stage simulated 12 weeks of heating. This duration was chosen in order to optimize computation time. The computational runtime per a simulation case was about 30 hours (using a PC with a processing speed of 3.0 GHz).

3 RESULTS AND CONCLUSION

In the TSX simulations, the heating and the potential effect of the filling material on the damage development in the floor region were the main concerns. The filling material was simulated by unbonded particles under gravitational loading. The weight of the free particles provided a confinement of about 100 kPa on the floor region, which was about the same as the value estimated in the in-situ experiment. The models showed damage around the opening, which was concentrated at the roof and floor. As the simulation continued, the effect of the filling material became evident, as the cracking in the roof area during the heating phase was predominant, when compared to that in the floor area. The damage extended about 1–2 meters from the tunnel perimeter outwards. In the roof area, the microcracking formed a notch-like shape. The apex of the notch reached about 1.5 meters from the tunnel wall.

Comparison between the simulation cases shows that the varying material strength is the main contributing factor determining the extent of the damage. Another, less important factor is the heating mechanism. All cases show the effect of the confinement at the floor level by a restrained damage below the tunnel. Also, parameter studies showed the sensitivity of the particle packing to the formed microcracking.

The models captured the first-order phenomena observed in-situ displaying the difference in the damage in the roof and floor regions, respectively. This difference was due to the filling material confinement of about 100 kPa on the tunnel floor, which occurred in the course of the heated experiment.

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Reasoned Argument Why Large-Scale Fracturing Will Not Be Induced by a Deep Geological Repository

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1 INTRODUCTION

The Canadian approach for long-term containment and isolation of spent nuclear fuel (termed ‘used fuel’) in a deep geological repository (DGR) relies on multiple barriers to prevent or retard the release of radionuclides to the biosphere. The system includes the host rock (or geosphere) as a natural barrier, and a series of engineered barriers placed in underground excavations in the host rock. Specific occurrences of both crystalline rock and sedimentary rock are considered potentially suitable host rock formations (NWMO 2005). These formations exhibit desirable mechanical and hydrological properties. They also cover large areas at sufficient depth below surface, and are not considered rich in mineral resources, thus limiting the potential for disturbance by erosion or accidental interception during drilling.

For the purposes of safety assessment of a DGR, the integrity of the natural barrier is assumed to remain substantially unchanged over the 100,000 year period following waste placement (i.e. the period in which release of radionuclides to the biosphere would constitute a possible safety risk). Over this time period, the host rock will experience mechanical effects from excavation and development of underground openings, thermal-mechanical effects from heat generated by the placed waste, and possible long-term mechanical effects associated with glaciation and seismicity. Taking these effects into account, a reasoned argument is presented that examines the potential for large scale fracturing or faulting induced by a DGR.

2 SYNOPSIS OF THE REASONED ARGUMENT

The Reasoned Argument (RA) presented in this study can be summarized as follows:

“The development and propagation of large-scale fractures either between repository rooms, or between the repository level and other remote natural hydraulic pathways, is improbable in the various DGR designs in each of the rock types considered. The rock properties in each case are sufficiently competent and stress conditions sufficiently benign to effectively impede the possible fracturing mechanisms discussed in this study. Specifically, the fact that the repository lies in a compressive stress field with relatively low deviatoric stresses in a thrust fault regime suggests that insufficient driving force would exist to initiate and propagate the two types of possible fractures of concern (large-scale horizontal extensile fractures, or thrust faults oriented at a shallow angle to horizontal) within the 100,000 year period following placement. A dramatic erosional event that might reduce the depth of cover by hundreds of metres could alter this conclusion, but the likelihood of such an event over the 100,000 years following placement is considered extremely remote.”

3 SUPPORTING STUDIES

The study to develop the RA (Read 2008) considers a number of possible scenarios for induced fracturing of the rock mass in response to DGR development. The study incorporates observational and experimental evidence from the Canadian and other national radioactive waste man-
agement programs, and information from literature related to rock mechanics, geology and seismology. Numerous far-field and near-field analyses (Figure 1) were completed for the study covering four used fuel containment and isolation concepts in crystalline rock (granite) and sedimentary rock (limestone and shale) at two depths and two different tunnel orientations.

Based on this study, it is concluded that the RA is supported by both the analysis results and practical observations from Canada and elsewhere. The primary conclusion from this study is that the development and propagation of large-scale fracturing either between repository rooms, or between the repository level and other remote natural hydraulic pathways, is implausible. Information reviewed as part of this study supports the RA.

The applicability of the RA to potential repository environments should be framed within the context of this study. For sites with rock mass properties or in situ conditions that differ significantly from the specific cases considered in this study, the RA provides an approach to assess the potential for large-scale fracturing. The essence of this approach is first to define strength criteria that must be exceeded for large-scale fracturing to occur (e.g. intact rock strength and slip along fractures), second to define expected stresses within the rock mass for given scenarios (e.g. excavation, heating, and glaciation), and then test the criteria using an accepted approach (e.g. Mohr circles). The RA highlights the need for thorough characterization of each potential DGR site to determine the in situ conditions, rock mass properties and the degree of variability and heterogeneity at the site.

Figure 1. Far-field analysis of a DGR in crystalline rock under glacial+thermal conditions (Read 2008).

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SESSION 10 ROCKMASS CHARACTERIZATION AND SITE INVESTIGATION I

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An Improved Definition Of Rock Quality Designation, Rqdc
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An improved definition of rock quality designation, RQDc

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1 INTRODUCTION
The rock quality designation (RQD) was initially introduced by Deere (1963) for civil engineering. But, its application has been largely extended to rock mechanics and has become a fundamental parameter in geotechnical engineering (e.g., Hoek & Brown 1980; Milne et al. 1998). The success of this definition is due, in great part, to its simplicity. However, this index includes a number of limitations. Among these limitations, a critical one is associated with the arbitrarily selected threshold value (e.g., Priest & Hudson 1976). Consequently, the RQD value would vary with different threshold length for the same core. Also, using the customarily and universally adopted but very arbitrarily selected threshold value of 10 cm (for NX cores) in the assessment of RQD, the RQD values usually tend to be either high or low in most rock engineering. Some intermediate values (50%) are less frequently encountered (Harrison 1999). The original definition of RQD does not well represent the actual quality of rock masses. In this paper, an improved definition of rock quality designation (RQDc) is given.

2 NEW DEFINITION OF RQD
The improved definition is proposed as follows:

\[ RQD_c = \frac{p_r}{f(N)} \]  

(1)

where RQDc is the new rock quality designation, \( p_r \) is the ratio of recovered cores in length (equivalent to the recovered rate commonly used by geological engineers), \( f(N) \) is a function of the total number of unbroken pieces. In this paper, the function \( f(N) \) was taken as:

\[ f(N) = N^a \]  

(2)

where \( a \) is a material parameter.

Figure 1a shows the variation of RQDc with parameter \( a \) varying from 0 to 1.0. It can be seen that RQDc remains constant if parameter \( a = 0 \). This is similar to the original definition of RQD, which is independent of the number of unbroken pieces.

Compared to the original definition of RQD, the new definition of RQDc not only keeps the original definition’s simplicity, but also describes the quality of rock masses from very bad to very good quality in a continuous manner.

3 DISCUSSION AND CONCLUSION
Using a fictive core with a uniformly distributed joint family, it can be shown that the new definition of RQDc can be expressed as the following:

\[ RQD_c = \frac{1}{JF \times L + 1} \quad \text{for} \ p_r = 100\%, \]  

(3)
Here, \( L \) is the scan interval and JF represents joint frequency (m\(^{-1}\)). This equation indicates that the RQD\(_c\) is subject to a scale (scan interval) effect, as shown in Figure 1b. It is interesting to note that the relationship shown in Figure 1b is very similar to those between other geomechanical properties (such as strengths and deformability) and rock size (e.g., Bieniawski 1968; Cunha 1990; Aubertin et al. 2000; Li et al. 2007). A correction can be made using the following equation:

\[
\frac{RQD_{cL}}{RQD_{cL0}} = \frac{JF \times L_0 + 1}{JF \times L + 1}
\]  

(4)

where \( RQD_{cL0} \) is the RQD\(_c\) with a standard scan interval of \( L_0 \) (usually 3 m), \( RQD_{cL} \) is the RQD\(_c\) with non standard scan interval of \( L \).

\[\begin{array}{c}
\text{Figure 1. Variation of RQD\(_c\) with number of pieces } N \text{ (a) and normalised scan interval } L \text{ (b) (}\rho_t = 100\%).
\end{array}\]

4 ACKNOWLEDGEMENT

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Rock strength characterization for excavations in brittle failing rock

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1 INTRODUCTION
It is hypothesized that the standard approach adopted to characterize the ground and to determine the rock strength properties at depth are flawed and generally tend to underestimate the rock strength and possibly the stiffness, particularly when the rock mass is characterized as massive to moderately jointed or blocky to very blocky. It is therefore necessary to conclusively established that the hypothesis, that the rock strength in highly confined rocks is greater than generally anticipated, is actually valid and generally applicable, and then to develop new procedures to establish rock parameters that can be reliably used for the design of deep excavation.

2 PROPOSED S-SHAPE BRITTLE FAILURE CRITERION
Kaiser and Kim (2008a, 2008b) revisited Dr. Hoek’s data base and showed that the rock strength was typically reduced to the left of a spalling limit of about $\sigma_1/\sigma_3 = 25$ to 20 (for intact rock) or typically for $\sigma_3/\sigma_1$ UCS/10.

The following Eqn (1) describes such an s-shape criterion:

$$\sigma_1 = k_2 \sigma_3 + UCS_{II} + \left[\frac{(UCS_I - UCS_{II})}{1 + e^{(\sigma_1 - \sigma_2)/\sigma_3}}\right]$$

where, $UCS_I$ is the unconfined compressive strength as determined in the laboratory, $UCS_{II}$ is the apparent $UCS$, obtained by linear back projection of a linear fit to high confinement data with slope $k_2$. The y-intercept at $\sigma_1 = 0$ represents the apparent uniaxial compressive strength ($UCS_{II}$) for the high confinement range or shear failure mode range.

Figure 1 presents an example using test data for Sandstone Darley in Dr. Hoek’s data base. In order to represent the variability the ±95 % confidence levels are plotted for the linear regression in the shear zone.

Figure 2 shows that $UCS_{II}$ is between 1 and 3-times the laboratory $UCS_I$ or on average between 1.5 and 1.7 times higher than $UCS_I$. In other words, the apparent $UCS_{II}$ for the rock strength in the highly confined zone ($>UCS/10$) is significantly greater than the $UCS$ obtained in the laboratory.

3 CONCLUSIONS
From the results, it is found that the apparent $UCS$ represented as $UCS_{II}$ is on average 50 to 60% higher than $UCS_I$ determined from laboratory test data. This is valid for a wide range of rock types. It is, therefore, suggested that the rock strength for the design of rock structure in confined and unconfined states, e.g. pillar under high confinement or near excavation at low confinement, should be obtained by fitting different regions of the s-shape criterion.
Figure 1. Result of fitting curve on s-shape criterion using the spreadsheet

Figure 2. Calculated a ratio of $\frac{UCS_{II}}{UCS_I}$ based on s-shape failure criterion for (mean: 1.56).

4 PREFERENCES


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Numerical Assessment of Factor B in Mathews’ Method for Open Stope Design

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1 INTRODUCTION

The stability graph method, developed by Mathews et al. (1980), later modified by Potvin et al. (1988), amongst others, is an empirical approach that has been developed for open stope design based on the depth of mining, rock mass quality and stope span. The stability graph is a plot of stope hydraulic radius versus the modified stability number, \( N' \), which is defined as:

\[
N' = Q'xAxBxC
\]  

where \( Q' \) = the modified Q rock mass classification (Barton et al., 1974); \( A \) = the rock stress factor; \( B \) = the joint orientation adjustment factor – Factor B; and \( C \) = the gravity adjustment factor. The Factor B is used to account for the influence of the relative orientation of dominate jointing relative to the excavation surface (stope wall or back) being assessed. The relative angle is referred to as the angle \( \beta \). The original Factor B proposed by Mathews et al. (1980) was based on expert discussion which considered change of behaviour based on the direction of loading with respect to the inclination of a plane of weakness. The Factor B curve was revised based on a larger number of case histories by Potvin et al. (1988). Figure 1 illustrates the two Factor B curves.

2 MODELLING THE FACTOR B

The program Phase\(^2\) (©RocScience, 2006) was used for the modelling and to assess the influence of various joint properties and states of stress on the shape of the Factor B curve. The purpose was to determine if a 2D continuum finite element modelling approach could account for discrete structures through the use of joint elements. An open unsupported stope of typical size for Canadian hard rock mines was modelled: 30m x 10m (height x span) dipping 80°. Models were created based on the joint orientations (\( \beta \)) of 90°, 60°, 45°, 20°, and 0° to the stope hanging wall and back.

The maximum depth of the \( \sigma_z = 0.5 \) MPa contour limit (confinement criteria chosen based on Cai et al. 1999 and Alcott & Kaiser 1999) was measured perpendicular to the hanging wall and stope back for each of the \( \beta \) angle cases analysed to recreate the Factor B curve. The measurements of the chosen confinement limit in the models were then normalized by dividing by the depth of \( \sigma_z = 0.5 \) MPa contour limit for the same stope geometry with the same material properties but containing joints oriented 90° to the stope wall. This process can be summarized by the following equation:

\[
\text{Numerical Factor B} = \left( \frac{d_{\sigma_z\beta}}{d_{\sigma_z90}} \right)^{-1}
\]  

where \( d_{\sigma_z\beta} \) is the depth of \( \sigma_z = 0.5 \) MPa contour limit for different \( \beta \) angles and \( d_{\sigma_z90} \) is the depth of \( \sigma_z = 0.5 \) MPa contour limit for the baseline finite element model where the joints are orientated 90° to the stope face being assessed. The results are normalized in this manner be-
cause joints oriented 90° to the stope face are assumed to have little to no influence on stability (Mathews et al., 1980 & Potvin et al., 1988).

A set of numerically derived Factor B results for the stope hanging wall are presented on Figure 1 and can be divided into two sections; (1) between 0° and <45° where the numerical curve did not match the empirically derived curves based on the confinement criteria alone; and (2) between 45° and 90° where the numerical curve matched the empirically derived curves based on confinement change around the stop boundary due to changing joint orientation. In order to match the B-curve at the lower β angles, GSI for the near boundary material had to be reduced from 80 to 30 as shown on Figure 2.

Taking both the confinement on joints and the strength of the rock mass surrounding the stope boundary into account, the Factor B curves proposed by Mathews et al. and Potvin et al. were reasonably reproduced numerically for the stope hanging wall.

Figure 1: Example set of modelling results for joint spacing = 2m, Kn/Ks = 10, k_o = 1, and φ = 40°.

Figure 2: Influence of GSI on the shape of the Factor B curve. Joint spacing = 2m, Kn/Ks = 10, k_o = 1, and φ = 40°.

3 ACKNOWLEDGMENTS

The main Author would like to thank: MIRARCO; the mining group at Golder Associates Ltd.; and Peter Kaiser for his time and patience. NSERC also deserves mention for the financial support provided through Dr. Kaiser and Trevor Carter for feedback regarding this paper.
Constitutive model for small rock joint samples in the lab and large rock joint surfaces in the field

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1 INTRODUCTION

A new constitutive model for rock joints is proposed for predicting the mechanical behavior of both small joint samples in the lab and large joint surfaces in the field. The normal and shear behavior of joints samples is predicted based on the strength and geometry of small-scale joint asperities. The behavior in the field is predicted based on the strength of small-scale asperities, determined from lab data, and the geometry of field-scale waviness determined from geologic observations.

The concept of available shear strength is introduced to describe the degradation of asperities and the shape of the mobilized shear stress-displacement curve.

Dilation and roughness degradation during shear, resulting from grinding, breaking, and crushing of asperities, is correlated to a dimensionless product of shear stress, incremental shear displacement, rock strength, and wavelength of irregularities, which fits direct shear tests results on natural joints and replicas of natural joints. The normal stress-joint closure behavior of rock joints is predicted based on the crushing analysis of asperities during normal loading and fitting available test results on fresh and weathered natural joints. The proposed model is fully incremental; hence it is not limited to monotonic loading.

The behavior of joints in the field is predicted by applying the model for lab scale samples to the small contact areas developed in large-scale joint surfaces. An equation for the contact area of joints in the field is derived, based on both the geometry of large-scale waviness and the normal stress-joint closure relation, which allows estimating actual contact stresses and degradation of asperities.

2 INPUT PARAMETERS

- Basic friction
  $\phi_c$: Basic friction angle - residual
- Lab-scale asperities
  $\alpha$: Initial average asperity angle
  $\lambda$: Asperity wavelength
- Field-scale waviness
  $\sigma_c$: Compressive strength
  $\lambda_w$: Waviness wavelength

Figure 1. (a) Parameters of proposed model (b) Available and mobilized friction at main shear stages.
3 SIMULATIONS OF DIRECT SHEAR TESTS

The proposed model was verified using experimental data from the literature that provides detailed stress-displacement curves, material parameters and joint profiles. The proposed model provides a very good correlation with the experimental data.

![Graphs showing comparison between proposed model and experimental results (Bandis, 1980)](image)

**Figure 2.** Comparison between proposed model and experimental results (experimental data from Bandis, 1980)
ABSTRACT: A complex network approach on a rough fracture is developed. In this manner, some hidden metric spaces (similarity measurements) between apertures profiles are set up and a general evolutionary network in two directions (in parallel and perpendicular to the shear direction) is constructed. Also, an algorithm (COmplex Networks on Apertures: CONA) is proposed in which evolving of a network is accomplished using preferential detachments and attachments of edges (based on a competition and game manner) while the number of nodes is fixed. Also, evolving of clustering coefficients and number of edges display similar patterns as well as are appeared in shear stress, hydraulic conductivity and dilation changes, which can be engaged to estimate shear strength distribution of asperities.

1 INTRODUCTION

Understanding of rock joint behaviors, either in single or mass form, under the several natural or artificial forces has been the subject of numerous researches during the evolution of rock mechanics field. Rock joint performance as a result of collective behavior of constructed elements (say fraction/pixel in each surface), interacting with each other, determines nonlinear picture of a changeable system. Obviously, one cannot predict the rich behavior of the whole by merely extrapolating from the treatment of its units (Boccara 2004, Vicsek 2002, Hakan 1989). Absent of fully prediction and nonlinear essences of behavior are prevalent gesture of complex systems. The term complex system formally refers to a system of many parts which are coupled in a nonlinear fashion.

When there are many non-linearities in a system (many components), behavior can be highly unpredictable. Complex systems research studies such behavior. Complex systems research overlaps with nonlinear dynamics research, but complex systems consist of a large number of mutually interacting dynamical parts. The success in describing of interwoven systems using physical tools as a major reductionism is associated with the simplifications of the interactions between the elements so that complexity reduction is a rescue pathway to regulation of approximated analyses of collective particles having swing states, complicated structures, and diversity of relations among elements. Complex networks have been developed in the several fields of science and engineering for example social, information, technological, biological and earthquake networks are the main distinguished networks (Albert & Barabasi 2002, Abe & Suzuki 2006). On the other hand, to catch on Hydro-mechanical and mechanical behavior of a rock joint, domination on to the surface morphology and its evolution as well as aperture is irrefutable. In addition to these procedures, the mechanical properties and hydraulic conductivity of the being joint are compared with the network properties. Upon this comparison, the distribution of shear strength –for each profile- is estimated.
Figure 1. The evolution of aperture patterns under successive shear displacements and 3 MPa normal stresses (the axis shows number of elements with a square element size of 0.2 mm).

Figure 2. The evolution of X-profiles networks (adjacency matrix visualization) using Euclidean distance and $d \leq 5$ - (white occupations: connected elements).

Figure 3. Frequency of nodes connectivity evolution over the shear displacements on: a) X-profiles and b) Y-profiles (Insets: results in log-log coordinate).
SESSION 11 INNOVATION IN GROUND SUPPORT AND INSTRUMENTATION I

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Instrumentation Of A Graphite Zone In The #3 Shaft At Brunswick Mine
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P. Mercier & R. Harrisson
Xstrata Zinc – Brunswick Mine, Bathurst, New Brunswick, Canada

4022
The Performance Of Mesh, Shotcrete And Membranes For Surface Ground Support
E.C. Morton, A.G. Thompson and E. Villaescusa
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Dynamic Testing of Friction Rock Stabilisers
J.R. Player, E. Villaescusa & A. Thompson
Western Australian School of Mines, Kalgoorlie, West Australia, Australia

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Numerical Modeling Of The Coupled Thermo-Chemo-Mechanical Response Of Cemented Paste Backfill Structures In Deep Mine Temperatures Conditions
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Instrumentation of a Graphite Zone in the #3 Shaft at Brunswick

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1 INTRODUCTION

Xstrata Zinc’s Brunswick Mine is an underground zinc mine situated near Bathurst, New Brunswick. The main shaft, the #3 Shaft, is critical to the life of the mining operation. The shaft intersects a soft graphite zone at a depth of approximately 1,050m below surface, just below the 3400 Level. Ground movements around the graphite zone cause the shaft’s concrete liner to be displaced towards the shaft’s interior. A particularly critical 3.0m by 5.5m area has been identified in the south wall of the shaft. This area encompasses a concrete wedge that is being pushed into the shaft by the material behind and requires monitoring.

2 APPROACH

Several challenges were identified. As the shaft contains a number of moving conveyances, instrumentation components in the shaft must not interfere with any of them. Any instrument installed in the shaft cannot cross the shaft’s interior. It was also desired for the system to avoid the use of point-measurement-type devices, which are only capable of detecting movement at a single point. The instrumentation called for the use of a transducer capable of providing area coverage. Finally, the installation of an instrument had to be fairly simple and quick. The installation was to proceed from the top of the cage where space is limited.

Taking into account the restrictions posed by the situation, Itasca Canada developed a unique surface-mounted extensometer. This instrument consists of a transducer and a cable that is designed to detect displacements. The movement-detecting cable can be virtually of any length. The extensometer is designed to be attached to rock surface using concrete fasteners. The principle of operation is illustrated in Figure 1. The transducer enclosure and the free end of the detection cable are attached outside the zone of potential movement. As movement occurs, the cable transmits movement to the transducer.

Figure 1. Sketch showing the principle of operation of the surface-mounted extensometer.
3 INSTALLATION

The conceptual design of the instrumentation system for the monitoring of the #3 Shaft at Brunswick Mine is shown in Figure 2. The system was designed to have four surface-mounted extensometers, each fitted with a 12m-long detection cable. The four instruments were installed inside trenches cut in the shaft’s liner. The installation of the four instruments took approximately 8 hours to complete by a team of two people.

4 PERFORMANCE

The analyses of extensometer data indicate that the new instruments are performing well. Variations in the readings are in the order of 1-3 mm, which is approximately 0.5% to 1.0% of the instruments’ movement range. These can be attributed to several factors. The instability of the extensometer excitation voltage may be one of the reasons (changes in this voltage will directly cause changes in the readings). Another reason for the variation may be the movement of the conveyances inside the shaft. For example, as the cage travels up and down the shaft it causes air in the shaft to move as well. Air moving along the shaft wall where the instruments are installed is capable of exerting pressure onto the extensometer cables and deflecting them, thus causing slight variations in the readings.

5 CONCLUSIONS

Measuring displacement in shaft walls is not a trivial exercise. The usage of conventional transducers may be limited by conveyance movements inside the shaft and by the space and time required to carry-out the preparation and/or the installation of the transducers.

A novel surface-mounted extensometer has been developed for installation in the #3 Shaft at Brunswick Mine. The instrument is rugged and therefore well-suited for long-term mining applications. The detection cables can be made of any reasonable length. The installation of four such extensometers at Brunswick Mine has shown that they require minimal preparation prior to installation and are easy to install. All four instruments have so far been performing well.

At present, the authors see the primary use for this extensometer in monitoring displacements on surfaces, such as in underground openings (backs and walls), pillars, and shotcrete posts.
The performance of mesh, shotcrete and membranes for surface ground support

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1 BACKGROUND

The Western Australian School of Mines (WASM) has undertaken a test program on mesh and sprayed layers (shotcrete and membranes) at its recently developed Static Test Facility. The aim of the program was to develop a method to enable the testing of all surface ground support products under realistic loading scenarios. The static test facility, shown in Figure 1 set up for a shotcrete test, comprises two steel frames; a load bearing, upper frame used to provide a loading reaction for the screw feed jack and a lower frame used to support the sample. The screw feed jack is driven at a constant displacement. The static testing results will be complemented by the results obtained by subjecting similar sample configurations to dynamic loading in the WASM Dynamic Test Facility.

2 TESTING PROGRAM

The testing program involved:
- two different types of mesh; a welded wire mesh and a woven, high strength wire mesh.
- punch tests on three different fibrecrete mixes; two with plastic fibres and one with steel fibres.
- two different tests on a single membrane product; bending of simulated discontinuous rock and a punch test.

![Figure 1. WASM Static Test Facility.](image-url)
The punch test was developed to involve the adhesion, shear and bending resistance of the sprayed layers (shotcrete and membrane products). The samples for the punch test were prepared by spraying the product onto a layer of sandstone into which an isolated, centrally located circular disc had been cut.

3 TEST RESULTS

The results from the mesh testing showed that the woven, high strength steel wire has significantly higher force, displacement and energy absorption capacities.

The results from the fibrecrete testing were more difficult to compare directly due to variations in both the thicknesses of the layers and age at which the samples were tested. There did not appear to be a relationship between rupture force and the displacement at which primary failure occurred. This may be related to the strain profile in a slab subjected to lateral loading and the critical tensile strain at which the brittle material fails. However, as expected from theoretical considerations, there was an approximate rupture force increase proportional to the square of the layer thickness. The plastic fibres were more securely held in the shotcrete matrix and were observed to fray; no steel fibres exhibited tensile failure and were able to pull out after straightening of the deformed ends. The steel fibres exhibited corrosion after being exposed to the atmosphere across cracks formed during testing.

The tests on the membrane product showed that it exhibited failure characteristic of a brittle material rather than a more ductile response required for it to be effective in a thin layer. That is, it behaves similarly to a very thin layer of shotcrete. Interestingly, the rupture force decreased as the layer thickness was increased. This may be attributed again to the critical tensile strain. Unlike fibrecrete, the residual capacity is not enhanced with internal fibre reinforcement.

Figure 2 provides a comparison of the force-displacement responses for some of the tests and clearly shows the differences in performance of the various support materials.

![Figure 2. Comparison of performance of weld mesh, shotcrete and a brittle membrane product.](image)

4 PRACTICAL IMPLICATIONS

Shotcrete, has a limited displacement capacity when compared with mesh. While shotcrete has the ability to provide immediate support to a rock mass and inhibit loosening, in high deformation environments the limited displacement capacity of shotcrete systems may result in a loss of effectiveness. In these circumstances, shotcrete surface support must be complemented with mesh that has much higher displacement capacities. Membranes that are formed from brittle materials will not be effective as surface ground support.
Dynamic Testing of Friction Rock Stabilisers

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**ABSTRACT:** A rough simulated borehole has been developed for the installation and dynamic testing of rock bolts at the WASM Dynamic Test Facility. The rough simulated borehole is used for rock bolts that are sensitive to equipment installation technique or borehole geometry (particularly friction rock stabilisers and resin encapsulated bolts). An extensive axial test program has quantified a difference between the static and dynamic capacity (kN/m of embedment) for friction rock stabilisers once sliding starts. The 47mm split tube bolt has a 50% reduction in embedment capacity (but with significant variation) dissipating 2.7kJ/100mm of sliding per metre of embedment, and the Omega (inflatable) bolt had a 70% reduction in embedment capacity dissipating 6.2kJ/100mm of sliding per metre of embedment. The Omega bolt dynamic capacity remained at least equivalent to the split tube bolt static capacity.

1 **INTRODUCTION**

The Rock Mechanics Group at the Western Australia School of Mines (WASM) commenced development, design and construction of a unique dynamic testing facility in Kalgoorlie during 2002 (Player et al., 2004). The facility is designed to quantify the force-displacement responses of reinforcement and support systems subjected to dynamic loading. Reinforcement system tests are undertaken on full scale double embedment samples, with the simulation of important parameters that are expected to affect the performance of the reinforcement system.

A wide range of reinforcement and support systems have been tested and the results reported by Player et al. (2008a, 2008b and 2008c) and Varden et al. (2008). Particularly it has been found that it is important to test the reinforcement with sufficient input energy to cause failure on the first loading and to report the critical states of energy dissipation, displacement, velocity and acceleration.

It is the testing philosophy at the WASM Dynamic Test Facility that we are establishing failure criteria for reinforcement system subject to axial dynamic loading. This is achieved by the establishment of an index test that catalogues reinforcement system performance.

The paper details what constitutes a friction stabiliser, how they function statically and reports their performance under dynamic loadings. The testing mechanism for applying a dynamic load to a reinforcement system is detailed, and this is related to the way in which the load is applied at the WASM Dynamic Test Facility and the importance of the instrument along with the data gathered is explained.

A significant time and technical challenge of the testing program was the construction and evaluation of the rough simulated bore holes by borehole profiling and static pull tests on the friction stabilisers. These bore holes are important in that they allow reinforcement systems that are sensitive to rock conditions and / or installation practices (operator or equipment) to be well evaluated as the bolt is installed by the equipment that would normally perform the task. They have been used for the dynamic testing of friction stabilisers and resin encapsulated bolts.

2 **RESULTS**

Figure 1 has the comparative axial static versus dynamic performance for the 47mm diameter split tube bolt and the Omega bolts. The Omega bolt even when sliding has an equivalent or greater capacity than the static capacity for the split tube bolt. The Omega bolt and split tube bolt have a reduce resistance to sliding on repetitive loadings. The split tube bolt shows a greater variation in performance than the Omega bolt.
2.1 47mm Split Tube Bolt

The 47mm split tube bolt should be expected to have an axial static capacity of 50kN/m of embedment, with a reduction of 30-25kN/m as the bolt slides under dynamic load. This can be calculated to an average energy dissipate of 2.7kJ (sd+/-1.0) per 100mm of slip normalised to per metre of embedment (the standard deviation is reported in brackets). This can be compared to Ortlepp and Stacey (1998) for 39mm split tube bolt projected at 1kJ/100mm of sliding for one metre of embedment, although the authors noted a low level of confidence in the result.

2.2 Omega Bolt

Omega bolt (equivalent to Swellex MN24) expect an axial static capacity of 160kN/m of embedment and a reduction to 70kN/m of embedment that will reduce to 50kN/m as it slides under dynamic load. This can be calculated to an average energy dissipation of 6.2kJ (sd+/-1.3) per 100mm of slip normalised to per metre of embedment. This can be compared to Ortlepp and Stacey (1998) for Swellex PM12 bolts of 3.0-5.8kJ/100mm of sliding with 1.4m of embedment. In Figure 21, bolt 110 has not been included in the average Omega bolt performance as it was as significant outlier and was not only axially loaded.

3 ACKNOWLEDGEMENTS


4 REFERENCES


Numerical modeling of the coupled thermo-chemo-mechanical response of cemented paste backfill structures in deep mine temperatures conditions

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The cemented paste backfill (CPB) is an engineered mixture of tailings from the processing operations of the mine, water and binders (2% to 7% by weight usually). It is commonly used for underground mine support and tailings disposal. It receives great interest as one of the most commonly used ways in mine backfilling around the world.

Mechanical stability and economical performance represent important performance criteria for CPB. Once placed, CPB has to satisfy certain mechanical stability requirements to ensure a safe underground working environment for all mining personnel. As a structural element, the mechanical stability of the CPB is mostly evaluated in practice based on its unconfined compressive strength (UCS). Knowing the time at which the CPB reaches its reasonable strength is very important for reducing the mining cycle (i.e. higher productivity) and ensuring the safety of mine workers. Binder consumption is the factor that has the most significant influence on the cost or economical performance of CPB.

Despite extensive use of the technology of CPB, many fundamental factors affecting the design of safe and economical CPB structures are still not understood well. Among these aspects, the effects of in situ (especially deep mine temperature) and/or backfill temperatures on the above performance properties of CPB structures are not well known. There is a need to increase our understanding of the impacts of high temperature on the performance of CPB structure and of the development of tools to assess and predict them. Indeed, as a cemented material, CPB strength (mechanical factor) is time and temperature (thermal factor) dependent, and a function of the degree of hydration (chemical factor: cement chemical reaction). Furthermore, the CPB structures can be subjected due to various thermal loading conditions during its life service such as the deep mine rock temperatures, the heat generated by the cement hydration, heat added to CPB during its preparation, the temperature of the mixing waters.

In consideration of the facts that are mentioned above, a research program has been conducted at the University of Ottawa to study the coupled effects of temperature (thermal factor, T), binder hydration (chemical factor, C) and mechanical loadings (mechanical factors, M) on the performance (e.g. mechanical stability, binder consumption) of CPB structure and to develop a model to predict the TCM behaviour of CPB structure. In this paper, a thermo-chemo-mechanical model is developed (implemented into FLAC software and validated) for predicting strength development and distribution within undrained hydrating CPB structure, temperature development and distribution within the CPB structures, heat transfer between CPB structures and deep mine rock temperatures. The validation tests show good agreement between the predicted and experimental (field and laboratory) results. The developed tool is then used to simulate the performance of CPB structure in several practical cases of deep mine backfill operations. Valuable results were obtained regarding the thermal-chemical-mechanical response of CPB structure and its optimal design taken into account the thermal loading conditions. The simulation results have shown the temperature has a significant effect on the thermal, chemical (binder hydration) and mechanical (strength) response of the CPB structure. Binder content, filling rate (e.g. Figure 1), stope size and shape ratio, initial CPB temperature (e.g. Figure 2) and CPB specific heat have a significant impact on the heat development and distribution within CPB structures as well as on the strength development of CPB. The developed tool can contribute to more cost-effective and safer design of CPB structures in deep mine temperature conditions.
Figure 1. Temperature distribution during the filling process for rate of backfilling 5m/day for big stope of 30 x 60 m.

Figure 2. UCS development for different initial CPB temperatures (mix characteristics per Célestin, 2008). (CPB surrounding by the rock mass in pink color).

References
SESSION 12  DEEP UNDERGROUND NUCLEAR WASTE REPOSITORIES  II

4253
Overview of Ontario Power Generation’s Proposed L&ILW Deep Geologic Repository Bruce Site, Tiverton, Ontario
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The Role of Rock Engineering in Developing a Deep Geological Repository in Sedimentary Rocks
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Kenneth Birch
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4154
Geotechnical Characterization of a Sedimentary Sequence for a Geological Repository
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Regional-Scale Paleoclimate Influences On A Proposed Deep Geologic Repository In Canada For Low And Intermediate Level Waste
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Overview of Ontario Power Generation’s Proposed L&ILW Deep Geologic Repository Bruce Site, Tiverton, Ontario

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ABSTRACT
The Nuclear Waste Management Organization on behalf of Ontario Power Generation (OPG) in 2006 initiated a multi-year program of geoscientific studies to confirm the suitability of the Paleozoic sequence beneath the Bruce site, near Tiverton, Ontario, for development of a proposed Deep Geologic Repository (DGR) for its Low and Intermediate Level nuclear Waste (L&ILW). This paper provides an overview of the DGR project and on-going regional and site-specific geoscience investigations.

1 INTRODUCTION
The Nuclear Waste Management Organization (NWMO) on behalf of Ontario Power Generation (OPG) is conducting multi-disciplinary geoscientific studies at the Bruce site to confirm the suitability of the underlying Paleozoic sequence for development of a proposed Deep Geologic Repository (DGR) for Low and Intermediate Level Radioactive Waste (L&ILW) from OPG owned nuclear generating facilities (Figure 1). An Environmental Assessment for the proposed DGR is currently underway in accordance with the Canadian Environmental Assessment Act. Bruce site, situated 225 km northwest of Toronto on the eastern shore of Lake Huron, is underlain by an 850 m thick sedimentary sequence of Cambrian to Devonian age near horizontally bedded, weakly deformed shales, carbonates and evaporites of the Michigan Basin. Within this sedimentary pile, the proposed DGR would be excavated within the low permeability argillaceous limestone Cobourg Formation at depth of 680 m, which is overlain by 200 m of upper Ordovician shale formations.

A key aspect of the DGR Safety Case is the integrity and long-term stability of the sedimentary sequence to contain and isolate L&ILW at timeframes on the order of 1 Ma. Early in the project, geoscientific studies that considered regional and site-specific public domain data sets indicated favourable geologic conditions for implementation of the DGR concept (Golder, 2003; Mazurek, 2004). In 2006, site-specific investigations were initiated following the development of a Geoscientific Site Characterisation Plan (GSCP) (Intera 2006; 2008). The GSCP represents a stepwise 4-year, multi-phase program of geoscience investigations that describes site-specific field and laboratory investigations to further develop and test the existing geoscientific knowledge of sub-surface conditions as they relate to understanding geosphere stability and evolution, engineered repository systems design, and long-term DGR safety.
This paper provides an overview of the DGR project and the on-going implementation of the GSCP.
Figure 1: Artist Rendering of Proposed Ontario Power Generation Deep Geologic Repository at the Bruce site, Tiverton, Ontario.

Figure 2: Illustration of Bruce site Descriptive Geosphere Model
The Role of Rock Engineering in Developing a Deep Geological Repository in Sedimentary Rocks

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1 INTRODUCTION

Considerable research has been conducted in Canada on concepts to contain and isolate used nuclear fuel in crystalline rock, focused largely on granitic rock of the Canadian Shield. Other countries are considering crystalline rock, volcanic rock, sedimentary rock, and salt as possible host media for a deep geological repository (DGR). A review of used fuel isolation concepts and management strategies (NWMO 2005) recommended that both crystalline rock and sedimentary rock should be considered as potential host media for a DGR in Canada.

2 ROCK ENGINEERING ASPECTS OF REPOSITORY DEVELOPMENT

Rock engineering considerations and issues related to development of a DGR in crystalline rock have been investigated in Canada, Sweden, and other countries. Andersson et al. (2000) provide a summary of geoscientific suitability indicators and criteria for siting and site evaluation in Sweden, with particular emphasis on host rock requirements for a KBS-3 repository. Based on SKB’s experience over many years of research and development, the report details required and preferred characteristics and conditions in the rock, and criteria to be used to identify potential candidate sites, and during site investigation at each candidate site. The reported requirements, preferences, and criteria provide the technical basis for SKB’s site selection process and site investigations. The results, and particularly the stipulated criteria, apply to a KBS-3 repository for used fuel, where the fuel is contained in copper canisters embedded in bentonite clay at a depth of 400 to 700 m in the Swedish crystalline basement. If the repository concept is changed or if new technical or scientific advances are made, certain requirements, preferences, or criteria may need to be revised. Andersson et al. (2000) emphasize that findings of the report cannot be used directly as a basis for siting other types of repositories, or in other geological settings.

Martin et al. (2001) synthesize important rock mechanics findings from the Canadian and Swedish research programs, and identify their relevance in assessing the stability of underground openings. The report draws heavily on published results from SKB’s ZEDEX Experiment in Sweden and AECL’s Mine-by Experiment in Canada, and incorporates examples from mining and tunneling to illustrate the application of these findings to underground excavations in general. The report describes the role rock engineering can play in siting and constructing a KBS-3 repository. The key rock mechanics parameters to be determined in order to facilitate repository siting and construction in crystalline rock are identified. Possible construction issues associated with rock stability that may arise during excavation of underground openings of a KBS-3 repository are discussed. The report provides a convenient reference document for major rock mechanics issues to be addressed during siting, construction and closure of a nuclear waste repository in hard crystalline rock in Sweden.

Read & Chandler (2002) describe the Thermal-Mechanical Stability Studies (TMSS), a comprehensive multi-disciplinary research project conducted in Canada between 1996 and 2001. The goal of the TMSS was to develop a suite of engineering tools and techniques to facilitate design of stable repository excavations with minimal excavation damage. The TMSS project advanced the state of knowledge in a number of areas including: numerical modeling of progressive failure and damage development around underground openings using the Particle Flow...
Code (PFC) and other micro-mechanical models; monitoring rock mass response to excavation using conventional instruments in conjunction with acoustic emission/microseismic (AE/MS) monitoring technology and methods; and characterizing rock properties and rock mass response under ambient and elevated temperature through specialized in situ and laboratory characterization and testing methods. The role of thermo-poroelasticity of the rock mass and pore pressure analyses in rock mass stability calculations were also assessed.

Read & Chandler (2002) summarize findings from the TMSS in the context of DGR rock mechanics studies, and assess the state-of-the-art in repository excavation design tools and capabilities. They also define a systematic design approach that integrates characterization, monitoring, and numerical modeling tools and capabilities into an engineering system for back analysis and forward prediction of short- and long-term rock mass responses, and associated changes in material properties. Although the focus of the report is a DGR in crystalline rock in Canada, many of the findings are applicable to other host rock environments and applications.

3 ROLE OF ROCK ENGINEERING IN DGR DEVELOPMENT

This paper summarizes the findings of a study conducted for the Nuclear Waste Management Organization (NWMO) in Canada (Read 2008). The purpose of this study was to provide an overview of the role of rock engineering in the siting, design and construction of a DGR in sedimentary rock. The review also considered the data, tools, and techniques required to advance DGR development. Based on the study, it is concluded that rock engineering will play an important role in each stage of DGR development, and during post-construction. The study provides a basis for prioritizing the various rock engineering aspects of DGR development, and initiating research and international collaboration to advance capabilities and information required for the siting, design and construction of a DGR in sedimentary rock in Canada.

REFERENCES


Geotechnical Characterization of a Sedimentary Sequence for a Geological Repository

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**ABSTRACT**

Ontario Power Generation (OPG) is currently conducting site characterization program for a Deep Geologic Repository (DGR) for the long-term management of operational Low and Intermediate Level Radioactive Wastes at the Bruce Nuclear site near Tiverton, Ontario. The proposed repository will be located at approximately 680 m depth in a sedimentary sequence of carbonates, shales and evaporites. The host horizon will comprise argillaceous limestones of Cobourg Formation underlain by the Sherman Fall Formation. To support the understanding of the geomechanical properties of Paleozoic bedrock formations in southern Ontario and the regional rock stress conditions around the Michigan Basin, information from over 700 geomechanical test measurements collected at 29 sites and in-situ stress measurements from 25 sites in southern Ontario and northern U.S. were assembled and reviewed.

1 INTRODUCTION

Ontario Power Generation (OPG) is proposing the development of a Deep Geologic Repository (DGR) for the long-term management of Low and Intermediate Level Radioactive Waste (L&ILW) from OPG owned nuclear generating facilities. A Site characterisation program is currently underway to determine the suitability of the Bruce Nuclear Site as the location to construct the underground repository. As part of the site characterization work, information regarding the geomechanical properties of the sedimentary formations intersected at the DGR and regional in-situ stresses was assembled and reviewed. This compilation of available rock strength and in-situ stress data from Southern Ontario and surrounding Great Lake region was used to establish input parameters for preliminary engineering analyses of the DGR facility. These parameters will be verified or modified by data from on-going site-specific field and laboratory investigations. The Bruce site Geoscientific Site Characterisation Plan and the activities associated with the L&ILW DGR work program are described in detail by Jensen et al. (2007).

This paper provides a summary of the compilation of the geomechanical rock properties for Ordovician rock formations relevant to the DGR concept as they occur in southern Ontario, and on in-situ rock stresses within the Appalachian and Michigan Basins. This work is a portion of a much larger database on the subject collected by OPG as a part of the DGR project. Figure 1 shows the stratigraphy of bedrock formations beneath the proposed DGR site.
Figure 1. Bedrock stratigraphy with deep boreholes DGR-1 and DGR-2
Regional-scale paleoclimate influences on a proposed Deep Geologic Repository in Canada for low and intermediate level waste

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1 INTRODUCTION

A Deep Geologic Repository (DGR) for Low and Intermediate Level (L&IL) radioactive waste has been proposed by Ontario Power Generation (OPG) for the Bruce site on the shore of Lake Huron near Tiverton, Ontario. The DGR is to be excavated at a depth of approximately 680 m within the argillaceous limestone of the Ordovician Cobourg Formation. In order to reasonably assure safety of the radioactive waste at the site and to better understand the geochemistry and hydrogeology of the formations surrounding the proposed DGR. This numerical modelling study provides a framework to investigate the groundwater flow system as it relates to and potentially affects the safety and long-term performance of the DGR.

In order to capture and recreate the regional-scale groundwater system, in both near-surface and deep environments, a groundwater flow model is developed, using FRAC3DVS-OPG (developed from FRAC3DVS (Therrien et al. 2004)), for a fully three-dimensional representation of the bedrock stratigraphy within a portion of south-western Ontario centered on the Bruce DGR site. Pore water in the deeper zone is thought to be stagnant and has high total dissolved solids (TDS) concentrations that can exceed 300 g/L with a corresponding specific gravity of 1.2 for the fluids.

2 MODEL DEVELOPMENT

The regional scale domain, occupies an aerial extent of approximately 18,775 km². The domain was discretized into slices with 27,728 nodes each, which were then used to create quadrilateral elements. Based on an areal discretization with 200 rows and 200 columns, these quadrilateral elements have sides of 762.8 m in the East-West direction by 900.9 m in the North-South direction. Each of the 31 units from the geological reconstruction was assigned a model layer so that the numerical model would closely resemble that of the geological framework model. This resulted in 31 layers in the numerical model. The elevation of the nodes for each slice were determined from the GLL00 geological framework model described in Frizzell et al. (2008). Each geologic interface in the geological framework model, representing the top of a geologic formation or unit, is comprised of a triangulated surface mesh. A computer script was written to interpolate the elevation of the slice nodes from the appropriate layer of the geological framework model.

In the absence of a source that can generate salt and hence total dissolved solids, the simulation of density-dependent flow using coupled flow and transport equations requires a transient analysis. The initial equivalent freshwater head distribution for the analysis was determined as the steady-state solution of density-independent flow subject to the same flow boundary conditions as that of the transient analysis.

After 1 million years, the model, having been allowed to reach pseudo-equilibrium, produces salinity profiles that are compatible with the geological framework, boundary conditions and hence the flow domain. The western portion of the domain, because of the absence of a velocity to transport the brine from the system, will maintain a high salinity concentration. The location of the proposed DGR repository is located within this area. At such a location, stagnation of the groundwater is expected due to both the low permeability of the Ordovician units and the effect that density will have on reducing the flow velocity.

The equations that describe the impact of glaciation and deglaciation on groundwater pressures and flow can be simplified by assuming that ice loads are areally homogeneous in which case, the
lateral strains are zero. The assumption is valid for cases where the speed of advance and retreat of the glacier is fast relative to the horizontal flow velocity in the groundwater system. For this case of purely vertical strain and following the development of Neuzil (2003), the density-dependent flow equation becomes:

\[
\frac{\partial}{\partial x_i} \left[ K_{ij} \left( \frac{1}{1 + \gamma_m C} \left( \frac{\partial h}{\partial x_j} + [c_w (p - p_0) + \gamma C] \eta_j \right) \right) \right] = S_s \frac{\partial h}{\partial t} - S_s \zeta \frac{\partial \sigma_{zz}}{\partial t}
\]  

(1)

where \( \sigma_{zz} \) is the vertical stress. The one-dimensional loading efficiency, \( \zeta \), is a function of Poisson’s ratio for the rock, the drained bulk modulus of the porous medium, the modulus of the solids and the porosity. Values for the one-dimensional loading efficiency vary between zero and one. The hydraulic conductivity of frozen porous media is assigned the value of \( 1.6 \times 10^{-3} \) m/year (5 \( \times \) \( 10^{-11} \) m/s) and is assumed to be isotropic (McCauley et al. 2002).

The effects of long-term climate change (e.g. permafrost) on the groundwater flow system are investigated by modifying the permeability of rock within the permafrost zone, by changing the surface boundary conditions to reflect a glacial scenario, and depending on the loading efficiency (refer to Equation (1)), by the inclusion of a pressure modifying term in the flow equation.

Using the deterministic University of Toronto Glacial Systems Model (GSM) of continental ice-sheet evolution, Peltier (2008) focuses on eight of the models of the ensemble that span the apparent range of model characteristics that provide acceptable fits to the totality of the observational constraints. Of the models, nn9930 was chosen for the paleoclimate analysis in this paper.

Zero flux Neumann boundary conditions were used for the lateral and bottom surfaces of the model domain. A Dirichlet boundary condition was applied to the upper surface. The ice loading was assumed to be applied as an equivalent freshwater head equal to the normal stress imposed by the ice sheet upon the domain. It was also incorporated as a pressure modifying term throughout the domain that, with the assumption of vertical strain and homogeneous loading, approximates the impact of the applied load on the rock. As described in Equation (1), this term includes a loading efficiency \( \zeta \); a loading efficiency of zero (\( \zeta = 0 \)) and a loading efficiency of one (\( \zeta = 1 \)) were applied.

3 CONCLUSIONS

This paper demonstrates the importance of including hydromechanical coupling for paleoclimate simulations. Simulation results show that hydromechanical coupling reduces vertical pore water gradients during glacial loading and unloading, thereby reducing fluxes into or out of the subsurface. Paleoclimate simulations that do not account for hydromechanical effects on pore water pressures can significantly overestimate the vertical gradients, thereby enhancing migration of surface waters deep into the subsurface environment. Rock compressibilities also affect calculated storage coefficients which allow elevated pore pressures generated during glacial loading to remain, and slowly dissipate once the glacial episode has ended.

REFERENCES


SESSION 13  HAZARD AND RISK ASSESSMENT

4019
Investigating Factors Influencing Fault-Slip in Seismically Active Structures
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Research To Reality: Application Of Mining-Induced Seismic Hazard Maps.
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4028
Re-entry Protocols for Seismically Active Mines Using Statistical Analysis of Aftershock Sequences
J.A. Vallejos & S.M. McKinnon
Queen’s University, Kingston, ON, Canada

4275
Characteristics Of Wenchuan Earthquake And Its Geological Hazard Effects
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ABSTRACT: As Canadian mines progress to greater depths, magnitudes of induced stresses increase, becoming high enough to cause rock mass damage and failure during excavation. In addition, the strain energy accumulated in pre-existing major fault and shear zones can often trigger severe rockbursts. Based on the magnitude of microseismic events and rockbursts experienced underground, two general zones of stress-induced rock mass failures have been typically observed in underground hard rock mines relative to the proximity of the excavations and the confinement conditions. In a simplified manner, stress-induced rock mass damage and failure can occur both under low and/or high confinement conditions. In both conditions, the damage process seems to almost always initiate by pre-conditioning the rock mass prior to actual failure by the creation of networks of new extension fractures (i.e., tensile fractures formed under a compressive stress field). Once sufficient damage or pre-conditioning has occurred to the rock mass, then overall loading and geometric system adjustment tend to exploit these damage zones through several mechanisms, thereby inducing actual “failure”.

Under low confinement, observations in the laboratory and at mine scale suggest that the onset of rock mass damage begins by the nucleation of extension fractures that tend to grow approximately parallel to the direction of the maximum induced principal stress. Under low confinement, the presence of pre-existing discontinuities (e.g., joints and bedding planes) appears to exert little influence on the process of nucleation and propagation of the new extension fractures.

In regions of high confinement, by contrast, macroscopic fault or shear zones may form as mining develops, such as those formed ahead of tabular stopes in South Africa. Depending on the orientation of key geological structures in relation to the in-situ stresses, induced shear loading can be developed on these structures, creating the potential for: a) remobilization of existing geological structures, b) formation of new seismically active structural zones, created through rock mass damage and coalescence along such structural zones, and/or c) interaction between existing and newly created structures. Several terms have been used in the technical literature to describe the formation of this type of macroscopic shear zone, such as: shear rupture, deformation band or principal slip zone. In this paper, because of morphology similar features have also been termed somehow interchangeably as “an embryonic fault zone”, a “seismically active structural zone” and/or in one specific case at the Garson mine in Sudbury as “the 45° Structure zone”.

Analysis of the mining induced seismicity that developed at Garson Mine indicated that the majority of microseismic events experienced in the last 2 years neither occurred immediately after a stope was mined nor were located close-to or around a stope, drift or excavation boundary; instead they tended to occur along new and/or along pre-existing major geological structures. Using a seismic plane clustering algorithm, it was found that a preferred orientation existed of seismically active planes with the dominant active planes dipping 35°-55° to the South. In addition, several known geologically interpreted structure zones that interconnected
with these preferred planes were also mobilized within an overall corridor of more seismically active features, designated the 45° Structure Zone.

Similarly to the low confinement conditions, the damage process for the formation of these macroscopic shear zones may also start by pre-conditioning the rock mass with the nucleation and growth of extension fractures. Such extension fracturing would likely tend to be a clean surface exhibiting no signs of shear displacement and being associated with high dilation in the direction normal to its propagation. The difference to the low confinement case, however, seems to be that when under high confinement, these extension fractures rather than immediately coalescing, they may develop in an en-echelon pattern, within a zone (corridor or deformation band) created under an overall shear loading or shear stress condition.

Except for a few observations from deep mines in South Africa and from surface outcrops, these fault or shear zones have not been mapped or described as such in Canadian underground mines. However, their formation may have been recorded by microseismic systems and described as fault-slip types of rockburst, as in general they inevitably generate high magnitude events when they fail.

In this paper, it is proposed that in addition to their being created essentially from “scratch”, on some occasions they may exploit previously weakened geological zones. This is evidenced by en-echelon patterns of pre-existing extension fractures that have already been formed and filled with quartz (or dykes) in the geological past. Such features could now be reactivated by shear loading to create a new macroscopic shear rupture in response to changes in induced stresses created by mining.

Using the South African experience that these major structural features (or zones) tended to only show significant damaging signs of shear displacement near stope zones, where the kinematic conditions for shear deformation were most favourable, for shear release, numerical simulations were carried out to investigate four factors that could help identify potential for adverse fault-slip on controlling structures, namely: (a) unclamping, (b) day-lighting, (c) stress rotation and (d) pillar shear.

Application to an underground mine in the Sudbury Basin allowed verification of these factors and the proposal of two alternative mine planning methods to limit fault slip on key structures: (1) pillar clamping and (2) stope sequencing.
Research to reality: Application of mining-induced seismic hazard maps

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1 INTRODUCTION

The Seismic Excavation Hazard Maps (SEHM) concept can be traced back to rockburst related research conducted in the 1990’s in Canada. While novel, the implementation of the concept was hampered by lack of suitable multi-dimensional analysis software and computational hardware to handle the large stress model and seismic datasets. With the current pace of technology development, the virtual reality (VR) developed technology will soon be available to the practitioner on the desktop.

1.1 Mine Data

The RAND report on the critical technologies for the mining industry (Peterson et al., 2001) recognizes that the knowledge management benefits of information technology (IT) will provide the greatest benefit to the industry. The report states that “although mine operations are generating more data, such information is rarely well utilized.” The major challenge is first to separate the important information and only then decide on how to use it. It is the underlying principal that “Data is not as interesting as insight”, that forms the basis for the new techniques in understanding the complexity of mining-induced seismicity hazards in the form of simplified, yet highly informative Seismic Excavation Hazard Maps.

1.2 Scientific and Information Visualization

Modeling software also provides visualization capabilities, however, the advantage of scientific visualization is that it uses the geometric (easting, northing, elevation) data as a structural support for thematic (property) data. The goals between the modeling visualization and scientific visualization are also different; the former is used to create data/information, whereas the latter is used explore datasets to find new trends/knowledge.

1.3 Virtual Reality

A number of mining researchers have applied principles of virtual reality to develop tactical applications such as safety training applications and line of sight for equipment design. In contrast, the foundational research work at MIRARCO using VR attempts to modify our perception of transient data to better understand the complex dynamics of deep mining.

1.4 Mine Map Overlays (MMO)

“Mine map overlays for excavation performance assessment in burst-prone mines” is the concluding chapter in the Canadian rockburst support handbook (Kaiser et al., 1996). With current 3D modeling, visualization and database technology, the MMO concepts are more easily integrated into a mine geomechanics model.

1.5 Geology models

Building a proper 3D deposit model is a complex procedure that requires a multidisciplinary geosciences approach since large volumes of space must be interpreted on “filled-in” based on limited data (surface maps, drill holes, geophysics, geochemistry, etc).
1.6 Seismic Excavation Hazard Maps

A (micro)seismic event records a physical rock mass damage occurrence. Stress-induced damage events are generally confined to the vicinity of mined openings, whereas structure-induced (shear-slip) seismicity may occur at some distance from the mine excavations. In terms of data handling and processing, the former are dealt with by seismic density hazard criteria, whereas the latter are determined by applying the space-time clustering algorithm to isolate individual clusters in 4-dimensional space (Vasak et al., 2004). To produce a practical and useful tool for the mine operator, the complexity of the data processing is reduced to produce the SEHM. Hazard ratings for the three factors are rated according to their impact on mine infrastructure. The hazard map is 3-dimensional and can represent discrete periods of time. The hazard factors may be combined or used independently depending on the site-specific conditions, since not all may be relevant to an operation.

2 SEISMIC EXCAVATION HAZARD MAPS IN VR

Under the Ontario Research Fund’s “Productivity Enhancement and Risk Management (PERM) for Underground Construction and Mining” initiative (www.mirarco.org/FeaturePrj/perm.htm), the Seismic Excavation Hazard Map logic is now implemented as a near-real-time excavation hazard system at a deep mine in the Sudbury Basin. The seismic hazards are calculated for the rock mass volume, but can be represented in terms of iso-surface contours, or more traditionally as plans and sections or the data can be projected onto any other geometric object such as geological structures (e.g. faults), mine stopes or drifts. A simple colour scale (blue, cool, low hazard to red, hot, high hazard) represents the hazard level.

3 CONCLUSIONS

It is not the quantity or quality of data, but rather the quality of the decisions that are made based on the data that makes scientific (engineering) visualization in an immersive, collaborative virtual reality environment an invaluable resource.

The stereoscopic hardware enables the use of more advanced software applications. Particularly if the use of the added dimension provided by the stereo display is not “wasted” by projecting 3-dimensional data, but leveraged to display hyper-dimensions. These dimensions can be used to create a link between space and time and to provide a means of evaluating and exploration the effects of uncertainty on measured and synthesized data.

Seismic Excavation Hazard Maps can be used to assess hazards and more importantly, to provide the resource for the operator to identify risk mitigating measures for anticipated rock mechanics problems as mining progresses deeper.

The SEHM is an additional tool for the rock mechanics practitioner, not to be used alone, but as an integral part of a comprehensive geomechanics evaluation process.

High-level risk mitigation measures developed in the immersive, collaborative virtual reality facility will ensure safe, profitable and reliable ore extraction.

REFERENCES


Re-entry Protocols for Seismically Active Mines Using Statistical Analysis of Aftershock Sequences

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1 INTRODUCTION

Immediately following large seismic events or blasts in mines, there is a short-term increase in levels of seismic activity that over time decays to background levels. Restricting access to areas of a mine affected by seismicity for sufficient time to allow this decay to occur is the main approach in most re-entry strategies.

As a part of a long term worker safety program, a MASHA sponsored and WSIB funded research project has been on-going at Queen’s University during the last three years. The overall goal of the project is to produce reliable practical guidelines for the development of re-entry protocols in seismically active mines for the range of mining conditions found in Ontario. The first phase of the project involved a review of current re-entry practices (Vallejos & McKinnon, 2008). The second phase focuses on the applicability of the Omori’s law as a formula for describing the event decay rate of mining-induced aftershock sequences (Vallejos & McKinnon, 2009). A total of 252 aftershock sequences were analyzed in seven different mines in Ontario. Guidelines for the use of the Omori formula for re-entry protocol development are presented.

2 CURRENT RE-ENTRY PRACTICES

Current re-entry practices were evaluated through a survey at 18 seismically active mines. Some of the main findings from the survey are:

- We found a large variety of re-entry protocols in use, many being specific to particular activities such as mining through dykes, crown blasts, or mining around identified hazardous zones.
- Blasting was found to be a significant factor in triggering seismic events. Ninety percent of re-entry incidents were reported to be triggered by blasting.
- The distance from active mining that seismic events typically trigger re-entry restrictions is between 50-100 m.
- Over half of the mines surveyed use event decay rate analysis as their primary decision making parameter.
- The decision to re-enter is based on the requirement for the monitored parameter to return to a previously defined background level for a specified time window.

3 MODIFIED OMORI’S LAW

The modified Omori’s law (Omori, 1894; Utsu, 1961) states that

\[
n(t) = \frac{K}{(c + t)^p}
\]

(1)
where $n(t)$ is the event rate since time $t$ measured from the main event, $c$ is a time offset constant, $p$ controls the speed of decay and $K$ is an activity parameter. The parameters $c$, $p$ and $K$ are estimated using the method of maximum likelihood (Ogata, 1983).

For estimating parameters on a consistent basis we considered only the time interval that satisfies power-law behaviour and $c=0$. The sites analyzed presented $p$ values that are normally distributed within a range of 0.4-1.6. The parameter $K$ can be approximated by: $K=\kappa N_{1\text{hour}}$, where $\kappa$ is a site specific parameter, which ranges between 0.30 and 0.50, and $N_{1\text{hour}}$ is the measured number of events occurring during the first hour after the principal event.

Determining the time for re-entry using Omori’s law to represent the time decay of seismic activity poses a problem in that it’s power law form implies there is no characteristic time scale. However, we found that the Omori formula has a characteristic point at the maximum curvature $T_{MC}$, which is a function of the Omori’s law parameters. This point has the interesting property that it defines the transition between the highest to lowest rate change. Therefore, we recommend $T_{MC}$ as a preliminary estimate of the time at which it may be considered appropriate to re-enter an area affected by a blast or large event.

The maximum curvature point $T_{MC}$ can also be correlated with $N_{1\text{hour}}$ by the expression: $T_{MC} = a(N_{1\text{hour}})^b$, where $a$ and $b$ are two parameters dependent on local conditions. Both parameters presented well constrained empirical ranges for the sites analysed: $a$ between 0.2 and 0.55, and $b$ between 0.5 and 0.8, with an average of 0.38±0.11 and 0.61±0.09, respectively.

These findings enable us to use the Omori’s law for developing a real time re-entry protocol after recording the number of seismic events during the first hour after the principal event.

4 CONCLUSIONS

Based on the results of a survey, current re-entry practices have been summarized. These most common practices provide advice on the development of re-entry protocols for those mines that are starting to experience seismicity and rockbursting and are faced with the difficulty of developing re-entry protocols without the benefit of significant local experience.

The event decay rate of mining-induced aftershock time sequences can be satisfactorily described by the modified Omori’s law. Based on the pattern described by the Omori formula and using the measured number of seismic events during the first hour as an input parameter it is possible to make a preliminary estimate of the time at which it may be considered appropriate to re-enter the area.

REFERENCES


1. INTRODUCTION

Seismically active areas are located in the middle of the complex fault-fold belt from the northeast to the Long Men Mountain (Fig.1-1). The Long Men Mountain Fault Zone is spreading to the northeast, and its southwest begins near Luding to the northeast along Guanxian, Wenchuan, Guangyuan into Ningqiang in Shanxi, Mianxian areas, totaling 500 kilometers. The basic structural framework of Long Men Mountain Fault is comprised of three northeast compression-shear faults: the Maowen fault, Yingxiuwan fault and Pengxian-Guanxian fault from north to south. Seismic areas sit in the block border surrounded only by Maowen fault and Yingxiu fault, with the structural plane dip direction of the sub-region mainly to northeast compression-shear faults.

The basic characteristics of the three main faults in Long Men Mountain can be summarized as follows:

- **Maowen fault:** The overall orientation is N30°-45°E/NW-45°-80°, along the Jiangyou areas in the northeast; three main fault zones developing in parallel compose the Maowen fault which can be divided into the west, the middle and the east branch faults; the line of the Maowen fault is closer than the others; its tectonic ingredients are tectonic breccia, sheet rock, cataclasite, tectonic lens, miliolite or fault clay.

- **Yingxiu fault:** The overall orientation is N40°-60°E/NW-50°-80°, with zone width 50 to 80 m, composed of crushed rock, sheet rock, miliolite and fault clay; it goes across the Min River in Yingxiu so that the Proterozoic Cheng River Jinningian granite (r2 4) overthrusts on Triassic Xujiahe Formation (T3XJ 3); it has characteristics of pressure or compression-shear.

- **Pengxian-Guanxian fault:** The overall orientation is N30°-60°E/NW-40°-53°, with zone width of about 100 m, composed of crushed rock, sheet rock, miliolite and fault clay.

These three faults are compression-shear thrust faults, with morpho-tectonics being linear. Later in geologic time, tectonic activity has been relatively strong. Historically, moderately
strong earthquakes occurred because of Maowen fault in Wenchuan, Yingxiu fault in Beichuan and Pengxian-Guanxian fault in Tianquan, Dayi. Under the control of these faults, secondary NE faults and fissures are developing.

The Wenchuan earthquake occurred because the northward movement of the Indian plate squeezed the Eurasian plate, causing uplift of the Qinghai-Tibet Plate, and eastward movement to squeeze the Sichuan Basin at the same time. The stress accumulation contributed to breakdown of the crust, and earthquakes along the Long Men Mountain fault zone.

In the earthquake zone the shock speed, high-energy accumulation, and the reversed dextro-rotation of the earthquake fault characteristics occurred, which caused the development of secondary geological disasters, mainly major landslides, collapses and landslides, potentially unstable slopes and landsliding barrier lakes.

The typical geological disasters had thrust rock collapse, the instability deposit, and the revival of potential debris flows whose genetic model has been analyzed.
SESSION 14 ROCKMASS CHARACTERIZATION AND SITE INVESTIGATION II

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Stress Measurements At Great Depth At Craig-Onaping Mines, Sudbury Canada
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B. Simser & S. Carlisle
Xstrata Nickel, Onaping Ontario Canada

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Statistical Multi-Scale Method Of Mechanics Parameter Prediction For Rock Mass
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1 Department of Applied Mathematics, Northwestern Polytechnical University, Xi’an, China
2 Academy of Mathematics and System Sciences, Chinese Academy of Sciences, Beijing, China

4035
Geomechanical Studies Of An Alpine Rock Mass
Tiziana Apuani, Gianpaolo Giani, Andrea Merri
University of Milan - Dept. of Earth Science

4169
A New Tool For The Field Characterization Of Joint Surfaces
D. Milne, C. Hawkes and C. Hamilton
University of Saskatchewan, Saskatoon, Canada
Stress measurements at great depth at Craig-Onaping Mines, Sudbury Canada

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1 Introduction
Knowledge of *in-situ* stress is required as input to numerical models of the planned extraction sequences in order to predict the performance of excavations and to design appropriate ground support schemes. Over the last ten years researchers at the Western Australian School of Mines (WASM) have developed techniques for measuring *in-situ* stress from oriented core, obtained from remote locations well in advance of mining development (Villaescusa et al, 2002; 2003). This information can be used to estimate the mobilized shear stress on large scale structures that are likely to cause failure, seismicity or influence the overall rock mass behaviour at a global mine scale (Windsor et al, 2007).

2 Acoustic Emission Technique
Acoustic emissions (AE) are bursts of high frequency elastic waves caused by localized failure of a material when it is placed under load. When a rock core sample is loaded and the acoustic emission monitored, it is observed that at a certain stress level the amount of acoustic emission increases markedly.

3 In-situ stress orientation and magnitudes for Craig-Onaping Mine
The methodology described above was used to estimate the *in-situ* stresses at several locations drilled at depth from three oriented cores at Craig-Onaping (See Figure 1).

![Figure 1. Section view (3D) of steeply dipping holes OD4984 and OD4985.](image)

Figure 2 shows the estimated principal stress orientations with respect to the borehole orientations (mine grid north) provided. The orientation of the main principal results
appear parallel to a common sub trend in NE/SW direction. Field evidence of horizontal stress consists of back overbreak, hole squeezing and fracturing in excavation back of previous cut within the cut-and-fill operations.

Figure 2. Principal stress orientations WASM AE–Craig Onaping Mine.

Figure 3 shows the stress magnitude profile with depth for the measurement sites at Craig-Onaping.

Figure 3. Principal stress magnitudes WASM AE–Craig Onaping Mine.
ABSTRACT: In this paper a statistical multi-scale method for the mechanics parameter prediction of the rock mass with random distribution of multi-scale cracks/joints is presented. First the micro-structure of the rock mass with random distribution of multi-scale cracks/joints is represented. Then the statistical second-order two-scale method for the mechanics performance predictions of the rock mass structure with random cracks/joints distribution is presented, including the statistical second-order two-scale expression on the vector-valued displacement, strain tensor and stress tensor, and the algorithm procedure of statistical multi-sale computation for the mechanics parameters. Finally some numerical results for mechanical parameters for the rock mass with random distributions of multi-scale joints/cracks by statistical multi-scale method are shown.

1 INTRODUCTION

With the rapid advance of engineering science, especially computing technology, the computational engineering science is developing very fast. A variety of numerical methods for the predicting the physical and mechanical performance of materials was developed in last decade.

According to their micro-structure the composite materials can be divided into two classes: composite materials with periodic configurations (Cui et al. 1997 Cui & Shan 2000) and composite materials with random distribution (Li & Cui 2004). A lot of random composite materials exist in nature and human life, such as rock mass and concrete (Shan et al. 2002). Due to the difference of their micro-configurations it needs to make use of different numerical methods to evaluate the physical and mechanical performance of them.

For the composite materials with random distribution some works have been done for predicting the physics and mechanical properties of random particulate composites (Li & Cui 2005 Yu et al. 2008). Many approaches can be used to the calculation of macroscopic stiffness parameters, such as the law of mixture, Hashin-Shtrikman upper and lower bounds method, self-consistent approach and Eshelby effective inclusion method etc. However, in regard to the prediction for strength parameters there are few theoretical techniques available, and most of them are based on the greatly simplification of real composite structures. Till now there is still no multi-scale analysis method to predict the physical and mechanical performance of the rock mass structure with random joints or/and cracks distribution.

In this paper a new statistical multi-scale method is presented to predict the mechanical performance of rock mass with random joint and/or crack distribution and related structures.

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The remainder of this paper is outlined as follows. In section 2 the representation of the rock mass with random distributions of multi-scale joints/cracks briefly described. The section 3 is devoted to the statistical second-order two-scale formulation for the prediction of the materials with random distribution and related structure. In section 4 the algorithm procedure for statistical multi-scale computation of rock mass with random distributions of multi-scale joints/cracks is given. In section 5 some numerical results for mechanical parameters of the rock mass with random distributions of multi-scale joints/cracks are shown.

![Joint statistical model of the four screen scales in rock mass.](image)

The joint statistical model of the four screen scales in rock mass.

![Statistical model of joints for ND(0°,10°) and mesh partition.](image)

The statistical model of joints for ND(0°,10°) and mesh partition.

In this paper one kind of structures of rock mass with plenty of joints or/and cracks is considered, they are defined as the structures of the materials with random distribution of multi-scale joints or/and cracks. And the micro-structure of rock mass with plenty of multi-scale joints or/and cracks is represented.

A new statistically second-order two-scale methods for the predicting the mechanics performances of them is presented, including the second-order two-scale asymptotic expression on the displacement vector, the formulations of the expected homogenization constitutive parameters, elasticity limit strength, and the algorithm procedures.

For some different random distribution models the expected homogenization constitutive parameters are predicted by SSOTS method. And the numerical experiments show that the micro-behaviors inside the structure with plenty of joints or/and cracks can be captured exactly by SSOTS method. And all of numerical results show that SSOTS method is valid and available.
Geomechanical studies of an alpine rock mass

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1 INTRODUCTION
The study refers to an area of about 70 km², extended in the Italian Central Alps along the San Giacomo Valley where different civil and mining works are present. The work is a contribute in assessing how the geometrical and mechanical properties of the rock masses can be regarded as regionalized variables. The spatial distribution of these properties is a function of the geological and structural phenomena at which the rock masses have been subjected. The presented geostatistical analysis regards the spatial variation of the orientation of the most representative joint set analysing the problem at two scale: same kilometres and hundred of meters scale.

2 GEOLOGICAL AND GEOMECHANICAL FEATURES OF THE AREA

2.1 Geology and structural geology
The regional geological setting is related to the Pennidic Nappe arrangement, characterized by the emplacement of sub-horizontal gneissic bodies resulting from the Mesoalpine isoclinal folding of crystalline basements, emplaced throw East, and separated by matasedimentary cover units. The investigated geological rocks, belong to the “Tambò” Unit, overlapped by its meta-sedimentary cover and by the “Suretta” Unit. The tectonic contact gently dips to the E–NE. The valley, furrowed by the Liro Torrent, follows a N–S-striking tectonic lineament, almost parallel to the nappe contact. Four main Alpine deformation phases were recognised in the upper eastern Pennine units; the two late pheses overprinted and steepened the previous structures, and produced an extensive fracturing pattern, dominated by two set orientated NW-SE and NE-SW, mainly expressed by normal faults cross-cutting all previous structures. The tributary valleys of the Liro are mainly elongated in the NE-SW direction. The Febbraro Valley which is one of them, presents well exposed outcrops of the Tambò nappe both basement and cover. A detailed analysis of the structural lineament orientation was carried out in this area, by fotoaereal observations, DEM interpretation, and in situ survey. The rose diagram shows the maximum frequency of lineament direction in the range between 300°-320°.

2.2 Geomechanical features of the rock masses
A number of more then 70 sites, distributed in the studied area, mainly along the Febbraro basin and on the left side of the San Giacomo valley around the Isola village, were chosen and detailed structural and geomechanical field surveys have been performed to characterize the rock masses, its intact rock and discontinuities. The procedure identifies the number of joint sets, and their representative orientation, the set spacing, the type of movement, the amount of dilation, the degree of alteration, the roughness coefficient, the presence and nature of infill. Rock mass quality indexes, such as the Rock Mass Rating (Bieniawski, 1989) or the Geological Strength Index (Hoek et al. 2002) have been evaluated in each surveyed site.

The 63 % of the sites follow in the class of “fair good quality” (RMR 41-60), and 36 % in the class of “good quality” (RMR 61-80), mostly bellow 70. To analyze the regional variation of joint orientation all the collected data were considered together and stereographic projection produced. The joint set SC, coincident with the more pervasive foliation, has average orienta-
tion 18°/91° (dip/dip direction), and shows a wide dispersion of value along the great circle 73°/260° (dip/dip direction). This orientation is quite parallel to the nappe contact.

Then, at least further 3 sets of joints are recognized, with mean orientation as follow: JN1 77°/276°; JN2 69°/155°; JN3 71°/215°. It was chosen to study the regional variability of the orientation of the JN1 set by means of geostatistical analysis.

3 GEOSTATISTICAL STUDIES

The tool applied to assess the structure of the joint orientation distribution is the variogram. The variogram function is defined as the expectation of the random variable

$$2\gamma(x, h) = \mathbb{E}\left[\theta(x) - \theta(x + h)\right]^2$$

(1)

where \(\theta(x)\) and \(\theta(x+h)\) are the angular distances between the unit vector in the \(x\) and \(x+h\) position. The variogram can be estimated by using the surveyed data: a variogram estimator is defined as the arithmetic mean of the square differences between two angular distance unit vectors at any two points separated by the distance \(h\),

$$2\gamma*(h) = \frac{1}{N(h)} \sum_{i=1}^{N(h)} [\theta(x_i) - \theta(x_i + h)]^2$$

(2)

where \(N(h)\) is the number of experimental pairs \([\theta(x), \theta(x+h)]\) of data separated by the distance \(h\).

Two variograms were constructed at different scale in this study: the first variogram was drawn at the surveyed discontinuity scale of about 70 meters, using the joint orientation (276°/77°, dip dir./dip), which is the mean values of the joint set (JN1) orientation obtained in all the examined stations selected in the San Giacomo valley; the second variogram was drawn at the San Giacomo valley scale of some kilometers, by using the same value.

4 CONCLUSION

A rock mass characterization of a Central Alps valley has been presented. Geomechanical work was carried out by surveying rock discontinuities in 73 different sites and by classifying, according to Bieniawski RMR system, the examined rock mass. Rock mass exhibits both good qualities and similar geometrical and mechanical parameters in each surveyed sites.

For this purpose a Geostatistical application was carried out to examine the spatial variability of a geometrical parameter of one of the surveyed joint sets: the orientation. The structure of the distribution of the joint orientation parameter was investigated by means of a variogram analysis.

Some auto–correlation of the joint set orientation in space has been determine both at a small scale and at a regional scale.

The structural variation of the joint set distribution shows that the direction along which the valley scale variogram exhibits greatest auto–correlation is about 315°: this direction is one of the most significant structural lineament, and those most frequent in the basin of the Febbraro Valley, area of structural detailed study.

The definition of auto-correlations to evaluate the spatial variability of the mentioned geometrical properties, can represent an useful tool: first in implementing the comprehension of the geological and geomechanical features related to the rock mass history, and to support provisional evaluation where no direct investigations are available.
A new tool for field characterization of joint surfaces

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ABSTRACT

Joint surface characterization is an important component to all rock classification systems and contributes about 30% to the overall classification value. In mine design work, it is important to improve our ability to estimate rock mass properties to try and keep up with improving techniques for modelling rock mass behaviour. Current methods of assessing the contribution of joint properties to rock mass properties often consist of very subjective descriptions of the general joint surface character. Improvements in joint roughness measurements were applied to rock characterization in mining in the early 1990s. These techniques were simple and designed to improve an engineer’s ability to characterize the joint properties into the very general and often subjective rock characterization categories used by rock classification systems.

Further improvement to methods of characterizing joint surfaces have not been developed for underground mining applications. There are no well established, commonly used design techniques which can apply improved data on joint surface conditions. Improved data gathering techniques will however improve the application of the very general joint property categories in current rock characterization systems, as well as provide much needed background information for future improvements to rock characterization systems.

In the petroleum geomechanics field, the success of thermal enhanced recovery operations, CO₂ sequestration and waste injection depends upon a tight, low permeability cap rock. The majority of any escape of reservoir fluids would, in most cases, be along discontinuities in the cap rock. Lab testing for estimating gas and liquid permeability through fractures is an important component in analyzing and predicting hydraulic integrity of cap rocks. One goal of an effective cap rock evaluation is to link laboratory test results to simple measurements of fracture roughness that can be conducted on core samples.

A simple hand held laser profilometer has been developed for recording the roughness characteristics of a joint surface (Figure 1). An example of the profile generated is shown in Figure 2. This paper presents this new field tool and discusses how data from this field instrument can be used to assist in designing stable underground openings, as well as effective steam, CO₂ or waste injection programs. It is hoped that this tool will also encourage the collection of improved rock classification data for mining geomechanics design.
Figure 1. Portable laser profilometer developed at the University of Saskatchewan

Figure 2: Sample joint surface profile generated using the portable laser profilometer.
SESSION 15  USE OF LIDAR AND DIGITAL PHOTOGRAMMETRY IN ROCK ENGINEERING II

3999
Investigation Of Block Geometrical Properties Of The Shale-Limestone Chaotic Complex Bimrock Of The Santa Barbara Open Pit Mine (Italy)
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4015
Enhancing the Collection of Rock Mass Fabric Data for Open Pit Mines
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Golder Associates Ltd., Mississauga, ON, Canada

4145
Use of a stereo-topometric measurement system for the characterization of rock joint roughness in-situ and in the laboratory
B.S.A. Tatone & G. Grasselli
Lassonde Institute, University of Toronto, Toronto, Ontario, Canada

4343
An in-Shovel Camera-based Technology for Automatic Rock Size Sensing and Analysis in Open Pit Mining
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Motion Metrics International Corp., Vancouver, British Columbia, Canada
Investigation of block geometrical properties of the Shale-Limestone Chaotic Complex bimrock of The Santa Barbara open pit mine (Italy)

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1 INTRODUCTION

The exhausted Santa Barbara open pit mine (Tuscany, Italy), owned by the main electric power company in Italy (ENEL S.p.A,) extracted lignite as a fuel for the Santa Barbara thermoelectric power plant.

The mining activity led to the excavation of a wide slope (with a maximum height of 200 m and a planar extension of about 1300 m x 500 m) in the Shale Limestone Chaotic Complex (SLCC) (Figure 1). Since the beginning of its excavation the slope underwent rotational landslides and diffuse instability phenomena.

From a geotechnical point of view the SLCC represents a typical bimrock (block-in-matrix rock) (Medley 1994, Medley 2002, Medley 2007) characterized by a mixture of a highly sheared, scaly-fabric, clayey matrix and calcareous blocks ranging from a few millimetres to tens of metres in size.

Since the geometrical properties of blocks (grain-size distribution, shape, orientation and volumetric proportion) have been proved to influence the mechanical behaviour of bimrocks (Lindquist & Goodman 1994, Goodman & Ahlgren 2000, Kim et al. 2004, Sonmez et al. 2006), the aim of the present study is to investigate the geometrical properties of calcareous blocks in order to properly characterize the mechanical behaviour of the SLCC bimrock and to further assess the stability of the slope.

Block size-distribution and shape were investigated by means of advanced image analysis techniques of SLCC pictures taken in the field.

Geostatistical semivariograms and variographic maps were also used in order to investigate the shape and spatial distribution of blocks.

Finally, in-situ large sized sieving tests were carried out to investigate the volumetric block proportion within the clayey matrix.

2 DATA ANALYSIS

An indirect non-destructive characterisation was carried out on 2 m x 2 m photographic images of SLCC outcrops by means of image segmentation (Sahoo et al. 1988, Gonzalez & Wood 2002) in order to investigate the size distribution of block Major Axes (MA) and block shape ratio.

The size distribution of MA appears to be self-similar at the 2 m x 2 m scale of investigation and it can be described as an inverse power-law relationship with a 2D fractal dimension $D$ ranging from 1.2 to 1.8. These data are in agreement with previous studies developed for the Franciscan Melange, California (Medley & Lindquist 1995, Medley 2002).

The size distribution shows a peak for the endclass value equal to $0.005\sqrt{A}$ (1 cm); this value is different from that observed in the Franciscan mélangé where it is equal to $0.05\sqrt{A}$.
Blocks shapes resulted in a an overall shape ratio ranging from 0.4 to 0.65 with a modal value of 0.55.

Experimental semivariograms and variographic surfaces were calculated in order to identify preferred directions of anisotropy of block shape and spatial distribution. The analysis evidenced both isotropic and anisotropic patterns at the scale of investigated samples. Layered patterns, however, do not show a common principal direction of anisotropy all over the whole slope.

The evaluation of volumetric block proportion within the clayey matrix was investigated by means of in-situ sieving tests on 1 m$^3$ bulk samples, using a special sieving cage developed for the purpose of the study. Sieving tests gave an average volumetric percentage of blocks within the matrix of 33%.

3 REFERENCES


Enhancing the Collection of Rock Mass Fabric Data for Open Pit Mines

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Stability of the rock slopes within a strong rock mass is primarily susceptible to kinematically controlled failures such as planar, wedge and toppling failures. As a result, knowledge of the major and minor discontinuity sets that form the rock mass fabric is paramount for pit slope designs. Due to the increased awareness of health and safety in mining projects, efforts have been made to limit structural mapping of the exposed bench walls in order to minimize the risks to personnel associated with approaching potentially unstable walls. Recognizing the importance of such safety precautions, Golder Associates Ltd. is applying a combination of techniques including 3D photogrammetry, conventional mapping and drilling with core orientation (or with televiewer surveys) to enhance structural data collection.

The case study described here was for the expansion design (both laterally and with depth) of an existing open pit. The on-going mining activity has exposed discontinuities on bench faces that could assist with the understanding of the rock mass fabric. However, due to blast induced damage, the previously mined pit walls contain unstable blocks which are prone to fall. Consequently, these potential hazards were considered to be dangerous for personnel to perform bench mapping, preventing the proper collection of joint measurements in some locations.

3D photogrammetry was used to further improve the understanding of the rock mass interpreted from conventional surface line mapping and geotechnical drilling with core orientation. Structural mapping through photogrammetry involves the use of digital photographs to produce a virtual model of a rock mass surface. Structural orientation data was obtained from the 3D images and analyzed for the major discontinuity sets that would influence slope stability.

Stereographic projections of the line mapping, photogrammetric method and geotechnical drilling emphasize three main discontinuity sets that exist in the east wall rock mass:
- Set 1 (S1): Sub-horizontal (dip < 20°) discontinuity set that generally dips northeast,
- Set 2 (S2): Sub-vertical (dip > 70°) discontinuity set that generally strikes east-west, and
- Set 3 (S3): Sub-vertical (dip > 70°) discontinuity set that generally strikes north to northeast.

Discontinuity Sets 1 to 3 are very pronounced on the photogrammetric stereonet and appear as major sets on the line mapping and oriented core stereonets. However, a fourth major set (S4) is also apparent from the drilling results. Set 4 is a foliation set that dips moderately (30°to 45°) to the east, into the pit wall. Therefore, this set does not act as a kinematic control on the east wall and is not prominent on the bench face surface.

Directional bias is introduced during data collection in all three techniques, where discontinuities that are perpendicular to the direction of surveying will be more evident than discontinuities that are sub-parallel. This means that structures that are oriented within up to about 20° to the drillhole axis or wall direction cannot be or will be less frequently measured than the structures with other orientations, creating “blind” zones in the stereographic projections. Despite the application of the Terzaghi correction in these projections, it is important to have a good understanding of the influence of these blind zones when interpreting the results. In addition, it highlights the need to carefully map walls with more than one orientation within the existing open pit and to ensure good coverage with directional geotechnical drilling.

Although the analysis of the line mapping and photogrammetry data produced analogous results, approximately six times the number of structural features were recorded with the photogrammetric method, with the same amount of time spent in the field for both techniques. The entire slope face can be mapped with photogrammetry, given that there is sufficient space to capture the rock face with the camera and the availability of the appropriate fixed focal length or...
zoom camera lenses. Whereas line mapping is limited to areas which are safely accessible and dependent on structural features that intersect the survey line (unless window style mapping is used). As a result, photogrammetric mapping can easily identify the critical discontinuity sets for slope stability, and this information is useful in validating the minor and major sets interpreted from geotechnical drilling.

In summary, there are limitations and benefits associated to each structural data collection technique. Traditional line mapping is simple, requiring little more than a compass, survey line and field staff. However, the risk associated with personnel approaching potentially unstable rock faces can limit coverage. Geotechnical drilling with core orientation offers structural data at depth and knowledge of in situ rock mass fabric conditions. Using this knowledge, shear strength data can be assigned to the major and minor discontinuity sets in a rock mass. The main drawback to geotechnical drilling is that it does not provide a clear insight to discontinuity persistence. 3D photogrammetric mapping can measure persistence and spacing, along with the orientation of discontinuity features, but lacks the capability to describe shear strength properties and other joint surface and rock mass characteristics. Photogrammetric mapping also offers large coverage in a short span of time, at distances away from potentially unstable rock faces. However, this method is limited to good weather conditions, clean unobstructed slopes and areas where a camera can easily capture the rock face.

Overall, photogrammetric mapping was a very useful tool in determining the major discontinuity sets seen in the existing open pit mine and complementing the other existing structural data collection techniques. In addition, it significantly improved the health and safety working conditions for the geotechnical field team, where access to the pit wall proved to be difficult and risky.
Use of a stereo-topometric measurement system for the characterization of rock joint roughness in-situ and in the laboratory

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1 INTRODUCTION

The surface roughness of unfilled rock discontinuities has a major influence on the deformational and hydraulic behaviour of discontinuous rock masses. Although it is widely recognized that surface roughness is comprised of a large-scale (waviness) and small-scale (unevenness) components, most investigations of surface roughness have been restricted to small fracture surfaces (< 1m²). Hence, the influences of the large-scale components of roughness are often neglected. Furthermore, these investigations typically focus on analyzing roughness in terms of two-dimensional profiles rather than the complete three-dimensional geometry, which can lead to potentially biased estimates of roughness. This contribution demonstrates the use of a new optical digital measurement system, based on the principle of triangulation, to digitize a large-scale natural planar rock discontinuity surface in granite gneiss at both outcrop-scale (~6m x 2m) and lab-scale (~100mm x 100mm). Subsequently, the digitized surfaces are systematically analyzed using the three-dimensional roughness methodology proposed by Grasselli to investigate the dependency of roughness (magnitude and directional anisotropy) on sample window size and measurement resolution.

2 EQUIPMENT

A 3D stereo-topometric measurement system, the Advanced Topometric Sensor (ATOS) II manufactured by GOM mbH, was adopted to digitize the large and small-scale fracture surfaces. The ATOS II system consists of a measurement head containing a central projector unit and two CCD cameras, and a high-performance Linux-based PC to pilot the system. The system is flexible in that it can be used in the laboratory with a boom-type stand and industrial PC or in the field with a laptop and tripod. Figure 1 illustrates the lab and field set-up of the ATOS II for the current study.

Figure 1. The 3D stereo-topometric measurement system utilized for this study, the Advanced Topometric Sensor (ATOS) II manufactured by GOM mbH: (a) lab configuration and; (b) field configuration.
3 EFFECT OF SAMPLING WINDOW SIZE AND MEASUREMENT RESOLUTION ON ROUGHNESS: METHODOLOGY AND RESULTS

To investigate the role of sampling window size on roughness estimates, several windows ranging from 100mm² up to 5x10⁶mm² were selected from the large-scale fracture surface and analysed. The results depicted in Figure 2 indicate that the magnitude of roughness was found to increase with the size of the sampling window (positive scale effect) while, anisotropy showed minimal variation.

To investigate the role of measurement resolution on roughness estimates, three small-scale samples of the large-scale field fracture were digitized with a range of measurement resolutions and analysed. Figure 3 illustrates the magnitude and anisotropy of roughness as a function of measurement point spacing. A sizable increase in the estimated roughness was observed as the measurement resolution was increased while, anisotropy showed minimal variation.

4 CONCLUSIONS

Based on the results of this study the following conclusions can be drawn:

– 3D roughness for the large-scale fracture surface displayed a positive scale-effect approaching a roughly constant value for sufficiently large sampling windows (> 3 x 10⁶ mm²).
– 3D roughness is more sensitive to measurement resolution compared to the size of the sampling window. Increasing the average spacing between measurements effectively smooth the surface; resulting in a 79% - 85% percent difference in roughness.
– Anisotropy in roughness was nearly the same for the large-scale and small-scale fracture surfaces and was found to be insensitive to both the sample window size and measurement resolution.
1. INTRODUCTION

Currently, if rock segmentation analysis of a particular location in an open pit mine is desired, the conventional method is to collect images manually from that region and later retrieve and feed them to a segmentation software at the office.

In this article, we present an autonomous rock fragmentation sensing and analysis system, with automation introduced in both image capturing (from desired digging operation sites using an in-shovel image collection system) and segmentation and fragmentation analysis with no manual correction (using a desktop software). However, as mentioned earlier, the process of rock segmentation is a difficult problem and cannot be considered providing flawless results.

Performing a fast and efficient rock segmentation and fragmentation analysis is directly affected by introduction of several regions on a single rock (over-segmentation) as well as formation of several rocks into one region (under-segmentation). Due to the difficulty in dealing with such over and under-segmented rock images, manual correction by a human operator is often needed to compensate for the errors in automatic fragmentation analysis in order to achieve a reasonable accuracy for blast engineers.

FragMetrics™ offers a fragmentation sensing solution with minimal requirement of manual corrections, as confirmed by the field results presented in this article in comparison with manual segmentation results.

2. APPROACH

Our rock segmentation and fragmentation analysis system collects the desired rock images from a camera installed on top of the boom structure of different mining shovels. The system automatically captures the suitable bucket images and stores them digitally on its permanent storage device. A bucket image is considered suitable if it is captured close enough to the camera to obtain a reasonable resolution and if it corresponds to a full bucket (i.e., not an empty or partially full bucket). The stored images are retrieved (via mine mesh network or manual data collection) and fed to the FM-Desktop office software. This software reads the bucket images and after performing the segmentation algorithms provides the fragmentation analysis results in different numerical and graphical representations.

3. SYSTEM DESIGN

FragMetrics™ is comprised of two main components: 1- FM-Logger: A shovel-based image capture and logging system. 2- FM-Desktop: An office software for fragmentation analysis and visualization of results. The diagram below shows a schematic of the system design and the various components in FragMetrics™.
A suitable bucket image refers to a bucket image with characteristics such as being close enough to the camera, high contrast, dust free and enough ore material to be analyzed. The MMI Bucket Extraction (BE) algorithm constantly delineates the bucket from its background during the swing action of the shovel. This algorithm and a suitability verification layer continuously select and pass the most suitable bucket image (during the swing action) to the Empty Bucket Detection (EBD) algorithm. The EBD algorithm decides whether the obtained bucket image is full or empty. The explained process is performed on the FM-Logger system. The suitable bucket images as well as the detected empty bucket images are stored on the FM-Logger storage device permanently.

FM-Desktop, as shown in Figure 1, is an intuitive and user-friendly desktop PC application. The collected bucket images from FM-Logger are fed to FM-Desktop for automatic fragmentation analysis and both graphical and numerical displaying of the size distribution results for any desired period of time. FM-Desktop displays different indications of segmentation results on the original images as follows:

1- A binary image indicating the rocks as white regions on black (fines) background.
2- The original image with region boundaries drawn in red color around the rocks.
3- A variable cell size graphical sieve shown on top of the original bucket image. The sieve provides an intuitive observation about how much of the material can be passed (depending on the selected P number, i.e., percentage passing) through a sieve with the indicated cell sizes.
4- A Color coded format to display the rocks that are larger than the sieve size (again, depending on the selected percentage passing).

4. CONCLUSIONS

In this paper, we presented an autonomous rock segmentation system to provide rock size statistics to mine blast engineers and geologists. The experimental results verify the performance of the proposed autonomous fragmentation sensing system with small mean absolute error between the automated and manual segmentation methods. Image pre-processing and preparation module is an important step in the proposed algorithm. Poor segmentation results will be experienced if this step is not performed precisely. At MMI, we continue to improve the accuracy and speed of the proposed automatic fragmentation analysis.
SESSION 16  DEEP MINING

3962
Difficult mining conditions in 153 Orebody at a depth of 4550 ft at Vale Inco’s Coleman Mine
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3994
Global Approach to Managing Deep Mining Hazards
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B. Valley, Ph.D.
MIRARCO, Sudbury, ON, Canada
Vale Inco Garson Mine, Sudbury, ON, Canada

4002
Modeling of mining-induced seismicity migration
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b Centre for Excellence in Mining Innovation, Sudbury, Ontario, Canada

4010
Prevention and Control of Rockbursts in Dongguashan Copper Mine
L. Tang
Central South University, Changsha, P. R. China
K. Xia
University of Toronto, Toronto, Canada

3986
Weak Rock Mass Span Design – Best Practices
A. Ouchi & R. Pakalnis
University of British Columbia, Vancouver, Canada
T. Brady
NIOSH, Spokane, USA
Difficult mining conditions in 153 Orebody at a depth of 4550 ft at Vale Inco’s Coleman Mine

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1 GENERAL INFORMATIONS

1.1 153 Orebody

Vale Inco’s Coleman Mine is located on the North Range of the Sudbury basin, approximately 28 miles north-west of the city of Sudbury and 3 miles east of the town of Levack, (Figure 1) Coleman Mine’s 153 Orebody is located 2500 feet west of and 1000 feet north of the Main orebody in the North range felsic footwall gneisses and breccias. The 153 Orebody consists of a complex system of copper-nickel-precious metal veins and stringers within an east west striking, southerly dipping sequence consisting of felsic gneiss, granite, and Sudbury breccia.

The 153 Orebody has a strike length of 1200 feet and has been defined from 3600 level to 5100 level (a vertical distance of 1500 feet). The ore zone has been divided into six horizons: 4250, 4400, 4550, 4700, 4810, and 4945 levels. The primary mining method in the 153 Orebody is mechanized cut-and-fill. Cut-and-fill production headings are mined to the width of the ore vein, up to a maximum width of 24 feet and with a cut height of 9 feet. With six mining horizons in the 153 Orebody, each cut-and-fill level successively mines upward, in individual cuts (each 9-ft high), creating a diminishing crown pillar. Cut-and-fill mining may eventually be converted to bulk recovery for the crown pillars where more challenging mining conditions are expected. To some extent, more difficult mining conditions are being encountered in the footwall of 4550 level at cut 13.

1.2 Seismicity in 4550 Level

During the mining of cut 13, an unrelenting seismicity trend was identified in a north-south direction. An assessment of the seismicity data showed that this trend has been consistent and repeatedly encountered in and around the same areas of the ore zone with the past mining activity.

To better identify the reason behind this persistent trend, a diamond drill program was planned and subsequently four holes were drilled in and around the area of interest in 4550 Level.
The cores and holes were analyzed by geotechnical core logging techniques in addition to Acoustic TeleViewer probing.

2 DATA COLLECTION AND ANALYSIS

Two of the holes were drilled horizontally in an area of high seismicity during the early stages of mining cut 13. These holes had a lower RQD and Q values and contained more fractured pieces. The other two holes were drilled at a dip of 40 degrees in the footwall of 4550. Logging of these holes showed that the footwall rocks are in good condition with no major structures present. However, a zone of jointed ground was noticed in one of them, hole 1252240, at a depth of 57-ft and this feature was further defined as a shear zone. This zone is aligned with the direction of the seismicity trend as shown previously.

In addition to core logging, the inclined holes were also assessed using the Acoustic TeleViewer probe. The results showed that there are no major fault/shear structures causing ground problems, however, in hole 1252240, at a depth of 57-ft, a joint set with a dip of 55 degrees and dip-direction of 290 degrees is noticeable. This data aligns with the results of coring and also fits with the observed trend of seismicity.

During the mining of block 2 of 4550, seismic activity in a direction opposite to the trend of 153 orebody was noticed. To confirm whether this seismicity trend was mainly due to stress redistributions, or due to an array error, or due to a geological structure in the footwall, a testing campaign was initiated. Both core logging and Acoustic TeleViewer inspections were implemented and assessed. The results of the TeleViewer and core logging determined that there is a joint-set in the same direction as the trend of seismicity. This structure, even though not extreme in nature, seems to be affecting the ground conditions in 4550 level and this impact seems to be increasing as the mining approaches 4400 level and as the crown pillar is diminished to a critical slenderness.

In a concerted effort to better control and manage seismicity and to minimize bursting while mining cut 13 on the 4550 level, a range of measures were undertaken as follow; development rounds were shortened to 6 or 8 ft in certain areas. Destressing techniques were utilized with holes drilled and blasted in the footwall of the mining cuts. Ground support was specified based on site conditions, Shotcrete was used as primary support for certain areas mainly in the wider veins located in the footwall, mechanical bolts were replaced with resin grouted rebars. The walls were supported to the floor with friction set bolts installed over #6 gauge screens. Trials with swellex bolts are underway for very broken ground conditions and in high grade cuts, i.e. where the bolt hole diameter cannot be controlled properly for effective use of resin grouted rebar bolts and mechanical anchor bolts. Seismicity was monitored constantly by both Mines Technical Services and Operations (shift supervisors) and protocols were followed accordingly. Temporary closure of areas and subsequent re-opening was based on data such as: the number of events, proximity of the events to the mining areas, magnitude of the events, etc.

3 CONCLUSION

The seismic data was helpful in identifying a geological structure in the footwall of 4550 level in the 153 Orebody. The existence of the structure was confirmed by geotechnical core logging and Acoustic TeleViewer technology. The seismic trend, primarily due to mining induced stress, is aggravated by the geological shear feature in the footwall. A well-developed plan was used to successfully mine 4550 level during cut 13. At this time, future mining in the footwall of the crown pillar is being studied in more detail to choose the most efficient mining sequence and method to ensure a safe and productive mining environment. The results of these studies and the conclusion of the crown pillar extraction will be published in the near future.
Global Approach to Managing Deep Mining Hazards

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1 INTRODUCTION

A series of fault slip seismic events have occurred at Garson mine. The events have occurred during development advancement exposing faults, after the extraction of single stopes due to mining induced stress change, and due to time effects after delays of many days during which no mining had been conducted. This paper discusses the study completed consisting of a global or holistic approach aimed at mitigating some of the risks associated with the structurally controlled rockbursts at Garson.

1.1 Background

Garson mine is located in the south east area of the Sudbury Basin (Ontario, Canada). Mining occurs in two main ore bodies (the #1 Shear and the #4 Shear) which are copper-nickel-sulphide and occur within shear zones that (as well as the ore bodies) strike approximately east-west and dip steeply south. From the north (footwall), lithologies progress (Figure 1a) from norite (NR), to greenstone (GS), and finally to meta-sediment (MTSD). A generally WNW-ESE striking, bifurcated sub-vertical dyke of olivine diabase (OLDI) cuts the series. The current active mining block at Garson is between 4700L and 5100L (levels denote depth in feet below surface).

2 COMPREHENSIVE ENGINEERING GEOLOGY MODEL

Garson mine data between years 1999 to 2007 were compiled in order to gain a more informed understanding of the mechanisms involved in the observed rock mass behaviour. The data included principally — but not exclusively — the exploration drillhole database, level mapping, seismic data, and interpreted faults zones. The main results of the EGM building are:

- The analysis of the fracturing at Garson mine could be used for the extraction of the dominant structural trends.
- The trends corresponded to some major structures with various characteristics such as; the 2500 structure system which is comprised of faults with anastomosed internal architecture and lenses of gouge with varying width; the 45° structure system which is made of not fully formed reverse faults occurring as bands of en-echelon tension gashes; and the non-continuous and offsetting OLDI dyke geometry.
- The micro-seismicity was not seen in direct geometrical relation with active mining but was strongly controlled by the identified structures. Moreover, interactions between the structures played a dominant role in the concentration of the observed seismic activity.
- The built EGM proved to be a reasonable model. Rockbursting experienced after the creation of the EGM has occurred in excavations projected to intercept the active structures.
3 NUMERICAL MODELLING

This section highlights two numerical modelling exercises to illustrate; (1) a back-analysis completed to further confirm the role of the 45° structure zone’s significance in the seismicity at Garson; and (2) modelling using displacement boundary conditions to more realistically account for the stiffness differences between the materials and faults to identify potentially highly stressed areas of ground. The main findings are:

- Modelling of the mining sequence and its impact on the stress and strain condition on the main identified structures confirms the importance of structure-structure interaction with respect to the slip pattern of a fault system and its related seismic activity.
- Displacement boundary numerical modelling of the area of interest including the main identified structures highlights the impact of the geological characteristics of the structures (dyke ‘undulations’ and holes, rock bridges between en-echelon fault systems or stiffness contrast between gouge filled rock and more brittle fault sections) on the possible initial stress state conditions. These conditions tend to induce heterogeneous stress fields.

4 CONCLUSIONS

A series of rockburst events at Garson mine which occurred over several years could not be explained using a direct relationship with stope extraction activity, as the events occurred relatively far from the active mining front or during periods without stope extraction in the project area. In order to better understand the involved mechanism in the rockburst activities, a global approach to address the risks associated with mining in such environments was developed. Such a global approach was proven to be necessary, as it appeared that the seismic activity at Garson mine was controlled by larger scale interactions between identified geological structures and thus can only be captured by a methodology considering interactions controlled by structures.

5 ACKNOWLEDGMENTS

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Modeling of mining-induced seismicity migration

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1. MINING-INDUCED SEISMICITY AND ENERGY FLOW

Mining-induced stress changes during excavation are generally associated with rock mass displacements, which are enhanced by chains of rotations of discontinuous rock mass grains/blocks. As a result, seismic events not only occur in the rock mass near the boundaries of mine excavations but also migrate further away at the mine wide scale, in deep underground mines. The latter type is more problematic and may be destructive to mining production due to their apparently unpredictable characteristics.

It is not practical to try to identify the structural geology on a mine wide scale with sufficient accuracy for conventional discrete element models to simulate the mine behavior. However, each seismic location is evidence of a rock mass damage, shown in Figure 1, by tracking and identifying the localization (through cluster analysis) and migration of damage at a mine wide scale, we can locate the seismically active zones and identify trajectories of damage migration.

![Figure 1. Seismic migration in rock mass.](image)

The faces advancing of openings triggers the energy flow, from the far ends to the openings, in the remnant rock mass. Under ideal conditions, e.g. elastic continuum assumption, the movement is localized by arching effects, and seismicity should not migrate beyond the localized zone.

But the heterogeneous and discontinuous nature of rock mass, together with locked-in unstable shear zones by historical movements, makes the energy flow structure unevenly distributed on site, deviating from the expected elastic behavior. The discontinuities and stress conditions form the energy flow corresponding to the openings disturbances on site, presenting a network structure (called channel element network in this research), in which mining-induced disturbances could migrate remotely away from the openings.

2. CHANNEL ELEMENT MODEL

A novel approach, the Channel Element Model (CEM), based on rock mass displacement and chain-like rotational movement along shear zones, is introduced in this paper. CEM is a mathematical method, with which to model rock mass remote interactions from source disturbances, in contrast to mechanical contact interactions by DEM, or constitutive relations by FEM to model near source disturbance responses.

In continuum mechanics, translations are governing and independent variables, whereas rotations are derived quantities from the translations. In the other word, rotation is not the freedom of the material movement. However, in discontinuous rock mass rotations are the freedom in the motion, and especially act as gears in shearing movement under high compressive condition at depth.
Figure 2. Rotation development at simple shear movement.

Figure 2. shows that rotations enhance the movement in the pre-existing shear zone, make a domino motion possible. The shear zones may outline as a network at a mine site, each zone is modeled by a channel element, and embedding in the rock mass, along which rotation and its induced slip movement are both possible under deep mining conditions. The paper presents a CEM network derived for a deep operating mine in the Sudbury Basin in Canada that is constructed by using historical seismicity records and exhibiting the space-time migration patterns at the mine site.

This model is a self learning system, its initial behavior is based on the historical data, however as mining excavation to depth progresses, the channel network is updated and its mechanical behavior re-evaluated to satisfy changing rock mass conditions and stress field.

Figure 3 shows a channel network, extending 1km in the longest dimension, which was identified with seismicity clustering. The links are unstable shear zones at a mine site, the nodes are intersection area of the links. A channel element model was built on it. As mining disturbances travel in the links between the nodes, the disturbances accumulate on nodes where they reach, so that the affected node is twisted, measured by moment. Once the moment exceeds the criterion on a node, a seismic event is triggered; and introduces new disturbances into the channel network. Here, the disturbance (a stress drop 50MPa) started at node A, triggered an event at B one hour later, then two disturbances interacted and migrated in the channel network, another event was generated at node C two hours later since the initial disturbance was introduced.

Figure 3. Disturbance and seismicity migration in a Channel Network.

3 CONCLUSION
Locked-in shear zones on site set up a network (channel network), in which rotations of discontinuous ensemble enhance rock mass movements, and migrate mining-induced disturbances away from mining openings, triggering seismicity in the remote area.

Shear slip is actually a multi-process composed of cracking, rotating, rolling and sliding, in which rotation acts as bearings.

The proposed Channel Element Model (CEM) presents an approach. Firstly, to identify lock-in marginal shear zones on mining site by seismicity clustering, the zones were exposed by seismicity induced by previous mining excavations historically. Secondly, to model disturbances (displacements by stress release) migrating in the channels with a phenomenological model.
Prevention and Control of Rockbursts in Dongguashan Copper Mine

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1 INSTRUCTION
A seismic monitoring system was installed in 2005 at Dongguashan Copper Mine, the deepest metal mine in China. Based on the seismic events monitored in situ, the possibility of rockburst prediction and prevention is investigated in this paper. The characteristics of space-time-intensity of mining-induced seismicity and the seismic response to mining are studied in detail using the seismic data in Dongguashan Mine. A conceptual model and a criterion of assessment of hazard seismic nucleation are proposed for Dongguashan Mine. In this model, the mine seismic stiffness method is adopted for rockburst prediction within the framework of the unstable failure theory. To analyze rock failure cases in Dongguashan Mine, the rate of the ratio of stiffness of rock in nucleation area to its surrounding area is proposed as a criterion for rockburst prediction.

2 GEOLOGIC AND MINING CONDITIONS
Dongguashan copper deposit is at 1000 m beneath the surface and it is controlled by an anticline. The average dip angle is about 20°. Its horizontally projected length is 1820 m and its horizontally projected width varies from 204 m to 882 m. The average thickness of the ore-body is about 40 m. The ore and its major surrounding rocks are very hard and thus are prone to rockburst during mining operation. The ore-body is divided into panels along its strike. The stopes are arranged along the long axis of the panels. The stope is mined and backfilled with cemented tailings afterwards.

3 SEISMIC RESPONSE TO MINING
3.1 The spatial-time-intensity of events
Analyzing the change of the spatial aggregations of events from September 1, 2005 to August 30, 2008 reveals that the events induced by driving shafts and developing drifts are located near these mining sites, and they are sensitive to these mining activities. On the other hand, the extracting of stope can induce seismicity in a much larger area. Generally speaking, aggregating events are located at temporary pillars and panel barrier pillars. A series of nephograms of displacement, $u$, and contour maps of logarithm of the apparent stress, $\log(\sigma_{\text{app}})$, at different levels are plotted for Dongguashan Mine to describe the variation of stress state (see Figure 1). The result shows that some stress concentration areas are not located at the aggregation areas of events, and that there are cases where the stress is not positively related to the deformation.
3.2 Conceptual model and the criterion for hazard seismic nucleation

Panel barrier pillars and temporary pillars restrain relative deformation of the walls, which can be roughly considered as compression-shear. In the framework of the theory of asperity in seismic source mechanism, these pillars are considered as asperities where seismic nucleation may occur. Therefore, we can develop a conceptual model of seismic nucleation at Dongguashan Mine, as shown in Figure 2.

![Diagram of conceptual model](image)

Figure 2 Conceptual model of seismic nucleation

4. MINE SEISMIC STIFFNESS METHOD FOR ROCKBURST PREDICTION

4.1 Measure of parameters for unstable failure

According to the mine stiffness theory, the rock burst occurs if the stiffness of rock in the seismic nucleation area is larger than that of the rock mass outside the nucleation area. We fit seismic data in equation \( \log E = c + d \log M \) to determine the average stiffness of this area in a given time period. The slope, \( d \), presents the stiffness of nucleation area. The \( b \) value in the G-R relation is used to determine the mine stiffness of mine loading system.

4.2 A criterion for rockburst prediction

We analyzed the \( b \) and \( d \) parameters using the data recorded at Dongguashan Mine to predict rockburst using seismic stiffness. A typical case is shown in this paper. From Sept. to Oct. 2006, a few rock failures occurred in surrounding rocks of drifts under the panel barrier pillar along the #54 exploratory line. We proposed the change of the ratio of \( d \) in the nucleation area to \( b \) in the loading system to measure the change of the relative stiffness of bath areas, and further to determine the possibility of rock burst. The ratio of \( d \) to \( b \) is defined as \( S \). The curse of \( S \) with time period is shown in Figure 3. The change of \( S \) reflects the relative change of the stiffness of nucleation area to that of the load system area. Therefore, we defined the \( dS/dt \) as a rock burst prediction criterion in mine, i.e., increase of possibility of rock burst if \( dS/dt > 0 \), decrease of possibility of rock burst if \( dS/dt < 0 \).

5. CONCLUSIONS

The characteristics of space-time-intensity of mining-induced seismicity are studied with the data of seismicity in Dongguashan Mine. A conceptual model of hazard seismic nucleation is proposed for this mine. A mine seismic stiffness method of rockburst prediction is proposed. The variable rate of ratio of stiffness of rock mass in nucleation area to its surrounding rock’s is used as a criterion of rockburst prediction.
Weak Rock Mass Span Design – Best Practices

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1 INTRODUCTION

As more and more mines are being developed in weak and difficult ground conditions, this presents potentially difficult and hazardous mining conditions to workers in the industry resulting in a higher frequency of injuries and fatalities. Evidence of this can be shown by the number of fatalities and injuries resulting from uncontrolled rock falls during the time period of 1990 through 2007 (Error! Reference source not found.) with a low of two (2) in 2004 and a high of 28 in 1995 and 1997 (Hoch, 2000 and Brady, 2008). In mid-1999 NIOSH started conducting visits and discussions with Nevada mines regarding weak rock and ground falls resulting in a statistical decline of ground fall related injuries over the next two years (Brady et al. 2005). An increase in ground fall related injuries occurred in 2002 and in the middle of that year, NIOSH commenced technical mine visits. There was another spike in injuries in 2005. The last two years have had relatively low numbers of ground fall related injuries. However, 2007 experienced one fatality from a fall of ground. Weak rock conditions are a concern and will continue to be in the years to come.

The relationship between span and rock quality has been studied for decades. One relationship of particular interest is that of the Critical Span Design Curve (Lang 1994) that has been widely used throughout the industry. The curve presented by Lang (1994) was later updated by Wang (2002). The Critical Span Curve is a simple and useful tool that aids in the design of underground man-entry openings. There is a need to update the Critical Span Curve for the RMR range of 20-50, as there are an increasing number of mines that are operating in these weak ground conditions. The augmentation of this design to include a larger database of 463 points in the range of RMR of 15-60 will increase its accuracy and reliability in such conditions.

Ground Fall Related Injuries in Nevada, 1990-2008
The University of British Columbia Geomechanics group and the NIOSH Spokane Research Laboratory have been conducting research in the development of safe and cost-effective underground design guidelines in weak rock environments with RMR$_{76}$ in the range of 20 to 60. An update of the Span Design Curve was conducted for this weak rock mass range. A total of 463 points were added to the database. The development of the weak rock augmented Span Design Curve has been separated into four different support categories; Pattern Friction Sets (A), Pattern Friction Sets with Spot Bolting of Rebar (B), Pattern Friction Sets with Pattern Rebar Bolts (C) and Cablebolting, Shotcrete, Spiling, Timber Sets or Underhand Cut and Fill under Cemented Rock Fill (D). Category D includes cablebolts and other engineering designed support systems such as cemented rock fill (underhand cut and fill mining), significant application of shotcrete (typically 76mm), spiling or timber sets.

Neural network analyses were conducted on the span-RMR$_{76}$ relationship for these four support categories. Categories A, B and C obtained acceptable correlation. Category D, however, did not. This is most likely due to the varied engineered support systems which act differently on the rock mass resulting in distinct support mechanisms with different factors of safety. Category A yields good results and follows what is seen in the field. These results also fit well with Barton’s relationship between Q and D$_e$. At an RMR$_{76}$ value of 25, the maximum stable span is 3m. However, at an RMR$_{76}$ of 25, there is a drastic transition between the Stable/Potentially Unstable zones and the Potentially Unstable/Unstable zones. There is a very small to nonexistent Potentially Unstable zone. Caution should be used when at these low RMR$_{76}$ values due to this lack of Potentially Unstable zone. Openings can very quickly go from being Stable to Unstable. Even though the database represents the North American mining industry well with 7 mines participating in the database, caution should be used as the dataset is small with 47 points. Categories B and C yielded similar results with the Stable/Potentially Unstable line moving up on the graph (increased span values). However the Potentially Unstable/Unstable line moved towards the right on the graph (increased RMR$_{76}$ values). This is unexpected, but may be explained by the difficulty experienced in the installation of resin grouted rebar. Due to this uncertainty in the accuracy of the data of Categories B and C, it would be imprudent to rely on the data interpretation in span design for these categories. Category D, the “heroic” category did not obtain positive results from the neural networks analysis, but still demonstrates that spans can be stable at lower RMR$_{76}$ values with detailed engineering support design.

The calculated ½ span failure Factor of Safety was found not to be significantly relevant in any category when applied to the prediction of stability in the relationship between the span and the RMR$_{76}$. In comparing the calculated Factor of Safety of all four categories, it was found that small spans in Category D were approximately eight times more supported (FS is eight times greater) than the corresponding small spans in Category A. As the span increased, the difference in the support capacities of the two categories diminished. At a span greater than 10m the difference in the support capacities became negligible. Due to the uncertainties identified previously, it would be imprudent to relate Categories B and C. Category A is deemed “Unsupported” with the Factor of Safety being less than 1.2. The rock mass design is valid for these spans; however, care must be taken to ensure that potential structural failure planes are not present. Category D is deemed “Supported” with the Factor of Safety being greater than 1.2 and is supported in terms of structurally controlled failures that encompass ½ span.

It has been observed that resin grouted rebar is difficult to install in weak rock. Full resin coverage of the bolt is difficult to achieve due to the jointed nature of the rock mass. This incomplete coverage, leaving the toe of the bolt ungrouted, would result in a decrease in effective length of the rebar bolts. This could be a reason why there are so many spans in the previous Potentially Unstable zone that have failed. The use of resin grouted rebar in weak rock environments could give an operator a false sense of security if the bolts are not installed properly. It has also been observed that the North American mining industry is moving away from the use of resin grouted rebar in weak rock masses and switching to frictions sets.

As with any empirical design, it is important to understand the data behind the design. These designs are for rock mass only. They do not incorporate design based upon structure and/or stress states. Small scale structure and/or changes in stress states may lead to a change in the RMR$_{76}$ of a given area. A new RMR$_{76}$ calculation may be done to reflect the change(s) and allow these empirical studies to remain valid. The empirical design graphs presented in this paper are intended to aid the experienced operator in making safe and economical design decisions.
SESSION 17  HIGH SLOPES AND OPEN PITS

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Importance Of Understanding 3-D Kinematic Controls In The Review Of Displacement Monitoring Of Deep Open Pits Above Underground Mass Mining Operations
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Numerical Modeling of Shear Stress and Displacement Reversals as a Pit Floor Passes a High Wall and Implications for Progressive Shear Strength Degradation
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Importance of understanding 3-D kinematic controls in the review of displacement monitoring of deep open pits above underground mass mining operations

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1 INTRODUCTION

Surface mining operations are increasingly involving the design and development of deeper and larger open pits, often with plans to transition into underground mass mining operations. Experiences at the Palabora Mine in South Africa demonstrate the complex interactions between underground workings and the steep slopes above that may lead to slope movements that evolve to pose safety and economic concerns. To properly assess the rock mass response to these interactions for planning of future developments, understanding the controlling influence of geological and geomechanical structures is crucial. Preliminary results are presented from a detailed review of historical slope monitoring data from the Palabora Mine, before and after the initiation of block caving 400-m beneath the 800-m deep pit. This paper demonstrates that a detailed analysis of geology and monitoring data can be used to better understand the kinematic controls of complex rock slope displacements and interactions between open pits and underground mines. This investigative treatment of the monitoring data is an important first step prior to its use for calibrating and constraining sophisticated 3-D numerical models.

Monitoring programs involving geodetic prisms form a key component of most slope stability programs for modern open pits which provide data that may be used to quantify the nature and extent of the hazard, the kinematics and stability state of the slope, or to provide early warning of an impending failure. Issues of uncertainty relating to the geological conditions, slope kinematics, and failure modes provide obstacles that contribute to a lack of understanding of the potential for failure.

Often, large-scale geological structures (e.g. major fault zones) play a dominant role in controlling the kinematics and stability of large pit slopes. With the added consideration of complex rock mass stress-strain interactions in response to rock mass failure and underground mass mining operations, the resulting 3-D deformation pattern of the pit slope becomes progressively more and more difficult to interpret with respect to how it will evolve. This creates a serious obstacle in the stability assessment of the slope or setting early warning alarm thresholds for workers in the pit (e.g. during scavenging operations), near sensitive surface structures located near the crest of the pit slope, or underground.

To contend with these complex interactions, numerical modeling offers a powerful means to break down problems into their constituent parts and analyse cause and effect relationships and their evolution. However, in order to effectively model the rock slope, a tight control on the representation of the geology is required for which monitoring data can be used as an important constraint. Yet the interpretation of monitoring data is far from straightforward and can be affected by the same issues of rock mass complexity and variability as affects the numerical models the data are meant to constrain.
The Palabora Open Pit Mine, located approximately 500 km northeast of Johannesburg, South Africa, began production in 1966. In 2002, as the pit neared completion, it measured approximately 800 m in depth and 1650 m across. A transition to an underground block cave mining operation was initiated in April 2001 with important caving milestones including: Dec. 2002, the crown pillar de-stressed; Mar. 2003, hydraulic conductivity between pit bottom and cave; and Dec. 2003 (estimated), cave break through into the pit.

There are four main faults and three dominant joint sets observed found within the pit which are mostly sub-vertical.

A series of complex slope movements within the northwest wall of the pit began to occur in 2003 in response to the developing block cave below. Pit slope displacement began to occur shortly after the crown pillar became de-stressed in December 2002. Movement of all pit walls increased substantially upon cave breakthrough into the bottom of the pit with the largest deformations observed in the north wall where cumulative displacements in excess of 1.5m were measured.

With planned future developments and expansion of the block cave at Palabora, involving an extension under the west wall and deeper second lift, understanding the kinematic state and key geological controls within the pit is of primary importance. An analysis is undertaken considering the large-scale structures such as faults, shear zones, and dykes found within the pit. Prisms around the pit were plotted with incremental displacement rates, cumulative displacement, and direction of displacement with respect to these major features. From this, any potential relationships between the movements observed and the geologic features present could be resolved. The assessment included geodetic data collected between 1984 and 2004.

Prior to caving, review of the prisms reveal that the rates were fairly consistent throughout the pit with movement typically explained by normal rebound associated with the excavation of the open pit (Figure 1).

As caving operations developed, from April 2001 up to before the 2004 pit wall failure, the pattern of displacement indicates that the rock slopes began to be drawn towards a location beneath the northwest wall (Fig. 1), roughly corresponding with the location of the eventual failure.

The example drawn from the Palabora Mine in South Africa shows that by integrating displacement monitoring and geological data, a better understanding of the kinematic controls of complex rock slope displacements and interactions between a deep open pit and an underlying block cave operation can be achieved.
Numerical Modeling of Shear Stress and Displacement Reversals as a Pit Floor Passes a High Wall and Implications for Progressive Shear Strength Degradation

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EXTENDED ABSTRACT

Surface coal mines in mountainous terrain are susceptible to footwall slope instabilities. The application of numerical modeling techniques for the analysis of the footwall failure modes has recently become popular. However, the important capability of numerical modeling in simulating the history of the mining operation has not been widely adopted by researchers and engineers. Simulation of the excavation history also makes it possible to understand progressive strength degradation of the remaining rock mass around an excavation. Rock excavation alters the initial state of stress adjacent to the excavation. The elastic rebound caused by the unloading process can reduce the strength of rock mass and bedding surfaces near the excavation. This paper deals with modeling the behaviour of footwall slopes and associated excavation-dependent progressive shear strength degradation or damage to bedding surfaces.

The finite element program, Phase2, was used to assess the effect of excavation unloading and the consequent elastic rebound on progressive shear strength degradation of bedding surfaces. For this purpose, a hypothetical planar footwall slab was modeled with multiple excavation stages that progressively exposed the slab. The initial maximum horizontal stress to vertical stress ratio was 2. An elastic-perfectly plastic joint boundary consisting of 30 joint elements was used to simulate the slip surface at the base of the slab. Stage 1 in this model simulates the development of the original ground surface. All the displacements were set to zero at this stage. Stages 2 and 3 represent the excavation of the rock mass above the slab. Stages 4, 5, 6 and 7 simulate the process of downcutting of the pit floor along the slab. In the last two stages (stages 8 and 9), the slab is fully exposed and free to slide along the joint boundary according to the Mohr-Coulomb strength parameters. Figure 1 displays the locations of the pit floor at stage 7.

The effects of the unloading process and elastic rebound on stresses and displacements along the joint were assessed. The distributions of shear stress and displacement along the joint at different stages were obtained from the joint data. Results demonstrated that initially the slab moves upward from stage 2 to 4. At stage 5, where half of the slab is exposed, the upper portion of the slab begins to slide downward. At this stage when this portion reverses its shear direction, the remaining part of the slab continues to move upward. With subsequent stages, a larger portion in the upper part of the slab reverses its shear direction and begins downward slip and a smaller portion in the lower part of the slab moves upward. After all the excavation stages, the joint still has a net upward displacement everywhere. This means that the slab started to move upward along the joint due to the elastic rebound from stage 2 to 4, and then progressively moved downward from stage 5 to 9 due to the weight of the slab. However, even at stage 9 the slab has not yet returned to the position from which it had started to move upward at stage 2.
The elastic-perfectly plastic joint has a ‘memory’ of its stress and displacement history associated with the pit floor being mined past the slab position. Figure 2 shows graphically the positions of shear stress or displacement reversals relative to pit floor levels at different stages. It is believed that this process can progressively degrade the shear strength of bedding surfaces.

The numbers and locations of yielded joint elements caused by stress relief due to excavation unloading at different stages were also obtained. The results suggest that the largest degree of shear strength degradation occurs when the footwall slab is being passed by the pit floor. The locations of yielded joint elements indicate that the shear strength progressively degrades from the top to the bottom of the joint as the pit floor approaches and passes the footwall slab to deeper levels. The global factors of safety for slab slip at different stages for different values of stress ratio were also determined. The lower factors of safety obtained at stages 4, 5 and 6 further supports the higher degree of shear strength degradation at these stages rather than when the slab is fully undercut. Moreover, the shear strength degradation at these stages increases (global factor of safety decreases) with increasing stress ratio from 1 to 3.
Influence of shear surface geometry on deformation processes in massive landslides

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1 INTRODUCTION

Slope movements of the Downie Slide, monitored using borehole inclinometers, extensometers, and survey monuments, show considerable spatial and temporal inconsistencies where different regions of the landslide mass do not move at the same rate and do not follow a common downslope sliding vector. Irregular displacements, indicative of compound or complex landslide processes, are influenced by a number of factors including: topography, groundwater, heterogeneous rockmass characteristics, and non-uniform shear surfaces (Hutchinson et al. 2006, Agliardi et al. 2001). This study assesses the influence that inferred shear surface geometry, as a key factor contributing to variable slope movements, has on resolving spatially discriminated slope deformations in three-dimensions.

Two-dimensional plain strain and simplified, spherical or bowl-shaped, three-dimensional models are commonly used in landslide analyses. However, these approaches have inherent geometric limitations, as the irregular nature of displacements in large landslides is an indication that the geometry of shear surfaces is more complex than such simplified models represent. Advances in three-dimensional numerical modelling techniques allow for complex and, in most cases, more geologically realistic shear surface geometries to be used for simulating slope behaviour. Using the Downie Slide, a massive active composite extremely slow moving rockslide located in southeastern British Columbia, Canada, as a case history, the sensitivity of slope behavior to various interpretations of three-dimensional shear surface geometry is explored.

2 METHODOLOGY

Three-dimensional shear surface geometry can be interpreted from geological mapping, aerial photographs, digital elevation models, and subsurface drilling campaigns. With this data, spatial prediction algorithms can be used to interpret fully three-dimensional shear surface geometries. Geometric analysis by Kalenchuk et al. (2009) has resolved four geologically reasonable, and one simplified interpretation of the principal shear zone, at Downie Slide (Fig. 1). There are large-scale geometric discrepancies between the continuous (a, b and c), stepped (d), and simplified (e) geometries, while small-scale variations distinguish each of the continuous geometries.

Figure 1: Interpretations (looking northwest) of the Downie Slide basal slip surface; continuous defined by (a) a minimum curvature algorithm, (b) krigging of a variogram model and (c) a multiquadratic function, stepped defined by (d) a discontinuous minimum curvature algorithm and simplified defined by (e) an elliptical parabola (modified from Kalenchuk et al., 2009).

The selection of a spatial prediction technique requires statistical analysis and expert judgment in order to define which algorithm best suits the available data set and produces the most realistic representation of the geological setting. Multiple, and different, surface geometries can be interpreted for any given data set, producing small- and large-scale discrepancies between different spatial predictor techniques and geological assumptions of surface continuity. Numeri-
cal models have been developed to compare slope behavior for each interpretation of shear surface geometry using 3DEC (3-Dimensional Distinct Element Code) (Itasca Consulting Croup, Inc. Minneapolis, Minnesota, 2003). Both continuum and discontinuum modeling methods are incorporated to model the rockmass matrix and shear surface, respectively.

3 RESULTS

Numerical simulations of varying shear geometries demonstrate that the three-dimensional shape of shear surfaces in massive landslides is a key factor controlling slope deformations. The simulated behavior of Downie slide proves to be sensitive to large-scale geometric variations, while small-scale discrepancies in slope geometry do not significant change simulated slope behavior. Figure 2 demonstrates that the continuous shear surfaces generated using minimum curvature algorithm and kriging of a variogram model best simulate observed slope behavior.

Figure 2: (a) Oblique view of Downie Slide 3DEC model. Contoured displacement rates (b) measured by survey monuments and sampled in numerical simulations using continuous (c) minimum curvature (d) kriging of a variogram model and (e) the multiquadratic function, stepped (f) minimum curvature, and simplified (g) the elliptical parabola geometries.

4 CONCLUSIONS

Geotechnical analysis of massive landslide behaviour is limited by oversimplified slope geometries in two-dimensional cross sections or three-dimensional spherical or bowl-shaped slip surfaces. Spatial prediction algorithms are used to interpret geologically realistic three-dimensional shear surface geometries, improving the analysis of massive landslides. Multiple, and different, surface geometries have been interpreted for the Downie Slide, small- and large-scale geometric discrepancies result from the use of different statistical interpolation methods and geological assumptions of surface continuity respectively. Numerical models have explored how variations in shear surface geometries influence the complex behaviour of this massive landslide, proving that large-scale geometric discrepancies do influence simulated slope behavior, while small-scale discrepancies are less significant.

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Selective highwall mining at the Dome Open Pit mine

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ABSTRACT: Following completion of planned mining operations at the Goldcorp Dome Open Pit Mine, a high-grade Shaft Pillar located in the pit highwall was recovered. Mining of this Shaft pillar faced numerous rock engineering design and implementation challenges due to its remote position on the highwall and the presence of mine workings within the wall.

The case example presented in this paper discusses the geotechnical investigation performed at the site, novel measures undertaken for installation of ground support required for rockmass reinforcement and for worker protection, as well as operational challenges to successful mining of the Shaft Pillar.

The geotechnical investigation described in the paper included a review of mined geometries in the vicinity of the area of interest (see Figure 1); compilation of available structural information (See Figure 2); from which an assessment of structural wedge potential was performed.

The findings from the geotechnical evaluation were used to develop strategies for ground reinforcement and excavation design and sequencing. Cablebolt pre-support consisting of plated, full column grouted cablebolts was required to stabilize anticipated wedge intersections. Locations of ground reinforcement are illustrated in Figure 3.

Support installation, drilling and blasting was performed by contractors skilled with working on highwalls. Because of the remote access to the site, all heavy or bulky materials were lifted up to the worksite by either long-reach cranes or by helicopter. Daily travel to and from the worksite required rappelling on the highwall face.

Figure 4 illustrates the outcome of the selective highwall mining project. Because of the non-standard activities required for mining of the Shaft Pillar, numerous non-routine hazardous task reviews were completed during the project. In all, the project was successfully completed with 15,792 incident free man-hours.
Figure 2. Geology plan from Dome mine, #7 Level. Insert: Pole plot concentration of compiled underground and surface data at investigation site

Figure 3. Cablebolt pre-support locations

Figure 4. Before and after comparison of Shaft Pillar recovery
A new design approach for highway rock slope cuts based on ecological environment protection

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ABSTRACT: With China's fast development, especially that of highways in mountainous areas, there are many unavoidable high cut rock slopes. Specifications for the design of highways in China prescribe that the design of any slope with a height of more than 30m should be compared with bridge and tunnel design schemes, economically. Despite existing guidelines, many high slopes are designed as equally high cuts based on traditional design methods, due to the engineering geological conditions and highway alignment. Actually, these excessively high cuts relative to the natural slope height seriously damages the original ecological environment and groundwater systems, in some cases irreversibly. With the aim of reducing ecological damage, we propose a new design approach which minimizes the ecological and environmental disturbance by decreasing the amount of cut slope through a steep excavation scheme. This approach requires that: (1) the geological conditions are good and the rockmass is self stable; (2) damage to the rock is minimized through use of controlled blasting (buffers, pre-splits); (3) a 20-m high bench be introduced if the local geological conditions are not very good; (4) reinforcement is added where needed to ensure stability during construction; and (5) deformation of the slope and working condition of the reinforcement be monitored to ensure stability. This new design is demonstrated through its application to a cut slope along the provincial trunk highway S223 in Guangdong province. Results from numerical modeling using UDEC reveal that compared with a traditional design scheme for which the cut would be 90 m high, a steeper cut limiting the height to 23.5 m, with the remaining height following its natural profile, would generate a comparable stability state. With cable reinforcement, the stability of slope excavated steeply can be further assured.

1 INTRODUCTION

With China's fast growing economy, numerous road construction projects have obtained great achievements. However, with highway construction, especially in mountainous areas, there are many unavoidable high cut slopes. Specifications for the design of highway subgrades prescribe that the design scheme of slopes with heights greater than 30m should be compared with bridge and tunnel schemes, economically (JTG-D-30 2004). Despite existing guidelines, it is still quite common to see many high slopes being designed as high cuts due to the engineering geological conditions, highway alignment, costs and issues related to traditional design ideas. In some cases, the traditional design method of “high backfill, high cut” for mountainous highways seriously damages the original ecological environment and groundwater systems, especially to those areas that are ecologically sensitive where damage is often irreversible. It is quite common practice to add vegetation to greenify a cut slope, however this still has problems such as low survival ratios, high maintenance costs and lack of species diversity. With increased awareness of the importance of the ecological environment, it becomes more urgent to decrease the damage to the ecological environment while ensuring stability during the engineering of high cut slopes.
With the aim of reducing ecological damage, we propose a new design approach which helps reduce ecological and environmental damage by reducing the height of a cut for a high rock slope by steepening the angle of the cut, while ensuring that the slope is both safe and economically feasible.

2 CONCLUSIONS

By introducing the steep excavation scheme to the case study, the following conclusions may be drawn:

(1) The steep excavation scheme requires that the engineering geological conditions are good, the rockmass self-stable; if the local geological conditions are not very sound, the excavation should be benched.

(2) The steep scheme also requires that the excavation process should not badly damage the rock. To do so controlled blasting measures may be required and/or the cut slope should be reinforced more strongly than usual.

(3) The deformation of the slope and the working conditions of the reinforcement should be monitored to see how the slope responds.

(4) The steep excavation is better than the traditional scheme in terms of ecological environmental effect, slope height, the excavated amount of rock and soil, the amount of disturbed area on the slope surface, and engineering cost.

(5) If the geological conditions permit, the steep cut scheme, which can ensure that slopes are both safe and economically sustainable, should be used preferentially.

With cable reinforcement, the stability of the steeply excavated slope can be guaranteed. Deformation of the slope and the cable force are monitored in detail during the construction period. The deformation data showed that slope movements decreased gradually with the final deformation being very small. The cable tensions indicated the need to re-tension the anchors, with subsequent tensions being small. It is optimistically estimated that the steep slope should be stable during operation. Comparison between the steep and common schemes affirms that the steep excavation is better than the traditional scheme in terms of ecological environmental impact, cut slope height, the excavated volume, the disturbed slope surface area, and engineering cost. It is concluded that if geological conditions permit, the steep cut scheme should be used given its benefits.

ACKNOWLEDGEMENTS

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REFERENCES


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Dynamic Testing of Threadbar used for Rock Reinforcement

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ABSTRACT: A comprehensive axial test program has been undertaken at the Western Australian School of Mines (WASM) Dynamic Test Facility on the performance of 20mm threadbar. The threadbar tests had a double embedment configuration, with variation to the encapsulation lengths, debonded length and surface hardware and fixtures to examine failure mechanisms. Performance was significantly affected by the collar embedment length and type of fixture nut used. Results ranged from thread shearing occurring between the nut and bar at 1kJ of energy dissipated, to plastic deformation of a central debonded length dissipating 22kJ.

1 INTRODUCTION
Threadbar is widely used in mining applications for the reinforcement of static and dynamic instabilities. Cement grout or resin is used to encapsulate the elements in a borehole drilled in rock. The elongation capacity of the overall reinforcement system may be increased by creating a free length between the toe and collar regions of the bar; either by having a short encapsulation length at the toe end or by decoupling the bar from the encapsulation medium. The latter method is preferred as it provides better resistance to shear movement across the axis of the borehole. The collar fixture comprises a domed plate, spherical washer and nut; the latter two components maybe integrated as one item.

The Western Australia School of Mines (WASM) Dynamic Test Facility has been used to quantify the performance of a number of commercially available threadbars. The effects of changes in embedment lengths and surface fixtures on the performance of 20mm threadbar to dynamic loading were investigated. WASM static and dynamic tests are carried out under axial load conditions, and may not entirely represent the shear loading that can occur underground. Shear loading of a reinforcement system underground is particularly uncertain. The performance will be significantly affected by the quantity of available dilation on the shearing structure, how intact the rock remains that is applying the shear loading, the rate of debonding / fracturing between the threadbar and encapsulation medium. An important influence for performance is whether the shear displacement is concentrated at one location on the bolt, or multiple locations. Concentrated shear will result in failure at a smaller displacement. The effective load transfer rate and hence system performance will be influenced by the components shear strengths, strain rate capabilities, and the interface conditions between the steel, grout and rock.

The testing program examined fully bonded threadbar and partially debonded threadbar both encapsulated in cement grout and resin encapsulated threadbar installed by a Jumbo and toe anchored.

2 RESULTS
2.1 Fully Bonded Threadbar
The encapsulated threadbar required plastic deformation of the steel bar at the simulated discontinuity to dissipate the input energy from the dynamic load. The dynamic axial loading and partial threads of the bar allow the grout to interlock and the shaft to break under some critical
loading conditions. The critical loading conditions are related to the rate at which the energy is consumed in plastic deformation of the steel bar compared with the fracture growth between the steel bar and grout interface. Different responses of the reinforcement system occur between critical and non-critical loading situations.

2.2 Partially Debonded Threadbar

The debonded threadbar required plastic deformation of the steel in the debonded length to absorb the input energy. To achieve this, the collar mass needed to transfer the load through the surface hardware and the side of the simulated bore hole onto the short length of encapsulated threadbar in the collar section. The performance of the reinforcement system was controlled by the surface hardware (particularly the nut fixture) that was used. Previous research has shown that for strain rates approximating one strain per second, a dynamic increase factor of approximately 1.3 in yield and ultimate strength capacities for reinforcing bar of nominal 550MPa yield stress can be experienced. For 20mm threadbar this increases the average yield load from 165kN to 213kN; a value that is in agreement with the dynamic yield load assessed in the facility.

2.3 Resin Encapsulated Threadbar – toe anchored

All reinforcement systems failed by early stripping of the nut and thread off. The short load duration leading up to failure of the bolt made analysis quite difficult. The capacity of the nut in response to dynamic loading is a major deficiency of the system.

3 CONCLUSION

A summary of the three reinforcement systems against two dynamically capable rock bolts (22mm Cone bolt and Garford Yielding Bolt) are shown in Figure 1.
Quantifying the rate of corrosion in selected underground mines

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1 INTRODUCTION

Corrosion of support systems can be a major safety and economic concern in underground hard rock mines. An improved insight on the factors that contribute to the corrosion of support systems can aid in the selection of appropriate support strategies and a reliable assessment of the predicted useful life of a support system.

This paper presents the preliminary results of on-going investigations in selected underground hard rock mines in Québec. This involved visual observations and the use of a corrosion classification system. Furthermore, a comprehensive testing program was established to monitor aqueous and atmospheric corrosion at selected sites. This involved direct measurements using corrosion coupons, electrical and analytical methods.

2 GROUNDWATER AND AQUEOUS CORROSION

The pH values of the water samples varied from 3.4 to 8.0. Oxygen solubility ranged from 5.9 to 15.6 ppm. The undertaken chemical analysis revealed a high concentration of aggressive ions such as Cl⁻ and SO₄²⁻. An operating mine is a dynamic environment where conditions and access are subject to production constraints. This can make long time observation difficult.

3 ATMOSPHERIC CORROSION

Atmospheric corrosion is the corrosion or natural degradation of material exposed to air and its pollutants. The rate of atmospheric corrosion is influenced by the relative humidity (the ratio of the quantity of water vapor present in the atmosphere to the saturation quantity at a given temperature). Corrosion rate increases beyond a critical humidity of over 60%. Atmospheric corrosion is further accentuated by the presence of pollutants such gas and particles. All these conditions are often present in underground mines. Furthermore, the ambient heat in deep mines also has a direct impact on the corrosion resistance of support systems. It is generally accepted that corrosion activity will double for each 10°C raise in temperature.

4 DETERMINATION OF CORROSION RATES BY TESTING OF COUPONS

A field testing program was designed to evaluate the type and rate of corrosion in a range of underground mining environments. This involved the use of corrosion coupon testing. In a mining context, coupons were used by Villaescusa et al (2008) to measure corrosion rates in several Australian underground mines.

Figure 1 illustrates installed coupons at two sites. The first one was installed in relatively dry conditions while the second example is from an area under direct water flow.
The initial total surface area and the mass lost during the test are determined. The average corrosion rate (mm/year) was obtained as follows (ASTM G1-03):

\[
\text{Corrosion Rate} = \frac{K \times W}{A \times T \times D}
\]  

(1)

where: \( K = 8.76 \times 10^4 \) (for units in mm/y); \( T \) = time of exposure in hours; \( A \) = area in cm\(^2\); \( W \) = mass loss of coupon in grams, and \( D \) = density of coupon in g/cm\(^3\).

Based on the 2.5 month and 5.5 month readings the corrosion rate of steel is higher for aqueous corrosion as compared to atmospheric corrosion. As expected corrosion rates are lower for galvanized steel. The results are still analyzed in order to address the impact of humidity and condensation.

5 CONCLUSIONS

A comprehensive research program on support system performance in corrosive environments has been undertaken with the collaboration of 6 mines. This investigation aims to characterize and monitor the performance of support systems over time several environments. Preliminary results suggest that it is possible to quantify the performance of support systems under corrosion. As more data become available these relationships can lead to useful information on the choice of support systems and reliable estimates of their longevity.
In-situ measurements of cemented paste backfill in long-hole stopes

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EXTENDED ABSTRACT

Cemented Paste Backfill has proven to be of critical importance to the operation of a number of Canadian mines. This is primarily due to the rapid backfilling rate this method permits. However, little field data exists on the in-situ behaviour of CPB, and so emplacement strategies are necessarily conservative. There exists the potential to improve the efficiency of this backfill system. For instance, to avoid exceeding the strength of a fence, many stopes are backfilled in two stages, with an initial plug being allowed to cure for 24 or more hours, to provide a barrier between the fill fence and the main volume of the stope pour. Eliminating the two stage pour would further reduce stope cycle time. Fill fence design may also be improved with a better understanding of the pressures generated by the CPB. To explore this opportunity, a program of large scale in-situ field tests is underway at three mines.

This paper presents field work conducted at Xstrata Copper’s Kidd mine. An open, long-hole stope was instrumented with geotechnical instrumentation and subsequently backfilled to provide a detailed picture of the spatial and temporal evolution of pressure within the backfill. The stope was 32 m high with a footprint of 28 x 19 m. It featured two fill fences to contain the CPB. Instrumentation included specially designed clusters, each containing three orthogonal total earth pressure cells (TEPC), a piezometer for pore water pressure, a heat dissipative sensor for negative pore water pressures, and an electrical conductivity probe for cement hydration information. Four of the clusters were suspended vertically in the centre of the stope to capture representative pressures throughout the fill volume. Two clusters were positioned just beyond the brow of the stope, and within the drift between a fill fence and the brow, in order to evaluate arching of stress with distance along a drift. Additionally, TEPCs and piezometers were mounted directly on both fill fences, and fence displacement was measured by an array of Linear Variable Displacement Transducers (LVDT) mounted on the free side of each fence. Accelerometers were also installed in the CPB and rock mass to measure blast induced vibration response, with a view to providing information on the liquefaction potential of CPB.

The stope was backfilled to a height of 8 m with CPB comprising 4% binder. The remainder of the stope contained CPB with 2% binder. The maximum total earth pressure during backfilling was measured in the vertical orientation, at cage 3, hanging 3 m from the floor near the centre of the stope. Generally, the lowest total earth pressures are measured in the drift in front of the fill fence. Pore pressures in this area are low and fall off rapidly due to local drainage through the fence. Also within the 4% binder CPB, cage 3 measures relatively low pore pressures that fall off within 1 day.
Hydrostatic loading persists for less than 8 hours in the 4% CPB. Within the 2% binder content CPB, loading is hydrostatic for in excess of 2 days.

Arching within a drift is demonstrated in Figure A1, where total pressures acting in the direction of the fill fence are plotted against time for four TEPCs mounted along the drift. Over the 6.5 day period of the pour, the pressures decrease with increasing distance from the stope brow. The pressures are very similar for the first 6 hours, due to the early stage of cement hydration and the limited head pressure. Between 22 and 24 hours after burial, the pressures measured on or close to the fence decline.

This paper also presents the results of long term monitoring. The effects of production blasting, and a Mn 3.8 rock burst in the near vicinity of the stope are manifest in changes in total earth pressures within the backfill. These changes are due to the evolution of local rock mass strain conditions inducing wall closure within the stope, thus increasing the confinement of the CPB.

Figure A1: (A) Total earth pressures during the 6 days of backfilling, for TEPCs on the fill fence, in the drift and just under the brow as shown in (B). All pressures are oriented towards the fill fence.

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Dynamic tests of a new type of energy absorbing rock bolt – the D bolt

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1 INTRODUCTION

The D bolt is a new type of energy-absorbing rock support element recently developed in Norway. The D bolt is not only able to accommodate large rock deformations, but can also provide a load-bearing capacity as high as that of rebar bolts. Furthermore, the anchoring quality of the D bolt is similar to rebar bolts because of its multi-point anchoring design.

2 D BOLT

The D bolt is made of a smooth steel bar that has a number of integrated anchors evenly or unevenly spaced along the length of the bar, Figure 1. In the current design, the anchors are stronger than the shank of the bolt. The shank, rather than the anchors, yields and ultimately fails under extreme loading conditions. D bolts use the anchor paddles to mix resin.

The D bolt is fully grouted in the borehole with either cement grout or polyester resin. The anchors are fixed in the grout, while the smooth bar sections between the anchors have no or very weak bonding to the grout. When rock dilation occurs between two anchors, the anchors limit the dilation so that a tensile load is induced in the bar section between the anchors. The section will elongate elastically in the beginning, but it will quickly get to yield with a small amount of elongation. Plastic elongation occurs next and continues until the ultimate strain limit. The D bolt fully utilises both the deformation and load capacities of the steel material. The energy absorption capacity of a 20 mm diameter smooth steel bar is approximately 30 – 40 kJ/m.

Every section between anchors works independently. Failure of one section or loss of one anchor only locally affects the reinforcement effect of the bolt. The other sections or anchors still provide reinforcement to the rock as usual.

The bolt samples for drop tests are 1.6 m long and 20 mm in diameter, Figure 2. Every bolt sample has three anchors along its length. The energy absorption capacity of the bolt section between anchors 1 and 2, which is 0.8 m long, will be examined by the drop tests.

![Figure 1. Layout of the D bolt.](image-url)
3 DYNAMIC DROP TESTS

Mixing tests were conducted before the drop tests in order to examine the ability of the D bolt to properly mix the resin. Boreholes are simulated by PVC pipes. The bolt was spun into the tube at a steady advancement rate and constant RPM. Once the samples were cured, the tubes were cut open and the resin was examined to determine the proportion of well mixed resin in the hole. The mixing study shows that the paddles of the 20 mm D-bolt can reliably mix cartridge resin in a 32 mm hole.

For drop tests, the bolt is installed in two sections of steel tubes, Figure 3. Each test consists of dropping a mass of 893 kg, from a height of 1.5 m, onto a plate connected to the lower section of the tubes. Dynamic drop tests were conducted on four D bolt samples. They were all installed in 32 mm diameter holes with a rotation speed of 300 – 350 RPM and an advancement rate of about 10 sec/m. Figure 4 shows the typical curves of the measured impact and plate loads in the course of impact. The test results are summarised as follows:

- Every drop induced a steel stretch of about 0.05 m;
- All bolts failed at the third drop on the smooth section of the bolt;
- The impact peak and average loads were in the range of 20 – 25 tons, that is at the level of the steel’s static ultimate strength;
  - Only a small portion of the impact load was transferred to the plate (1-11 tons), indicating that the anchor closest to the bolt plate provides protection to the threaded portion of the bolt, which is usually generally considered the weakest link;
  - The average energy absorption capacity of the samples is 44 kJ per metre of bolt.

Figure 3. Dynamic test arrangement of the D bolt.

Figure 4. The impact and plate loads recorded during the first drop of sample D-1.
Back-analysis of extreme squeezing conditions in the exploratory adit to the Lyon-Turin base tunnel

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1 DRIVING AND MONITORING THE ADIT

The S.Martin La Porte incline adit, France, is an exploratory and access gallery to the projected base tunnel of the new railway link Lyon-Turin. During construction of this adit, 9m in width, extreme squeezing conditions occurred while crossing the alpine Carboniferous formations, graphitic and schistose in composition. Under 330m only of overburden, the full face excavation produced more than 2m of diametric convergence (Fig. 1), in spite of face reinforcement by means of fibreglass axial elements and heavy radial bolting. The shortening of the lining perimeter induced the complete crushing of the primary support and the consequent necessity of both systematic reshaping of the cavity and additional stabilization measures.

In order to interpret correctly the excavation behaviour, an intensive program of monitoring was implemented, including the measurement of convergences, of the plastic radius around the cavity (by means of rod and wire extensometers, up to 18m in length), of the tunnel face extrusion, and of the state of stress in the concrete final lining (by means of flat jacks and vibrating wire extensometers).

![Convergences en fonction du temps: PM 1260 à 1320](image)

Figure 1. Synthesis of the convergence measurements vs time (days after excavation) in Carboniferous schists, between chainage 1+260 and 1+320.

2 BACK ANALYSIS OF CONVERGENCE

On the basis of such measurements and systematic geological survey of tunnel face, a deepened back-analysis was carried-out by implementing either analytical and numerical methods, as well
as different geotechnical modelling. The analytical formulation of Sulem et al. (1985) for the convergence prediction and the “characteristic lines” method were originally combined to derive the instantaneous and time-depending rock mass properties and simulate the observed behaviour.

The results of such analysis have been further implemented in numerical calculation by FLAC™ code (Itasca Group), in order to take into account rock mass anisotropy (by means of “Ubiquitous Joints”), as well as the progressive decay of geomechanical parameters after excavation. A very good fitting between calculated and measured stress/strain behaviour could be obtained (Figs 2, 3); in spite of an evident time-dependent rock mass behaviour, the reference to simple elasto-plastic laws provided satisfactory results by considering an progressive reduction of the residual geomechanical parameters and a relevant increase of dilatancy.

Figure 2. Anisotropic deformation of the excavation contour in the Carboniferous schists.

Figure 3. Radial displacements of the anisotropic model, by considering ubiquitous joints.

MAIN REFERENCES

SESSION 19  UNDERGROUND MINING

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The Effect Of Stope Inclination And Wall Rock Roughness On Backfill Free Face Stability
Dirige, A. P. E., McNearny, R. L., and Thompson, D. S.
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Influence Of Finger Configuration On Degradation Of Ore Pass Walls
K. Esmaieli
Université Laval, Quebec City, Canada
J. Hadjigeorgiou
University of Toronto, Toronto, Canada

4044
Characterization And Empirical Analysis Of Block Caving Induced Surface Subsidence And Macro Deformations
K. Woo & E. Eberhardt
University of British Columbia, Vancouver, Canada
A. van As
Rio Tinto Technical Services, Brisbane, Australia

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Ground Support Audit at Brunswick Mine – Data Collection and Results Management
A. Turichshev & P. P. Andrieux
Itasca Consulting Canada Inc, Sudbury, Ontario, Canada
P. Mercier & R. Harrisson
Xstrata Zinc – Brunswick Mine, Bathurst, New Brunswick, Canada

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CCSM Stability Graph And Time Evaluation Of Open Stope Stability
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EXTENDED ABSTRACT

Backfilling is conducted for reasons that include ground control, economics, environmental considerations, and the need to prepare working floors in cut-and-fill operations (Archibald and Hassani, 1998). For ground control, the critical roles of backfill are for wall support in blasthole and cut-and-fill stoping, and as sillmats in undercut mining. The stability of the paste fill exposed face during adjacent mining and of sillmats when exposed by undercut mining is a prime concern due to the high costs associated with maintaining stable paste fill structures.

The optimum recovery of ore in variably dipping ore zones of moderate width often require the use of cemented backfill to serve as structural support. With the intention of saving on costs, backfill of low cement content can be used, which are often supported by a sillmat of higher strength. The stability of the low cement backfill face, exposed during adjacent mining, must be carefully studied to provide very effective, safe and economic mining operations. Improper design of these stope support structures may result in fill mass failure resulting in relaxation and failure of the stope walls, with consequent losses of production, ore dilution, and in safety problems.

This paper presents results of a study conducted to assess fill performance during adjacent pillar mining and to provide an understanding of backfill behaviour, the possible failure modes that may occur and the consequences to production, ore dilution and to safety problems, and accurately predict their stability. The design study investigated the effects of various parameters such as stope width and height, orebody geometry and inclination, and wall roughness on the stability of cemented backfill during adjacent pillar mining. In most cases, stability analysis was based on paste fill prepared at 80% pulp density using unclassified tailings mixed with Type 10 Normal Portland cement (NPC) and Type C fly ash (FA), and cured for 28 days, since this corresponds to the cycle time used at the mine. Binder content used were 7% NPC/FA combination for the sillmat and 2.5% NPC/FA combination for the overlying low binder content paste fill.

Analytical and numerical modeling studies were used for the different conditions to assess paste fill performance. Analytical modeling was carried out using limiting equilibrium analysis adapted from a method introduced by Mitchell et al (1982). Numerical modeling was carried out using FLAC3D (Fast Lagrangian Analysis of Continua), a powerful three-dimensional elastic plastic-finite difference code capable of solving a wide range of complex problems in mechanics. Models were prepared to simulate two different stope conditions of a mine: stopes 3 m wide, 15 m long, and 30 m high and stopes 7.5 m wide, 15 m long, and 40 m high. Stope walls were inclined at 90°, 75° and 60°. Smooth and rough rock wall conditions were established for simulating typical boundary modes in analytical and numerical modeling.

While the analytical modeling approach was demonstrated to be useful in providing some approximate parameters for predicting the behaviour of paste fill exposed faces during adjacent mining, it cannot predict the mode or mechanisms of failure. Numerical modeling not only assess the stability behaviour of paste fill free faces, but also is able to provide a better idea of paste fill failure modes and possible failure mechanisms. The depth of failure and the potential for instability for a simulated stope filled with paste fill can be predicted, and may be useful in estimating the mass of material that could possibly fail and the resultant ore dilution levels.

Based on the modeling results, some general recommendations can be made concerning the overlying paste fill binder content required to avoid instability. The analytical and numerical modeling results indicated comparable modeling results in any of the simulated stope dimen-
sions (3 m wide, 15 m long and 30 m high, and 7.5 m wide, 15 m long and 40 m high stopes) with rough stope rock wall conditions in any of the simulated orebody inclinations. However, numerical modeling results do not agree with the analytical modeling results which indicated that, for all the simulated stope sizes and orebody inclination with smooth boundary conditions which were filled with 2.5% overlying paste fill, the factor of safety against failure would be less than 1. Agreement of modeling results was only achieved when the footwall rock conditions were considered rough.

Increasing the binder content of the overlying paste fill to 7% placed in the same stope sizes and with smooth boundary conditions, would increase the stability of the paste fill exposed face compared with the 2.5% paste fill blend. All models with smooth boundary conditions indicated stable conditions in both modeling techniques. With rough wall conditions, all the simulated stope widths would become stable. The paste fill free-standing height analysis also indicated that, when the stope walls are inclined, failure, driven by the fill self-weight, is dependent on the binder content and is reduced by resisting forces developed on the footwall-fill contact and fill failure plane. The case is not the same for vertically inclined orebodies. Failure, which is also driven by the fill self-weight, is dependent on the binder content and is reduced only by the resisting forces developed on the fill failure plane.

Based on the modeling results, some general recommendations can be made concerning the overlying paste fill binder content required to avoid instability. Stable fill free face exposure conditions exhibited by almost all of the modeling results using the two engineering approaches in any of the simulated mining conditions indicated the appropriateness of adding 2.5% binder content at 1.5% T-10 NPC and 1% T-C FA to the paste fill recipe when the orebody is inclined and with rough rock wall conditions. Recipes with higher cement contents would result in unnecessarily high operational costs. For exposure in narrower stopes (3 m wide) with rough rock wall conditions, a lower than 2.5% binder content paste fill recipe may be appropriate.

2 REFERENCES


Material transfer in underground mines often relies on the use of ore pass systems. Fragmented ore is transported from stopes, or production faces, to a tipping point where it is dumped into the ore pass. When an ore pass intersects two or more production levels finger raises are employed to funnel material into the ore pass. In this configuration, material flows into the finger raise and falls into the ore pass at the junction between the ore pass and finger raise. Material is subsequently drawn out from the ore pass using a chute system.

Finger raises are often associated with operational problems. The drop of rock fragments results in high impact loads acting on the walls of an ore pass that can contribute to the degradation of the ore pass system. This can result in enlargement of the area where a finger raise intersects the ore pass, Figure 1.

The present work addresses issues associated with ore pass degradation caused by material impact. A total of 33 ore pass and finger raise configurations were modeled, using the 2D particle flow code, to quantify the influence of finger raise inclination on the resulting impact loads on the ore pass wall. Three different ore pass inclinations ($\alpha = 90^\circ, 80^\circ$ and $70^\circ$) were considered, Figure 2. The inclination of the finger raise ($\beta$) ranged from $30^\circ$ to $80^\circ$, at $5^\circ$ increments. The ore pass inclination ($\alpha$) and finger raise inclination ($\beta$) result in different angles of intersection ($\gamma$).

The physical and mechanical properties of rock fragments simulated in the PFC2D model include: rock size distribution, particle shape, normal and shear stiffness, density, friction coeffi-
cient and coefficient of restitution. The main source of input data for the numerical models is based on material properties collected in several Quebec underground mines.

During the simulations the following parameters were monitored and measured: velocity and kinetic energy of particles in the impact zone, impact duration, average normal and shear impact force and peak impact load on the ore pass wall.

The results demonstrated that particle impact velocity and kinetic energy increase with finger raise inclination. The impact duration decrease with increase of finger inclination. These observations can be used to evaluate different options of finger inclination for any particular ore pass inclination. In order to compare the influence of both ore pass and finger inclination it is necessary to account for the resulting intersection angle. This consideration does not appear to have been taken into account in current design practice. The results of the undertaken analysis however clearly demonstrate that the choice of intersection angle can have significant influence on the resulting impact loads on the ore pass wall and the location and magnitude of damage to the ore pass. The highest impact loads were reported for intersection angles of 140° and 145°.

Figure 2. Ore pass and finger raise configurations used in the numerical modeling.
The increasing move to block cave mining methods (including sublevel and panel caving) to access deeper and lower grade ore deposits as near surface sources become exhausted raises questions as to the extent and magnitudes of caving-induced subsidence as it may affect strain-sensitive infrastructure on surface. Predicting these surface deformations is difficult as they tend to be discontinuous and asymmetric due to large movements around the cave controlled by geologic structures, rock mass heterogeneity and topographic effects. Of further impediment is the sparse number of case histories available, in which actual subsidence measurements are reported to provide guidance and empirical constraints for subsidence calculations.

Given that such data is essential to better understanding discontinuous subsidence and its controls, this paper reports the development of a comprehensive database of available (i.e. public domain) in-situ subsidence information from block cave mining operations from around the world. The objectives of the database are to lay the foundation for more effective empirical and numerical analysis of surface subsidence induced by block cave mining. Based on the database, empirical relationships can be drawn to demonstrate the limitations of analysis techniques that do not explicitly incorporate geological structures and rock mass anisotropy. In addition, preliminary guidelines are provided that contribute to improving the accuracy of subsidence prediction based on correlations between different geological factors influencing subsidence and characteristics of the subsidence patterns measured.

The database compiled in this study is populated by more than ninety cave mining operations throughout the world including both historical mines that have ceased to operate and those still producing. A tabular format adopted for the database is designed to systematically display diverse basic information on a mine including the name, location, and major factors that affect subsidence including topography and structural geology. Topography is critical because the scope of surface deformation induced by block cave mining can be expanded and the shapes are more likely to be irregular due to the terrain's ruggedness. Structural geology is important as faults, foliation, and/or the discontinuity network create paths of weakness that may influence cave propagation and the development of discontinuous surface deformations.

Again, typically symmetry is assumed when predicting mining induced subsidence. While only 5 percent of the surveyed mines report detailed subsidence measurement data, those mines that do report measured surface subsidence profiles confirm that subsidence above block cave mines is largely asymmetric, meaning that the angle of break from the hangingwall and footwall differs in relation to the structural geology. Structural geology is the overriding factor that governs the asymmetry of subsidence.

Due to this limited availability of surface subsidence profiles, the majority of subsidence information is qualitative focusing on large-scale deformations and outlines of collapse structures (i.e. glory holes). Still, the database represents an important undertaking and forms a solid framework for continued data collection. Updating of the database is continuous with ongoing efforts seeking to incorporate in-house data not available in the public domain.
Subsidence above block caving operations can be divided into micro- and macro deformations (Butcher 2005). Micro deformations include those detected as tilting ground, small strains and/or vertical and horizontal displacements detectable through deformation monitoring. Although relatively small compared to macro deformations, they can still be significant in causing differential displacements of several centimeters, which in turn can affect the structural integrity of strain sensitive materials (e.g. concrete). Macro deformations involve those ground movements that are visually detectable such as the opening of tension cracks, development of scarps, fracturing and break back of the surface above and around the cave’s footprint, and breakthrough of the cave itself to form a large crater.

In the database, high resolution satellite images of some 30 mining operations were captured using the Google Earth image database. These satellite images provide a source of information on the development of large-scale deformations (referred to as macro-deformations) particularly, by phase. Satellite imagery technology, including satellite radar interferometry (InSAR), is already showing potential for providing a valuable new source of caving associated subsidence 2-D and 3-D data (Eberhardt et al. 2007).

Based on the above considerations, the database was analyzed to serve the following objectives:

i). To provide relationships correlating geological information to subsidence observations as a means of guiding and constraining future numerical analyses;
ii). To provide a framework on which to base the selection and integration of investigation, monitoring and analytical techniques to be used to analyze the effects of geology in promoting asymmetry and discontinuous caving-induced subsidence; and ultimately;
iii). To develop empirical characterization tools based on the diverse information compiled in the database to aid in the projection of expected subsidence patterns during the planning stages of future new mining projects.

In total, images were found for just over 30 mine sites in the database. The different patterns identified in these images were first grouped according to three visually distinct topographic controls: operations below relatively flat surface topography (40%), operations under mountainous topography (25%), and operations beneath open pit mines (35%). The distinct patterns seen in these groupings confirm that topography, both natural and anthropogenic (i.e. open pit), is a key factor that affects the extent and shape of the subsidence profile.

Comparative analyses of these images were used to develop a preliminary classification system for macro deformations and block caving-induced collapse structures. The results highlight the degree of asymmetry in the subsidence profiles and the inferred influence of topography, orebody type and structural geology in controlling it.

These findings are constrained though by limitations in the data available in the public domain. Of the ninety block caving operations that populate the database, satellite imagery was only found for thirty cases. This limited availability is due to the short history of the Google Image service which became available just a few years ago. Furthermore, some of the released satellite images are out of date and do not necessarily show the current state of the mining operations. Despite these drawbacks, the significance of the databased images lies in the quick and visual information they provide on characteristics of mining-induced macro deformations.

As more data is acquired, refinement of the classification system will continue. One key additional data set that will be incorporated into this study is that from high resolution satellite imagery. Differential interferometric analysis of these images (DInSAR) will allow for the additional characterization of the micro-deformation component of block cave mining-induced subsidence.

Efforts will continue to update this database with additional geological, topographical, operational and surface deformation information, with the next phase of development to focus on including mine data not available in the public domain.
Ground Support Audit at Brunswick Mine – Data Collection and Results Management

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1 INTRODUCTION
In the life of any ageing underground mine comes a moment when the quality of previously-installed ground support needs to be evaluated. At such times, a mine-wide ground support audit may be commissioned. Usually, its purpose is to identify areas where ground support rehabilitation work is required and to what extent, and to categorise these areas based on urgency. Unfortunately, after enormous efforts were spent on data collection there is often no efficient mechanism that allows the mine’s ground control personnel to organise and quantify the gathered information, track rehabilitation work, and report on progress with respect to the initial state.

In the Fall of 2005, Itasca Consulting Canada, Inc. was asked by the Xstrata Zinc Brunswick Mine to assist the operation’s Ground Control Department with a mine-wide ground support audit. This paper presents some aspects of this project, particularly how the information was collected and organized following the audit.

2 INSPECTIONS
The results of the audit were to be in the form of colour codes implemented directly on paper level plans, with concise accompanying notes. The inspection teams were to assign colour categories to the back and both walls everywhere the audit took place. Colour categories were green (ground support is per standard), blue (ground support is below standard but the ground conditions are good), yellow (ground support is below standard and the ground conditions are poor), and red (the ground support system is ineffective).

Three two-member teams carried-out the inspections. The audit covered mine workings on 49 levels and sublevels, with a total linear extent of nearly 60 km. At the end, the work represented a snap shot of the state of the ground support in the mine at the time of the inspections. Importantly, it was not a direct assessment of the risk of ground falls. A green rating, for example, did not guarantee that no ground instability would ever occur in the area, but, rather, that the ground support installed in it was at the time of the inspection as per mine standards.

3 DATA MANAGEMENT
Once the audit was completed, its main product was a stack of mine level plans full of coloured lines and written comments. Leaving collected information in such form significantly limits its future usefulness. Having coloured lines on a paper mine level plan, however, has one significant advantage: it is a very visual and efficient mechanism for data presentation. This was something that had to be preserved when converting the collected data from “analogue” form into a “digital” form. In addition, the presentation of the data had to be dynamic, i.e., able to show conditions at present day, not only conditions at the time of the audit.

The solution was to develop a two-component system for the audit data storage. The first component was a database containing the collected information and tracking changes as rehabilitation work took place. The second component consisted of a series of CAD drawings that showed audited areas using the same system as when the information was collected, i.e., coloured lines on excavations’ backs and walls. The system had to have the ability to convey any changes made in the database into the drawings.
3.1 Data Organisation

The Microsoft Access database software package was chosen to facilitate data storage. The database was designed with two functions in mind: to be a repository for the collected data and to track progress with respect to rehabilitation work.

The two main challenges faced prior to the development of the system were 1) how to organise the collected data such that they conform to a relational database record structure and 2) how to reference the database records to physical locations underground and to CAD drawings. Coming up with a concept of Area helped solve both issues. In a physical sense, an Area is a section of mine workings where classification category for the ground support conditions, as determined by the audit, remains unchanged. To constitute an Area, categories for each of the walls and the back do not have to be the same but only remain constant in a section of an excavation.

3.2 Database Organisation

The design of the database developed for the management of the audit data is relatively simple. The database contained only ten tables, the functions of which being separated into three categories: storage of collected data, tracking of data related to reconditioning underground, and storage of application’s metadata or data about data. Area, which is uniquely defined by its ID number, is used as the governing entity of the database. The existing audit data and the data related to reconditioning work are based on it.

3.3 Reconditioning Layout Tracking

The dynamism of the audit data was accommodated by including functionalities geared towards tracking the reconditioning work issued for inspected underground workings. The main purpose of having the application being capable of tracking reconditioning work was to provide the Ground Control Department with the information on how the results of the 2005 audit were being addressed. The application was designed to provide its users and mine management with up-to-date quantitative information on reconditioning and update information in the CAD system for visual presentation of the progress through reporting and drawing updates.

3.4 Reporting and CAD Updates

Reporting functionality was implemented as part of the system’s development. Its main purpose was to provide a summary of existing ground support conditions at specific timeframes with respect to the original results generated by the audit.

The purpose of updating the CAD system was to have mine plans show updated information related to reconditioning work carried out since the ground support audit. By looking at updated mine plans one could immediately see which areas of the mine had been reconditioned. Updates to the CAD system were initiated directly from inside the database application. Updating changed the colour of reconditioned areas from their original to grey.

4 CONCLUSIONS

Ground support audits are fairly common in ageing underground mines. The one carried out at Brunswick Mine in 2005 was standard, focusing on identifying areas requiring ground support rehabilitation. The unique part about it, however, was the approach adopted for the audit data storage and management. Following an audit, the collected information quite often remains in its original form, as comments scribbled on paper plans, i.e., in the “analogue” so to speak form. In the case of Brunswick Mine, the collected information was digitised, catalogued in a database and linked to the mine’s CAD system. It this form, it became available electronically and, hence, easy to query. Having an ability to relate collected information to ongoing rehabilitation work made the database application more versatile, providing its users with means of assessing how the information collected by the audit was addressed.
CCSM stability graph and time evaluation of open stope stability

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ABSTRACT: The main purpose of this study is to assess footwall and hangingwall stability of Copper Cliff South Mine (CCSM) stope walls using Mathew’s Stability Graph Method. The secondary purpose of this study is to determine whether time influences the footwall and hangingwall stability. This work is a continuation of a previous study at Copper Cliff North Mine. Nineteen stopes are evaluated to test the hypothesis and none of these stopes are located adjacent to major geological features. A specific CCSM Stability Graph is developed using the guidelines of the Mathew’s Stability Method. As well, a Dilution Plot is developed using a revised approach of overbreak dilution in three classes: (1) less than 10% “Stable”, (2) 10 to 20% “Stable With Support” and (3) greater than 20% “Unstable”. This graph is developed to estimate the amount of dilution from walls of open stopes. Geology joint mapping and diamond drill core logs provide the largest source of data. Factors A, B and C are calculated using GDA (Geomechnical Design Analysis) software. Likewise, $\sigma_1$ wall stresses at specific elevations and distances in the stopes are calculated using MAP3D. The numerical modelling reveals that the assumption of Factor A of unity is not correct in most cases. It was noticed, based on intuition, that opening time has an effect on opening stability. This study found a rudimentary correlation based on the data of 18 stopes whereas an improved correlation was found when 6 high dilution stopes were removed from the data set.

1 INTRODUCTION

There is a need to better understand rockmass behaviour with respect to open stope size so operations can maximize mining cycle while maximizing production and all the while controlling dilution.

This study utilizes the Stope Stability Graph (Mathews et al., 1981) approach to evaluate the stope stability of nineteen stope walls at Vale Inco’s Copper Cliff South Mine (CCSM) in Canada. The time component associated with footwall and hangingwall stability is also investigated. Currently, Copper Cliff South Mine produces ore from five orebodies (790, 800, 850, 865 and 880). The ore lies along a north-south striking quartz diorite dyke. The dyke extends approximately 3-kilometres south of the Sudbury Basin.

The following points highlight some unique features of the study:

1.) Previous work was based on traditional Equivalent Linear Overbreak and Sloughage (ELOS) measurement to quantify dilution and assess stability. This new study uses a mark of 10% external dilution (Figure 1) from a stope face to measure stability. There are two types of dilution, internal (planned) and external. Internal dilution is rock that is blasted during mining of the ore. External dilution is rock that is not part of the planned stope boundary, i.e. it is extended beyond the planned.
stope outline, as shown in Figure 1. In this study, $\geq 10\%$ unplanned (i.e. external) dilution is set as the marker to indicate instability of a stope.

2.) Rockmass quality, in particular RQD, was assessed for an area surrounding each of the studied stopes.

3.) Rays, 10 m long, were extended from the walls of the completed Cavity Monitor Survey (CMS) surface, joined with strings in Mine2-4D and these strings are linked to make wireframes for analysis of RQD. In this fashion, the rock mass quality assessment is considered stope specific.

4.) MAP3D numerical modeling was undertaken to assess stope wall stress estimates taking geometry and excavation sequence into account. In this fashion, values of Rock Stress Factor A are obtained.

5.) Factor B (Joint Orientation Factor) and Gravity Adjustment Factor C, are calculated using a software program called GDA (Geomechanical Design Analysis).

6.) Horizontal slicing for overbreak calculations is used in this study.
SESSION 20 INNOVATION IN GROUND SUPPORT AND INSTRUMENTATION III

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Compilation Of Industry Practices For Control Of Hazards Associated With Backfill In Underground Mines - Part I Surface And Plant Operations
Euler De Souza, Jamie Archibald
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Compilation Of Industry Practices For Control Of Hazards Associated With Backfill In Underground Mines - Part II Underground Transport And Stope Placement
Euler De Souza, Jamie Archibald
Queen’s University, Kingston, Canada
Luc Beauchamp
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New Progress In Ground Control Monitoring System In Leeville Underground Mine
Changshou Sun
Newmont Mining Corp.

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Rockweb – An Innovative TSL Technology For Today’s Mining Environments And The Next Generation Of Hard Rock Mine Operations
T. Macpherson
Spray On Plastics Ltd, Rockwood, Ontario, Canada

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Numerical modeling simulations of spray-on liners support potential in highly stressed and rockburst prone rock conditions
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Compilation of industry practices for control of hazards associated with backfill in underground mines - Part I surface and plant operations

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ABSTRACT: The authors conducted a study of backfill practices in underground mines across Canada to assess the current state of backfill usage and to overview potential hazards associated with backfill practice, from preparation to transportation to placement. The primary aim of this study was to cover each component of a typical mine backfill system, and to identify potential incidents leading to the disruption of backfilling operations and any associated corrective actions. This paper, Part I of the study, deals with surface operations consisting of backfill material supply and backfill plant preparation. Part II of the study, which deals with underground operations consisting of backfill transportation and backfill placement, is also presented in these proceedings. A review of safety measures associated with current backfill operations and recommendations for procedures of best practice that should be implemented to reduce fill-related hazards in all mine backfill workplace environments are presented. In general it has been found that system failures normally lead only to minor losses to the operations.

1 INTRODUCTION

The primary aim of this study (Parts I and II) was to study backfill practices in each component of a typical mine backfill system and to identify potential incidents leading to the disruption of backfilling operations and any associated corrective actions to mitigate against disruption or hazard occurrence.

In general, it has been found that, although mine fill types have become progressively more engineered products, the selection of fill components is usually site-specific, and the mix formulations used and cement additions made are still based on experience and various empirical techniques. Although typical backfill plants are well monitored, relatively little engineering data is gathered or known once the backfill enters the mine borehole, travels through the underground distribution system and is placed in stopes.

A distribution of the types of backfill system failure, previously identified by the authors (De Souza, E., Archibald, J.F. and Dirige, A.P. 2003, 2004), is shown in Figure 1. Pipeline and borehole plugging events were the primary failure types followed by exposed fill sloughing and pipeline bursting. Other primary types of failures were found to be associated with events including pipe hammering, bulkhead failure and fill segregation. Such failures have been associated with a multitude of causes including operator error, poor drainage, inadequate bulkhead construction, poor quality control (low cement content or high variability in moisture), inadequate flushing practices, system wear (pipeline and pump), and indirect causes (excessive blast vibrations, stope wall failure, etc.). System failures normally lead only to minor operational losses.
Based on this study, the authors are of the opinion that there is much about backfill technology, specifically, that is not known; a number of improvements in current backfill operations, including production, transportation and placement, are still required to establish safer, more efficient and less costly fill practices. The areas that have been identified through the course of this study indicate that considerable research and industry education remains to be undertaken. Significant change is, however, noted to be underway, particularly in the areas of total tailings and paste backfill development and use.

Figure 1. Backfill system failures.

2 CONCLUSIONS

A study of backfill practices in underground mines has been conducted to assess the current state of backfill usage and to overview potential hazards associated with backfill practice, from preparation to transportation to placement. This paper (Part I) deals with surface operations consisting of backfill material supply and backfill plant preparation. A review of safety measures associated with current backfill operations and recommendations for procedures of best practice that should be implemented to reduce fill-related hazards in all mine backfill workplace environments have been presented. In general it has been found that, by adopting appropriate management practices, system failures normally lead only to minor losses to the operations.

REFERENCES


Compilation of industry practices for control of hazards associated with backfill in underground mines - Part II underground transport and stope placement

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ABSTRACT: The Department of Mining Engineering at Queen’s University conducted a study of backfill practices in underground mines across Canada to assess the current state of backfill usage and to overview potential hazards associated with backfill practice, from preparation to transportation to placement. The primary aim of this study was to cover each component of a typical mine backfill system, and to identify potential incidents leading to the disruption of backfilling operations and any associated corrective actions. This paper, Part II of the study, deals with underground operations consisting of backfill transportation and backfill placement. Part I of the study, which deals with surface operations consisting of backfill material supply and backfill plant preparation, is also presented in these proceedings. A review of safety measures associated with current backfill operations and recommendations for procedures of best practice that should be implemented to reduce fill-related hazards in all mine backfill workplace environments are presented. In general it has been found that system failures normally lead only to minor losses to the operations.

1 INTRODUCTION

The primary aim of this study (parts I and II) was to study backfill practices in each component of a typical mine backfill system and to identify potential incidents leading to the disruption of backfilling operations and any associated corrective actions to mitigate against disruption or hazard occurrence.

In general, it has been found that, although mine fill types have become progressively more engineered products, the selection of fill components is usually site-specific, and the mix formulations used and cement additions made are still based on experience and various empirical techniques. Although typical backfill plants are well monitored, relatively little engineering data is gathered or known once the backfill enters the mine borehole, travels through the underground distribution system and is placed in stopes.
2 CONCLUSIONS

A study of backfill practices in underground mines has been conducted to assess the current state of backfill usage and to overview potential hazards associated with backfill practice, from preparation to transportation to placement. This paper (Part II) deals with underground operations consisting of backfill transportation and backfill placement. A review of safety measures associated with current backfill operations and recommendations for procedures of best practice that should be implemented to reduce fill-related hazards in all mine backfill workplace environments have been presented. In general it has been found that, by adopting appropriate management practices, system failures normally lead only to minor losses to the operations.
New progress in ground control monitoring system in Leeville underground mine

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Newmont Mining Corp.

ABSTRACT: The Leeville underground mine has been in operation since 2006 and currently produces 3,300 tons of ore per day with an average gold grade of 0.4 opt. The mine has a large horizontal extent of 2,500 feet by 500 feet and a vertical extent of 20 to 200 feet. High density of joints occur in the hosted rock. Based on the RMR rate and observation on the field, the ground condition of Leeville underground mine is poor to very poor. A heavy ground support is required for the mining operation in Leeville. The long hole stope mining method is currently used in Leeville underground.

Because of the poor ground condition, the establishment of ground control monitoring system is the key of the successful operations in Leeville underground mine. The ground control monitoring system is composed of comprehensive instrument design and installations. Instruments include Multi-Point Borehole eXtensometers (MPBX) and Stretch Measurement to Assess Reinforcement Tension (SMART) cables. The monitoring system covers the mining zones and infrastructure openings. After more than two years applications and measurements, many useful data are collected to use in the stope and infrastructure structure design. The system provides the information making engineer and operation teams to do their job more confident and efficiency. It proved that ground support principle in Leeville works. In mining zones, the changes of ground conditions in critical locations with mining process are completely recorded by the ground control monitoring system. The monitoring helps us to know the change trend of ground conditions as the mining operation continues. The information from monitoring system can help us to optimize the support system and cut the support cost. In stope design, a large stope could be used in the tertiary stope mining, based on the results of ground control monitoring system and a series of numerical simulations. The width of stope can be widened at 30 ft to 40 ft. The large stope can help us improve the productivity and efficiency. It also dramatically could improve the safety in mining tertiary stope. The ground control monitoring system can timely response to any emergent hazard ground conditions, such as blast vibration and earthquake etc.
8. CONCLUSIONS AND REMARKS

The ground control monitoring system successfully collected the data of ground conditions in mining zones and infrastructure areas which can answer and solve the problems mentioned in the previous section. In infrastructure areas, every critical location of large size structure is monitored by the instrument. This monitoring makes engineer and operation teams more confidently and efficiently to do their work. It proved that ground support principle in Leeville works. In mining zones, the changes of ground conditions in critical locations with mining process are completely recorded by the ground control monitoring system. The monitoring helps us to know the change trend of ground conditions as the mining operation continues. The information from monitoring system can help us to optimize the support system and cut the support cost. For instance, using the rule of one third of span for the large structures to determine the length of rock bolts are satisfied in Leeville.

In stope design, a large stope, the width larger than 20 ft, could be used in the tertiary stope mining, based on the results of ground control monitoring system and a series of numerical simulations. The width of stope can be widened at 30 ft to 40 ft. The large stope can help us improve the productivity and efficiency. It also dramatically could improve the safety in mining tertiary stope.

The ground control monitoring system can timely response to any emergent hazard ground conditions, such as blast vibration and earthquake etc..
INTRODUCTION

Spray On Plastics Ltd. is a manufacturer and applicator of hot-spray polymer linings for many industries including finishing, industrial containment, concrete fire protection and mining. In the early nineties, Spray On Plastics, was hired as the contractor to the first North American TSL manufacturer for application of a polyurethane-based, elastomer. For a period of five years, the company served as applicators of the Mineguard™ TSL. Many underground applications were undertaken at various mines throughout Canada, including the Sudbury Neutrino Observatory at INCO’s Creighton Mine.

It became evident in those early days that the traditional polyurethane systems were not going to work. The liquids were sprayed in a fashion similar to paint, coating either side of a crack in the rock, but failing to bind the rocks together. In order to obtain complete coverage, the product had to be applied well above design thickness. Furthermore when applied to the back, the hot liquids would drip from the back, wasting more material and often landing on the applicator. When applied in damp areas the urethanes would react with water causing them to foam and become weak.

TECHNOLOGY

RockWeb is a 3 part polyurea elastomer. Polyureas are stronger than polyurethanes, with better resistance to tearing, heat and chemical degradation. They also are significantly less sensitive to moisture during spraying, allowing them to maintain their properties when applied in damp areas rather than foaming as occurs in polyurethanes.

The two liquid components are pumped to the spray gun where 2 high pressure streams collide inside the mixing chamber and are then passed out through open barrel of the gun. Once the polymer is mixed, the fire retardant is blown into the stream of the reacting polymer to create a flame-reactive composite. This composite has displayed exceptionally low flame spread ratings. When touched by a flame, the surface of the composite expands to nearly 100 times its volume, shielding the underlying composite from heat.

Figure 1: RockWeb sprayed in-booth, 4mm thick across an 80mm gap.

RockWeb possesses a unique, patented ability to be sprayed in a “cobweb” like consistency (Figure 1) allowing the applicator to span fissures in the rock, ensuring no initial defects in the liner, and maintaining design thickness over highly irregular surfaces. This ability, coupled
with the high strength and elasticity of polyurea, allows RockWeb to mitigate rock movement through yielding, while maintaining intimate confinement of the smaller material between key blocks that can not be held by screen.

RockWeb is an RHSF (reactive, high strength, flexible) TSL. (Spearing, 2003) It becomes solid in less than 10 seconds, achieves over 1 MPa tensile strength (UTS) within minutes after application, realizes over 75% of its UTS and bond strength in one hour, and is fully cured in 24 hours. Cure time is only marginally affected by temperature, and there are no solvents or water to evaporate. RockWeb’s UTS is 13 MPa, and it can achieve up to 100% elongation. Average adhesion values in excess of 2 MPa have been recorded and are high relative to other TSL materials, and meet or exceed adhesion levels created between shotcrete and rock. (Ozturk & Tannant, 2004)

3 APPLICATION

RockWeb provides reduction in logistics through the transport of as little as 1/25 of the material quantities required for dry shotcrete. Deployment rates of 300 m² per shift have been achieved, making it possible to spray an average round in as little as 30 minutes. Bolting can be completed immediately after spraying, with no harm resulting to the liner, or the bolting equipment. Since no screen is being installed, a bolting pattern suited to the ground conditions may be used instead of the pattern needed to pin up screen.

RockWeb has been used in a number of Canadian mines as a supplement to shotcrete in order to prevent the shotcrete from cracking and to protect large underground infrastructure from damage by falling shotcrete debris. It has also been used in the rehabilitation of problem areas in both shaft and drift applications. Three completed projects will be discussed.

Health and safety issues associated with the application of polyurethane liners have been addressed and virtually eliminated through engineering controls. Isocyanate levels are monitored for every application, and have been shown to be below acceptable limits. Personal protective equipment has been evaluated and simplified, and the air leaving the worksite can be easily processed by the applicators for complete safety in the mine environment.

REFERENCES


Numerical modeling simulations of spray-on liners support potential in highly stressed and rockburst prone rock conditions

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1 EXTENDED ABSTRACT

This paper provides the preliminary results of numerical modeling performed to complement field assessment of support capabilities of TSLs and conventional spray products for mitigating rockburst or like damage in highly stressed mine environments. Extensive field study had been performed by the Queen’s University Mining Engineering Department and funded by the Workplace Safety and Insurance Board of Ontario (WSIB) to characterize the support capabilities of innovative mining support agents, designated as TSL materials, conventional spray supports, as well as combinations of TSLs and conventional spray supports, for mitigating dynamic failure effects created by simulated rockbursts. The field assessment of support capabilities of TSLs and conventional spray products is novel and constitutes work that is unique in the field of underground excavation support design. In the research, various TSL products, shotcrete, fibrecrete and “superliner” combinations of TSL products with ultra-thin shotcrete or fibrecrete layers (at 5 and 3 cm thicknesses, respectively) have been tested. Support performance was studied using field scale explosive detonation trials to simulate dynamic failure effects that are known to develop during typical rockburst events. Multiple seismic and high speed photographic monitoring techniques were used to provide detailed information concerning rock motion, surface fracturing, ejected fragment motion, and support liner survivability characteristics. Results of this study have demonstrated that TSL’s and variant layer combinations may be as effective as or better than conventional support materials for mitigating rockburst or like damage in highly stressed mine environments. The research on TSL support application has been performed under the sponsorship of the Workplace Safety and Insurance Board of Ontario (WSIB), with the principal goals being reduction in the incidence of underground worker injuries and enhancement of excavation stability or durability. A number of papers on the field study of TSLs were published in various forums and journals (Archibald et al., 1997, 1999, 2000, 2001, 2004, 2005, 2006 and 2007).

In this study, field results have been verified using numerical modeling procedures to better understand the support behaviour of TSLs when subjected to highly stressed mine environments and mining-induced rockbursts, a factor that is essential when designing any rock support system. Numerical modeling was carried out using FLAC® (Fast Lagrangian Analysis of Continua), a powerful three-dimensional elastic plastic-finite difference code, with a three-dimensional dynamic analysis option. A series of numerical modeling assessments were conducted on a half circular TSL-lined tunnel, influenced by anisotropic stresses and simulated mining-induced rockbursts, to investigate the support potential of TSLs under prototype conditions. The effect of a mine rockburst was simulated using the three-dimensional dynamic analysis option of FLAC®. In this work, modeling was performed using a polymer based TSL and shotcrete, a conventional spray support that is commonly used in underground mines. The TSL product is currently commercially available for mine support application. Two numerical modeling simulations were conducted: 1) determination of the rock support capability of TSL’s on a half circular tunnel in a Mohr-Coulomb material within a bi-axial stress field; and (2) assessment of the rock support capability of TSL’s on a half circular tunnel in a Mohr-Coulomb material for preventing rock and support material damage due to mining-induced rockbursts.

The numerical modeling simulations successfully verified, if not complemented, field tests and standard material characterization test results conducted for a polymer based spray-on lining
material, commonly designated as a TSL. The results indicated that the numerical modeling simulations of underground excavations, surface-coated with thin spray-on liners, indicated TSL capabilities for generating significant area support potential against gravity falls of loose rock in backs or sidewalls of excavations. This was substantiated by the stalled deformations at tunnel crowns in numerical modeling simulations of underground excavations coated with TSLs under high stress and mining-induced rockburst conditions.

2 REFERENCES


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McGill University, Montreal, Canada
The influence of temperature on Mode II fracture toughness using the Punch-Through Shear with Confining Pressure experiment

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ABSTRACT: The influence of temperature on Mode II fracture toughness was determined using the Punch-Through Shear with Confining Pressure (PTS/CP) experiment. A total of 30 experiments was carried out and the temperature was varied from 25 to 250°C. Ultrasonic measurements before and after heating provided a basis to estimate the microcrack density development during the heating stages. It is shown that significant thermal microcracking occurs at T > 150°C and that the tested fine-grained granite shows an anisotropy of approximately 15%. Experimental results show that Mode II fracture toughness remains roughly constant up to 150°C and increases slightly by 10% for elevated temperatures. The increase corresponds to the onset of thermal microcracking.

The experimental set-up used in the study is the Punch-Through Shear with Confining Pressure (PTS/CP) experiment. It is suggested that the PTS/CP test utilizes the left over from the ISRM Suggested Method of the Chevron Bend experiment (K_{IC}) to obtain K_{IIC} from the same sample. Into the cylindrical specimen of diameter equal length (typically 50 mm) notches are drilled centered into the end surfaces. The specimen is placed between a bottom support and a load stamp. During testing the inner cylinder is punched down at a constant displacement rate of 0.2 mm/min inducing a shear load between the drilled-in notches (see Fig. 1 C for specimen geometry and schematic loading). Reaching peak load the notches act as a friction free initiation locus for the propagation of a shear-loaded fracture that connects the notches.

A servo-controlled MTS loading frame establishes the load with a maximum load capacity of 4600 kN, and a stiffness of 11 MN/mm. Confining pressures up to 200 MPa can be applied via a pressure vessel, enabling the PTS/CP experiment to apply confining pressure independently of axial load in comparison to others, i.e. Rao et al. (2003), Ayatollahi & Aliha (2007).
A total of 30 specimens was tested at temperatures from 25°C to 250°C in steps of 25°C. No confining pressure was applied to the specimens. Figure 3A displays the results from the testing. From room temperature up to 150°C, $K_{IC}$ is roughly constant at $3.4 \pm 0.6 \text{ MPa}\sqrt{m}$; for $T > 150°C$ $K_{IC}$ increases by approximately 10% to $3.7 \pm 0.2 \text{ MPa}\sqrt{m}$. Radial ultrasonic measurements are conducted on specimens prior to and after heating. The difference in p-wave velocity ($\Delta v_p$) increases significantly for $T > 150°C$. The figure below summarizes the results.

- It has been shown that the amount of thermally induced microcracks increases significantly for $T > 150°C$.
- $K_{IC}$ for the fine-grained granite increases when heated above 150°C.
- At $T > 150°C$ the effect of blunting is larger than the effect of fracture-parallel microcracks.
- The difference in ultrasonic velocities prior and after heating proved to be a good indicator to the development of thermal microcracks.

ACKNOWLEDGMENT

The work was carried out partly in the context of an international research and development project aiming at further development of a coupled thermo-hydro-mechanical fracture mechanics software called fracod2D. The project is sponsored and carried out by CSIRO, Australia, SK Engineering and Geology, Korea, KIGAM, Korea, Fracom Oy., Finland, Leibniz-Institut for Applied Geophysics (LIAG), Germany, and GeoFrames GmbH, Germany. Further support to the work was granted by the European Union, European Fund for Regional Development, program 'Investment to Future', Period 2007-2013. The rock material for this study was kindly provided by KIGAM.

REFERENCES


Granitic rocks usually exhibit strongly anisotropy due to pre-existing microcracks induced by geologic loadings. The understanding of the rock anisotropy in mechanical properties such as tensile strength is critical to quarrying and stabilization of underground structures. In this paper, Brazilian tests are conducted in combination with MTS material testing machine and split Hopkinson pressure bar (SHPB) system to measure both static and dynamic tensile strength of anisotropic Barre granite. Samples are cored and labeled using the three principle anisotropic directions of Barre granite (BG). These directions are also chosen as the loading directions for the Brazilian discs. For dynamic tests, pulse shaping technique is used to achieve dynamic equilibrium in the samples during the test. Finite Element Method is implemented to formulate equations that relate the failure load to the material tensile strength employing a orthotropic model for Barre granite. For samples in the same orientation group, the tensile strength shows clear loading rate sensitivity. The tensile strengths exhibit clear anisotropy under static loading; while less anisotropy under dynamic loading. The tensile strength anisotropy of Barre granite is interpreted as interactions of the pre-existing microcracks.

2 EXPERIMENTAL SETUP

Static measurement is conducted with an MTS hydraulic servo-control testing system and Dynamic test is conducted using a 25 mm SHPB system (Fig.1). For dynamic tests, a pulse shaping technique is used to minimize the loading inertial effect and validate a quasi-static stress analysis. Our BG block is directly taken from quarried stones with clear identification of three splitting planes. P-wave velocities are measured along three orthogonal axis of the block, labeled as X, Y and Z axes with respect to slow (3.57 km/s), intermediate (4.00 km/s) and high P-wave velocity (4.75 km/s) respectively (Fig. 2). The rule of nomenclature for the Brazilian disc groups is also shown schematically in Fig. 2, with the first index represents the direction normal to the fracture plane and the second index indicates the propagation direction of the crack.
3 RESULTS AND DISCUSSION

A Photron Fastcam SA1 high speed camera is utilized to monitor the fracture processes of the dynamic Brazilian test. The disc specimen is split completely into two identical fragments (Fig. 3). Several secondary cracks are visible near the loading ends in the last frame. Since those secondary cracks emerge after the initiation and complete propagation of the primary crack, they have no influence on the tensile strength measurement.

All the testing results are shown in Fig. 4. The measured static fracture toughness exhibits very strong anisotropy. The plane normal to X axis remains the weakest plane to split while Z axis is the toughest. The highest toughness value is almost twice of the smallest one. Dynamically, the tensile strength increases with the increase of the loading rate for each group. Compared to the static one, the dynamic anisotropic ratio is much lower. The maximum anisotropic ratio of tensile strength drops drastically from the static value of 1.72 to the dynamic value of 1.13 with a loading rate of 1800 GPa/s.

BG obviously exhibits stronger anisotropy under static loading, while relatively lower anisotropy during dynamic loading. This tensile strength anisotropy is mainly attributed to the distribution and orientation of micro-crack sets. The YZ plane, XZ plane and XY plane corresponds to the quarryman’s description of “rift plane”, “grain plane” and “hard-way plane" respectively. This explains that in our static tensile strength measurements, the minimum tensile strength is obtained from sample split in the rift plane YZ (normal to X axis); while the maximum are obtained from sample with a hard-way splitting plane XY (normal to Z axis). However, the loading in dynamic case is transferred in the sample quickly. It takes time for the unloading information to propagate from the critical crack to its neighboring microcracks. Only part of the microcracks will contribute in the response. Thus, the effects of anisotropy on the tensile strength of Barre granite are overshadowed by the loading rate effects of tensile strength.

4 CONCLUSION

In this paper, we systematically measured the tensile strength of the anisotropic Barre granite with Brazilian tests statically using a MTS hydraulic servo-control testing and dynamically using a split Hopkinson pressure bar system. The disc samples are cored and loaded along three predetermined material symmetrical planes. In the dynamic test, with proper pulse shaping, dynamic far-field force balance is achieved and quasi-static analysis is thus valid for deducing the tensile strength from the SHPB measurements. Rate dependence of the tensile strength of Barre granite is observed. The Barre granite exhibits strong tensile strength anisotropy under static MTS loading, while relative lower anisotropy during dynamic loading. Under very high loading rates such as shock wave loading, it is anticipated that the tensile strength anisotropy can be ignored. The reason for the tensile strength anisotropy is interpreted with the dominant microcracks orientation in the Barre granite.
Rate Dependence of Flexural Tensile Strength of Laurentian Granite

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1 INSTRUCTION

Due to difficulties associated with gripping in direct tension, indirect methods are commonly used to measure rock tensile strength. In this work, we adopt an innovative indirect tensile test method: Semi-Circular Bend (SCB) to measure the tensile/flexural strength of Laurentian granite (LG). The static tests are carried out with a servo-controlled material testing machine and the dynamic tests are conducted with a Split Hopkinson Pressure Bar (SHPB) system. Pulse shaping technique is adopted to achieve dynamic force equilibrium in SHPB to facilitate data regression. Finite element method is used to relate the failure load to the tensile strength. Strong rate dependence of the tensile strength is observed. The flexural strength is higher than the tensile strength measured using Brazilian Disc (BD). This difference is rationalized using the non-local theory. To further validate the method, a coupled Finite element/Discrete element method (Fem/Dem) in conjunction with the smeared crack model is utilized to simulate the fracture process during dynamic SCB tests. The resulting fracture patterns are qualitatively consistent with those from recovered samples. Both the experimental and numerical methods have validated the results and this new method is readily to be used for other rocks and other brittle materials.

2 EXPERIMENTAL SETUP

Static measurement is conducted with an MTS hydraulic servo-control testing system and Dynamic test is conducted using a 25 mm SHPB system (Fig.1) and also schematically shown in Fig.2). The impact of a striker bar on the free end of the incident bar induces a longitudinal compressive wave propagating in both directions, upon interacting with the sample and gets reflected back and transmitted. With strain gauges mounted on in incident and transmitted bar respectively. These three stress waves can be measured. The dynamic forces history on both ends of the sample P1 and P2 thus can be calculated. For dynamic tests, a pulse shaping technique is used to minimize the loading inertial effect and validate a quasi-static stress analysis. During tests, the striker impacts the pulse shapers before the incident bar, thus generating a non-dispersive ramp pulse propagating into the incident bar and thus facilitating the dynamic force balance for the SCB specimen.
3 RESULTS

To visualize the rock dynamic fracture process of SCB specimen, a combined FEM/DEM method (Y2D code) is utilized to qualitatively simulate our experiments. Fig. 3 illustrates the macroscopic crack initiation from the centre of the sample’s diameter, where the generated tensile stress is the maximum. The fracture pattern of the recovered SCB sample for this test is shown in Fig. 3f. Good agreement has been reached between the simulated fracture pattern and experimental observation. This shows that the primary failure of the SCB test is tensile and the failure indeed started from the failure spot O, where the tensile stress is the largest.

Both the static and dynamic results are illustrated in Fig. 4, accompanied by the tensile strength measured from dynamic BD test for comparison purpose. Overall, the measured strengths of LG exhibit a linear increase with the loading rates. The flexural strength measured with the SCB method is higher than the tensile strength measured with BD method for a given loading rate. A non-local approach is utilized here to reconcile the discrepancy of measured dynamic results from SCB and BD tests. Since the dynamic equilibrium is ensured for all SCB tests, the non-local approach should work for our dynamic tests. This theory states that the material fails when the local stress averaged over a distance $\delta$ along the prospective fracture path reaches the tensile strength $\sigma_f$. With non-local theory, the relationship between flexural tensile strength $\sigma_f$ and the resulting tensile strength $\sigma_f$ is determined as $\sigma_f/\sigma_f = 1.8410$. Employing non-local failure theory, the actual tensile strength can be deduced from the measured static or dynamic maximum flexural strength $\sigma_f$ at failure. The results are also included in Fig. 4. Overall, the corrected tensile strengths from flexural strength agree well with those measured from BD tests.

4 CONCLUSION

We measured flexural tensile strength of LG with a newly proposed semi-circular bend method statically using a MTS hydraulic servo-control testing and dynamically using a split Hopkinson pressure bar system. A numerical code (Y2D) using FEM/DEM coupled method is utilized to visualize the fracture process of dynamic SCB test and the results validate the tensile nature of the failure. With proper pulse shaping, dynamic far-field force balance is achieved and quasi-static analysis is thus valid for deducing the flexural tensile strength from the SHPB measurements. The flexural tensile strength of LG exhibit strong rate dependence, an almost linear increase with measured loading rates ranging from ~0 GPa/s to ~ 3000 GPa/s. The flexural tensile strength measured from SCB test has a higher value than that the tensile strength measured using BD method under similar loading rates. We rationalize this discrepancy using a non-local failure theory.
A simple method to estimate tensile strength and Hoek-Brown strength parameter $m_i$ of brittle rocks

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1 INTRODUCTION

Design of engineering structures in and on rocks requires many mechanical properties of the rock mass such as tensile strength ($\sigma_t$) and compressive strength ($\sigma_c$). Direct tensile strength tests of rocks are not routinely conducted because of the difficulty in specimen preparation. When tensile strength test data are not available, the general approach to estimate rock tensile strength makes use of the relationship of $\sigma_t / \sigma_c = 1/10$. This is a very rough estimate and it cannot be generalized for all rock types.

The Hoek-Brown failure criterion (Hoek et al. 2002) is widely used to describe the strength of rocks and jointed rock masses. To use this failure criterion, a few parameters such as $\sigma_c$, $m_i$, and GSI, are required. $\sigma_c$ can be obtained from uniaxial compression tests. The $m_i$ values are believed to vary with rock type, and it is recommended that these values be determined from a series of triaxial tests. Quite often, triaxial tests are not routinely conducted for most projects and engineers are forced to determine the $m_i$ values empirically. Hoek (2007) provided some $m_i$ values for reference. However, the suggestion should be used with caution as $m_i$ does not depend on rock only. In this study, a simple method to estimate $\sigma_t$ and $m_i$ using uniaxial compression test data is presented.

2 ESTIMATE OF TENSILE STRENGTH AND HOEK-BROWN PARAMETER $m_i$ OF ROCKS FROM UNIAXIAL COMPRESSION TESTS

2.1 Tensile strength of strong brittle rocks

Crack initiation in tensile loading often means tensile fracture is imminent. On the other hand, crack initiation in compression is only linked to a lower stress as compared to the peak compressive strength. When both crack initiation and peak strength are considered the same (e.g. Griffith’s approach), it leads to a $\sigma_c / |\sigma_t|$ ratio of 8. However, additional loading is required to bring the stress level from the crack initiation stress $\sigma_{ci}$ to $\sigma_c$ in uniaxial compression. The ratio of $\sigma_c / \sigma_{ci}$ indirectly reflects the gap between crack initiation and peak strength in compression. Hence, it is proposed to estimate the tensile strength of strong brittle rocks from the crack initiation stress ($\sigma_{ci}$) and the uniaxial compressive strength ($\sigma_c$) using the strength ratio from the following equation

$$R = R_{ci} \frac{\sigma_c}{\sigma_{ci}} = 8 \frac{\sigma_c}{\sigma_{ci}}$$

(1)

It follows that the tensile strength of the rock can be obtained from

$$|\sigma_t| = \frac{\sigma_c}{R} = \frac{\sigma_{ci}}{8}$$

(2)

2.2 Hoek-Brown parameter $m_i$ of strong brittle rocks

For strong brittle rocks, the Hoek-Brown parameter $m_i$ can be approximated by $R$, i.e.,

$$m_i = R = 8 \frac{\sigma_c}{\sigma_{ci}}$$

(3)
When a rock is subjected to a high confining stress, microcrack initiation from most pre-existing defects in the rock will follow the stress state defined by 3D Griffith ellipsoidal cracks. It is observed that the estimated $m_i$ from Eq. (3) provides a $m_i$ value for the tension to low confinement zones (e.g. $\sigma_3 < 5$ MPa, for rocks near the underground excavation boundary, which are of a concern for excavation stability), since all cracks are assumed to follow 2D Griffith’s stress state. Under a high confining stress state, ellipsoidal microcrack initiation and propagation will dominate and hence Murrell’s strength ratio ($R_M$) = 12, instead of Griffith’s strength ratio ($R_G$) = 8, is considered more appropriate for the estimation of $m_i$. It is proposed to estimate the $m_i$ value for the high confinement zone from

$$m_i (m_i) \approx R_M \frac{\sigma_c}{\sigma_{ci}} = 12 \frac{\sigma_c}{\sigma_{ci}}$$

(4)

2.3 Application example

As an example, the method is applied to the Lac du Bonnet granite from the Mine-by tunnel in Canada. The mechanical properties of the medium to coarse grained pink granite are: elastic modulus – 65±5 GPa, Poisson’s ratio – 0.25±0.05, uniaxial compressive strength – 213±20 MPa, tensile strength (obtained from Brazilian test) – 8.9±1 MPa, crack initiation stress 70 to 80 MPa. According to Eq. (1), the strength ratio $R$ is found to vary between 21 and 24. For comparison, the average strength ratio from the test data is $R = 213 / 8.9 \approx 24$. If only the uniaxial compression test is conducted, we can estimate the tensile strength as $\sigma_t / \sigma_c = (70 \text{ to } 80) / 8 = 8.75$ to 10 MPa with an average of 9.4 MPa, the Hoek-Brown strength parameter $m_i$ as 34 and 23 for the high confinement and low confinement zones, respectively. The $m_i$ value for the high compression zone is very close to the triaxial compression test data listed in Martin and Stimpson (1994). The $m_i$ values from the test data for the medium to coarse grained granite are 30.8 from the shallow ground and 34.8 from the 240 m level. More examples can be found in the full paper in the conference proceeding CD.

3 CONCLUSIONS

A practical simple method is proposed to estimate the tensile strength ($\sigma_t$) of strong brittle rocks from the strength ratio of $R = \sigma_c / \sigma_t = 8 \sigma_c / \sigma_{ci}$, where $\sigma_c$ is the uniaxial compressive strength and $\sigma_{ci}$ is the crack initiation stress in an uniaxial compression test. In addition, it is suggested to estimate the Hoek-Brown strength parameter $m_i$ by $m_i = 12 \sigma_c / \sigma_{ci}$ for strong, brittle rocks in high confinement zone and $m_i = 8 \sigma_c / \sigma_{ci}$ for low confinement to tension zones. It is found that the predicted tensile strengths and Hoek-Brown strength parameter $m_i$ using this method are in good agreement with test data.

Rock type cannot be used directly to define the strength ratio $R$ and the $m_i$ value. Data inferred from the databases can only be used when there are no test data available at the initial design stage. Whenever possible, laboratory tests should be conducted to determine the tensile strength and the $m_i$ values more accurately. The method suggested in this paper provides an easy and yet accurate way to determine these two important parameters for brittle rocks from conventional uniaxial compression tests.

4 REFERENCES

ABSTRACT: Permeability is a key geomaterial property that is important to many problems in geoenvironmental and geotechnical engineering. Problems in hydrogeology, including groundwater extraction, contaminant transport, aquifer contamination by sediments, etc., are controlled by the permeability characteristics of the porous materials involved. In hydrological geosciences in particular, the property of permeability is considered to be sessile, whereas in geomechanics factors such as the stress state acting on the geomaterial can have a significant influence on the permeability. Relatively new areas in environmental geomechanics, including carbon dioxide sequestration, nuclear waste disposal and deep injection of hazardous wastes require accurate estimates of permeability of the geologic media encountered. The estimation of permeability of geomaterials becomes even more critical when the information is used in computational models of flow and transport processes to predict the long-term performance of strategies for geoenvironmental remediation.

Permeability of rocks is scale-dependent. These can range from crustal scales of 0.5 km to 5.0 km to borehole scales ranging from 30 m to 300 m to laboratory scales of 5 cm to 15 cm. The experimental work associated with this research focuses on the measurement of the permeability of a relatively intact block of Indiana Limestone; in disposal endeavours, such as geologic sequestration of carbon dioxide, the intact units of rock offer a substantial pore volume for sequestration and access to this volume is controlled by the intact permeability of the rock. Laboratory procedures offer the most convenient techniques for the experimental determination of permeability of natural geologic materials that are void of dominant defects such as fractures, fissures and other visually observable defects including solution channels. Several laboratory procedures have been developed, but these methodologies invariably involve the application of steady state flow, usually in the longitudinal direction of the sample. The main advantage of a steady state test in hydraulic property measurement is that the interpretation of the results is relatively straightforward, dependent only on the hydraulic boundary conditions of the test and the dimensions of the sample. In materials with low permeability an inordinate amount of time is required to attain steady state flow conditions, and recourse is usually made to transient methods that are referred to as hydraulic pulse tests. Pulse tests are rapid tests but require a significantly larger number of extraneous parameters including the compressibility of the fabric of the geologic medium to interpret the test data.

This paper presents the results of a research investigation leading to the development of an innovative technique for the measurement of the surface permeability of Indiana Limestone. The tests are conducted on a cuboidal block of the limestone with a flat surface that can be sealed to create a circular aperture through which water influx can take place. Computational results for Darcy flow are used to interpret the results of the experiments. The water entry area can be moved over the surface of the block to investigate, quite conveniently, the distribution of permeability across the limestone block sample. The paper is a departure from the use of rock cores for determining the permeability characteristics of geologic materials, in that the permeability tests are conducted on a substantially larger intact sample of the geomaterial. The test involves the application of a constant flow rate to an open region of the test specimen, which is provided with an adequate seal to enable the development of steady flow conditions in the flow domain. The test configuration involves the development of a perfect seal over an annular region in contact with the surface of the cuboidal specimen and the application of a constant potential within the internal
circular region, to attain steady state flow conditions. The annular region was sealed against the surface of the rock by the application of a normal load to a confined rubber gasket. The experimental configuration consisted of a reaction frame for the sealing loads, a hydraulic cylinder with a manual pump to apply the sealing load, a load cell to measure the applied load, liquid chromatographic pump, a designed permeameter to allow sealing of the annular patch, a water supply and a water reservoir to maintain the cuboidal block under water. The cross-head of the reaction frame can be moved to accommodate specific test locations on the surface of the sample; this is necessary in order to quantify the spatial distribution of the permeability and is important for the computational analysis of the test. The hand pump allows for the accurate, constant load required to generate sealing, to be maintained constant during a test. Filtered de-aired water was used as the permeating fluid to ensure uniformity in the test procedure. The pressure induced during the attainment of a steady flow rate was monitored. The results of nearly 45 steady state permeability tests conducted on one face of the block of Indiana Limestone are presented. Tests were conducted at nine locations each on two faces and the test results interpreted using the computational approach described in the ensuing section. At each location the maximum and minimum pressures reached during attainment of a steady state flow were recorded. The temperature of the percolating fluid was also recorded for each test since this information is required for the correct interpretation of the viscosity.

Since the steady state Darcy flow problem associated with the surface permeability testing deals with a three-dimensional problem, the interpretation of the experimental data is more conveniently accomplished using a computational model of the steady state flow problem. The finite element code COMSOL Multiphysics was used. The computational code COMSOL solves the steady flow problem by employing a Galerkin finite element scheme; meshes of 165 634, 102 237 and 101 463 elements respectively, were employed A robust solver SPOOLES, provided in COMSOL, was used to generate the steady state solution. The computed results for permeability varied over the face of the Limestone block but showed very good repeatability at a specific location.

Once the data was collected it was possible to infer that the average surface permeability of the first face of the limestone block was $29.4 \times 10^{-15} \text{ m}^2$ with a standard deviation of $11.21 \times 10^{-15} \text{ m}^2$. These are acceptable values and are in the range of the values obtained previously ($16 \times 10^{-15} \text{ m}^2$) for Indiana limestone tested using axial flow tests. The second face showed the same degree of spatial variability that was shown on the first face. The test data from the series of tests on one face of the Indiana limestone block were compiled to visually illustrate the spatial representation of the permeability over the surface area. This was done using the contour mapping algorithm in MATLAB.

Results from experiments conducted on a cuboidal Indiana limestone sample using the surface permeability technique in a laboratory environment show good correlation with those obtained using traditional cored cylindrical samples. The average values for face one and two, $29.4 \times 10^{-15} \text{ m}^2$ to $44.3 \times 10^{-15} \text{ m}^2$ respectively, obtained from the surface permeability tests are of a comparable range to those found previously using core samples of the same material. When performing the surface permeability test the confirmatory experiments (sealing test and Reynolds test) must be carried out to ensure the applicability of the test methodology and the underlying assumptions of the theoretical developments. The non-invasive technique proposed in this paper may prove beneficial in situations where coring of the sample is not an option. These tests might take longer to perform and be more costly, but for situations where the overall permeability is needed and the samples cannot be altered by drilling or coring, the surface permeability technique is considered to be a valuable approach. This research investigation will be extended to include the surface permeability measurement of the four remaining sides and an interpretation of the permeability of the cuboidal block in terms of global statistical estimates.

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