Management of Squeezing Ground Conditions in Hard Rock Mines

by

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Abstract

Squeezing ground conditions can result in large scale deformation and instability of underground excavations in rock. In hard rock mines, squeezing ground conditions are often associated with the presence of dominant foliation and high stress with resulting deformation often exceeding the capacity of installed ground support. This can result in failure of ground support making the excavations non-operational.

This thesis is a contribution to improved understanding of structurally defined squeezing mechanisms observed in underground hard rock mines. A comprehensive field campaign was undertaken in two mines where squeezing was a major ground control issue. This has allowed the development of a database comprised of 147 squeezing case studies. Reported information included drift configuration and orientation, deformation monitoring, rock mass characterization and the performance of ground support. The data were reviewed to identify the failure mechanisms and to quantify the influence of ground conditions, excavation orientation, and the performance of different support strategies.

The distinct element method (DEM) was used to successfully reproduce the buckling mechanism resulting in squeezing and large deformations observed in mining drifts in the database. The developed methodology was implemented in 3D models and explicitly considered the role of foliation. Subsequently, a comprehensive strategy for explicitly modeling rock reinforcement using the DEM was developed and implemented in a series of 3D numerical models. The models were calibrated based on field testing of
reinforcement and observations at the LaRonde Mine. They were used to investigate the impact of different reinforcement strategies and the time of installation of the support. The numerical results were in agreement with the field observations and demonstrated the practical implications of using yielding reinforcement elements. This was supported by field data where the use of yielding bolts reduced the drift convergence and rehabilitation.

An analysis and interpretation of the field data and the numerical modelling investigations were used to provide tools for the assessment and mitigation of the anticipated deformation in squeezing ground. The developed strategies can be used to select optimal reinforcement options. The developed tools and procedures can be successfully used to manage large structurally controlled deformations in underground hard rock mines.
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<tr>
<td>$\delta_{total}$</td>
<td>Total convergence</td>
</tr>
<tr>
<td>$\varepsilon_{total}$</td>
<td>Total strain</td>
</tr>
<tr>
<td>$F_s$</td>
<td>Shear force in the grout for global reinforcement elements</td>
</tr>
<tr>
<td>$J_a$</td>
<td>Joint alteration number</td>
</tr>
<tr>
<td>$J_r$</td>
<td>Joint roughness number</td>
</tr>
<tr>
<td>$K_a$</td>
<td>Axial stiffness of local reinforcement elements</td>
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<td>$K_{bond}$</td>
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</tr>
<tr>
<td>$K_s$</td>
<td>Shear stiffness of local reinforcement elements</td>
</tr>
<tr>
<td>$p_o$</td>
<td>Overburden stress</td>
</tr>
<tr>
<td>$u_c$</td>
<td>Axial displacement of the bolt for global reinforcement elements</td>
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<tr>
<td>$u_m$</td>
<td>Axial displacement of the medium (rock) for global reinforcement elements</td>
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<td>$\Delta F^t$</td>
<td>Incremental axial force of global reinforcement elements</td>
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<tr>
<td>$\Delta u^t$</td>
<td>Displacements at the structural nodes of each bolt element for global reinforcement</td>
</tr>
<tr>
<td>$\sigma_{ci}$</td>
<td>Uniaxial compressive strength of the intact rock</td>
</tr>
<tr>
<td>$\sigma_{cm}$</td>
<td>Uniaxial compressive strength of the rock mass</td>
</tr>
<tr>
<td>$\sigma_t$</td>
<td>Tensile strength</td>
</tr>
<tr>
<td>BEM</td>
<td>Boundary element method</td>
</tr>
<tr>
<td>CMS</td>
<td>Cavity monitoring system</td>
</tr>
<tr>
<td>DBS</td>
<td>Depth below surface</td>
</tr>
<tr>
<td>DEM</td>
<td>Distinct element method</td>
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<tr>
<td>E</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>FDM</td>
<td>Finite difference method</td>
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<td>FEM</td>
<td>Finite element method</td>
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<tr>
<td>FRS</td>
<td>Friction rock stabilizers</td>
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<tr>
<td>GRC</td>
<td>Ground reaction curve</td>
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<td>H</td>
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LDP  Longitudinal displacement profile
MPBX  Multi-Point Borehole Extensometer
N    Rock mass number
q    Constant which depends upon the end conditions of the rock slab
Q    Tunnelling index
r    Reduction factor
SRF  Stress reduction factor
t    Thickness of rock slab
v    Poisson's ratio
θ    Angle between the active length and the reinforcement for local reinforcement elements
σ₁  Maximum principal stress
σ₂  Intermediate principal stress
σ₃  Minimum principal stress
σ₇  Critical buckling load for a rock slab
A    Cross sectional area of global reinforcement elements
L    The element length of the global reinforcement
a    Joint amplitude
l    Joint length
s    Surveyed drift width
ΔFₐ  Incremental change in axial force of local reinforcement elements
ΔFₛ  Incremental change in shear force of local reinforcement elements
ΔUₐ  Incremental change in axial displacement of local reinforcement elements
ΔUₛ  Incremental change in shear displacement of local reinforcement elements
α    Displacing angle of bolts
δ    Highest recorded wall convergence
ε    Highest wall strain
Angle of interception, defined as the angle between the normal to the foliation planes and the normal to the sidewall
Chapter 1
Introduction

1 Introduction

1.1 Background

Squeezing ground conditions are encountered in both tunnelling and mining and often result in large scale deformation and instability. They are caused by a variety of failure mechanisms but are typically associated with a reduction of the cross sectional area and a combination of induced stresses and relatively weak material properties (Potvin & Hadjigeorgiou, 2008). Controlling the very large deformations is challenging and has important cost implications.

In a mining context, squeezing conditions can result in severe economic and safety repercussions in deep and high stress excavations. The Task Force on Squeezing Rock in Australian and Canadian mines, Potvin and Hadjigeorgiou (2008) reported that "in a mining environment squeezing ground conditions are encountered when the total displacement of an excavation, the drive closure, will reach at least tens of centimetres within the life expectancy of a supported drive. In general, mine drives are designed to be in operation up to 18 months to two years. In squeezing ground the resulting loads will be greater than the capacity of a “stiff” support system. This results in significant failure of ground support and necessitates extensive rehabilitation work.” Figure 1.1 shows an example of extreme squeezing conditions in mining during rehabilitation. At that stage there was significant floor heave and a buckling mechanism in the sidewall driven by the presence of foliation.

Figure 1.1: Example of extreme squeezing in a mining drift during the stage of rehabilitation.
In civil applications, the implications in ground support failure are similar with mining. Figure 1.2 shows an example of severe squeezing in tunnelling. Large deformation encountered in a number of locations where inadequate support had been installed (Hoek & Marinos, 2009). The tunnel had to be re-mined and rehabilitated at several locations.

![Image of a civil engineering tunnel](image)

**Figure 1.2:** Re-mining and rehabilitation of a civil engineering tunnel due to severe squeezing (Hoek & Marinos, 2009).

The analytical, numerical or empirical methods used to study squeezing conditions in civil engineering tunnelling often assume that the rock mass deforms and fails in a uniform way. In a hard rock mining context, squeezing is often controlled by structures and high stress (Potvin & Hadjigeorgiou, 2008). This often results in a buckling mechanism. The study of these conditions should take into account the anisotropy of the rock mass.

Another important variation in tunnelling, as opposed to mining are the available options for reinforcement and support. As civil engineering tunnels are permanent infrastructures used by the public, the choice and economics of support are quite different than mining. Civil engineering applications have less tolerance for deformation. At the same time they have the option to invest in support that can be more effective such as steel arches or ductile tunnel linings with yielding elements, although they are more expensive. On the other hand, the lifetime of mining drives is often limited to 18 months to two years. In addition, there is higher tolerance in deformation in a mining environment than in civil engineering tunnelling due to operational purposes. Mining drives often remain operational even after significant deformation has occurred provided they can still allow the movement of mining equipment.
For the purposes of this thesis, the following definition of squeezing has been chosen: Structurally defined deformations driven by stress and resulting in a buckling failure mechanism including detachment and rotation of rock slabs. The magnitude of these deformations in a mining context can exceed tens of centimeters or 10% wall-to-wall strain within the life expectancy of a supported drive. The associated loads are often greater than the capacity of conventional support systems resulting in significant failure of ground support and extensive rehabilitation work.

Hard rock mines in this thesis refer to metalliferous mines. The strength of the rock can be low or high. For example, large deformation can be observed in hard rock mines with relatively high uniaxial compressive strength ($\sigma_c$), driven by high in-situ stresses (Mercier-Langevin & Turcotte, 2007a). Martin et al. (1999) suggested that instabilities due to high stresses can occur when the ratio of the maximum far-field stress $\sigma_1$ to the unconfined compressive strength ($\sigma_c$) is higher than 0.4. This thesis examines squeezing conditions in hard rock mines in both low and high strength rocks.

1.2 Objectives

The thesis focuses on how large, structurally controlled deformations, in underground hard rock mines, can be controlled avoiding frequent rehabilitation and high support cost. The primary objectives of this thesis are:

- To improve our understanding of the structurally defined squeezing mechanisms observed in hard rock mines.
- To propose ways to mitigate the resulting deformations and investigate the impact of different support strategies.

The secondary objectives of this thesis are:

- To develop field based empirical tools to manage structurally controlled deformations.
- To develop 3D stress analysis procedures to capture the phenomenon and tools to investigate the effect of reinforcement.

1.3 Methodology

A literature review was undertaken focusing on the reported squeezing mechanisms in hard rock mining, the parameters controlling the deformation and the support strategies used to control the deformation. The numerical methods used for modelling the large deformations in hard rock were also reviewed.

Subsequently, an extensive data collection campaign took place in mines facing squeezing ground conditions. The LaRonde and Lapa mines were selected, located in northwest Quebec. Both mines face
some of the highest deformations reported in hard rock mines that involve rehabilitation and provide a wide spectrum of squeezing ground conditions. The data collection included the formation of a series of case studies recording the drift convergence, the angle of interception ($\psi$), the dip angle of the foliation, the condition of the foliation planes, intact rock strength data, the observed damage, the development date, the stress effect due to mining activity, the support system used, the additional support installed, the presence of water and any rehabilitation or purging.

A detailed investigation was performed on the ground control records including the above parameters as well as any monitoring and rock mass quality data available. The deformation was recorded using cavity monitoring systems (CMS) surveys. The measurements were supplemented by laser measurements whenever CMS surveys were not possible. Borehole extensometers were used to record the extent of deformation in the walls for various case studies. The ground support strategies employed under squeezing conditions were recorded.

The combined interpretation of the field data allowed investigating of the impact of rock mass characteristics, mining and time on the deformation. The investigation was focused on the presence of structural features (foliation), the angle of interception between the foliation and the drift, the foliation spacing, the condition of the foliation planes, the stress and the intact rock strength, the support system used and the progress of deformation over time. The results contributed towards the validation and extension of the empirical “Hard Rock Squeezing Index” (Mercier-Langevin & Hadjigeorgiou, 2011).

A methodology was developed for the simulation of the squeezing mechanism observed underground using the 3D distinct element method (DEM). The proposed technique considered explicitly the role of fractures within the rock mass and resulted in the reproduction of the buckling mechanism. Progressive reduction of the forces acting at the boundaries of the excavation by a reduction factor through a series of modelling steps, overcame the computational and time constraints on modelling the development of a mining drift. The effect of different ground conditions on the deformation was also examined. The numerical modelling results were in agreement with the observed squeezing mechanism and deformation levels recorded in Canadian and Australian hard rock mines.

The influence of reinforcement in the 3D DEM was subsequently explored, extending the numerical work that captured the squeezing buckling mechanism. A methodology was proposed for capturing the impact of different reinforcement elements under these conditions using the distinct element method. The mechanical behaviour of different rock bolts was examined based on in-situ pull out tests from the LaRonde mine. Structural elements in the 3D DEM were calibrated to simulate the behaviour of different rock bolts in pull conditions as observed in the field. The calibrated elements were introduced to the simulated squeezing
conditions. Different reinforcement scenarios were examined taking into account the sequential installation of reinforcement based on the sequence followed at the mine. The numerical work captured the influence of reinforcement in squeezing conditions as observed and monitored at the LaRonde mine. The impact of other reinforcement strategies was subsequently investigated.

The results from the interpretation of the field data and the numerical modelling using the 3D DEM were used to recommend tools for the design of drifts in structurally controlled deformation in hard rock mines. Tools for the assessment of the anticipated deformation were presented. Proactive ways to mitigate the deformation and effective support strategies were recommended.

1.4 Thesis structure

The thesis consists of 10 chapters. Chapter 1 provides an introduction to the thesis and chapter 10 includes the main contributions and recommendations for future work. An outline of the remaining chapters is presented below:

Chapter 2 provides a description of the problem. It presents the results from the literature review on the reported squeezing mechanisms in underground hard rock mines. A discussion follows on the critical parameters controlling the deformation in hard rock mines. A review of the available methods to assess the squeezing conditions is also provided. The magnitude of deformation reported in several underground hard rock mines is summarized. The rehabilitation measures required to keep the drifts operational under excessive deformation are described.

Chapter 3 includes a description of the rock mass behaviour and the ground conditions at the LaRonde and Lapa Mines. Information on the geology, the mechanical properties of the rock units and the in-situ stresses is provided. The reported conditions driving the squeezing process in the two mines are compared.

Chapter 4 presents the data collection process from the LaRonde and Lapa Mines. The methodology followed to collect the rock mass characterization data is described. The orientation of the drift with respect to the foliation, the dip angle of the foliation, the foliation spacing, and the condition of the foliation planes were recorded in a series of case studies for both mines. Deformation monitoring data was collected using cavity monitoring systems (CMS) surveys and laser measurements. Mechanical borehole extensometers were manufactured to capture the deformation and the extent of the depth of failure in the back and the sidewalls. The support installed in each case study was recorded including the existence of any additional support and the existence of any rehabilitation work.
Chapter 5 presents the interpretation of the field data collected from the two mines. Squeezing was expressed as strain based on the CMS surveys and the laser measurements. It includes a quantitative interpretation of the effect of the orientation of the excavation with respect to the foliation on the degree of squeezing. The foliation spacing data and the condition of the foliation planes are presented as examined in the case studies. The data were used to validate the hard rock squeezing index. The effect of mining induced stresses on the squeezing process was identified based on monitoring data. Finally, the progress of deformation over time was examined.

Chapter 6 addresses the critical failure mechanism observed in foliated rock under stress in underground hard rock mines. It includes a review of the numerical methods for modelling large deformations in hard rock. It presents a methodology for capturing the buckling mechanism using the distinct element method. Progressive reduction of the forces acting at the boundaries of the excavation by a reduction factor through a series of modelling steps overcame the computational and time constraints on modelling the development of a mining drift. The numerical modelling results were in agreement with the observed squeezing mechanism and deformation levels recorded in Canadian hard rock mines. The effect of different ground conditions on the deformation was examined including the foliation spacing, the dip angle of the foliation and the stress conditions.

Chapter 7 describes the support strategy followed to manage the excessive deformation under squeezing conditions at the LaRonde and Lapa mines. The results from the investigation of the impact of different support strategies in squeezing at LaRonde are presented. The evolution and effectiveness of the employed support strategy are discussed based on field observations. The effectiveness of hybrid bolts as part of a secondary support strategy was examined based on borehole extensometer results and the rehabilitation records at LaRonde.

Chapter 8 explores the implication of reinforcement in the 3D DEM extending the numerical work that captured the squeezing buckling mechanism in chapter 6. A methodology is proposed for capturing the effect of different reinforcement elements under these conditions using the DEM. The behaviour of rebars, FRS, hybrid bolts and cable bolts was examined based on in-situ pull out tests from LaRonde. The results were compared with laboratory tests available for each bolt type. A review of the structural elements in 3DEC is presented focusing on their effectiveness in capturing the mechanical behaviour of bolts under squeezing conditions. Structural elements in the 3D DEM were calibrated to simulate the behaviour of different rock bolts in pull conditions as observed in the field. The calibrated elements were introduced to the simulated squeezing conditions, taking into account the sequential installation of reinforcement based on the sequence followed at the mine. The impact of the discrete effect of reinforcement was also taken into account. The modelling results were in agreement with the impact of reinforcement observed and
monitored at LaRonde. The impact of other reinforcement strategies was examined using the developed methodology including the early installation of cable bolts and the use of D-Bolts.

Chapter 9 presents the recommendations for the design of drifts in structurally controlled deformations in hard rock mines based on the interpretation of the field data and the numerical modelling. It includes tools for the identification of structurally controlled deformations and the assessment of the anticipated squeezing level. Proactive measures for mitigating squeezing were recommended based on the field data. The factors that need to be considered for the selection of an appropriate surface support and reinforcement are discussed. Reinforcement strategies that can mitigate large deformations are suggested.
Chapter 2
Squeezing rock in hard rock mines

2 Squeezing rock in hard rock mines

2.1 Introduction

This chapter presents a description of the squeezing problem in hard rock mines. An overview of the literature review on the failure mechanisms in Canadian and Australian hard rock mines is presented. A discussion follows on the critical parameters that can influence the deformation. A review of the methods and classification systems available to assess the potential of squeezing is provided. Deformation measurements from various underground hard rock mines experiencing squeezing ground conditions are summarized. The problems observed in the operation of the mine drives experiencing such phenomena are discussed including the necessity of rehabilitation of the underground openings.

2.2 Problem definition – mining under squeezing conditions

A large number of underground hard rock mines experiences large scale deformations, often referred as squeezing ground conditions. These conditions have major economic and safety implications for the mining industry especially where deeper excavations are developed. A definition of squeezing rock has been provided by the International Society of Rock Mechanics (ISRM): “Squeezing of rock is the time dependent large deformation which occurs around the tunnel and is essentially associated with creep caused by exceeding a limiting shear stress. Deformation may terminate during construction or continue over a long time period”, Barla (1995). The large convergences associated with squeezing often result in the need for non-standard excavation and support methods.

In addition to squeezing, large deformations may also occur underground due to swelling. Terzaghi (1946) distinguished between squeezing and swelling phenomena. In his description of the rock condition he states that “Squeezing rock slowly advances into the tunnel without perceptible volume increase. Prerequisite of squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or of clay minerals with a low swelling capacity.” For the swelling phenomenon he mentioned that “Swelling rock advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks which contain clay minerals such as montmorillonite, with a high swelling capacity”. Squeezing is mechanically treated whereas swelling is a chemical process involved with the exchange of ions between some minerals and water (Aydan et al., 1996).
In a mining environment, squeezing conditions are usually related with a significant failure of ground support, considerable rehabilitation work and high support cost, Potvin and Hadjigeorgiou (2008). Figure 2.1 shows an example of extreme squeezing conditions from the LaRonde Mine. Significant rehabilitation work is necessary to maintain the drift in operation.

![Extreme squeezing conditions at the LaRonde Mine.](image)

**Figure 2.1: Extreme squeezing conditions at the LaRonde Mine.**

### 2.3 Squeezing mechanisms in hard rock mines

Aydan et al. (1993) presented a phenomenological description of three failure mechanisms related to squeezing conditions in tunnels:

- **Complete shear failure:** This involves complete process of shearing of the medium observed in continuous ductile rock masses or in masses with widely spaced discontinuities;

- **Buckling failure:** This type of failure is generally observed in metamorphic rocks (i.e. phylite, mica schists) or thinly bedded ductile sedimentary rocks (i.e. mudstone, shale, siltstone, sandstone, evaporatic rocks); and

- **Shearing and sliding failure:** Observed in relatively thickly bedded sedimentary rocks and it involves sliding along bedding planes and shearing of intact rock.
These main mechanisms can potentially be used in a mining context. Complete shear failure of the medium can occur in salt or coal mines (Gao et al., 2015) while shearing and sliding has been reported in gold mining in South Africa (Malan, 2002). Buckling is the failure mechanism of interest in this thesis.

Potvin and Hadjigeorgiou (2008) studied squeezing conditions in five different mines in Australia and Canada. They reported that high deformations in deep and high stress mines are generally related to the presence of prominent structural features such as a dominant fracture set, intense foliation or a shear zone and high stress.

Beck and Sandy (2003) recorded squeezing ground conditions in several Western Australian underground mines where the drives were parallel to foliation and the major principal stress was orthogonal to the excavations. Based on typical observations, they showed the modes of ground deformation presented in Figure 2.2, reporting that shearing of foliation in both sidewalls results in "guillotining" of the support elements. Stress induced fractures sub-parallel to the drive back and floor result in bulking as the fractures shear and dilate. This leads to back and floor deformation and probably drives much of the shearing in the walls. Significant bulking above the back is initiated by the development of an open crack where the back meets the hanging wall. Buckling of the drive walls may occur if shearing is present.

Figure 2.2: Modes of large deformations ground behaviour (Beck & Sandy, 2003).
Sandy et al. (2007) noted that the support elements installed at the top of the hanging wall are often the first to be sheared. The buckling tends to be developed in the lower part of the foot wall as a result of the confinement provided by the drive floor and the beneficial effect of the support at the upper part of the walls.

A similar failure mechanism has been observed in Canadian hard rock mines. Potvin and Hadjigeorgiou (2008) noted that squeezing in LaRonde promotes shearing in the top of the hanging wall and the bottom of the foot wall and clamping in the top of the foot wall and bottom of the hanging wall. Figure 2.3 shows a schematic representation of this process.

![Figure 2.3: Shearing in the top of the hanging wall and the bottom of the foot wall and clamping in the top of the foot wall and bottom of the hanging wall (Potvin & Hadjigeorgiou, 2008).](image)

An interpretation of the failure mechanism based on experience in LaRonde and Lapa has been provided by Mercier-Langevin and Wilson (2013) and is presented in Figure 2.4. They identified that the stress redistribution around an opening results in loading of the intact rock in a parallel direction to the foliation planes. This leads in contraction along the foliation and dilation towards the opening. This dilation increases the deflection of the rock layers and decreases the critical buckling load. Bulking appears orthogonal to the foliation as the foliation planes open up. As buckling occurs in the sidewalls, this process is transferred deeper into the rock mass. The buckling process in the sidewalls results in an increased effective span, and reduces the confinement provided to the back and the floor as well as the friction between the foliation planes in these areas. At the early squeezing stages, there is a higher convergence rate in sidewalls whereas later in the process the closure rate reduces in the sidewalls and increases in the back. The authors also noted that deformation in competent sediments is usually minimal as opposed to weak schist.
Figure 2.4: Failure mechanism based on experience at Lapa and LaRonde Mine; a) Initiation of buckling failure in foliated rock; and b) loss of confinement on the back and floor following buckling of sidewalls, after Mercier-Langevin and Hadjigeorgiou (2011).

2.4 Assessment of squeezing in hard rock mines

Although many hard rock mines around the world have problems associated with large scale deformations, there are only few documented case studies in mining under squeezing rock conditions. These are usually related with the presence of foliation and the orientation of the foliation with respect to the drift direction.

Yun-Mei et al. (1984) examined failure modes of foliated rock using physical modelling. Kazakidis (2002) studied the eccentric loading applied in a slab under buckling loading around underground openings. Analytical methods, such as the Euler’s formula (Gere & Timoshenko, 1991) can describe only idealized situations for boundary conditions and load direction, determining the critical load necessary for buckling to occur. The critical buckling load ($\sigma_b$) for a rock slab of thickness ($t$) and length ($l$) shown in Figure 2.5 is given by equation 2.1.

Figure 2.5: Simplified representation of the buckling in foliated rock, after Kazakidis (2002)
\[ \sigma_b = \frac{\pi^2 E}{12q^2(l/t)^2} \]  

(2.1)

Where \( E \) is the Young modulus of the rock, \( (l/t) \) is the slenderness ration and \( q \) is a constant which depends upon the end conditions of the slab. The constant \( q \) has the following values:

- Both ends pin-joined \( q = 1 \)
- Both ends clamped \( q = 1 \)
- One end clamped, one free \( q = 2 \)
- One end clamped, one pin jointed \( q = 1/\sqrt{2} \)

Euler’s solution cannot give quantitative information on the deformation around excavations. It can, however, help in the understanding of the parameters that affect the columnar bending/ buckling effect and the ways to prevent or mitigate it. The overall strength can be increased by providing rigid support or pin support to the rock columns. Reduction of the length of the exposed wall (e.g. by rounding the corners) can decrease the slenderness ratio and increase the critical buckling load. In addition, the pinning action of the reinforcement can result in thicker slabs increasing the strength of the rock slabs.

A review of theoretical criteria available to quantify squeezing ground for both civil and mining applications has been provided by Potvin and Hadjigeorgiou (2008). Barla (2002) has also provided an overview of the methods used for the identification and quantification of squeezing focusing on tunnelling applications. Singh et al. (1992) suggested that rock squeezing occurs when the tangential stress is higher than the uniaxial crushing strength of the rock mass. Aydan et al. (1993) developed theoretical criteria using the ratio of the peak tangential strain at the circumference of the tunnel to the elastic strain to define various degrees of squeezing. Hoek (2001) used the strain produced in a tunnel and the ratio of the rock mass strength to in-situ stress to predict squeezing potential and propose squeezing levels. He classified that strain higher than 1% results to squeezing problems, Figure 2.6. Singh et al. (2007) used the ratio between the expected strain and the critical strain (the strain beyond which squeezing problems may be encountered during construction) to identify different levels of squeezing potential in tunnels.
Empirical classification systems have been used to identify the potential of squeezing ground conditions. Barton (2002) defines mild squeezing rock pressure when the ratio between the maximum tangential stress and the unconfined compression strength is between 1 and 5, and heavy squeezing rock pressure when the ratio is greater than 5. Singh et al. (1992) suggested that squeezing in tunnels can occur when $H > 350Q^{1/3}$ where $H$ is the overburden in (m) and $Q$ is the Tunnelling index proposed by Barton et al. (1974). A statistical analysis using the theory of linear regression was used by Jiménez and Recio (2011) to predict squeezing ground conditions, working on 100 case histories for tunnels up to a depth of approximately 800 m. The authors concluded that the probability of squeezing increases proportionally with depth. In addition, squeezing probability increases as the Rock Mass Number $N$ (i.e. the $Q$ number computed for SRF=1) decreases. It was identified that the span does not have strong influence on squeezing behaviour. Mercier-Langevin and Hadjigeorgiou (2011) discussed the limitations of using rock mass classification for squeezing as same $Q$ rating can occur for different conditions and although the stress increases with depth, the variability of the horizontal stresses can result in different stress conditions for different parts of the world.

Mercier-Langevin and Hadjigeorgiou (2011) presented a “Hard Rock Squeezing Index” for underground mines based on case studies conducted in various mining operations in Australia and Canada and calibrated based on in-situ observations from LaRonde Mine in Quebec, Canada. The authors noted that the index was a preliminary indicator of the squeezing potential of a rock mass and can potentially be useful in other hard

![Figure 2.6: Squeezing levels in tunnelling and the ratio of rock mass strength to in-situ stress (Hoek, 2001).](image)
rock mines experiencing squeezing ground conditions. As shown in Figure 2.7, the index was based on the foliation spacing of the rock mass and the stress to intact rock strength ratio.

\[
\begin{array}{c|c|c|c}
\text{Foliation spacing (s)} & \text{Stress to strength ratio} \\
\hline
s>100mm & \sigma_1/\sigma_{ci} > 0.7 \\
100mm > s > 10mm & 0.3 < \sigma_1/\sigma_{ci} < 0.7 \\
s<10mm & \sigma_1/\sigma_{ci} < 0.3 \\
\end{array}
\]

Where \( \psi \) is the angle between the normal to the plane of the walls of interest and the normal to the foliation planes.

**Figure 2.7:** Squeezing ground index: a) stress matrix to predict areas of squeezing based on foliation spacing; and b) squeezing matrices for varying angle of interception (modified from Mercier-Langevin and Hadjigeorgiou, 2011).

Different squeezing matrices were proposed for varying angles of interception (the angle between the normal to the plane of the wall and the normal to the foliation plane) indicating the importance of the foliation orientation in the squeezing mechanism. More severe squeezing is expected in areas with lower foliation spacing, higher stress to intact rock strength and lower angle between the orientation of the excavation and the foliation. It was also identified that the degree of squeezing increased in areas where localized alteration such as mica, chlorite and tochilinite was present.
The classifications proposed by Hoek (2001), Singh et al. (2007) and Aydan et al. (1993) for the squeezing potential in tunnels are limited to the isotropic behaviour of the materials as they do not take into account any effect of a dominant structure which is commonly observed in mining squeezing problems. Likewise, the Q system considers squeezing rock as plastic flow of incompetent rock under high pressure (Barton et al., 1974). The use of rock mass classification must be done carefully as same Q number can result for different conditions and although stress increases with depth, horizontal stresses in different parts of the world can vary resulting in different stress conditions. Jiménez and Recio (2011) suggest that the span of the tunnel does not have strong influence on squeezing behaviour. In foliated ground, however, this would result in a longer slab on the sidewalls of the underground opening decreasing the critical load for a slab to buckle (Kazakidis, 2002).

The hard rock squeezing index has the potential to be used in mining. Since its introduction in 2010, the Hard Rock Squeezing Index has now been used at Wattle Dam mine in Western Australia (Marlow & Mikula, 2013), the Waroonga underground complex (Woolley & Andrews, 2015) and Westwood mine in Canada (Armatys, 2012). Strain between 1% and 5% recorded in the walls of ore drives in Wattle Dam mine matched successfully the squeezing index expectations for a range of stress to strength ratios. The squeezing areas for Wattle Dam have been included in Figure 2.6. Mercier-Langevin and Wilson (2013) demonstrated that the index predicted effectively the strain in a variety of conditions from no squeezing to extreme squeezing in Lapa Mine. Some cases outside the strain limits of the index with higher strain (upwards of 40%) were discussed.

### 2.5 Magnitude of squeezing in underground hard rock mines

The severity of squeezing can be quantified by deformation or damage. The drift closure magnitude related to squeezing conditions is often reported as wall-to-wall deformation, deformation in a single wall or strain defined as the total wall deformation over the initial drift width. Table 2.1 summarizes the reported level of deformation from several hard-rock mines. The LaRonde and Lapa Mines report some of the highest deformations. This was also identified by the “Hard Rock Squeezing Index” showing only LaRonde, Lapa and Perseverance mines in extreme squeezing conditions. Based on the index, the extreme squeezing conditions at LaRonde are caused in areas with high stress to strength ratio and moderately foliated ground (with foliation spacing approximately between 2 cm and 8 cm) whereas in Lapa in a range of stress to strength ratio and highly foliated rock (with foliation spacing in the scale of mm).
Table 2.1: Squeezing ground conditions reported in hard rock mines, after Karampinos et al. (2015c).

<table>
<thead>
<tr>
<th>Mine site</th>
<th>Strain range % (defined as total wall deformation over the drift width)</th>
<th>Magnitude of deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Lower bound</td>
<td>Upper bound</td>
</tr>
<tr>
<td>LaRonde (Hadjigeorgiou et al., 2013)</td>
<td>2.5%</td>
<td>41%</td>
</tr>
<tr>
<td>Lapa (volcanics and ultramafics) (Mercier-Langevin &amp; Wilson, 2013)</td>
<td>1%</td>
<td>upwards of 40%</td>
</tr>
<tr>
<td>Wattle Dam (Marlow &amp; Mikula, 2013)</td>
<td>1%</td>
<td>5%</td>
</tr>
<tr>
<td>Westwood (Armatys, 2012)</td>
<td></td>
<td>up to 8.5%</td>
</tr>
<tr>
<td>Perseverance (Gaudreau, 2007) (Potvin &amp; Slade, 2007) (Struthers et al., 2000)</td>
<td>2.5 total wall convergence</td>
<td>Wall convergence in excess of 2 m 3 m total sidewall closure, over 1 m of floor heave</td>
</tr>
<tr>
<td>Yilgarn Star (Potvin &amp; Slade, 2007)</td>
<td></td>
<td>Up to 2 m in the hanging wall</td>
</tr>
<tr>
<td>Black Swan (Potvin &amp; Slade, 2007)</td>
<td></td>
<td>Up to 1.5 m in one wall</td>
</tr>
<tr>
<td>Maggie Hayes (Mercier-Langevin &amp; Hadjigeorgiou, 2011)</td>
<td>1%</td>
<td>2.5%</td>
</tr>
<tr>
<td>Casa Berardi (Mercier-Langevin &amp; Hadjigeorgiou, 2011)</td>
<td>1%</td>
<td>5%</td>
</tr>
<tr>
<td>Waroonga (Mercier-Langevin &amp; Hadjigeorgiou, 2011)</td>
<td>1%</td>
<td>5%</td>
</tr>
<tr>
<td>Bousquet (Mercier-Langevin &amp; Hadjigeorgiou, 2011)</td>
<td>1%</td>
<td>10%</td>
</tr>
<tr>
<td>Doyon altered zone (Mercier-Langevin &amp; Hadjigeorgiou, 2011)</td>
<td>2.5%</td>
<td>10%</td>
</tr>
</tbody>
</table>

Another measure under squeezing conditions is the depth of excavation damage. This is important for the design of reinforcement strategies. Mercier-Langevin and Turcotte (2007a) reported on areas at LaRonde Mine where the total wall convergence can be in excess of one meter and the visible fractures can extend as deep as 6 m into the rock mass. Sandy et al. (2010) presented a qualitative and conceptual damage scheme based on observations, Table 2.2. The scheme identifies that, in extreme damage conditions the depth of damage is greater than 4 m and the drives are classified as collapsed openings, beyond rehabilitations. Comparing this description to conditions at LaRonde and Lapa it would appear that what is considered beyond rehabilitation is in this scheme is different than at LaRonde and Lapa. This is possibly due to the differences in the reinforcement and support strategy.
### Table 2.2: Damage classification scheme for stress driven mine tunnel damage for assumed drive width of 5 m, after Sandy et al. (2010).

<table>
<thead>
<tr>
<th>Damage level</th>
<th>General Description</th>
<th>Rock Mass/ Tunnel Damage</th>
</tr>
</thead>
</table>
| S0           | No visible damage (Low stress conditions) | Description: No stress-induced damage visible  
**Area of Damage (% of drive profile):** 0% of profile affected  
**Depth of Damage (m):** 0 m indicated depth of damage  
**Area of Damage (% of drive profile):** 0% of profile affected  
**Ground control:** Easily controlled with minimal support e.g. split sets and mesh |
| S1           | Minor damage (spalling) | Description: Superficial damage only, easily scaled back to good rock  
**Depth of Damage (m):** 0 m to 2 m indicated depth of damage  
**Area of Damage (% of drive profile):** < 10 % of profile affected  
**Ground control:** Easily controlled with minimal support e.g. split sets and mesh |
| S2           | Moderate damage (or spalling) | Description: Spalling clearly developed and more widespread in walls  
**Depth of Damage (m):** indications of damage/loosening to up to 0.5 m depth into walls or backs (~ 10 % of wall or back span)  
**Area of Damage (% of drive profile):** 10 % to 50 % of profile affected  
**Ground control:** Minor rehabilitation required in highly utilization excavations |
| S3           | Significant damage to excavations. | Description: Damage evident in all excavation surfaces. “Bagging” in the mesh clearly developed; Shearing on foliation/bedding clearly indicated, isolated split set head failures.  
**Depth of Damage (m):** Indications of damage/loosening to a depth up to 1.5 m (~ 30 % of wall or back span)  
**Area of Damage (% of drive profile):** > 50 % of profile affected  
**Ground control:** Significant rehabilitation effort required to maintain safe access |
| S4           | Significant damage to excavations. | Description: Severe damage with up to 2 m wall or backs to floor convergence and/or significant floor heave. Passable on foot, with extreme caution, but serviceability significantly reduced. Many bolts broken in shear, mesh severely bagged, some local rockfalls.  
**Depth of Damage (m):** indications of damage/loosening to a depth up to 1.5 m but less than 4.0 m (~ 50 % of wall or back span)  
**Area of Damage (% of drive profile):** > 80 % of profile affected  
**Ground control:** Limit of rehabilitation with conventional support |
| S5           | Extreme damage to excavations. | Description: Widespread support failure and large rockfalls (> 1000 tonnes) and in some cases complete or nearly complete drive closure.  
**Depth of Damage (m):** Indications of damage/loosening to a depth greater than 4.0 m (~ 100 % of wall or back span)  
**Area of Damage (% of drive profile):** > 100 % of profile affected  
**Ground control:** Access not advisable, Beyond rehabilitation |

### 2.6 Support strategies under squeezing conditions

Most research studies have concentrated on squeezing ground conditions in tunneling. Many support systems that can manage ground deformation are available in tunnelling. The use of lining stress controllers (i.e. yielding steel sets embedded in shotcrete lining containing a number of open gaps) can improve the
effectiveness of shotcrete linings used in tunnelling support systems under squeezing conditions. These support systems have been described by Barla and Barla (2008) and Schubert (2008), nevertheless, their use in mining would result in high cost and production delays. Mining operations have shorter service life and higher advancement rates.

In a mining context, Potvin and Hadjigeorgiou (2008) indicated that it is not a realistic approach to stop deformation in squeezing ground conditions as this results to significant rehabilitation work and high support cost. They discussed the differences on the support systems used to control large deformations in Australian and Canadian underground mines in hard rock. Australian mines use soft reinforcement such as FRS. The support system becomes stiffer by using fibre reinforced shotcrete. However, as shotcrete has a limited deformation capability, a further layer of mesh is required to achieve a ductile behaviour after shotcrete has cracked. On the other hand, Canadian mines use a high density of bolts with yielding capability (such as inflatable bolts or hybrid bolt) and weld-mesh complemented sometimes with mesh straps. Mercier-Langevin and Turcotte (2007a) reported some success with a hybrid bolt developed for squeezing ground conditions at LaRonde. The support practices of five mines in Australia and Canada are summarized in Table 2.3. The mines are classified based on the encountered ground conditions (Potvin & Hadjigeorgiou, 2008).

**Table 2.3 Support practices at mines operating under squeezing ground conditions (Potvin & Hadjigeorgiou, 2008).**

<table>
<thead>
<tr>
<th>Mine Site</th>
<th>Support</th>
<th>Mine Site</th>
<th>Support</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mine #1</strong></td>
<td>Split sets (1 x 1 m); 75 mm fibrecrete + weld mesh</td>
<td><strong>Mine #4</strong></td>
<td>Split sets and cable bolts (1.3 x 1.5 m); 50 to 75 mm fibrecrete + weld-mesh</td>
</tr>
<tr>
<td><strong>Mine #2</strong></td>
<td>De-bonded bar and Swellex (1 x 1.5 m); 75 mm fibrecrete + weld-mesh + 50 mm fibrecrete</td>
<td><strong>Mine #5</strong></td>
<td>Split sets, resin bar and cable bolts (50 to 75 mm); fibrecrete + weld-mesh</td>
</tr>
<tr>
<td><strong>Mine #3</strong></td>
<td>Hybrid bolts (0.8 x 0.8 m); weld-mesh</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 2.7 Rehabilitation of squeezed drifts

Most production drives are designed to be in operation up to 18 months to 2 years. In squeezing ground conditions, the resulting loads are usually greater the capacity of a “stiff” support system resulting to considerable failure of ground support and extensive rehabilitation work is required to maintain the drives.
in operation (Potvin & Hadjigeorgiou, 2008). This may include the installation of additional reinforcement elements and surface support.

When convergence reaches a point where wall stability is compromised or if equipment clearance is insufficient, a scoop may be used to “purge” the walls, i.e. to remove excess material and maintain the drives in operation. Figure 2.8 shows an example of extreme deformation at Lapa resulting insufficient space for equipment. Purging can result in very large spans, consequently, additional support such as cables are used to stabilize the greater spans created from this process. This process, delays production and has important cost implications.

![Figure 2.8: Insufficient equipment clearance at a production drift at Lapa Mine.](image)

### 2.8 Conclusions

Squeezing is a mechanical phenomenon related with large deformations around an underground opening. The deformation may stabilize during the construction of the tunnel or continue over a long period. Three main failure mechanism can result in squeezing: complete shear failure of the medium, buckling failure and sliding and shearing failure. This thesis focuses on the buckling mechanism. This phenomenon is associated with failure of the ground support, the need of rehabilitation and high support cost.

A similar buckling mechanism has been reported in Australian and Canadian hard rock mines. This includes the rotation of the major principal stresses and dilation of the rock layers towards the underground opening. Shearing of the foliation promotes the development of buckling in the lower part of the foot wall and the upper part of the hanging wall. This shearing can result in “guillotining” of the support elements.
There are several ways assessing squeezing conditions. The buckling mechanism can be captured using the empirical hard rock squeezing index that implicitly recognizes structure. Generic classification systems are not capable of capturing the structurally driven squeezing mechanisms as they are limited to the isotropic behaviour of the materials or consider squeezing as plastic flow of rock under high pressure.

A review of the measured deformations in underground hard rock mines experiencing squeezing conditions showed that LaRonde, Lapa and Perseverance mines have recorded wall-to-wall deformation in excess of 2.1 m or 40% strain. The depth of excavation damage can extend as deep as 6 m into the rock mass in Canadian mines.

Stopping the deformation is not a realistic approach as this can result in significant rehabilitation and high support cost. Different support approaches are used in Australian and Canadian hard rock mines to control the deformation. The support strategy in Australian mines relies on the use of soft reinforcement and fibre reinforced shotcrete. A further layer of mesh is required when the stiff shotcrete layer cracks. Canadian mines use high density of bolts with yielding capability and weld mesh. Rehabilitation of the affected mine drives is necessary under severe squeezing including the installation of further reinforcement and support elements. Under extreme deformations, the excess material is removed to maintain the drives in operation.
Chapter 3
Squeezing conditions at the LaRonde and Lapa Mines

3 Squeezing conditions at the LaRonde and Lapa Mines

3.1 Introduction

Squeezing conditions were investigated in two underground Canadian mines owned by Agnico-Eagle Mines Ltd: LaRonde and Lapa. Both mines are located in the Abitibi region of Northwest Quebec and have to manage large rock mass deformations that require rehabilitation of affected drifts. Of interest is that the two mines are located 11 kilometers from each other as shown in Figure 3.1 and provide a spectrum of squeezing ground control conditions.

![Figure 3.1: LaRonde and Lapa Mine location in the Abitibi region.](image)

This chapter describes the ground conditions at the LaRonde and Lapa Mines. It includes information about the geological units around the mining infrastructure and the development areas. A summary of the mechanical properties of the rock units pertinent to rock mechanics and the in-situ stress conditions are provided. Reference is made to the mining methods used.
3.2 Squeezing at LaRonde

LaRonde Mine presents a wide spectrum of squeezing conditions. The mine is located in northwest Quebec and has been in operation since 1988. The ore body is a world class Au-Ag-Cu-Zn massive sulphide lenses complex extending below 3110 m depth. The mining methods used are transverse open stoping with cemented and unconsolidated backfill and longitudinal retreat with cemented backfill. Current production is below 830 m depth using a 2,240 m deep shaft. After the construction of an 832 m long internal shaft, a new production horizon was established in 2013 at 2,930 m. Figure 3.2 shows a longitudinal view of the LaRonde ore body.

Figure 3.2: Longitudinal view of LaRonde (Mercier-Langevin & Turcotte, 2007a).
3.2.1 Ground characterization at LaRonde

The ore body is comprised of four tabular zones (7, 19, 20 and 21). Zone 20 (shown in green in Figure 3.2) is the largest of the four with a strike length of 1 km dipping at 80 to the south and a thickness between 3 m and 50 m. Most permanent infrastructures are developed in basalt whereas the majority of the ore access developments are located in felsic tuff (Mercier-Langevin & Turcotte, 2007a). This formation is characterized by tightly spaced foliation dipping between 70° and 85° with an orientation parallel to the ore body. A significant variation of the rock mass behaviour is present around the mine. Although high seismic events are recorded in certain areas, the presence of the foliation under high stress results in highly anisotropic squeezing. In certain areas of the mine the rock mass is more silicified resulting in more brittle behaviour. A conceptual sketch of the major features of mine geology between levels 1940 m and 3000 m is shown in Figure 3.3. Variation of the intact rock strength can also occur around the mine due to the presence of alteration. This process (sericitic alteration) has also a significant influence on the strength of the foliation planes. The mechanical properties of the rock units for the upper part of the mine are shown in Table 3.1. The UCS values refer to the strength of the rock in a direction perpendicular to the foliation. In the new production horizon (2,930 m below surface), alteration is less present and the rock mass is more silicified resulting in a more brittle behaviour.

![Conceptual sketch of major features of mine geology between 194 and 300 levels (Mercier-Langevin, 2003).](image)

Figure 3.3: Conceptual sketch of major features of mine geology between 194 and 300 levels (Mercier-Langevin, 2003).
Table 3.1: Mechanical properties of rock units (Mercier-Langevin & Turcotte, 2007a; Turcotte, 2014).

<table>
<thead>
<tr>
<th>Rock type</th>
<th>UCS (MPa)</th>
<th>E (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Upper part of the mine</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Basalt</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>Intermediate tuff</td>
<td>100</td>
<td>48</td>
</tr>
<tr>
<td>Massive sulphide</td>
<td>140</td>
<td>53</td>
</tr>
<tr>
<td>Semi-massive sulphide</td>
<td>85</td>
<td>47</td>
</tr>
<tr>
<td>Mineralized felsic tuff</td>
<td>140</td>
<td>58</td>
</tr>
<tr>
<td><strong>Level 290 (2,900 m depth)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Basalt</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>Rhyolite</td>
<td>260</td>
<td>66</td>
</tr>
<tr>
<td>Rhyodacite</td>
<td>200</td>
<td>63</td>
</tr>
<tr>
<td>Semi-massive sulphide</td>
<td>200</td>
<td>70</td>
</tr>
</tbody>
</table>

The foliation is present in all rock types having a dominant effect on the rock mass behaviour. Other joint sets are present but with much lower intensity than the foliation. Figure 3.4 shows the joint mapping in the foot wall drifts at level 245. While other joints are present the rock mass behaviour is dominated by the presence of foliation. The foliation spacing varies between millimetres to decimeters, the spacing of other joints is in the scale of multiple metres. Of interest is the localized structures in drifts and this is addressed in chapter 5.

![Figure 3.4: Mapped structures on the foot wall at level 245.](attachment:figure34.png)
A series of stress measurements has been performed in the past to examine the in-situ stress field at the LaRonde Mine. McKinnon (2004) summarized the results from the in-situ stress measurements in levels 146, 150, 218 and 219 between 1999 and 2004. Additional stress measurements were performed on level 215 in 2006. The results from all measurements agree that the major and intermediate principal stress components (σ₁ and σ₂, respectively) are sub-horizontal and the minor principal stress component (σ₃) is sub-vertical. Minor discrepancies on the orientation of the principal stresses have been reported that could be due to the influence of mining induced stresses and yielding of the rock mass. The stress gradients used at the LaRonde Mine are based on numerical modelling work done by Andrieux et al. (2007) after the calibration of a 3DEC model against seismic activity recorded in the two pyramids at 1940 and 2150 m depth. Table 3.2 presents equations used to estimate the in-situ stresses based on the DBS (depth below surface). Calculated values at the elevation of 2,270 m below surface (level 227) are given as an indication. These values give similar stress magnitudes as those recorded in the stress measurements and assume that the maximum principal stress is horizontal with a north-south orientation.

<table>
<thead>
<tr>
<th>Component</th>
<th>Equation</th>
<th>227 Level</th>
<th>Plunge/direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>σ₁</td>
<td>8.62 + (0.04 x DBS)</td>
<td>99 MPa</td>
<td>0°/000°</td>
</tr>
<tr>
<td>σ₂</td>
<td>5.39 + (0.0262 x DBS)</td>
<td>65 MPa</td>
<td>0°/090°</td>
</tr>
<tr>
<td>σ₃</td>
<td>0.0281 x DBS</td>
<td>64 MPa</td>
<td>90°/000°</td>
</tr>
</tbody>
</table>

Variations at the rock mass quality result in differences on the ground response to mining. Seismic activity and squeezing ground conditions may be present in different areas at the same level (Turcotte, 2010). This is illustrated in Figure 3.5. It has been observed that below level 215 the rock mass in the east part of the mine has higher foliation thickness and lower degree of sericitic alteration resulting in a more brittle behaviour and relatively higher seismic activity. On the contrary, the west part of the mine has lower foliation thickness and higher degree of alteration resulting in areas more prone to buckling. Squeezing ground and seismic activity may be present in one single level.
Figure 3.5: a) Typical level layout below level 215, after Turcotte (2010); and b) example of squeezing conditions at LaRonde.

3.3 Squeezing at Lapa

Lapa is a high grade gold mine that began production in 2009 through a 1,369 m deep shaft using two mining methods – longitudinal retreat with cemented backfill, and locally transverse open stoping with cemented backfill. The mine has reported squeezing conditions and efforts were made to investigate the type of squeezing mechanism and support strategy to control large deformations (Mercier-Langevin & Wilson, 2013). Figure 3.6 shows a longitudinal view of the Lapa ore body.

Figure 3.6: Longitudinal view of Lapa (Mercier-Langevin & Wilson, 2013).
3.3.1 Ground characterization at Lapa

The Lapa deposit is hosted by silicified volcanic and sedimentary rocks (minor pyrite, arsenopyrite and stibnite along with visible gold). The gold deposit has an approximate east west direction and is dipping vertical. The deposit begins at approximately 400 m depth up to 1600 m. The horizontal dimension is 400 m. The deposit is comprised of five narrow sub-parallel lenses averaging less than 5 m in thickness. The lithological sequence is shown in Figure 3.7. Three distinct geological units pertinent to rock mechanics are present. The sediments on the north and the volcanics on the south are relatively more competent than the very weak schist zones (ultramafics). These rock units are foliated with the foliation planes oriented sub-parallel to the orebody. The ultramafics have significantly smaller foliation spacing and are subjected to talc-chlorite alteration. Most of the reserves come from the contact zone which is typically found at the contact of the sediments and the ultramafics. The mechanical properties of the three distinct geological units are shown in Table 3.3. The UCS values refer to the strength of the rock in a direction perpendicular to the foliation. The stress field at Lapa is the same as in LaRonde (Mercier-Langevin & Wilson, 2013).

![Figure 3.7: Plan view of lithological sequence (Mercier-Langevin & Wilson, 2013).](image)

Table 3.3: Mechanical properties of the distinct geological domains at Lapa (Dubuc et al., 2012; Mercier-Langevin & Wilson, 2013).

<table>
<thead>
<tr>
<th>Unit</th>
<th>UCS (MPa)</th>
<th>E (GPa)</th>
<th>v</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sediments</td>
<td>80-90</td>
<td>37.9</td>
<td>0.16</td>
</tr>
<tr>
<td>Ultramafics</td>
<td>20-25</td>
<td>6.7</td>
<td>0.18</td>
</tr>
<tr>
<td>Volcanics</td>
<td>50-60</td>
<td>35.2</td>
<td>0.195</td>
</tr>
</tbody>
</table>

The main mining method used at Lapa is longitudinal open stoping with cemented backfill. The sublevels are developed at 30 m intervals and the ore is mined between the sublevels using the longhole blasting technique. Longitudinal retreat is used as illustrated in Figure 3.8. Stopes are typically 12 m wide by 30 m
high and varying thickness from three to five meters. In areas where the width of the ore exceeds 6 metres, primary-secondary transverse stoping is used.

Figure 3.8: Longitudinal retreat open stoping (Mercier-Langevin & Wilson, 2013).

In areas where the longitudinal retreat method is used, stress concentration in ore drifts developed sub-parallel to the foliation results in squeezing ground conditions. Keeping the drifts open for the entire life of an extraction panel (up to two years) can be challenging. On the other hand, when the transverse mining method is used, the access drifts are developed perpendicular to the ore zones and are used for short periods of time. Negligible squeezing occurs in the drawpoints.

A phenomenological description of typical drift deformation in squeezed ground has been provided by Mercier-Langevin and Wilson (2013). They reported that deformation in the competent sediments or volcanics are easily managed, whereas in ultramafic deformation is massive and often requires time consuming rehabilitation that interferes with production. Figure 3.9 shows typical deformed drift profiles based on the location of the weak and strong rock types relative to the drift. Deformation mainly occurs in the part of the drift where the ultramafics are present. It can therefore occur only in one sidewall, in one sidewall and the back or in the back and sidewalls walls.
Figure 3.9: a) Deformation in a haulage drift with south wall (left) in ultramafics; b) early stage deformation in an ore drift where only the north wall (right) is in competent sediments; and c) late stage deformation in an ore drift entirely in ultramafics (Mercier-Langevin & Wilson, 2013).

3.4 Conclusions

LaRonde is a deep and high stress mine with production between 830 m and 2,930 m depth using the transverse open stoping mining method in most places. The rock units are foliated to varying degrees. Variations in the rock mass quality related to the degree of alteration and the foliation spacing result in different ground conditions around the mine. In areas where the rock mass quality is relatively lower, the
rock is subjected to aseismic failure and squeezing ground conditions. On the contrary, in areas where the rock mass quality is relatively better, violent failure and seismic activity has been recorded.

Lapa is a narrow vein mine using the longitudinal retreat mining method in most places. Squeezing conditions appear in drifts below 500 m. Large deformations are associated with the presence of ultramafics with low foliation spacing and talc-chlorite alteration. More competent sediment and volcanic formations are located near the ore. The location of the weak ultramafics relative to the drift influences the deformation profile of the drifts. Walls driven in ultramafics are more prone to squeezing conditions.

Overall, LaRonde and Lapa Mines face large rock mass deformations and a range of conditions can be identified. Further investigations of these conditions can provide a better indication of the ground conditions on the degree of squeezing. This can be achieved by examining a series of squeezing case studies.
Chapter 4
Data collection

4 Data collection

4.1 Introduction

This chapter reports on the methodology followed for the collection of data from squeezing conditions in hard rock mines. An extensive data collection campaign was conducted between October 2012 and May 2013 at the LaRonde and Lapa Mines. The author spent overall 8 months collecting all the pertinent information to study the squeezing conditions encountered at the mines. Every day underground visits were conducted to collect a series of field data. The ground control practices and records were examined. The collection process was complemented by several interviews with the ground control teams from both mines. Supplementary data were collected during additional mine visits and the last data were collected in August 2015.

Rock mass characterization data were collected including measurements of the orientation of the foliation with respect to the direction of mining, the dip angle of the foliation, the foliation spacing, and the condition of the joints. A qualitative assessment of the degree of squeezing was undertaken for each case study based on observations. Deformation monitoring data were collected using cavity monitoring system (CMS) surveys and laser measurements. The extent of the depth of failure in the sidewalls and the back was quantified using electrical and mechanical borehole extensometers. The support installed in each case study was recorded including the presence of any rehabilitation work. The tools used to collect this information are presented in this chapter. The data collection allowed definition of several case studies, based on the ground behaviour. These case studies form the squeezing database for LaRonde and Lapa.

Collecting data is not an objective by itself and unless they are analyzed and summarized they are of limited use (Hadjigeorgiou, 2012). The data collection followed a reparative process in which data was analyzed and transformed into information. The need of supplementary information was subsequently examined along with the characteristics of any required additional data. Part of the data collection was based on the principle of redundancy. In certain cases, several techniques were tried and the most successful was adopted. The experience of the ground control team at each mine was used in determining the best strategy for the collection process.
The first step was to study the ground control plans and procedures at each mine. Several underground inspections were necessary to identify areas suffering from squeezing conditions. Once these areas were identified, a series of case studies was chosen.

4.2 Squeezing case studies

Data were collected from areas in both mines around where squeezing was evident. Each case study was referring to a cross section of an examined drift. In some cases more than one cross section was chosen for one drift. A wide spectrum of squeezing ground conditions was examined from both mines. The following parameters were recorded: the angle between the orientation of the foliation and the drift, the dip angle of the foliation; the orientation of the drift; the observed damage; the geological unit; the development date; the stress effect due to mining activity; the support system used; the additional support installed; the presence of any corrosion on the support and the presence of water. Any intervention such as rehabilitation or purging was also noted. The drift dimension was recorded at 1.5 m and 2.5 m from the floor using laser measurements. The list used to collect information at LaRonde is shown in Figure 4.1.

Figure 4.1: Collected information for LaRonde.
A similar list was used for Lapa. It was, however, modified to account for the different geology and the position of each wall relative to the different geological units. Changes were also made to account for the different support strategy used in each wall when ultramafics are present.

Several case studies were defined. Each case study referred to a cross section of a drift in a direction normal to the direction of development. A total of 61 case studies from LaRonde were examined covering 26 drifts at 17 different levels between 1790 m and 2690 m depth. At Lapa, a total of 86 case studies were recorded in 12 drifts from 12 levels between 510 m and 950 m depth. Given the main mining method used in each mine, most data were collected at LaRonde from haulage drifts whereas at Lapa from longitudinal drifts. Supplementary data on the deformation, the foliation spacing, and the condition of the foliation planes were necessary, in addition to the information listed in Figure 4.1. Sections 4.3, 4.4 and 4.5 describe the process followed to collect these data. The detail of the information recorded between the case studies varied. However, the assessment provided important information about the effect of different ground conditions on the squeezing conditions.

4.3 Rock Mass characterisation data

The rock mass characterisation focused on the presence of foliation including the orientation of the foliation with respect to the direction of the drift, the foliation spacing and the condition of the foliation planes.

4.3.1 Angle of interception

The orientation of the drift with respect to the foliation was recorded using the angle of interception which was defined as the angle between the normal to the foliation planes and the normal to the sidewall, Figure 4.2. The influence of the angle of interception was evident at 2,150 m depth (Mercier-Langevin & Hadjigeorgiou, 2011). As shown in Figure 4.3 there was little evidence of squeezing when the foliation was perpendicular to the drift orientation, relatively more evidence at 45 degrees and severe squeezing when the foliation was parallel to the drift.
This orientation phenomenon is supported by mechanistic analysis. Auto-confinement of foliation planes is greater when the angle between the normal to the free face and the normal to the foliation increases. It has been shown analytically that even a small confining pressure is sufficient to prevent buckling failure (Kazakidis, 2002).

Measurements with a geological compass were influenced by the magnetic properties of the excessive steel support installed in the walls (wire meshes and straps). Consequently, a goniometer with a bubble level adjustment was used to identify the angle between the strike of the foliation planes and the orientation of the drift. The design plans were also used to identify the orientation of the drifts in the case studies. A dip angle of 90° was used for the sidewalls. The measurements were supplemented in certain cases by the geological structural mapping available at the mines. The angle between the foliation plane and the sidewall was measured for each case study using DIPS 7.0 (Rocscience Inc, 2016a).
4.3.2 Foliation spacing

The foliation frequency was measured in 39 case studies at the nearest intersections at LaRonde. The frequency was measured in scan lines of a maximum of 5 m length for each side of the drift and recorded as breaks per meter in a horizontal direction, vertical to the strike of the foliation planes as shown in Figure 4.4. Measurements were taken from both sidewalls when possible.

Figure 4.4: a) Foliation frequency measurements from scanlines at LaRonde; and b) estimation of foliation frequency vertical to the strike of the foliation (plan view).

The mining method at Lapa did not facilitate any measurements near the examined areas. Most case studies were located in longitudinal drifts, away from intersections. Furthermore, there are not ultramafic exposures at the intersections. The foliation spacing of sediments was measured in scan lines for certain case studies. It was, however, not possible to assign a single foliation frequency value for each case study due to the variations in the geology and the foliation spacing. A range of foliation spacing was assigned instead in each geological unit based on underground measurements and observations.
4.3.3  Condition of foliation planes

4.3.3.1  Alteration

The joint alteration is a critical parameter in the characterization of a jointed rock mass. Milne and Hadjigeorgiou (2000) noted that in the most popular classification systems used, the alteration is described in only subjective terms resulting in difficulties in the making accurate and repeatable assessments of this property. The approach by Milne and Hadjigeorgiou (2000) was used for estimating the alteration of the foliation planes ($J_a$) as presented in Table 4.1 was used.

Table 4.1: Field measurements approach for estimating $J_a$ (Milne & Hadjigeorgiou, 2000).

<table>
<thead>
<tr>
<th>Description</th>
<th>$J_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cannot be scratched by a knife</td>
<td>0.75</td>
</tr>
<tr>
<td>Can be scratched by a knife</td>
<td>1.0-1.5</td>
</tr>
<tr>
<td>Can be scratched with fingernail and feels slippery when touched</td>
<td>2.0</td>
</tr>
<tr>
<td>Can be dented with a fingernail and feels slippery when touched</td>
<td>4.0</td>
</tr>
</tbody>
</table>

At LaRonde, the effect of the sericitic alteration in the intermediate tuff has been discussed by Turcotte (2010) and Mercier-Langevin and Turcotte (2007b). Localised sericite alteration zones present a tighter foliation spacing and lower rock mass properties due to lower intact rock strength and joint resistance. A range of alteration values was recorded from the scan lines in the 39 case studies.

Mica altered zones at Lapa result in more slippery interfaces. Biotite and silica alteration are present in the Piche Group volcanics. Talk chlorite alteration is present in the ultramafics (schist) and sericitic alteration is present in the more competent sedimentary rock (Dubuc et al., 2012; Mercier-Langevin & Wilson, 2013). The variation in the geology in all the case studies at Lapa made it impossible to assign a single joint alteration number for each case study. Ranges of joint alteration values were recorded for the different geological units based on field measurements in a series of case studies.

4.3.3.2  Roughness

The joint roughness is another critical parameter for the behaviour of the jointed rock masses. The methodology proposed by Milne and Hadjigeorgiou (2000) was used to determine the roughness of the foliation planes for each case study. A small ruler and a 1 m long stick were used to evaluate the relationship between the joint amplitude ($a$) and the joint length ($l$). The ratings were based on the approach presented in Table 4.2. The measurements were focused on the small scale roughness as shown in Figure 4.5.
Table 4.2: Field measurement approach for obtaining $J_r$ values (Milne and Hadjigeorgiou, 2000).

<table>
<thead>
<tr>
<th>$J_r$</th>
<th>Large scale roughness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Small Scale Roughness</td>
<td>Planar ($a/l&lt;10\text{mm/m}$)</td>
</tr>
<tr>
<td>Slickensided</td>
<td>$J_r = 0.5$</td>
</tr>
<tr>
<td></td>
<td>$JRC_{2m} = 5, JRC_{1m} = 4$</td>
</tr>
<tr>
<td>Smooth ($a/l&lt;2.5\text{mm/10cm}$)</td>
<td>$J_r = 1.0$</td>
</tr>
<tr>
<td></td>
<td>$JRC_{2m} = 1.5, JRC_{1m} = 0.9$</td>
</tr>
<tr>
<td>Rough ($a/l&gt;2.5\text{mm/10cm}$)</td>
<td>$J_r = 1.5$</td>
</tr>
<tr>
<td></td>
<td>$JRC_{2m} = 2.5, JRC_{1m} = 2.3$</td>
</tr>
</tbody>
</table>

Figure 4.5: (a) Smooth foliation planes in intermediate tuff at LaRonde and (b) slickensided foliation planes in ultramafic formations at Lapa.

4.4 Deformation monitoring data

Previous analysis of the squeezing phenomenon at LaRonde was focused on qualitative information (Mercier-Langevin, 2005) for the drift closure. For the purposes of this investigation, the convergence on the walls was derived from cavity monitoring system (CMS) surveys and laser measurements. The deformation inside the drift walls was also recorded in certain cases using electrical and mechanical multipoint borehole extensometers.

4.4.1 Cavity monitoring system surveys

The cavity monitoring survey system CMS V400 (Optech Incorporated, 2010) was used to capture the drift profile during the various stages of the squeezing process. In operation, the CMS scanning head was placed near the centre of the drift. The instrument produces a laser beam that is reflected from the wall surface.
measures the time of flight of the laser pulse to a surface and back and converts it into a distance reading. The theory of operation and an example survey at Lapa are shown in Figure 4.6. The system takes distance measurements as the scan head rotates clockwise by an angular increment (azimuth step). After a complete rotation it elevates by a specified angle (elevation step). The elevation angle and the azimuth step chosen for the surveys was 1°. The position of the instrument in space was determined from each scan using a total station. Multiple scans were undertaken to capture the 3D profile of drifts. The instrument was placed along the drift every 10 m to 15 m depending on the conditions, minimizing any blind spots due to the deformed walls.

![CMS theory of operation](image1)

**Figure 4.6: CMS theory of operation and survey in drift GM-86-C1-37E at Lapa.**

The author worked with the mine survey technicians and conducted a series of scans. Four drifts were scanned at LaRonde and five drifts at Lapa. Some drifts were surveyed more than once at different time intervals. At Lapa, it was possible to capture the profile of the walls during the development of one drift through multiple scans at 11 time intervals. An example of multiple CMS surveys in a drift at Lapa is shown in Figure 4.7 illustrating the profile of the drift in cross section at two different dates.
4.4.2 Laser distance measurements

In cases where it was not possible to conduct CMS surveys due to operational and time restrictions, a laser measuring device was used. The distance between the two sidewalls was measured and recorded for each case study at heights of 1.5 m and 2.5 m from the drift floor. The back–to-floor distance was also estimated at the centre of each drift and at 1 m on each side from the centre. To minimize any error from the presence of muck on the floor, rock bolts were installed in the floor at LaRonde, without being anchored. The bolts would have acted as a reference point for the back–to-floor measurements; however, they were damaged from machinery before any measurements were taken.

4.4.3 Borehole Extensometers

4.4.3.1 Electrical Extensometers

LaRonde and Lapa use electrical multipoint borehole extensometers to monitor the deformation at various depths within the drift walls. Measurements from extensometers installed in the back at intersection between drifts are available at the mines. The instruments used are the Multi-Point Borehole Extensometer (MPBX) (Mine Design Technologies, 2016) and the D-EXTO (YieldPoint, 2016). Both extensometers use potentiometer transducers to measure the deformation at 6 anchor points. They are fully grouted in the borehole and have a high accuracy which is in the scale of micrometers. Figure 4a shows a schematic representation of the D-EXTO. Measurements from extensometers installed in both mines indicated that these instruments cannot measure the excessive deformation in the walls. The extensometers are susceptible
to shear and, depending on the ground conditions, fail after several tens of centimetres of deformation. In particular, when installed in the sidewalls, the instruments fail in shear in earlier squeezing stages. The depth of failure recorded at the intersection in the back is above the length of the rebars. Figure 4.8 shows the measurements from a 10 m long MPBX installed in the back at the intersection between to longitudinal drifts at Lapa. The instrument failed after recording approximately 60 mm of deformation indicating a depth of failure of 3.3 m. Depth of failure in excess of 8 m has been recorded in the back of intersections at Lapa.

Figure 4.8: a) Schematic representation of the D-EXTO extensometer (YieldPoint, 2016); and b) measurements from 10 m long extensometer installed in the back at the intersections of drift EP-83-E and EP-83-W.

4.4.3.2 Mechanical Extensometers

Mechanical extensometers were installed in a series of case studies to record the deformation in the back and the sidewalls. The extensometers were manufactured by the ground control teams of each mine under the guidance of the author. The instruments were based on extensometers used for routine monitoring as roadway deformation indicators in the coal mining industry (Bigby et al., 2010). At Lapa, extensometers were typically comprised of six Ø4 mm wires anchored mechanically at different depths into the walls in Ø64 mm boreholes. Figure 4.9 shows a schematic representation of the mechanical extensometer and the installation of an extensometer at Lapa. The wires were extending outside the borehole. A reference tube was anchored at the end of the borehole using polyurethane foam. The movement of each anchor point was determined from measurements on the length of each wire extending outside the borehole, relative to the bottom of the reference tube. The movement between anchors was calculated by subtracting the movement of the lower anchor from the movement of the upper anchor.

A pull force was applied to each wire, before taking any readings, to determine the extent of shear in the walls. Wires trapped in the hole due to shearing were not returning to their previous position when released and were being marked as sheared. The stiffness of the extensions springs was high enough to allow
tensioning the wires without extending the springs. The readings were taken by applying a minimum force and tensioning each wire separately, without extending the springs, and measuring the length of the wire extending outside the borehole. The mechanical extensometers provided lower accuracy in the measurements than the electrical extensometers. However, they were proven to be less susceptible to shear and allowed to record deformation in the sidewalls and the back that had not been measured before. Single height extensometers were installed at LaRonde using the same concept as at Lapa.

Figure 4.9: a) Schematic representation of the mechanical extensometer used to capture the deformation into the walls; and b) installation of a mechanical extensometer in the back of drift GM-54-45E at Lapa.

4.5 Ground support data

The timing of support placement and the density of the support was recorded. The presence of any additional support or any rehabilitation work was also noted. For LaRonde, the total length of purging along the drifts around the mine was examined and compared for the different support standards used at the mine. Wall purging is a critical operation and is always performed under close supervision from ground control personnel. As it is difficult to control the amount of broken material that falls off during purging, the final size of the drift often exceeds its original dimensions. It is then necessary to install additional support, such as cable bolts, in order to stabilize the greater spans created by scaling of the side walls. This rehabilitation process is costly and time-consuming and only used as a last resort. Figure 4.10 shows an example from a purged drift at LaRonde.
Figure 4.10: Purging of excessive deformation in the walls in drift GM-182-20E at LaRonde; a) view of the drift before purging; b) view of the drift after purging; and c) additional support installed in the sidewalls.

4.6 Summary and conclusions

Ground characterization, ground support and deformation data were collected in a series of squeezing case studies at LaRonde and Lapa Mines between October 2012 and August 2015. Each case study was referring to a cross section of an examined drift. The collected information was related to the angle between the orientation of the foliation and the drift, the dip angle of the foliation, the orientation of the drift, the observed damage, the geological unit, the development date, the stress effect due to mining activity, the support system used, the additional support installed and the presence of water. Any intervention such as rehabilitation or purging was also recorded.

Deformation monitoring data were collected using CMS surveys and laser measurements. In certain cases the deformation was recorded at different time intervals. Information from electrical borehole extensometers was collected. Mechanical extensometers were designed and manufactured at the mine sites to overcome the limitation of the early shear failure of the electrical extensometers when installed in the walls. The ground support used and the presence of any rehabilitation were also recorded. The length of purging along the drifts at LaRonde was recorded for a period of eight years.

The combined interpretation of the data can provide essential information for the assessment of the parameters controlling the squeezing ground conditions in hard rock mines. Deformation monitoring data can be used for the validation and calibration of numerical models.
Chapter 5
Interpretation of in-situ information from LaRonde and Lapa Mines

5  Interpretation of in-situ information from LaRonde and Lapa Mines

5.1  Introduction

This chapter investigates the interaction of the rock mass characteristics, mining and time on the resulted deformation. The data from the CMS surveys and the laser measurements collected in chapter 4, were used to quantify the observed convergence in the case studies. The methodology used to estimate the strain in the walls is presented.

The strain data were used to study the effect of the drift orientation with respect to foliation on the recorded strain and strain rate. The data from the investigation of the foliation spacing measurements and the condition of the foliation planes are presented. The interpretation of the CMS survey and the borehole extensometer data indicated the effect of the stress disturbance on the recorded deformation due to mining in the vicinity of the case studies. The strain measured in the case studies is compared with the strain predictions from the squeezing index to assess the effectiveness of the index in predicting squeezing conditions in hard rock mines. The progress of deformation over time was explored as captured by CMS surveys during the development of a drift. The convergence data were used to identify any trends on the displacements and displacement rates recorded at the LaRonde and Lapa Mines.

5.2  Quantifying the observed convergence

The data from the CMS surveys were used along with the laser measurements to determine the total sidewall and total back–to-floor convergence for 61 case studies at LaRonde and 86 case studies at Lapa. The distance between the two sidewalls was extracted from the CMS surveys, and recorded at heights of 1.5 and 2.5 m from the drift floor. The back–to-floor distance was estimated at the centre of each drift and at 1 m on each side from the centre.

The greatest wall convergence was derived from CMS survey readings for 35 case studies at LaRonde and 73 case studies at Lapa. 3D surveys, showing the initial drift dimensions after the development of every drift, are also available for both mines. These surveys are made by surveying one point at the back, the floor, and each sidewall immediately after the development of a drift. The convergence was determined by comparing the initial 3D survey results during development and the CMS survey profile after a time
interval. Data collection was complicated by the frequent presence of muck on the side of each drift. Figure 5.1 shows the various methods used to identify the convergence in each case study and the reference points along the drift.

![Diagram of drift convergence](image)

**Figure 5.1: Estimation of drift convergence (Karampinos et al., 2015c).**

The total wall-to-wall convergence \( (\delta_{\text{total}}) \) was estimated from the difference between the surveyed width \( (s) \) and the lowest sidewall distance measured. Similarly, the total back-to-floor convergence was derived from the lowest back-to-floor distance and the height of the drift. The total wall-to-wall and back-to-floor convergences were expressed as percentages of the total strain \( (\varepsilon_{\text{total}}) \):

\[
\varepsilon_{\text{total}} = \frac{\delta_{\text{total}}}{s} \times 100
\]  

(5.1)

It is recognized that operational restrictions can influence the data collection process. In particular, when the wall-to-wall closure is close to 3.5 m, the drift becomes nonoperational for equipment and therefore it is purged. Consequently, a value of 3.5 m was used as the lowest wall-to-wall distance measured in these cases. Typical squeezing examples observed in LaRonde and Lapa Mines are presented in Figure 5.2.
Figure 5.2: Examples of drifts subjected to squeezing at the LaRonde and Lapa Mines (Hadjigeorgiou et al., 2013).
A revised classification scheme for quantifying squeezing in hard-rock mines is proposed:

- No or low squeezing \( (0\% < \varepsilon < 5\%) \)
- Moderate squeezing \( (5\% < \varepsilon < 10\%) \)
- Pronounced squeezing or rehabilitated drifts \( (10\% < \varepsilon < 35\%) \)
- Extreme squeezing \( (\varepsilon > 35\%) \)

Published work in civil engineering tunnelling applications summarized by Potvin and Hadjigeorgiou (2008) reported considerably lower ranges than those proposed here. This is a result of the higher tolerance of rock mass failure in a mining environment. The Hard Rock Squeezing Index used the same strain ranges as Hoek (2001). A comparison between the previous squeezing classifications in tunnelling and mining and the proposed classification is shown in Table 5.1. At the time of the introduction of the squeezing index the observed deformation in the developed drift was considered extreme if it was higher than 10%. Since 2011, however, mines such as LaRonde and Lapa have managed, through the use of effective reinforcement strategies, to keep drifts under operation in more extreme conditions.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Squeezing Level</td>
<td>Tunnel Strain</td>
<td>Squeezing Level</td>
</tr>
<tr>
<td>Few support problem</td>
<td>( \varepsilon &lt; 1% )</td>
<td>No squeezing</td>
</tr>
<tr>
<td>Minor squeezing problem</td>
<td>( 1% &lt; \varepsilon &lt; 2.5% )</td>
<td>Minor squeezing</td>
</tr>
<tr>
<td>Severe squeezing problem</td>
<td>( 2.5% &lt; \varepsilon &lt; 5% )</td>
<td>Severe squeezing</td>
</tr>
<tr>
<td>Very severe squeezing problem</td>
<td>( 5% &lt; \varepsilon &lt; 10% )</td>
<td>Very severe squeezing</td>
</tr>
<tr>
<td>Extreme squeezing problem</td>
<td>( \varepsilon &gt; 10% )</td>
<td>Extreme squeezing</td>
</tr>
</tbody>
</table>

For comparison purposes, the convergence was also examined for each wall separately. This was defined as the ratio of the highest recorded convergence (\( \delta \)) for each wall to half of the surveyed width (\( s \)) for the sidewalls or to half the surveyed height for the back and the floor. The convergence for each wall was expressed as percentage strain (\( \varepsilon \)):

\[
\varepsilon = \frac{\delta}{s/2} \times 100
\]  
(5.2)
This methodology was also used to determine the strain for each wall along drifts at Lapa, when developed between different geological units.

### 5.3 Influence of drift orientation

The influence of the angle of interception ($\psi$) on the resulting total wall-to-wall and back-to-floor strain is shown in Figure 5.3.

Figure 5.3: Influence of angle of interception ($\psi$) on resulting total strain at the LaRonde and Lapa Mines; a) total wall-to-wall strain; and b) total back-to-floor strain.

The influence of the angle of interception ($\psi$) on the resulting strain for each wall at the LaRonde and Lapa Mines is presented in Figure 5.4. These diagrams capture only a part of the behaviour of a rock mass under squeezing ground conditions. There are further factors that can influence the resulting strain, such as the time of measurement, the foliation spacing, the stresses, the strength of the rock, and the condition of the joints. In addition, operational constraints do not allow for a drive with a wall-to-wall distance less than 3.5 m.
Figure 5.4: Influence of angle of interception ($\psi$) on resulting back and floor strain at the LaRonde and Lapa Mines; a) south wall; b) north wall; c) back strain; and d) floor strain (Karampinos et al., 2015c).

The graphs include a threshold of the highest expected strain for a given angle of interception. It is noted, however, that there is limited data for an angle of interception less than 10° in extreme squeezing conditions. Nevertheless, the field data clearly indicates that an increase in the angle of interception between the drift and the inherent foliation will invariably reduce the resulting level of squeezing.

The south walls demonstrated the highest strain, exceeding 50% in certain case studies. These values are also higher than the total sidewall strain recorded. The difference between the convergence on each sidewall (north walls and south walls) at Lapa was also identified by Mercier-Langevin and Wilson (2013). Higher strain, resulting in frequent rehabilitation, was linked with the presence of ultramafics, whereas lower strain, easily managed, was associated with relatively competent sediments and volcanics. The ultramafics have
typically much smaller foliation spacing and talc-chlorite alteration is usually present. The examined south walls at Lapa were comprised of ultramafic formations while the north walls were mostly driven in sediments.

The strain is estimated by comparing the 3D surveys with which the CMS surveys. Visual observations at Lapa suggest that the recorded strain in the north walls is sometimes higher. Higher strain can be a result of errors in the positioning of the CMS or any shape irregularities on the wall, as the 3D survey considers the wall as a plane surface. The reported total sidewall and total back-to-floor strain has significant practical implications for the functionality of the drifts and the need for rehabilitation to maintain them in operation. The determination of the strain for each individual wall can allow for a more representative consideration of the geological and mineralogical conditions that can influence the squeezing level. Consequently, the influence of other factors controlling the degree of squeezing, such as the foliation spacing, the alteration, intact rock strength, and the stress can be explored in greater detail.

The influence of the angle of interception ($\psi$) on the resulting total wall-to-wall and back-to-floor strain rate is illustrated in Figure 5.5. The graphs indicate that an increase in the angle of interception reduces the strain rate. The strain rates measured at Lapa were higher. This is related to the other parameters affecting the squeezing process such as the foliation spacing, the degree of alteration, the stresses and the strength of the rock.

![Figure 5.5: Influence of angle of interception ($\psi$) on resulting sidewall strain rate at the LaRonde and Lapa Mines; a) total wall-to-wall strain rate; and b) total back-to-floor strain rate.](image)

Currently there is a lack of a ground support system that can fully control pronounced and extreme squeezing ground conditions. Consequently, exploring changes in the orientation of development can be an
effective strategy in mining under such conditions. While this has been an opportunity in LaRonde, it is more difficult at Lapa, due to the lower flexibility allowed by the mining method.

Hoek and Marinos (2000) used the ratio of the uniaxial compressive strength ($\sigma_{cm}$) of the rock mass, to the overburden stress ($p_o$) to predict the extent of squeezing in tunnelling, using a similar method to estimate the wall strain. This approach does not consider the anisotropic behaviour of the rock mass and the presence of any dominant structure. A successful classification system for the prediction of the level of squeezing at LaRonde and Lapa should take into account the influence of foliation and the angle of interception ($\psi$).

## 5.4 Influence of foliation spacing

The foliation frequency was measured at the nearest intersections in 39 case studies in LaRonde. Measurements were taken from both sidewalls in most cases. The frequency was measured in scan lines of a maximum of 5 m length for each side of the drift and recorded as breaks per meter in a horizontal direction normal to the strike of the foliation planes. The thickness of the largest and the smallest rock block was also recorded for each scan line. The foliation frequency measurements at the north wall and the south wall for each case study are shown in Figure 5.6. The graph shows that there is no significant difference between the measurements for each wall for each case study. This indicates that the highest strain recorded in the south walls at LaRonde is not related to any changes in the frequency of the foliation between the two walls and could potentially be attributed to the higher induced stresses in the south part of the mine.

![Figure 5.6: Foliation frequency measured in the south wall and the north wall at LaRonde.](image)

The mean value of the measured frequency between the two walls was used for each case study. This value was transformed to mean foliation spacing for each case study. Figure 5.7 shows the highest and lowest foliation spacing recorded in the scan lines along with the mean foliation spacing derived from the measurements. The foliation spacing measured varied between 1 mm and 650 mm whereas the mean
foliation spacing derived from measurements was between 18 mm and 76 mm. Although higher strain values might be expected in areas of high foliation frequency, this was not fully supported by the field measurements.

Figure 5.7: Foliation spacing measured at the LaRonde case studies.

At Lapa, it was not possible to assign single foliation frequency to each case study due to the variation in the geology between the walls. Furthermore there are no ultramafic exposures at the intersections of longitudinal drifts where the foliation frequency can be measured in scan lines. Underground measurements from scan lines in sediment units at 540 m, 570 m and 860 m depth indicated foliation spacing between 10 mm and 500 mm. Measurements on ultramafic walls on longitudinal drifts indicated foliation spacing between 1 mm and 100 mm. Overall, the underground measurements and observations at the walls near the case studies were similar to the foliation spacing reported by Mercier-Langevin and Wilson (2013) and are presented in Table 5.2.

Table 5.2: Foliation spacing at Lapa.

<table>
<thead>
<tr>
<th>Unit</th>
<th>Foliation spacing (mm)</th>
<th>Highest</th>
<th>Lowest</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sediments</td>
<td></td>
<td>500</td>
<td>10</td>
</tr>
<tr>
<td>Ultramafics</td>
<td></td>
<td>100</td>
<td>1</td>
</tr>
<tr>
<td>Volcanics</td>
<td></td>
<td>200</td>
<td>50</td>
</tr>
</tbody>
</table>
Figure 5.8 illustrates the large influence of the geology on the recorded strain at Lapa. The graph shows the recorded strain derived from CMS surveys along the longitudinal drift GM-54-45E at 540 m depth. The north wall strain was overestimated in certain cases. As discussed in section 5.2, this can be due to the fact that the inclination of the walls in not considered in the 3D survey or due to errors in the positioning of the CMS surveys. In addition, the strain measurements in the floor were influenced by the presence of muck. Nevertheless, the high strain recorded in the south walls correlates well with the presence of ultramafics. On the other hand, the low strain values in the north wall are related with the presence of sedimentary units. The plotted cross sections show that in the majority of the cases the highest deformation in the back was recorded in the part of the wall located in ultramafic units. It is recognized that local variation in the geology and localized stress changes can also influence the recorded deformations and strain.

Figure 5.8: Influence of the geology on the highest measured wall strain in drift GM-54-45E at Lapa.
5.5 Influence of the condition of foliation planes

5.5.1 Alteration

Sericitic alteration is present at various areas of the LaRonde Mine, arguably influencing the strength of the foliation planes and the resulting deformation. A joint alteration number was assigned from each case study following the methodology approach by Milne and Hadjigeorgiou (2000). The measurements for the alteration are shown in Figure 5.9. Although higher strain values might be expected for higher joint alteration number, this was not fully supported by the field measurements at LaRonde.

![Mean alteration](image)

**Figure 5.9: Field stain measurements and joint alteration at LaRonde.**

The joint alteration number was examined separately for each geological unit at Lapa. Mica altered zones results in more slippery interfaces along the foliation planes. Talc-chlorite alteration is present at the ultramafic units. Figure 5.10 shows examples of biotite and chlorite schist exposures. The measurements resulted in ranges of the joint alteration number for the distinct geomechanical units shown in Table 5.3.

**Table 5.3: The joint alteration number ranges measured for the different geological units at Lapa.**

<table>
<thead>
<tr>
<th>Unit</th>
<th>Joint alteration number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sediments</td>
<td>1-1.5</td>
</tr>
<tr>
<td>Ultramafics</td>
<td>3-4</td>
</tr>
<tr>
<td>Volcanics</td>
<td>1.5-3</td>
</tr>
</tbody>
</table>
5.5.2 Roughness

The roughness of the foliation planes measured at the LaRonde case studies was consistently smooth and planar with values of $J_r = 1$. The roughness of the foliation planes at Lapa was examined separately for each geological unit. All the examined planes were planar. The joints in volcanics and ultramafics are slickensided whereas the joints in sediments are either smooth or rough. The ranges of $J_r$ recorded for the different geological units are presented in Table 5.4.

Table 5.4: The joint alteration number ranges measured for the different geological units at Lapa.

<table>
<thead>
<tr>
<th>Unit</th>
<th>Joint roughness number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sediments</td>
<td>1-1.5</td>
</tr>
<tr>
<td>Ultramafics</td>
<td>0.5-1</td>
</tr>
<tr>
<td>Volcanics</td>
<td>0.5-1</td>
</tr>
</tbody>
</table>

5.6 Effect of stress

LaRonde is characterized by areas of high seismicity and squeezing. A series of rockbursts has been reported (Turcotte, 2014). In areas where the foliation thickness is lower, the stress redistribution around the drifts results in axial loading of the foliation planes and triggers the buckling squeezing mechanism. The rock layers are more prone to buckling when the stress is higher or the intact rock strength is lower. The effect of the stress to intact rock strength ratio at LaRonde was described by Mercier-Langevin and Hadjigeorgiou (2011). The index indicated the clear difference on the lower degree of squeezing in the upper part of LaRonde, where the far field stresses are lower, and the larger deformation observed at the lower part of the mine, where the stresses are higher. It used the far field stresses for LaRonde to determine
the range of the stress to strength ratio for the areas where large deformation was evident at the mines. The mining sequence results in higher induced stresses that can arguably influence the level of squeezing.

Numerical stress analysis could provide further insight on the stress redistribution around the mine. There has been some success for predicting areas with rockburst conditions but these models are not calibrated based on squeezing conditions. Previous attempts at LaRonde developing global mining models have failed to capture the squeezing areas. Following a series of investigations, it was decided that the buckling mechanism had to be addressed specifically around specific drifts. This has been the motivation for the numerical approach followed in chapter 6.

5.6.1 Sensitivity of monitored deformation to stress disturbance

The monitored deformation is sensitive to the mining sequence and production blastings at the vicinity of the case studies. This was observed from the CMS surveys measurements for LaRonde and from borehole extensometer results for Lapa.

5.6.1.1 LaRonde case studies

Figure 5.11 shows an example of squeezing over time as captured from multiple CMS surveys, in a haulage drift developed sub-parallel to the foliation at 2690 m depth. The surveys reported a displacement of 1.3 m in the hanging wall after 679 days from the development of the drift. The analysis included an interpretation of the highest deformation in each wall over time and the mining activity at the vicinity of the section. There was a correlation between the increase of the deformation rate in the hanging wall and the back, and the stopes blasted 232 days after the development of the drift. This area was subsequently rehabilitated.
5.6.1.2 Lapa case studies

Mechanical borehole extensometers at Lapa captured the depth of failure in the walls. The interpretation of the data took into account the progress of deformation over time and the time of the production blastings at the mine. This allowed examination of any correlation between the recorded displacement and the mining activities at the vicinity of the case studies.

Figure 5.12 shows the results from mechanical extensometers installed in the southwall and the back at drift GM-54-45E (540 m depth). The extensometers were installed after squeezing had already progressed in the
drift. A CMS survey prior to the installation of the extensometer indicated that the wall displacement was 500 mm. The peak on the displacement graph at 3 m into the south wall may be due to movement of the rock layers inside the walls or to slipping of the anchor point in the borehole. The results suggest that the depth of failure is between 3.5 m and 5 m in the south wall and 5 m and 6.5 m in the back. The graph shows a correlation between the increase on the displacement rate in the back and the south wall and the mining activity at the vicinity of the examined area. The displacement rate decreases several days after blasting each stope.

Figure 5.12: Increase in the displacement and displacement rate in the back and the south wall after stopes mined at the vicinity of extensometers at drift GM-54-45E.
Figure 5.13 summarizes the results from mechanical extensometers installed in the southwall and the back at drift GM-54-45W. The measurements show that the depth of failure is between 3.5 m and 5 m in both the southwall and the back. Same as the extensometer results in GM-54-45E, blasting in a stope near the examined area, increased the displacement rate. The displacement tends to stabilize 50 days after blasting.

Figure 5.13: Increase in the displacement and displacement rate in the back and the south wall after stopes mined at the vicinity of the extensometers at drift GM-54-45W.

In drift GM-86-C1-37E (860 m depth), mechanical borehole extensometers were installed in the south wall, the north wall and the back. The north wall was developed in sediments whereas the back and the floor were developed in ultramafics. The measurements show large deformation in the back and the south wall and minor deformation in the north wall, Figure 5.14. The depth of failure is between 3.5 m and 5 m in the south wall and above 6.5 m in the back. A measurement is missing from the south wall extensometer from
2 m inside the wall, 120 days after the installation of the instrument. This may have been due to a reading error or slipping of the anchor point inside the borehole. The displacement rate in the back and the southwall increased after blasting a stope at the vicinity of the examined area and was reduced 50 days later.

Figure 5.14: Increase in the displacement and displacement rate in the back, the south wall and the north wall after stopes mined at the vicinity of the extensometers at drift GM-86-C1-37E.
5.7 Validation of the squeezing index prediction from the measured strain in the field

The field data from the LaRonde and Lapa Mines were plotted on the squeezing index graph to compare the measured strain with the level of predicted squeezing. Figure 5.15 includes the total wall-to-wall strain for 39 case studies at LaRonde. The highest, lowest and mean foliation spacing values were measured underground for each case study as described in section 5.4 and presented in Figure 5.7. The upper, lower and mean foliation spacing were included in the squeezing matrixes for each case study as single points. Each point is color coded based on the assessed squeezing condition of the case study that it refers to. The revised classification scheme for quantifying squeezing in hard-rock mines proposed in section 5.2 was taken into account. For example, for the pronounced squeezing conditions in “EP-255-34W sect 2” presented in Figure 5.7, red points were included in the matrix for an angle of interception less than 25° for 5 mm, 200 mm and 36.6 mm foliation spacing. The set of data from all the case studies form an ellipse for each matrix covering areas of low to high foliation spacing and a range of σ\(_1\) to σ\(_{ci}\) ratios.

For an angle of interception less than 25°, part of the ellipse is located in the “extreme squeezing” prediction area of the index. This indicates that strain in excess of 10% can be expected. This is in agreement with the ground conditions measured in the case studies. For low foliation spacing (less than 10 mm), the strain can be in excess of 10%. For large foliation spacing (in excess of 100 mm) the strain can be less than 5%. For an angle of interception between 25° and 40°, the index slightly underestimated the level of squeezing in certain areas. In these cases, the largest strain predicted by the index is below 5% while the strain recorded in the case studies was in excess of 10%.
Figure 5.15: Superposition of the measured total wall-to-wall strain at LaRonde and the squeezing index prediction.

At Lapa, Mercier-Langevin and Wilson (2013) noted that the predictions from the squeezing index for the competent volcanic and sedimentary rocks concur with their experience at the mine. The authors treated the three distinct geological domains separately and reported that when drifts are developed perpendicular to the ultramafic rock the squeezing index prediction (2.5% strain) is in agreement with underground observations. For drifts developed parallel to the foliation in ultramafic rock, the squeezing index accurately places them in the extreme category. Their squeezing predictions for Lapa, were based on observations and are shown in Figure 5.16.
Figure 5.16: Squeezing predictions for Lapa (Mercier-Langevin and Wilson, 2013).

Figure 5.17 shows the strain recorded at the Lapa case studies for sediment and ultramafic rocks. The upper, lower and mean values for the foliation spacing are used for each case study same as at LaRonde. In certain areas, the drifts were developed only in sediments or ultramafics. In these cases, the wall-to-wall strain estimated in section 5.2 was used in the graphs. For the drifts driven parallel to the foliation, most sections were developed between the sediment and ultramafic units. In these cases, the highest strain captured by the CMS surveys was used in the graphs. As the deformation in the sediment walls was insignificant compared to the one in the ultramafic walls, the strain used corresponds to the total deformation for these sections.

For the case studies with angle of interception less than 25°, the index correctly predicts that the strain can be more than 10% in ultramafics (low foliation spacing) and less than 5% in sediments (larger foliation spacing). Many points overlap in the ultramafics ellipse. These are cross sections from five longitudinal drifts in four mine levels. For an angle of interception between 25° and 40°, the index correctly predicted the low strain in sediments but did not capture the strain in excess of 10% recorded in the ultramafic walls.
Figure 5.17: Superposition of the measured strain at Lapa for ultramafic and sediment rocks and the squeezing index prediction.

The classification scheme used for the case studies is different from the one used in the index. This is due to the larger strain recorded in the field. LaRonde and Lapa Mines face some of the largest deformations encountered in hard rock mines. Figure 5.3 provides a better indication of the highest level of squeezing that may be expected for a given angle of interception. The index is based on case studies reported from several mining operations and calibrated based on in-situ observation from the LaRonde Mine (Mercier-Langevin & Hadjigeorgiou, 2011). Overall, the data validates the squeezing index. The higher strain recorded in certain case studies could be due to large induced stresses or/and high degree of alteration in these areas.
5.8 Deformation over time

5.8.1 Progress of squeezing during the drift development

The advancing face of a drift creates a complex stress path with the plastic zone generated in front of the face. The plastic zone around the drift and the elastic closure of the surrounding rock create a wall displacement profile as shown in Figure 5.18a. Figure 18b shows a case where the stress redistribution has caused rotation of the foliation planes in front of the face during the development of a haulage drift. The drift was developed at 2450 m at LaRonde and rotation of the foliation planes is visible on the face.

![Diagram of deformation profile](image)

**Figure 5.18:** a) The longitudinal displacement profile for an unsupported tunnel (Vlachopoulos & Diederichs, 2014); and b) rotation of foliation planes on the face at 2450 m depth at LaRonde.

Multiple CMS surveys in drift GM-51-44E captured the early deformation during the development of the drift. Figure 5.19 shows the geological units along the drift. The methodology described in section 5.2 was used to measure the displacement in the walls. An example cross section shows the profile of the excavation and the progress of the deformation during different time intervals up to 520 days after the development of the section. The deformation in the north wall was overall minor. This was due to the presence of sediments along the north side of the drift. The maximum measured displacement was 1.41 m in the sidewall and 0.77 m in the back and 0.86 m in the floor. This is used as a consistent reference point to illustrate the development of deformation over time as discussed subsequently in this section and in Figure 5.20.
Figure 5.19: Progress of squeezing over time as captured by CMS surveys at drift GM-51-44E at Lapa.

Figure 5.20 shows the displacement measured in the south wall and the back at different cross sections along the whole length of the drift. The initial dimensions of the drift were determined from the 3D surveys after the development of each section as described in section 5.2. These surveys are made by surveying one point at each wall after blasting and can introduce an error in the measurements. The high deformation rates resulted from the first measurement of some sections may be due to this error. In sections 2, 3, 10, 11, 19 and 20 the cavity monitoring surveys appear to be above the 3D survey. This could be due to errors in positioning the CMS in space or due to the low detail of the 3D survey. In these cases, the initial drift dimensions were measured from the first CMS survey. Despite the limitations, the measurements indicated that displacement initiates immediately after the development of the drift. The displacement rate is high, during the development of the drift and reduces over time.
Mercier-Langevin and Wilson (2013) in their interpretation of the squeezing mechanism at LaRonde and Lapa suggested that the displacement rate in the sidewalls is high early in the squeezing process while the back is mostly stable. The authors noted that later in the process, the displacement rate in the sidewalls decreases and the convergence in the back and floor accelerates. The data indicate that the displacement rate in the back is overall lower than the displacement rate in the sidewalls. However, the closure in the back and the sidewalls happens simultaneously and the displacement rates follow the same trends. The interpretation of the mechanism by Mercier-Langevin and Wilson (2013) was based in underground observations and was most likely influenced by the lower displacement observed in the back than the sidewalls. The field measurements from drift GM-51-44E show that the deformation in the south wall reached 0.4 m after approximately 50 days while, in most cases, the deformation in the back reached the same levels after 400 days.

The drift was excavated in approximately 77 days. Figure 5.21 shows the progress of the south wall displacement in several cross sections during the development of the drift. Part of the deformation recorded in this graph is a result of continued stress redistribution around each cross section during the development of the drift. The deformation continues after the development of the drift is over.
Figure 5.21: South wall displacement relative to the position of the face captured by CMS surveys during the development of drift GM-51-44E at Lapa.

5.8.2 Progress of deformation over time at LaRonde and Lapa

Figure 5.22 shows the highest measured wall-to-wall, and back-to-floor displacement in 147 case studies from the LaRonde and Lapa Mines. The highest measured displacement for each wall in 108 case studies from the LaRonde and Lapa Mines is presented in Figure 5.23. The graphs include the displacement rates from the measurements. The case studies are drawn from a range of squeezing ground conditions between 1790 m and 2690 m depth for LaRonde and between 540 and 950 m for Lapa. The displacement is widely scattered over time. Operational restrictions did not allow the continuous recording of deformation over time for each case study. The field data indicate that the displacement rate reduces over time and higher displacement and displacement rates are recorded in the sidewalls than the drift backs and floors. Higher displacement was recorded for the floor rather than the back in most case studies. This could be due to the lack of reinforcement or muck deposition on the floor. Overall, the displacement rates at Lapa are larger despite the greater depth at LaRonde. This is due to the presence of weakest rock units with lower foliation frequency and higher degree of alteration at Lapa than at LaRonde.
Figure 5.22: Wall-to-wall and back-to-floor displacement and displacement rate over time; a) LaRonde Mine and b) Lapa Mine.

Figure 5.23: Highest displacement and displacement rate over time for each wall; a) LaRonde Mine and b) Lapa Mine.
Underground measurements and observations have shown that the displacement continues at a low rate over the working life of the drifts. This suggests a time dependant deformation mechanism. Squeezing at LaRonde and Lapa is characterised by deformation due to ground stress redistribution during the development of the drift (associated with high initial deformation rates) and deformation caused by stress changes due to mining activity. The latter occurs after the development of the drift, and results in high deformation rates for a certain period. After a certain time the deformation rate decreases.

5.9 Conclusions

The LaRonde and Lapa Mines exhibit large-scale deformation in a range of ground conditions. This chapter investigated the interaction of the rock mass characteristics, mining and time on the resulted deformation in the drifts. The first step was to use the data collected in chapter 4 and quantify the observed convergence. The field data, indicated a range of squeezing ground conditions. The wall-to-wall and back-to-floor convergence were estimated and expressed as strain. The highest strain for each wall was also estimated. The data revealed wall-to-wall closure up to 2.5 m or wall-to-wall strain in excess of 40%.

A classification scheme was proposed. The estimated strain showed that acceptable squeezing levels in a mining environment are considerably higher than in civil engineering tunnelling operations. Although this allows more flexibility, mining operators have to work under greater economic constraints in terms of support. Squeezing ground conditions in mining applications often involve considerable failure of ground support and necessitate significant rehabilitation work. The estimation of the total sidewall and the total back-to-floor strain indicated the problems encountered in the functionality of the drifts and the need for rehabilitation work when pronounced and extreme squeezing conditions are faced.

The second step investigated the impact of rock mass characteristics, mining and time on the resulted deformation. The squeezing conditions at LaRonde and Lapa are influenced by a combination of parameters. A quantitative interpretation of the influence of the angle of interception between the drift and the foliation on the resulted deformation was presented. The choice of a favourable angle of interception can result in a more manageable squeezing level and increase the performance of an appropriate support system for squeezing ground conditions. At Lapa, the interpretation of the reported strain for each drive wall revealed the strong influence of the geology on the squeezing level. There was clear correlation between the weak ultramafic units with low foliation spacing and high degree of alteration and the large wall strain recorded in the case studies.

Subsequent adjacent mining activity can result in higher induced stresses and increase significantly the displacement. This was captured in the case studies from the CMS and the borehole extensometer data. The
depth of failure recorded from the borehole extensometers was up to 5 m in the walls and in excess of 5 m in the back. This is beyond the length of the primary support used at the mine.

The results from this study overall validate the squeezing index proposed by Mercier-Langevin and Hadjigeorgiou (2011). In certain case studies, with angle of interception higher than 25°, the measured strain was slightly higher than the one predicted by the index. This was probably due to large induced stresses or/and high degree of alteration in these areas that are not taken into account in the index.

The displacement rates recorded in the case studies at LaRonde and Lapa are higher early in the process and drop over time. Greater displacement, and displacement rates, were recorded in the walls than the back and the floor. Overall, larger displacement rates were recorded at Lapa. Closure in the back and the sidewalls occurs simultaneously and the displacement rates follow the same trends. The displacement rate in the back is overall lower than the one in the sidewall.
Chapter 6
Simulation of the buckling mechanism in hard rock mines using the distinct element method

6 Simulation of the buckling mechanism in hard rock mines using the distinct element method

6.1 Introduction

The study of the nonlinear anisotropic response of foliated rock masses around an underground opening, under the influence of high stress, requires the use of numerical modelling methods. Although continuum methods are used to simulate the rock mass behaviour around underground excavations, they are quite limited and do not fully capture the buckling mechanism observed in the field. Discontinuum numerical methods, even though more appropriate, have not been widely used due to long solution times and other computational restrictions.

This chapter provides a review of the numerical methods used to simulate squeezing ground conditions including continuum and discontinuum modelling approaches. The distinct element method is described. The chapter reports on the application of the distinct element method to capture the buckling mechanism in foliated ground under high stress conditions. The constructed numerical models focused on reproducing the mechanics of the failure mechanism in hard rock mining. They explicitly consider the influence of structure thus overcoming limitations of continuum models. Furthermore, they have addressed some of the time and computational constraints in running large discontinuum models. The numerical investigations are based on the case studies from LaRonde and Lapa presented in chapter 5. The impact of the ground conditions on the resulting deformation is also investigated using numerical modelling.

6.2 Numerical modeling methods for the simulation of squeezing ground conditions

Numerical modelling is a powerful tool for the simulation of the in-situ rock mass behaviour. The numerical methods can represent the rock mass either as a continuum or as a discontinuum medium. The most broadly continuum methods used are the boundary element method (BEM), the finite element method (FEM) and the finite difference method (FDM). The BEM methods such as Map3D and Examine 2D and 3D (Rocscience Inc, 2016b, 2016c) provide the capability of a fast 3D stress analysis but the analysis is usually limited to elastic. The FEM such as Phase² (Rocscience Inc, 2014) and the FDM such as FLAC and FLAC
3D (Itasca Consulting Group Inc, 2011a, 2012) are usually used to reproduce the plastic behaviour of the rock mass, However, they are not suitable for highly jointed rock and do not allow large displacement. The advantages and limitations of the numerical methods used for the analysis of rock failure around excavations have been discussed by Coggan et al. (2012) and are presented in Table 6.1.

### Table 6.1: Numerical methods for analysis of rock failure around excavations (Coggan et al., 2012)

<table>
<thead>
<tr>
<th>Analysis method</th>
<th>Input assumptions</th>
<th>Advantages</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Continuum : Boundary element</strong></td>
<td>Representative tunnel geometry, usually adopt simple constitutive criteria</td>
<td>Elastic analysis, capability of three dimensional modelling, rapid assessment of designs and stress concentrations</td>
<td>Normally elastic analysis only, (non-linear and time dependent options are available)</td>
</tr>
<tr>
<td><strong>Continuum: Finite-element and finite difference</strong></td>
<td>Representative tunnel geometry, wide range of constitutive criteria, including weakness plane, groundwater, shear strength/stiffness of discrete interfaces, in-situ stress, support properties</td>
<td>Allow for material deformation and failure, can model complex behaviour, capability of three-dimensional modelling, able to assess simulate both saturated and unsaturated (multiphase) flow/water pressures, recent advances in hardware mean that complicated models can now be PC-based and run in reasonable time periods, can incorporate coupled dynamic/groundwater analysis, suitable for soil, rock or mixed soil rock analysis, time dependent deformation readily simulated</td>
<td>Must be aware of model/software limitations including effects of mesh size, boundaries, symmetry and hardware restrictions (i.e. memory and time constraints) and data input limitations (such as effects of variation of critical input parameters etc.); simple structures can be simulated with interfaces, but not suitable for highly jointed-blocky media; well trained and experience users and familiarity with numerical analysis methods essential; validation through surface/subsurface instrumentation important</td>
</tr>
<tr>
<td><strong>Discontinuum: Discrete element</strong></td>
<td>Representative tunnel and discontinuity geometry, rock mass constitutive criteria, discontinuity shear strength and stiffness, groundwater, in-situ stress, support properties</td>
<td>Able to model complex behaviour; including both block deformation and relative movement of blocks (translation/rotation); three-dimensional models possible; effect of parameter variations on instability can be investigated easily; dynamic loading, creep and groundwater simulated; can incorporate synthetic rock masses to represent the fracture network; use of Voronoi polygonal blocks allows simulation of rock fracture between blocks</td>
<td>As above. Scale effects: simulate representative discontinuity geometry (spacing, persistence); limited data on joint stiffness available; predominantly used for jointed rock; validation through surface/subsurface instrumentation important</td>
</tr>
<tr>
<td><strong>Hybrid codes incorporating intact fracture capability (finite discrete element)</strong></td>
<td>As above. Use fracture mechanics criteria or particle flow code (parallel/shear bonds) to simulate intact rock fracture</td>
<td>Able to allow for extension of existing fractures and creation of new fractures through intact rock, capable of three dimensional modelling (although limited application to-date), can incorporate dynamic effects</td>
<td>Limited use and validation, state-of-the art codes requiring in-depth knowledge/experience of modelling methods/mechanics, must incorporate realistic rock fracture network, little data available for contact properties and fracture mechanics properties, limited capability to simulate effects of groundwater, extremely long run times will require use of parallel processing for large models</td>
</tr>
</tbody>
</table>
6.3 Continuum modeling approach

Material deformation in underground excavations in hard rock is often modelled using finite element and finite difference methods. These methods can provide a rapid design assessment in relatively fast solution times. The analysis can incorporate the effect of the support installation and the influence of the 3D development of an underground opening using 2D models (Vlachopoulos & Diederichs, 2014) or 3D models (Vakili et al., 2013). The main limitation is their ability to capture the buckling failure mechanism observed underground.

6.3.1 Implicit representation of the role of fractures

There is a plethora of documented modelling case studies in squeezing ground conditions referring to civil engineering structures. More specifically, previous studies using field deformation data from several tunnels in good quality rock concluded that it was possible to predict deformations based on elasto-plastic numerical models (Aydan et al., 1996; Kontogianni et al., 2004; Steiner, 1996).

Hoek (2001) simulated squeezing ground conditions using a 2D finite element program, Phase² assuming zero dilation of the rock mass. Considering different rock mass properties and in-situ stresses, he developed a ground response curve that can be used as a first estimate of tunnel squeezing problems, indicating the anticipated strain based on the ratio of the rock mass strength to in-situ stress. His analysis was based on the assumption that the rock mass was homogeneous without any specific structure and has an elastic perfectly plastic behaviour. It was argued that the numerical modelling results were appropriate as lower bound conditions for very poor quality rock masses.

The Geological Strength Index (GSI) (Hoek et al., 1995) is widely used as a preliminary estimate of input rock mass parameters for numerical models. For example, Barla et al. (2007) used FLAC to model squeezing conditions in a civil engineering tunnel. They used an elasto-plastic ideally plastic constitutive law with a subsequent transition to a viscoelastic-plastic behaviour intending to reproduce the time dependent behaviour of the rock mass. The strength parameters of the rock mass were derived using the GSI. The authors reported that the model was not able to describe the time-dependent response of the tunnel for long time period (more than 50 days).

The absence of geological structures in continuum elastic-plastic numerical models results in the consideration of squeezing rock only as ‘plastic flow’ of rock under the influence of high stress. This is not a valid assumption for the study of squeezing ground condition in hard rock mines. The reduction of the intact rock properties in continuum models using the GSI involves similar limitations under anisotropic
conditions. Although these models, after calibration may match the observed damage, they cannot capture
the observed squeezing mechanism driven by the presence of geological structures.

The use of ubiquitous joints for the modelling of foliated ground under high stress conditions has similar
limitations. Although the ubiquitous joint material model takes into account the presence of an orientation
of weakness, it fails to reproduce the buckling failure mechanism observed underground.

6.3.2 Explicit representation of the role of fractures

Sandy et al. (2007) noted that simple elastic models cannot predict the location and degree of stress induced
fractures but they can indicate slip in foliation. A FLAC3D model, with an explicit representation of the
foliation, predicted failure along the foliation indicated by separation of joints. It was not possible, however,
to identify the location and magnitude of stress induced fracturing. This was a conceptual model, and was
not compared to field data.

Some success in capturing large scale deformation has been reported by Beck et al. (2010) using a multi
scale approach with an explicit representation of the discontinuities. Each modelling scale was comprised
by a strain softening, dilatant, discontinuum finite element model. This model incorporated only tunnel
scale or larger discontinuities and discontinuities smaller than 1 m were homogenized.

A continuum Phase² model was used by Mercier-Langevin and Turcotte (2007a) to demonstrate the
resulting large deformations in some drifts at LaRonde. The model included an elastic-plastic material
behaviour with an explicit representation of the foliation. The authors identified a good correlation between
the resulting model and the observed failure patterns. Nevertheless, there was no discussion on the
magnitude of deformation and the model was not related to any particular drift. A more comprehensive
investigation using Phase² was undertaken by Mellies (2009) for the representation of failure patterns in
three drifts at LaRonde using an elastic-plastic model with a strain softening material behaviour. The
models indicated an agreement between the measured on site and the modelled deformation and were
characterized as plausible solutions of the recorded deformations, Figure 6.1. It was not possible however
to capture the buckling mechanism.
An explicit representation of foliation in continuum finite difference and finite element models, following extensive calibration can result in an improved representation of the squeezing mechanism in hard rock mines. Given that these methods do not allow any block rotation or relative movement of blocks, the mechanism is not reproduced.

### 6.4 Discontinuum modelling approach

Discontinuum discrete element methods have the ability to model the complex behaviour of rocks including block deformation and relative movement of blocks. Hsu et al. (2004) modelled squeezing conditions in a civil engineering tunnel constructed in interbedded sandstone and shale using UDEC (Itasca Consulting Group Inc, 2011c). They reported that the resulting failure shape correlated well with the actual failed profiles, identifying flexural tensile buckling of the interbedded formation at the beginning of the rock mass failure.

Goricki et al. (2005) studied the effects of the dominating discontinuities and the tunnel axis on the resulting displacements. A 2D conceptual UDEC model with different foliation dips showed that deformations arise from either slip or opening along the joints. A FLAC3D model with ubiquitous joints was used for parametric studies demonstrating the influence of discontinuity orientation on the resulting tunnel displacement.
Vakili et al. (2012) used 3DEC (Itasca Consulting Group Inc, 2013b) for a back analysis of the failure mechanism of a raise bored shaft developed in a foliated rock mass under high stress. The authors noted that 3DEC modelled the observed buckling mechanism. Figure 6.2 shows the modelled displacement and the time dependent depth of breakout based on in-situ observations. It is interesting to note that there is an inconsistency in the presentation of Figure 6.2, whereby the order in the depth of breakout seems to have been reversed. In this application, however, computational constraints did not allow the use of the 3DEC model to simulate the full length of the shaft. The discontinuum model was used to determine the continuum input parameters simulating triaxial testing. A continuum FLAC3D model was subsequently generated using a strain softening material model with ubiquitous joints and matched the observed damage.

Figure 6.2: 3DEC modelled displacement for a raise bored shaft in highly stressed foliated rock mass (Vakili et al., 2012).

The buckling mechanism and the block detachment observed in underground hard rock mines could be better reproduced using the distinct element programs UDEC and 3DEC. Nevertheless, there is a scarcity of field data to support this.

6.5 The discrete element method

The discrete element methods offer the capability of modelling the behaviour of discontinuous systems by representing the motion of multiple intersecting discontinuities explicitly. A discontinuous medium is comprised of multiple discrete bodies and contacts or interfaces between these bodies. A definition of the discrete element method was provided by Cundall and Hart (1992): “The name discrete element method should apply to a computer program only if it (a) allows finite displacements and rotations of discrete bodies, including complete detachment, and (b) recognizes new contacts automatically as the calculation
progresses.” The codes that use an explicit time-marching scheme to solve the equations of motion directly for deformable or rigid blocks with deformable contacts are called distinct element programs. The distinct element method was first created as a two dimensional representation of the jointed rock mass. Cundall (1988) reported on the development of a three dimensional version introducing the three dimensional distinct element code 3DEC.

6.5.1 3DEC

3DEC is a DEM code that simulates the behaviour of intact material between discontinuities as nonlinear continuum, allowing the interaction of separate blocks. The blocks can be either rigid or deformable. In deformable blocks, each polyhedral block is subdivided into internal finite difference mesh comprised of tetrahedral elements. Failure of the intact material can be simulated in this case. A contact between two blocks is discretized into sub-contacts where the interaction forces are applied (Itasca Consulting Group Inc, 2013a).

The code is based on a dynamic (time-domain) algorithm that solves the equations of motion of the block system by an explicit finite difference method. Sub-contact force displacement relations are set for both rigid and deformable blocks at each time step. The Newton’s second law (force = mass × acceleration) and the constitutive equations are applied. The integration of the law of motion gives the new positions of the blocks and the contact displacement (or velocities) increments (Itasca Consulting Group Inc, 2013a). The sub-contact force displacement law is then used to obtain the new sub contact forces. These forces are applied to the blocks in the next time step of the new cycle of mechanical calculations as shown in Figure 6.3.

Figure 6.3: 3DEC calculation cycle (Itasca Consulting Group Inc, 2013a).

The code uses a cell mapping method where the space containing the blocks is divided into rectangular 3D cells identifying neighboring blocks. All contacts are assumed to be soft. A contact detection scheme identifies the type of contact (e.g. face to face, face to edge etc.). The detection scheme identifies a normal
vector which defines the plane along which sliding can occur and the gap or contact overlap between blocks (Itasca Consulting Group Inc, 2013a). To achieve this, the code uses the idea of a common-plane. Based on that, a “common-plane” that bisects the space between two blocks is determined and each block is tested separately for contact with the common-plane. The scheme was described by Cundall (1988).

The contact forces are directly related to the deformations or overlaps at contacts. A normal force and a shear force component are used for each sub-contact. The constitutive laws that can be used for the contacts include an elastic model, a coulomb slip model considering both tensile and shear failure and a “continuously yielding” model which employs continuous functions for the force displacement relations. The block constitutive models available in the code include elastic, Mohr-Coulomb plasticity models and a bilinear strain-hardening/softening ubiquitous-joint plasticity models (Itasca Consulting Group Inc, 2013a).

The model is considered to be in equilibrium when the maximum unbalanced force is small compared to the representative forces in the problem (Itasca Consulting Group Inc, 2013a). Mechanical damping is used in the distinct element method. The equations of motion are damped to reach force equilibrium state as quickly as possible. In adoptive global damping (default damping), the viscosity constant for damping forces is continuously adjusted such that the power absorbed by damping is proportional to the rate of change of kinetic energy in the system. Local damping is also available where the damping force at a gridpoint is proportional to the magnitude of the unbalanced force with the sign set to ensure that vibrational modes are damped (Itasca Consulting Group Inc, 2013a).

6.6 Application of the distinct element method in foliated rock masses under high stress

The objective of this investigation was to employ the distinct element method to capture the observed buckling mechanism associated with squeezing in hard rock mines. The initial analysis using the 32-bit UDEC version 5.00.272 indicated the memory limitations of the program in modelling a dense foliation spacing. The subsequent use of the 64-bit 3DEC version 4.10.135 overcame the computational constraints. Finally, the presented work was undertaken using the multithreaded 3DEC version 5.00.164.

6.6.1 Applications at LaRonde

The in-situ measured foliation spacing in LaRonde varies between 0.018 m and 0.076 m. A lower foliation frequency was chosen in this analysis to avoid any increased running times. Figure 6.4 shows the conceptual model used. It was assumed that the drift was excavated parallel to the foliation at 2,270 m depth. The dip angle of the foliation was 80°. A foliation thickness of 0.2 m was chosen near the excavation and was
increased to 1.2 m 20 m away from the centre of the model. Horizontal construction joints with infinite strength and high stiffness were also added in the model to assist the block zoning process. The external model boundaries in the out of plane direction (x-axis) were fixed in the x direction. The 60 m long external boundaries in the y and z axes were fixed in all directions. Global dumping was used.

![Figure 6.4: Model geometry for 0.2 m foliation spacing. LaRonde model, after Karampinos et al. (2014).](image)

**6.6.1.1 Straight excavation method**

The material models used in this analysis were calibrated following a series of numerical tests and iterations. All formed blocks in the model were deformable and the drift was excavated in a single stage. An elastic constitutive behaviour was first used for all the blocks and the discontinuities. Subsequently, after the elastic model had reached equilibrium, a perfectly plastic Mohr Coulomb constitutive model was assigned to the blocks and the model was cycled further. The "area contact elastic/plastic with Coulomb slip failure" joint material model (Itasca Consulting Group Inc, 2013a) was used for the foliation. An elastic constitutive model was assigned to the horizontal joints at all times. The initial use of the elastic material model for all the blocks and joints was necessary to avoid the effect of a dynamic stress wave. This wave could cause failure in zones around the opening that would have not normally failed under the static stress caused by the excavation. A more extensive plastic zone, extending up to the external boundaries of the model was evident when this modelling stage was bypassed.
The elastic mechanical properties of the intact rock (intermediate tuff) were determined from uniaxial compressive strength tests (Turcotte, 2010). The plastic properties for the intact rock and the foliation were a result of a series of modelling iterations. Table 6.2 shows the mechanical properties that resulted in comparable squeezing levels to those recorded underground. The mechanical properties used in the numerical analysis resulted from a calibration process. A lower foliation frequency than the one recorded underground was used resulting in intact rock blocks that are less prone to buckling. The calibration process resulted in reduced material properties to attain a comparable squeezing level in the model with the one recorded underground.

Table 6.2: Material properties used in the numerical analysis. LaRonde model (Karampinos et al., 2015a).

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Input values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus (GPa)</td>
<td>48</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.16</td>
</tr>
<tr>
<td>Tensile strength of intact rock (MPa)</td>
<td>9</td>
</tr>
<tr>
<td>Friction angle of intact rock (°)</td>
<td>35°</td>
</tr>
<tr>
<td>Cohesion of intact rock (MPa)</td>
<td>12</td>
</tr>
<tr>
<td>Dilation angle (°)</td>
<td>0</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>2.81</td>
</tr>
<tr>
<td>Normal stiffness of foliation (GPa/m)</td>
<td>10</td>
</tr>
<tr>
<td>Shear stiffness of foliation (GPa/m)</td>
<td>1</td>
</tr>
<tr>
<td>Tensile strength of foliation (MPa)</td>
<td>0</td>
</tr>
<tr>
<td>Friction angle of foliation (°)</td>
<td>8</td>
</tr>
<tr>
<td>Cohesion of foliation (MPa)</td>
<td>0</td>
</tr>
<tr>
<td>Normal stiffness of construction joints (GPa/m)</td>
<td>100</td>
</tr>
<tr>
<td>Shear stiffness of construction joints (GPa/m)</td>
<td>10</td>
</tr>
</tbody>
</table>

The input parameters were arrived at following a comprehensive set of calibration trials. These parameters were successful in capturing the field observations for the model. As in every calibration exercise it is possible that a calibrated model can be developed based on a different set of input parameters. The parameters outlined in Table 6.2 were specific to the LaRonde mine. The values appear reasonable for the purposes of this investigation. The calibration process requires a trade-off of what appears to be ‘reasonable’ values and ratios (for example uniaxial compressive strength and $\sigma_0$) and managing to execute a successful stress analysis model overcoming any numerical instabilities.
The model managed to reproduce successfully the observed buckling mechanism as illustrated in Figure 6.5. The deformation in the sidewalls was 0.85 m (33% strain) which corresponds to pronounced squeezing based on the working classification presented in Chapter 5. This correlates well with the observed squeezing levels in LaRonde. For example, the drift at 2550 m depth shown in Figure 6.5 experienced a wall-to-wall convergence of 1.79 m 306 days after development.

**Figure 6.5: Modelled squeezing conditions in LaRonde (Karampinos et al., 2015a).**

The extent of the buckling zone is illustrated in Figure 6.6. Plasticity state indicators were used to illustrate the condition of the zones at every step. The zones may be in an elastic state, under shear failure (shear-n), sheared in the past (shear-p), under tensile failure (tension-n) or subjected to tensile failure in a past step (tension-p). A combination of these states can occur for each zone given the constant stress redistribution. The block state graph shows that most of the zones failed in shear. A tensile failure can also be observed in the sidewall around the opening. The extent of plastic zones around the opening indicates a direction of squeezing normal to the foliation planes. The extent of the joints slip reveals a similar pattern. In comparing these results with any continuum analysis, it was demonstrated that the distinct element model allowed for
a more refined consideration of the role of fractures within the rock mass including rotation of separate blocks, opening of fractures and detachment of blocks from their initial position.

Figure 6.6: Extent of plastic zone and joint slip in LaRonde (Karampinos et al., 2015a).

The normal and shear displacement of the joints are presented in Figure 6.7. The graphs indicate a dilation of the foliation planes in the sidewalls towards the opening and are in agreement with the model proposed by Mercier-Langevin and Wilson (2013). The extent of the dilation zone has a similar size and orientation with the buckling zone. Contraction in the back and the floor due to the stress redistribution after the excavation of the drift results in closure of the foliation planes. Figure 6.6 and Figure 6.7 show slipping of joints and shear displacement at the top of the hanging wall and the bottom of the foot wall. This is in agreement with the mechanism discussed by Beck and Sandy (2003) and Sandy et al. (2007) and observed underground at LaRonde and Lapa.

Figure 6.7: Joint normal and shear displacement in LaRonde (Karampinos et al., 2015a)
Although the numerical models captured the mechanics of the failure mechanism and reproduced similar deformation levels to those recorded underground, they are not calibrated to specific case studies. However, they can be particularly useful for the investigation of squeezing ground conditions under different conditions and to examine the effect of reinforcement.

A complete representation of the in-situ rock mass behaviour should take into account the influence of the reinforcement. The primary reinforcement around underground openings developed in squeezing ground conditions is often installed after significant deformation has already occurred in the excavation. In cases of large deformation, additional support is often required in later stages during the development of the drift. The study of the effect of different reinforcement strategies, therefore, requires a 3D model that can capture the effect of the advancement around an underground opening.

6.6.1.2 Pseudo-3D approach

Computational restrictions and time limitations did not allow modelling the development of a drift using a 3D model. The sequential excavation and advancement of a face, however, can be examined following a pseudo-3D approach. A review of the pseudo-3D methods commonly used to simulate the development of a tunnel has been provided by Vlachopoulos and Diederichs (2014), namely, straight excavation, field stress vector/average pressure reduction, excavation of concentric rings and face de-stressing (with or without softening).

As a first approach, the face de-stressing method with softening was examined at the 3DEC model. This was achieved by applying an elastic material model and zero stresses at the face. This approach was not successful as it resulted in modelling errors and unrealistic block overlaps due to the high block deformation.

A pressure reduction method was subsequently investigated. An algorithm was created to incrementally decrease the forces acting at the boundaries of the excavation by a reduction factor (r) through a series of modelling steps. The force reduction was made possible by applying a point load at all the grid points located at the boundaries of the excavation. The load had an opposite direction to the unbalanced forces at each grid point and a magnitude equal to the unbalanced forces multiplied by the reduction factor. Each modelling step included a stress analysis for a given value of the reduction factor from one to zero. A range of step numbers was used. It was observed that for a low number of steps the effect of a stress wave can increase the extent of the yielding zone and the resulting displacement as described in section 6.6.1.1.

Figure 6.8 shows the numerical modelling results and the progress of the deformation using this method. A total of 13 steps were employed, using the same model geometry as the straight excavation LaRonde model.
The material properties used were the same as in Table 6.2. For the case of \( r=1 \) the face is considered to be ahead of the modelled section and no displacement or failure appears in the model. As \( r \) reduces, the face approaches the modelled section and overpasses it. At the final step (\( r=0 \)) the position of the face has no influence on the modelled section.

This method successfully captured the actual progress of the deformation that was not possible with the straight excavation technique. The use of the progressive pressure reduction method showed the same extent of joint slip and plastic failure as the straight excavation method. However, the incremental stress redistribution using the progressive pressure reduction method resulted in a slightly higher displacement. In the absence of deformation data at the exact face location, it is difficult to determine the value of the reduction factor corresponding to the position of the face.

The model showed a joint slip initiation at the top of the hanging wall and the bottom of the foot wall. This is in agreement with the failure mechanism and the presence of shearing in reinforcement elements installed in these areas as reported by Sandy et al. (2007) in high stress Australian mines and observed underground at Canadian mines.

The direction and the magnitude of the maximum and minimum principal stresses for different values of the reduction factor are presented in Figure 6.9. The graphs show that the rotation of the maximum horizontal stress (\( \sigma_1 \)) in the sidewalls results in an axial load of the foliation planes as described by Mercier-Langevin and Wilson (2013). As buckling occurs, the process propagates further into the rock mass. The rock mass failure around the excavation results in a large zone of relaxation. At the early squeezing stages, there is a high stress concentration around the face that propagates further into the rock mass later in the process.
Figure 6.8: Numerical modelling results for different values of the reduction factor (r) and examples of squeezing conditions in LaRonde (Karampinos et al., 2015a).
Figure 6.9: Principal stresses redistribution for different values of the reduction factor ($r$). LaRonde model (Karampinos et al., 2015a).

A longitudinal displacement profile and a ground reaction curve can be obtained from this approach in a similar way with the convergence confinement method. Figure 6.10 displays the modelled longitudinal displacement profile (LDP) illustrating the progressive displacement for each wall and the ground reaction curve (GRC) demonstrating the reduction of the inner pressure (expressed as the reduction factor) and the displacement.
6.6.2 Applications at Lapa

More than one material was introduced to the model to study the different squeezing levels observed at Lapa for drifts developed in different geological units. The model comprised of both a weak and a strong material that represented the schist and sediment units respectively, Figure 6.11. The far field stresses assigned to the model were those of level 54 (540 m deep).

Field measurements suggest that the foliation spacing varies between 0.01 m and 0.5 m for the sediments and that it is lower than 10 mm for the schist. In the model, a 0.3 m foliation spacing was used for the sediments and a spacing of 75 mm was assigned to the schist due to computational constraints. The foliation was dipping at 85° to the north. The cases examined are shown in Figure 6.11; one with weak material in the foot wall; and one with weak material extending towards the back and the floor.
Figure 6.11: Examined geometry for drift developed in different geological units. Lapa model (Karampinos et al., 2015a).

The drift was excavated in a single stage following a similar methodology with the straight excavation LaRonde model. The elastic material properties for the intact rock were determined from uniaxial compressive strength tests (Dubuc et al., 2012). The plastic properties for the intact rock and the foliation were determined from a series of modelling iterations. Table 6.3 shows the mechanical properties that reproduced the displacement recorded in the foot wall at level 54 for the case where the weak material was present only in the foot wall.

Table 6.3: Material properties used in the numerical analysis. Lapa model (Karampinos et al., 2015a).

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Input values for strong material</th>
<th>Input values for weak material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus (GPa)</td>
<td>37.92</td>
<td>6.68 GPa</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.16</td>
<td>0.18</td>
</tr>
<tr>
<td>Tensile strength of intact rock (MPa)</td>
<td>9</td>
<td>7</td>
</tr>
<tr>
<td>Friction angle of intact rock (°)</td>
<td>35</td>
<td>25</td>
</tr>
<tr>
<td>Cohesion of intact rock (MPa)</td>
<td>23.9</td>
<td>6.37</td>
</tr>
<tr>
<td>Dilation angle (°)</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>2.81</td>
<td>2.81</td>
</tr>
<tr>
<td>Normal stiffness of foliation (GPa/m)</td>
<td>100</td>
<td>10</td>
</tr>
<tr>
<td>Shear stiffness of foliation (GPa/m)</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>Tensile strength of foliation (MPa)</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Friction angle of foliation (°)</td>
<td>15</td>
<td>8</td>
</tr>
<tr>
<td>Cohesion of foliation (MPa)</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Normal stiffness of construction joints (GPa/m)</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Shear stiffness of construction joints (GPa/m)</td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>
The resulting displacement from the models along with photos from representative case studies at level 54 in Lapa are shown in Figure 6.12. For the case study where the weak rock is present in the foot wall at 540 m depth, the measured displacement in the sidewall was 0.54 m 185 days after the development of the drift which correlates well with the 0.53 m modelled displacement. Although there is actually some weak material present in the back and the floor in most case studies, the models resulted in comparable squeezing profiles as illustrated in the photos. A similar buckling mechanism with the LaRonde model was observed. Nevertheless, squeezing was only present on the part of the drift located in the weak material. The case with the weak material extending to the back and the floor resulted in higher total displacement. This was in agreement with field observations.

Figure 6.12: Squeezing mechanisms in Lapa (Karampinos et al., 2015a).
6.7 Numerical investigation of the effect of ground conditions

6.7.1 Influence of foliation spacing

A parametric analysis indicated the effect of the foliation spacing on the degree of squeezing. The model geometry and the in-situ stresses presented in section 6.6.1 were used. A minimum number of two zones were introduced in each block formed between the foliation. Two sets of material properties were used. Material A had higher intact rock and joint properties than material B. Same as in section 6.6.1, reduced material properties were used to attain large deformation.

Table 6.4: Material properties used for the parametric analysis of the foliation spacing.

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Input values for relatively strong material (material A)</th>
<th>Input values for relatively weak material (material B)</th>
</tr>
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<tbody>
<tr>
<td>Young's modulus (GPa)</td>
<td>48</td>
<td>48</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.16</td>
<td>0.16</td>
</tr>
<tr>
<td>Tensile strength of intact rock (MPa)</td>
<td>20</td>
<td>12</td>
</tr>
<tr>
<td>Friction angle of intact rock (°)</td>
<td>38</td>
<td>35</td>
</tr>
<tr>
<td>Cohesion of intact rock (MPa)</td>
<td>23</td>
<td>16.9</td>
</tr>
<tr>
<td>Dilation angle (°)</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>2.81</td>
<td>2.81</td>
</tr>
<tr>
<td>Normal stiffness of foliation (GPa/m)</td>
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<td>40</td>
</tr>
<tr>
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<td>4</td>
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<td>Tensile strength of foliation (MPa)</td>
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<td>0</td>
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<tr>
<td>Friction angle of foliation (°)</td>
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<td>4</td>
</tr>
<tr>
<td>Cohesion of foliation (MPa)</td>
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<td>0</td>
</tr>
<tr>
<td>Normal stiffness of construction joints (GPa/m)</td>
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<td>100</td>
</tr>
<tr>
<td>Shear stiffness of construction joints (GPa/m)</td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>

Figure 6.13 shows the resulting displacement for material A in models with foliation spacing of 0.3 m, 0.2 m, 0.1 m and 0.075 m. The wall displacement was reduced from 0.75 m for the model with 0.075 foliation spacing to 0.1 m for the model with 0.3 m foliation spacing.
Figure 6.13: Variation in the displacement for foliation spacing between 0.075 m and 0.3 m.

Figure 6.14 shows the effect of the foliation spacing on the modelled wall-to-wall strain for the two materials examined. Lower strain appeared in models with higher foliation spacing for both materials. The wall-to-wall strain was relatively higher for the weakest material (material B) in all cases. The observed trend is in agreement with Euler’s analytical formula (Gere & Timoshenko, 1991) for the critical load for a column to buckle under axial load. The formula indicates that the critical load decreases as the thickness of the column decreases.
It should be clarified that the numerical models were not calibrated to specific case studies and did not include the effect of any support. Nevertheless, the resulting displacement in the models follows the same trends as described by the squeezing index and provides further confidence in its use.

### 6.7.2 Influence of the dip angle of the foliation

An increased model thickness is required to model a case when the strike of the foliation is not parallel to the direction of mining. A series of modelling tests indicated the computational and time constraints for an increased model thickness. The effect of the angle of interception on the modelled displacement was investigated instead by modifying the dip angle of the foliation.

The dip angle of the foliation measured at LaRonde and Lapa was between 90° and 70°. The model geometry presented in section 6.6.1 was modified to examine the displacement for a dip angle of 90°, 80° and 70°. The material properties used are shown in Table 6.5.

Figure 6.15 shows the variation in the displacement for the examined cases. Figure 6.16 shows the influence of the angle of interception on the modelled wall-to-wall strain. The results indicated a similar trend with the one observed in the field measurements presented in section 5.3. An increase in the angle of interception between the drift and the inherent foliation reduces the resulting level of squeezing. Limited field data were available for an angle of interception less than 10 degrees. The modelling investigation indicated that the strain rate decreases for an angle of interception less than 10°.
Table 6.5: The material properties used for the parametric analysis of the dip angle of the foliation.

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Input values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus (GPa)</td>
<td>48</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.16</td>
</tr>
<tr>
<td>Tensile strength of intact rock (MPa)</td>
<td>9</td>
</tr>
<tr>
<td>Friction angle of intact rock (°)</td>
<td>35</td>
</tr>
<tr>
<td>Cohesion of intact rock (MPa)</td>
<td>12</td>
</tr>
<tr>
<td>Dilation angle (°)</td>
<td>0</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>2.81</td>
</tr>
<tr>
<td>Normal stiffness of foliation (GPa/m)</td>
<td>15</td>
</tr>
<tr>
<td>Shear stiffness of foliation (GPa/m)</td>
<td>1.5</td>
</tr>
<tr>
<td>Tensile strength of foliation (MPa)</td>
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</tr>
<tr>
<td>Friction angle of foliation (°)</td>
<td>12</td>
</tr>
<tr>
<td>Cohesion of foliation (MPa)</td>
<td>0</td>
</tr>
<tr>
<td>Normal stiffness of construction joints (GPa/m)</td>
<td>100</td>
</tr>
<tr>
<td>Shear stiffness of construction joints (GPa/m)</td>
<td>10</td>
</tr>
</tbody>
</table>

Figure 6.15: Variation in the displacement for angle of interception between 0° and 20°.

Figure 6.16: Influence of the angle of interception on the modelled wall-to-wall strain.
6.7.3 Influence of stress conditions

The effect of the magnitude and the orientation of the in-situ stress conditions applied to the model was examined. The LaRonde model, presented in section 6.6.1 was used for this purpose.

6.7.3.1 Stress magnitude

In-situ stresses that corresponded to the far field stresses at 1820 m, 2270 m, 2690 m and 2900 m depth in LaRonde were applied to the model. The material properties used are shown in Table 6.6. The modelling results showed that the total displacement increases with depth, Figure 6.17.

Table 6.6: The material properties used for investigation of the effect of the in-situ stress magnitude.

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Input values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus (GPa)</td>
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</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.16</td>
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<tr>
<td>Tensile strength of intact rock (MPa)</td>
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<tr>
<td>Friction angle of intact rock (°)</td>
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<tr>
<td>Cohesion of intact rock (MPa)</td>
<td>12</td>
</tr>
<tr>
<td>Dilation angle (°)</td>
<td>0</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>2.81</td>
</tr>
<tr>
<td>Uniaxial compressive strength (MPa)</td>
<td>46.8</td>
</tr>
<tr>
<td>Normal stiffness of foliation (GPa/m)</td>
<td>30</td>
</tr>
<tr>
<td>Shear stiffness of foliation (GPa/m)</td>
<td>3</td>
</tr>
<tr>
<td>Tensile strength of foliation (MPa)</td>
<td>0</td>
</tr>
<tr>
<td>Friction angle of foliation (°)</td>
<td>8</td>
</tr>
<tr>
<td>Cohesion of foliation (MPa)</td>
<td>0</td>
</tr>
<tr>
<td>Normal stiffness of construction joints (GPa/m)</td>
<td>100</td>
</tr>
<tr>
<td>Shear stiffness of construction joints (GPa/m)</td>
<td>10</td>
</tr>
</tbody>
</table>
The modelled wall-to-wall strain and the back-to-floor strain for the examined stress conditions are shown in Figure 6.18. The graph includes the ratio of the initial maximum principal stress applied to the zones over the intact rock strength of the intact material. The results show that strain increases as the stress to strength ratio increases. The observed trend is in agreement with the squeezing index and contributes towards its validation and extension.
The effect of the magnitude of the initial minimum principal stress on the modelled displacement was examined separately. The vertical stress was reduced for the 2270 m depth case from 64 MPa to 40 MPa and 20 MPa. The material properties presented in Table 6.6 were used for this case. Figure 6.19 shows the resulted displacement in the models. There was no clear effect on the displacement when $\sigma_3$ was reduced to 40 MPa. When it was reduced to 20 MPa there was minor increase on the displacement in back the floor and the sidewalls.

**Figure 6.19:** Modelled displacement for reduced magnitude of $\sigma_3$.

### 6.7.3.2 Stress orientation

The initial in-situ stress conditions applied to the zones assumed that $\sigma_1$ was horizontal and $\sigma_3$ was vertical. The stress measurements in LaRonde suggest that the far field maximum principal stress is horizontal. The orientation of the induced stresses at drifts near mined stopes may vary. The initial in-situ stresses applied to the zones for 2690 m depth were rotated as shown in Figure 6.20.

**Figure 6.20:** Effect of rotated initial in-situ stresses on the modelled displacement.

Mercier-Langevin and Hadjigeorgiou (2011) reported that the in-situ stress orientation was found to be of relatively low importance for the buckling foliated rock that resulted in squeezing. This was based on the
fact that the stress redistribution results in axial loads on the foliation planes. The numerical modelling results show that the rotation of the stresses increases the axial load on the foliation planes extending the buckling mechanism farther into the walls. The initial in-situ stress orientation has an impact on the magnitude of the axial load applied on the foliation planes during the stress redistribution. In this particular localized model, it is observed that that there is larger displacement when the initial orientation of the maximum principal stress is subparallel to the foliation planes for this drive.

6.7.4 Correlation between modeled and recorded deformation

The straight excavation and the pressure reduction method reproduced the squeezing buckling mechanism observed in hard rock mines. For the same material properties, the final modelled displacement derived from the two methods was similar. The underground measurements and observations presented in Chapter 5 showed that the displacement continues over the working life of the drifts, suggesting a time dependant deformation mechanism. The model is calibrated to reproduce a similar magnitude of deformation recorded in drifts at the later squeezing stages. At that point in time, the initial high displacement rates have been reduced.

The progress of deformation was modelled using the pressure reduction method. The modelled displacement in this case is related to the displacement during the development of the drift as described in section 6.6.1.2. The deformation, however, continues after the development of the drift. A conceptual representation of the correlation between the recorded deformation over time at LaRonde and Lapa and the modelled deformation using the pressure reduction method is presented in Figure 6.21. The method captures the progress of squeezing during the development of the drift as presented in section 5.7.1. However the modelling steps cannot be directly linked to the actual steps of a 3D advancing face. In the absence of a time dependant constitutive equation in the model, it is not possible to relate directly the modelling steps with time. However, this methodology captures the progressive deformation and can allow the investigation of the effect of different reinforcement elements installed at different stages.
6.8 Summary and conclusions

The reproduction of the buckling mechanism observed in underground hard rock mines using numerical modelling requires the explicit representation of foliation. The distinct element method resulted in a better simulation of the role of fractures within the rock mass. The employed 3D distinct element model simulated the buckling mechanism allowing for block rotation, fracture opening or detachment of a rock block from its original domain that would not be feasible with any continuum modelling approach. The squeezing levels observed along drifts crossing different geological units were successfully reproduced following this methodology.
The trends derived from the examination of the impact of the foliation spacing and the stress conditions using the distinct element method are in agreement with the trends from the squeezing index and contribute towards its validation. The numerical investigations identified a correlation between the stress orientation and the displacement. When the maximum stress is sub-parallel to the foliation the axial load acting on the foliation planes is higher extending the buckling mechanism farther into the walls and increasing the modelled displacement.

The reproduction of the progressive deformation during the development of a drift was made possible with the progressive reduction of the forces acting at the boundaries of the excavation. The development of the plastic zone and the joint slip as well as the stress redistribution around the drift are in agreement with the interpretation of the failure mechanism reported in Australian and Canadian hard rock mines. This method is necessary for the investigation of the right support strategy in hard rock mining squeezing ground conditions.
Chapter 7
Investigation of the impact of different support strategies in squeezing

7 Investigation of the impact of different support strategies in squeezing

7.1 Introduction

This chapter presents the employed ground support strategy under squeezing conditions at the LaRonde and Lapa Mines. The effectiveness of different support strategies in controlling the large deformations at LaRonde has been first examined by Mercier-Langevin and Turcotte (2007a) and Turcotte (2010) and is summarized here. The advantages on the use of the hybrid bolt for squeezing conditions are discussed. This chapter provides further information on the effectiveness of different reinforcement elements based on field data collected at LaRonde. Monitoring data from borehole extensometers are used to examine the effectiveness of the current support strategy on the displacement. Results on the impact of the use of hybrid bolts on the required rehabilitation are presented.

7.2 Field behaviour of reinforcement elements at LaRonde

The failed zone in squeezing conditions can extend several meters behind the walls and the reinforcement is often entirely contained within the failed rock mass. The aim in these conditions is to provide some confinement within the broken rock mass and create a "reinforced shell". Stiff bolts cannot accommodate high deformation. Bolts that can yield and have ductile behaviour can be more effective. Ductility can be obtained by stretching the bolt in partially de-bonded bolts or by sliding between the tendon and the rock mass in friction bolts.

Some success on controlling the large deformation at LaRonde has been recorded by Mercier-Langevin and Turcotte (2007a) using FRS. The capacity of the FRS is, however, limited in shear. The bolts ‘lock up’ when buckling of foliation reaches a certain level as shown in Figure 7.1. The bolts usually fail at the collar, when further deformation occurs. Cemented grouted cable bolts were found too stiff to yield under load. Yielding cable bolts had limited success as they either eventually failed or sunk into the walls as the regular cemented cables. Cone bolts were locking up due to the shearing in the walls and were behaving very rigid like rebars. In these cases, either the plate tore, or the bolt snapped in its threaded section. The use of the
hybrid bolt has overcome most of these limitations (Mercier-Langevin & Turcotte, 2007a). For these reasons the mine moved away from the use of de-bonded cables and cone bolts.

Figure 7.1: Locking of support elements from internal wall shearing due to buckling (Mercier-Langevin & Turcotte, 2007a).

7.2.1 Description of the hybrid bolt

The hybrid bolt was developed in LaRonde and is comprised of a rebar installed in a FRS. The installation procedure is shown in Figure 7.2. Turcotte (2010) provided a description of the procedure:

- A 2.0 m friction bolt is installed with the resin already inside for preventing broken rock from entering the bolt and hampering the next steps. The friction bolt is installed with a plate to hold the screen.
- A 1.9 m rebar is installed inside the friction bolt and spun following the specific requirements of the resin used. The rebar has its own plate to prevent early failure of the friction bolt head.
- The rebar nut is tightened to push the plate against the head and plate of the friction bolt.

Figure 7.2: Installation procedure followed in LaRonde (Mercier-Langevin & Turcotte, 2007a).

The advantages of the hybrid bolts have been discussed by Mercier-Langevin and Turcotte (2007a). This setup prevents the resin from escaping in fractured ground. In addition, it increases the resistance to shear and the frictional resistance of the friction bolt. A stronger head to the bolt is also provided. In-situ and laboratory pull test have shown that it provides a stiff early reaction at low displacement and almost plastic
behaviour in high loads (15 to 20 tonnes). The hybrid bolt yields at a constant high load (140 to 180 kN) and has high resistance to shear (Turcotte, 2010).

7.3 Support strategy at LaRonde under squeezing conditions

The ground control standards for squeezing ground conditions at LaRonde, take into consideration the angle of interception between the foliation and the drift. For an angle greater than 45°, the support is comprised of Ø22 mm 1.9 m long rebars for the back and Ø39 mm 2 m long FRS in the walls, complemented with 4.1 mm galvanized weld mesh installed 1.8 m from the floor. For an angle of interception lower than 45°, alternate rows of 1.9 m and 2.3 m long rebars (Ø22 mm) are installed in the back and the sidewall support extents up to 0.6 m from the floor. Under these circumstances secondary support is installed in the sidewalls 12 m behind the face, comprised of hybrid bolts and zero gauge mesh straps, Figure 7.3. The hybrid bolts are installed at an offset height on the south wall in order to account for the expected deformation associated with the foliation.

Figure 7.3: Ground support standards for squeezing rock conditions at LaRonde, after Hadjigeorgiou et al. (2013).
Rehabilitation may be required after two months. The haulage drifts are cable bolted before mining the secondary stopes. Purging of the walls may be necessary when the drifts are narrower than 3.5 m for operational purposes. Cable bolts are used after purging of the walls occurs. Overall, the use of shotcrete at the mine is limited to holding together the broken rock mass during rehabilitation.

7.4 Support strategy at Lapa under squeezing conditions

The selection of the ground support at Lapa takes into account the complex geology at the mine and depends on the presence of ultramafics in or near the drift. Figure 7.4 presents the selection process used to define which support standard is used for any given excavation. In areas where no squeezing conditions are anticipated (DS and DM), rebars are used in the back and FRS in the sidewalls. Longer support is used in the ore drifts as they are subjected to higher stresses than the haulage drifts developed in waste.

Figure 7.4: Selection process for ground support systems at Lapa (Mercier-Langevin & Wilson, 2013).
In areas where squeezing conditions are likely to occur, when a drift is developed sub-parallel to the foliation and ultramafics are present within 5 metres of the excavation, the ground support was inspired by the success of the hybrid bolt at LaRonde. The mine employs three standards for areas where squeezing occurs. Longer support is used in the walls, and FRS bolts are upgraded to higher capacity hybrid bolts in the ultramafic walls susceptible to squeezing. In all ground support standards, screen (# 6 gauge, 4.1 mm diameter) is brought within 0.6 m from the floor to prevent unravelling of the lower part of the sidewalls. Figure 7.5 shows the ground support standard for the case when ultramafics are present in both walls and the back.

![Diagram](image)

**Figure 7.5:** Ground support standards for squeezing rock conditions at Lapa: hybrid bolts are installed in both north and south walls in the presence of ultramafic rock, after Hadjigeorgiou et al. (2013).

Cable bolting is used as a secondary support. The cables bolts are installed in the walls that are prone to squeezing with Ø8.3 mm mesh straps to strengthen the link between the surface support and the stiffer cables to prevent plates from punching through screen. The spacing used is 2 m with 8 m long cable bolts in the back and 6.4 m cable bolts in the walls. Mercier-Langevin and Wilson (2013) reported that the installation of the stiff cable bolts is delayed in time to allow the drift to deform first. When cable bolts are
installed early they either loaded to failure very quickly or “sunk” into the walls and broke the mesh, losing the connection with the surface support. The use of shotcrete is limited at the mine. It was found that using shotcrete, over long term, as deformation progresses the shotcrete breaks and needs to be supported.

Same as in LaRonde, when the width of the drifts becomes less than 3.5 m, the walls are purged to maintain the drifts operational. The excessive material is removed using a 6-yard scoop tram. This operation is carried out under the presence of the ground control personnel.

7.5 Investigation of the ground control practices at LaRonde based on field data

It was recognized that hybrid bolts cannot accommodate the high inward ground pressure if the bolt is installed too early in the process. Consequently, the mine has adopted a practice of delayed installation to allow for some ground deformation and reduction of the ground pressure without loading the primary support to failure. The installation of the hybrid bolt has a direct effect on the observed drift displacement. Figure 7.6 shows the displacement, and the displacement rate, in the north wall and back as captured by single height mechanical extensometers during the development of a drift at a depth of 2690 m. The instruments were installed approximately one week from the excavation of the section when some deformation had already occurred. Although the displacement continued after the installation of the hybrid bolts, the extensometers recorded a significant reduction of the displacement rate.
In places where the drifts become too narrow (less than 3.5 m wide), the walls are “purged” using a scoop. This process is conducted under the supervision of ground control personnel and is followed by the installation of cable bolts. A reduction of the purging distance after the introduction of the cable bolt in the support standard was reported by Turcotte (2010). Figure 7.7 presents an update of the cumulative length of purging below level 215. The graph shows a significant reduction on the purging distance per day after the introduction of the bolts. This trend was maintained even though that mining activity progressed deeper and the drifts were subjected to higher stresses and indicates the effectiveness of the hybrid bolt in controlling the deformation at the mine.

Figure 7.6: Recorded reduction of the deformation rate after the introduction of hybrid bolts at 2,690 m depth (Karampinos et al., 2015b).
Cable bolts are installed in haulage drifts prior to mining the secondary stopes. At most places ground support usually fails at the connection between reinforcement and surface support. A rehabilitation process, including the installation of additional reinforcement and surface support is necessary under these conditions to maintain the confinement in the rock mass. The use of shotcrete at the mine is limited in holding together broken rock mass, during rehabilitation.

Although rehabilitation may be required in certain cases, the ground control practice at LaRonde is considered to be effective given the extreme squeezing conditions encountered at the mine. The employed support strategy maintains the serviceability in the drifts and can sustain 40% strain or 2.5 m total wall displacement in places where rock fracturing extends up to 6 m into the rock mass.

The hybrid bolt is also used in areas susceptible to high seismicity. Cases where the hybrid bolt absorbs the load where the FRS has failed have been reported (Turcotte, 2010). The ground support used in seismic areas around the mine was described in detail by Turcotte (2014).

### 7.6 Conclusions

This chapter presented the ground support standards followed at the LaRonde and Lapa Mines. It discussed the effectiveness of different reinforcement elements used in the past at LaRonde and illustrated the effectiveness on the current strategy at the mine based on field data.

Both mines follow similar ground support strategies for developing excavations in squeezing ground conditions. In pronounced squeezing, the reinforcement elements are contained entirely within the failed rock mass. Under these conditions, the aim is to integrate the failed rock and provide some confinement.
within the broken rock. The support systems aim to control the extreme deformation rather than prevent it, which is not a realistic objective in a mining context.

The use of cement grouted cable bolts, yielding cable bolts and modified cone bolts has shown limited success in squeezing conditions at LaRonde as the bolts were either too stiff or did not yield enough by design. The mine has developed a ground control strategy that manages the extreme deformations rather than completely arresting them. The ground support standard for squeezing ground conditions calls for FRS in the walls and rebars in the back with screen. Hybrid bolts are installed as secondary support with mesh straps, 12 m behind the face. The haulage drifts are cable bolted after mining the secondary stopes. Rehabilitation may be required after a couple of months and when drifts become narrower than 3.5 m wide wall purging is necessary.

At Lapa the selection process for the ground support employed at the mine takes into account the presence of ultramafics relative to the drift. The mine employs three standards for areas where squeezing occurs. FRS are used in the walls and rebars in the back. The FRS are upgraded to hybrid bolts in the ultramafic walls. Cable bolts and mesh straps are used at the mine as secondary support strategy allowing for some deformation to occur prior installation. Same as in LaRonde, rehabilitation may be required for severe squeezing conditions and wall purging when the drifts become non-operational.

Monitoring results showed that the hybrid bolts have a direct effect on the displacement. The introduction of hybrid bolts, as part of the ground support strategy, has decreased the rehabilitation. When these bolts are installed as part of a secondary support strategy, they contribute to a significant reduction in drift convergence. Numerical modelling can provide a further insight on the effect of ground support under squeezing conditions in hard rock mines and contribute towards the optimization of the ground support minimizing the rehabilitation costs.
Chapter 8
Modelling of reinforcement strategies at LaRonde Mine

8 Modelling of reinforcement strategies at LaRonde Mine

8.1 Introduction

LaRonde provides a spectrum of squeezing conditions including some of the largest deformations recorded in hard rock mines. The reinforcement strategy at the mine is comprised of rebars, FRS, hybrid bolts and cable bolts, installed at different deformation stages. Field data from LaRonde presented in chapter 7 showed that the introduction of hybrid bolts as part of the ground support strategy has decreased the rehabilitation. Numerical modelling can provide further insight on the effectiveness of the reinforcement strategy used under these conditions. The numerical modelling methodology developed in Chapter 6 using the 3D DEM captured the observed buckling mechanism. This chapter explores the influence of reinforcement in the 3D discrete element method extending the previous numerical work.

The performance of grouted rebar rock bolts, FRS, hybrid bolts and cable bolts is first examined based on in-situ pull out tests from LaRonde. The results are compared with laboratory tests. The means for the representation of the reinforcement in the 3D DEM are presented. The use of the structural elements in the 3D DEM for squeezing conditions is explored indicating their advantages and limitations in capturing the behaviour of rock bolts under squeezing conditions. The structural elements were calibrated to simulate the behaviour of the different rock bolts in pull conditions as observed in the field.

A comprehensive strategy for modelling rock reinforcement in the 3D DEM is presented. There are three alternatives for the explicit representation of reinforcement in the 3D DEM. The first alternative would have been to construct a true 3D numerical model. In this investigation this was not a practical option for the reasons discussed in chapter 6 where it was decided to construct a pseudo-3D model. The second option implemented was to model a thin slice of a drift. The progressive reduction of the forces acting at the boundaries of the excavation in the model captured the deformation as observed in the field. This is presented in section 8.7. Finally, a third option was a variation of the pseudo-3D model whereby scaling the properties of the reinforcement elements was used to account for the distinct effect of elements in the out of plane direction. This is presented in section 8.8.
Different reinforcement scenarios were examined studying the effect of rebars, FRS, hybrid bolts and cable bolts. The numerical work took into account the sequential installation of reinforcement based on the sequence followed at LaRonde.

The influence of other reinforcement strategies was subsequently examined using the developed methodology. The impact of the early installation of cable bolts was investigated. The structural elements in 3DEC were calibrated to simulate the behaviour of the D-Bolt based on laboratory and in-situ pull out tests from LaRonde. The performance of D-Bolts under simulated squeezing conditions was then investigated.

### 8.2 Performance of reinforcement in laboratory and field tests

Reinforcement elements can be classified based on their performance as stiff or ductile and energy-absorbing rock bolts (Li, 2010). Stillborg (1994) investigated the performance of several rock bolts under pull loading across a simulated discontinuity in the laboratory using two blocks of reinforced concrete. Figure 8.1 presents the results from these tests, redrawn by Li et al. (2014) to include the concept of the energy-absorbing bolt type. Cement or resin grouted rebar bolts are characterised by a high load capacity but display stiff behaviour with limited deformation capability. On the other hand, ductile reinforcement elements, such as friction and expandable bolts can accommodate large deformations, albeit with relatively lower load capacity. Finally, energy-absorbing or yielding bolts such as the D-bolt, the cone bolt or the hybrid bolt can carry relatively high load, accommodate large displacement and absorb a large amount of energy before failure.

![Graph showing performance of different reinforcement elements](image)

**Figure 8.1:** Performance of different reinforcement elements under pull load in the laboratory (Stillborg, 1994), redrawn by Li et al. (2014).
Understanding and quantifying the behaviour of reinforcement elements is a prerequisite in selecting appropriate rock bolts for squeezing ground conditions. This is necessary, both for conceptual design, but also for acquiring the necessary input for analytical and numerical models.

The behaviour of different rock bolts has been examined through analytical studies (Dight, 1982; Li & Stillborg, 1999). Rock bolts installed in hard rock conditions are subjected to opening, shearing of rock fractures or a combination of the two (Li, 2010).

The behaviour of rock bolts can be captured by both in-situ and laboratory tests. Laboratory tests can be performed using a steel tube for direct pull tests at one end of the bolt or through a split tube to test across a simulated joint (Doucet & Voyzelle, 2012). Quantifying the behaviour of rock bolts in shear is a greater challenge. Chen and Li (2015b) has summarised different methodologies used to test the behaviour of bolts in shear across simulated joints. Stjern (1995) performed a series of tests on rock bolts under pure pull and pure shear load using two 0.95 m long cubic blocks of high strength concrete. Chen (2014) modified this test rig to apply both pull and shear load on bolts simultaneously and reported the results from a series of pull and shear tests and various combinations of the two in rebars and D-bolts.

Laboratory tests on bolts installed across an artificial discontinuity can represent the behaviour of bolts underground better than standard laboratory tests applying a load at one end of the bolt. However, they cannot reproduce the in-situ conditions and the rock mass state in the field.

There is limited laboratory testing done on rock bolts. Most of the tests available are done in concrete and is difficult to extrapolate them in field conditions. This thesis uses pull out tests from LaRonde to investigate the performance of rebars, FRS, hybrid bolts and cable bolts. While the tests do not fully capture the load application on the bolts under squeezing, they are the best available tests for the different rock bolts types. The advantage is that the pull tests correspond to the ground conditions that the rock bolts are employed in squeezing ground. The LaRonde Mine uses pull tests on a routine basis as part of a comprehensive quality control program. The derived information was used to identify the mechanical properties required to model the behaviour of the rock bolts.

The in-situ pull out test results provide information on the performance of each rock bolt type. The combined performance of the reinforcement elements used for squeezing conditions can be examined through field observations and monitoring. This was addressed in chapter 7.
8.2.1 Grouted rebar rock bolts

The anchoring mechanism in rebar is provided by the mechanical interlocking between the ribs on the bolt surface and the grout. Li et al. (2014) provided a theoretical model of the axial load on the bolt and the shear load on the interface when a rebar is subjected to pull load. They indicated that the maximum axial load is at the loading point and decreases with distance from that point. For an Ø20 mm rebar grouted with cement in pull out laboratory test, failure at the interface commences at the loading point and extends approximately 150 mm towards the end of the bolt. Under pull load, the rebar provides a stiff behaviour with high loading capacity. After the yielding load, the bolt deforms following a strain hardening behaviour. The bolt failed in tension in the bolt shank at the joint at 40 mm displacement. A similar behaviour was observed for a displacing angle of 20°, 40° and 60° (Chen & Li, 2015b). Lower stiffness was observed when the bolt was subjected to pure shear due to crushing on the grout around the bolt. Figure 8.2 shows the load-displacement curves from these tests done by pulling two concrete blocks apart.

Figure 8.2: Load-displacement curves of cemented grouted rebar in a) pure pull; b) pure shear (Stjern, 1995); and c) different displacing angles (Chen & Li, 2015b).

Pull out tests from LaRonde on Ø22 mm, 1.9 m long fully resin grouted rebars presented in Figure 8.3, indicated a stiff behaviour of the rock bolt. It was noted that in certain tests, instead of the bolt deforming, there was deformation on the rock mass around the bolt. This resulted in larger recorded deformation. In-situ pull out tests do not load the rebar to failure. Given the setup of the test, however, the bolt is more likely to fail near the thread which constitutes the weakest part of the rebar.
8.2.2 Friction rock stabilizers (FRS)

Under pull load, the strength of the FRS is mainly controlled by the friction at the interface between the bolt and the rock along the entire length of the bolt. The shear strength is first reached at the loading point. When this happens, the bolt starts to slide and the shear stress at the interface between the bolt and the rock along the bolt is approximately the same magnitude as the strength of the interface. Stjern (1995) reported on pure shear test of a 46 mm FRS installed in two cubic concrete blocks. The bolt failed at the plate and broke at the shank of the joint formed between the blocks. Figure 8.4 shows load-displacement curves for FRS subjected to pull and shear loading in laboratory tests performed by Stjern (1995). The graphs indicate that under shear load the strength of the bolt is not controlled by the interface between the bolt and the concrete but it fails in the plate and the bolt shank. The different failure mechanism under shear load, allows the bolt to accommodate higher load than under pull load.

Figure 8.3: Pull out tests at LaRonde in 1.9 m long, 22 mm diameter rebars.

Figure 8.4: Load-displacement curves for FRS subjected to; a) pull; and b) shear load (Stjern, 1995). The symbols “o” and “x” refer to failure in the plate and bolt shank respectively.
A series of in-situ pull out tests have been performed at LaRonde in 39 mm and 35 mm, 2 m long FRS, and are presented in Figure 8.5. The bolts were installed in boreholes drilled with different drill bit sizes. The results showed no clear correlation between the drill bit size and the overall performance of the bolt. The variation on the measured capacity is related with the rock mass quality. There is lower confinement on the bolt surface when the bolt is installed in broken rock. Shearing in the rock can “lock” the bolts and prevent them from sliding (Mercier-Langevin & Turcotte, 2007a). In this case, the bolt can fail in tension. Failure in the plate may also occur.

Figure 8.5: Pull out tests results at LaRonde in 2 m long, 35 mm and 39 mm FRS.

8.2.3 Hybrid bolt

The hybrid bolt is a rebar installed inside a friction bolt. A detailed description of the bolt has been provided in section 7.2.1. Under pull load, the hybrid bolt slides in the hole in a similar way as the FRS. A failure is observed at the interface between the bolt and the rock. Laboratory pull out tests showed that it provides larger shear strength than the FRS with high deformation capability (Mercier-Langevin & Turcotte, 2007a). Field pull out tests at LaRonde showed that the load increases drastically in the first 5 to 10 mm of movement, after which the friction bolt begins to slide at a steady rate, Figure 8.6. The strength varied from 140 to 180 kN (Mercier-Langevin & Turcotte, 2007a). Three bolts failed at the threaded section with one failed at the shear pin. The variation of the stiffness and the strength of the bolt could be related to changes in the quality of the rock mass and in the confinement acting on the bolt annulus. More recent in-situ pull out tests at the mine indicated similar bolt stiffness. While the bolts is expected to slide after the shear strength at the interface between the rock and the bolt is reached, these tests aimed to examine whether the bolts can accommodate sufficient load in-situ and were stopped before this load was reached. The axial load, under in-situ pull out tests, is applied only on the rebar grouted in the FRS for operational reasons.
This setup resulted in failure of the rebar on the head in certain tests. Under squeezing conditions, however, the load is applied to the head of the FRS and the rebar increasing the capacity of the bolt.

Figure 8.6: Performance of the hybrid bolt; a) underground pull out tests (Mercier-Langevin & Turcotte, 2007a); and b) recent pull out tests.

8.2.4 Cable bolts

A plethora of pull out tests on cable bolts is available in the literature (Hutchinson & Diederichs, 1996; Stjern, 1995). Figure 8.7 shows pull out and shear tests from a twin strand cable of 2 x Ø12.7 mm cement grouted in a borehole reported by Stjern (1995). The embedment length was 0.95 m. Under pull loading, the cable yielded at 170 kN after 25 mm of displacement due to the dilation effect of the grout strand interface. The 380 kN tensile strength of the bolt was not reached. The test was terminated at 250 mm of displacement. Under shear loading, the wires of the cable failed at 60% of the tensile strength of the cable with a shear displacement of 134 mm. Hyett et al. (1992) conducted a series of laboratory and field tests indicating that the bond strength is influenced by the embedment length, the cement ratio and the radial confinement on the borehole.

Figure 8.7: Load-displacement curves of twin strand under pull and shear loads (Stjern, 1995).
LaRonde uses 15.9 mm bulbed strand cables installed in Ø64 mm boreholes. Two cables per hole are installed in the back with one cable only being platted. One cable per hole is installed in the sidewalls. No specific tests were undertaken for the purposes of this thesis. It was possible to have access to pull tests conducted by the mine on bulbed strand cables, Figure 8.8. The embedment length used was 1.8 m. The tests aimed to examine whether the bolts can accommodate sufficient load in-situ and did not load the bolts to failure.

![Figure 8.8: Pull out tests at LaRonde for cable bolts.](image)

8.3 Representation of reinforcement in 3D discrete element models

The introduction of reinforcement in numerical models is not a trivial exercise. Numerical models have been relatively successful in reproducing the results of laboratory tests by explicit modelling of the borehole, the grout and the bolt steel. Chen and Li (2015a) used FLAC3D (Itasca Consulting Group Inc, 2012) to model the performance of rebars and D-bolts under combined pull and shear load using a tri-linear material model for the bolt steel and defining different interfaces to simulate the different bonding mechanisms. The modelling results were in good agreement with the experimental results of the recorded axial load in the bolt and the shear stress along the bolt–grout interface. Grasselli (2005) used a 3D finite element code to model explicitly shear tests on expandable bolts and fully grouted rods capturing the dissimilar response of the two structural elements under shear.

8.3.1 Structural elements in 3D discrete element method

The mechanical behaviour of reinforcement can be simulated in numerical modelling using structural elements. The structural elements available in the 3D distinct element method are the local and global reinforcement elements (Itasca Consulting Group Inc, 2013a).
8.3.1.1 Local reinforcement elements

The local reinforcement elements consider only the local effect of reinforcement when it intersects fractures. They are commonly used to simulate rock bolts in rigid blocks. Figure 8.9 shows the formulation of the local reinforcement element (Itasca Consulting Group Inc, 2013a). An active length is defined for the formulation simulating the short length along the reinforcement that changes orientation during shear displacement. The element can be described as an axial and a shear spring located parallel and perpendicular to the discontinuity interface. It is assumed that the active length changes orientation as a geometric result of shear and normal displacement. Following shear displacement the axial spring is oriented parallel to the active length while the orientation of the shear spring remains the same. The forces developed from the displacements are applied at the endpoints of the active length. The axial and shear forces are resolved into components parallel and perpendicular to the discontinuity. Limits can be set for the shear and the axial forces. Strain limits can be also assigned.

\[
\Delta F_a = K_a |\Delta U_a| \quad (8.1)
\]

Where \( \Delta F_a \) is the incremental change in axial force, \( \Delta U_a \) is the incremental change in axial displacement and \( K_a \) is the axial stiffness. The force displacement relation for shear response is given by Equation 8.2.

\[
\Delta F_s = K_s |\Delta U_s| \quad (8.2)
\]

Figure 8.9: Local reinforcement model; a) geometry after shear displacement; and b) orientation of the shear and axial springs prior and after shear displacement (Itasca Consulting Group Inc, 2013a).
Where $\Delta F_s$ is the incremental change in shear force, $\Delta U_s$ is the incremental change in shear displacement and $K_s$ is the shear stiffness. The mechanical behaviour of the local reinforcement elements in axial and shear load is shown in Figure 8.10. The forces developed from the incremental displacements are applied at the endpoints of the active length. The axial and shear forces are resolved into components parallel and perpendicular to the discontinuity.

Figure 8.10: Mechanical behaviour of local reinforcement elements; a) axial load; b) shear load; c) resolution of axial force into components; and d) resolution of shear force into components (Itasca Consulting Group Inc, 2013a).

8.3.1.2 Global reinforcement elements

The global reinforcement elements take into account that the intact rock may experience inelastic deformations in which the bonding agent along the reinforcement length may fail in shear. They allow the modelling of shearing resistance as provided by the bond between the grout and either the reinforcement element or the rock. The mechanical behaviour of the global reinforcement elements is summarized in Figure 8.11. The reinforcement element is assumed to be divided into a number of segments with nodal
points located at each segment end. Each structural node is associated with a finite difference zone for the calculation of shear forces between the reinforcement element and the zones. The mass of each segment is lumped at the nodal points. The axial stiffness is a function of the cross sectional area and the Young’s modulus of the steel. A tensile strength and a strain limit can be assigned to the element. The shear behaviour is represented as a spring-slider system between the nodal points. A limit can be set at the maximum shear force that can be developed per length of element.

Figure 8.11: Conceptual mechanical representation of the global reinforcement elements (Itasca Consulting Group Inc, 2013a).

The axial stiffness is described in terms of the reinforcement cross sectional area $A$ and the Young’s modulus, $E$. The incremental axial force is given by Equation 8.3.

$$
\Delta F^t = -\frac{EA}{L} \Delta u^t
$$

(8.3)

Where $\Delta u^t$ corresponds to the displacements at the structural nodes associated with each bolt element. A tensile yield force can be assigned to the bolt so forces greater than the tensile limit will not be developed. A strain limit can also be used.

The shear behaviour is represented as a spring-slider system between the nodal points. The shear behaviour of the grout during relative displacement between the reinforcement/grout and the grout/rock interfaces is described numerically by Equation 8.4.

$$
\frac{F_s}{L} = K_{bond}(u_c - u_m)
$$

(8.4)

Where: $K_{bond}$ is the grout shear stiffness;

$F_s$ is the shear force in the grout;
$L$ is the element length;

$u_c$ is the axial displacement of the bolt; and

$u_m$ is the axial displacement of the medium (rock).

The maximum shear force that can be developed per length of element is limited to the cohesive strength of the bond. The mechanical behaviour of the cable and the grout material are shown in Figure 8.12. The stiffness of the grout can be estimated from pull out tests.

![Mechanical behaviour of a) the bolt and b) the bond](image)

Figure 8.12: Mechanical behaviour of a) the bolt and b) the bond (Itasca Consulting Group Inc, 2013a).

### 8.4 Use of structural elements in 3D DEM for squeezing conditions

A series of numerical tests were performed to examine whether reinforcement can be introduced in the 3D DEM for squeezing conditions and assess the performance of the structural elements under these conditions. The global and the local reinforcement elements were introduced to the 3D DEM. The pressure reduction method was used to introduce the structural elements at different deformation stages based on the sequence followed at LaRonde. The geometry of the model and the mechanical properties used were those presented in section 6.6.1.

#### 8.4.1 Use of global reinforcement elements

Global reinforcement elements were used to simulate the mechanical behaviour of rebars, FRS, hybrid bolts and cable bolts. The material properties assigned to the global reinforcement elements were based on the pull out test results presented in Section 8.2 and are summarized in Table 8.1. The area of the FRS was derived from the effective cross sectional area of the bolt annulus. The area chosen for the hybrid bolt was
the sum of the rebar and the FRS areas. The bond stiffness assigned to each bolt was based on the in-situ pull out test results from LaRonde. The bond strength assigned to the FRS was 70 kN. The bond strength for the rebars and the hybrid bolts was 160 kN. Ten times higher bond strength was assigned to the first node of each global reinforcement element to simulate the effect of a plate on each bolt. The bond stiffness of the cable bolts was estimated from Figure 8.8. The empirical formula used (Itasca Consulting Group Inc, 2013a), to estimate the cohesive strength of the bond neglects any frictional confinement effects. A perfect bonding was assumed using high values and preventing any sliding on the cables.

Table 8.1: Material properties used for the global reinforcement elements.

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>FRS (sidewalls)</th>
<th>Rebars (back)</th>
<th>Rebars (back)</th>
<th>Hybrid Bolts (sidewalls)</th>
<th>Cables (sidewalls)</th>
<th>Cables (back)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (m)</td>
<td>2</td>
<td>2.3</td>
<td>1.9</td>
<td>2</td>
<td>6.4</td>
<td>8</td>
</tr>
<tr>
<td>Area (m²)</td>
<td>2.71E-4</td>
<td>3.8E-4</td>
<td>3.8E-4</td>
<td>6.51e-4</td>
<td>1.99E-4</td>
<td>1.99E-4</td>
</tr>
<tr>
<td>Young's modulus (Pa)</td>
<td>200E9</td>
<td>200E9</td>
<td>200E9</td>
<td>200E9</td>
<td>200E9</td>
<td>200E9</td>
</tr>
<tr>
<td>Tensile yield strength (N)</td>
<td>150E3</td>
<td>250E3</td>
<td>250E3</td>
<td>420E3</td>
<td>275E3</td>
<td>275E3</td>
</tr>
<tr>
<td>Bond stiffness (N/m/m)</td>
<td>2.33E6</td>
<td>9.1E6</td>
<td>11.1E6</td>
<td>6.4E6</td>
<td>4.17E6</td>
<td>5.56E6</td>
</tr>
<tr>
<td>Cohesive strength of bond (N/m)</td>
<td>35E3</td>
<td>72.7E3</td>
<td>88.9E3</td>
<td>80E3</td>
<td>3,316E3</td>
<td>3,326E3</td>
</tr>
</tbody>
</table>

The global reinforcement elements were introduced to the model at different deformation stages to simulate the installation sequence used at LaRonde. In case 1, rebars were installed in the back and FRS in the sidewalls after the numerical equilibrium at stage r=0.45. Hybrid bolts were subsequently installed after the numerical equilibrium at r=0.2 and the reduction factor was reduced up to zero. In case 2, cables were introduced to the model after the numerical equilibrium at r=0.1 in case 1. The displacement at the stage when each reinforcement was installed is shown in Figure 8.13.
Figure 8.13: Modelled displacement at installation stage of each bolt type for cases 1 and 2.

Figure 8.14 shows the performance of the rock bolts at the final modelling stage (r=0) for cases 1 and 2. The Figures in this chapter are reproduced at larger size for improved clarity in Appendix A. Higher load was applied to the hybrid bolts than the FRS in both cases. The axial load applied to the cables was near the 261 kN ultimate strength assigned to the cable bolt steel. The bond strength was exceeded at the rebars, the FRS and the hybrid bolts. This allowed the FRS and the rebars to slide inside the rock mass as the deformation was increasing. The high bond strength assigned to the cables did not allow any bond failure. This is partially attributed to the fact that they were installed at a later stage in the process. Lower overall displacement was recorded in the model when cables were used.

Figure 8.14: Modelled block displacement and performance of the global reinforcement elements for a) case 1 and b) case 2.
The plastic behaviour of the rebars in case 1 and 2 was controlled by the shear bond at the interface between the bolt and the blocks. The bolts slid after the bond strength was exceeded. This mechanism cannot allow any rupture in the bolt as captured in pull out tests presented in section 8.2.1. The global reinforcement elements cannot capture the localized shear on the reinforcement elements. The bolts installed in the sidewalls are subjected to a combined pull and shear load in the field. This is discussed further in section 8.4.3 along with the performance of the global reinforcement elements under combined pull and shear load.

### 8.4.2 Combined use of global and the local reinforcement elements

The global reinforcement elements did not provide any direct shear resistance at block interfaces when crossed by rock bolts at the discontinuities. The combined use of global and local reinforcement was examined to introduce direct shear at the interfaces crossed by FRS, rebars and hybrid bolts. There are not any shear tests available for the rock bolts used at LaRonde. Therefore, different shear properties were assigned to the local reinforcement elements in a series of case studies. In case 3, the shear properties used were based on the shear test results by Stjern (1995) as presented in section 8.2 for each bolt type. In case 4, the shear stiffness and shear strength assigned to each bolt were equal to the axial stiffness and strength used for the global reinforcement elements. This was an initial assumption to initiate the calibration process. These values were revised considerably during the calibration. In both cases it was assumed that the axial behaviour of the rock bolts was controlled by the global reinforcement elements. Consequently, no axial material properties were assigned to the local reinforcement. The material properties used for the local reinforcement elements for cases 3 and 4 are summarized in Table 8.2.

**Table 8.2: Material properties used for the local reinforcement elements in cases 3 and 4.**

<table>
<thead>
<tr>
<th></th>
<th>FRS (sidewalls)</th>
<th>Rebars (back)</th>
<th>Rebars (back)</th>
<th>Hybrid Bolts (sidewalls)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Case 3</td>
<td>Case 4</td>
<td>Case 3</td>
<td>Case 4</td>
</tr>
<tr>
<td>Length (m)</td>
<td>2</td>
<td>2</td>
<td>2.3</td>
<td>2.3</td>
</tr>
<tr>
<td>Area (m²)</td>
<td>2.71E-4</td>
<td>2.71E-4</td>
<td>3.8E-4</td>
<td>3.8E-4</td>
</tr>
<tr>
<td>1/2 active length (m)</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>Axial Stiffness (N/m)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Axial rupture strain</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Ultimate axial capacity (N)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Shear Stiffness (N/m)</td>
<td>2.35</td>
<td>4.7E6</td>
<td>4E6</td>
<td>20E6</td>
</tr>
<tr>
<td>Shear rupture strain</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Ultimate shear capacity (N)</td>
<td>160E3</td>
<td>160E3</td>
<td>160E3</td>
<td>160E3</td>
</tr>
</tbody>
</table>
The global reinforcement elements were introduced to the model at the same modelling stage as in case 1. It was found that it was not possible to install the local reinforcement after significant deformation had already occurred in the model. The local reinforcement element were introduced to the model from the first modelling stage using material properties that did not influence the deformation. The material properties were then modified as per Table 8.2 to simulate the installation of the rock bolts.

The modelling results for cases 3 and 4 are presented in Figure 8.15 and Figure 8.16 respectively. The plots present the performance of the global and the local reinforcement elements separately. Overall, the behaviour of global reinforcement elements was similar with case 1. Low shear force was applied to the local reinforcement elements representing the rebars in the back and the FRS in the sidewalls. Higher shear force was applied to the hybrid bolts with the highest values recorded at the top of the hanging wall and the bottom of the foot wall. This is in agreement with the presence of shearing in reinforcement elements installed in these areas as reported by Sandy et al. (2007) in deep and high stress Australian mines and observed underground at Canadian mines.

![Figure 8.15: Performance of the structural elements (global and local reinforcement elements) for case 3.](image)
Figure 8.16: Performance of the structural elements (global and local reinforcement elements) for case 4.

When the shear properties in the local reinforcement were based on the tests by Stjern (1995), in case 3, the reduction on the displacement is minor compared to case 1. There was slightly larger reduction on the displacement when shear properties equal to the axial properties were used for each reinforcement element (case 4).

8.4.3 Assessment of the use of structural elements in 3D DEM for squeezing conditions

The main advantage of the local reinforcement elements is that they can take into account the bending resistance of the structural element. However, they consider only the local effect of reinforcement when it passes through discontinuities. They are most applicable in cases when the deformation of individual blocks can be neglected in comparison with the deformation of the reinforcing element. This cannot be a valid assumption for squeezing conditions in hard rock mines. The simulation of the squeezing mechanism presented in section 5, indicated that the resulting deformation is a combination of block deformation and shearing and opening of discontinuities.

Global reinforcement elements can take into account the presence of reinforcement along their entire length throughout the rock mass and can simulate bonding (grout) failure. Unlike local reinforcement elements, they consider the deformation of individual blocks with respect to the deformation of the reinforcing element. This is a significant advantage, resulting in an improved simulation of the role of reinforcement under squeezing ground conditions in hard rock mines. Nevertheless, they do not provide any direct bending resistance along the block interface when intersecting fractures. Given the present limitations of the reinforcement elements available in 3D distinct element method, the effect of both global and local...
reinforcement on the squeezing mechanism and the modelled deformation was examined. The combined use of the global and local reinforcement elements could take into account both the direct shear resistance at an interface when crossed by a rock bolt and the deformation of blocks relative to the rock bolt. This approach was examined through a series of modelling tests and revealed several limitations.

The numerical tests revealed that it was not possible to install the local reinforcement elements at later modelling stages, after significant deformation had already occurred in the model. To overcome this limitation, the elements were introduced in the model from the first modelling stage. Material properties that did not influence the block deformation were initially assigned to the local reinforcement elements and were modified at later modelling stages to simulate the installation of rock bolts. This approach, however, results in changes of the angle $\theta$ between the active length and the reinforcement, as shown in Figure 8.10, before the actual rock bolt properties were assigned to the element. The shear force component applied to the end of the active length is higher for a larger $\theta$ and lower for a smaller $\theta$. Consequently, the resulting shear resistance applied on the blocks is overestimated or underestimated depending on the direction of shearing relative to the reinforcement. In addition, for bolts that are not installed perpendicular to the discontinuities, the presence of shearing in the foliation planes triggers a normal force component. This force may have a direction towards or away the discontinuity depending on the direction of shearing relative to the discontinuity.

Given the limitations of local reinforcement elements, the behaviour of the global reinforcement elements in the presence of shear in the modelled squeezing conditions was investigated. Figure 8.17 shows the final joint normal and shear displacement for the LaRonde model presented in section 6.6.1. The largest joint shear displacement appeared near the corners is approximately 0.06 m. At these areas, the normal displacement is approximately 0.025 m. The displacing angle ($\alpha$) for a bolt installed perpendicular to a joint is given by Equation 8.7:

$$\alpha = \arctan\left(\frac{\text{joint shear displacement}}{\text{joint normal displacement}}\right)$$

(8.7)

The maximum displacing angle at the last modelling stage ($r=0$) for bolts installed perpendicular to the foliation would be approximately 60°. In practice, it is not always possible to install the rock bolts perpendicular to the foliation. In addition, as squeezing progresses, the displacement angle changes continuously. Figure 8.17 suggests that the reinforcement is subjected to a combined pull and shear displacement.
The global reinforcement elements cannot capture the doweling effect of bolts under pure shear. A numerical experiment was conducted to examine the behaviour of the elements for displacing angles between 0° and 60°. This was achieved by applying a velocity boundary condition to a block bonded at the last node of the global reinforcement element. High bond stiffness and strength were assigned to that node. The material properties used are shown in Table 8.3. It was assumed that the element modelled a 2 m Ø22 mm rebar. The block properties used were derived from LaRonde model in section 6.6.1. A strain limit was assigned to the structural elements to simulate bolt rupture. The block was 2.5 m long.

Table 8.3: Material properties for the global reinforcement and the block.

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Rebar</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Global reinforcement</strong></td>
<td></td>
</tr>
<tr>
<td>Length (m)</td>
<td>2 m</td>
</tr>
<tr>
<td>Area (m²)</td>
<td>3.8E-4</td>
</tr>
<tr>
<td>Young's modulus (Pa)</td>
<td>200E9</td>
</tr>
<tr>
<td>Tensile yield strength (N)</td>
<td>200E3</td>
</tr>
<tr>
<td>Strain limit</td>
<td>0.12</td>
</tr>
<tr>
<td>Bond stiffness (N/m/m)</td>
<td>1.3E7</td>
</tr>
<tr>
<td>Cohesive strength of bond (N/m)</td>
<td>25E4</td>
</tr>
<tr>
<td><strong>Block properties</strong></td>
<td></td>
</tr>
<tr>
<td>Young's modulus (MPa)</td>
<td>48,000</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.16</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>35</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>12</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>2.81</td>
</tr>
</tbody>
</table>
The load-displacement curves are shown in Figure 8.18. The performance of the bolt was similar for 0°, 20°, 40° and 60° displacing angles. The results were similar with the load tests on rebars in different displacing angles performed by Chen (2014).

Figure 8.18: Load tests for different displacing angles; a) elastic displacement along the bolt for 40° displacement angle; and b) load-displacement curves for the examined displacement angles.

Consequently, based on the limitations on the use of local reinforcement, it was decided to investigate if the use of global reinforcement and calibration would capture the behaviour of reinforcement in a way that can be comparable with field observations.

8.5 Implementation of global reinforcement elements in the LaRonde 3D DEM model

The global reinforcement elements were used to simulate the field behaviour of rebars, FRS, hybrid bolts and cable bolts. The material properties used for each rock bolt were calibrated through numerical pull tests. Each bolt was pulled from a 2.5 m long block. Forty segments were used for the rebars, the FRS and the hybrid bolts and sixty segments were used for the cable bolts. A velocity boundary condition was applied to a block bonded at the last node of each bolt. High bond strength was applied to the part of the bolt attached to the loading block. The load on each bolt was estimated by adding the reaction forces on the loading block as the bolt was pulled. A strain limit was assigned to the elements. The derived material properties are shown in Table 8.4. The mechanical properties for the blocks were those used at the LaRonde model and are presented in Table 8.3. The confinement effect on the bolts was not taken into account.
Table 8.4: Calibrated material properties for the reinforcement elements used in LaRonde.

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>FRS (sidewalls)</th>
<th>Rebars (back)</th>
<th>Rebars (back)</th>
<th>Hybrid Bolts (sidewalls)</th>
<th>Cables (sidewalls)</th>
<th>Cables (back)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (m)</td>
<td>2</td>
<td>2.3</td>
<td>1.9</td>
<td>2</td>
<td>6.4</td>
<td>8</td>
</tr>
<tr>
<td>Area (m$^2$)</td>
<td>2.71E-4</td>
<td>3.8E-4</td>
<td>3.8E-4</td>
<td>6.51E-4</td>
<td>1.99E-4</td>
<td>1.99E-4</td>
</tr>
<tr>
<td>Young's modulus (Pa)</td>
<td>200E9</td>
<td>200E9</td>
<td>200E9</td>
<td>200E9</td>
<td>200E9</td>
<td>200E9</td>
</tr>
<tr>
<td>Tensile yield strength (N)</td>
<td>127E3</td>
<td>185E3</td>
<td>185E3</td>
<td>367E3</td>
<td>261E3</td>
<td>261E3</td>
</tr>
<tr>
<td>Strain limit</td>
<td>0.12</td>
<td>0.35</td>
<td>0.35</td>
<td>0.12</td>
<td>0.035</td>
<td>0.035</td>
</tr>
<tr>
<td>Bond stiffness (N/m/m)</td>
<td>2.33E6</td>
<td>1.5E7</td>
<td>1.5E7</td>
<td>6.4E6</td>
<td>4.17E6</td>
<td>5.56E6</td>
</tr>
<tr>
<td>Cohesive strength of bond (N/m)</td>
<td>35E3</td>
<td>55E4</td>
<td>55E4</td>
<td>80E3</td>
<td>400E3</td>
<td>400E3</td>
</tr>
</tbody>
</table>

This investigation took into account the specificities of each bolt type employed at LaRonde Mine. The approach can eventually be extended to new bolt types. These parameters are based on the actual field conditions at the LaRonde mine whereby the rock mass is extremely fractured. This is reflected in the results of the pull tests. The resulting stiffness may appear to be lower than pull tests in good ground conditions. This is to be anticipated. A practical incentive is the development of new rock reinforcement elements for extreme fractured conditions such as LaRonde. A promising avenue for this research is the use of yieldable reinforcement elements such as the D-Bolt with the use of pumpable resin.

8.5.1 Rebar

The results of the numerical modelling axial load experiments for a 2.3 m long rebar bolt are presented in Figure 8.19. Before the element yields, the axial load increases linearly along the bolt (point 1). A higher axial load is observed on the pulled side of the bolt. When the bolt reaches its axial capacity, plastic deformation occurs (point 2). The bolt ruptures when the strain limit is reached (point 3). During laboratory pull out tests the bolt was detached near the loading point. In order to simulate rupture of the bolt, it was assumed that the nodal point contacts between the element and the blocks remained intact while the bolt was allowed to displace between the nodes (Li et al., 2014). It was assumed that the bolt failed in tension after 38 mm of elongation. The peak load used was 180 kN (Jenmar, 2015). The shear force was higher near the loading point.
Figure 8.19: Performance of a 2.3 m rebar under axial load using global reinforcement elements.

8.5.2 Friction rock stabilizers (FRS)

Figure 8.20 summarises the numerical modeling results for FRS. The axial load increases linearly along the bolt (point 1) with higher axial load observed on the pulled side of the bolt. The maximum shear force developed per length of element is limited to the cohesive strength of the bond. For the 2 m FRS the maximum load was 70 kN and was distributed equally along the 40 nodes used (approximately 1.8 kN per node). When the peak load is reached, the bonds fail and the bolt starts to slide (point 2). The bolt continues to slide under a constant applied load (point 3).
8.5.3 Hybrid bolt

The hybrid bolt is stiffer and has greater capacity than FRS under axial loading. Figure 8.21 shows the linear increase of the axial load along the length of the bolt (point 1). Higher axial load is observed on the pulled side of the bolt. When the peak axial load is reached, the bonds fail and the bolt starts to slide (point 2) as observed in FRS and continues to slide under a constant load (point 3). The tensile capacity of the bolt was not exceeded.
8.5.4 Cable bolts

In the numerical experiment, axial load was applied to a single strand cable. The embedment length used was 1.8 m, same as at the LaRonde pull out tests. A linear load-displacement curve was derived. The stiffness of the bolt was similar to the field pull out tests in Figure 8.8. The test was stopped at 150 kN and the axial force on the cable was higher near the loading point. The shear force decreases away from the loading point. Laboratory tests using 1.8 m embedment length showed that the bolt ruptures when the tensile capacity of the strand (261 kN) is reached (Hyett & Bawden, 1996). In the absence of field data for the bond strength of the cables at LaRonde, the strength assigned to the global reinforcement elements for practical purposes was 400 kN/m. This allowed keeping the bond between the bolt and the blocks intact during the test as shown in Figure 8.22. The laboratory values for other bolts have shown that mechanical properties measured in the field at LaRonde are relatively lower. This is attributed to the ground conditions specific to LaRonde in squeezing ground. The bond strength assigned to the global reinforcement elements for the cables is considered to be a lower bound value. Plain strand cables have lower bond strength than bulbed cables (Hyett et al., 1995). The value chosen is equal to published values on plain strand cables by
Hutchinson and Diederichs (1996) for a Young’s modulus of 48 GPa and a water: cement ratio of 0.35 at 40 mm slip.

Figure 8.22: Cable bolt performance under axial load using global reinforcement elements.

This simulated test reproduced the performance of cables as observed in the in-situ pull out tests which did not load the bolts to failure. Laboratory tests with the same embedment length indicate rupture on the cable (Hyett & Bawden, 1996). When installed in simulated squeezing conditions, the structural elements reproduced the strand rupture when the tensile capacity of the bolt was reached keeping the bonds intact. This is discussed further in section 8.7.1.

8.6 Influence of reinforcement under simulated squeezing conditions

As demonstrated in section 8.5, following a calibration process, it was possible to capture the behaviour of bolts in simulated pull test conditions. The calibrated material properties were used to examine the performance of the rock bolts under squeezing conditions. Rebar, FRS, hybrid bolts and cable bolts were introduced to the LaRonde DEM squeezing rock model. The pressure reduction method was used to introduce global reinforcement elements to the model at different deformation stages based on the
installation sequence followed at LaRonde. The geometry of the model and the mechanical properties used as presented in section 6.6.1.

The pressure reduction method can model the progress of deformation during the development of a drift. However, the modelling steps cannot be directly linked to the actual steps of a 3D advancing face. When the reduction factor \( r \) is equal to one, the face was considered to be ahead of the modelled section with no displacement appearing in the model. As \( r \) was reduced, the face was approaching the modelled section and was over passing it. At the final modelling step (\( r=0 \)), the face had no effect on the modelled section. When the support is installed during the development of drift, right after blasting, deformation has already occurred in the rock mass. This corresponds to a value of the reduction factor lower than one.

### 8.6.1 Impact of sequential installation of reinforcement

The mine, based on empirical experience, has recognized that reinforcement in squeezing ground conditions is best installed in stages. The primary support, installed after blasting, is comprised of FRS in the sidewalls and rebars in the back. Secondary support comprised of hybrid bolts in the sidewalls is subsequently installed 12 m behind the face. Finally, tertiary support comprised of cable bolts is installed in the back and the sidewalls before mining the secondary stopes.

The numerical experiments investigated a series of case studies, based on the sequence followed at LaRonde Mine. The base case model looked at the case where no support would be installed. This was a reference case as in practice the mine installs reinforcement in all mining drives. In case 5a primary support was added in the model at stage 6 (\( r=0.3 \)). In case 5b, in addition to the primary support, secondary support was installed in the model at stage 8 (\( r=0.15 \)) and in case 5c, in addition to the primary and secondary support, tertiary support was added at stage 10 (\( r=0.066 \)). The reduction factor (\( r \)) was reduced up to zero (step 13) in all the cases examined. Figure 8.23 shows the modelled displacement at the stage when each reinforcement element was introduced to the model. The cable bolts were installed following secondary support and before the reduction factor was zero.
The total displacement for each case at the last modelling stage (r=0) and the mechanical behaviour of the reinforcement elements are presented in Figure 8.24. As anticipated, the use of reinforcement reduced the level of displacement. The reduction is small when only primary support (rebars and FRS) was used and larger when secondary support (hybrid bolts) was introduced to the model. The rebars installed in the back yielded in cases 5a and 5b. The bond strength of the FRS and the hybrid bolts was exceeded in all cases resulting in slipping of the bolts. The hybrid bolts were subjected to higher load than the FRS due to their larger shear capacity. The introduction of tertiary support (cables) provided a further reduction on the displacement. The bond strength was not exceeded in any cable bolt while those installed in the centre of the sidewalls were subjected to the highest load.
Figure 8.24: Modelled displacement and performance of rock bolts for case 5; a) no support; b) case 5a; c) case 5b and d) case 5c.

The longitudinal displacement profile at the centre of each wall is shown in Figure 8.25. The effect of the primary support on the displacement in the foot wall and the hanging wall is minor (4%). The use of rebars in the back reduced the displacement for approximately 10 cm. When hybrid bolts were installed in the sidewalls, the reduction on the displacement in the sidewalls was 18 cm (or 18%). The use of secondary support reduced the level of squeezing overall, having also an effect on the displacement in the back and the floor. The installation of cable bolts in the back and the walls reduced the displacement further (23%).
8.6.2 Effect of time of installation of the secondary support

The effectiveness of the secondary support was further examined. As previously the reinforcement was introduced in the model sequentially. In case 6a the primary support (rebars in the back and FRS in the sidewalls) was introduced to the model at stage 5 \((r=0.45)\). This was one stage earlier in the model than in case 5. In case 6b secondary support was installed in the model at stage 6 \((r=0.3)\) and in case 6c the secondary support was installed at stage 7 \((r=0.2)\). The reduction factor \((r)\) was reduced up to zero (step 13) for all the cases examined.

The mechanical behaviour of the rock bolts and the modelled displacement are shown in Figure 8.26. The primary support was subjected to higher loads than in case 5 due to the earlier installation of the support. The strain limit of the rebar installed in the centre of the back was exceeded breaking the bolt. When
secondary support was introduced to the model, in cases 6b and 6c, the additional reinforcement elements absorbed the excessive load reducing the total displacement and preventing the failure of the reinforcement in the back.

Figure 8.26: Modelled displacement and performance of rock bolts for case 6; a) no support; b) case 6a; c) case 6b and d) case 6c.

The longitudinal displacement profile at the centre of each wall for case 6 is shown in Figure 8.27. The reduction on the displacement is higher in the sidewalls and the back when the hybrid bolts are installed at step 6 than at step 7.
Figure 8.27: Longitudinal displacement profile at the centres of wall for case 6.

8.6.3 Influence of the time of installation of the primary support

As described in section 8.6, the primary support is installed after some closure has already occurred at the drift. There are no monitoring data available that can indicate the drift deformation before the installation of the primary support. In addition to cases 5a and 6a, the primary support was installed at stage 3 when $r=0.7$ (case 7a). The mechanical behaviour of the bolts and the modelled displacement are presented in Figure 8.28. The plots show that if the primary support is installed at earlier deformation stages, the rebars yield and fail in tension.
Figure 8.28: Modelled displacement and performance of rock bolts for case 6; a) no support; b) case 6a; c) case 6b and d) case 6c.

The longitudinal displacement profile at the centre of each wall for cases 5a, 6a and 7a is shown in Figure 8.29. The graphs show that although the rebars may fail in the back when the primary support is installed at earlier squeezing stages, the modeled displacement in the hanging wall and the foot wall is lower. Failure of the reinforcement in the back can actually result in loss of the rock mass confinement and larger displacement. This mechanism cannot be captured by the numerical model. Although the modelling methodology reproduces the buckling mechanism observed in LaRonde, it does not capture the unravelling of rock material in the walls.
The modelled performance of the FRS and the hybrid bolts is based on the friction at the interface between the bolts and the rock. The global reinforcement elements cannot model the localized shear on the reinforcement elements and the “locking” mechanism when the bolts are installed early in the deformation process. Structural elements that take into account the bending moments generated when the bolt is subjected to shear and prevent sliding when bending occurs, could capture better the mechanical behaviour of reinforcement under shear load.

The numerical simulations focused only on the mechanical behaviour of rock bolts and their effect on the deformation. Under squeezing conditions, the surface support maintains the necessary confinement while distributing the load from the bolts on the surface of the drift walls. This effect was not taken into account.
The displacement rate is influenced by the ground conditions. In addition, the modelling stages are not directly linked with the actual steps of a 3D advancing face and the time after the excavation of the drift. However, the reduction on the displacement rate observed in the field after the installation of the hybrid bolt is in agreement with the numerical modelling results. The use of hybrid bolts, as a secondary support strategy, reduces the displacement and decreases the required rehabilitation.

8.7 Investigation of other reinforcement options

8.7.1 Installation of cable bolts at early deformation stages

The performance of cable bolts installed at later modelling stages, when most of the deformation has already occurred, was examined in section 8.6.1. This section examines the behaviour of cable bolts when installed early in the squeezing process. The mechanical properties for the bolts presented in section 8.5 were used. The cable bolts were installed as secondary support replacing the hybrid bolts in case 6b presented in section 8.6.2. The primary support (rebars in the back and FRS in the sidewalls) was introduced to the model at stage 5 (r=0.45) and the cable bolts were installed at stage 6 (r=0.3) and then the reduction factor (r) was reduced up to zero (step 13). The modelled displacement and the mechanical behaviour of the bolts are presented in Figure 8.30a. The displacement was slightly lower than in case 6b. While the hybrid bolt can yield and slide inside the rock allowing deformation to occur without any bolt failure the cable bolts provide a stiffer support reaction. As shown in Figure 8.30a, the capacity of the cable bolts, 261 kN, was exceeded resulting in rupture.

Figure 8.30: a) Modelled displacement and performance of rock bolts when cable bolts are installed as a secondary support at stage 6; b) failure of the cable plates and the surface support when cable bolts are installed early in the squeezing process at LaRonde (Mercier-Langevin & Turcotte, 2007a).
Early installation of fully grouted cable bolts at LaRonde showed that the cable bolt plates either failed or the bolts sunk into the drift walls as shown in Figure 8.30b. Failure in the cable plate and the surface support would result in loss of confinement in the rock mass. This can result in unravelling and increase of the wall convergence. The numerical model cannot reproduce this mechanism.

8.7.2 Effect of the D-Bolt

The D-Bolt is a yielding bolt made of a smooth steel bar with a small number of anchors (Li, 2012). The bolt is fully encapsulated with resin or cement grout. The weak bonding between the grout and the smooth section of the bolt allows the bolt to elongate when the rock dilates while the anchor points remain intact. The performance of D-Bolts has been studied in the past through laboratory and field tests.

8.7.2.1 Performance of D-Bolts in laboratory and field tests

Li (2012) reported on tests carried out in CANMET Mining and Mineral Sciences Laboratories, Canada. The load displacement graph from three 22 mm bolts, installed in split steel tubes using resin, are shown in Figure 8.31a. The distance between the anchor points was 1.5 m. Chen (2014) tested the performance of cemented grouted D-Bolts subjected to pull and shear load simultaneously using two cubic blocks of high strength concrete. The results for different displacement angles are shown in Figure 8.31b.

![Figure 8.31](image)

Figure 8.31: Load displacement curves from laboratory tests on D-Bolts: a) Pull tests on resin grouted 22 mm D-Bolt with 1.5 length between anchor points (Li, 2012); and b) Different displacing angles on cement grouted 20 mm D-Bolt with 1 m length between anchor points (Chen, 2014).
In-situ pull tests on D-Bolts were carried out at LaRonde. The tests aimed to evaluate the quality of installation and the anchorage capacity for 2.2 m and 1.8 m long bolts. The two bolt configurations are presented in Figure 8.32.

![Figure 8.32](image)

**Figure 8.32:** The two D-Bolt configurations examined at LaRonde; a) 2.2 m long bolt and b) 1.8 m long bolt (Charette, 2015).

The results from the in-situ pull out tests are shown in Figure 8.33. The four tests performed on the 2.2 m long configuration indicated a wide variation in stiffness. This was attributed to low rock mass quality (fractured ground) and weak anchorage due to the loss of resin during the installation of the bolts. The six tests made on the 1.8 m long configuration, with relatively lower distance between the anchors, showed lower variation in the stiffness. These tests were also made in fractured rock while a smoother surface was provided reducing accidental displacement (Charette, 2015). The results from the 1.8 m long bolt configuration were consistent. The bolts were loaded approximately up to their yielding point.

![Figure 8.33](image)

**Figure 8.33:** Load-displacement curves for the in-situ pull tests on 2.2m and 1.8 m long bolts at LaRonde, after Charette (2015).
8.7.2.2 Application of the global reinforcement elements for the D-Bolt

The global reinforcement elements were used to simulate the performance of the D-Bolt as captured in the in-situ pull out tests at LaRonde. The methodology and the block properties presented in section 8.5 were used for this purpose. It was assumed that the bolt can slide between the anchor points while the anchor points remained intact. Two sets of material properties were used as shown in Table 8.5. Low shear strength was assigned to the part of the structural element between the anchor points. Large shear strength was assigned to the three nodes representing the anchor points to prevent shear failure. Laboratory tests on D-bolts have indicated a strain hardening material behaviour. A perfectly plastic behaviour was assumed in which the bolt continues to deform under constant load when the tensile strength is reached. The peak load chosen was slightly higher than the yielding load measured in laboratory tests.

Table 8.5: Calibrated material properties for the reinforcement elements used in LaRonde.

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>D-Bolts (between anchor points)</th>
<th>D-Bolts (at anchor points)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (m²)</td>
<td>3.8E-4</td>
<td>3.8E-4</td>
</tr>
<tr>
<td>Young's modulus (Pa)</td>
<td>200E9</td>
<td>200E9</td>
</tr>
<tr>
<td>Tensile yield strength (N)</td>
<td>230E3</td>
<td>230E3</td>
</tr>
<tr>
<td>Strain limit</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>Bond stiffness (N/m/m)</td>
<td>1.3E8</td>
<td>1.3E8</td>
</tr>
<tr>
<td>Cohesive strength of bond (N/m)</td>
<td>1</td>
<td>950E4</td>
</tr>
</tbody>
</table>

The modelling results are presented in Figure 8.34. The axial load is constant between the anchor points while the shear load on the bolt is concentrated at the anchor points (point 1). Plastic deformation initiates when the pull load reaches the axial capacity of the bolt (point 2). The anchor point at 0.3 m is the one subjected to the highest shear force. The sum of the shear forces at the three anchor points is equal to the pull load applied to the bolt. The bolt continues to undergo plastic deformation between the load point and the anchor point at 0.3 m until it eventually fails when the strain limit is reached (point 3). Although the bolt was not tested to failure in the field, it was assumed that the 0.3 m long part of the bolt can sustain an ultimate deformation of approximately 40 mm. This was based on laboratory test results by Chen (2014) who reported deformation up to 120 mm for a stretched length of 1 m.

The results from the use of global reinforcement elements are in agreement with the in-situ and laboratory pull out tests. The derived material properties can be used to investigate the performance of the D-Bolt in simulated squeezing conditions.
Figure 8.34: Performance of a 1.8 m long D-Bolt under axial load using global reinforcement elements.

8.7.2.3 Performance of D-Bolts under simulated squeezing conditions

The D-Bolts were installed as secondary support in simulated squeezing conditions. The case 5 presented in section 8.6.1 was used to compare the effect of D-Bolts and hybrid bolts in the model. Rebars were first installed in the back and FRS in the sidewalls at the base case model (model with no support) after the numerical equilibrium at stage 6 ($r=0.3$). Three D-Bolts were subsequently installed in each sidewall at the same location as the hybrid bolts in case 5b. Figure 8.35 shows the final modelled displacement when D-Bolts were used in the model. The results are compared with the performance of hybrid bolts in case 5b. As expected, the D-Bolts were subjected to higher load than the hybrid bolts. The peak load was reached in all D-Bolts. The D-Bolt installed in the bottom of the hanging wall failed between the first and second anchor points at stage 12.

The longitudinal displacement profile at the centre of each wall is presented in Figure 8.36. The modelled displacement using D-Bolts is marked as case 8. The graphs show that the use of D-Bolt as secondary support reduces the displacement in the model for 23 mm compared to the numerical model without any reinforcement. The reduction obtained by the use of the D-Bolt is even larger than the one provided by the hybrid bolts. If this trend is observed in the field this has significant implications.
Figure 8.35: Comparison of the modelled displacement and the performance of rock bolts; a) D-Bolts installed at stage 8; and b) Hybrid bolts installed at stage 8.

Figure 8.36: Longitudinal displacement profile at the centre of each wall for cases 5a, 5b and 8.
Bolt failure can result in loss of the wall confinement, unravelling and excessive deformation that cannot be captured by the model. The failure of the D-Bolt in case 8 could trigger such a mechanism. However, only one bolt was loaded up to complete failure and rupture occurred at the last modelling stages. At that time the D-Bolts were already providing higher reduction on the displacement than the hybrid bolts in case 5b. It should also be noted that the strain limit assigned to the global reinforcement elements was based on laboratory tests. No bolts have been tested to failure in the field. The installation of cable bolts at later deformation stages would have reduced the displacement and prevented the bolt failure.

8.8 3D impact of the discrete effect of reinforcement

The impact of the discrete effect of reinforcement in three dimensions can be simulated in numerical models in different ways. The first option would be to use a 3D numerical model to simulate explicitly the sequential advancement of the face and the installation of the support. As discussed in chapter 6, computational restrictions and time limitations did not allow modelling the development of a drift using the 3D discrete element model that captured the buckling mechanism. This was made possible, however, following a pseudo-3D approach and modelling a thin slice of a drift. The progressive reduction of the forces acting at the boundaries of the excavation in the model allowed capturing the progress of deformation as observed in the field. This approach can be used to introduce explicit support in the model at different deformation stages as presented in section 8.6. In this case, one row of bolts can only be introduced to the model and the investigation on the impact of the reinforcement in the out of plane direction is limited on the thickness of the model (0.05 m). Scaling the material properties of the reinforcement elements can distribute the discrete effect of elements in the out of plane direction (Itasca Consulting Group Inc, 2011b). This approach can examine the sensitivity of the modelled displacement in any changes of the reinforcement spacing.

The use of global reinforcement element and the calibrated material properties for the rock bolts in LaRonde represented successfully the impact of reinforcement in the discrete element model. The resulted models captured the impact of different reinforcement scenarios in the simulated squeezing conditions as observed in the field. Given that the analysis compares different reinforcement scenarios the values used for the material properties are definitely justifiable and appropriate.

Further work was conducted to examine whether taking into account the out of plane effect maybe more appropriate in this analysis. The 3D impact of the discrete effect of reinforcement can be examined by linear scaling of the material properties of the reinforcement. This approach averages the effect of reinforcement in 3D and takes into account a spaced pattern of reinforcement (Itasca Consulting Group Inc, 2011b).
The work investigated whether scaled down properties can be successfully used and if there are significant differences in the numerical experiments for the LaRonde case between the two approaches. The impact of the reinforcement in a 0.6 m spacing was examined based on the spacing used at LaRonde. The calibrated material properties of the global reinforcement presented in Tables 8.4 and 8.5, were linearly scaled to distribute the discrete effect of the bolts in the out of plane direction (x-axis in Figure 6.4).

The material properties of the structural elements were divided by 12 to account for a 0.6 m spacing in the 0.05 m thick discrete element model presented in section 6.6.1. For the FRS and the hybrid bolt the parameters scaled were those controlling the axial force and the shear force of the elements namely the Young’s modulus, the tensile yield strength, the bond stiffness and the cohesive strength of the bond. For the rebar and the D-Bolt the parameters scaled were those controlling the axial force of the elements and the shear stiffness. Based on the calibration of the elements, as presented in sections 8.5 and 8.7, and the modelled failure mechanism, it was assumed that the bolts do not slide at the anchorage points in both scaled and non-scaled cases. Therefore, the cohesive strength of the bond was not modified. The scaled material properties for the FRS, the hybrid bolt, the rebar and the D-Bolt are shown in Table 8.6.

**Table 8.6: Scaled material properties for the reinforcement elements used in LaRonde to represent a 0.6 m spaced pattern of structural elements.**

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>FRS (sidewalls)</th>
<th>Rebars (back)</th>
<th>Rebars (back)</th>
<th>Hybrid Bolts (sideways)</th>
<th>D-Bolts (between anchor points)</th>
<th>D-Bolts (at anchor points)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (m)</td>
<td>2</td>
<td>2.3</td>
<td>1.9</td>
<td>2</td>
<td>1.8</td>
<td>1.8</td>
</tr>
<tr>
<td>Area (m²)</td>
<td>2.71E-4</td>
<td>3.8E-4</td>
<td>3.8E-4</td>
<td>6.51E-4</td>
<td>3.80E-04</td>
<td>3.80E-04</td>
</tr>
<tr>
<td>Young's modulus (Pa)</td>
<td>1.67E+10</td>
<td>1.67E+10</td>
<td>1.67E+10</td>
<td>1.67E+10</td>
<td>1.67E+10</td>
<td>1.67E+10</td>
</tr>
<tr>
<td>Tensile yield strength (N)</td>
<td>1.06E+04</td>
<td>1.54E+04</td>
<td>1.54E+04</td>
<td>3.06E+04</td>
<td>1.92E+04</td>
<td>1.92E+04</td>
</tr>
<tr>
<td>Strain limit</td>
<td>0.12</td>
<td>0.35</td>
<td>0.35</td>
<td>0.12</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>Bond stiffness (N/m/m)</td>
<td>1.94E+05</td>
<td>1.25E+06</td>
<td>1.25E+06</td>
<td>5.33E+05</td>
<td>1.08E+07</td>
<td>1.08E+07</td>
</tr>
<tr>
<td>Cohesive strength of bond (N/m)</td>
<td>2.92E+03</td>
<td>5.50E+05</td>
<td>5.50E+05</td>
<td>6.67E+03</td>
<td>1.08E+07</td>
<td>9.50E+06</td>
</tr>
</tbody>
</table>

The scaled material properties were used in the reinforcement scenarios from cases 5a, 5b and 8 presented in sections 8.6 and 8.7. Figure 8.37 shows the modelled displacement, the performance of rock bolts and LDP of the hanging wall for case 5a (when only primary support was introduced to the model) using scaled and non-scaled material properties. The reduction on the displacement in the hanging wall when non-scaled
properties were used was 4% whereas when scaled properties were used, there was no reduction on the displacement. The plots show that, as expected, the axial force applied to the global reinforcement elements when scaled properties were used was lower than in the case of non-scaled properties. The failure in rebars in the back when scaled properties were used was logical given the lower material properties used. The bonds of the rebars in the back remained intact in both cases while the bonds of the FRS failed in both cases resulting in sliding of the bolts.

Figure 8.37: Modelled displacement, performance of rock bolts and LDP of the hanging wall for case 5a using scaled and non-scaled material properties.
Figure 8.38 shows the final modelled displacement, the performance of rock bolts and LDP of the hanging wall for case 5b using scaled and non-scaled material properties. When hybrid bolts were introduced to the side walls in addition to the primary support, the reduction on the displacement in the hanging wall for non-scaled properties was 18%. When scaled properties were used, the reduction on the displacement was 10%. The axial force applied to the reinforcement elements was lower when scaled properties were used as expected. The plots illustrate that the bonds of the FRS and the hybrid bolts failed to a larger extent when scaled properties were used due to the lower bond strength assigned to the elements. The results are logical.
Figure 8.39 shows the results for case 8, when D-Bolts were introduced to the model instead of hybrid bolts, including the final modelled displacement, the performance of the rock bolts and the LDP of the hanging wall for both scaled and non-scaled material properties. The reduction of the final displacement for non-scaled properties was 23%. When scaled properties were used the final displacement was reduced for 20%. The maximum axial force that the D-Bolts could sustain when scaled properties were used was lower and the bolts failed in certain areas as shown in the plots.

Figure 8.39: Modelled displacement, performance of rock bolts and LDP of the hanging wall for case 8 using scaled and non-scaled material properties.
Figure 8.40 shows the combined interpretation of the LDP for all the walls in cases 5a, 5b and 8 for both scaled and non-scaled material properties. The results from all the walls indicate similar trends. Scaling the material properties averaged the effect of reinforcement in 3D and increased the modelled displacement for each case. The modelled displacement was higher when scaled material properties were used than when non-scaled properties were assigned to the bolts. The modelled displacement using scaled and non-scaled properties indicated the same trends. The introduction of hybrid bolts, as part of a secondary support strategy, contributes to a significant reduction in drift convergence. The D-Bolt can provide a larger reduction on the displacement than the hybrid bolt.

![Graph showing longitudinal displacement profiles for all walls in cases 5a, 5b, and 8 for scaled and non-scaled material properties.](image)

Figure 8.40: Longitudinal displacement profiles for the all the walls in cases 5a, 5b and 8 for scaled and non-scaled material properties.
Non-scaled elements show relatively larger reduction in displacement. The obtained results from both approaches are consistent with field observations. The non-scaled results are relatively closer to field observations. However, none of the two approaches explicitly take into consideration the impact of surface support. This would have to account for load transfer between the different elements which is not a trivial exercise and is beyond the scope of this thesis.

8.9 Conclusions

This chapter has built on previous work, presented in chapter 6 that captured the structurally defined squeezing mechanism observed in hard rock mines using the 3D DEM. The buckling mechanism at LaRonde has been reproduced by progressive reduction of the forces acting at the boundaries of the excavation by a reduction factor through a series of modelling steps. The use of the distinct element method overcomes the computational and time constraints on modelling the development of a mining drift.

The work from this chapter focused on the role of reinforcement in managing the level of deformation at the LaRonde Mine using the 3D DEM. The explicit introduction of reinforcement elements is not a trivial exercise. The first step was to investigate whether reinforcement can be introduced in the 3D DEM for squeezing buckling conditions and assess the behaviour of the structural elements under these conditions. The global reinforcement elements were installed successfully in the model. It was identified, however, that the local reinforcement elements cannot be introduced in the model at later modelling stages, after significant deformation had occurred in the model.

A review of the structural elements available in the 3D distinct element method identified the advantages and limitations of different modelling strategies for structurally driven squeezing ground conditions in hard rock mines. The use of local reinforcement elements in the numerical model considers the bending resistance of rock bolts when crossing discontinuities. They cannot, however, account for the deformation of individual blocks to the reinforcement elements and consequently are not suitable for squeezing conditions. Global reinforcement elements take into account the deformation of individual blocks but do not provide a direct bending resistance on the block interface when crossing discontinuities. The combined use of local and global reinforcement elements revealed a series of limitations.

The behaviour of global reinforcement in the presence of shear in the modelled squeezing conditions was subsequently investigated. The interpretation of the joint normal and shear displacement in the 3D DEM LaRonde model showed that the reinforcement installed in the model is subjected to a combined pull and shear displacement. The maximum displacement angle estimated for bolts installed perpendicular to the foliation was approximately 60°. While the global reinforcement elements cannot capture the doweling effect of bolts under pure shear, numerical investigations showed that the performance of the elements
under combined pull and shear load, in displacement angles between $0^\circ$ and $60^\circ$ is similar. It was consequently decided to investigate whether the use of global reinforcement and calibration would capture the behaviour of reinforcement in a way that can be comparable with field observations.

Following a series of numerical experiments, the calibration of the global reinforcement elements captured the behaviour of the rock bolts in simulated pull conditions. The mechanical properties used were based on in-situ pull tests at the mine. The in-situ pull tests illustrated the difference in the behaviour of the bolts on site. Rebar bolts provided a stiff response under axial load with limited deformation capability. The FRS showed low loading capacity with high deformation capability. The hybrid bolts demonstrated a stiff reaction with high load capacity and high deformation capability. Finally, cable bolts provided a stiff reaction to the displacement.

The calibrated material properties were used to examine the performance of the rock bolts under squeezing conditions. Different reinforcement scenarios were investigated. The impact of rebars, FRS, hybrid bolts and cable bolts on the displacement was studied.

The LaRonde Mine has recognized that reinforcement in squeezing conditions is best installed in stages. Field observations presented in chapter 7 indicated that the hybrid bolt has a direct effect on the displacement and when is introduced as part of a secondary support strategy contributes to a significant reduction of the drift convergence and reduces the rehabilitation. The rock bolts were introduced to the model reproducing the installation sequence followed at LaRonde.

The use of rebar bolts in the back and FRS in the sidewalls had a minor effect on the modelled squeezing level. The use of hybrid bolts, as a secondary support strategy, reduced the displacement. The installation of cable bolts at later modelling stages contributed to a larger reduction on the deformation. When the primary support was installed in an early modelling stage, the rebars did not manage to accommodate the large deformation in the back and failed in tension. The installation of hybrid bolts in this case, prevented the failure of the rebars and reduced the final displacement.

The effect of surface support was not taken into account in the model. In addition, the structural elements did not consider explicitly the radial confinement effect on the bolts. Although the sliding mechanism of FRS and hybrid bolts was explicitly represented in the model, the “locking” effect reported in certain cases when the bolts are installed earlier in the squeezing process, was not reproduced. Structural elements that take into account the bending moments generated when bolts are subjected to shear and prevent sliding when bending occurs could provide a better representation of that effect.
The numerical investigations are in agreement with field observations and monitoring data from the LaRonde Mine. In both cases it was noted that the hybrid bolt, installed as part of a secondary support strategy, contributes significantly, towards the reduction of the drift convergence and decreases the rehabilitation. The developed methodology is applicable to other mine sites experiencing structure control deformations. In combination with pull tests it provides a template for investigating the potential impact of new type of rock bolts in structurally defined squeezing conditions.

Other reinforcement scenarios were investigated following the developed methodology. The use of cable bolts as secondary support strategy in the place of hybrid bolts was first examined. The numerical models showed that the cable bolts fail in tension in this case. This failure could result in unravelling, loss of the wall confinement and excessive deformation in the field.

The global reinforcement elements were calibrated to simulate the behaviour of D-Bolts based on pull out tests from the LaRonde Mine. It was assumed that the bolt does not slide at the anchorage points. The calibrated global reinforcement elements were introduced to the 3D DEM LaRonde model. The model indicated a potential advantage in the use of D-Bolt. The introduction of 1.8 m D-Bolts as secondary support in the numerical models showed that they can provide more than 5% larger reduction on the displacement than the hybrid bolts. It is interesting to determine if these predictions are verified in the field.

The calibrated material properties of the global reinforcement elements were linearly scaled to examine whether taking into account the out of plane effect maybe more appropriate in this analysis. This approach averaged the effect of reinforcement in 3D taking into account a spacing pattern of reinforcement and increased the modelled displacement for each case.

The obtained results from scaled and non-scaled input reinforcement properties are consistent with field observations. It was evident that both approaches capture the same trends on the impact of reinforcement elements and gave similar magnitudes in reduction. The use of hybrid bolts contributed to a significant reduction of the convergence. The use of the hybrid bolt as part of a secondary support strategy can potentially reduce the displacement and decrease the required rehabilitation in cases of large initial deformation. The modelling results suggest that the D-Bolt can provide larger reduction of the displacement that the hybrid bolt.

This investigation used a comparative modelling approach indicating successfully the difference between different reinforcement strategies. The work took into account the specificities of each bolt type employed at LaRonde Mine and explored the potential use of D-Bolt. This approach can eventually be extended to other bolt types that can in theory provide effective yielding support such as the de-bonded cables.
Chapter 9
Recommendations for structure controlled deformations in hard rock mines

9 Recommendations for structure controlled deformations in hard rock mines

9.1 Introduction

This chapter presents recommendations for structure controlled deformations in hard rock mines based on a comprehensive literature review and the interpretation of the field data and the numerical investigations from the LaRonde and Lapa Mines. A review of the types of squeezing in underground excavations and the necessary conditions that lead to a buckling type of failure is provided. Tools for assessing the anticipated level of squeezing are presented. Proactive mitigation measures are proposed based on the interpretation of the in-situ information from the LaRonde and Lapa Mines.

The behaviour of different support elements under squeezing conditions is reviewed. The factors to be considered for the selection of an appropriate surface support and reinforcement under squeezing conditions are discussed using field data and underground observations. Industry ground control practices are also taken into consideration for this purpose. The advantages of certain reinforcement strategies under pronounced conditions are presented based on field observations and numerical investigations. Reinforcement design strategies that can mitigate large deformations are presented.

9.2 Identification of the failure mechanism

The first step in examining the potential of squeezing ground conditions in hard rock mines is the identification of the likely driving mechanism. Information on the geology and the rock mass characteristics is necessary to identify the potential modes of failure.

A general phenomenological description of the failure mechanism resulting in squeezing ground conditions in underground excavations has been provided by Aydan et al. (1993) and was described in section 2.3. A schematic representation of the possible mechanisms is shown in Figure 9.1. Large deformations can result from complete shear of the medium in cases of continuous ductile rock masses or rock with widely spaced discontinuities. Shearing and sliding failure of the rock can be observed in thickly bedded rocks involving shear failure of the intact rock and sliding along the bedding planes.
In cases of thinly layered rocks (up to tens of centimeters) (Potvin & Hadjigeorgiou, 2008), a buckling failure mechanism may occur. The design recommendations provided in this chapter refer to deformations from layered rock masses that can result in a buckling type of failure. In these cases, the stress redistribution around the excavation loads axially the layers of the intact rock, promoting shearing along the foliation planes. Bulking in the rock appears as the fractures shear and dilate. When the foliation is sub vertical the sidewalls may buckle. In these cases, shearing in the top of the hanging wall and the bottom of the foot wall is apparent from the early squeezing stages.

9.3 Assessment of the anticipated level of squeezing

Once buckling is recognized as a potential failure mechanism due to the presence of thinly layered rock masses, the anticipated level of squeezing can be identified using empirical tools. Chapter 2 included a review of the available methods to assess squeezing conditions. Several criteria have been proposed for the squeezing potential of tunnels that are limited to the isotropic behaviour of materials and do not consider the presence of a dominant structure feature. The Q system has been used for squeezing (Barton et al., 1974). The limitations of the Q system specific to squeezing have been discussed by Potvin and Hadjigeorgiou (2008) and Palmstrom and Broch (2006). Analytical methods such as the Euler’s formula (Gere & Timoshenko, 1991) can provide information for the buckling potential of a rock slab in idealized situations but cannot provide an estimate of the anticipated level of squeezing in an underground drift.

The hard rock squeezing index (Mercier-Langevin & Hadjigeorgiou, 2011) takes into account the presence of a dominant fracture set and provides information on the anticipated level of squeezing for drifts. The interpretation of in-situ information collected from the LaRonde and Lapa Mines in chapter 5 and the numerical modelling results in chapter 6 validated the index as an indicator for the prediction of structure
defined squeezing conditions in hard rock mines. Since its introduction, the index has become an industry standard for comparing squeezing conditions in hard rock mines, Figure 9.2.

**Figure 9.2: Hard rock squeezing index (Mercier-Langevin & Hadjigeorgiou, 2011).**

The index provides a first indication for the potential of squeezing and the long term strain level based on ranges for the foliation spacing and the stress to strength ratio. The degree of alteration and high induced stresses during the lifetime of an excavation can increase significantly the degree of squeezing. In cases of high induced stresses and/or highly altered rock mass larger deformation than the one suggested by the index may occur. In cases when extreme squeezing conditions are predicted by the index, ground control problems are expected and mitigation measures should be taken to avoid or manage the deformations.

### 9.4 Proactive mitigation measures for structure controlled deformations in hard rock mines

The proactive mitigation measures refer to the actions that can be taken before the development of the excavation and during the design process to minimize the anticipated deformation. The most critical factor controlling the deformation in hard rock mines is the orientation of the excavation with respect to the foliation. The influence of this factor on the displacement cannot be captured by classification systems which assume that the rock behaves as a continuum isotropic material such as the Q system or the GSI. It was, however, taken into account in the squeezing index using the angle of interception. This approach was validated and extended from field data at the LaRonde and Lapa Mines in chapter 5.
When large deformation is expected, the choice of a more favourable angle of interception can result in a more manageable squeezing level. The squeezing index can give a first estimate of the anticipated level of squeezing for different ranges of the angle of interception, Figure 9.3a. More favourable rock mass conditions can also contribute to the reduction of the rock mass deformation (higher foliation spacing, lower in-situ stresses, and higher intact rock strength).

A quantitative interpretation of the effect of the orientation of the excavation with respect to the foliation was presented in chapter 5, Figures 9.3b and 9.3c. The graphs are based on some of the largest strains recorded in hard rock mines. When used for design purposes, they can indicate the highest anticipated wall-to-wall and back-to-floor strain for a given angle of interception. Based on these graphs, drifts can be designed with a larger angle of interception to avoid pronounced squeezing conditions.

![Graphs of squeezing index and strain for different angles of interception](image)

**Figure 9.3**: Influence of the angle of interception on resulting strain; a) squeezing index (Mercier-Langevin & Hadjigeorgiou, 2011); b) total wall-to-wall strain; c) total back-to-floor strain.
While changes in the orientation of development can be an effective strategy in mining under squeezing conditions, certain mining methods allow limited flexibility in such modifications. This was observed at Lapa where the longitudinal drifts are developed sub-parallel to the foliation following the ore. For cases when large deformations cannot be avoided, reactive mitigation measures can be adopted to control the excessive deformation. These measures are based on the use of an appropriate support strategy for the anticipated level of squeezing.

9.5 Selection of support for structure controlled deformations in hard rock mines

When squeezing is anticipated based on Figures 9.2 and 9.3, an effective support strategy should be adopted to mitigate the resulting deformation. Experience in structure controlled deformations in hard rock mines has shown that it is not a realistic expectation to completely arrest the deformation. Such an approach can lead to significant rehabilitation and high support cost (Potvin & Hadjigeorgiou, 2008). An effective support strategy should concentrate in controlling rather than arresting the deformation.

The depth of failure in structure controlled deformations can extend up to 6 m into the walls. Under these conditions, the aim is to integrate and provide some confinement to the failed rock mass using a combination of reinforcement elements and surface support creating a reinforced shell around the excavation. Maintaining the confinement of the rock is critical for the stability of the excavation. Higher reinforcement density can result in a stronger shell around the excavation and more uniform surface deformations.

Potvin and Hadjigeorgiou (2008) identified a dichotomy on the ground control practices used to control structurally defined squeezing mechanisms between Australian and Canadian hard rock mines. Australian mines use wide spacing of FRS creating a relatively softer reinforcement shell. The system becomes stiffer by using fibre reinforced shotcrete. In cases of large deformation, a further layer of mesh is required as shotcrete cracks due to its limited deformation capability. On the other hand, Canadian mines use a higher density of bolts with yielding capability (such as inflatable bolts or hybrid bolt) and weld-mesh complemented with mesh straps.

An effective support system under squeezing conditions should be comprised of compatible surface support and reinforcement elements in terms of their ability to deform and yield under excessive load. When soft surface support is used with stiff reinforcement elements, the bolts may sink inside the walls. On the other hand, when soft reinforcement is used, a stiff surface support may fail at early deformation stages. Overall, a ground support system is as strong as its weakest part.
9.5.1 Surface support

Under squeezing conditions, the surface support aims to maintain the necessary confinement in the rock mass. It prevents any unravelling of smaller rocks between the reinforcement and distributes the support load around the excavation connecting the reinforcement elements.

Shotcrete provides an immediate resistance to the ground pressure preventing degradation and maintaining the rock mass confinement. Under squeezing conditions, it can allow widening of the bolting pattern. Nevertheless, it provides low deformation capability. The use of fibres can increase its ability to deform. However, even fibre reinforced shotcrete cannot accommodate larger deformations than those corresponding to moderate squeezing conditions. In cases of large deformation, mesh is installed on the top to support the cracked fibercrete (Potvin & Hadjigeorgiou, 2008).

The welded mesh, in contrast with the shotcrete, allows for some deformation to occur before providing any pressure on the rock. Its higher deformation capability makes it more suitable for pronounced squeezing conditions. Failure may occur along overlaps where mesh sheets are joined together. This can be prevented by using zero gauge mesh straps. The welded mesh has a lower load capacity compared to shotcrete. A higher density of bolts is therefore required to overcome this limitation. A tighter reinforcement strategy results in a more uniform wall deformation limiting the excessive stretching of the mesh (Potvin & Hadjigeorgiou, 2008).

Chain link mesh is more difficult to install than welded wire mesh and its use is overall limited. Its main disadvantage compared to the welded mesh is that it tends to unravel when damaged and it is difficult to apply shotcrete through it (Hadjigeorgiou & Potvin, 2011). The chain link mesh provides a larger ductility than the welded mesh that can lead to excessive baggage loads between the reinforcement elements under pronounced squeezing conditions (Mercier-Langevin & Turcotte, 2007a). High-tensile chain link mesh has a stiffer reaction to the displacement. The installation of this mesh is labour intensive. The mesh comes in roles and needs to be manually unrolled during installation. Mechanization of the installation has been reported using a twin-boom jumbo (Louchnikov et al., 2014).

Figure 9.4 summarizes the performance of shotcrete, welded mesh and high-tensile chain link mesh published by the Western Australian School of Mines (Louchnikov et al., 2014). The tests were carried out under the same boundary conditions. The graph indicates the relatively higher load capacity of the shotcrete and its lower displacement capability. The welded mesh has relatively lower capacity with higher deformation capability.
The combined use of mesh and shotcrete has been explored in Australian mines. When mesh is embedded between shotcrete or fibercrete layers, the stiffness of the support is increased. While this approach reinforces the shotcrete layers, under large deformation the surface support will fail creating large free shotcrete blocks (Potvin & Hadjigeorgiou, 2008). The use of a stiff shotcrete layer and a subsequent installation of mesh can accommodate large deformations but increases the support cost. This may be acceptable if the required rehabilitation is significantly reduced.

9.5.2 Reinforcement

In pronounced squeezing conditions, the reinforcement can be contained entirely within the failed rock around the excavation. Under these conditions, the aim is to integrate the failed rock rather than support large unstable blocks of intact rock. Stiff bolts tend to sink into the walls and break when large deformation occurs. Bolts that allow large deformation can be more effective in pronounced squeezing conditions.

Conventional bolts can provide stiff behaviour with high load capacity or ductile behaviour with low load capacity. On the other hand, yielding bolts have high load capacity and can accommodate large deformation. The performance of different bolts in the laboratory and the field was discussed in chapter 8. Figure 9.5 summarizes the behaviour of rebars, FRS, cable bolts and hybrid bolts as captured from in-situ pull out tests at LaRonde. While the tests did not load the bolts to failure, the graph indicates the differences
in stiffness and load capacity between the examined bolt types. Resin grouted rebars and cemented grouted cable bolts have high load capacity but are stiff and cannot accommodate large deformation. The FRS have the ability to accommodate large deformations but provide low load capacity. Finally, the hybrid bolt has high load capacity and can accommodate large deformation.

![Graph showing load vs. displacement for different types of bolts](image)

Figure 9.5: In-situ pull out tests at LaRonde a) rebars, FRS, hybrid bolts and cable bolts; and b) hybrid bolts (Mercier-Langevin & Turcotte, 2007a).

Ductility in reinforcement can be achieved by partially de-bonded bolts, sliding between the bolt and the grout or sliding between the bolt and the rock. The ductility in partially de-bonded bolts is proportional to the length of the de-bonded section and the properties of the steel. Li et al. (2014) described the available energy absorbing or yielding bolts (D-Bolt, Garford bolt, Roofex bolt and Yield-Lok bolt) and reviewed their performance under pull loading. These bolts can provide an axial load capacity in excess of 100 kN with some of them exceeding 200 kN and have the ability to yield and sustain large axial deformations. Yielding bolts have the potential to manage large rock mass deformations. The inflatable bolt has also large deformation capability and has been used in squeezing conditions (Potvin & Hadjigeorgiou, 2008).

The choice of reinforcement elements for structurally controlled deformations should take into account a series of problems that may be encountered in large structurally controlled deformations. Under these conditions, the reinforcement elements are subjected to a combined pull and shear loading. Some bolts may not perform well under localized shearing. The shearing may lock the bolt and reduce its deformation capability. This mechanism was described in section 7.2 and has been observed at LaRonde in FRS, inflatable bolts and cone bolts (Mercier-Langevin & Turcotte, 2007a). Under these conditions, the FRS fail at the bolt head and the cone bolts sink into the walls. Hallow bolts such as FRS and inflatable bolts may fail in shear. Finally, as bolts are installed in broken ground, inadequate anchorage may occur due to resin escaping from the borehole. It is possible that this limitation can be overcome by using thick cement grout. In-situ pull out tests can examine whether adequate bolt anchorage is achieved.
9.6 Reinforcement design strategies and installation sequence

Potvin and Hadjigeorgiou (2008) reviewed the support practices at mines operating under squeezing rock conditions from Australian and Canadian hard rock mines, Table 9.1. For heavy squeezing (strain in excess of 10%) the support systems include bolts with high deformation capability. In cases when fibercrete is used, a wider bolting pattern is employed and weld mesh is installed on top. A tighter bolting pattern employing yielding bolts can allow the use of welded mesh. For moderate squeezing conditions (strain lower than 10%) FRS are used in conjunction with stiff cable bolts and fibercrete. Weld mesh is still necessary after the fibercrete cracks.

**Table 9.1: Support practices at mines operating under squeezing ground conditions (Potvin & Hadjigeorgiou, 2008).**

<table>
<thead>
<tr>
<th>Mine Site</th>
<th>Support</th>
<th>Mine Site</th>
<th>Support</th>
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<tbody>
<tr>
<td>Mine #1</td>
<td>Split sets (1 x 1 m); 75 mm fibrecrete + weld mesh</td>
<td>Mine #4</td>
<td>Split sets and cable bolts (1.3 x 1.5 m); 50 to 75 mm fibrecrete + weld-mesh</td>
</tr>
<tr>
<td>Mine #2</td>
<td>De-bonded bar and Swellex (1 x 1.5 m); 75 mm fibrecrete + weld-mesh + 50 mm fibrecrete</td>
<td>Mine #5</td>
<td>Split sets, resin bar and cable bolts (50 to 75 mm); fibrecrete + weld-mesh</td>
</tr>
<tr>
<td>Mine #3</td>
<td>Hybrid bolts (0.8 x 0.8 m); weld-mesh</td>
<td></td>
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</table>

Underground observations at the LaRonde Mine presented in chapter 7 suggested that the use of soft reinforcement has a minor effect on the displacement and results in excessive rehabilitation work. The use of the hybrid bolt has been justified by reducing the required rehabilitation. The bolt prevents the resin from escaping, provides a greater resistance to shear, increases the frictional resistance of the FRS and provides a stronger head to the bolt. Cable bolts can be used to reduce the displacement further.

The use of the 3D DEM in chapter 8 captured the effect of different reinforcement strategies under squeezing conditions at LaRonde. The modelled displacement for the reinforcement strategies examined is shown in Figure 9.6. When the reinforcement in the sidewalls is comprised only of FRS, the reduction on the displacement is minor. The introduction of hybrid bolts in the support system has a significant effect on the overall displacement reducing the displacement by 18%. When cable bolts are used in the support system, the reduction on the displacement is even higher (23% in total).
Figure 9.6: Modelled reduction on the displacement for different reinforcement strategies: a) FRS; b) FRS and hybrid bolts; c) FRS, hybrid bolts and cables; and d) longitudinal displacement profiles for the examined strategies.

The time of installation for each reinforcement element is critical. Mining experience has shown that for pronounced squeezing conditions, the ground support is more effective if it is installed progressively. When yielding bolts are installed early in the squeezing process, they may lock due to shearing, become stiffer, lose their yielding capability and sink into the wall. When large deformation is anticipated, the yielding bolts should be installed as a secondary support, allowing some deformation to occur without loading the initial support to failure. When additional reduction on the displacement is necessary, cable bolts can be used at later deformation stages. Underground observations and modelling results presented in section 8.7.1 showed that when cable bolts are installed early in the squeezing process, they tend to either sink into the walls or fail.
While there are no reported cases using the other yielding bolts discussed in section 9.5.2 under pronounced squeezing conditions, numerical modelling investigations can provide information on the effectiveness of bolts under such conditions. 3D DEM models indicated a potential advantage in the use of D-Bolt instead of the hybrid bolts as a secondary support strategy, Figure 9.7.

![Figure 9.7: Influence on yielding bolts on the modelled displacement: a) D-Bolts; b) hybrid bolts; and c) longitudinal displacement profiles for the examined strategies.](image)

The D-Bolt can carry higher load than the hybrid bolt and when installed as a secondary support, can provide more than 5% larger reduction on the displacement. Pull out tests should be done to ensure than the necessary anchorage is achieved for the D-Bolt. The potential impact of other rock bolts in squeezing conditions can be identified using the methodology presented in chapter 8.

Overall, there is no unique system to control large deformations. Maintaining the rock mass confinement is critical in squeezing conditions as unravelling and excessive deformation may occur otherwise. The
numerical models examined only the effect of reinforcement and did not take into account the influence of surface support. When mesh is used, a tighter reinforcement strategy that can lead to more uniform deformations is recommended. If shotcrete is used, a further layer of mesh will be necessary in cases of large deformation. This approach increases the support cost but may be justified if reduces the required rehabilitation.

For pronounced squeezing conditions, reinforcement is more effective if installed in stages. Yielding bolts that allow large deformation such as the hybrid bolt and the D-Bolt can be more effective. These bolts should be installed after some deformation has already occurred and before the primary support is loaded to failure. Stiff support such as cable bolts is more effective if installed at later deformation stages.

9.7 Summary and conclusions

This chapter presented recommendations for identifying, assessing and mitigating structurally controlled deformations in hard rock mines. In general, different rock mass failure mechanisms can lead to squeezing conditions. A buckling type of failure can occur in thinly layered rock masses. In this case, rock mass classification which assume that the rock mass behaves as continuum isotropic material cannot provide adequate information on the squeezing potential. The anticipated level of squeezing under these conditions, can be assessed using the hard rock squeezing index which takes into account the presence of a dominant fracture set in the rock mass.

Proactive mitigation measures can be taken during the design process to avoid large deformations. A choice of a higher angle of interception between the orientation of the drift and the foliation can result in more favourable conditions. Graphs that can provide a quantitative indication of the degree of squeezing based on the angle of interception have been proposed in this thesis and can assist during the design process.

Changes on the orientation of development are sometimes restricted due to the employed mining method. In these cases an effective reinforcement strategy should be adopted to manage the anticipated deformations. The aim of an effective reinforcement strategy is to control rather than arresting the deformation as significant rehabilitation may be necessary otherwise. The factors that need to be considered for an effective reinforcement and surface support strategy were presented. Stiff surface support such as fibercrete would eventually crack under large deformation and a layer of mesh would be required to provide additional support. Reinforcement elements that can sustain high load and have large deformation capability can be more effective in squeezing conditions.

There is no unique support strategy to control large deformations, the advantages of certain reinforcement strategies under pronounced squeezing conditions were discussed. Reinforcement can be more effective if
installed in stages. The use of yielding bolts as a secondary support strategy can reduce the displacement and decrease the required rehabilitation. The use of hybrid bolts has reduced the required rehabilitation in LaRonde. A potential advantage in the use of D-Bolt has been indicated from numerical modelling. Stiff reinforcement such as cable bolts can be installed later in the squeezing process to provide further reduction on the displacement. Welded mesh can sustain large deformation. When shotcrete is used a wider reinforcement strategy can be adopted. Under large deformations, a further layer of mesh will be necessary after the shotcrete cracks. While this approach increases the support cost it may be justified if it reduces the required rehabilitation.
10 Conclusions

10.1 Introduction

This chapter summarizes the work undertaken towards the management of squeezing ground conditions in hard rock mining. It identifies the main contributions of this thesis while at the same time acknowledges the limitations of this work. Finally, it provides recommendations for future work.

10.2 Motivation and originality

A large number of underground hard rock mines report squeezing ground conditions. Different failure mechanisms and ground conditions can result in large deformations but are often related with a combination of high stresses and relatively low material strength properties. In general, squeezing conditions in hard rock mines are encountered when the wall convergence reaches at least tens of centimeters within the life expectancy of a supported drive. Under these conditions, the resulting loads are usually greater than the capacity of conventional support systems. Controlling the excessive deformation is challenging, and can have severe implications on the cost effectiveness and safety of underground excavations.

In many hard rock mines squeezing conditions are related to the presence of major structural features. This thesis focuses on structurally controlled deformations in underground hard rock mines examining how these deformations can be controlled to avoid frequent rehabilitation and high support cost. This thesis is a contribution to improved understanding of structurally controlled deformations and can be used as a reference for engineers designing underground excavations in squeezing ground conditions in hard rock mines.

The primary objectives of this thesis have been to:

1) Improve our understanding of structurally defined squeezing mechanisms observed in hard rock mines; and

2) Propose ways to mitigate the resulting deformation and investigate the impact of different support strategies.

The secondary objectives have been to:

1) Develop field based empirical tools to manage structure controlled deformations; and
2) Develop 3D stress analysis procedures to capture the phenomenon and numerical tools to investigate the effect of reinforcement.

10.3 Contributions

10.3.1 Improved understanding of structurally defined squeezing mechanisms in hard rock mines

It was demonstrated that the acceptable levels of squeezing in a mining environment are considerably higher than in civil engineering tunnelling operations. This was accomplished by conducting an extensive data collection campaign of squeezing case studies at the LaRonde and Lapa mines. A database of 147 case studies was formed by a wide range of squeezing conditions. Information collected and analysed included deformation monitoring data using CMS surveys and laser measurements. In certain case studies, the deformation was monitored for a long period of time.

Some of the largest deformations reported in hard rock mining were recorded including wall-to-wall closure up to 2.5 m or wall-to-wall strain in excess of 40%. Of interest is, that despite the very large deformations, in most cases the drifts remained operational without any rehabilitation required. Large deformations, up to approximately 10% wall-to-wall strain, have been reported in civil engineering using ductile lining with yielding elements (Barla et al., 2010). Larger deformations in tunnelling most often require significant rehabilitation or re-mining of the tunnel. Overall in mining there is higher tolerance for deformation allowing more flexibility to mining operators. At the same time, however, they have to work under greater economic constraints in terms of support.

The strong influence of the geology on the level of squeezing was demonstrated using quantitative deformation data from Lapa. The combined interpretation of the geology information and the recorded strain showed that there was a clear correlation between the weak ultramafic units with low foliation spacing and high degree of alteration and the large wall deformation recorded at the case studies.

The hard rock squeezing index developed by Mercier-Langevin and Hadjigeorgiou (2011) was validated as a tool for the indication of the potential of squeezing in hard rock mines. This was done by collecting rock mass characterization data for each case study: the angle between the orientation of the excavation with respect to the foliation; the foliation spacing; the condition of the foliation planes; the strength of the rock; the presence of alteration and the encountered geological units. Higher strain values than those suggested by the index were recorded in certain case studies, probably due to large induced stresses or/and high degree of alteration.
The characteristics of the deformation of the drives over time were identified. The interpretation of the CMS surveys and the mechanical extensometer data showed that the displacement rates in the sidewalls and the back are high during the development of the drifts and drop over time. The displacement continues at a low rate over the working life of the drifts. This suggests a time dependant deformation mechanism. Stress changes due to mining activity result in high deformation rates for a certain period. Lower displacement rates appear in the back than the sidewalls for the conditions of the studied mines.

Deformations over time were measured for the first time inside the sidewalls under these conditions. This was achieved by using mechanical borehole extensometers. The instruments manufactured on site, were inspired by extensometers used for routine monitoring as roadway deformation indicators in the coal mining industry. The use of these instruments overcame the limitations of grouted electrical borehole extensometers which were damaged early in the squeezing process due to the presence of shear in the walls. While the mechanical extensometers provided lower accuracy in the measurements than the electrical extensometers, they were proven to be less susceptible to shear and allowed deformations in the sidewalls and the back to be recorded. The depth of failure recorded was up to 5 m in the walls and in excess of 5 m in the back. This is beyond the length of the primary support used at the mine.

The failure mechanism at the LaRonde and Lapa mines was described. The field observations and the interpretation of the field measurements suggested that the stress redistribution around the opening results in loading of the intact rock in a direction parallel to the foliation planes leading in contraction along the foliation and dilation orthogonal to the foliation planes. The dilation decreases the critical buckling load. As buckling occurs in the sidewalls, this process is transferred deeper into the rock mass. This interpretation is similar to the conceptual mechanism provided by Mercier-Langevin and Wilson (2013) based on their experience at the two mines. However, investigations on the displacement rates in the back and the sidewalls were in contrast with what was reported by the authors. The monitoring results suggested that the displacement in the back and the sidewalls happens simultaneously and the displacement rates follow the same trend.

10.3.2 Quantifying anticipated extreme conditions

A quantitative interpretation of the effect of the orientation of the excavation with respect to the foliation was developed. This was done by recording the angle of interception for each case study (defined as the angle between the normal to the foliation planes and the normal to the sidewall). The combined interpretation of the collected data captured the influence of the angle of interception on the resulting total strain at LaRonde and Lapa, Figure 10.1. The graphs are based on the largest strains recorded in hard rock mines for different angles of interception. They can, therefore, indicate the highest anticipated wall-to-wall
and back-to-floor strain for a given angle of interception. This is transferable to mines with similar geotechnical ground conditions as in the database. It is expected that similar trends will be observed at other mines but the actual strain may vary considerably based on local conditions.

This study evolved by recognizing that, irrespective of the employed support system, it was difficult to maintain open drifts developed sub-parallel with respect to foliation. The angle of interception is the most critical parameter controlling the deformation. The choice of a larger angle of interception can result in a more manageable squeezing level and increase the performance of an appropriate support system for squeezing ground conditions. The results were used to extend the hard rock squeezing index as a tool for assessing the anticipated deformation in structurally controlled deformation in hard rock mines.

![Figure 10.1: Influence of angle of interception ($\psi$) on resulting total strain at the LaRonde and Lapa Mines; a) total wall-to-wall strain; and b) total back-to-floor strain.](image)

10.3.3 Capturing the buckling mechanism

The observed buckling squeezing mechanism in hard rock mines was captured using the 3D DEM. The employed methodology considered the role of fractures within the rock mass. It overcame the limitations of continuum numerical modeling methods in capturing the non-linear anisotropic response of foliated rock masses to high stresses and excavations. This was achieved by progressive reduction of the forces acting at the boundaries of the excavation by a reduction factor through a series of modelling steps. This approach overcomes the computational and time constraints on modelling the development of a mining drift. It allowed for a more refined consideration of the role of fractures within the rock mass including rotation of separate blocks, opening of fractures and detachment of blocks from their initial position. The numerical modelling results were in agreement with the observed squeezing mechanism and deformation levels recorded in Canadian and Australian hard rock mines, Figure 10.2.
The extent of plastic zones around the opening indicated a direction of squeezing normal to the foliation planes. The extent of the joint slip revealed a similar pattern. The numerical model showed slip of the foliation in the top of the hanging wall and the bottom of the foot wall. Field observations by Sandy et al. (2007) and Potvin and Hadjigeorgiou (2008) suggested that this was the prevailing mechanism in several Canadian and Australian mines. The dilation of the foliation planes in the sidewalls was simulated as well as the contraction in the back and the floor due to the stress redistribution.

The numerical models successfully reproduced the rotation of the maximum horizontal stress ($\sigma_1$) in the sidewalls that results in an axial load of the foliation planes as described by Mercier-Langevin and Wilson (2013). As buckling occurs, the process propagates further into the rock mass. The rock mass failure around the excavation results in a large zone of relaxation. At the early squeezing stages, there is a high stress concentration around the face that propagates further into the rock mass later in the process. The squeezing
levels observed along drifts crossing different geological units were successfully reproduced. The influence of the foliation spacing and the in-situ stress conditions was also captured in the model.

### 10.3.4 Capturing the effect of reinforcement

The impact of reinforcement under the structurally controlled deformations observed in the field was successfully captured using the 3D DEM. The first step was to identify the influence of the ground support used at LaRonde. A comprehensive strategy for modeling rock reinforcement in squeezing ground conditions was subsequently developed extending the numerical work in chapter 6 that captured the buckling mechanism using the DEM.

The influence of different support strategies on the resulting deformation was captured based on field observations and monitoring data. Field observations and monitoring showed that the installation of hybrid bolts has a direct impact on the displacement. The installation of hybrid bolts, as a secondary support strategy, reduces the displacement and decreases the required rehabilitation.

The interpretation of data from pull out tests at LaRonde illustrated the differences in the behaviour of various bolt types in-situ. Rebar bolts provided a stiff response under axial load with limited deformation capability. The FRS showed low loading capacity with high deformation capability. The hybrid bolts demonstrated a stiff reaction with high load capacity and high deformation capability. Finally, cable bolts provided a stiff reaction to the displacement.

The 3D DEM that captured the buckling mechanism and the pressure reduction method were used to explore the impact of the reinforcement elements on the modelled displacement. Two approaches were investigated for the representation of reinforcement. The first approach introduced one row of bolts explicitly at different deformation stages in the model. The impact of reinforcement in this case was limited on the thickness of the model (0.05 m). The second approach distributed the discrete effect of the reinforcement in the out of plane direction to examine the sensitivity of the modelled displacement to any changes of the reinforcement spacing. This was achieved by scaling the material properties of the reinforcement elements (Itasca Consulting Group Inc, 2011b). Both approaches successfully captured the same trends on the impact of reinforcement elements when modelling different support strategies and were consistent with field observations. In effect, both numerical modelling approaches provided consistent indication of the relative performance of the investigated reinforcement elements.

The introduction of the structural elements in the modelled squeezing conditions showed that global reinforcement elements can be introduced successfully in the model. However, local reinforcement elements cannot be introduced in the model at later modelling stages, after significant deformation had
occurred in the model. Local reinforcement elements take into account the bending resistance of rock bolts when crossing discontinuities but cannot account for the deformation of individual blocks to the reinforcement elements and are not suitable for squeezing conditions. Global reinforcement elements consider the deformation of individual blocks with respect to the reinforcement elements but do not provide a direct bending resistance on the block interface when crossing discontinuities. The combined use of local and global reinforcement elements revealed a series of limitations.

Reinforcement elements installed at LaRonde are subjected to combined pull and shear displacement. This was illustrated by studying the joint normal and shear displacement in the model. Numerical experiments showed that under combined pull and shear, the global reinforcement elements provide similar load displacement curves. The results were similar with laboratory tests for rebars.

Following a series of numerical experiments it was possible to calibrate the global reinforcement elements to simulate the performance of rock bolts in pull conditions as observed in the field. The elements simulated the performance of FRS, hybrid bolts and cable bolts based on the in-situ pull out tests from LaRonde.

Different reinforcement scenarios were investigated based on the reinforcement strategies used at LaRonde. The scenarios included the use of rebars in the back while the performance of FRS, hybrid bolts and cable bolts in the sidewalls was examined. The effect of the time of installation of each reinforcement element was taken into account. The reinforcement elements were introduced in the model following the same sequence as used at LaRonde. The primary support comprised of rebars in the back and FRS in the walls had a minor effect on the displacement. The use hybrid bolts in the sidewalls as a secondary support strategy reduced the displacement significantly (18%). The installation of cable bolts at later deformation stages provided a further reduction to the displacement (23%), Figure 10.3. Figure 10.4 shows the longitudinal displacement profiles resulted from the different reinforcement scenarios.

The investigation of other reinforcement options indicated a potential advantage in the use of yielding bolts such as the D-Bolt. The global reinforcement elements were calibrated to reproduce the behaviour of D-Bolts as captured in in-situ pull out tests. The introduction of 1.8 m D-Bolts in the modelled squeezing conditions showed that the D-Bolts can provide more than 5% larger reduction on the displacement than the hybrid bolts, Figure 10.5. This has considerable implications in the choice of reinforcement elements. The same methodology developed in this thesis can be extended to investigate the potential behaviour of other yielding bolts such as the Yield-Lok etc. It provides the means to consistently investigate different reinforcement scenarios, and can be a prelude to comprehensive field trials.
Figure 10.3: Modelled displacement and performance of rock bolts for case 5.

Figure 10.4: Longitudinal displacement profiles illustrating the modelled effect of different reinforcement strategies on the modelled displacement.
10.3.5 Methodology for managing squeezing conditions

Recommendations for managing the structurally controlled squeezing conditions in hard rock mines were suggested. The recommendations included tools for the assessment of the anticipated deformation, proactive ways to mitigate the deformations and suggestions for effective support strategies under squeezing conditions. They were based on a comprehensive literature review, the interpretation of the field data in a range of squeezing conditions at the LaRonde and Lapa mines and the numerical modelling results using the 3D DEM. The recommendations are summarized below:
• Squeezing conditions in hard rock mines can be caused by a variety of failure mechanisms such as complete shear of the medium and sliding and shear. In thinly layered rock masses, a buckling type of failure can appear under high stresses.

• The anticipated level of squeezing in structurally controlled deformations in hard rock mines can be assessed using the hard rock squeezing index. Rock mass classifications do not take into account the anisotropy in the rock mass and cannot predict the squeezing potential of structure controlled deformations. The index considers the presence of a dominant fracture set in the rock mass and can indicate the long term expected strain levels in drifts. If the drifts are subjected to high induced stresses and/or developed in highly altered rock masses, larger deformation than the one suggested by the index may occur. The largest deformations recorded for different angles of interceptions have been presented in Figure 10.1.

• A more favourable angle of interception between the orientation of the drift and the foliation can be chosen as a proactive mitigation measure for squeezing conditions. A larger angle of interception can result in more favourable squeezing conditions. Figure 10.1 can be used to identify the optimum angle of interception during the design process. However, changes on the orientation of drifts are sometimes restricted from the employed mining method.

• An effective support strategy under squeezing conditions should manage rather than arrest the deformation as significant rehabilitation may occur otherwise. Compatible surface support and reinforcement elements in terms of their ability to deform and yield under excessive load can be more effective under squeezing conditions. Stiff surface support such as fibercrete would eventually crack under large deformations and a layer of mesh would be required to provide additional support. Reinforcement elements that can sustain high load and have large deformation capability can be more effective in squeezing conditions.

• Overall, there is no unique support strategy to control large deformations in hard rock mines. Reinforcement can be more effective if installed over time, at different stages. Yielding bolts as a secondary support can reduce the displacement and decrease the required rehabilitation. This was observed both in the field and in the numerical models. The hybrid bolt has reduced the rehabilitation at LaRonde while numerical modelling identified a potential greater advantage in the use of the D-Bolt. For pronounced squeezing conditions, stiff reinforcement such as cables is more effective if installed later in the squeezing process.
10.4 Limitations of the employed methodology

The thesis focused on structurally controlled deformation in underground hard rock mines. The derived conclusions are based on data collected from a range of squeezing conditions at the LaRonde and Lapa Mines. These mines report some of the most extreme deformations recorded in hard rock mines. This work can be a reference for engineers designing underground excavations in structurally defined squeezing conditions in hard rock mines. However, it is not readily applicable for squeezing mechanisms controlled by shearing of the medium observed in continuous ductile rock masses or in masses with widely spaced discontinuities.

Most of the limitations of the numerical modelling methodology for capturing the buckling squeezing mechanism in hard rock mines and the performance of the support under these conditions have been discussed in chapters 6 and 8. These limitations are summarized below:

- The use of the distinct element method allowed capturing the buckling mechanism at LaRonde and Lapa simulating rotation of separate blocks, opening of fractures and detachment of blocks from their initial position. The methodology, however, did not allow the explicit modelling of stress induced fractures and any rock mass unravelling between the rock bolts.

- The employed numerical models simulated the effect of reinforcement at LaRonde Mine. The impact of the surface support (mesh and straps) was not taken into account in the models.

- The global reinforcement elements used to simulate the effect of the reinforcement under squeezing conditions does not capture the doweling effect of bolts. This does not allow capturing the localized shear on reinforcement elements. The use of the global reinforcement elements was, however, justified by showing that the reinforcement elements in-situ are subjected in combined pull and shear load. Numerical tests on global reinforcement elements under combined pull and shear load indicated similar results with those observed in laboratory tests on rebars.

- The performance of bolts is influenced by the radial confinement acting on the bolt annulus. This effect was not taken into account explicitly in the models. The calibration of the structural elements with in-situ pull out tests considers the confinement on the bolt in the field implicitly. It does not, however, account for mining induced stress changes over time and their impact on radial confinement on the bolt.
10.5 Future work

Further work is required to fully capture the effect of the ground support under squeezing conditions in hard rock mining. Additional work can incorporate the effect of the surface support in the distinct element numerical models. This work would simulate the confinement provided by the surface support on the rock mass and the distribution of the load between the reinforcement elements. The potential calibration of structural liners or beam structural elements (Itasca Consulting Group Inc, 2013a) available in 3DEC can be examined to capture the effect of mesh or shotcrete in the field. A successful calibration should simulate the relative large displacement allowed by the mesh and the limited deformation capability of shotcrete under squeezing conditions.

The comprehensive numerical approach in modelling reinforcement using 3DEC included the simulation of FRS, rebars, hybrid bolts, cable bolts and D-Bolts. The methodology followed in chapter 8 can potentially be extended to the use of other reinforcement elements. The results from scaled and non-scaled input reinforcement properties indicated the same trends on the impact of reinforcement and were consistent with field observations. The use of the two approaches has to be explored further for other cases and support elements in the future to identify the best strategy for modelling a spaced pattern of reinforcement for squeezing conditions.

Structural elements that simulate the bending moments generated when bolts are subjected to shear and prevent sliding of bolts when bending occurs could provide a better representation of the dowelling effect. The global reinforcement elements used in the thesis do not capture this mechanism. The rockbolt elements (Itasca Consulting Group Inc, 2011b) provide shear resistance along the length of the bolt, same as the global reinforcement, including bending resistance and inelastic bending. These elements incorporate normal coupling springs that could be used to simulate the interaction of the grout or resin between the bolt and the rock in a direction normal to the bolt. These elements should be calibrated in pure pull and pure shear conditions and a combination of the two based on laboratory tests in the different bolt types.
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References


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Appendix A: Numerical results from the modelling of the reinforcement strategies at LaRonde Mine

Figure A.1: Modelled block displacement for the model without support (base case model).
Figure A.2: Modelled block displacement and performance of the global reinforcement elements for case 1.

Figure A.3: Modelled block displacement and performance of the global reinforcement elements for case 2.
Figure A.4: Performance of the structural elements (global and local reinforcement elements) for case 3.
Figure A.5: Performance of the structural elements (global and local reinforcement elements) for case 4.
Figure A.6: Modelled displacement and performance of the global reinforcement elements for case 5a.

Figure A.7: Modelled displacement and performance of the global reinforcement elements for case 5b.
Figure A.8: Modelled displacement and performance of the global reinforcement elements for case 5b.

Figure A.9: Modelled displacement and performance of the global reinforcement elements for case 6a.
Figure A.10: Modelled displacement and performance of the global reinforcement elements for case 6b.

Figure A.11: Modelled displacement and performance of the global reinforcement elements for case 6c.
Figure A.12: Modelled displacement and performance of the global reinforcement elements for case 7a.

Figure A.13: Modelled displacement and performance of the global reinforcement elements when cable bolts were installed as secondary support in case 6b instead of hybrid bolts.
Figure A.14: Modelled displacement and performance of the global reinforcement elements for case 8.
Figure A.15: Modelled displacement and performance of the rock bolts for case 5a using scaled and non-scaled material properties.
Figure A.16: Modelled displacement and performance of the rock bolts for case 5b using scaled and non-scaled material properties.
Figure A.17: Modelled displacement and performance of the rock bolts for case 8 using scaled and non-scaled material properties.