APPLICATION OF GEOTECHNICAL NUMERICAL MODELLING OF SILL PILLAR MINING OPTIONS TO AID VALUE-DRIVEN DECISION-MAKING

by

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B.A.Sc., The University of British Columbia, 2012

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The following individuals certify that they have read and recommend to the Faculty of Graduate and Postdoctoral Studies for acceptance, a thesis entitled:

Application of Geotechnical Numerical Modelling of Sill Pillar Mining Options to Aid Value-Driven Decision-Making

submitted by  Craig James Archibald  in partial fulfillment of the requirements for the degree of  Master of Applied Science  in  Mining Engineering

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Abstract

Sill pillars divide vertically extensive orebodies into mining blocks to allow concurrent mining operations. Sill pillars also provide regional stability to a mining district and are usually one of the last volumes of ore to be mined. The initial design for sill pillar mining is often only high level, made without knowledge of the ground performance. The final design and planning occurs many years or decades after the initial design was made, which did not account for operational learnings. This thesis describes the decision process for positioning the overcut development within a sill pillar while considering operational and geotechnical issues.

There are three possible scenarios for the placement of the overcut development: in the sill pillar, in the backfilled stope directly above the sill pillar, or a combination of the sill pillar and backfill. Numerical modelling (finite element) was used to aid in the decision-making process for the location of the overcut development. In addition, this modelling was used to understand the potential failure mechanisms and forecast the mining conditions during development and production in a sill pillar. Results from the numerical modelling have shown that mining in the adjacent blocks caused a decrease in stress in the sill pillar. Potential failure mechanisms include the deformation and unravelling of the overcut development and the failure of the longhole stope walls due to this decrease in stress.

To apply to an operating mine, the numerical model must be of sufficient detail to capture the mechanics of the problem yet be completed in a timeframe to support the decision-making process. Each of the operational strengths and weaknesses of the overcut development scenarios were evaluated alongside the numerical models to understand the implications of each development scenario. Using a combination of numerical modelling and operational
considerations, the location of the overcut development can be decided to maintain safe access for underground personnel during the recovery of the sill pillar.
Lay Summary

Rock pillars are left to increase safety during the production of an underground mine and may be removed near the end of the mine life. Mining a pillar can create regional instability, and understanding the risks is essential so that these factors can be mitigated, and the overall level of risk is reduced. This thesis uses numerical modelling to explain the ground response to removing pillars between adjacent mining areas. The numerical modelling results can be used in conjunction with operational considerations to aid in the decision-making process of locating the development in the sill pillar. The location of the development can significantly increase the safety of personnel during the mining of the pillar.
Preface

The thesis is an original and independent work completed by the author. The author presented Chapters 3, 4, and 5 of this thesis at the Canadian Institute of Mining, Metallurgy and Petroleum (CIM) 2023 conference.
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<th>Description</th>
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<tr>
<td>a</td>
<td>Hoek-Brown Material Constant (Rock Mass)</td>
</tr>
<tr>
<td>D</td>
<td>Disturbance Factor</td>
</tr>
<tr>
<td>E, E_i</td>
<td>Young’s Modulus</td>
</tr>
<tr>
<td>m</td>
<td>Hoek-Brown Material Constant</td>
</tr>
<tr>
<td>m_b</td>
<td>Hoek-Brown Material Constant (Rock Mass)</td>
</tr>
<tr>
<td>m_i</td>
<td>Hoek-Brown Material Constant (Intact)</td>
</tr>
<tr>
<td>s</td>
<td>Hoek-Brown Material Constant (Rock Mass)</td>
</tr>
<tr>
<td>v</td>
<td>Poisson’s Ratio</td>
</tr>
<tr>
<td>( \rho )</td>
<td>Density</td>
</tr>
<tr>
<td>( \sigma_1 )</td>
<td>Major Principal Stress</td>
</tr>
<tr>
<td>( \sigma_2 )</td>
<td>Intermediate Principal Stress</td>
</tr>
<tr>
<td>( \sigma_3 )</td>
<td>Minor Principal Stress</td>
</tr>
<tr>
<td>( \sigma_{ci} )</td>
<td>Intact Unconfined (or Uniaxial) Compressive Strength</td>
</tr>
<tr>
<td>( \sigma_n )</td>
<td>Normal Stress</td>
</tr>
<tr>
<td>( \sigma_t )</td>
<td>Tensile Strength</td>
</tr>
<tr>
<td>( \sigma_{im} )</td>
<td>Rock Mass Tensile Strength</td>
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List of Abbreviations

<table>
<thead>
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<th>Abbreviation</th>
<th>Description</th>
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<tr>
<td>2D, 3D</td>
<td>Two-Dimensional, Three-Dimensional</td>
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<tr>
<td>BEM</td>
<td>Boundary Element Method</td>
</tr>
<tr>
<td>CRF</td>
<td>Cemented Rockfill</td>
</tr>
<tr>
<td>DEM</td>
<td>Discrete Element Method</td>
</tr>
<tr>
<td>DWG</td>
<td>Dewatering Gallery</td>
</tr>
<tr>
<td>FDM</td>
<td>Finite Difference Method</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite Element Method</td>
</tr>
<tr>
<td>FLAC</td>
<td>Fast Lagrangian Analysis of Continua (Itasca)</td>
</tr>
<tr>
<td>GSI</td>
<td>Geological Strength Index</td>
</tr>
<tr>
<td>HD</td>
<td>Haulage Drift</td>
</tr>
<tr>
<td>MPa</td>
<td>Megapascal</td>
</tr>
<tr>
<td>MK</td>
<td>Magnetic Lapilli-Rich Macrocystic Volcaniclastic Kimberlite</td>
</tr>
<tr>
<td>REV</td>
<td>Representative Elementary Volume</td>
</tr>
<tr>
<td>SLR</td>
<td>Sub-level Retreat</td>
</tr>
<tr>
<td>UCS</td>
<td>Unconfined (or Uniaxial) Compressive Strength</td>
</tr>
<tr>
<td>URF</td>
<td>Uncemented Rockfill</td>
</tr>
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Chapter 1: Introduction

1.1 Background

The main objective of mining is to extract as much of the orebody as safely and economically as possible, which is aided in underground mining by leaving pillars (Jessu, 2018). Pillars are designed to maintain the stability between underground mine openings, providing for safe extraction, and can separate adjacent mining areas, allowing for concurrent mining. Pillars that separate adjacent mining areas prevent operational and geotechnical interactions, but the adjacent mining damages the rockmass of the pillar. Pillar recovery involves the extraction of the pillars between adjacent mining areas after primary mining has taken place and inherently creates a complex situation.

Numerical modelling provides a pathway to analyze complex situations. The use of realistic geological and geotechnical parameters in advanced numerical techniques, results in valuable insights into the stress distribution, deformation pattern, and potential failure mechanism within the rockmass. Different scenarios can be explored to assess the impact on pillar stability for a systematic evaluation of the interplay between safety considerations and mineral extraction. Numerical modelling serves as a powerful decision-support tool by identifying optimal recovery strategies and understanding the limits of pillar integrity.

1.2 Research Objectives

A sill pillar separates two adjacent mining horizons to allow concurrent mining operations while providing regional stability. The initial design for sill pillar mining is often only high level, made without knowledge of the ground performance. The final design and planning occurs many years or decades after the initial design was made, and cannot not account for operational learnings.
The objective of this thesis is to describe the decision-making process for positioning the overcut development within a sill pillar while considering operational and geotechnical issues.

![Figure 1.1 Cross-section showing the sill pillar with development (option 3)](image)

The main objectives of this thesis include:

- Use numerical modelling to inform the decision-making process by comparing three different development options for mining the sill pillar.
- Determine the optimal development option by outlining the operational considerations for each development option and discussing the risks, opportunities, and mitigating strategies.

To achieve these objectives, the following tasks were completed:

- Develop a numerical model that is easily replicable at an operational level to compare the three development options. This includes simplifying the geological setting and mining sequence, and calculating material parameters from laboratory data.
- Complete an assessment of the three development options considering the failure mechanisms and ground reaction identified in the numerical modelling. Define the
operational controls that will prevent these failure mechanisms from occurring or mitigation strategies that will reduce the risk to operational personnel.

1.3 Thesis Organization

This thesis is organized into six chapters, including the introductory chapter and the conclusions. Chapter 2 presents a literature review of the concepts and techniques to evaluate the recovery of a sill pillar. It includes information about pillar design, failure mechanisms, and numerical modelling techniques. It concludes with a couple of case studies of numerical modelling of sill pillars.

Chapter 3 describes the sill pillar arrangement/layout of the Diavik Diamond Mine and outlines the three development options for recovery of the sill pillar. The chapter outlines the four failure mechanisms to be controlled to ensure the safe recovery of the sill pillar.

Chapter 4 defines the development of a finite element model to compare the three development options. The input parameters for the numerical model are outlined and justified along with the numerical modelling parameters.

Chapter 5 examines the site-specific conditions of each option, how numerical modelling of the scenarios identifies the geotechnical risks, and the operational considerations for each scenario. The chapter concludes with a comparative ranking of the three development options to determine the preferred option for the sill pillar development.

Chapter 6 summarizes the research completed, key findings and conclusions, limitations and assumptions, and recommendations for future work.
Chapter 2: Literature Review

2.1 Introduction

Chapter 2 reviews the main concepts and techniques used to evaluate the recovery of a sill pillar. Three areas were researched in the literature review:

i. Pillar design and failure mechanisms.

ii. Numerical modelling techniques, to understand what techniques are most applicable to the problem being investigated.

iii. Case studies of numerical modelling of sill pillars.

2.2 Pillar Mining and Pillar Failure

2.2.1 Pillar Scale and Geometry

Pillars are designed to maintain the stability between underground mine openings and can be separated into two groups depending on the scale of the pillar (Jessu, 2018):

- Regional support pillars,
- Local support pillars.

Regional support pillars separate mining districts or essential infrastructure and provide mine scale stability. A barrier pillar is used in horizontal orebodies to separate mining districts and prevent interactions on the same mining horizon. A crown pillar is used in vertical orebodies to separate the underground mine workings from interaction with surface structures. A shaft pillar is a vertical pillar that divides the underground mine workings from immediate infrastructure, such as a mine shaft.
Local support pillars are smaller pillars that separate mining areas and are designed to maintain stability during production activities. Square or irregular pillars are used in flat-lying or gently dipping orebodies as primary support between development drifts. Vertical orebodies have two types of pillars to separate adjacent mining areas and to prevent interactions between production stopes. A sill pillar is a horizontal pillar separating stopes of two different mining horizons, forming the back or sill of a stope. A rib pillar is a vertical pillar that separates stopes of the same mining horizon and forms the walls of a production stope.

Table 2.1 Types of mining pillars

<table>
<thead>
<tr>
<th>Orebody Geometry \ Pillar Scale</th>
<th>Regional</th>
<th>Local</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal Orebody</td>
<td>Barrier</td>
<td>Square / Irregular</td>
</tr>
<tr>
<td>Vertical Orebody</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Horizontal Pillar</td>
<td>Crown</td>
<td>Sill</td>
</tr>
<tr>
<td>Vertical Pillar</td>
<td>Shaft</td>
<td>Rib</td>
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2.2.2 Pillar Failure Mechanisms

Pillar recovery creates an inherently unstable situation, and safe pillar recovery requires global stability and local stability (Mark et al., 2003; Mark & Gauna, 2017). Global stability is the prevention of section-wide pillar failure, and local stability is the prevention of roof falls in the working area. Mark et al. (2003, 2017) identified that a proper pillar design is critical to ensuring global stability, ensuring pillars can carry the loads applied to them. Local stability is controlled by sequencing and ground support. Ghasemi et al. (2010) created a risk assessment chart to identify the most hazardous areas of a pillar recovery mining operation. They noted that rock quality and mine depth are the most essential uncontrollable parameters leading to a back failure or pillar instability. Back support, excavation size and age, and pillar ratios are controllable parameters that may influence the overall risk of pillar recovery. Pillar recovery in a room and pillar coal mine differs from the recovery of a sill pillar between mining blocks of a hard rock.
mining operation. However, the significant risks, controls, and experience gained from a room and pillar operation can be used to understand how to mitigate these risk factors, and the overall level of risk can be minimized.

Carter (2014) investigated the design and failure of pillars and found several instances of sill pillar collapses. Very few pillars show failure due to the sill pillar cracking in the center or a natural major feature such as a fault or shear zone; most seem to occur along the hanging wall and footwall of the orebody, which causes dislocation or sliding. Carter (2014) identified that pillar failures can be separated into five principal failure mechanisms:

- **Plug failure**: a sudden fall of the crown pillar by gravity into the stope, typically delineated by its boundary planes or a major structure.
- **Unravelling**: a block-by-block rockmass failure where a self-supporting cavity is not reached, caused by a jointed rockmass that is not properly confined.
- **Chimneying**: upwards progression within a weak rockmass, forming a cavity that does not meet a naturally stable dome.
- **Caving**: a failure of the rockmass along persistent discontinuities, which forms blocks of small size.
- **Delamination**: sliding or buckling of thin horizontal structure within the pillar; one layer opens, causing the back to drop, allowing subsequent layers to start buckling.

Figure 2.1 presents an illustration of each of the five principal failure mechanisms.
The failure mechanism depends on the in-situ rockmass, and three approaches can be taken for evaluating the stability of pillars: empirical methods, structural analysis and cavability assessments, and numerical modelling procedures (Carter, 2014). Carter (2014) and McKinnon (2013) agree that the failure mechanism must be understood before a numerical modelling code is chosen, and the numerical modelling code must be capable of handling the correct failure mechanism.

2.3 Numerical Modelling Methods

“Numerical modelling code will be chosen based on user experience, cost, and time and must be chosen to assess the underlying mechanics of the problem” (McKinnon, 2013). McKinnon outlined three general requirements for the choice of a code:

- Sufficiently represent the relevant material behaviour,
- Represent the required boundary conditions,
- Enable realistic failure mechanisms to develop.

Numerical methods in rock mechanics can be classified into continuum, discontinuum, and hybrid models (Jing & Hudson, 2002). A continuum method divides the rockmass into a set of
simple sub-domains called “elements” and assumes that the elements of the model share continuity of displacements and stresses. This method shows an accurate representation of complex geometries and includes dissimilar materials. Common continuum methods include boundary element, finite element, and finite difference. A discontinuum method represents the rockmass as an assemblage of distinct interacting blocks, allowing for block deformation and relative movement, and can model complex behaviour and mechanisms. (Hoek et al., 1991). A common discontinuum method is the discrete/distinct element method. A hybrid model will combine two methods, allowing for the modelling of specific behaviours better represented by different methods.

2.3.1 Boundary Element Method

The boundary element method is a continuum method that focuses on the domain boundary and is more suitable for homogenous and isotropic material properties (Hoek et al., 1991). The boundary element method uses stress and displacement boundary condition and can provide stress and displacements within the domain. The main advantage of the boundary element method is reducing the computation model dimension by one, simplifying the mesh generation. This reduction allows for a more straightforward calculation while being more accurate than the other continuum methods (Jing, 2003).

2.3.2 Finite Element Method

The finite element method is a continuum method that breaks the domain into elements, connected at nodes, and uses global equilibrium to yield a system of equations for the displacement at the nodes. Stresses and strain within an element are calculated given the displacement at the element nodes. The finite element method has flexibility in handling material
heterogeneity and anisotropy, complex boundary conditions and dynamic problems (Jing, 2003). This method is unsuitable for highly jointed-blocky media (Coggan et al., 2012).

2.3.3 Finite Difference Method

The finite difference method is a continuum method which overlays a regular grid of nodes over the domain, and approximates the derivatives of the equation of equilibrium at a node by the finite differences using values from adjacent nodes. The result is a system of equations for displacement at each node, and given these displacement, strains and stresses may be calculated. The conventional finite difference method is inflexible when dealing with fractures, complex boundary conditions, and material heterogeneity, making it generally unsuitable for modelling rock mechanics problems (Jing, 2003).

2.3.4 Discrete / Distinct Element Method

The discrete element method is a discontinuum method where the domain is broken into deformable blocks which can move in relation to the surrounding blocks. Individual blocks may be free to rotate and translate, and the deformation of the contacts is more significant than the deformation of the intact rock (Hoek et al., 1991). Identifying the appropriate contact properties can be difficult due to limited data from borehole logging or underground/surface mapping. A synthetic fracture system is generated that shares the sampled properties from the logging and mapping (Jing, 2003).

2.3.5 Hybrid Models

The objective of a hybrid method is to combine two methods to eliminate as many undesirable characteristics as possible while retaining as many advantages as possible (Hoek et al., 1991). Hybrid models do this by separating the model into a far-field domain and a near-field domain,
and using different numerical methods for each area, allowing the modelling of specific behaviours. One common type of hybrid model typically uses a boundary element for simulating the far-field and a finite element or discrete element for the near-field representation of behaviour (Jing, 2003).

2.3.6 Selection of Modelling Method

“The need to use continuum or discontinuum methods depends on the size and scale of discontinuities to those of the problem under consideration” (Sari, 2021). The size of a structure that will affect an underground crusher chamber will be significantly smaller than that of a structure that will affect a pit slope. Since many length scales of structures and excavations affect mines, it is impossible to represent all structural discontinuities in a numerical model (Beck et al., 2009). An essential task in numerical modelling is establishing the length scale in which a rockmass can be treated as homogeneous (Witherspoon et al., 1981, as cited in Beck et al., 2013) which determines when a continuum or discontinuum method is applicable. Sari (2021) outlines that the representative elementary volume (REV) value can be calculated by dividing the representative dimension of an engineering structure by the mean spacing of joint sets. The rockmass can be treated as continuous, homogeneous, and isotropic if the calculated value equals or exceeds a threshold value. Sari (2021) summarized that there is no definitive threshold REV value, but a comparison of REV size ranges proposed by different researchers indicates a commonly reported range between five and ten.
2.4 Case Studies of Comparisons using Numerical Modelling

This section investigates previous studies where numerical modelling was used to compare multiple options for pillar mining. These previous studies were reviewed to determine different evaluation characteristics that can be used in the numerical modelling of pillar mining.

2.4.1 Mine A – Hard Rock Stope using Discrete Element Modelling

The Louvicourt mine in Quebec, Canada, was a polymetallic orebody using a primary/secondary mining sequence with longhole stoping and paste backfill (Doucet et al., 2006). Two sill pillars were created under the paste backfill, with the rockmass being described as highly fractured because of high mining stresses. Access to the top of the sill pillars was achieved using two different methods: drifting through the paste and drifting under the paste. Drifting through the paste was used where the main drift was located close to the ore reserves and helped reduce risks associated with elevated ground stresses and seismicity. Drifting under the paste was completed in ore but through the highly fractured rockmass. Support of drifts in both methods was achieved using a shotcrete shell, two layers of shotcrete with weld wire mesh between the shotcrete layers. Numerical modelling was used to determine the stope length, with the stope width fixed at fifteen meters (Doucet et al., 2006). The primary stope design was simple, with the numerical modelling showing that the back was stable if the drift pillars remained stable and shotcrete was used. Three modified configurations were modelled for the secondary stopes using 3DEC by Andrieux and O’Connor (2004, as cited in Doucet et al., 2006). Of the three scenarios proposed, it was decided that a four-meter-thick arched skin floor pillar would be left under the sill pillar drift (Doucet et al., 2006).

Primary stoping was considered a general success due to optimizations of blasting practices and the design of support in the drifts (Doucet et al., 2006). Secondary stoping was generally
successful, but problems were encountered, primarily associated with rehabilitating the drifts after blasting. Rehabilitation required additional time to install screen or shotcrete, impacting the mining sequence and preventing other stopes from being mined. The mining plan for secondary stopes was revised, and an action plan was established to identify rehabilitation needs ahead of production, allowing for higher reserve recovery rates than initially planned.

2.4.2 Mine B – Stope Sequence Optimization using 3D Finite Element Modelling

Xu (2021) presented a three-dimension finite element model of a longhole stoping operation and their assessment of the optimal recovery of two sill pillars. The orebody was split into three mining blocks adjacent to the two sill pillars that would be recovered with these blocks, and these blocks were stoped and backfilled according to the mine plan. The model was calibrated using reflective prisms on the surface to align the subsidence between the model and the mine site. Three different sill pillar mining sequences were modelled. The base case was to mine the two sill pillars concurrently, while the other two cases involved mining each sill pillar sequentially. The objective was to determine if the sequence of mining the two sill pillars caused an adverse stress change that would impact recovery.

Xu (2021) used two parameters to determine which of the three sequences was optimal for the sill pillar recovery, prism movement and stress concentration. The movement of the surface prisms was compared in all three models to determine if one sequence caused higher levels of movement than the others. In all cases, the movement of the surface reflective prisms was less than what would be considered significant for this analysis, and there was no difference between the three mining sequences. In their work, the author investigated the rock-bursting potential using the tangential stress criterion and the energy-based burst potential index criterion of three headings in the sill pillar. One heading was located near the center of the sill pillar, and the other
two were near the northern and southern pipe contacts. The stress concentration was determined at the four corners of each heading. It was determined that there is a higher stress concentration near the pipe contacts than in the center of the pipe.

Xu (2021) concluded that there was no significant difference between the three mining sequences, and mining both sill pillars simultaneously was the optimal sequence.

2.5 Chapter Summary

Pillars are designed to maintain the stability between underground mine openings, and a sill pillar prevents mining interactions between two mining horizons. Mining a pillar creates an unstable situation; therefore, mining a sill pillar occurs after mining in neighbouring mining horizons has been completed. Understanding the risks of this hazardous situation and the relevant failure mechanisms that could lead to a failure helps reduce the overall risk level. Numerical modelling can aid in understanding the risks with sill pillar recovery. Still, the chosen numerical model must accurately represent the rockmass and the failure mechanism. Numerical modelling has been used to model different pillar recovery problems, and lessons such as the impact of mine sequencing can be gleaned and applied to determine the best pillar recovery option.
Chapter 3: Diavik Sill Pillar Arrangement/Layout

3.1 Diavik Diamond Mine

Diavik Diamond Mine is in Canada's Northwest Territories, approximately 300 km Northeast of the city of Yellowknife. Located on an island, the Diavik Diamond Mine is a self-contained community that generates electricity and potable water, manages wastes, and maintains emergency and medical services (Yip & Pollock, 2017).

Three active kimberlite pipes are located near the island: A21, A154N, A154S, with one completed pipe: A418. Two dikes were built to allow open pit mining to start in 2003 in the A154S and A418 pipes. An additional dike was built in 2017 for open pit mining of A21 pipe. Underground development began in 2007, with underground production commencing in 2012 to recover the remainder of the A418, A154S, and A154N pipes. Underground development began in 2023 to recover the remainder of the A21 pipe. The sub-level retreat (SLR) mining method is used for mining the A418, A154S and A21 pipes, while longhole stoping with cemented rockfill (CRF) is used for the A154N due to its proximity underneath the dike.

Figure 3.1 Diavik Diamond Mine surrounded by Lac de Gras (Yip & Pollock, 2017)
3.2 Underground Mining – A154N

The A154N orebody has been divided horizontally into four mining blocks to allow for production on multiple levels. Figure 3.2 shows the four main mining blocks of the A154N kimberlite pipe: A, B, C, and D. Each mining block is divided into wide vertical slices called ‘stopelines’ (Lewis, Auld, & Karami, 2017). Development of each stopeline starts at the bottom of a mining block and progresses upward to the top level of the mining block. Stopes are mined in sequence between two levels; the length and width of the stopes depend on the mining sequence and the ground conditions. Stopes are blasted in sequence and are backfilled before blasting the next stope in the stopeline. Once stopes on one level are complete, stoping moves up one level and begins again.

The upper two mining blocks (A & B) are extracted in a primary/secondary sequence with a stope width of 7.5 meters (Figure 3.3). The primary stopes are developed and mined, leaving a 7.5-meter kimberlite pillar between primary stopelines. As mining of primary stopes progressed up each block, mining of the 7.5-meter kimberlite pillar, or secondary stopes, followed. The primary/secondary sequence used in A & B blocks was altered for C-Block due to geotechnical concerns. For C-Block, the stope width was increased to 15 meters to maximize ore recovery relative to development meters. The mining sequence was also changed to a more continuous mining front, unlike the previous primary/secondary stoping sequence used in A & B blocks. The width of the development in C-Block was decreased to five meters to maintain drift stability during development and production. During stoping, a five-meter overhang or shoulder is created in each drift wall with secondary support being installed in the drift to maintain these shoulders. C & D mining blocks are being recovered using 15-meter-wide stopes (Figure 3.5).
Figure 3.2 A154N stoping blocks - view looking west
Figure 3.3 A & B block stopes – 7.5-meters wide

Figure 3.4 B-block - 7.5-meter-wide stope
Figure 3.5 C & D block stopes – 15-meters wide

Figure 3.6 C-block - 15-meter-wide stope
A 20-meter-high sill pillar (one level) is left between mining blocks to prevent mining interactions. The mining plan includes recovering these sill pillars near the end of the mine life and is the focus of this thesis.

### 3.3 Sill Pillar Options

Mining of sill pillars will not commence until near the end of the mine life to reduce the potential of destabilizing the block above each sill pillar. Stopes are designed to be 10 meters wide to increase recovery in one pass while maintaining a stable geometry above the stope shoulder. The overcut drifts will be developed near previously blasted and backfilled stopes to allow for longhole drilling and backfilling of the sill pillar stopes. These drifts will be excavated using an arched back profile, which has had good stability performance in previous kimberlite and CRF excavations. There are three options for developing the overcut drifts: development in CRF, development in kimberlite, and development through a combination of CRF and kimberlite. These options are described in more detail in the following sections.

#### 3.3.1 Option 1 – Development in CRF

Development in CRF would align the sill of the new development drift with the bottom of the backfilled stope. CRF forms the back and walls with kimberlite in the floor (Figure 3.7).
3.3.2 Option 2 – Development in Kimberlite

Development in kimberlite would ramp down from the contact to align the back of the new development drift at the bottom of the backfilled stopes. The grade of the drift would change to align the back of the drift with the CRF, and the walls and floor are in kimberlite (Figure 3.8).

3.3.3 Option 3 – Development in CRF & Kimberlite

This development option divides the face of the drift into two meters of CRF and three meters of kimberlite. The bottom of the backfilled stopes forms the upper walls and the back of the overcut drift, and the floor is kimberlite (Figure 3.9).
3.4 Risk Assessment

A risk assessment was completed by technical and operational personnel to conceptualize the hazards that may occur during sill pillar mining. The sequence of events was reviewed rather than individual options, as each of the three options would have similar hazards. As a result, four significant hazards or failure mechanisms were identified during the risk assessment that need to be controlled:

- Failure of the back during development,
- Failure of the stope walls/shoulder during production,
- Failure of the support elements due to changes in stress,
- Instability of the mining block above the sill pillar.

3.4.1 Failure of the Back during Development

Overcut development for sill pillar mining will occur near previously backfilled CRF stopes. The CRF placed into these stopes was designed for future entry for mining of the sill pillar. The failure types of concern would be the unravelling, chimneying, and caving of the backfilled CRF. Therefore, back failure during development could occur in three scenarios: unconsolidated material near the bottom of the stope, widely segregated material due to quality and placement, and poor-quality material that does not meet design specifications.

- Unconsolidated material may occur if the material falls from the stope walls during backfilling. Scans of the open stopes were completed before backfilling commenced to confirm that the stope was empty, but interim scans were not completed. Therefore, the location of unconsolidated material is unknown and may cause the unravelling of the back across the drift or a chimneying failure along one of the sides of the drift. Figure
3.10 presents a long section identifying the location of unconsolidated material along the bottom of the stope, the back of the development drift.

- Widely segregated CRF would occur near the far end of a backfilled stope where larger fragments from end dumped CRF would roll and collect at the bottom. The cement slurry would not bond the larger fragments and would act as an unconsolidated material that would fall out or unravel during development if encountered. Figure 3.10 presents a long section identifying the location of segregated CRF at the end of a production stope.

- Poor quality CRF that did not meet design specifications would lead to unravelling of the back or caving of the backfilled stope. Quality control is completed at the backfill plant and the underground CRF dump, which checks the strengths after production and before deposition into a stope. Therefore, identification of poor quality CRF can be made before development begins.

*Figure 3.10 Unconsolidated material failure mechanism (option 2)*
3.4.2 Failure of Stope Walls/Shoulders during Production

Failure of the stope walls is governed by the material strength of the unsupported walls, which are managed by the exposure length of each stope, the shorter the stope length, the lower the risk of a wall failure. The length of a stope is determined by assessing the ground conditions and comparing the stability against an empirical stability chart of known stable lengths. This assessment is completed to reduce the likelihood of the stope walls unravelling, causing a failure of the stope shoulders and the material above the excavation as presented in Figure 3.11.

The stope shoulders are the undercut walls of the overcut drift, created because the production stope is wider than the overcut drift. Different support standards are used depending on the width of the stope shoulder. For instance, a 10-meter-wide stope has a 2.5-meter shoulder supported through longer tendons embedded outside the boundaries of the production stope. These tendons act in shear to prevent the gravity-driven failure of the stope shoulder. The shoulders of a 15-meter-wide stope are too long to be supported with a standard support tendon, and therefore, pre-tensioned cables are used to increase the normal forces in the stope shoulder.

![Figure 3.11 Stope wall/shoulder failure mechanism (option 3)](image-url)
3.4.3 Failure of Support Elements due to Changes in Stresses

The risk assessment identified that support elements may be impacted due to changes in stress that occur during production mining. Three locations have been identified where this type of failure may occur:

- At the granite/kimberlite contact. Movement along the granite/kimberlite contact has caused failures within the wide granite intersections due to the failure of the support tendons. Cablebolts were installed, increasing the support length and capacity in these areas, and instrumentation was installed to monitor ground movement. This failure mechanism is located outside of the sill pillar and is not within the scope of this thesis.

- At the adjacent undercut drifts. Changes in the stress regime due to longhole stoping have previously caused shearing of the support tendons in the backs of adjacent drifts. A secondary support regime of a solid bar tendon, such as a rebar or cablebolt, has been implemented as part of the standard support requirements to prevent shearing. Figure 3.12 presents the contours of maximum shear plastic strain concentrating in the back of adjacent undercut drifts. Shear strain is the ratio of the change in deformation to its original length perpendicular to the axes of the member due to shear stress.

- Within the overcut drifts during production. The strength of the CRF above the sill pillar was designed to prevent failure due to the stress changes caused by longhole stoping (Golder Associates, 2011). Failure of the CRF would lead to unravelling and bulking in the back and walls of the drift. Primary surface support of shotcrete creates a shell in the development drift, increasing the confinement of the CRF and preventing unravelling.
Standard support designs have already been developed and implemented to reduce the likelihood of these failure mechanisms occurring.

![Figure 3.12 Contours of maximum shear plastic strain showing concentrations in the back of undercut drifts](image)

### 3.4.4 Instability in Sill Pillar and Mining Block above Sill Pillar

Sill pillars prevent interactions between different mining blocks, and mining a sill pillar may cause local or regional instability in the block above each pillar. Local or regional instability could lead to unravelling, chimneying, or caving of the backfill above the sill pillar. The high strength backfill above each kimberlite sill pillar was designed to prevent the following two issues:

- A small-scale failure causing local instability and the potential loss of mining in one heading. Figure 3.13 presents a failure in the left wall of a production stope and pillar that impacts the production drift. This area would be stabilized though backfilling of the stope. Local stability is controlled by the strength of the backfill in the CRF pillar and the stope stability.
A significant failure that spans multiple headings causes regional instability, impacts mining in the block above the sill pillar and future stoping in the stopeline. Figure 3.14 presents a significant regional-scale unravelling failure caused by a small-scale wall failure. The small failure impacts the rib pillar, and as it fails, it continues to unravel into the CRF pillar. Regional stability is controlled by the strength of the backfill and the production sequence.
Chapter 4: Sill Pillar Numerical Model Set-Up Parameters

4.1 Introduction

This chapter outlines the numerical modelling set-up parameters used in this thesis and will discuss the material properties used for the kimberlite and the backfill materials. It will identify the numerical modelling code used and why it applies to this case study. The parameters for the numerical modelling will be defined. This chapter concludes with a summary of the outcomes from the numerical modelling and how they may affect operational activities.

4.2 Geotechnical Parameters

Pipe contacts, lithologies and geotechnical parameters are determined through core logging, laboratory testing and underground mapping. Geotechnical domains are defined by grouping areas of similar geotechnical properties closely aligning with the kimberlite lithologies. The sill pillars comprise a magnetic lapilli-rich macrocrystic volcaniclastic kimberlite (MK). MK was ejected during a volcanic eruption and then reformed by flowing back into the open crater that remained after the volcanic eruption (Diavik Technical, 2021). MK is characterized by its highly competent nature compared with other lithological units in the pipe and will be used in modelling the recovery of the sill pillar.

4.2.1 Tensile Strength

The tensile strength of the samples was determined using the indirect tensile strength test, also known as the Brazilian test, which indirectly determines the tensile strength in megapascals (MPa) by using the test's sample dimensions and maximum load. Twelve samples have been tested in MK with an average tensile strength of 5.5 MPa (Figure 4.1).
4.2.2 Uniaxial Compressive Strength

Uniaxial compressive strength (UCS) tests have been conducted for the various lithological units, with the offsite testing being completed by the University of Alberta Rock Mechanics Laboratory (2014 – 2019). Over forty UCS tests have been conducted on MK, corrected to an equivalent UCS with a core diameter of 50 mm. The average corrected uniaxial compressive strength of MK is 75 MPa (Figure 4.2).
4.2.3 Young’s Modulus

Young’s Modulus is a property that indicates the stiffness of the rock. Young's Modulus was determined in conjunction with the UCS testing completed by the University of Alberta (2014 - 2019). The average Young’s Modulus for MK is 17 GPa (Figure 4.3).

![Figure 4.3 Young's Modulus of MK]

4.2.4 Poisson’s Ratio

The Poisson ratio measures the relative expansion and compression in the axial and radial direction of the sample and is a dimensionless value between 0.0 and 0.5. The Poisson’s Ratio was determined in conjunction with the UCS testing completed by the University of Alberta (2014 - 2019). The average Poisson’s Ratio for MK is 0.25 (Figure 4.4).

![Figure 4.4 Poisson's ratio of MK]
4.2.5 Triaxial Test & Failure Criterion

The triaxial test determines the shear components of a rockmass by applying a constant lateral (confining) fluid pressure and then applying an axial load in the same manner as in a UCS test. At least three confining pressures can be plotted vs. the axial loads to determine the Hoek-Brown fit (Figure 4.5). Hoek & Brown (2018) identified that using all the available uniaxial compressive strength data may result in a significant bias. Therefore, an average of UCS data should be used to represent the value of the principal stress at zero confining stress. The unconfined compressive strength ($\sigma_{ci}$) and the intact material constant ($m_i$) can be calculated from the triaxial data-set, presented in Table 4.1.

![Figure 4.5 Hoek-Brown failure envelope](image)

**Table 4.1 Material constant parameter from the testing dataset**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined Compressive Strength ($\sigma_{ci}$)</td>
<td>62 MPa</td>
</tr>
<tr>
<td>Material Constant ($m_i$)</td>
<td>10.1</td>
</tr>
</tbody>
</table>
4.2.6 MK Modelling Parameters

The numerical modelling software, failure mode, and user experience will indicate what failure criterion should be used. The Generalized Hoek-Brown uses the compressive strength and the material constants, \( m_b, s, \) and \( a \). The material constants are determined through an indirect method using the Geological Strength Index (GSI) and blast disturbance (D) (Hoek, Carranza-Torres, & Corkum, 2002). Applying the typical GSI descriptions to kimberlite is difficult as discontinuities are inconsistent. An estimated GSI can be applied based on field observation of the behaviour of the rockmass or through the RMR-GSI Correlation (Hoek, Kaiser, & Bawden, 1995). The blast disturbance factor was chosen for poor-quality underground blasting as kimberlite fractures easily, and damage is expected. Table 4.2 presents the parameters used as continuum modelling input parameters for MK.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength</td>
<td>62 MPa</td>
</tr>
<tr>
<td>Geological Strength Index</td>
<td>60</td>
</tr>
<tr>
<td>Disturbance Factor</td>
<td>0.8</td>
</tr>
<tr>
<td>Broken Material Constant ( (m_b) )</td>
<td>0.93</td>
</tr>
<tr>
<td>( s )</td>
<td>0.002</td>
</tr>
<tr>
<td>( a )</td>
<td>0.5</td>
</tr>
</tbody>
</table>

4.3 Geotechnical Parameters – 8% Cemented Rockfill

The design of the cemented rockfill (CRF) was completed by Golder Associates (2009, 2011) using the finite difference code FLAC®. Undercutting spans were modelled to determine the required backfill strength, resulting in a strength of approximately 4 MPa, with no safety factor. A factor of safety of 2.5 was recommended for CRF strength to allow for variability resulting from the batching process and backfill placement. This initial design recommended a cemented
rockfill with a strength of 10 MPa for a 7-meter exposure width and an estimated 8% (by dry weight) cement content of the cemented rockfill.

Daily samples are taken for uniaxial compressive strength (UCS) testing to maintain quality control of the CRF. Cylindrical samples are tested after 3, 7, 28, and 56 days of curing time. The 3- and 7-day samples ensure early strengths are being reached and allow for changes in the CRF recipe due to changes in the backfill material, such as temperature and moisture content. The 28 and 56-day UCS tests ensure the cemented rockfill meets the design specifications and will help identify potential risks for future stoping.

The 28 and 56-day tests for 8% CRF are summarized in Figure 4.6 and Figure 4.7 below, showing the average strength with a one-standard-deviation range. The 28 and 56-day cylinder UCS testing shows an average value of 16 MPa and 17 MPa, respectively. These results are above the design specifications of 10 MPa, but some samples failed to meet the requirement and would be identified to determine future risks.
The numerical modelling of cemented rockfill uses a strength of 8.0 MPa, which implies a factor of safety of 2.0 compared to design requirements. Backfill is used to fill voids created by mining, and it is assumed that the material in a stope binds together: GSI value of 100 with a disturbance factor of 1.0. Therefore, the material constants $s$ and $a$ would be each equal to 1.0. Table 4.3 presents the parameters used as the continuum modelling input for 8% CRF:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength</td>
<td>8 MPa</td>
</tr>
<tr>
<td>Geological Strength Index</td>
<td>100</td>
</tr>
<tr>
<td>Disturbance Factor</td>
<td>1.0</td>
</tr>
<tr>
<td>Broken Material Constant ($m_b$)</td>
<td>6.8</td>
</tr>
<tr>
<td>$s$</td>
<td>1.0</td>
</tr>
<tr>
<td>$a$</td>
<td>1.0</td>
</tr>
</tbody>
</table>

4.4 Numerical Modelling Code

A continuum model was selected because it allows the model to represent the deformation in kimberlite and allows the identified failure mechanisms to develop. A continuum model requires that the rockmass act as a continuous, homogenous, and isotropic material. The representative elementary volume (REV) is a method to determine the length scale in which a rockmass can be treated as homogeneous. It is calculated by dividing the representative dimension of a structure
by the mean spacing of the joint sets (Sari, 2021). The REV value was calculated as 4-7 for production mining and is on the lower end of the range recommended for continuum modelling. A low REV value indicates that a discontinuum model should be investigated to see if the representation of individual block movement alters the comparison of the numerical models. A 2D model was chosen as it requires less user experience and time to set up and run multiple iterations. The modelling cycle must fit into the decision-making cycle to influence operational decision-making. Many geotechnical problems can be assumed to be a plain strain (2D assumption) without a significant loss of accuracy if one dimension is considerably more significant than the other two, and strains along the out-of-plane direction can be assumed to be zero (Eberhardt, 2020a). Due to the geometry of the sill pillar, a 2D model would be appropriate for numerical modelling.

RS2 is a 2D finite element program for soil and rock applications and can be used for a wide range of engineering projects, including excavation design (Rocscience, 2022). RS^2 was chosen as it is readily available and does not require significant experience. In addition, a 2D model with RS^2 would be easy to set up, process, and analyze within the constraints of an operational setting.

### 4.5 Boundary Conditions

External boundaries are the extent of the finite element mesh, and it is recommended that the external boundaries be greater than five times the size of the excavation (Australian Centre for Geomechanics, 2021). The ensures that the boundary conditions do not impact the results near the excavation. Figure 4.8 presents the contours of the major principal stress out to the boundary of the numerical model. The external boundary does not impact the major principal stress contours as they are uniform at the external boundary.
Each boundary is given a displacement restraint, which allows for free movement, movement to be limited in one direction, or for the node to be pinned, not allowing movement (Rocscience, 2022). The surface of the model is allowed free movement, with no restraint. The left and right boundaries of the model allow for movement in the $Y$ direction only, restrained $X$. The bottom boundary allows for movement in the $X$ direction, restrained $Y$.

### 4.6 Field Stress Conditions

Mining the A154N kimberlite pipe will cause a redistribution of stresses around the pipe and through the sill pillars that will remain until the end of the mine life. The starting *in-situ* stresses can be determined through far-field measurements or by using a regional stress model. In 2015, the Geomechanics Research Centre (GRC) of Mirarco completed an *in-situ* stress determination. The stress states were determined by the overcore strain-relief technique employing the United States Bureau of Mines Borehole Deformation Gauge (Mirarco, 2015). Table 4.4 summarizes the results from the *in-situ* stress determination.
Mirarco completed a set of measurements at the A9080HD in the granite host rock between the two main underground mining areas. The results from the A9080HD were determined to be the best location, as it is the farthest from production mining and, thus, the farthest from potential stress changes. The stress state at the A9080HD also aligns well with the regional structural context of the Lac de Gras dike swarm (Mirarco, 2015). The results were converted into an orthogonal system aligned with the mine grid, outlined in Table 4.5.

Table 4.4 Summary of in-situ stress states

<table>
<thead>
<tr>
<th>Location</th>
<th>Principal Stress</th>
<th>Magnitude</th>
<th>Orientation</th>
</tr>
</thead>
<tbody>
<tr>
<td>A8995</td>
<td>Maximum/Major, $\sigma_1$</td>
<td>23.0</td>
<td>358/42</td>
</tr>
<tr>
<td></td>
<td>Intermediate, $\sigma_2$</td>
<td>19.5</td>
<td>268/01</td>
</tr>
<tr>
<td></td>
<td>Minimum/Minor, $\sigma_3$</td>
<td>11.5</td>
<td>177/48</td>
</tr>
<tr>
<td>A9080 HD</td>
<td>Maximum/Major, $\sigma_1$</td>
<td>11.6</td>
<td>219/09</td>
</tr>
<tr>
<td></td>
<td>Intermediate, $\sigma_2$</td>
<td>10.7</td>
<td>001/79</td>
</tr>
<tr>
<td></td>
<td>Minimum/Minor, $\sigma_3$</td>
<td>5.2</td>
<td>128/07</td>
</tr>
<tr>
<td>D8925 DWG</td>
<td>Maximum Horizontal, $\sigma_H$</td>
<td>41.7</td>
<td>215/00</td>
</tr>
<tr>
<td></td>
<td>Minimum Horizontal, $\sigma_h$</td>
<td>17.3</td>
<td>125/00</td>
</tr>
</tbody>
</table>

*plunge is positive down from horizontal

Table 4.5 Cartesian components of the tensor for A9080 HD

<table>
<thead>
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<tr>
<td>Normal (Vertical)</td>
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<tr>
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<tr>
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</tr>
<tr>
<td>Shear (Vert / N-S)</td>
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</tr>
</tbody>
</table>

4.7 Model Mesh

A numerical model is broken into elements, with nodes at the corners and the midpoints. A more significant number of nodes increases the complexity of the model, allowing for an increase in the detail the model provides, but it will also increase the calculation time. In an operational setting, the calculation time needs to be balanced with the required level of detail, which is
determined by the purpose of the numerical model. The performance of stope walls will require less detail than assessing the performance of support elements.

Three types of mesh can be used to create the elements: uniform, graded, and specific. A uniform mesh remains the same size over the entire model. A graded mesh will have a higher concentration of nodes near excavation surfaces, increasing the level of detail in proximity to these surfaces. A specific mesh can be placed in areas of more significant concern, increasing the node density and the level of detail, but it will increase the length of the calculation time. A graded mesh was chosen for modelling the sill pillar recovery as it decreases the node density around the boundaries and increases the node density around the excavations. A specific mesh was created around the overhangs as it was identified as a potential failure mechanism during the risk assessment. The specific mesh was constrained to a one-meter element spacing, which will provide details about the deformation of the walls. Still, the mesh size will not give enough detail to analyze the performance of the support elements. Figure 4.9 presents the model mesh around the sill pillar.

*Figure 4.9 Cross-section showing the model mesh around the sill pillar (option 3)*
4.8 Groundwater and Dynamic Effects

Active dewatering of the kimberlite pipes reduces the ingress of ground water, and the mine site is not located in an active seismic zone. This analysis did not apply the exposure of the rockmass to groundwater or dynamic effects (earthquake and blasting vibrations). This assumption was seen as reasonable as the rockmass has been dewatered.

4.9 Model Sequencing

Mining of the sill pillars will occur near the end of the mining sequence after mining-induced stress changes have occurred, and the model should provide a representation of these stress changes. The mining-induced stress changes were introduced by simplifying the mining sequence, achieved by mining and backfilling each level in the model. It is acknowledged that this is a simplified representation, but it should show an approximation that will be acceptable for a comparison between the three different models. Figure 4.10 presents the geometry of four stages of the mining sequence leading to the sill pillar production. The colour contours in the figures represent the major principal stress. Step 2 presents the start of mining in the block, with Step 6 presenting the changes halfway through the mining of the block. Step 10 presents the last level being mined in the mining block, and Step 12 is the last stage with changes to mining in the pipe. Step 12 is the final stage between the three different models, with the mining of the sill pillar occurring after this stage.
The mining sequence progresses one stopeline at a time from the center of the pipe to the pipe extent, alternating sides. The production sequence was broken down into three numerical modelling steps: development, production stoping, and backfill, as presented in Figure 4.11. The overcut drift development occurred in the same step as the backfilling of the previous stope.
4.10 Numerical Modelling Results

This section is broken into two parts; the first discusses a single-stope extraction, and the second discusses changes after several stopelines have been extracted. Each part will outline the conclusions from the numerical modelling and the implications for future operational activities.

4.10.1 One-Stope Extraction

The extraction of the first stope does not change the stress regime in the sill pillar as the sill pillar is already in a low-stress state. Low stress in the sill pillar allows the relaxation of the stope walls and causes the walls of the production stope to fail. Stope wall failure would impact the stope shoulders and access to the overcut drift. Figure 4.12 presents a cross-section of the sill pillar with the $\sigma_1$ stress contours after extracting the first stope. The contours follow the stress arching seen in the model sequencing step.
Figure 4.13 presents the extraction of the first stope in the sill pillar, showing the maximum plastic shear strain that starts at the shoulder of the stope and arches downward towards the undercut level. Shear strain is the ratio of the change in deformation to its original length perpendicular to the axes of the member due to shear stress. The maximum shear plastic strain follows the same arching as the maximum principal stress.
The concentration of the plastic shear strain in the backs of the undercut drifts would impact the primary support tendons due to shear. Shearing of the primary support tendons would cause failure in the back of the drift and was identified during the risk assessment. Secondary support would be installed in these headings, preventing back failure caused by the shearing of the primary support tendons.

Figure 4.14 presents the maximum shear plastic strain with the yielding elements. The yielding elements indicate failure at the node in the model mesh but does not indicate the magnitude of failure. All three options show a significant amount of shear and tension-yielding elements in the walls and shoulders of the open stope. These yielded elements indicate that failure in these areas will likely occur, impacting production mining and access to the overcut drift. The yielded elements are more pronounced with the higher exposed kimberlite shoulders in Option 2 and Option 3 and do not continue into the CRF pillar in any of the options.

The strength factor measures the rock strength over the induced stress and would indicate a rockmass failure for values less than one. Figure 4.15 presents the contours for the strength factor, which shows no significant areas of potential failure due to the induced stresses. The
major principal stresses are relatively low compared to the rockmass strength, indicating that this
is not likely a failure mechanism that will occur during the mining of the sill pillar.

![Figure 4.15 Strength factor (option 2)](image)

4.10.2 Full Extraction

Subsequent production stoping does not change the stress profile of the sill pillar, presented in
Figure 4.16 and Figure 4.17.

![Figure 4.16 Sigma1 - step 18 (option 1)](image)
The figures show the major principal stress profile between production stoping steps 18 and 22. There is a negligible change in any concentration of the major principal stress between the two production steps, with low stresses prevailing throughout the sill pillar. Relaxation and unravelling of the rockmass remains the principal failure mechanism during production stoping of the sill pillar.

Figure 4.17 Sigma1 - step 22 (option 1)

Figure 4.18 and Figure 4.19 presents the maximum shear plastic strain with yielded elements of the production stoping steps 18 and 22. The maximum shear plastic strain does not increase between the two steps, indicating that shearing of the support tendons in the back of the undercut drift may occur before production stoping starts. Additional damage is done to the rockmass with each production stope, shown through the increasing amount of shear and tension-yielding elements. The yielding elements indicate that the shoulders and walls of the production stope are damaged, which could lead to a failure during production stoping. A stope shoulder failure would impact the overcut drift during backfilling and could expand, causing a more significant regional-scale failure that may affect the CRF above the sill pillar.
4.11 Chapter Summary

This chapter outlined the numerical modelling set-up and input parameters of a sill pillar to aid in determining the optimal placement of an overcut drift. A finite element model was chosen to replicate the production stoping with the Generalized Hoek-Brown used as the failure criterion. The input parameters used for the model set-up were presented with how these parameters affect
the modelling process, which needs to fit the decision-making cycle for the numerical modelling to be used effectively. Finally, the outcome of the numerical modelling was discussed, presenting a low-stress environment during the production stoping of the sill pillar. The low stress and damage caused by production stoping affects the stope walls and stope shoulders in the kimberlite, leading to wall failure. A failure would increase the difficulty during backfilling, affecting adjacent production stoping and regional stability.
Chapter 5: Operational Considerations for the Mining Sequence

5.1 Introduction

Three scenarios for the overcut development of sill pillar recovery have been assessed:

- Development through the cemented rockfill (CRF) above the kimberlite,
- Development through the kimberlite under the CRF,
- Development partially through each material.

This chapter will outline the site-specific conditions of each option, how numerical modelling of the scenarios identifies the geotechnical risks, and the operational considerations for each scenario. These factors impact the decision-making process to determine the preferred location for the overcut development.

5.2 Pre-Development Conditions

The numerical model must be easily altered to reflect different options to use the modelling results in operational decision-making effectively. The time to set up and change the model, process the model, evaluate the options, and discuss the results must fit within the on-site planning and decision-making cycle. A simplified model of the mining sequence was used to provide the same starting conditions for each option, as the models are used to compare stress changes due to the development and production activities of the sill pillar.

The simplified model will provide an approximation that can be used to compare the three development options. Still, it should be noted that it will not give the actual in-situ stress conditions of the sill pillar as the model as not been calibrated. Following the mine plan, the simplified mining sequence removes and replaces each mining level with cemented rockfill. As
this sequence progresses up each mining block towards the sill pillar, the model indicates stress arching through the sill pillar, as seen in Figure 5.1.

![Figure 5.1 Sigma1 - step 12 (option 1)](image)

5.3 Development & Primary Support

Development occurs near the top of the sill pillar underneath previously blasted and backfilled stopes. The numerical modelling of the development did not show any differences between the three assessed options. Therefore, this section describes the potential operational challenges and the potential risk of the failure of the back during development, which was identified as a major failure mechanism during the risk assessment.

5.3.1 Option 1 – Development in CRF

Development in CRF would align the sill of the new development drift with the bottom of the backfilled stopes. CRF forms the back and walls, and the sill is kimberlite. There are two main challenges with development occurring entirely in CRF:

- Increased difficulty in developing,
• Reduced CRF pillar thickness.

Cemented rockfill comprises granitic rock encompassed in a cementitious binder, making it more difficult than mining in kimberlite. The operational difficulty begins with drilling a development round, where the drill holes can be prone to collapse. Breakage of the cement bonds can cause loose stones to fall from the perimeter of the hole, which will plug the hole when the drill steel is removed. These holes would need to be cleaned before blasting, mechanically with the drill or manually by the blasting crew. The holes drilled for ground support also require cleaning before installing the support tendons; the bolter completes this mechanically. The CRF is prone to unravelling after blasting, so shotcrete is applied to prevent unravelling during the installation of the support tendons. The standard support for CRF is shown in Figure 5.2. This support regime will be used for any development under or in CRF and, therefore, for all three options.

![Figure 5.2 CRF development support regime (Diavik Technical, 2023)](image)

The second challenge with the development through CRF is the reduction in the thickness of the 8% CRF above the kimberlite sill pillar. A minimum of a ten-meter vertically thick 8% CRF pillar has been placed above the kimberlite sill pillar, with the remainder of the stope being filled with a lower-strength material. Figure 5.3 presents a typical long section of a backfilled stope.
above the sill pillar, showing the location of the reduction in the CRF pillar thickness. The darker
grey is 8% CRF, with the lighter grey being a lower-strength material.

![Diagram showing reduction in pillar size](image)

*Figure 5.3 Long-section of backfilled stope above the sill pillar (option 1)*

By developing along the bottom of the 8% CRF stope, the distance between the top of the drift
and the transition of binder contents will be reduced. The numerical modelling completed by
Golder Associates (2009, 2011) did not detail a minimum height of high-strength CRF above
development drifts and only outlined the parameters used in the numerical modelling. The
recommendation was to use a CRF with a strength of 10 MPa, which implies a factor of safety of
2.5. Even with generally higher strengths than the recommended 10 MPa, the reduction in the
thickness of the high-strength CRF pillar may impact production mining. This scenario was not
modelled in this thesis.

### 5.3.2 Option 2 – Development in Kimberlite

Development in kimberlite would align the back of the new development drift at the bottom of
the backfilled stopes with the back of the drift in CRF and kimberlite forming the walls and sill.
This option reduces the impact of drilling and installing support in CRF but increases the
potential for failure of the back during development. Failure of the back during development would occur when loose or unconsolidated material encountered at the bottom of the stopes falls out after blasting a development round. The risk assessment identified three scenarios that would lead to the failure of the back during development:

- Loose or unconsolidated material,
- Widely segregated CRF,
- Poor quality CRF.

Loose or unconsolidated material is left on the sill before backfilling because mucking a stope is an imprecise task, and there are areas near the sides and back of the stope that are difficult to recover. These problematic areas can be created due to the geometry of the stope or poor blasting results, leaving ridges in the floor of the stope. The stope walls may also unravel after backfilling has begun and may not be identified until these areas are encountered during development. Scans of empty stopes are completed to track their location, blasting performance and to determine the amount of backfill to fill the void. After backfilling has started, there is no reason to complete another scan, leading to the potential for unconsolidated material left by an unknown wall failure. This material would be covered by the backfill and be encountered in the back during development through the sill pillar. Figure 5.4 presents the development sequence for Option 2 as it meets unconsolidated material at the bottom of a backfilled stope. As development progresses, the unconsolidated material falls out of the back during the blasting of the development round. The higher back would then be supported, which, depending on the height of the failure, would increase the difficulty of supporting the development round.
Unconsolidated material at the bottom of the backfilled stope.

Material falls out during the blasting of the development round.

Supported high back.

*Figure 5.4 Development sequence at the bottom of a backfilled stope (option 2)*

The placement of CRF into a stope may create bands of loose or unconsolidated material by the segregation of larger material and reduced binder content. As CRF is dumped into a stope, it will create a slope where larger material will preferentially collect at the bottom. A lack of binder to encase this larger material creates weak CRF layers. Figure 5.5 presents a picture of CRF in a secondary heading showing the layering of segregated and poor-quality CRF.

*Figure 5.5 Segregated CRF in a secondary heading – partially covered by shotcrete*
This layering has been encountered in the walls of secondary headings with lower strength CRF, designed only for side wall exposure. This scenario is seen as unlikely to occur as this type of layering has not been encountered in any development through higher-strength CRF. Poor quality CRF would be encountered when CRF did not meet design specifications. Quality control is completed at the backfill plant to check for strength and after production. An additional sample is completed at the underground CRF dump, which checks the CRF strength before deposition into a stope. The underground samples are not taken daily and are used to correlate the infield strength to the backfill plant production strengths. Identification of poor quality CRF can be made before development begins by checking the CRF database. Recovery of the development drive after a back failure will depend on the height of the failure. Underground equipment would need to reach the top of failure to secure the ground with shotcrete and support tendons. If the top of the failure were higher than the equipment can reach, then other recovery methods for the drift would need to be evaluated.

5.3.3 Option 3 – Development in CRF & Kimberlite

The likelihood of a failure of the back during development is reduced in Option 3 by mining the lowest two meters of the backfilled stope where there is a higher likelihood of poor or unconsolidated CRF. Kimberlite forms the lower three meters of the drift with the transition zone from kimberlite to CRF in the upper face and walls. The drill and blast process will have increased operational difficulty as it must account for two varying rock types. The blast performance will be monitored, and adjustments will be made to prevent damage to the back of the development drift. The ground support regime, shown previously in Figure 5.2, will control the transition zone in the upper walls and is a lower hazard than a transition zone in the back.
Each development round will have kimberlite and CRF, increasing the dilution of recovered ore. The mine plan must account for the additional dilution introduced through the development of ore and waste.

5.4 Production & Secondary Support

The production stopes are designed to be 10 meters wide with an overhang of 2.5 meters in both walls. The numerical modelling of the production stoping did not show any differences between the three assessed options. Therefore, this section describes the potential operational challenges of the three options in the following zones:

- Failure of support due to a stress change,
- Stope wall stability,
- Stope shoulder stability,
- CRF pillar stability.

5.4.1 Failure of Support due to Stress Change

The pre-development steps of the numerical modelling indicate a change in the stress direction in the backs of the undercut drifts. Figure 5.6 presents the contours of the maximum shear plastic strain showing concentration in the backs of the undercut drifts. The primary support tendons may shear in the potential failure zones, causing back failure. This failure mechanism would occur in all three options.
A secondary support regime of solid bar support tendons, such as rebar or cablebolts, has been implemented as part of the support requirements to prevent shearing, presented in Figure 5.7. Secondary crown support was implemented due to observed bulking of the back and failure of the primary support tendons caused by a change in the stress profile. Since the secondary crown support was implemented, no additional bulking in the back or failures have occurred.

Figure 5.6 Contours of maximum shear plastic strain showing concentrations in the back of adjacent undercut drifts

Figure 5.7 Secondary crown support regime (rebar) (Diavik Technical, 2023)
5.4.2 Stope Wall Stability

Overbreak in a production stope is when the stope walls break larger than the drill and blast design. It is difficult to determine the cause of overbreak as the two primary causes lead to the same outcome and have similar failure mechanics. The two primary causes of overbreak are:

- Ground conditions,
- Blast design.

The sill pillar rockmass is competent compared with the other lithological units in the pipe, but its stability is impacted by the changes induced through mining activities in the area. The numerical modelling results for production stoping are outlined in Section 4.10, with Figure 4.18 and Figure 4.19 presenting the changes to the yielded elements during production stoping of the sill pillar. Production stoping increases the number of yielded elements in the numerical model, which indicates increased damage to the walls of the production stope. During mining, the walls of the production stope may fail due to the damaged ground conditions, leading to the undercutting of the stope shoulders.

Overbreak is also caused by the blast design, as a poor blast design will cause excessive damage to the rockmass. Excessive blast damage is caused when more energy is imparted into the rockmass than is required to break the rock, causing damage outside of the drill and blast design. Figure 5.8 presents a cross-section of the sill pillar showing a production stope with an overbreak highlighted in the left wall. The overbreak undercuts the stope shoulder, reducing the effectiveness of the installed secondary support and allowing the rockmass to unravel around the ends of the installed support.
Reducing rockmass damage caused by production stoping and blasting is a priority to ensuring a stable stope. Optimization of the drill and blast designs can reduce the amount of rockmass damage and is an iterative process. The performance of a production stope is evaluated against the design, and then the design inputs are adjusted to optimize the powder factor for the rockmass conditions. When wall failures still occur due to poor ground conditions, the stope design must be changed to create a more stable stope geometry. The geometry of the stope can be altered in two ways that will affect the stability of the production stope:

- Stope length,
- Production ring profile.

The exposure of the damaged rockmass is increased with longer stope lengths, which increases the potential of a wall failure. The likelihood of a wall failure must be balanced against the increased productivity of the longer stope, which will adjusted based on stope performance. Similar ground conditions have allowed for production stopes of 15 to 18 meters in length without significant failures and will be the initial length for the sill pillar production stopes.
The production ring profile is the two-dimensional cross-section of a stope and specifies the angle of the stope shoulders. Shallower stope shoulders increase the likelihood of failure in poor ground conditions and increases the potential for blast damage by having the production blast holes closer to the installed secondary support. Increasing the steepness of the stope shoulders reduces recovery and leaves a larger intact pillar, increasing stope stability in poorer ground conditions. Figure 5.9 presents production ring profiles of C & D Blocks with different stope shoulder angles, 55° vs 63° respectively. The different ground conditions in the two mining blocks have led to different shoulder angles, which were determined by evaluating the stope performance. The steeper angles in D-Block improves stope stability but reduces ore recovery from the stope.

![C-Block – 55 degrees](image1)
![D-Block – 63 degrees](image2)

*Figure 5.9 C & D blocks production ring cross-sections*

Overbreak in a production stope is typically caused by poor ground conditions or a poor blast design that damages the rockmass. The three development options will use the same stope geometry and drill and blast design, causing the same stope stability issues. The drill and blast designs must be optimized after production stoping begins.
5.4.3 **Stope Shoulder Stability**

The sill pillar production stopes are designed to be 10 meters wide, creating an overhang, or stope shoulder, of 2.5 meters in both walls. Two failure modes were identified that would cause a stope shoulder failure during production mining:

- Failure of the kimberlite/CRF rockmass,
- Failure along the horizontal kimberlite/CRF interface.

The rockmass of the sill pillar will be damaged by production mining in nearby mining blocks, indicated in the numerical modelling through the yielded elements in the sill pillar. A yielded element does not indicate the magnitude of the failure, so it can only be assumed that the rockmass is damaged. The yielded elements in the numerical model are isolated to the kimberlite, but the development and production blasting will damage the CRF, leading to unravelling and failure. Figure 5.10 outlines the steps of a stope shoulder failure presenting the unravelling of the kimberlite rockmass.

![Figure 5.10 Kimberlite unravelling stope shoulder failure (option 3)](image-url)

*Figure 5.10 Kimberlite unravelling stope shoulder failure (option 3)*
The damaged kimberlite rockmass is undercut by the production stoping, reducing confinement, and will unravel around the primary support tendons. The installation of secondary support tendons, offset from the primary tendons, will increase tendon density, reducing the likelihood of unravelling between the installed tendons. The secondary support tendons will also be longer than the primary tendons to be embedded outside the extent of the production stope. The longer support will reduce the potential for unravelling around the ends of the primary support tendons when the production stope overbreaks. Figure 5.11 presents the secondary overcut support to be used in the right wall of the overcut development. The same support standard will be installed in the left wall above the stope shoulder.

Increasing confinement by installing secondary support has been effective when installed in kimberlite, but it has not been effective in CRF. The additional drilling damages the rockmass, breaking the cement bonds that provide the CRF strength. Secondary support tendons will not be used in Option 1, where CRF forms the walls.

Figure 5.11 Secondary overcut support of kimberlite overhangs (up to 2.5 meters)
Production stoping damages the primary surface support, causing unravelling in the overcut drift. Increasing the strength of the surface support by adding shotcrete-embedded strap arches has effectively prevented damage caused by production mining. Figure 5.12 presents the secondary surface support for strap arches with the primary support tendons.

![Secondary surface support (strap arches) (Diavik Technical, 2023)](image)

To prevent a stope shoulder failure due to unravelling, the secondary support design consists of 0-gauge welded wire mesh straps installed over the initial 50 mm layer of shotcrete. Next, a second 50 mm layer of shotcrete embeds the mesh straps, creating a ridged shell. Finally, the longer secondary support tendons are installed through the 100 mm ridged shell in Option 2 and Option 3. The longer secondary support tendons are not installed in Option 1 due to the damage that drilling causes to the CRF rockmass.

Loose or unconsolidated material may be encountered at the bottom of the stopes, affecting production stoping by creating a weak horizontal interface between the CRF and kimberlite. This interface allows for the kimberlite stope shoulder to detach as a block. Figure 5.13 presents the failure of a kimberlite stope shoulder caused by the kimberlite/CRF interface.
Production stope is blasted. Stope shoulder separates from kimberlite/CRF interface. Stope shoulder fails as large blocks.

Figure 5.13 Kimberlite/CRF interface stope shoulder failure (option 3)

Production stoping undercuts the stope shoulder, allowing the kimberlite to separate from the CRF and fall into the stope. The primary support tendons will fail with the block as the length of the tendon is inadequate to support the block. The long secondary support tendons installed in Option 2 and Option 3 will support the kimberlite shoulder to prevent a block failure by creating a beam from the rigid surface support to outside the extent of the production stope. The secondary tendons will act in shear and require adequate capacity to support the kimberlite block, which will differ for each option.

5.4.4 CRF Pillar Stability

The CRF pillar is a series of backfilled stopes filled with high strength, 8% CRF, to act as a future pillar and back during the recovery of the kimberlite sill pillar. The strength of the CRF was modelled using the finite difference software FLAC® for various undercutting spans, as outlined in Section 4.3. The CRF pillar cannot be changed, and operational decisions may impact the integrity of the pillar, causing a failure. There are two significant concerns caused by the placement of the overcut drift in proximity to the CRF pillar:
• Notching of the CRF pillar,
• Local and regional stability.

Notching is the term used to describe the amount that development impacts the CRF pillar by reducing its thickness and creating the potential for additional breakage. A large notch significantly affects the CRF pillar by decreasing the thickness and having a larger area that may fail when undercut during production stoping. Figure 5.14 presents two development options showing the impact on the thickness of the CRF pillar and the potential additional breakage.

![Diagram showing notching in CRF pillar due to location of development](image)

*Figure 5.14 Notching in CRF pillar due to location of development*

A minimum CRF pillar thickness of 10 meters was sufficient when backfilling the production stopes. Therefore, the upper portion of the stope was backfilled with a lower-strength CRF. Development through the CRF will reduce the thickness of the CRF pillar, increasing the risk of a pillar failure. The reduction in the pillar thickness will only be in a couple of locations, and these areas may be reinforced with more significant surface support. Option 1 reduces the thickness of the CRF pillar to five meters, and Option 3 reduces the pillar thickness to eight meters. Option 2 does not reduce the pillar thickness as it is entirely developed in kimberlite.
Notching also allows for the potential of additional unravelling in the CRF pillar. A more prominent notch will allow a larger area to be impacted and undercut during production stoping. If the kimberlite detaches from the CRF pillar, the CRF may unravel, causing a local failure. A local failure will be controlled by backfilling the stope and tightfilling the failure area. Tightfilling provides confinement to the area to prevent additional unravelling but does not provide support. A more significant regional failure could occur if one local failure connects with a previously tightfilled failure. Figure 5.15 presents a scenario where a failure in the left wall of a production stope unravels and causes the backfill and tightfill of an adjacent stope to fail. The larger failure creates a wide span in the production stope, and the 8% CRF pillar fails. A larger regional failure would impact the mining sequence of the sill pillar and any remnant mining in the block above the sill pillar.

*Figure 5.15 Large notching and failure of production stope causing a regional failing of the sill pillar*
5.5 Backfilling and Recovery of a Failure

The sill pillar production stopes will be backfilled with a combination of CRF and rockfill. CRF will be used to create an endwall, with the remainder of the stope being filled with uncemented rockfill. The sill pillar stopelines are single pass, and the overcut development will not be used again; therefore, the overcut development will be tightfilled. Tightfilling of the overcut development limits voids and provides confinement to the back and walls. The location of each of the three development options will allow for backfilling and tightfilling of the overcut drift to occur without any issues.

A stope shoulder or CRF pillar failure may create an unstable span and prevent personnel access to backfill the production stope. Rehabilitation is an option to allow personnel entry into the overcut drift to complete backfilling. Backfilling on teleremote is an additional option that is more time-efficient and would be preferable since the overcut development is only used as an overcut.

The most likely failure is that the kimberlite fails up to the kimberlite/CRF interface which is in the sill of Option 1. Figure 5.16 presents a stope shoulder failure in Option 1 and shows that the overcut development would be unaffected by this failure. Tightfilling the stope shoulders would be difficult and ineffective in providing confinement to the CRF. A failure in an adjacent stope may create the geometry for a future regional-scale failure.

Failure to the kimberlite/CRF interface in Option 2 would create a wide span in the overcut drift. An unravelling failure would leave the shotcrete arches in place, unravelling around the installed tendon support. The arches would need to be removed to allow for tightfilling of the overcut drift. A block failure would remove the shotcrete arches with the failure of the kimberlite. Figure 5.17 presents a stope shoulder failure in Option 2, with the dashed line representing the location
of the walls of the overcut development. Teleremote tightfilling of a five-meter drift is difficult due to equipment limitations, and a 0.5-meter gap would be expected.

CRF forms the upper walls and back of Option 3, and a kimberlite stope shoulder failure would create a similar span to Option 2. The shotcrete arches may or may not remain, depending on the type of failure. Figure 5.18 presents a stope shoulder failure in Option 3, showing the potential...
failure profile. Teleremote tightfilling of Option 3 is the easiest at a three-meter-high failure and allows for the placement and ramming of the tightfill material.

![Diagram of 3-meter-high failure is the easiest to tightfill.]

*Figure 5.18 Recovery of a kimberlite stope shoulder failure (option 3)*

### 5.6 Ore Recovery

A financial discussion on the value of recovered ore between the three sill pillar options is outside the scope of this thesis. This section compares the geometrical impacts on ore recovery and the overall extraction ratio for the sill pillar between the three development options. Figure 5.19 compares the recovered ore volume in each option using Option 2 – Development in Kimberlite as the base case. The designs of Option 1 – Development in CRF, and Option 3 – Development in CRF & Kimberlite recovered 12.5% and 7.5% more ore, respectively. The additional recovery of kimberlite from the stope shoulders accounts for the difference between the three options. These values represent a stable production stope and do not account for overbreak or stope shoulder failures.
Table 5.1 summarizes the extraction ratio for the three sill pillar development options. The extraction ratio is the ratio of mined ore volumes to total ore volumes from the design shapes. These values represent a stable production stope and do not account for overbreak or stope shoulder failures.

<table>
<thead>
<tr>
<th>Sill Pillar Options</th>
<th>Extraction Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option 1 – Development in CRF</td>
<td>52%</td>
</tr>
<tr>
<td>Option 2 – Development in Kimberlite</td>
<td>47%</td>
</tr>
<tr>
<td>Option 3 – Development in CRF &amp; Kimberlite</td>
<td>49%</td>
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</table>

### 5.7 Comparison of Operational Considerations

This chapter breaks down the production cycle into its major constituent steps to determine how each production cycle step is affected by the three development options. Each development has strengths and weaknesses when compared against the other options. Table 5.2 presents a comparison between the three development options, showing the comparative ranking. An arrow pointing up indicates that the option is preferred comparatively to the other options, while a
downward-facing arrow suggests the opposite. No ranking was given if there was no perceived
difference between the options.

Table 5.2 Comparison of operational considerations

<table>
<thead>
<tr>
<th></th>
<th>Option 1</th>
<th>Option 2</th>
<th>Option 3</th>
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<td>Numerical Modelling</td>
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<td>Development</td>
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<td>Secondary Support</td>
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<td>Backfilling</td>
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<td>Recovery</td>
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Option 2 is the least preferred because of the risk during development caused by unconsolidated
material leading to a back failure. Development in Option 1 is perceived as the safest option as
the back and walls are entirely in CRF, and the unconsolidated material is fully removed. The
development in Option 1 reduces the thickness of the CRF pillar, which will impact production
stoping by weakening the pillar above the production stope. The installation of secondary
support above the stope shoulders is also not recommended in Option 1 due to the additional
damage caused by the drilling.

Option 3 is a merger of the first two options with the development of removing the
unconsolidated material while leaving a larger pillar above the drift, and secondary support may
be installed to prevent failure of the kimberlite stope shoulders. Each of these are elements of the
mining sequence that have already been successfully completed and increase the confidence of
successful implementation. Option 3 is the preferred option for the overcut development to
recover the sill pillar.
Chapter 6: Conclusion

6.1 Research Summary

Pillars are designed to maintain the stability between underground mine openings, providing for safe extraction, and can separate adjacent mining areas, allowing for concurrent mining. Pillars that separate adjacent mining areas prevent operational and geotechnical interactions, but the adjacent mining damages the rockmass of the pillar. Sill pillar recovery involves extracting the pillars between these adjacent mining areas, and three development options were identified for the overcut development.

A numerical modelling analysis of the three development options was conducted to ascertain distinctions among them and to inform operational decisions regarding the placement of the overcut drift. Notably, no significant disparities were observed in the major principal stress, maximum shear plastic strain, or strength factor among the three development options. However, the numerical modelling revealed an overall reduction in the major principal stress within the sill pillar, creating a low-stress environment and allowing the rockmass to fall apart.

Furthermore, the numerical modelling highlighted substantial rockmass damage occurring before the initiation of development and stoping activities in the sill pillar. This damage poses a potential risk of failure in the stope shoulders and production stope walls, necessitating careful consideration in the design of production stopes and the implementation of ground support measures.

The three development options were compared through the operational cycle of development, production, and backfilling, allowing for an understanding of the opportunities and risks associated with each step of the cycle. Development partially through cemented rockfill and the kimberlite offered the most balanced compromise solution.
6.2 Main Findings and Conclusions

The main findings are summarized as follows:

- Simplifying the geological setting and mining sequence allows for an easily replicable model that can be used at the operational level, where changes can be easily made and compared against other scenarios.
- The optimal solution to a problem needs to encompass the entire operational cycle. The development through cemented rockfill offers the most strengths during development but causes significant production risks. Development through cemented rockfill and kimberlite is the optimal compromise, addressing both development strengths and production risks.

6.3 Limitations and Assumptions

Numerical modelling is constrained by the information inputted into the model and the user experience in interpreting the results. The absence of calibration warrants caution in drawing absolute conclusions. Additionally, the following limitations were identified:

- The initial mining sequence generated stress arching through the sill pillar and reduced the major principal stress that persisted throughout the rest of the modelling sequence. Ground support regimes were suggested based on the low stress environment of the numerical model, but operational experience from development and production stoping indicates that a low-stress environment is reasonable.
- The thesis focused on the failure mechanisms and support regimes in the kimberlite. The cemented rockfill was modelled as an isotropic material, which is reasonable if the placement of the cemented rockfill causes no layering issues. Operational experience has
identified that layering is not an issue in the higher strength cemented rockfill. Modelling the cemented rockfill as an anisotropic material would allow for a better understanding of the geotechnical mechanisms involved with the cemented rockfill failure but would need to be completed as a discontinuum model.

### 6.4 Recommendations for Future Work

Numerical modelling informed the decision-making process in comparing the three development options. Subsequent modelling of the chosen option would further aid the decision-making process through:

- A sensitivity analysis of the modelling inputs would provide confidence in the failure modes identified in the numerical model. A sensitivity analysis would reduce the input parameters of the material components to determine if there are significant changes. New mitigation strategies would be introduced for a substantial change, such as an enhanced ground support regime.

- The optimization of the stope design by changing the production ring profile to achieve stable production stope shoulders. The stope shoulders were identified as a major failure mechanism, and stability can be improved by increasing the angle of the stope shoulders. Increasing the stope wall angle reduces the stope recovery but could be offset by increasing the width of the stope, which introduces new challenges that need to be investigated.

- A continuum model was chosen for ease of use and the ability for the rockmass to act as a homogenous, isotropic material. The representative elementary volume (REV) was on the lower end of the recommended range for a continuum model, and a discontinuum model should be investigated. A discontinuum model, such as a discrete element model, would
show an appropriate failure mechanism for the detachment of the kimberlite and CRF from their interface. It would also model individual block movement and might be able to show the damage and failure mechanism of the cemented rockfill. A comparison between a continuum and discontinuum model may lead to different conclusions, especially with a REV number near the lower end of the recommended range.
Bibliography


Hammah, R., & Curran, J. (2009). *It is better to be approximately right than precisely wrong: Why simple models work in mining geomechanics*. Asheville, NC, USA: In Proceedings


Appendix A: Numerical Modelling Sections

Sigma 1 - option 1

Sigma 1 - option 2

Sigma 1 - option 3
Sigma 1 - yielded elements - option 1

Sigma 1 - yielded elements - option 2

Sigma 1 - yielded elements - option 3
Strength factor - option 1

Strength factor - option 2

Strength factor - option 3
Sigma 1 - step 18 - option 1

Sigma 1 - step 18 - option 2

Sigma 1 - step 18 - option 3