ABSTRACT

Difficult ground conditions negatively affect both mine production and the safety of underground workers. Underhand cut-and-fill mining is a potential solution to these issues. Discussions with mine sites revealed the use of sill beams in underhand cut-and-fill mining is not optimized. Optimization in ground support, development of in-situ strength test, and revisions of design standards are desired. Ultimately, the operations require the minimum cemented paste backfill (CPB) strength for a stable span.

Operational concerns were addressed by a multi-prong approach investigating stability of CPB sill beams using observational, experimental and analytical techniques. A case study approach summarizing the design of five mines utilizing underhand cut-and-fill with CPB is presented for different ground conditions. A historical study of span widths and beam strengths for the Stillwater mine is presented.

Laboratory testing determined the stress-strain behaviour of CPB. CPB follows a hyperbolic elastic loading path to peak stress followed by a strain-softening associated with the decay of the cohesion values. Additional testing found that cohesive and tensile strength values were on average 35 and 20 percent respectively of the unconfined compressive strength. This finding impacts sill beam design strengths as previous assumptions were conservative. Test database analyses from three mine sites found that sample size and location preparation has no effect on the strength of the sample. In-situ testing methods common in other industries were not practical with CPB; rather the in-situ strength can be represented by a site specific moisture content index relationship.

Review of current design methodology noted closure stresses were absent from the majority; a method was developed to assess closure for sill beam stability. The potential for critical failures were determined through a Monte Carlo probabilistic model. Methods reducing the risk of failure based on the simulation are investigated. Analysis found ground support does not improve the structural stability of the sill beam. Ground support keeps the beam intact: beam equations govern stability. The stability of sill beam in a seismic environment was analyzed based on the strain-energy density of the beam. The research concludes with a design guideline for CPB sill beams.
PREFACE

The research presented has not been previously submitted for any other degree. The hypothesis was developed by the researcher as a response to questions raised through discussion with mine operators in North America and gaps in knowledge raised from the literature study. The researcher designed the experimental, analytical and numerical studies that form the body of the dissertation. When published material was used to guide the dissertation, to the extent of the researcher’s knowledge, the work was appropriately referenced.

Portions of this thesis include work from papers that are published in conference proceedings. Work presented with respect to Stillwater mine, in parts, was presented in the following paper:


For this paper, the researcher performed the analytical work, discussion on technical aspects of the mine and the writing and submission of the document. The co-author were responsible for detailing the geological setting and mining methods at the mine.

In Chapter 5, the discussions of the Kencana mine and discussions of sill beam closure was part of the following paper:


For the above paper, the researcher’s work included the writing of the document, assisting in the development of the numerical modeling, compiling of the numerical model results, literature review and organizing the submission of the paper for the conference.

The preparation of the cemented paste backfill samples at the University of British Columbia (Chapter 4) was performed by undergraduate students under the direct supervision of the researcher.
Portions of the work presented in Chapter 4 are part of an incomplete Ph.D. thesis performed by Kathryn Dehn (nee Clapp). Work performed with regards to the Windsor Pin method and gathering of data used in the statistical analysis of paste strength at Red Lake were performed by Ms. Dehn.

Bond strength testing results were not performed by the researcher but were made available by the mine site for the purpose of furthering the understanding of ground support within cemented paste backfill. Mine site consulting reports were used as part of the design process of the numerical modeling chapter, work was cited where appropriate.

The research presented was guided by the research supervisor Dr. Pakalnis. This dissertation is a continuation of one aspect of the supervisor’s research field. This dissertation is, in parts, a continuation of work performed by Caceres (2005) in the study of mining under consolidated backfill.

This research was not subject to the University of British Columbia’s ethics approval.
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1 INTRODUCTION

“In many ways, over the last 50 years, Canadian mines have been in the forefront in developing cut-and-fill techniques... This development and innovation is still proceeding to further improve the efficiency and productivity in mining especially as the mines go deeper... Although research and development, including rock mechanics, contributed to these past developments, it was mainly an engineering approach that was used, relying to a large extent on trial and error methods.”

-Singh and Hedley (1981)

“....despite the intensive and increasing use of CPB (cemented paste backfill) in underground mining operations, it remains a relatively new technology. Consequently, many fundamental aspects are still not very well understood.”

-Fall, Belem, Samb, and Benzaazoua (2007)

“Management has said that the (Stillwater) mine would likely have never reopened after 1986 had it not been for the all the benefits of LFUL (Underhand cut and fill with cemented paste backfill) mining.”

-Pakalnis, Caceres, Clapp, Morin, Brady, Williams, Blake and MacLaughlin (2005)

The above quotes were selected to show the advantages gained by mining with backfill, as well as the unknowns of working directly under a consolidated backfill. Difficult ground conditions present significant challenges to underground mining operations. These conditions negatively affect both mine production and the safety of underground workers. Underhand cut and fill mining is a potential solution to the safety and production concerns that result from challenging ground conditions.

Many ore bodies that were unable to be mined with traditional large tonnage methods are economically viable with the underhand cut and fill mining method (UCF). Successful UCF mining is dependent on cemented backfill. The cemented backfill forms the back in UCF mining, replacing weak or highly stressed rock where back stability is a concern. Traditional cemented fill, such as cemented rock fill (CRF) and consolidated hydraulic fill are not suitable in certain conditions conducive to UCF. Traditional fills pose issues with transportation over long distances or require costly planned production delays for the material to develop the design strength. These issues are not present with cemented paste backfill (CPB). CPB underhand cut-
and-fill mining UCF allows for challenging mining conditions to be tackled in a cost-effective, productive and safe manner.

UCF mining method is conducive to high grade, narrow or irregular ore bodies, where local underground stability is of concern (Darling, 2011). The need to mine difficult ore bodies will become increasingly important as conventional deposits are depleted and mining depths increase. This research hopes to provide guidance for mines selecting the UCF CPB mining method in difficult conditions.

1.1 Research Statement

This research grew from discussions with mine operations personnel regarding the stability of CPB sills. Simply put, from an operations perspective, the widest stable span with the cheapest backfill is optimal. Researching published studies, the optimal backfill strength versus span relationship provided no general conclusions. As the focus of this dissertation, a conclusive optimal strength vs. span relationship will be established. In framing the research, the following questions have been identified to be important to this study:

- Can all previous research on hydraulic fill, cemented rock fill and paste backfill be unified into a design guideline?
- How does CPB respond to applied loads?
- What are the post-peak behaviours of backfill and will post-peak behaviour dictate stability?
- Are current constitutive models realistic of actual behaviour of paste under loads?
- What effect does ground support have on structural stability of CPB sills?
- Is the quality control and quality assurance of backfill practices for CPB optimized?
- Is there a low-cost method in determining the in-situ strength of CPB?
- What is the correct methodology for designing a CPB sill?
- What are the commonalities of mines that employ underhand cut-and-fill mining?
- Most importantly, what is the required strength of backfill for an arbitrary span width?

In addressing the issue posed by mine personnel, it is difficult to construct a clear pathway for a complete design. From a ground control perspective, the current state of CPB UCF mining has effectively remained constant since its inception. It will be shown that the research disseminated on underhand cut-and-fill tends to focus on two aspects of the design: the behaviour of CPB and the stability of the sill beam. Little published research exists on a holistic
approach that unifies the stability of sill beams with the behaviour of CPB. This research will investigate both the behaviour of CPB and sill beam stability and unify both these in a comprehensive guideline.

My hypothesis is that a conceptual CPB sill beam design can be built through a rigorous engineering study. By applying fundamental engineering techniques and modern numerical models, a comprehensive design guideline can be developed. Observational, experimental, analytical and numerical methods will be used in developing the guidelines. This research provides a framework for CPB sill beam design that can then be validated in the field in future studies.

1.2 Contributions of Research

The contributions of this research to mining operations can be articulated in terms of mine production and safety. The production benefits of this research to mine operations will be as follows:

- Reduction of ground support requirements based on analytical and numerical study;
- Improved access time to ore blocks based on design strength of CPB through statistical forecasting models;
- Improved efficiency of mining wider ore blocks based on structural sill beam stability; and
- Cost-efficiencies through understanding of CPB cement mix-designs.

The safety improvements for operations are:

- Development of in-situ testing methods of CPB;
- Improved understanding of sill beam stability reduce the risk posed to miners; and
- Improved understanding of ground support will reduce risk of rock fall.

The theoretical contributions will be on disseminating knowledge of the CPB UCF in a unified project. Current research is typically restricted to partial studies of the overall picture. This research will be comprehensive regarding the stability of CPB sill beams. The refinement of current analytical models is necessary since few studies have created new methods since the original work by Mitchell, Olsen and Smith (1981). This research will provide an updated set of analytical equations for or beam stability. Numerical model studies have provided their methodology and procedures. A black-box approach to the results is presented in the literature.
with only the results discussed. This research will provide an open-sourced version of the constitutive models. Further, explicit design methodology of numerical analysis will be provided. This will develop a framework for future CPB sill designs. As CPB gains usage in the mining industry, the stress-strain path of the material will be crucial in further studies. This research will build upon current constitutive models in defining the behaviour of the CPB under axial load. The research will achieve an understanding of the role of numerical models in sill beam design. This research will provide an investigation for in-situ testing methods of CPB. This will contribute insight into the strength development of CPB with respect to its environment. In conclusion, this research hopes to update the theoretical knowledge of CPB sill beam stability from previous studies.

UCF mining is becoming more prevalent in mining due to the exploitation of weak or highly stressed ore bodies. Despite its increase in application, research on UCF has been narrow in its scope. Of the studies presented, the majority have been site-specific or lacking depth to all facets of design. As such, there is no comprehensive design package that is suitable for the mining industry. The outcome of the research is a comprehensive design package for underhand cut-and-fill mining with cemented paste backfill. The design package will detail individual design parameters. By addressing design parameters individually as part of building the holistic model, it allows the research to be modular. The advantage of this is it allows applicable portions of the research to be applied to other mining methods or backfill types. In my opinion, the most important aspects of this research to the reader are the empirical data and field observations. This provides the reader with an empirical reference to current practices. The empirical guide provides a context in which future research and design can be built within and, where warranted, beyond.

1.3 General Overview

Underhand cut-and-fill is a sequential mining method. Generally the ore is mined from a top down method (Figure 1-1), with some mines using a drift and fill strategy when performing UCF mining. The sill beam is defined as the consolidated cemented past backfill that is directly above the current mining level replacing weak rock that may otherwise create an unstable back.
The sill beam consists of the CPB and the necessary ground support to maintain structural stability. For UCF mining the typical production cycle is as follows:

- Mining and mucking of planned stope block;
- Sill beam preparation:
  - Construction of a sill beam consisting of either waste rock, stand-up rebar, shear paddles or timber lagging is performed in some operations;
  - Construction of a bulkhead to retain fill, and installation of paste lines to the mined out stope;
- Backfill placement and curing; and
- Mining, with support on advance, of the subsequent block underneath the recently backfilled stope (sill beam).

Mah, Tessier and Clelland (2003) discuss the advantages of UCF:

- the back is engineered to provide a safe working environment in high stress and weak rock areas;
- wall dilution is low in comparison to mechanical overhand cut-and-fill; and
- seismic risk is reduced.

Figure 1-1: Mechanized underhand cut and fill

From the researcher’s observations of mines employing this method, there was very little commonality between operating mines. Although the concept of underhand mining method is the same, the execution varies. One of the objectives of this research is to outline the common themes of UCF mining across different operations.
One difference between operations that needs to be addressed is consistency in terminology. The sill beam is defined in this research as the consolidate backfill and included support that is directly above the current stope. The sill mat is defined as the ground support specifically related to the sill beam placed in advance of the placement of backfill. It was found during this research that these two terms are used interchangeably.

UCF mining is apt in mitigating the presence of weak rock mass, burst prone or converging ground. A weak rock mass is a rock mass with an RMR\textsubscript{76} rating less than 45 (Brady, Pakalnis and Clark, 2005); or a GSI rating less than 30 (Martin, Kaiser and Christiansson, 2003). The NGI Q rating described by Barton (1973) is difficult in defining weak ground due to the influence of the SRF factor. However, a Q rating below 1.0 is considered weak rock. Despite difference in rock mass ratings, the characterization of weak rock is consistent. Weak rock masses tend to yield and results in squeezing ground.

Mining a weak back or in a yielding environment poses large risks to the miner. Wedge failures, excessive displacement and rehabilitation of failed ground support are common. The Kencana mine typifies a weak rock mass environment. The use of the UCF mining method in weak ground provides an engineered back that can accommodate strains, and improved ground conditions in the back and adjacent walls. In addition, the UCF mining method assists in redistributing stress around the stope blocks while providing localized support. For cut and fill mining in weak rock masses, the cemented fill provides far-field and local support to the mine.

Paste fill in weak rock provides an engineered back that is comparatively stronger than the host rock it replaces. In high stress mining, the advantage of paste fill differs: the lower stiffness of CPB material to the host rock is beneficial. Martin et al. (2003) define high stress ground as areas where the principal in-situ stress is 15% of the unconfined compressive strength of the rock mass. In high stress areas there is a risk of rock bursting. Rock bursting will occur in areas where the mine loading systems is soft in comparison to the surrounding rock mass (Blake and Hedley, 2003). The 'softening' of the mine loading system is due to the mining of stopes within the mine boundary. The change in local stiffness reduces the ability of the mine system to ‘shed’ stresses. This results in built up energy released in a sudden violent failure, or rock-burst. The advantage of using a CPB sill beam in high stress conditions is that the CPB does not take
significant load due to the differential stiffness. The sill is not affected by seismic events and large voids are not present in the fill.

The advantages of UCF mining in difficult ground conditions, both weak rock and high-stress, provides operations with mineral resources that were previously uneconomical. The engineering justification to the use of UCF with paste backfill in challenging ground conditions is a research objective of this dissertation.

The choice of backfill is typically the following three: paste backfill, cemented rock fill and hydraulic backfill. Cemented rockfill and hydraulic fill present their own respective challenges: material handling at depth and drainage issues. There was a need for a new backfill type for deposits at depth that support rapid mining.

Following in the footsteps of hydraulic fill, paste backfill was developed as a way to limit environmental footprint. The fill was pumped underground, reducing the volume of tails deposited on surface. Hydraulic backfill has low hydraulic conductivities and required excess water to pump the slurry. This combined to cause ponding water and large hydraulic gradients within the fill. These issues posed risks: the risk of static-liquefaction, excess use of limited water and variability within the mix due to the hydraulic transportation of fines within the material (including cement). To mitigate the risks lengthy backfill placement was required to let water percolate before placement of subsequent lifts. Paste backfill, on the other hand, does not settle and free water does not pond above the paste. This is the main advantage of paste in comparison to hydraulic fill: it allows for faster placement. The faster placement of paste has a large effect on mine productivity.

Potvin, Thomas and Fourie (2005), define paste as follows:

- contains at least 15% passing 20 microns,
- when placed does not bleed water,
- does not settle or segregate in a pipeline,
- has a slump of less than 230 mm,
- contains typically between 75% and 85% solid by weight, and
- contains between 1% and 10% binder.
With the realization on the transportation and operational efficiencies, industry was quick to adopt paste backfill (Udd, 1989 and Udd & Annor, 1993). However, the incorporation of paste backfill at first was poorly utilized. Initial designs were based on principals relating to hydraulic fill (Udd, 1989). This led to over-designed CPB strengths and unrealistic behaviours of CPB. CPB design in UCF mining was not intrinsically developed; instead CPB behaviour was based on hydraulic fill.

1.4 Thesis Outline

This thesis consists of eleven chapters. The Chapters include an introduction to cemented paste backfill application to mining, a literature study of current research, experimental and observational results, subsequent analysis, development of design guidelines and suggestions for future work.

Chapter 2 studies the current literature on cemented paste backfill and underground cut and fill mining. This chapter outlines the current gaps in research and supports the development of the research methods.

Chapter 3 outlines the methodology of the research. This chapter details the observational, empirical and analytical techniques used as the framework for the study. Limitations, bias and repeatability of the proposed methods are discussed.

Chapter 4 presents the results of the observational portion of the research. The chapter present results from field and laboratory studies. The material in Chapter 4 is incorporated into later analytical and numerical analysis.

Chapter 5 is a review and description of mines that utilizes underhand cut and fill mining method with CPB. Using the case-study format, this chapter presents relevant information of five mine sites. This chapter’s purpose defines the common practices at mines and allows for a comparison between operations.

Chapter 6 investigates paste backfill under load. This chapter discusses the suitability of current constitutive models. A new approach to constitutive modeling of CPB under axial load is presented. Chapter 6 also discusses the ability of CPB to withstand seismic energy.
Chapter 7 is an analytical approach to sill beam stability. Comparison between existing analytical models is investigated. Sill beam stability is discussed in terms of parametric and probabilistic methods. The chapter will establish a proper analytical model for determining sill beam stability.

Chapter 8 determines the stability of the sill beam with numerical analyses. The numerical model analysis will incorporate model properties and parameters that are difficult to capture in a simple analytical model.

Chapter 9 presents a discussion between the observational, analytical and numerical analysis. Design curves will be established in this chapter.

After all the hard work is complete, Chapter 10 presents the detailed guidelines of designing CPB sill beams. This chapter outlines required strengths, ground support and maximum widths for various ground types for stable sill beams.

Chapter 11 presents the conclusions of the research. Key findings, answering the original hypothesis and summarizing the contribution of this research to the state-of-the-art of underhand cut and fill research is summarized. Chapter 12 discusses research goals that were not answered with the research and outlines possible future studies.
2 LITERATURE STUDY

The literature, as it will be presented, demonstrates that individual portions of the complete design of CPB UCF mining has been well researched and presented; however, there lacks a unified approach to the research. The literature separates itself into two distinct streams: sill beam stability and CPB behaviour.

The sill beam stability is based on the culminations of other disciplines including material storage (Janssen, 1895; Koenen, 1896; modified by Reimbert and Reimbert, 1976) and soil excavations (Marston, 1930 and Terzaghi, 1943). From these traditional approaches, work by Blight (1984) and the works of Mitchell (Mitchell, Smith and Libby (1975), Mitchell and Smith (1979), Mitchel, Olsen and Smith (1982), Mitchell and Wong (1982), Mitchell and Stone (1987), Mitchell (1989a), Mitchell (1989b), Mitchell (1991), and Mitchell and Roettger (1989), Nnandi and Mitchell (1991) began investigating the development of stresses within placed fill and the stability of the fill after a horizontal face was exposed. Building on undercut studies, Krauland and Stille (1993); Ley, Steed, Brockhurst and Gustas (1998); Jordan, Langston, Kirsten, Marjerison, Jacobs, and Stahlbush (2003); Williams, Denton, Larson, Rains, Seymour, and Tesarik (2001) discuss these application to underhand cut and fill mining. Stresses within fill have been discussed by many, with much of the work focused on the early-strength and stresses developed in backfill, summarized by Fahey, Helinski and Fourie (2009).

Properties of CPB and the behaviour of studies have been presented by Fall, Belem, Samb and Benzaazoua (2007), Rankine and Sivakugan (2007), Klein and Simon (2006), Tesarik, Seymour, and Jones (2003) and Amaratunga & Yaschyshyn (1997).

Differing from laboratory study and theoretical stability of sill beams, valuable information on UCF and CPB has been provided by the case study approach. Williams et al. (2001); Jordan et al. (2003); Newman, Pine and Ross (2001); Febrian, Wahyudin, Gunadi, Peterson, Mah and Pakalnis (2007) amongst others present case studies on operating mines using UCF.

With respect to the design guidelines for UCF mining with CPB, Pakalnis et al. (2005); Caceres (2005); Mitchell (Mitchell and Stone (1987), Mitchell (1989a), Mitchell (1991),
Mitchell (1989b) and Mitchell and Roettger (1989); Stone (1993); and Jordan et al. (2003) provide methodologies for designing sill beams from a minimum strength standpoint. The stability of the CPB sill is integral to this thesis. The stability of CPB sill beams will be built on the analytical assessment derived in the above studies with the incorporation of findings from this study.

Analytical assessments are limited by their assumptions and simplification of design elements. Numerical modeling is able to address the simplifications. The use of numerical code in determining the performance of CPB during mining is discussed by Swan and Brummer (2001); Pierce (2001); Andrieux, Brummer, Mortazavi (2003); Helinski (2008); Helinski, Fourie and Fahey (2010a, 2010b); and Veenstra, Bawden, Grabinsky and Thompson (2011). The methodology and approach towards numerical modeling from these studies will be incorporated in this research.

The literature on CPB is well researched but fragmented and is not robust enough to be considered a design guideline. This research anticipates the development of a design guideline by combining the available research with the knowledge gained from a thorough study. The literature search will be presented based on the design components of the sill beam. For visual reference, Figure 2-1 (after Pakalnis et al., 2005) is a schematic of the design requirements.

Figure 2-1: Design elements of underhand cut-and-fill (after Pakalnis et al., 2005)
The literature search discusses the papers that were presented on each of the design elements as indicated above. The intent of the literature study is to provide an outline of previous research, determining where the state-of-the-art requires further study and to outline which materials will be included in the analysis and construction of design guidelines for underhand cut-and-fill mining.

### 2.1 Development of Vertical Stresses

Vertical stresses generated in backfill are less than a static load due to the transference of stress to the side walls due to arching. The study of vertical stress distribution in backfill stopes has been performed by Mitchell, Olsen and Smith (1982); Handy (1985); Grice (2004); Aubertin, Li, Arnoldi, Belem, Benzaazoua, and Simon (2003); Pirapakaran and Sivakugan (2007); Fahey et al. (2009); Singh, Shukla and Sivakugan (2011). The application of these studies to UCF CPB mining is limited as they consider the fill un cemented (cohesion = 0 kPa). These analyses have little bearing on CPB UCF as the CPB has a cohesive element integral to the success of the mining method.

The equation proposed by Blight (1984), built on the works of Reimbert and Reimbert (1976) and Terzaghi (1943) is suitable in assessing vertical stress within a CPB stope. Blight (1984) proposes the first order linear equation is used to calculate the lateral earth pressures:

\[
\sigma_z = \frac{\gamma \sin \beta - \frac{2c'}{w}}{K_o \tan \phi} \left(1 - e^{-K_o \tan \beta \cdot z}\right)
\]

*Equation 2-1*

where:
- \( K_o \) = co-efficient of lateral earth pressure at rest
- \( \phi \) = friction angle of fill
- \( c' \) = cohesion (kPa)
- \( K_o = 1 - \sin \phi \)
- \( \beta \) = Stope dip
- \( w \) = sill width perpendicular to the hanging wall (m)

The Blight model (Blight, 1984) will be used to analytically assess the vertical stress generated within CPB stopes before undercutting. This is due to the analysis considering the effects of cohesion. The limitation of the Blight model is the consideration that the stopes satisfy
plane-strain conditions. However, Mitchell and Roettger (1989) found that plane strain conditions are typically satisfied in UCF conditions.

2.2 Development of Horizontal Stresses in Fill

The understanding of horizontal stresses, both internal and applied, are essential in stability of sill beams. As the sill is undercut, the lower boundary becomes unrestricted and vertical movement can occur. The horizontal stresses in the fill allow the sill to interact with the sidewalls limiting movement through confinement and frictional forces. The issue with the horizontal stresses for CPB operations is the transitions from a saturated and uncemented fill (no cohesion) to a hydrated mass with cohesion. Over time the hydration process provides a cohesive element to the backfill during the consolidation of the fill; Helinski et al. (2010a) discuss the complicated process of stresses generated within the fill during placement and hydration of the backfill.

Once under cut, the consolidated sill relies on the internal strength and shear strength along interfaces to prevent failure (Mitchell and Roettger, 1989). Below is a discussion on internal stresses developed in the fill from classic soil mechanics theory, followed by discussion on the effect of closure on stresses and strains within the fill.

Ultimately, the discussion is to lead into study on the appropriate lateral-earth co-efficient and horizontal stress profiles within the fill.

2.2.1 Development of Self-Arch

The horizontal earth pressure models by Marston (1930), Terzaghi (1943), and Reimbert and Reimbert (1976) are based on the development of lateral earth pressures through the mobilization of shear strength within this fill. With these approaches the backfill develops a lateral earth pressure that is less than vertical pressure. Further, with the mobilization of shear strength the values of the vertical pressures are less than the unit weight multiplied by the height. This is due to a mobilized shear strength on the side walls which creates a bridging effect.

For cemented paste backfill (CPB) the calculation of horizontal pressures are required to determine the shear strength resistance between the sidewall and paste backfill. It should be
noted, that CPB utilizes a cohesive value and as such the models need to be tailored to accommodate this parameter of the CPB.

The seed of lateral earth pressure models as related to soil mechanics are based on soil mechanic theory presented by Marston (1930) and Terzaghi (1943). These theories are discussed in detail with significant modifications discussed below.

2.2.1.1 The Marston’s Cohesionless Model

The Marston’s Cohesionless Model was developed to quantify the lateral earth pressures at the base of trench excavations. This method is not well suited to CPB cut-and-fills analysis, as it uses the active earth pressure. In CPB cut-and-fill the material must be considered at rest or in passive conditions as the sidewalls are either exerting a force on the walls, or have completed all displacement prior to placement of paste. As such, the CPB would be considered at rest (Fall et al., 2007). This idea of the fill being at rest is addressed by Terzaghi (1943) in his Cohesive Materials Model.

2.2.1.2 Terzaghi’s Cohesive Materials Model

Terzaghi’s Cohesive Materials Model (1943) predicts horizontal earth pressures at the base of an excavation. In comparison to silo models (Reimbert and Reimbert, 1976) and Marston’s cohesionless model, the Terzaghi model is better suited to CPB as it has both a cohesive value and relies on at rest conditions for the fill. For a cohesive material the Terzaghi model equation is as follows:

\[
\sigma_h = \frac{\gamma B - 2c}{2 \tan \phi} \left[ 1 - \exp \left( \frac{-2KH \tan \phi}{B} \right) \right]
\]

*Equation 2-2*

where:

- \(K = \frac{1}{1+2\tan^2 \Phi}\)
- \(\Phi = \) fill internal friction angle
- \(c = \) fill cohesive strength (kPa)
- \(\gamma = \) fill bulk unit weight (kN/m3)
- \(B = \) Stope Width (m)
- \(H = \) fill height (m)
2.2.1.3 Blight’s Lateral Earth Pressure Model

Blight (1984) assessed the stress within a column of fill utilizing theories of Terzaghi (1943), and Reimbert and Reimbert (1976). Blight argued that three conditions exist that require stresses in columns of fill that need to be calculated, they are:

1. Ensuring that cumulative frictional force along the sidewalls is adequate to retain the fill in the stope,
2. Estimate the distribution of lateral stress provided by the fill to quantify the amount of deformation of sidewalls, and
3. Cemented plugs need to have the frictional components mobilized to ensure a safe design against failure.

For underhand cut-and-fill, point #1 is of significant concern as the frictional force between the backfill and sidewalls is of utmost importance. The mobilized shear friction in absence of ground support is the only ground support that is provided to the sill beam when undercut. Blight quantifies the vertical and horizontal pressures in the following equation.

\[
\sigma_z = \left( \frac{\gamma \sin \beta - \frac{2c'}{w}}{K \tan \phi} \right) \left( 1 - e^{-K_z \tan \phi \frac{z}{z_o}} \right)
\]

where:

\[ \sigma_h = K_o \sigma_z \]

In essence, Blight’s (1984) is similar to that of Terzaghi (1943), with the differences contained within the methods in which the properties of the material are defined. The incorporation of the dip of the stope and cohesion makes the Blight model suitable for use for assessing both vertical and horizontal stresses within a CPB sill beam.

2.2.1.4 Silo Theory

Silo theory discussed by Reimbert and Reimbert (1976) calculates the lateral pressures of granular material founded on the works of Janssen (1895) and Koenen (1896) through experimental testing. Silo theory relies on the internal frictional properties of cohesionless
materials to create a self-supporting network of interlocking grains that reduce both the horizontal and vertical loads.

Caceres (2005); Handy (1985); Grice (2004); Aubertin et al. (2003); Pirapakaran and Sivakugan (2007); Fahey et al. (2009); and Singh, Shukla and Sivakugan (2011) considered the Reimbert model in assessing horizontal stresses. In these cases large stopes and, in some cases, cohesionless material was part of the analysis. This is not the case with UCF CPB: cohesion is present within the sill beam. This study will not use the Reimbert silo model (1976) based on the lack of cohesion within the horizontal stress calculations.

An additional horizontal stress model was developed by Aubertin, et al. (2003). Their proposed model is similar to the Terzaghi Cohesionless Model but uses the fill’s effective friction angle rather than the internal friction angle. By using the effective friction angle, it is argued by Aubertin et al. (2003) that it accounts for the role of the cohesion. This statement is confusing when applied to CPB with intrinsic cohesion and makes the model poorly suited for CPB horizontal stress calculations.

Models by Van Horn (1963) and Li, Aubertin and Belem (2005) propose alterations to the above models to be used in three dimensions by means of introduction of a parameter for stope length. As plane-strain conditions are satisfied (Mitchell and Roettger, 1989) for UCF mining, these models were not considered.

Another method to determine lateral stresses is the Voussoir Arch. Studies of this method, as applied to rock mechanics, are Beer and Meek (1982); Brady and Brown (1993); and Sofianos and Kapenis (1998). The principal of the arch is the development of lateral stresses within a horizontal beam based on non-uniform horizontal thrust. The assumptions of the Voussoir Arch are: no wall closure, no volumetric expansion of material (ν = 0) and elastic, homogenous conditions. These assumptions do not reflect the environment of UCF mining.

The models presented account for the horizontal and vertical load acting on the sill floor and sidewalls. For all the models the following are essential in calculating horizontal loads:

- The state of the backfill (active, at rest or passive conditions),
- The effect of arching (dependant on the internal friction angle),
• The weight of the fill (the larger the weight requires a larger mobilized shear resistance),
• The shear resistance (the effective friction angle between the sidewalls and the backfill),
• The width of the stope (the wider the stope, the more the backfill is required to self-support its own weight), and
• The cohesion of the material has a large effect on the lateral earth pressures.

The models suggested by Terzaghi and Blight are the most applicable in determining horizontal loads within cemented paste backfill: both account for the effect of cohesion. The models diverge in accounting for the state of the backfill; the state of the backfill is difficult to assess (per Marcinyshyn, 1996). The Terzaghi model assesses the horizontal stress without considering the state of the fill. Chapter 6 will present a comparison of the two models to determine the correct horizontal stress model for this research.

### 2.2.2 Wall Closure

Apart from stresses generated within the fill from self-arching, stresses within the fill can also be generated by closure of wall stopes. Elastic convergence of wall stopes and plastic deformation are possible methods of imposed lateral loads. The applied closure against a beam can lead to failure (Timoshenko and Gere, 1961). However, per Mitchell and Roettger (1989), the closure can also increase the factor of safety from increasing normal forces along frictional interfaces within the stope. The question with horizontal stresses within fill is the following: do the self-developed or closure stresses dominate? This will be addressed in the analytical research.

The idea of the behaviour of fill in closure is discussed by Grice (2004) and Marcinyshyn (1996). Grice (2004) addresses closure stresses by investigating the role of fill stiffness with respect to closure stresses. Marcinyshyn (1996) argues that in narrow stopes the fill is in passive conditions, during stress changes, the backfill approaches at rest conditions. The state of the backfill during mining will be effect by the amount of closure of the stope.

Krauland and Stille (1993) demonstrate that the rock side walls complete the elastic displacement prior to the placement of fill. However, that only accounts for the placement of fill; it was found that once the stope was undercut, additional convergence occurred. According
to elastic theory (Hoek and Brown, 1980), convergence occurs and will continue to occur during the mining of a lower stope in a UCF environment. Each additional sequential stope will increase the amount of convergence on a sill beam.

Previous studies (Mitchell, 1991; Marcinyshyn, 1996; Pierce, 2001; Helinski et al., 2010a & 2010b) discuss closures for stopes with heights larger than stope widths. These proportions are not common dimensions in UCF environments: stope heights are typically equal to the widths. The closure stresses of these studies are not applicable to UCF mining.

Krauland and Stille (1993) discuss the effect of closure on the sill beam stability at UCF Garpenberg Mine. The findings with respect to convergence, demonstrate that 25 m wide fill mat converges 24 mm after the mining of cut one, and the 60 mm after the final three cuts (Krauland and Stille, 1993, p. 3). This demonstrates that closure occurs over the course of mining and attenuates after completion of mining and stress has stabilized. Further, Krauland and Stille (1993) found that the vertical and horizontal stresses were measured throughout the mining process. In comparing the horizontal and vertical stress, for the initial cuts of mining the horizontal stresses are greater than the vertical. Once the mining ceases in the area the vertical stresses exceed the horizontal stresses. This is the only known study that allows a direct comparison of horizontal and vertical earth pressures. Krauland and Stille (1993) is an important study as it demonstrates that when closure occurs the closure stresses generated exceed those of the self-generated horizontal stresses.

Williams et al. (2001) studied lateral closure as part of a study on reinforced backfill at Lucky Friday Mine, Idaho. The study encountered site specific issues making comparisons to other mines difficult. However, the instrumentation program found approximately 150 mm of closure over a 3 m wide stope after the mining of three successive undercuts. This corresponds to approximately 5 percent strain within the backfill and exceeds the maximum allowable strain of the backfill at Lucky Friday. This finding is significant: the sidewall closure strains can exceed the compressive strength of the backfill and sill beams remain stable in post-peak stress-strain environments.

It can be seen that estimation of lateral stresses and the distribution of lateral stresses are not unified in the literature. Terzaghi theory presented by Blight (1984) considers the stresses
generated within the fill at rest and the closure of the stopes are absent from design. Although internal stresses are being generated, the closure of the stope walls must be considered as discussed by Marcinyszyn (1996) and Krauland and Stille (1993). The issue of horizontal stresses has a large effect on stability as it increases the normal force along the interfaces. In addition, closure can lead to buckling or compressive failures if the strains exceed the ultimate strain of the material. This research will study the proposed analytical models and determine which failure types are the most critical.

2.3 Failure Modes

Understanding the stresses within the fill is essential in knowing the initial behaviour of the backfill. However, once the fill is undercut, and a free face is formed, the strength of the backfill sill beam must be strong enough to prevent failure. Failure of sill beams can take many forms and numerous methods are presented in their analysis. Model testing, analytical, numerical, observational, and empirical studies are employed in engineering design of CPB sill beams.

This section discusses the initial studies of Mitchell (1987, 1989a, 1989b and 1991) with comments from Caceres (2005); Stillwater methodology (Jordan et al., 2003); Stratoni Mine (Newman, Pine and Ross, 2001); beam theory design (Stone, 1993); numerical approaches (Brechtel, Struble and Gunther, 1999)); observational approach (Krauland and Stille, 1993); and empirical studies by Pakalnis et al. (2005). This section will demonstrate the research work that has been published with respect to failures of sill beams. What is surprising is the research presents modeled sill beam failures, but none present the analysis in terms of actual sill beam failures.

2.3.1 Mitchell Method for Sill Beam Design

R.J. Mitchell developed a majority of research on sill beam failures typically focusing on centrifuge modelling. The key papers on this research are: Mitchell and Stone (1987), Mitchell (1989a), Mitchell (1991), Mitchell (1989b) and Mitchell and Roettger (1989). The Mitchell analytical approach to sill beam stability does not take into account the effect of ground support,
in situ stresses, and, as noted by Caceres (2005) the effect of shear strength on the hangingwall in rotational failure.

Mitchell observed five types of possible failures within a sill beam: rotational, sliding, caving, crushing and flexural. Rotational and sliding are considered functions of the backfill/rock interface and caving, crushing, flexural are functions of the internal properties of the backfill. Figure 2-2 is a schematic of these failure methods.

Mitchell’s analysis defines a stable sill beam as one that has sufficient mobilized bending and shear strength to counter the following stresses:

- a non-uniform vertical stress (due to arching of overlying backfill),
- the sill beam self-weight,
- lateral closure stresses,
- movement along the sidewalls, and
- the rotation of the sill beam.

An important finding of Mitchell (1989a & Mitchell and Roettger, 1989) is that the sill beams are of such a length that plane strain can be considered. In this, only the width and height of the stope needs to be considered, and a two dimensional analysis is all that is required for sill beam stability.
To achieve sliding along the wall/backfill interface, Mitchell argues that this will only occur if the interface is of low friction, such that limit shear strength is achieved. The issue with the sliding equation is that the shear strength along the interface is hard to quantify. Research by Fall and Nasir (2010) attempt to parameterize the interface shear strength parameters for a proposed mix design. However, the general behaviour and range of values of the shear strength and shear parameters respectively is not available at present. The effect of stope dip, in-situ stresses, wall roughness, water pressure, all need to be accounted for in the sliding equation; the author has found that these values are not measured but are assumed based on engineering assumptions. One of the key constraints to assessing the sliding failure is that the sill beam fails as a single element and is unable to have a composite failure.

A caving failure will occur in stopes that are thick and narrow. The caving failure is independent of the excess vertical stress. Mitchell assumes that caving would extend to a stable arch with a height that is half the width of the stope.

The caving equation by Mitchell is either in error, or has an unstated conservative approach. The resistance of the fill is only provided across the length of the stope and not over
the curved failure surface, assumed by Mitchell to be half the stope width. That is to say, that the resistance to failure should be the tensile strength acting along the perimeter of the cave surface, and not the tensile strength of the beam across the length of the stope.

Flexural failure is critical in certain conditions given the Mitchell equations (Stone, 1993; Mitchell and Roettger, 1989). Flexural failure occurs in stopes that have a comparatively small height to width ratio; causing the beam to fail in tension due to excessive displacements from self-weight loading. The flexural failure is similar to beam theory models presented by Timoshenko and Gere (1961).

In the Mitchell analysis the sill beam is fixed to the sidewall and no movement is to occur, thereby implying that flexural failure will occur before shear failure. Mitchell (1989a) describes the beam as simply a supported beam. This description leads to a more conservative approach to a fixed supported beam as it requires greater strength to fail in flexure.

Rotational failure, according to Mitchell occurs when the hangingwall/backfill interface separates and the sill beam rotates along a linear failure surface. This failure type was shown to be valid in centrifuge models studied by Mitchell (1989a and 1991) and was prevalent in stopes with low dips. Caceres (2005) found that the effect of the friction of the hangingwall should not be ignored. Caceres argues that in steeply dipping stopes there exists contact between the hangingwall and backfill and as such shear resistance would need to be overcome to have a rotational failure.

Further, Caceres proposes that with Cemented Rock Fill (CRF), in comparison to hydraulic fill (HF) and CPB, the interface between the hangingwall and the fill will have a larger frictional component that should be accounted for in the stability equation. Additionally, Dirige, De Souza, and Chew (2001) found, through centrifuge modelling, that the roughness of the footwall provides additional strength to the sill beam due to the increase in shear resistance. This implies that the roughness of the fill has the same impact on the interface as the wall rock roughness.

Caceres proposes, based on his calculations that the shear resistance of the hangingwall should be included in the rotational analysis. The equation proposed by Caceres properly
represents the actual conditions present in rotational failures; however, the Mitchell method is more conservative. Both methods require parameters values to be either tested or assumed. The Mitchell method, as noted by Caceres (2005), has numerous assumptions and simplifications that, as argued here, limits its use to engineering outside of a scoping study.

Marcinyshyn provides a good summary of sill beam stability with respect to Mitchell’s (Marcinyshyn, 1996, p.19):

- Cemented sills are stable if large lateral closures are not present in the stope provided there is good wall anchorage;
- Supported is necessary when large lateral closures are expected, noted as timber support;
- Steel support is preferred in comparison to timber support; and
- Rotational failure is most prominent for cases where stopes have a dip of 70° or less.

The suitability of the Mitchell analysis in terms of design strengths for stable sill beam configurations will be investigated as part of the analytical study of this research.

### 2.3.2 Stillwater Mine Design Methodology

Jordan et al. (2003) presents the design guidelines for sill beams at Stillwater Mine, Nye, Montana. The Stillwater design addresses beam detachment, rotational, tensile cracking, shear failure, beam sliding, beam compression and beam relaxation. These failure mechanics are discussed in how they apply to the critical design of the sill beam.

The principle behind beam detachment is the sill beam will fail if the footwall and hanging wall dip in opposite directions and diverge more than 15 degrees. This type of failure could occur since little to no frictional resistance is mobilized along the backfill/side walls and the shear resistance would rely purely on the cohesion between the backfill and the side walls. It should be pointed out, that few stopes would have this geometry as its layout would leave ore in the stope or have increased dilution and should be eliminated at the planning stage of operation.

Jordan et al. (2003) found that rotational failure is not common within their stopes and will only occur when the critical angle of rotation ($\psi$) is equal to or exceeds 90 degrees, as defined in the following equation:
\[
\cot \psi = -\cot \alpha + \frac{h \cdot \cos ec^2 \alpha}{h \cdot \cot \alpha - h \cdot \cot \beta + w}
\]

*Equation 2-4*

where:

- \( \psi \) = critical angle of rotation
- \( h \) = height of paste beam (m)
- \( w \) = width of stope (m)
- \( \alpha \) = inclination of hangingwall in degrees
- \( \beta \) = inclination of footwall in degrees

The rotational theory presented by Jordan et al. (2003) differs from Mitchell (1989a) in that the constrictions on failure are geometric and not associated with the strength of the material. Rotational failure was not considered the critical failure at Stillwater.

Tensile cracking was found to be the most critical of failure types for sill beam according to Jordan et al. (2003). The tensile failure will occur when the tensile stress in a simply supported beam at mid span exceeds the tensile strength of the paste fill. The equation for tensile stress is defined below and parallels the flexural failure of a sill beam discussed by Mitchell (1989a). Beam failure would occur if the tensile stress exceeds ten percent of the unconfined compressive strength of the backfill. The equation as presented is considered conservative as it does not take into account the effect of the normal forces provided by the converging sidewalls. The beam failure equation is the fundamental equation for simply supported beams (Timoshenko and Gere, 1961; Mott, 2002)

\[
\sigma_t = \frac{3}{4} \left( \frac{\gamma \cdot L^2}{h} \right)
\]

*Equation 2-5*

Shear failure proposed by Jordan et al. (2003) is defined as follows:

\[
\tau = \frac{1}{2} \frac{\gamma \cdot L}{h}
\]

*Equation 2-6*

The one-half factor in the equation is an approximation for the coefficient of lateral earth pressure. No attempt to measure the shear strength is provided by Jordan et al. (2003); the approach presented contains numerous simplifications and ignores the properties of the hangingwall and backfill interface. Jordan et al. (2003) argue that since tensile failure
dominates, the need to quantify shear strength is unnecessary, however for a complete sill beam design the shear strength of the material must be considered.

In general, sill beam sliding is defined as when the FW is steeper than the HW and sliding occurs along the steeper plane. Jordan et al. (2003) point out that tensile failure is more critical than sliding failure and that sliding failure would not occur prior to a tensile failure.

Beam crushing occurs due to the convergence of the stope. The resistance against failure is based on the stiffness and strength of the fill. Jordan et al. (2003) state that although crushing failure is numerically possible, crushing failures have not been observed at Stillwater. Pakalnis and Associates (2012) provides the explanation on why crushing is not an issue with sill beam failures: the displacement of the sidewalls occurs almost immediately and once the backfill is in place the majority of sidewall movement has occurred.

Beam relaxation is the opposite of beam crushing and occurs to outward movement of the stope due to relaxation of the stresses surrounding the stope. This type of failure is more of a cause of sliding, rotational, or shear failure and not a failure type in its own right.

The work by Jordan et al. (2003) outlines the failure modes that are possible in sill beams. The rotational, crushing, sliding and tensile failures are discussed in depth by Mitchell (1989a). As indicated by Mitchell, HW relaxation and detachment failure must be addressed in any sill beam design.

### 2.3.3 Stratoni Mine Design Criteria

Newman, Pine and Ross (2001) detail the development of the paste sill mats at Stratoni Operations, Greece. Stratoni operates with a Longitudinal and Transverse mining method, with each method providing varying types of failures. For the Longitudinal mining, block sliding, rotational, buckling, arching and caving are possible failure modes; whereas for transverse mining, buckling, arching caving and shear failure are possible. The difference between the transverse and longitudinal mining is that transverse mining spans between cemented fill and un-mined ore (secondary cut), while longitudinal mining spans between the hangingwall and un-mined ore. Newman et al. (2001), carry out the Mitchell analysis as described above for stability of the sill mat, however they apply Beer and Meek (1982) theory to determine the arch concept
for cemented fill beams. This method proposes that a lateral force is developed through arching of the fill that increases with shortening of the arch.

As noted by Newman et al. (2001), three types of failure are possible with the arch analysis: compressive arch failure, abutment shear failure and buckling failure. The compressive arch failure occurs if the peak compressive stresses in the arch exceed the UCS of the material. Abutment shear failures will occur if the shear load exceeds the available shear strength at the abutment. The loss of the shear strength is due to a weak arch that cannot support the weight above and slides, these are more common in shorter beams as they do not develop sufficient lateral thrust. Buckling failures occur with sills with a low height to width ratio. Newman et al. (2001) describe the conditions for buckling as follows:

“The arch will be unable to form before excessive compression allows the beam to shorten significantly and 'snap-through’” (Newman et al., 2001).

From Newman et al.’s analysis, it was found that a beam of 6 m span and backfill strength of 2.1 MPa has a factor of safety of 1.5 against all types of failures. It was also found that rotational failure as analyzed by Mitchell (1989a, 1991) was more critical than flexural failure as proposed by Beer and Meek (1982). The reasons for this is that the Beer and Meek approach takes into account the arching of the sill mat thereby gaining some internal strength and confinement; whereas the Mitchell method looks at a beam failing under self-weight according to beam mechanics.

### 2.3.4 CRF Methodology

Stone (1993) uses a beam equation formula approach to develop guidelines for UCF sill beams for Cemented Rock Fill (CRF). The equation for the sill beams is not provided in the paper, however a replicated graph is provided in Figure 2-3. An analysis regarding the flexural strength of the beam is performed; however back calculation of the results does not yield flexural beam formula: it can be deduced that an altered beam formula is applied. Further, a factor of safety of 2.0 and an assumption that the UCS of the material is equal to 1/10 of the flexural strength is assumed. Stone looked at only vertical sidewalls as the investigation was for Nevada cut-and-fill operations where the HW and FW are vertical.
Figure 2-3: Required strength of CRF required for a FS of 2.0 (vertical sidewalls), from Stone (1993)

The approach presented by Stone (1993) provides minimum required UCS strength for a stable CRF beam. However, this analysis only looks at flexural failure as Stone (1993) estimates:

“…although other failures include shear at the sidewalls (slippage), rotation, caving and crushing…..(these modes) are less critical provided that sill thickness is greater than 0.5 times the unsupported span, the strength of the CRF is above 1.5 MPa and there are no closure stress…”

Stone’s approach results in very high strength values of CRF. The use of a factor of safety of 2.0 and the assumption that the flexural strength of CRF is 1/10 of the UCS are likely factors to the high strength values obtained with the use of the design chart.

It is the researcher’s experience that the design chart presented by Stone (1993) is used for design of CRF spans in Nevada operations. A refinement on the beam equation and assumed
parameters would improve Stone’s approach. In addition, Stone suggests that flexural strength governs stability, an investigation into all types of failures should be provided for a thorough analysis.

### 2.3.5 Numerical Modeling Approach

Brechtel et al. (1999) investigated the use of ITASCA’s UDEC code for numerical design of sill beams at Murray Mine, Nevada. The scope of the study was to determine the largest span possible based on the strength of the UCS of the material. Brechtel et al. (1999) determined failure would occur due to crushing. This was concluded by the observations of tension cracks within the sill beams at the mine. The numerical analysis determined that between 3.1 and 4.5 MPa backfill was suitable for a 9.1 m span. It is not clear from the study what defined a stable sill beam. The work defines a stable sill beam as one with "adequate proportions of factor of safety less than one or less" (Brechtel et al., pp. 486, 1999) with no definition of what quantifies adequate.

### 2.3.6 Empirical Approach

Pakalnis et al. (2005) provide an empirical approach to span stability. Investigation between stable spans and backfill strength at operating mines are presented in Figure 2-4 and the empirical data points are listed in Table 2-1.

The study presents the limit of span widths for stable stopes based on observed mines. The presentation of the design lines in Figure 2-4 is based on the following, per Pakalnis (2014):

> “... the database of underhand mining operations with vertical sidewalls with a FS of 2.0. The chart is based on flexural instability employing fixed beam analysis with surcharge loading. The observed is generally lower than the strengths dictated by analytical method and this could be due to QA/QC, confinement or other reasons. “

The empirical approach is essential in providing a basis for design: engineers designing UCF sill beams can reference stable cases. The need for an empirical approach cannot be overstated. It is anticipated that this research can provided further cases of stable or unstable sill beams to add to the empirical database of Pakalnis (2014).
Figure 2-4: Sill width vs. UCS with case histories (after Pakalnis, 2014)
<table>
<thead>
<tr>
<th>DATA POINT</th>
<th>MINE</th>
<th>FILL TYPE</th>
<th>% CEMENT</th>
<th>UCS (MPa)</th>
<th>SPAN (m)</th>
<th>SILL THICKNESS (m)</th>
<th>COMMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Red Lake</td>
<td>Paste</td>
<td>10.0</td>
<td>1.5</td>
<td>6.1</td>
<td>3.0</td>
<td>Design Strength Govern time to mine under sill beam (14 - 28 Days); 0.6 m Air Gap between lifts</td>
</tr>
<tr>
<td>2a</td>
<td>Anglo Gold (1999 Site visit)</td>
<td>CRF</td>
<td>6.5</td>
<td>5.5</td>
<td>7.6</td>
<td>4.6</td>
<td>Mined Remote</td>
</tr>
<tr>
<td>2b</td>
<td>Murray Mine</td>
<td>CRF</td>
<td>8.0</td>
<td>6.9</td>
<td>9.1</td>
<td>4.6</td>
<td>2” Minus Aggregate</td>
</tr>
<tr>
<td>2c</td>
<td>Queenstake (2004)</td>
<td>CRF</td>
<td>8.0</td>
<td>6.9</td>
<td>21.0</td>
<td>4.6</td>
<td>Mine under 14 day cured back; CRF 'jammed' tight to back</td>
</tr>
<tr>
<td>3</td>
<td>Eskay</td>
<td>CRF</td>
<td>7.0</td>
<td>12.0</td>
<td>3.0</td>
<td>3.0</td>
<td>Design Strength 4 MPa; 28 Day strength is 11 MPa</td>
</tr>
<tr>
<td>4a</td>
<td>Turquoise Ridge</td>
<td>CRF</td>
<td>9.0</td>
<td>8.3</td>
<td>13.7</td>
<td>4.0</td>
<td>CRF test panel</td>
</tr>
<tr>
<td>4b</td>
<td>Turquoise Ridge</td>
<td>CRF</td>
<td>9.0</td>
<td>8.3</td>
<td>3.7</td>
<td>3.0</td>
<td>CRF drift and fill</td>
</tr>
<tr>
<td>4c</td>
<td>Turquoise Ridge</td>
<td>CRF</td>
<td>9.0</td>
<td>8.3</td>
<td>7.3</td>
<td>3.0</td>
<td>CRF panel</td>
</tr>
<tr>
<td>5</td>
<td>Midas</td>
<td>CRF</td>
<td>7.0</td>
<td>3.4</td>
<td>2.7</td>
<td>3.0</td>
<td>Go under paste 7 to 28 days after pour</td>
</tr>
<tr>
<td>6</td>
<td>Deep Post</td>
<td>CRF</td>
<td>6.8</td>
<td>4.8</td>
<td>4.9</td>
<td>4.3</td>
<td>Go under paste 7 to 28 days after pour; Stress (~800 m)</td>
</tr>
<tr>
<td>7a</td>
<td>Stillwater - Nye Operation</td>
<td>Paste</td>
<td>10.0</td>
<td>0.3</td>
<td>1.8</td>
<td>2.7</td>
<td>Design based on beam theory</td>
</tr>
<tr>
<td>7b</td>
<td>Stillwater - Nye Operation</td>
<td>Paste</td>
<td>10.0</td>
<td>0.5</td>
<td>2.4</td>
<td>2.7</td>
<td>Factor of safety of 1.5 in design</td>
</tr>
<tr>
<td>7c</td>
<td>Stillwater - Nye Operation</td>
<td>Paste</td>
<td>10.0</td>
<td>0.7</td>
<td>3.0</td>
<td>2.7</td>
<td>Factor of safety of 1.5 in design</td>
</tr>
<tr>
<td>7d</td>
<td>Stillwater - Nye Operation</td>
<td>Paste</td>
<td>10.0</td>
<td>1.0</td>
<td>3.7</td>
<td>2.7</td>
<td>Factor of safety of 1.5 in design</td>
</tr>
<tr>
<td>7e</td>
<td>Stillwater - Nye Operation</td>
<td>Paste</td>
<td>10.0</td>
<td>1.4</td>
<td>4.3</td>
<td>2.7</td>
<td>Factor of safety of 1.5 in design</td>
</tr>
<tr>
<td>7f</td>
<td>Stillwater - Nye Operation</td>
<td>Paste</td>
<td>10.0</td>
<td>1.8</td>
<td>4.9</td>
<td>2.7</td>
<td>Factor of safety of 1.5 in design</td>
</tr>
<tr>
<td>7g</td>
<td>Stillwater - Nye Operation</td>
<td>Paste</td>
<td>10.0</td>
<td>2.3</td>
<td>5.5</td>
<td>2.7</td>
<td>Factor of safety of 1.5 in design</td>
</tr>
<tr>
<td>7h</td>
<td>Stillwater - Nye Operation</td>
<td>Paste</td>
<td>10.0</td>
<td>2.0</td>
<td>6.1</td>
<td>2.7</td>
<td>Factor of safety of 1.5 in design</td>
</tr>
<tr>
<td>8</td>
<td>Mikel South</td>
<td>HF</td>
<td>7.0</td>
<td>5.5</td>
<td>4.6-6.1</td>
<td>4.6</td>
<td>Factor of safety of 1.5 in design</td>
</tr>
<tr>
<td>9</td>
<td>Gold Fields</td>
<td>HF</td>
<td>10.0</td>
<td>4.5</td>
<td>5.0</td>
<td>5.0</td>
<td>Factor of safety of 1.5 in design</td>
</tr>
<tr>
<td>10</td>
<td>Stratoni Mine</td>
<td>HF</td>
<td>12.8</td>
<td>2.0</td>
<td>6.0-9.0</td>
<td>6.0</td>
<td>78% Solids by weight</td>
</tr>
<tr>
<td>11</td>
<td>Galena- Coeur d'Alene</td>
<td>HF</td>
<td>10.0</td>
<td>2.5</td>
<td>3.0</td>
<td>2.1</td>
<td>73-75 % Solids by weight; 0.9 m air gap between lifts</td>
</tr>
<tr>
<td>12</td>
<td>Lucky Friday</td>
<td>Paste</td>
<td>8.0</td>
<td>4.8</td>
<td>2.4-4.6</td>
<td>2.4</td>
<td>Go under paste 3 days after pour; 0.6 m air gap between lifts</td>
</tr>
<tr>
<td>13</td>
<td>Kencana</td>
<td>Paste</td>
<td>12.0-24.0</td>
<td>1.2 - 1.5</td>
<td>6.0-8.0</td>
<td>5.0</td>
<td>Go under paste 7 to 28 days after pour; 1.2 MPa Paste in back, 0.5 Mpa in sidewalls</td>
</tr>
<tr>
<td>14</td>
<td>Lanfranchi- Helmut South</td>
<td>Paste</td>
<td>4.0-8.0</td>
<td>1.2 - 2.0</td>
<td>6.0-12.0</td>
<td>5.0</td>
<td>Span width is 6 m, intersection width is 12 m; 6 m cable bolts used in intersection; Go under paste 14 days after pour</td>
</tr>
<tr>
<td>15</td>
<td>Cortez Hills</td>
<td>CRF</td>
<td>7.8</td>
<td>6.0</td>
<td>6.0-11.0</td>
<td>4.6</td>
<td>Span width is 6 m, intersection width is 11 m</td>
</tr>
<tr>
<td>16</td>
<td>Andaychagua Mine</td>
<td>Concrete</td>
<td>14.0</td>
<td>16.0+</td>
<td>5.0-15.0</td>
<td>3.5</td>
<td>Cement aggregate fill; Span is 15 m; Aggreate 3/4&quot;</td>
</tr>
</tbody>
</table>

Table 2-1: Empirical Database for UCF mines (after Pakalnis, 2014)
2.4 Observational/Analytical Approach

Krauland and Stille (1993) studied the behaviour of sill beam design at the Garpenberg Mine in Sweden. Specifically, their research studied the in-situ behaviour of sill beams, and presents a numerical and analytical assessment of the CPB span stability.

Krauland and Stille’s analytical assessment of the sill beam used elastic-beam theory. The analytical model investigates two methods of beam failure: compressional convergence and flexural failure. The findings of the analysis are acceptable sill beam aspect ratios for stope convergences.

The important finding of Krauland and Stille (1993) analytical model is the consideration of different failure methods dependent on the mine’s stress environments. It was found that in low-stress environments, flexural failure is dominant; in high stress environments, convergence failures dominate.

The equations used for design are not presented in the analytical study, and no considerations for hybrid failures (rotational, sliding or relaxation) are part of the analysis. Although not stated in their work, beam theory is thought to of been used in determining the flexural failure.

Within Krauland and Stille (1993), Itasca’s FLAC 2D (Itasca, 2012) was used to perform the numerical analysis of sill beam stability. The constitutive model was a strain-softening relationship for CPB. The analysis applied step wise convergence until failure occurred. This type of numerical modeling reflects the nature of stresses within the stope. This approach is considered appropriate. Convergence and strain-softening will be used in the numerical studies within my research.

The strain-softening constitutive approach allows the material to have post-peak strength behaviours. This differs from a brittle behaviour response where no post-peak behaviour exists. Further, the strain-softening model reduces the post peak strength as opposed to the perfectly plastic models suggested by Mohr-Coulomb models.

Krauland and Stille (1993) determined that observed deflections in the field agree with the numerical approach. The analytical model better represented the measured stresses within the beam.

The design of sill beams at Garpenberg are based on the findings of this study with no known failures reported. Krauland and Stille (1993) find that modulus of elasticity is the most important critical parameter to the behaviour of the sill beam. This is a key finding as typical studies focused only on the strength, and not strain, characteristics of the backfill.

There exist numerous stability assessments for the sill beams. Most are built on the work of Mitchell (Stone, 1993; Newman et al., 2001; Caceres, 2005; Pakalnis et al., 2005) that are first principal equations based on the results of centrifuge modeling. Jordan et al. (2003) present sill beam stability with respect to traditional beam equations. Numerical approaches have been successfully used to design sill beams (Brechtel et al., 1999 and Krauland and Stille, 1993). Observational approaches have been presented by Krauland and Stille (1993) and Pakalnis et al. (2005). These observation approaches have merit in that they demonstrate the application of theory in mine operations.

With all these approaches, an analysis identifying the critical failure of sill beams over a range of model inputs needs performed. This analysis will be presented in the analytical section of this dissertation.

2.5 Seismic Loading of Paste Backfill

With UCF mining being a solution to mining in high stress and burst prone environments, the concern on the behaviour of CPB under seismic loading is valid. Cyclical loading has been performed on CPB (Nnandi and Mitchell, 1991) to determine the unload and reload response of fill. Further, studies on energy impacts on seismic events has been performed in numerous
The use of backfill in UCF mines with high seismicity is discussed by Williams et al. (2001) and Williams, Brady, Bayer, Bren, Pakalnis, Marjerison and Langston (2007). Dynamic behaviour on the long-term stability of CPB, to the author’s understanding has not been performed on the effect of seismicity on sill beam stability. Without field scale testing, it will be difficult to assess the effect of the material. What is proposed is that the resiliency of the material (Timoshenko and Gere, 1961) be assessed for the purpose of energy absorption. The resiliency of the backfill will then be compared to the energy of seismic events.

The seismic interaction of rock and fill is considered a research project in its own right. Determining the capability of fill to withstand seismic events will be considered within this research; however, the behaviour of the fill during seismic events and the dynamic response will not be considered. The purpose of this research is to validate the observations of mining operations in that the paste fill holds up better than the host rock in seismic events.

2.6 Performance of Paste Under Loading

To have proper numerical model results, the behaviour of paste is essential. The behaviour of CPB is one of the two aspects of the design guidelines investigated in this research. Works by Fall, Belem, Samb and Benzaazoua (2007), Rankine and Sivakugan (2007), Klein and Simon (2006), Tesarik et al. (2003), Pierce (1997), Aref, Hassani and Churcher (1989) and Amaratunga & Yashchysyn (1997), amongst others provide the stress-strain behaviour of laboratory samples of backfill under loading. Further works by le Roux, Bawden and Grabinsky (2005) provide the behaviour of these materials in-situ and from field specimens.

It is essential to know the behaviour of these materials should valid numerical models of paste sill beams be desired. Numerical modeling work on paste sill beams by Swan and Brummer (2001) assume that strain softening of backfill occurs within paste backfill and that the material loses its cohesive values and the frictional properties of the material drop off dramatically at 1.5% strain. Studies by those listed above show that this assumption is not valid.
especially for backfill with low binder content, and it is these assumptions that have limited the development of valid numerical models.

The theory of the backfill stresses and the design of sill beams have more to do with the behaviour of a sill beam as a single element and failing along the interfaces or breaking according to solid beam mechanics. What the presented models do not incorporate is the failure or yielding of the paste during a sill beam loading. Numerical studies by Pierce (1997); Brechtel et al. (1999); Caceres (2005); Dirige, DeSouza and Archibald (2007); Fahey, Helinski & Fourie (2009); Sainsbury & Urie (2007) and Hughes, Pakalnis, Caceres, Blake and Brady (2006) demonstrate that during the failure mechanism, the backfill undergoes significant amount of strain during failure. It is this stress-strain response that Pierce (1997); Aref et al. (1989); Amaratunga & Yashchysyn (1997); Fall et al. (2007); Rankine & Sivakugan (2007) have studied with the hopes of understanding the constitutive behaviours of CPB. In-situ studies of the behaviour of CPB under loading have been carried out by Gurtunca & Adams (1991); Ouellette & Servant (2000) and; le Roux et al. (2005) studied the in-situ behaviour of paste backfill while Piciachia, Scoble, & Robert (1987) studied the in-situ behaviour of hydraulic fill. Large-scale studies of backfilled stopes have been undertaken by Tesarik et al. (2003); Williams et al. (2007); and Donovan, Dawson, and Bawden, 2007 to determine the in-situ behaviour of the CPB with respect to changes in stress due to undercutting and mining activity.

The constitutive model, or mathematical representation of materials, is essential in simulating the behaviour of the failure mechanics of CPB in numerical models. Conclusive research needs to occur to determine if CPB behaves as an elastic material under Hooke’s law (Timoshenko and Gere, 1961), a non-linear elastic (Duncan and Chang, 1970), a linear-elastic – simple plastic model (Mohr-Coulomb, after Coulomb, 1776), Hoek-Brown (Hoek, Carranza-Torres and Corkum, 2002), or a non-linear plastic failure (Drucker and Prager, 1952). Other possible behaviours are of a hyperbolic-elastic model for CPB (Duncan and Chang, 1970 and Borgesson, 1981) or a strain-softening model (Roscoe, Schofield and Wroth (1958); and Swan and Brummer (2001)).

Presentations of results of Aref et al. (1989), Fall et al. (2007) demonstrate that CPB behaves linearly at low strain rates, commences an elasto-plastic yield at near peak stress,
followed by strain softening behaviour after peak stress. The constitutive relationship shown by these test results before failure is similar to that of the Duncan and Chang (1970) hypo-elastic code. After failure, it is true that a strain softening approach is present as suggested by Roscoe et al. (1958); Swan and Board (1989); and Swan and Brummer (2001). However, the approach to apply the constitutive behaviour of CPB under loading from laboratory testing to the behaviour of sill beams has not been performed.

This research project will model the constitutive behaviour of the CPB under loading, including strain softening behaviour, in the laboratory and apply the constitutive model to numerical models of sill beams during the undercutting process. This behaviour will allow the beam to withstand larger strain before failing, and provide a realistic post-failure behaviour as opposed to a perfectly plastic behaviour as suggested by a Mohr-Coulomb model.

The application of a non-linear hyperbolic elastic model has been applied to backfill by Borgensson (1981). The application of the hyperbolic model has the following limitations: dilation is not possible, confinement issue, poor in undrained conditions, and poorly models volumetric expansion (Duncan and Chang, 1970). Initial testing demonstrates that the hyperbolic model models the initial post-failure behaviour of backfill. Testing of the model with lab data will be performed to determine the applicability of the hyperbolic behaviour to CPB behaviour under load.

As Fall et al. (2007) note, the constituents of the CPB are paramount in determining the behaviour of stress-strain path of the material. As such, the discussion will be confined to general behaviour of the material and not absolute values. It is in the study by Fall et al. (2007) that outlines the stress-strain behaviour of the CPB, testing 300 lab cylinders of various constituents. Through Fall’s study, it was determined that CPB demonstrates a non-linear yielding behaviour. Fall found that an increase in the UCS correlates with an increase in the Young’s Modulus. Further, it was determined that confinement of the sample has a large effect on the stress-strain behaviour of the material. I argue that the confinement of the sample is more of a concern in long hole-delayed filling mining than in drift and fill UCF mining. In drift and fill mining, the material is not under the large confinement and self-weight as in other methods,
and its elastic properties will not be affected to a noticeable degree as suggested by Fall et al. (2007).

Pierce (1997) studied CPB at Golden Giant mine and by completing testing on samples, he was able to determine that the Mohr-Coulomb model reflected the stress-strain behaviour of the samples. This type of constitutive model is also used by modeling by Brechtel et al. (1999), Caceres (2005) and, le Roux et al. (2005) in determining the behaviour of backfill. I suggest that further verification of the use of the Mohr-Coulomb is necessary as with stronger samples there is evidence of a strain-softening type behavior as seen in tests by Pierce (1997) and Fall et al. (2007).

Krauland and Stille (1993) used a non-linear, strain softening approach to CPB backfill. The constitutive model was piece wise in its stress-stain response. This approach seems realistic in that it allows for post peak behaviour and can be easily programmed into numerical codes. However, the work is presented for one backfill mixture; this approach is limited as it does not allow material properties to be used in determining the stress-strain response. My research will attempt to use material properties to build the stress-strain response of backfill under load.

The study on understanding CPB behaviour under stress is essential to this research. By understanding the constitutive response a better understanding of the sill beam failures will be achieved. The stability of sill beams to this point treats the sill as a rigid body and ignores the strain. By understanding the stress-strain response of CPB, the expectation is that an understanding of the development and propagation of failure within the sill beam can be garnered.

From the literature it was found that numerous constitutive relationships exist for modeling CPB. Elastic, simple plastic and strain-softening constitutive relationships have been applied to CPB. No consensus on the correct approach was found. This research will attempt to build a constitutive code that uses material properties to define the stress-strain response. This will be investigated through laboratory study, followed by constructing constitutive models to fit the stress-strain codes. Once completed, it is anticipated that the constitutive code can reflect the behaviour of CPB sill beams.
2.7 Effect of Ground Support on Cemented Paste Backfill Sills

To date there exists little research/analysis on the effect of ground support on the performance of CPB sill mats. Work by Williams et al. (2007) and Donovan, Dawson and Bawden (2007) provide initial research on determining the effectiveness of ground support within the sill mat. Ground support and its purpose are founded in the book “Underground Excavations in Rock” by Hoek and Brown (1980). Case studies of the effect of bolts in standard mining operations have been discussed by Hyett (2006), Hyett, Moosavi and Bawden (1996) and Hutchinson & Diedrichs (1996) amongst others. With respect to ground support within CPB, Williams et al. (2001) argues that the bolts within the fill are loaded to near capacity as subsequent undercuts are mined. The bolts are tensioned and thus active and provide a ‘cone of influence’ within the backfill. These theories are similar to the Voussoir’s arch theory presented by Sofianos’ analytical model.

The ideas proposed by Williams et al. (2001) should be tested against the Sofianos (2006) and how they relate to Voussoir’s arch theory. This discussion will be performed as part of the study on ground support within the sill beam.

The effect of confinement of the bolts in the CPB sill needs to be verified. Williams et al. (2007) propose that the bolts play a large role in confinement of the sill beam, whereas Pells (2008) argues that non-tensioned tendon support offers little to no confinement, rather the tendons transmit tensile strength within the rock mass. The effect of bolts within sill mat will be assessed. It is initially thought by the researcher that the ground support acts only in unifying the beam and containing small skin failures; support does increase the structural stability of the sill beam outside of ensuring the integrity of the beam. This hypothesis will be tested in the research.

2.8 Operations Mining Under Cemented Paste Backfill

Underhand cut and fill employing cemented paste backfill is becoming more prevalent as difficult ore bodies are mined. The most common use of CPB is to backfill large stopes underground (DeSouza, Archibald and De Souza, 2003). In these large stopes, it is typical that the CPB will not form a sill beam for miner entry, rather narrow drill drives are developed within
paste or remote operated equipment is employed. The research that involves large scale CPB mines without undercut paste provides an operational overview on the use of backfill. The literature study focused, where possible, on mines performing UCF with CPB for laterally developed stopes.

Foster, Jessop and Andrews (2011) discuss mining under paste backfill at Lafranchi mine in Australia. The paper provides a good summary of the learning curve at Lafranchi utilizing paste backfill. Lafranchi mine has 5 m wide drifts supporting CPB paste backfill with galvanized 2.4 m split sets that either pushed from the collar or pushed in the final 1.0 m of bolt depth to create a quasi-grout filled bolt; welded wire mesh is installed. Foster and Jessop report that 7 to 8 tonnes of pullout strength is achieved with these bolts. Cable bolts are used in intersections or when drift spans are greater than 6 m. No stope preparation is performed apart from a full clean out of the stope prior to placement of CPB. The paste backfill at Lafranchi has a design strength of 500 kPa; 500 kPa is a low strength value from the researcher’s experience. One item of interest at Lafranchi is how the CPB information is centralized into a database that can be accessed by the paste plant, scheduler and mine personnel. A discussion and the benefits on the centralized database for paste backfill that is integrated into all levels of the operation are provided. The unique aspects of Lafranchi are large span widths that are achieved with a low strength CPB and cable bolt ground support.

Febrian et al. (2007) discuss mining under CPB at Newcrest’s Kencana mine. The geotechnical setting at Kencana is much different than typical high stress environments: in comparison to the weak rock mass and closing ground, the paste is a strong element in the design. For mining under CPB, the original and published strength is 2.5 MPa at Kencana. However, with optimization of the CPB at Kencana the current UCS of the CPB is between 1.2 and 1.5 MPa with between 10 and 16% binder. The ground support of the fill is not discussed in detail, but it would appear from the figures shown, that no ground support is installed in the backfill upon exposure; however, sill preparation is performed that includes shear support in the wall and stand-up rebar prior to placement of the fill. The paper also shows useful statistical relationships of the backfill testing.
Jordan et al. (2003) provides a summary of the mining under CPB at Stillwater Mine. The paper explains the design philosophy and details the requirements for strength of backfill necessary for stope widths. At Stillwater a 1.0 MPa, 12% binder (Portland cement) is used. Ground support discussed in the paper is no longer utilized at the site. Detailed discussion on Stillwater is presented in the analysis section of this research.

Lucky Friday is the first mine to successfully employ underhand cut and fill utilizing paste backfill in a narrow vein orebody. Discussions on the mining at Lucky Friday are found in Williams et al. (2001 & 2007). These reports detail the design, reasoning and implementation of CPB at Lucky Friday. Lucky Friday uses an extensive sill mat prep that involves stand up rebar and the placement of mesh prior to the placement of CPB. Once fill is placed the use of spot bolting with split sets is carried out. The CPB at Lucky Friday is a 2 MPa design at 8-10% binder content. Further discussion on Lucky Friday mining process is provided in Chapter 5.

Similar to the instrumented study at Lucky Friday, Donovan et al. (2007) discuss test stopes at the David Bell Mine. This looks at testing the stope preparations for CPB sills. Three scenarios were tested: stand-up paddles; chain-link mesh; and geofabrics. The CPB was a 2MPa design with 6-8% binder. Ground support on advance was not performed in the case of the stand-up bolts; in the case of the chain-link and geofabrics, Swellex bolts 1.8 m long on a 1.2 m x 1.2 m pattern was utilized. At time of the article, the CPB sills were showing no signs of failure. This paper also provides the results of pull tests on Swellex in CPB; testing indicates bond strengths of 1.9 tonnes/foot were achieved. Further publications were to be provided to discuss on going performance of the sill tests; no work was found on the matter.

Rankine and Sivakugan (2007) outline the geotechnical properties of the backfill at Cannington by process of extensive laboratory testing. The paper underlines that the cement content and percent solids are the main factors that dictate the strength of the backfill. Further, drained and undrained properties for Cannington backfill are presented. These properties are useful for determining the behaviour during the placement process and the once the CPB has hydrated.
2.9 Quality Control of Cemented Paste Backfill

With the placement of an engineered material, the assurance the material is being placed as designed is essential. In all engineering design, the goal of backfill is that the material is placed such that its properties are homogenous; quality control ensures that the materials are being placed as designed. Typically testing for the backfill is carried out on surface (Yu, 1995; Stone, 1993; De Souza; Archibald, Dirige, 2003; and Farsangi, Hayward and Hassani, 1996), with special projects undertaken to test the backfill in-situ (Ouellette and Servant, 2000).

The issue with the testing of CPB is that no standard exists, as such the testing of the material is a hybrid of UCS for Concrete- ASTM Standard C39 (ASTM, 2010a); two Rock UCS standards -ASTM Standard D7012 (ASTM, 2010b) and ISRM- 1979 (Ulusay and Hudson, 2007). The common thread is that the axial load is applied to the sample until the peak load is reached. Details on the testing of the backfill at various mine sites are discussed by Hughes, Pakalnis, Deen and Ferster (2013); Rai, Sandbak and Kallu (2013); and Jordan et al. (2003).

Farsangi et al. (1996) describes the differential strength of CRF when backfilled in a stope; this is of great concern as the segregation of binder can severely impact the strength of the undercut beam. This issue is more pertinent to that of CRF in that the fines contain the binder, however it illustrates the point quality control issues are inherent to backfill operations.

The question from operations remains “is the lab strength reflective of the in-situ strengths?” Study of literature found that this question is not fully answered.

The comparison of in-situ values to lab samples is discussed by Yilmaz, Benzaazoua, Belem and Bussiere (2009). The paper finds that when a sample is tested without an applied vertical pressure during hydration the sample will have a lower strength than those prepared with an applied load similar to that experienced within the stope. This effect was most noticeable on samples that had low cement content. It must be stressed these are not field samples; rather the tests are for samples cured in the laboratory under conditions that reflect the field. The testing apparatus and procedures are difficult for most operations to set-up. This was verified during my research, labs at mines are not to set to academic research standards. The practicality of an
operation performing these tests regularly is not feasible. The need for a simple in-situ testing methods is required.

The shotcrete industry has a novel technique that utilizes both a digital pocket-penetrometer and a Hilti 450 pneumatic gun to test material strengths between 0 and 1.5 MPa, and 1.5 to 8 MPa respectively (Bracher, 2005). These ranges of values are within the desired range of strengths for paste. As this method of testing is widespread in the Shotcrete industry (Bracher, 2005) and no studies have been disseminated in CPB industry it is suggested that this in-situ testing method be incorporated to the proposed research. From this, a database can be established for in-situ vs. laboratory strengths.

Yu (1995) and Stone (1993) point out that the in-situ values of CRF should be less than the QA lab test values due to the effect of grain-size distribution; however, le Roux et al. (2005) and Pierce (1997) suggest the strengths of the backfill is greater in-situ than in the laboratory values. Jordan et al. (2003) provide a solution to this problem as they cured samples underground and test in the laboratory, thereby accounting for differences in strength due to curing conditions. The method described by Jordan et al. (2003) will be investigated as part of this research.

With the differences in strength noted between lab and field values, studies should be focused on the comparison between in-situ strength and laboratory. Yu (1995) and Stone (1993) proposed separate empirical formulae for this question. Yu (1995) goes further in suggesting various types of tests can be carried out for in-situ testing, however the methods Yu suggests would slow down production or be carried out in old mining areas. Discussion with operations found that little buy-in to these methods existed.

The comparison between the in-situ strength and that of the lab test value needs to be determined. Former UBC/Red Lake student researcher Kathryn Clapp performed in-situ testing using the Windsor pin method and Schmidt Hammer test at Red Lake with little success. It was noted that scatter was common and no correlation with test values were achieved. The shotcrete industry (Bracher, 2005) performs in-situ early strength testing of backfill using a Hilti gun test. The test performed by measuring the depth of embedment of a pneumatically charged stud projected into shotcrete. The depth of embedment is then correlated to a UCS value. Hughes et
al. (2013) has found that this method is not conducive to testing in-situ values of CPB as there was too large a variance in test results over small distances. The strength of cement core lab sample, as presented by Bartlett and Macgregor (1994), is correlated to the uniform moisture content in the sample. As the moisture content goes up, strength goes down in tested concrete cylinders. Applying this to CPB, the strength of the placed CPB could be achieved. Hughes et al. (2013) explored the correlation of CPB UCS with moisture content at Stillwater Mine. The research by Hughes et al. (2013) will be presented as part of the research.

In keeping with the concept of database testing, studies by Shrestha (2008), Williams et al. (2007), Pakalnis et al. (2005), De Souza et al. (2003) and Annor, Tarr, and Fynn (2003) that provides indexed values of backfill strength (predominantly on CRF) related to either span or binder content. This type of database is invaluable to the design of paste backfill mining as it provides a basis for costing and allows for empirical design concepts. It would be time well spent to amalgamate these databases and form a general relationship for both strength vs. binder content and strength vs. sill beam span. With access to databases from Red Lake, Stillwater and Macassa it is hoped that these relationship can be established. This work will be performed in Chapter 4 and 5.

The purpose of quality control and assurance program is to ensure that the design strengths are being achieved for the CPB in-situ. From the literature, there is no simple method to determine the in-situ strength of CPB. The investigation into simple QA/QC methods will be studied in attempt to fill this shortcoming. Pneumatic penetration tests (Bracher, 2005); moisture content vs. UCS (Bartlett and Macgregor, 1994); Schmidt Hammer and Windsor pin method will be studied to determine if they can provide a simple in-situ relationship.

### 2.10 Instrumentation and Performance of Cemented Paste Backfill Sills

The instrumentation of backfill is discussed in the literature by Williams et al. (2001); Donovan et al. (2007); DeGagne, DeSouza and Nantel (2001); Tesarik et al. (2003); Helinski et al. (2010a); Veenstra et al. (2011) and Thompson et al. (2012). Apart from the Donovan et al. (2007) and Williams et al. (2001), the others discuss the monitoring of paste in large stopes or in
the early strength of the material (placement and hydration of CPB). The concepts discussed in these papers are relevant as they can be applied to UCF mining; however, the studies are for long-hole CPB operations, the work is not directly transferrable to CPB UCF

The study of in-situ testing backfill has been carried out by means of self-boring pressure meters (le Roux et al., 2005; Ouellette, Bidwell and Servant, 1998), vibrating wire instrumentation (Tesarik et al., 2003; Williams et al., 2001) and cone-penetrometer-testing (CPT) as found by Aref et al. (1989). Le Roux determined that the in-situ strength of the material is higher than the laboratory strength of the material, and sill beam design based on laboratory strength results in a conservative design. This is in contrast with theories, although based on CRF, provided by Stone (1993) and Yu (1995) that suggest that laboratory strengths are upwards of 150% stronger than the in-situ strengths. Further findings of le Roux et al. (2005) showed that the deformations of stopes do occur prior to the movement of the stope, as suggested by Pakalnis and Associates (2012), and that the material is undergoing loading due to self-weight. The study by le Roux et al. (2005) has provided some strong findings of the in-situ behaviour, however as they noted, the testing apparatus is difficult to use in an operating mine and the tests were done in a dormant part of the mine.

Other papers listed above discuss the instrumentation program and lessons learned during the study. Of note, the DeGagne, DeSouza and Nantel (2001) paper discusses the types of instruments that can be used to measure different characteristics of the paste backfill.

In-situ behaviour was carried out by Tesarik et al. (2003) to determine the modulus of backfill at three mines across the USA. This method involved the placement of instruments prior to the placement of CRF backfill and was able to determine the in-situ modulus of the backfill based on the measured stresses and strain within the backfill. The study by Williams et al. (2007) was able instrument the CPB at Luck Friday, Idaho. Unfortunately, the findings of Tesarik et al. (2003) were not replicated by Williams et al. (2007) and the instrumentation program was not adequately designed to gain the proper information. A positive outcome from this study is that long-term instrumentation is possible with CPB mining environments (validated by Donovan et al., 2007).
The challenge with in-situ stress and in-situ modulus measurements is that they only provide the current stress behaviour of the material and do not represent the entire stress-strain path or ultimate values. The in-situ testing must be coupled with a laboratory study in order to have both the ultimate, and full-stress regime with the in-situ performance of the material.

Aref et al. (1989) provided an evaluation of cemented paste backfill with both laboratory studies and in-situ testing with piezometer-cone-penetration-tests (CPT). The purpose of the study was to measure the shear strength characteristics of the material as well as the susceptibility of CPB to liquefaction. The results of these tests are similar to those by Pierce (1997) and Fall et al. (2007): the laboratory samples had dilative behaviour; the stiffness of the material increases with higher cement content, and at higher confinement, the friction angle instead of cohesion dominates. It should be noted that these test were undertaken at high water content and low cement content. What was unique about this study is that the seismic response of the material was measured to determine the in-situ liquefaction potential of the material. It was determined that due to the dilative behaviour and high cohesion of the material liquefaction was not possible. This statement by Aref et al. (1989) is referenced when the liquefaction potential of CPB is discussed.

The instrumentation and monitoring of CPB coupled with the QA/QC verifies the design properties and parameters. The instrumentation programs do illustrate the two fields of research present with UCF CPB work: the behaviour of the fill and the stability of the sill beam.

2.11 Chapter Summary

The literature study was a rewarding endeavor. For this research, two streams of study were apparent. One stream was studying the stability of sill beams; the other the behaviour of CPB. In an effort to construct design guidelines these two research paths need to be linked together with in-situ testing methods instrumentation to ensure a complete design. The literature study provided insight to the original research questions. From the literature the following questions still need addressed:

- What are the critical failure methods of a sill beam?
- Is the incorporation of the mine environment (closure stresses) critical to sill beam stability?
• What are the governing equations for sill beam stability?
• What role does ground support play in sill beam stability?
• What is the minimum strength of CPB for an arbitrary width of sill beam?
• What is the correct constitutive code for CPB?
• Is there a simple method in determining the in-situ strength of CPB?
3 METHODOLOGY

In order to have complete design of the sill beam, two distinct items need studied: the mechanical behaviour of cemented paste backfill and the stability of the sill beam. These two paths will be linked in the end, but initially CPB behaviour and sill beam stability will be studied independently. These two streams are necessary as they both relate directly to sill beam stability. One addresses the behaviour of the material during loading; the other addresses the conditions that lead to failure.

This research does not look at one specific element of design; rather it studies all elements of sill beam design methodology through analytical, observational, empirical and numerical means. This research follows a mixed method methodology format, in that numerous studies will be undertaken to prove the hypothesis and research questions. Figure 3-1 provides a flow chart of the research methodology indicating the stages to reaching the research objectives. Table 3-1 summarizes the research methodologies and their proposed outcomes. Detailed discussion of the methodologies is provided within this chapter.

The field research for this dissertation occurred between June, 2009 and June, 2013. During this time, laboratory studies, mine site visits, analytical, numerical and empirical studies occurred. Laboratory, analytical, numerical and empirical analysis occurred at the University of British Columbia campus. Mine site visits occurred at Macassa Mine, Kirkland Lake, Ontario; Red Lake Mine, Red Lake, Ontario; Goldhunter (Lucky Friday), Kellogg, Idaho; and Stillwater Mine, Nye, Montana. In depth study on the design methodologies of Kencana Mine, Gooswong, Indonesia was also performed, but no site visit occurred.
3.1 Research Scope and Limitations

This research looks to identify, qualify and quantify key design elements of CPB sills as shown in Figure 2-1. The scope of this thesis consisted of the following research methods:
• on-site inspection of operating underhand cut and fill CPB mines,
• researching the current-state-of-knowledge on CPB and underhand cut and fill mines,
• performing laboratory tests on CPB samples to determine material properties,
• performing analytical and numerical analysis, and
• testing the research findings against empirical evidence.

The operating underhand cut and fill mines selected for this study offer a good representation of high stress, weak rock and challenging ground conditions that are conducive to underhand cut-and-fill mining. By studying the selected mines, the ground conditions that are suitable for UCF mining are properly represented.

Further limitations with laboratory equipment did not allow for primary study of the tri-axial behaviour of CPB; due to mine safety and production requirements, planned field observation was not completed at sponsoring mines; the 3D behaviour of the material was not investigated. As discussed later, plane-strain provides a good model of sill beam behaviour.

The selection of numerical code for analysis was limited due to research funds and software available to the University. Initially the following software was investigated for the study: Plaxis 2D, Phase2, FLAC 2D, FLAC 3D, UDEC 2D, UDEC 3D, MIDAS GTS, and MAP 3D. In the end, due to financial limitations, the investigation was between available FEM, FDM and BEM models; as such, MIDAS GTS and Plaxis 2D were not considered.

For the observational studies at the mine site, certain mines led themselves to different research approaches. Due to the different approaches, the scope was not uniform across operations. Table 3-2 summarizes the work performed at the individual mines.
<table>
<thead>
<tr>
<th>Design Element</th>
<th>Research Tools</th>
<th>Outcome</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Loading</td>
<td>• Numerical Analysis of loading effect&lt;br&gt;• Analytical analysis of loading effects&lt;br&gt;• Parametric study on effect of cohesion and friction angle on vertical loads</td>
<td>• Understanding effect of vertical surcharge on CPB beam stability</td>
</tr>
<tr>
<td>Lateral Stresses and Closure</td>
<td>• Literature study of arching theory and lateral loads in fill&lt;br&gt;• Back analysis of reported closure readings&lt;br&gt;• Numerical modeling of stress transfers between fill/rock&lt;br&gt;• Analysis of lateral loading and co-efficient of lateral earth pressure</td>
<td>• Determination of lateral loading factors&lt;br&gt;• Understanding of effect of lateral closure on failure</td>
</tr>
<tr>
<td>Failure Modes</td>
<td>• Review of literature on known failures&lt;br&gt;• Analytical assessment of failure&lt;br&gt;• Numerical Analysis of failure&lt;br&gt;• Observations of failures at mines</td>
<td>• Understanding of Critical failure modes&lt;br&gt;• Ability to back analyze failures&lt;br&gt;• Anticipated analytical model of failure</td>
</tr>
<tr>
<td>Performance of CPB Under Loading</td>
<td>• Laboratory Tests of paste under constant load&lt;br&gt;• Laboratory Tests of paste under cyclical load&lt;br&gt;• Laboratory Tests of paste under constant strain&lt;br&gt;• Review of available literature</td>
<td>• Determine resiliency of CPB&lt;br&gt;• Create a constitutive model for CPB under uniaxial loading</td>
</tr>
<tr>
<td>Support Implications</td>
<td>• Review of available literature&lt;br&gt;• Observation of mine site ground support&lt;br&gt;• Analysis of ground support and sill beam interaction</td>
<td>• Understanding of interaction of ground support with CPB beam</td>
</tr>
<tr>
<td>Cold Joints/Air Gaps</td>
<td>• Review of available literature&lt;br&gt;• Analysis of critical depth of cold joint/support layer interface</td>
<td>• Understanding on behaviour of cold joints and effect on stability</td>
</tr>
<tr>
<td>Seismic Effects</td>
<td>• Review of available literature&lt;br&gt;• Lab testing of resiliency of CPB&lt;br&gt;• Analytical analysis of energy impact of seismic events&lt;br&gt;• Laboratory Testing based on size of sample</td>
<td>• Understanding of paste backfill under seismic loads</td>
</tr>
<tr>
<td>QA/QC</td>
<td>• Statistical analysis of lab testing values&lt;br&gt;• Laboratory testing to investigate index relationships&lt;br&gt;• In-situ testing of backfill to determine in-place strength</td>
<td>• Index relationships of CPB to be used in planning and quality control&lt;br&gt;• Creation of standard for CPB testing at mine sites</td>
</tr>
</tbody>
</table>
Table 3-2: Design element researched from observational method

<table>
<thead>
<tr>
<th>Mine</th>
<th>Research Performed</th>
<th>Design Element Researched</th>
</tr>
</thead>
</table>
| Red Lake   | - Detailed review of design guidelines  
              - Study of support performance in pull-out tests  
              - Assessment of stability of sill beams  
              - Study of cold-joint behaviour  
              - Study of historic failures  
              - Underground tour to determine operational issues  
              - Site tour of paste plant           | - Seismic loading strength  
              - Failure modes  
              - Performance under loading  
              - Ground control requirements       |
| Stillwater | - Underground tour to determine operational issues  
              - Review of design methodology  
              - In-situ testing  
              - Statistical analysis of testing database  
              - Underground tour to determine operational issues  
              - Assessment of stability of sill beams  
              - Study of historic failures  
              - Laboratory Testing of CPB          | - Failure modes  
              - Performance under loading  
              - Ground control requirements  
              - QA/QC Guidelines                  |
| Macassa    | - Underground tour to determine operational issues  
              - Statistical analysis of testing database  
              - Site tour of paste plant  
              - Laboratory Testing of CPB  
              - In-situ testing  
              - Analysis of ground control elements | - QA/QC Guidelines  
              - Performance under loading  
              - Ground support elements           |
| Lucky Friday | - Review of design methodology  
              - Review of instrumentation  
              - Underground tour to determine operational issues  
              - Review of ground control           | - Closure/horizontal loading  
              - Ground support elements  
              - Failure modes                     |
| Kencana    | - Review of design methodology  
              - Back analysis of failure           | - Failure modes  
              - Ground support elements  
              - Loading (Vertical/Horizontal)      |
3.2 Research Methods

This research will be a combination of field observations, laboratory testing, empirical study, analytical assessment and numerical modeling that will assist in the engineering design of CPB sills. The purpose of the field monitoring and laboratory testing is to determine the behaviour of the material under loading. The numerical modeling enabled a parametric study utilizing the results of the laboratory and field studies.

3.3 Research Tools

The following section describes the laboratory, field, observational, analytical and numerical tools that will be used to answer the research questions.

3.3.1 Laboratory Testing

Laboratory testing was performed to determine strength and index properties of backfill. Select batch tests were performed at Norman B. Keevil Rock Mechanics Laboratory under the control of the researcher. Outside testing was performed by mine sites and by third-party laboratories organized by the mine sites. SK Geotechnical performed the external lab testing for Stillwater; Shaba Testing for Macassa. Both of these labs were visited as part of this research.

Laboratory tests performed for determining material properties as part of this research are as follows: Unconfined Compressive Strength Test (UCS) under constant force; UCS with post-peak behaviour under constant strain; cyclical loading of cylindrical samples with pre-programmed displacement; Brazilian tensile testing; moisture content; and specific gravity. Secondary testing to classify the material, (not integral to this research) included the following: particle size gradation; mineral x-ray diffraction (XRD); and slump tests. The secondary tests were performed to differentiate between the compositions of the CPB from the different mine sites.
3.3.1.1 Test Laboratory

Primary research of CPB strength and index properties occurred at the Norman B. Keevil Rock Mechanics Laboratory (NBK rocklab). Equipment used in the laboratory included the servo-controlled MTS Rock Testing System Model 815 II apparatus utilizing Model 7933.00 System Software with Basic TestWare®; Sartorius Universal Scale Model U4600p + **V40; and Lab Line Instruments L-C Oven.

The laboratory tests performed for this study at the NBK Rock Mechanics are as follows:

- UCS under constant load,
- UCS under constant strain,
- Brazilian Tensile testing,
- Bulk Density Measurements, and
- Elastic Modulus Readings.

3.3.1.2 Description of Laboratory Tests

The purpose of the testing is described within this chapter in detail. However, in summary, there are two main reasons for the testing:

- Determine the appropriate parameters and properties for analysis;
- Verify published laboratory data.

3.3.1.2.1 Unconfined Compressive Strength test under constant rate of force

The UCS testing under constant rate of force consisted of primary research performed by the researcher at the University of British Columbia Norman B. Keevil Rock Mechanics Laboratory (UBC NBK). In addition, secondary UCS testing was performed by Shaba Testing Limited and SK Geotechnical for QA/QC at the Macassa and Stillwater Mine respectively. The UCS testing database for the in-house QA/QC performed by Red Lake was also provided. The external testing results were obtained for statistical analysis. The primary testing was to determine the behaviour of CBP.

The testing of CPB has yet to be standardized; as such, the methodology for the unconfined compressive testing is not clearly defined. It was observed at the visited mines
that every mine has its own ‘standard.’ Primary UCS testing was performed using ASTM D7012-10 (ASTM, 2010b); ISRM-79 (Ulusay and Hudson, 2007); and ASTM C39-10 (ASTM, 2010a) as guidelines.

The issue with the ASTM C39-10 is that no provisions are made to measure elastic modulus properties during loading. ASTM C39-10 is used for simple QA/QC guidelines (ASTM, 2010a). Whereas, ASTM D7012-10 (ASTM, 2010b) and ISRM-79 (Ulusay and Hudson, 2007) provide guidelines on measuring elastic modulus readings during testing of samples under constant loading rate. Where possible, samples tested at the UBC NBK were tested with recordings of elastic modulus properties.

Tests were performed per ASTM C39-10 standards at the two external testing facilities. These two labs performed the tests with amendments made based on the nature of CPB and the preparation of cylinders at the mine site. Further, due to time constraints, the recommended 7, 14 and 28 day strengths tests were not strictly followed. Other noted alterations at these facilities included adjusting the loading rate for weaker samples, testing samples below the required height to length ratio, and, in the case of Macassa UCS samples, testing of samples that were not within planar tolerances.

Primary UCS testing was performed on samples from Red Lake Mine, Stillwater Mine and Macassa Mine. Two rounds of UCS under constant load were completed on Red Lake Samples; one round of testing was completed for the Stillwater Mine at UBC NBK, with the database of Stillwater QA/QC testing being provided; and testing at UBC NBK was completed on Macassa CPB with access to the testing database provided by the mine site.

3.3.1.2.2 Unconfined Compressive Strength test under constant rate of displacement

The UCS test performed under constant rate of displacement was performed to determine the post-peak characteristics of the CPB. Displacement rates of 0.5 to 1 mm per minute were used to determine the post-peak characteristics of the backfill.

The loading rate was selected based on the ASTM standard D-7012 (ASTM, 2010b) specification that the test should take approximately 10 minutes; trial and error found that for
CPB tests a displacement rate 0.5 mm allowed the samples to be tested within the standardized time.

CPB per Fairhurst and Hudson (1999) is classified as a Type I ‘material’: it will behave in a ductile manner. The purpose of the post-peak testing is to determine the stress-strain response of the material once the peak is reached. It is hypothesized that the strength of the material past peak load will reduce gradually as shown by Fall et al. (2007) and described by Rankine and Sivakugan (2007). Further, the behaviour of the material as a Mohr-Coulomb material, per Pierce (2001) or the behaviour of the CPB as a strain-softening material per Andrieux et al. (2003) will be analyzed.

3.3.1.2.3 Unconfined Compressive Strength test under cyclical loading

In order to determine the load-unload characteristics of CPB, hysteresis loop loading was applied to the select samples. As per the tests under constant strain, the MTS machine was controlled with a pre-programmed displacement sequence. The displacement vs. time of the hysteresis loading is shown in Figure 3-2.
The purpose of the tests is to determine the modulus of permanent deformation (M) as described in Goodman (1980) and to determine a numerical method in determining the secant modulus as discussed by Fairhurst and Hudson (1999). In addition to determining the load and reloading characteristics of CPB, these tests will be used in conjunction with the constant strain tests to determine the post-peak behaviour and characteristics of CPB.

### 3.3.1.2.4 Brazilian Tensile Testing

The Brazilian Tensile test is an indirect testing method to determine the tensile strength of an object. A compressive load is placed diametrically across cylinders with thickness between 0.2 and 0.7 times the diameter. This applied cylindrical load induces tensile stresses perpendicular to the applied load leading to tensile failure. Standards for the Brazilian Tensile testing can be found in ASTM D3967 (ASTM, 2008a). Discussion on the failure mechanics of Brazilian tensile test is provided by Munz and Fett (2001).
3.3.2 In-situ Strength and Relationships

With CPB, it is typical of operations that the majority of the testing is performed at the lab, with only verification of underground testing done on occasion. Red Lake Mine has drilled in-place CPB and performed UCS testing on the material. Stillwater, as discussed in Jordan et al. (2003) and Hughes et al. (2013) perform underground sampling of CPB; however, these samples are brought to surface after the end of shift.

The absence of in place strength is of great concern to operations. The competency of the sill is based on design strength of the in-situ beam: a direct measurement of the strength of the beam is needed.

As part of this study, the researcher performed primary in-situ tests at mine sites to obtain an indexed strength value for the CPB. Secondary research was performed by others in determining in-situ values and index testing. The type of in-situ testing and the methodology considered for this research are presented below.

3.3.2.1 Schmidt Hammer

The Schmidt Hammer, or Swiss Hammer, is a non-destructive testing device that correlates the strength of material based on the measured rebound strength a calibrated spring mounted device when pushed against said material. The details and specifics of the test are described in ASTM C805 (ASTM, 2013a) and ASTM D5873 (ASTM, 2013b).

The Schmidt Hammer testing was not pursued for testing due to two discrepancies with the standards: 1) Material must be between 1 and 100 MPa; 2) CPB is comparable to that of a soft rock: penetration into the surface of the sample will tend to give erroneous readings.

It was determined at the early stages of the research that due to the low-strength and ductile nature of CPB, CPB does not conform to the standards of ASTM D5873 (ASTM, 2013b).
3.3.2.2 Pocket Penetrometer

The researcher and mine operations had a preferred in-situ test apparatus: the pocket penetrometer. It is a cheap tool, easy to perform and it is a standardized test.

Pocket Penetrometer tests were performed to establish the early strength of backfill per standards ASTM D1558 (ASTM, 2010d). The pocket penetrometer is a calibrated spring loaded instrument that correlates the spring’s resistance to penetration to known strengths. Typical use of penetrometers is for non-cohesive soils that have a bearing capacity less than 1.5 MPa. It is thought that given the design strengths of paste backfill less that 2 MPa, this will provide a guide to the early strength of the backfill only.

The pocket penetrometer trials were performed at Red Lake Gold Mine to investigate the in-situ strength of the material.

3.3.2.3 Cored UCS strength tests

In-place CPB was cored as part of this research in an effort to determine the in-situ strength. The coring was performed by two methods: directly at the mine with diamond drill exploration rigs and at the NBK rock lab. The purpose of the coring is to get a direct measurement of the CPB in-situ. These will reflect the specific mine conditions in which the CPB is cured. Knowing the in-situ strength value will allow proper design values to be used in analytical design methods like Mitchell (1989a) and Pakalnis et al. (2005). This finding could ultimately lead to a lower initial strength design that could provide mine sites with cost-savings.

3.3.2.4 Hilti Penetration gun method

Tests were performed at Stillwater to determine in-situ strengths of backfill. The in-situ Penetration Depth Test (IPDT) performed is commonly used in shotcrete. The test is summarized by Bracher (2005) and discussed by Hughes et al. (2013) for the purposes of CPB in-situ testing. The IPDT test involves the firing of a pneumatically charged nail into a cohesive medium; the penetration depth of the nail is compared to an empirical relationship of
the penetration depth vs. UCS. The testing procedures are comparable to ASTM C803 (ASTM, 2010c).

This research will attempt to determine if a relationship between IPDT and CPB UCS exists. To determine if a relationship exists backfill samples from Macassa Mine of various constituents and cement contents were poured in ore sample bins (60 cm deep, 60 cm wide, and 90 cm long) and were cured for 56 days, to ensure that the CPB had properly hydrated. Upon curing, 63.5 mm long concrete nails were driven into the backfill with a Hilti DX36 semi-automatic powder-actuated tool using white cartridges (extra-light). Once the depths of embedment are measured, cored samples will be obtained from surrounding backfill and subsequent UCS tests will be performed. The depth of embedment of the nail vs. UCS will be compared to the results of Bracher (2005) to see if the backfill behaves in similar fashion to the shotcrete and if a similar relationship for backfill exists.

After testing at UBC NBK, the relationship will be field tested at Stillwater Mine, to determine the in-situ strength of backfill at Stillwater based on the determined relationship. Hilti penetration depths will be performed in accessible stopes and the values will be correlated to the determined relationship and then compared to test values from the Stillwater database.

3.3.2.5 Windsor Pin method

The Windsor Pin Penetration test is an index test that correlates the penetration depth of a pin to the UCS strength of a material: the strength of the material is inversely related to the penetration depth of the pin. The penetration test is described in ASTM C803 (ASTM, 2010c). The Windsor Pin Method is designed for in-situ determination of in place structural concrete with nominal strength of 34 MPa (Kosmatka, Kerkhoff, and Panarese, 2002). Due to the high strength values of the material the test is designed for, the Windsor Pin Method might not be suitable.

A strength relationship will be experimentally established between the strength of CPB and the penetration depth of the pin. To calibrate the Windsor Pin instrument for the site, cylindrical paste backfill samples from the plant are to be prepared and tested for their UCS at
the NBK rock lab. Windsor Pin tests will be performed on each sample cylinder. The average penetration depth will be correlated to the UCS. A regression analysis will be performed to determine the relationship between the UCS of the CPB and Windsor Pin Penetration depth. Once the relationship is established, Windsor Pin tests will be performed to determine the correlated UCS value of in-situ CPB strength.

The source material for the calibration of the Windsor Pin Method was boulder sized material from Red Lake Mine sent to UBC for testing. Samples were cored, where possible and cut and polished per ASTM C39 Standard (ASTM, 2010a).

In total eight (8) CPB test cylinders were suitable for UCS testing from the shipped boulders. Windsor Pin Tests were performed on the CPB boulders outside of the coring area prior to drilling of the samples. The number of Windsor Pin Tests varied per sample; as many samples as practically possible were performed on each sample in an effort to increase the precision of the calibration.

3.3.2.6 Point Load test

The Point Load Test is an index test that compares an indirect tensile value of core or ‘lump’ sample to the UCS value through a correlation value. The Point Load Test procedures are described in ISRM-89 (Ulusay and Hudson, 2007) and ASTM D5731 (ASTM, 2008b). For CPB core cylinders are difficult to obtain through coring methods. Typical point load test apparatuses cannot accommodate core larger than 75 mm in diameter; the standard CPB sample is 100 mm diameter. Point load tests require hard and brittle samples; CPB is characteristically ductile and severe indentation occurs when pressed. From observation, the failure of CPB during point load testing does not conform to the standards discussed in ASTM standards (ASTM, 2008b). The sample ravels around the point load and severe indentations are observed during the test. Due to the poor test performance coupled with the difficulty in obtaining proper samples, point load testing will not be pursued in this study.

3.3.2.7 Index property relationship

A statistical study will be undertaken to determine if index tests can be correlated to the UCS of CPB. Cement content of mix designs, moisture content, slump tests, and pulp
density results will be analyzed to determine if a relationship exists between the CPB UCS and these test results.

Bartlett and Macgregor (1994) found that as the moisture content increases, the compressive strength of a sample decreases. This relationship will be investigated with testing from the UBC NBK and, where available, from the Stillwater and Red Lake databases.

The database from Macassa has the following QA/QC measurements that will be investigated to determine if a correlation to the UCS can be determined from the following: grain size of constituents, pulp density, and slump tests. The relationship will be studied with statistical testing to determine the co-relation and relationship between the QA/QC tests and the test results of the CBP. If a relationship is found, a predictive model will be generated with in predicting future CPB strength from other relationships.

The purpose of the investigation of the relationship between CPB UCS and QA/QC or index values is to determine, one, if a relationship exists with strong correlation, and secondly to determine the suitability of this relationship in forecasting strengths of CPB once placed in-situ.

3.3.3 Mine Site Observations

The mine site observations will be based on primary research of mine sites. Over the course of this study four mines were visited and one mine was conceptually investigated.

The observations of the mine site will be based on the following criteria:

- Geotechnical setting;
- Mine operations;
- Paste backfill strengths and observations;
- Span widths under paste;
- Minimum ground support standards; and
- Design methodology.

The observations from the mine sites will form the empirical portion of this research. These observational findings will be used to support or counter findings of analytical and/or numerical analyses.
3.3.4 Analytical Modeling

The development of underhand cut-and-fill mining methods has its foundation in theories proposed by Marston (1930) and Terzaghi (1943) as well as in the silo theories developed by Janssen (1895) and Koenen (1896). These theories provide the idea of the mobilization of shear strength due to self-weight lateral earth pressure that allows the loads of the materials to be distributed to the sidewalls.

The analytical modeling will look at critical properties and parameters of design and provide analytical solutions to CPB sill stability. Theories presented by Mitchell (1989a), Stone (1993), Pakalnis et al. (2005), Newman, Pine and Ross (2001) and Jordan et al. (2003) will be investigated for their application to design with a focus on which failure method are critical for given stope geometries and CPB strength.

Analysis will be performed to determine the critical properties and parameters for all reasonable sill beam configurations. The following properties and parameters will be investigated: vertical loads, horizontal stress transfers, interface properties, material properties, the effect of ground support, the critical depth of cold joints for beam stability and the necessary height/width ratios. For the purpose of this study, a cold joint is defined as a discontinuous layer between successive paste pours within a sill beam. Cold-joints are problematic as the discontinuity between successive pours can have weaker shear and tensile strength that the homogenous CPB. Improvements and discussion on the analytical models will be discussed.

The issue with the analytical modeling is that it incorporates numerous properties and parameters that make parametric assessment difficult. To account for the large range of model properties and parameters, a Monte Carlo simulation will be performed to determine the most critical failures. In performing a Monte Carlo simulation, the analysis is no longer parametric, but probabilistic. Probabilistic stability studies in Rock Mechanics are common in limit-equilibrium studies (Hammah, Yacoub and Curran, 2009). A computer program will be designed to calculate the probability of failure of a sill beam given the range of variables determined from the experimental, historical and empirical research portions of this study.
3.3.5 Numerical Modeling

Numerical analysis of sill beam stability will be an ongoing objective of this research as it allows for an understanding of the strain developed within CPB sills during deformation. Software selection will be presented to determine the optimum sill beam behaviour by numerical analysis. Continuum models (RocScience Phase2, Midas’ Geotechnical Software (GTS), Plaxis 3D, and Itasca’s FLAC 2D and FLAC 3D) and discontinuum models (UDEC 3D, UDEC 2D) will be evaluated for the research. The software will need to be able to monitor both behaviour of the elements involved and the interfaces between them and be able to model these interactions in stages. Further, customization of the code will be required for constitutive models and the ability to perform parametric assessment using scripts.

A study will be conducted to determine the proper constitutive code for paste backfill. This will be a culmination of the laboratory testing and the constitutive models found in the literature search. The constitutive model code will be developed based on the appropriate findings. Once the constitutive model is created, it will be integrated into the numerical modeling software.

The selected software will be used for parametric studies and forward modeling will take place to determine the critical parameters of sill beam failure. The forward model will then be calibrated to the instrumentation and laboratory testing programs.

The following provides a description of considered numerical models, the justification in selection of the models, the methodology in selecting the appropriate constitutive models, and the anticipated finding of the numerical model analysis.

3.3.5.1 Model Requirements

The analytical solution provides a conceptual framework for sill beam behaviour; however, analytical solutions provide a non-time dependent, generally rigid approach to material displacements. The need for time dependent strain (not necessarily limited to creep) due to changing stress regimes within an active mining area need to be considered in design. The effect of closure, relaxation and increased vertical loads over time are essential in determining the performance of a sill beam. In addition, with analytical solutions, the three
The satisfaction of plane-strain conditions and the knock-on effect of using two-dimensional software needs to be discussed. Plane strain is defined as the state of strain such that the strain normal to an arbitrary x-y plane (typically aligned with \( \sigma_1 \) and \( \sigma_3 \)), defined as \( \varepsilon_z \), and the shear strains \( \gamma_{xz} \) and \( \gamma_{yz} \) are assumed to be zero (Timoshenko and Gere, 1961). This is to say that the stress state on the body in question is such that strains in z-direction are infinitesimally small in comparison to the z-direction length: the strain in z-direction is negligible. Further, the stress condition in the x-y plane is considered constant along the z-direction for plane-strain conditions.

Pakalnis, Tenney and Lang (1991) found that for numerical modeling, plane-strain conditions are approximated when the length of the stope is such that it is four times the height. One can question if this is satisfied with UCF mining, as stope height blocks can be hundreds of meters high; however, this is inconsequential to the plane-strain conditions of a sill beam under investigation. The stope block height only affects the boundary conditions on the sill beam under investigation: the original geometry of the sill beam is intact, only the loading on the block changes with mining above the sill beam. From studies of plans and from observations underground, it was found the length of the stopes were at a minimum four times that of the height of the sill beam. In considering this, Pakalnis et al. (1991) condition of plane-strain analysis is satisfied. With plane-strain being verified in the field, general design of sill beams can be determined with 2D software. Three dimensional analyses have the benefit of being able to model z-direction edge effects and the complex geometry of a stope layout. This benefit comes at the expense of model simplicity and ease of use in design. Considering that plane-strain conditions are typically satisfied, based on observation, the use of 2D software is appropriate.

Outside of the influence of the geometry and the ability to incorporate changes in boundary conditions, the numerical model has to handle the internal behaviour of CPB during deformation and that the appropriate stress path is followed. CPB is used in both high stress and high closure environments: the material is subject to high deformations that can lead the
material entering a plastic phase. The issue with the plasticity is the behaviour of the material, in one, entering the plastic phase, and second the plastic behavior needs to reflect the actual behaviour. Laboratory testing will be performed to determine the stress-strain response of CPB. This behaviour will need to be reflected in the numerical model: is the material linear elastic, simple-plastic; strain-softening; or strain hardening? Once known, the behaviour of the material will need to be incorporated into the model, either by pre-defined constitutive models or the ability for user-defined codes.

Typically, CPB sills have been designed analytically as was demonstrated in the literature search. With the analytical approach beam detachment and block displacement occur; no distortion of the beam occurs. The ability of grid elements to displace and distort is crucial in comparing numerical techniques to analytical. The selected numerical model will need to allow ‘physical’ displacement of grid elements.

The purpose of any model is to reflect the actual behaviour of the geotechnical problem. This poses the issue when dealing with the incorporation of support materials in analytical models: the complexity of the behaviour of bolts is either simplified or ignored. The selected numerical code will need to incorporate the actual behaviour of the support elements through numerically defined support elements, or by allowing aspects of the model to behave as supported beams.

Since part of the study on beam stability will be parametric assessment of material properties and model parameters of width, sill height, and the like, the model-cycle time and the ability for the numerical model to run script-based, or parametric study is essential. The viability of the model to run parametric studies will be investigated in the numerical model selection.

In addition to the above, cost will be factored into the numerical model selection. The goal of this research is to provide an accessible design for all users. High-end, esoteric numerical code that requires intense numerical programming and expensive software will be considered inappropriate with the research goals.
3.3.5.2 Numerical software codes

A look at commercially available software and their pros and cons will be weighed against the requirements from the software. For the study Phase2 (Rocscience, 2007); FLAC 2D and 3D (Itasca, 2012); UDEC 2D and 3D (Itasca, 2004); Map3D (Map3D, 2013); Midas GTS (Midas, 2013) and; Plaxis 2D (Plaxis, 2013) were considered for the numerical modeling analysis.

Table 3-3 is a summary of the advantages and disadvantages of the numerical models. FLAC 2D will be used for the numerical analysis. One downside to FLAC is that it is a two-dimensional model. The argument has been presented that plain-strain conditions are valid (Pakalnis et al., 1991) for UCF CPB mining: two dimensional analyses is suitable. Further, FLAC allows for incorporation of discrete failure surfaces with the use of interfaces. FLAC 2D will allow for simple model construction, the use of custom constitutive codes and batch file running to ease the parametric. The use of FLAC 2D to model backfill is also supported by work performed by Hughes et al. (2006); Andrieux, Brummer, Mortazavi, Simser & George (2003); Pierce (2001); and Swan and Brummer (2001).
<table>
<thead>
<tr>
<th>Numerical Codes</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phase2</td>
<td>• Simple design process</td>
<td>• FEM does not allow for monitoring of strains</td>
</tr>
<tr>
<td></td>
<td>• Non-linear, plastic constitutive models</td>
<td>during closure</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Non-custom constitutive models</td>
</tr>
<tr>
<td>FLAC 2D</td>
<td>• FDM model allows for 'real-time' strain</td>
<td>• Plain-strain conditions must be valid</td>
</tr>
<tr>
<td></td>
<td>development</td>
<td></td>
</tr>
<tr>
<td>FLAC 3D</td>
<td>• 3D geometry can be incorporated into analysis</td>
<td>• Large learning curve</td>
</tr>
<tr>
<td></td>
<td>• Custom constitutive codes</td>
<td>• Complex discretization can cause long-model</td>
</tr>
<tr>
<td></td>
<td></td>
<td>build/run times</td>
</tr>
<tr>
<td>UDEC 2D</td>
<td>• Discrete element models allows for discontinues to be explicitly added</td>
<td>• Plain-strain conditions must be valid</td>
</tr>
<tr>
<td></td>
<td>• Custom constitutive codes</td>
<td>• Discrete element not essential in research</td>
</tr>
<tr>
<td>UDEC 3D</td>
<td>• Custom constitutive codes</td>
<td>• Large learning curve</td>
</tr>
<tr>
<td></td>
<td>• 3D geometry can be incorporated into analysis</td>
<td>• Advantage of discrete element not essential in research</td>
</tr>
<tr>
<td>Midas GTS</td>
<td>• Powerful Graphical Presentation</td>
<td>• FEM does not allow for monitoring of strains</td>
</tr>
<tr>
<td></td>
<td></td>
<td>during closure</td>
</tr>
<tr>
<td>Plaxis 2D</td>
<td>• Common across numerous sectors</td>
<td>• Plain-strain conditions must be valid</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• FEM does not allow for monitoring of strains</td>
</tr>
<tr>
<td></td>
<td></td>
<td>during closure</td>
</tr>
<tr>
<td>Map 3D</td>
<td>• Boundary Element Method can accommodate</td>
<td>• Not well suited to small scale investigation</td>
</tr>
<tr>
<td></td>
<td>complex large geometry</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Well suited for stress analysis</td>
<td>• Time stepping of developed strains not possible</td>
</tr>
</tbody>
</table>

Table 3-3: Summary of numerical models considered
3.4 Research Data

The data used in this research consists of the following:

- data obtained from literature;
- data obtained from laboratory tests performed at the UBC rock mechanics lab;
- data obtained from laboratory tests performed by/for mines;
- data obtained from in-situ testing at mines; and
- data obtained from observations and documents provided during site visits to underground mines.

The data from literature is necessary in determining the friction angle and cohesion properties of CPB since no triaxial testing was performed as part of this study.

Data obtained from the lab testing at UBC NBK consists of the following: UCS, Brazilian Tensile strength, stress-strain behaviour, unit weight and moisture content. These test values were obtained through four distinct data sets: Red Lake test samples cored from boulders obtained from site, Red Lake Samples mixed and prepared at UBC, Macassa mine samples mixed and prepared at UBC, and samples obtained from Stillwater Mine. Details on the sample mineralogy, grain size, cement content/mix design, age of sample at testing date, sample size and quantities of each sample are discussed in a subsequent section of this chapter.

Three mine sites (Stillwater, Red Lake, and Macassa) provided their testing databases for this study. The data provided from these databases are as follows: UCS, and where applicable, moisture content, slump tests, unit weight, cement content, sample size.

Data from in-situ testing will be presented in subsequent chapter and is discussed within Section 3.3.2.

Observations and discussion on data obtained from visits to mine sites are presented in a subsequent section.

3.4.1 Cemented Paste Backfill Samples

In total, five CPB tests data sets are included for analysis as part of this research. The five sets CPB samples were of three forms: samples made in the UBC NBK rock mechanics laboratory, samples cast at the Stillwater Paste plant; and samples cast at the Macassa Fill plant.
A description of the five sets is presented with respect to the origin of the samples, date range of tests, cement content, grain size, mineralogy, sample preparation, slump and pulp density, size of the samples, and age of samples at time of testing.

Detailed information on the sample mineralogy, preparation (mix designs), and particle size distribution for all the samples sets are presented in Appendix 1.

3.4.1.1 UBC/Red Lake Cemented Paste Samples

The Red Lake Mine testing was performed to determine the optimum mix design for cement content from a UCS standpoint, post-peak behaviour of the CPB, and the effect of sample size on the UCS of CPB. The effect of the sample size on the UCS value was performed in an attempt to determine a correction factor for the various cylinder sizes used across industry; no standard size is currently used for test cylinders. Series of comparable samples are tested for their UCS values. A correction factor for the various samples sizes, similar to the Is50 Values from the Point Load Test (Ulusay and Hudson, 2007), is the desired outcome of this testing.

In addition to testing performed at UBC NBK described above, additional testing of in-situ testing performed by UBC NBK and independent testing of in-situ core was provided to the researcher.

3.4.1.2 UBC/Macassa Mine Samples

Primary research and testing was performed on samples from Macassa Mine. Full stream tails and mined sand was shipped to the UBC NBK. Three separate paste recipes were mixed to compare the strengths and to have three separate recipes to calibrate index testing.

The backfill at Macassa is considered a blended paste backfill, or a high density sand fill. The backfill solid aggregate consists of 70% esker sand and 30% mine tailings. The backfill binder content varies at the mine depending on the demands of the underground mining environment; backfill binders of 3, 5, 7 and 10 percent are used at the mine. The binder at the mine is a standard Type 10 Portland Cement. These blends were followed as closely as possible when the samples were made at UBC NBK, such that the values at Macassa could be replicated.
However, due to the need for the testing to have a large array of UCS values, alterations were made to the original recipe used at Macassa.

The Macassa backfill was mixed at UBC for determining an index value for an in-situ test. The three mixed tails batches were placed in separate mineral ore container, and the pneumatic penetration test was performed on the samples. Then, core samples were drilled from the cemented fill and UCS tests were performed. A correlation between the values recorded for the in-situ tests and the UCS is to be assessed.

### 3.4.1.3 Stillwater Cemented Paste Samples

Detailed discussion of Stillwater paste samples are discussed by Hughes et al. (2013) and by Jordan et al. (2003). The majority of the sample results from Stillwater are secondary results as preparation and testing was performed by Stillwater Mine and SK Geotechnical in Billings, Montana.

The purpose of the Stillwater test database is as follows: Perform statistical analyses to determine time based strength gains, the strength of samples based on cement content, the strength of samples based on moisture content, and the effect of sample curing underground versus the head of the reticulation system at the paste plant. The investigation into relationships of parameters and properties of the paste backfill used the Stillwater Paste Backfill testing database that was made available during the site visit. Percent solid information was not provided as part of the database. The testing database is inclusive of all samples tested between January 3, 2004 and July 7, 2012. In total 17,540 samples were tested over this period.

Further additional testing at UBC was performed on Stillwater samples to determine geotechnical properties of the material and verify results from SK Geotechnical labs. The purpose of the additional testing at UBC NBK is to determine CPB properties that can be used in numerical modeling. These properties include the UCS, Brazilian Tensile strength, stress-strain response and unit weight of the material. In addition, moisture contents of the sample were obtained to test relationships determined in the statistical analysis.

The testing database is inclusive of all samples tested between January 3, 2004 and July 7, 2012. In total 17,540 samples were tested over this period.
330 samples had to be omitted due to missing pertinent information; the following summarizes the omitted samples:

- 323 samples contained no cement content;
- 1 sample contained no age of specimen at test date; and
- 6 samples contained no reported strength values.

The following table summarizes the number of cylinder per cement content.

Table 3-4: Stillwater CPB testing database

<table>
<thead>
<tr>
<th>% Cement</th>
<th>Number of Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.0</td>
<td>393</td>
</tr>
<tr>
<td>8.0</td>
<td>12</td>
</tr>
<tr>
<td>9.5</td>
<td>6</td>
</tr>
<tr>
<td>10.0</td>
<td>1180</td>
</tr>
<tr>
<td>10.5</td>
<td>163</td>
</tr>
<tr>
<td>11.0</td>
<td>349</td>
</tr>
<tr>
<td>12.0</td>
<td>14827</td>
</tr>
<tr>
<td>12.5</td>
<td>149</td>
</tr>
<tr>
<td>13.0</td>
<td>113</td>
</tr>
<tr>
<td>15.0</td>
<td>8</td>
</tr>
<tr>
<td>18.0</td>
<td>12</td>
</tr>
<tr>
<td>Total</td>
<td>17212</td>
</tr>
</tbody>
</table>

From the above it can be clearly seen that the majority of backfilled poured at Stillwater is 12% binder content. The 8, 9.5 15 and 18 percent binder content samples did not have enough population for meaningful statistical information; these samples were omitted from further analyses. Statistical analysis will be performed on the 7, 10, 11 and 12 percent binder mix as there exists a large enough data population to allow meaningful relationships.

The data set includes outliers of higher and lower than expected values of UCS. These outliers are likely due to poor mix design, issues in sampling, testing error, or caused by environmental factors. It was determined that the outliers would influence the general trend of the data and should not be included in any statistical relationship.
An outlier was considered to be a UCS value of the same age and cement content that did not pass a hypothesized normalcy test. From a statistical point of view, an outlier was considered a UCS test that prevented a straight line fit to a UCS versus rank based Z-score (inversed normal distribution) plot; an $R^2$ coefficient of 0.9 was considered suitable linear fit. In total, after the data manipulation had taken place 15,344 UCS samples were suitable for statistical analysis of UCS vs. age and UCS vs. cement content.

### 3.4.1.4 Macassa fill plant test database

The investigation into the procedures for the Macassa testing was investigated by the researcher as part of the study and presented to the mine site. The review is based on the information found within the testing database and from detailed study by the researcher into the testing methods at the site. The purpose of the study was to identify the cause of a drop in strengths of the cemented backfill through statistical investigation of material properties and QC test values.

The Macassa testing database consists of 831 tested UCS samples. Cement content, age of testing and mix design is not constant for the tests. The UCS tests were performed by Shaba testing of Kirkland Lake, Ontario.

Investigation into the relationship between age of testing, cement content, particle size analysis, slump values and pulp density will be performed to determine the causation of a drop in strength over time and to be able to forecast future strength based on QC test data.

### 3.4.1.5 Red Lake cemented paste backfill test database

Investigation of the Red Lake paste plant test database is part of this study. The investigation purpose is to determine statistical relationships between cement content and the age of sample with the UCS of the material.

The Red Lake database was provided by the mine site as part of this research. The database consisted of 1508 UCS test samples. The tests were performed by Red Lake mines paste plant operators.
### 3.5 Chapter Summary

The research data and tools are described in detail for two purposes:

1) To provide a framework for the tests to be repeated and/or validated.
2) To provide the reader with an understanding of the tests, instruments, sample constituents, and standards used in this research.

The rationale for the development of this research was illustrated by the gaps in the current body of research as presented in the literature study, and the methodology provides a plan for addressing these gaps and answering the questions outlined in the research objectives. From here, the information provided follows two paths: sill beam stability and mechanical behaviour of CPB. In the end these two paths will be unified in a design guideline for CPB sill beam stability in the UCF mining method.
4 EXPERIMENTAL AND OBSERVATIONAL RESULTS AND ANALYSIS

Chapter 4 presents the observational and experimental results of CPB obtained from mine site data and performed at the NBK rock lab. The purposes of the experimental and observational studies are the following:

- Investigate statistical relationships between the UCS and backfill constituents;
- Investigate statistical relationships between the UCS and test sample diameter;
- Investigate statistical relationships between the UCS and age of tests;
- Investigate statistical relationships between the UCS and cement content;
- Investigate statistical relationships between the UCS and moisture content;
- Experimental study to determine the post-peak behaviour;
- Experimental study of the cyclical loading response of cemented paste backfill in unconfined compressive strength tests; and
- In-situ strength relationships.

This chapter presents the observational studies of backfill testing at Macassa Mine, Red Lake Mine and Stillwater Mine. From the observational approach, it is anticipated that the relationship between backfill strength and the following tests or properties: moisture content, cement content, and age of sample. In addition, a comparison between surface and underground sampled tests is presented.

The experimental studies presented are tests performed on Macassa, Red Lake and Stillwater Mine samples performed at the University of British Columbia NBK lab. The experimental tests performed determined the tensile strength, unit weight, UCS, post-peak behaviour and cyclical loading response of paste backfill. Tests are presented to determine if a scaling factor is appropriate for UCS tests based on the sample diameter of the sample, analogous to the ISRM point load index ‘Factor (Ulusay and Hudson, 2007).

A combination of both observational and experimental testing was performed to determine the in-situ strength of the paste backfill in comparison to the UCS values.
4.1 Observational Studies

The design of UCF sills is designed typically on the UCS of the backfill. As such the UCS value of CPB samples is monitored by technical services to assure that design strength are being targeted. Presentation and discussion on testing of backfill at mines is discussed by Hughes et al. (2013) and Rai et al. (2013), from the discussion it can be seen that large amount of data is available for analysis. Two questions asked by operating mines during this research are how much cement is required and when is the minimal time for the CPB to reach satisfactory strength.

Detailed investigation into the age of sample and cement content with respect to the UCS strength of the material is presented within. From these relationships, not only can an understanding be made on gain in strength over time, but also the ability to reduce the cement content of the CPB, realizing a cost-saving.

The issue with the UCS testing is that the results are typically obtained at 7, 14 and 28 days of testing per ASTM C39 (ASTM, 2010a). This causes an issue as stope turn-around times can be within less than seven days, and proper UCS strength values have not been reported. In addition to UCS values, typically other index values (slump, moisture content, or pulp density) are recorded at the time of testing. Investigation into the relationship between index values and UCS testing will be investigated to see if UCS strengths can be forecasted from other tests. The tests that will be investigated are moisture content, pulp density, slump test and particle size diameter to determine if a correlation with UCS exists.

Part of the database of the observational study has some unique data that requires separate analysis as it provides some interesting findings. These unique data sets consist of underground and surface strength of CPB tests for the same batches and coupled moisture content/UCS tests.

4.1.1 Unconfined Compressive Strength vs. Age of Sample

A noted strength increase is found as the cement within the CPB hydrates. The process of the hydration creates a hardened bond within the CPB matrix as discussed by Klein and
Simon, 2006; and Bartlett and Macgregor, 1994. Typically CPB samples are tested at 7, 14 and 28 day strength, although it was found in the databases that test of all ages occurred due to lab availability and scheduling. However, based on ASTM C39 (ASTM, 2010a) it is acceptable to take UCS values of the samples between 2.1% and 3.6% (6 to 20 hours) either side of the 7, 14 and 28 day samples. For all intents and purpose, this is only applicable to samples within the 28 day range. Where necessary 27and 29 day samples will become 28 day test values per standard. The advantage of knowing the development of strength over time of the samples is that once the material has reached its designed strength within a stope, the stope can then be scheduled to be mined and the beam should behave as designed.

An analysis of the strength gains over time of the UCS samples for the Stillwater, Macassa and Red Lake test database is performed. Outliers are eliminated per Section 3.4.1.3.4. Once outliers were eliminated, the only cement content that had significant population (greater than 5.0% of remaining data) was analyzed.

### 4.1.1.1 Stillwater Mine UCS vs. Age of Sample

An investigation into the strength gain of the paste backfill over time was investigated using the Stillwater testing database. Samples were filtered according to the cement content and age of test; no consideration was made for samples cast on surface or underground. The following tables summarize the quantity of tests per age of backfill, median strength, and the standard deviation for 7, 10, 11 and 12 percent cement content respectively.

Table 4-1: Median strengths of 7% cement CPB samples

<table>
<thead>
<tr>
<th>Age of Sample (days)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
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<td>9</td>
<td>257</td>
<td>259</td>
<td>28</td>
<td>0.11</td>
</tr>
<tr>
<td>4</td>
<td>11</td>
<td>243</td>
<td>234</td>
<td>44</td>
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</tr>
<tr>
<td>6</td>
<td>19</td>
<td>262</td>
<td>262</td>
<td>28</td>
<td>0.11</td>
</tr>
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<tr>
<td>14</td>
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<td>14</td>
<td>444</td>
<td>434</td>
<td>46</td>
<td>0.10</td>
</tr>
</tbody>
</table>
Table 4-2: Median strengths of 10% cement CPB samples

<table>
<thead>
<tr>
<th>Age of Sample (days)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
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</thead>
<tbody>
<tr>
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<td>328</td>
<td>69</td>
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<td>4</td>
<td>37</td>
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<td>351</td>
<td>92</td>
<td>0.25</td>
</tr>
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<td>5</td>
<td>79</td>
<td>385</td>
<td>359</td>
<td>124</td>
<td>0.32</td>
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<td>6</td>
<td>64</td>
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<td>317</td>
<td>130</td>
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<td>240</td>
<td>427</td>
<td>430</td>
<td>175</td>
<td>0.41</td>
</tr>
<tr>
<td>8</td>
<td>32</td>
<td>445</td>
<td>430</td>
<td>122</td>
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<td>465</td>
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<td>0.46</td>
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<td>13</td>
<td>410</td>
<td>442</td>
<td>84</td>
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<td>531</td>
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<td>0.29</td>
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<td>20</td>
<td>702</td>
<td>645</td>
<td>204</td>
<td>0.29</td>
</tr>
</tbody>
</table>

Table 4-3: Median strengths of 11% cement CPB samples

<table>
<thead>
<tr>
<th>Age of Sample (days)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
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<td>738</td>
<td>737</td>
<td>58</td>
<td>0.08</td>
</tr>
<tr>
<td>10</td>
<td>22</td>
<td>766</td>
<td>737</td>
<td>126</td>
<td>0.16</td>
</tr>
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<td>13</td>
<td>27</td>
<td>1080</td>
<td>1132</td>
<td>206</td>
<td>0.19</td>
</tr>
<tr>
<td>14</td>
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<td>825</td>
<td>47</td>
<td>0.06</td>
</tr>
<tr>
<td>28</td>
<td>80</td>
<td>1069</td>
<td>1040</td>
<td>165</td>
<td>0.15</td>
</tr>
<tr>
<td>29</td>
<td>21</td>
<td>1276</td>
<td>1397</td>
<td>348</td>
<td>0.27</td>
</tr>
<tr>
<td>36</td>
<td>5</td>
<td>1386</td>
<td>1397</td>
<td>83</td>
<td>0.06</td>
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</table>
Table 4-4: Median strengths of 12% cement CPB samples

<table>
<thead>
<tr>
<th>Age of Sample (days)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>454</td>
<td>476</td>
<td>460</td>
<td>196</td>
<td>0.41</td>
</tr>
<tr>
<td>4</td>
<td>656</td>
<td>555</td>
<td>518</td>
<td>213</td>
<td>0.38</td>
</tr>
<tr>
<td>5</td>
<td>758</td>
<td>594</td>
<td>537</td>
<td>240</td>
<td>0.40</td>
</tr>
<tr>
<td>6</td>
<td>826</td>
<td>569</td>
<td>515</td>
<td>197</td>
<td>0.35</td>
</tr>
<tr>
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<td>2058</td>
<td>657</td>
<td>589</td>
<td>275</td>
<td>0.42</td>
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<td>617</td>
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<td>565</td>
<td>235</td>
<td>0.38</td>
</tr>
<tr>
<td>9</td>
<td>467</td>
<td>659</td>
<td>621</td>
<td>221</td>
<td>0.33</td>
</tr>
<tr>
<td>10</td>
<td>224</td>
<td>645</td>
<td>640</td>
<td>144</td>
<td>0.22</td>
</tr>
<tr>
<td>13</td>
<td>570</td>
<td>721</td>
<td>654</td>
<td>224</td>
<td>0.31</td>
</tr>
<tr>
<td>14</td>
<td>1916</td>
<td>786</td>
<td>717</td>
<td>304</td>
<td>0.39</td>
</tr>
<tr>
<td>15</td>
<td>499</td>
<td>753</td>
<td>678</td>
<td>266</td>
<td>0.35</td>
</tr>
<tr>
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<td>402</td>
<td>731</td>
<td>683</td>
<td>200</td>
<td>0.27</td>
</tr>
<tr>
<td>27</td>
<td>569</td>
<td>840</td>
<td>797</td>
<td>162</td>
<td>0.19</td>
</tr>
<tr>
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<td>2681</td>
<td>984</td>
<td>911</td>
<td>346</td>
<td>0.35</td>
</tr>
<tr>
<td>29</td>
<td>642</td>
<td>902</td>
<td>837</td>
<td>285</td>
<td>0.32</td>
</tr>
<tr>
<td>30</td>
<td>399</td>
<td>1002</td>
<td>957</td>
<td>304</td>
<td>0.30</td>
</tr>
</tbody>
</table>

The plots of the data using the medians of the samples versus age are shown in Figure 4-1; best fit curves are plotted along with the data.
Figure 4-1: UCS vs. age of Stillwater samples for varying cement content

Table 4-5 shows the modeled curves for the estimated strength and for each cement content. The design strength value at Stillwater is 1000 kPa. Using the relationships found in Table 4-5, the required time needed for the design strength to be reached is 18 years, 298 days, 15 days and 64 days for 7, 10, 11, and 12 percent cement content respectively. The time needed to get to design strength generally decreases as the cement content increases. The turn-around time for the stope based on the cost of the samples needs to be considered when performing scheduling of the orebody.

Table 4-5: Equations for estimated strength of Stillwater CPB based on age and cement content

<table>
<thead>
<tr>
<th>Cement Content</th>
<th>Estimated Strength (kPa)</th>
<th>Standard Error (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7%</td>
<td>Strength (kPa) = 100 ln (Age of Sample) + 122</td>
<td>57</td>
</tr>
<tr>
<td>10%</td>
<td>Strength (kPa) = 156 ln (Age of Sample) + 110</td>
<td>41</td>
</tr>
<tr>
<td>11%</td>
<td>Strength (kPa) = 398 ln (Age of Sample) + -84</td>
<td>163</td>
</tr>
<tr>
<td>12%</td>
<td>Strength (kPa) = 189 ln (Age of Sample) + 212</td>
<td>47</td>
</tr>
</tbody>
</table>
4.1.1.2 Macassa Mine: UCS Strength vs. Age of Sample

The Macassa Mine has performed UCS tests for QA for the period between January, 2009 and April, 2011. To ensure that meaningful relationships can be derived, the data presented had outliers removed per Chapter 3.4.1.3.4; 1068 samples were determined suitable for analysis. After the data manipulation, significant data (greater than 5% of the overall data population) existed for the 14 and 28 day samples for the data analysis for 3, 5, 7 and 10 percent cement content samples. The following tables summarize the quantity of tests per age of backfill, mean strength, median strength, standard deviation and coefficient of variation for 3, 5, 7 and 10 percent cement content respectively; the data is plotted in Figure 4-2.

Table 4-6: Median strengths of 3% cement backfill samples

<table>
<thead>
<tr>
<th>Age of Sample (days)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>72</td>
<td>1688</td>
<td>1669</td>
<td>142</td>
<td>0.08</td>
</tr>
<tr>
<td>28</td>
<td>241</td>
<td>1596</td>
<td>1691</td>
<td>656</td>
<td>0.41</td>
</tr>
</tbody>
</table>

Table 4-7: Median strengths of 5% cement backfill samples

<table>
<thead>
<tr>
<th>Age of Sample (days)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>77</td>
<td>1367</td>
<td>1310</td>
<td>294</td>
<td>0.22</td>
</tr>
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<td>28</td>
<td>219</td>
<td>1461</td>
<td>1373</td>
<td>452</td>
<td>0.31</td>
</tr>
</tbody>
</table>

Table 4-8: Median strengths of 7% cement backfill samples

<table>
<thead>
<tr>
<th>Age of Sample (days)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>88</td>
<td>2815</td>
<td>2849</td>
<td>739</td>
<td>0.26</td>
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<td>167</td>
<td>2933</td>
<td>2776</td>
<td>608</td>
<td>0.21</td>
</tr>
</tbody>
</table>
Table 4-9: Median strengths of 10% cement backfill samples

<table>
<thead>
<tr>
<th>Age of Sample (days)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>68</td>
<td>4308</td>
<td>4250</td>
<td>891</td>
<td>0.21</td>
</tr>
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<td>28</td>
<td>136</td>
<td>4631</td>
<td>4564</td>
<td>1021</td>
<td>0.22</td>
</tr>
</tbody>
</table>

Figure 4-2: UCS vs. age of Macassa samples for varying cement content

Figure 4-2 (dotted lines indicate values of one standard deviation) shows the modeled curves for the estimated strength and standard error for each cement content. Table 4-10 provides calculations for estimated strength of the Macassa backfill. From Figure 4-2 and Table 4-10 it can be seen that the highest strengths are found with the highest percent of backfill. The 4500 kPa median strength of the 10% backfill samples is very high for a true CPB. The
explanation of the strength could be due to the gradient of the samples as shown in Figure A1.3. and is corroborated by research of Weatherwax (2007).

Table 4-10: Equations for estimated strength of Macassa backfill based on age and cement content

<table>
<thead>
<tr>
<th>Cement Content</th>
<th>Estimated Strength (kPa)</th>
<th>Standard error (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3%</td>
<td>Strength (kPa) = 542 * ln (Age of Sample) + 41</td>
<td>503</td>
</tr>
<tr>
<td>5%</td>
<td>Strength (kPa) = 435 * ln (Age of Sample) + 28</td>
<td>173</td>
</tr>
<tr>
<td>7%</td>
<td>Strength (kPa) = 901 * ln (Age of Sample) + 81</td>
<td>255</td>
</tr>
<tr>
<td>10%</td>
<td>Strength (kPa) = 1,436 * ln (Age of Sample) + 79</td>
<td>492</td>
</tr>
</tbody>
</table>

4.1.1.3 Red Lake Mine: UCS Strength vs. Age of Sample

The Red Lake Mine has performed UCS tests for QA for the period between 2001 and 2004. The data presented had outliers removed per Chapter 3.4.1.3.4; 1505 samples were suitable for analysis. After the data manipulation, there was significant data (greater than 5% of the overall data population) existing for the 3, 7 and 28 day samples for the data analysis for percent cement content samples. Table 4-11 through

Table 4-13 summarize the quantity of tests per age of backfill, mean strength, median strength, standard deviation and coefficient of variation for 5, 10 and 15 percent cement content respectively; the data is plotted in Figure 4-3.

Table 4-11: Median strengths of 5% Red Lake CPB samples

<table>
<thead>
<tr>
<th>Age of Sample (days)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>222</td>
<td>126</td>
<td>125</td>
<td>42</td>
<td>0.33</td>
</tr>
<tr>
<td>7</td>
<td>231</td>
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<td>218</td>
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<td>0.31</td>
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<tr>
<td>28</td>
<td>225</td>
<td>466</td>
<td>467</td>
<td>116</td>
<td>0.25</td>
</tr>
</tbody>
</table>
Table 4-12: Median strengths of 10% Red Lake CPB samples

<table>
<thead>
<tr>
<th>Age of Sample (days)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>248</td>
<td>283</td>
<td>270</td>
<td>97</td>
<td>0.34</td>
</tr>
<tr>
<td>7</td>
<td>239</td>
<td>547</td>
<td>529</td>
<td>150</td>
<td>0.27</td>
</tr>
<tr>
<td>28</td>
<td>273</td>
<td>1171</td>
<td>1179</td>
<td>327</td>
<td>0.28</td>
</tr>
</tbody>
</table>

Table 4-13: Median strengths of 15% Red Lake CPB samples

<table>
<thead>
<tr>
<th>Age of Sample (days)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>22</td>
<td>555</td>
<td>607</td>
<td>148</td>
<td>0.27</td>
</tr>
<tr>
<td>7</td>
<td>22</td>
<td>919</td>
<td>947</td>
<td>151</td>
<td>0.16</td>
</tr>
<tr>
<td>28</td>
<td>23</td>
<td>1991</td>
<td>2016</td>
<td>753</td>
<td>0.38</td>
</tr>
</tbody>
</table>

Figure 4-3: UCS vs. age of Red Lake samples for varying cement content
Figure 4-3 (dotted lines indicate values of the standard error of the model) shows the modeled curves for the estimated strength and standard error for each cement content. Table 4-14 provides calculations for estimated strength of the Red Lake backfill. Within Table 4-14 and Figure 4-3, it can be seen that the highest strengths are found with the highest percent of backfill. This corroborates the higher strength with higher cement content relationships that was found within the Macassa and Stillwater database.

Table 4-14: Equations for estimated strength of Red Lake backfill based on age and cement content

<table>
<thead>
<tr>
<th>Cement Content</th>
<th>Estimated Strength (kPa)</th>
<th>Standard error (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>$\text{Strength (kPa)} = 139 \ln (\text{Age of Sample}) - 20$</td>
<td>133</td>
</tr>
<tr>
<td>10%</td>
<td>$\text{Strength (kPa)} = 354 \ln (\text{Age of Sample}) - 70$</td>
<td>32</td>
</tr>
<tr>
<td>15%</td>
<td>$\text{Strength (kPa)} = 596 \ln (\text{Age of Sample}) - 58$</td>
<td>100</td>
</tr>
</tbody>
</table>

As expected per Bartlett and Macgregor (1994), for all cement contents and mine sites as the age of sample increased as did the strength of the test sample. This finding was then applied to a forecasting model to determine the strength gain as a function of time for each cement mix design studied.

4.1.2 Unconfined Compressive Strength vs. Cement Content

As the cement content is increased, the strength of CPB increases (Klein and Simon, 2006; Huang, Xia and Qiao, 2011; Fall et al., 2007; Bartlett and Macgregor, 1994). The following looks to determine the relationship between cement content and UCS strength using existing databases from Stillwater, Macassa Mine and Red Lake Mines QA databases of UCS testing. Although this relationship was inadvertently proven in section 4.1.1; this section directly studies the effect of cement content on the strength of backfill.

4.1.2.1 Stillwater Mine UCS vs. Cement Content

The Stillwater Mine testing database was discussed in Section 3.4.1. Outliers were eliminated per Section 3.4.1.3.4. Once outliers were eliminated, the only cement content that had significant population (greater than 5.0% of remaining data) was the 7%, 10% and 12% cement content. Per ASTM C39 (ASTM, 2010a) samples that were tested within 20 hours of the 28 day
are considered representative of the 28 day sample and included within the 28 day test bin. Figure 4-4 summarizes the relationships between UCS and binder content; Table 4-15 through Table 4-17 provides this data.

Figure 4-4: UCS vs. Stillwater CPB binder content for various ages of samples

![Figure 4-4: UCS vs. Stillwater CPB binder content for various ages of samples](image)

Table 4-15: 7 Day UCS values of CPB by binder content

<table>
<thead>
<tr>
<th>Percent Cement (%)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>74</td>
<td>397</td>
<td>382</td>
<td>129</td>
<td>0.32</td>
</tr>
<tr>
<td>10</td>
<td>240</td>
<td>427</td>
<td>430</td>
<td>175</td>
<td>0.41</td>
</tr>
<tr>
<td>11</td>
<td>32</td>
<td>738</td>
<td>737</td>
<td>58</td>
<td>0.08</td>
</tr>
<tr>
<td>12</td>
<td>2058</td>
<td>657</td>
<td>589</td>
<td>275</td>
<td>0.42</td>
</tr>
</tbody>
</table>
Table 4-16: 14 Day UCS values of CPB by binder content

<table>
<thead>
<tr>
<th>Percent Cement (%)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>84</td>
<td>424</td>
<td>397</td>
<td>131</td>
<td>0.31</td>
</tr>
<tr>
<td>10</td>
<td>148</td>
<td>533</td>
<td>531</td>
<td>238</td>
<td>0.45</td>
</tr>
<tr>
<td>11</td>
<td>20</td>
<td>831</td>
<td>825</td>
<td>47</td>
<td>0.06</td>
</tr>
<tr>
<td>12</td>
<td>1916</td>
<td>786</td>
<td>717</td>
<td>304</td>
<td>0.39</td>
</tr>
</tbody>
</table>

Table 4-17: 28 Day UCS values of CPB by binder content

<table>
<thead>
<tr>
<th>Percent Cement (%)</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>79</td>
<td>547</td>
<td>521</td>
<td>135</td>
<td>0.25</td>
</tr>
<tr>
<td>10</td>
<td>211</td>
<td>662</td>
<td>679</td>
<td>264</td>
<td>0.40</td>
</tr>
<tr>
<td>11</td>
<td>80</td>
<td>1069</td>
<td>1040</td>
<td>165</td>
<td>0.15</td>
</tr>
<tr>
<td>12</td>
<td>2681</td>
<td>984</td>
<td>911</td>
<td>346</td>
<td>0.35</td>
</tr>
</tbody>
</table>

From the data shown in Figure 4-4, two trends are apparent:

- As age of sample increases, so does the UCS strength;
- The chart shows a non-linear increase to a data-confined maximum strength for the 11% cement content, before dropping off.

The first point is discussed in detail in Section 4.1.1.1. For the second point it appears that a maximum strength is achieved at a cement content of 11%. The reason for this could be as described by Revell (2004), in which there exists optimum cement content for CPB given the same yield strength. It is noted that the slump performance for the material at Stillwater is measured to be 18cm and is kept constant outside of the cement content. Discussion with Stillwater indicated that the 11% cement mix had an additive to assist in the flowability of the material; the additive could have an effect on the strength of the material. The issue with the data as presented is that the 11% cement content has a small population in comparison to the 12% and further testing would be required to determine if it is a statistical anomaly, or a true trend indicating optimum cement content for CPB at Stillwater is 11% binder content. The
higher strength with lower cement content has the potential to save Stillwater money while providing a higher quality of CPB.

Table 4-18 provides strength estimation for Stillwater Mine to determine the strength of the CPB at certain times based on the cement content.

Table 4-18: Estimated strength of CPB based on cement content

<table>
<thead>
<tr>
<th>Age of Sample</th>
<th>Estimated Strength (kPa)</th>
<th>Standard Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>44 (Cement %)-61</td>
<td>52</td>
</tr>
<tr>
<td>7</td>
<td>55 (Cement %)-15</td>
<td>134</td>
</tr>
<tr>
<td>14</td>
<td>76 (Cement %)-145</td>
<td>118</td>
</tr>
<tr>
<td>28</td>
<td>93 (Cement %)-140</td>
<td>144</td>
</tr>
</tbody>
</table>

### 4.1.2.2 Macassa Mine UCS vs. Cement Content

The Macassa Mine testing database was discussed in Section 3.4.1. Outliers were eliminated per Section 3.4.1.3.4. Once outliers were eliminated, the only cement content that had significant population (greater than 5.0% of remaining data) were the 3%, 5%, 7% and 10% cement content. Figure 4-5 summarizes the relationships between UCS and binder content; Table 4-19 and Table 4-20 list the data.

Table 4-19: 14 Day UCS values of Macassa backfill by binder content

<table>
<thead>
<tr>
<th>Cement Content</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>3%</td>
<td>72</td>
<td>1688</td>
<td>1669</td>
<td>142</td>
<td>0.08</td>
</tr>
<tr>
<td>5%</td>
<td>77</td>
<td>1367</td>
<td>1310</td>
<td>294</td>
<td>0.22</td>
</tr>
<tr>
<td>7%</td>
<td>88</td>
<td>2815</td>
<td>2849</td>
<td>739</td>
<td>0.26</td>
</tr>
<tr>
<td>10%</td>
<td>68</td>
<td>4308</td>
<td>4250</td>
<td>891</td>
<td>0.21</td>
</tr>
</tbody>
</table>
Table 4-20: 28 Day UCS values of Macassa backfill by binder content

<table>
<thead>
<tr>
<th>Cement Content</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>3%</td>
<td>241</td>
<td>1596</td>
<td>1691</td>
<td>656</td>
<td>0.41</td>
</tr>
<tr>
<td>5%</td>
<td>219</td>
<td>1461</td>
<td>1373</td>
<td>452</td>
<td>0.31</td>
</tr>
<tr>
<td>7%</td>
<td>167</td>
<td>2933</td>
<td>2776</td>
<td>608</td>
<td>0.21</td>
</tr>
<tr>
<td>10%</td>
<td>136</td>
<td>4631</td>
<td>4564</td>
<td>1021</td>
<td>0.22</td>
</tr>
</tbody>
</table>

Figure 4-5: UCS (median strength) vs. Macassa backfill binder content for 14 and 28 day samples

From the data shown in Figure 4-5, the following is concluded:

- The 5% samples do not follow the trend that as the cement content increases, so does the strength. This is considered an anomaly to this data set as this general
trend is proven in literature increases (Klein and Simon, 2006; Huang, Xia and Qiao, 2011; Fall et al., 2007; Bartlett and Macgregor, 1994).

- Peak cement content similar to that found in Figure 4-4 is not found. However, slump records indicate that the slump is not controlled as is done in the QA at Stillwater.

In general the data for Macassa has poor control and large variance is noted in the test data. A cause for the variance may be due to the change of the coarse fraction or due to the change in pulp density. Further study on this work is presented below. Table 4-21 provides strength estimation for Stillwater Mine to use to determine the strength of the CPB at certain times based on the cement content.

Table 4-21: Estimated strength of Macassa backfill based on cement content

<table>
<thead>
<tr>
<th>Age of Sample</th>
<th>Estimated Strength (kPa)</th>
<th>Standard Error (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>412(Cement %)-54</td>
<td>614</td>
</tr>
<tr>
<td>28</td>
<td>448(Cement %)-199</td>
<td>652</td>
</tr>
</tbody>
</table>

### 4.1.2.3 Red Lake Mine UCS vs. Cement Content

The Red Lake Mine testing database was discussed in Section 3.4.1. Outliers were eliminated per Section 3.4.1.3.4. Once outliers were eliminated, the only cement content that had significant population (greater than 5.0 % of remaining data) was the 7%, 10% and 12% cement content. Per ASTM C39 (ASTM, 2010a) samples that were tested within 20 hours of the 28 day are considered representative of the 28 day sample and included within the 28 day test bin. Figure 4-4 summarizes the relationships between UCS and binder content; Table 4-22, Table 4-23, and Table 4-24 lists the data.

Table 4-22: 3 Day UCS values of Red Lake CPB by binder content

<table>
<thead>
<tr>
<th>Cement Content</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>222</td>
<td>126</td>
<td>125</td>
<td>42</td>
<td>0.33</td>
</tr>
<tr>
<td>10%</td>
<td>248</td>
<td>283</td>
<td>270</td>
<td>97</td>
<td>0.34</td>
</tr>
<tr>
<td>15%</td>
<td>22</td>
<td>555</td>
<td>607</td>
<td>148</td>
<td>0.27</td>
</tr>
</tbody>
</table>
Table 4-23: 7 Day UCS values of Red Lake CPB by binder content

<table>
<thead>
<tr>
<th>Cement Content</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>231</td>
<td>221</td>
<td>218</td>
<td>68</td>
<td>0.31</td>
</tr>
<tr>
<td>10%</td>
<td>239</td>
<td>547</td>
<td>529</td>
<td>150</td>
<td>0.27</td>
</tr>
<tr>
<td>15%</td>
<td>22</td>
<td>919</td>
<td>947</td>
<td>151</td>
<td>0.16</td>
</tr>
</tbody>
</table>

Table 4-24: 28 Day UCS values of Red Lake CPB by binder content

<table>
<thead>
<tr>
<th>Cement Content</th>
<th>Count</th>
<th>Mean Strength (kPa)</th>
<th>Median Strength (kPa)</th>
<th>Standard Deviation (kPa)</th>
<th>Coefficient of Variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>225</td>
<td>466</td>
<td>467</td>
<td>116</td>
<td>0.25</td>
</tr>
<tr>
<td>10%</td>
<td>273</td>
<td>1171</td>
<td>1179</td>
<td>327</td>
<td>0.28</td>
</tr>
<tr>
<td>15%</td>
<td>23</td>
<td>1991</td>
<td>2016</td>
<td>753</td>
<td>0.38</td>
</tr>
</tbody>
</table>

Figure 4-6: UCS vs. Red Lake CPB binder content for various ages of samples
From the data shown in Figure 4-6, the following is validated or apparent:

- the higher cement content has higher strengths;
- the older the sample the higher the strengths;
- ‘peak’ cement content similar to that found in Figure 4-4 is not found. However, slump records indicate that the slump is not controlled as is done in the QA at Stillwater.

Table 4-25 provides strength estimation for Red Lake Mine to use to determine the strength of the CPB at certain times based on the cement content.

<table>
<thead>
<tr>
<th>Age of Sample</th>
<th>Estimated Strength (kPa)</th>
<th>Standard Error (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>48 (Cement %)-148</td>
<td>78</td>
</tr>
<tr>
<td>7</td>
<td>59 (Cement %)-240</td>
<td>44</td>
</tr>
<tr>
<td>28</td>
<td>155 (Cement %)-328</td>
<td>51</td>
</tr>
</tbody>
</table>

4.1.3 Unconfined Compressive Strength vs. Moisture Content

Stillwater Mine performed a series of tests that measured both the moisture content and unconfined compressive strength of CPB samples for the 12% cement mix design. In total 30 samples had coupled tests taken, one sample was eliminated from the data set due to erroneous readings. In addition, the researcher performed four UCS tests with moisture contents at the NBK rock lab.

Bartlett and Macgregor (1994) found that as the moisture content increases, the compressive strength of a sample decreases. This relationship was found with the test data presented by Stillwater.

Figure 4-7 demonstrates that a natural logarithmic relationship exists between the moisture content and the UCS of CPB; the testing by NBL lab verifies the data. The Pearson Product Moment Correlation ($R^2$) of 0.7299 for the data suggests that a strong relationship exists.
From a best fit line of the data, an equation estimating the UCS based on moisture content is presented in Equation 4-1. The standard error for the estimated UCS from a moisture content measurement was found to be 90 kPa.

\[
UCS(kPa) = -410.1 \ln(Moisture\ Content(\%)) + 2490
\]

Equation 4-1

In addition to data available from the Stillwater database, coupled moisture content and UCS values were obtained from the Red Lake Mine testing performed commercially at the UBC NBK Lab in July of 2008. Samples were cored from boulders sent from Red Lake Mine, the cement content for the moisture content is 10%, the ages of samples were not released by the mine. In total 20 samples were tested for both moisture content and UCS. The relationship between the moisture content and UCS is shown in Figure 4-8.
Figure 4-8: UCS vs. moisture content of Red Lake CPB Samples

The relationship to estimate the UCS based on the moisture content is shown in Equation 4-2; the standard error present with this relationship is 244 kPa.

\[
UCS(kPa) = -2286\ln(Moisture\ Content(\%)) + 9415.8
\]

Comparing the two equations it can be seen that the Red Lake relationship is more sensitive to moisture content. This is likely due to the long hydration process associated with the long-term strength gain of Red Lake samples (slope of log of time of data presented in Figure 4-3) in comparison to those of Stillwater (as above, shown in Figure 4-1). As the cement hydrates, the free moisture is incorporated into the chemical reaction hardening the samples. This strength gain takes longer as seen in Figure 4-3 for Red Lake compared to Figure 4-1 of the Stillwater samples.

4.1.4 Slump Test and Pulp Density Test

Slump tests are performed as part of the QA/QC procedures at mine sites in an effort to monitor the flowability of the backfill. As part of the Macassa QA/QC CPB testing database, slump values were included with UCS results. The slump values were recorded on site in inches and were converted to metric values for this study. Figure 4-9 is a histogram plot of the slump values sorted by cement content.
There is no relationship in the data between slump value and cement content. However, investigation was performed to determine if a relationship exists between the slump value and the UCS of the material. Figure 4-10 presents the slump vs. UCS data of the Macassa CPB QA/QC data.
There is a general negative linear relationship between the slump values and the UCS of the material. The slump test has an inherent imprecision; the measurements are rounded off to the nearest ¼ inch (0.5 cm); however, in discussion with operations the slump is typically rounded off to the nearest ½ inch (1 cm). This will have an effect on the correlation of the data. Yet, the negative relationship is still viable with a moderate correlation. It can be concluded from the data that as the slump value decreases, the UCS strength increases.
The slump of the material is directly correlated to the pulp density of the material. Figure 4-11 shows the relationship of the pulp density to the slump of the material at Kirkland Lake operations. It can be seen that there is a general negative relationship between pulp density and slump. Simply stated: the density of the backfill increases (less water added), the slump decreases. Inversely, as more water is added, the slump increases.

There are two interpretations of the pulp density and slump correlation. One, the pulp density measurement could be approximated by the slump values. However, since the slump test has low precision (1/4 inch), the pulp density provides a more precise measurement of the yield strength of the material. Pulp density should be added to the QA/QC procedures due to its improved accuracy over the slump tests.

4.2 Experimental Test Results and Analysis

Experimental tests were performed to determine properties of backfill that can be subsequently used in analytical and numerical analysis. The properties that are required for analysis are compressive strength, cohesion, tension, friction angle, and stress-strain behaviour.
(including Young’s modulus and Poisson’s ratio). In order to determine these values, samples were obtained from mine sites or cast at the NBK lab and tested for stress-strain characteristics and strengths.

The following sub-sections detail UCS with stress strain measurements and tensile strength testing performed at UBC. In addition to properties of CPB, the testing program attempts to verify the relationships found in Section 4.1.

4.2.1 Unconfined Compressive Strength Tests

This section discusses all UCS values obtained during all testing programs performed at the UBC NBK lab; the results are summarized in Appendix 2. This includes constant stress, constant strain and cyclical testing, and includes sample from the Macassa, Red Lake and Stillwater laboratory testing programs. In total 64 UCS tests were performed for the observational study portion of this chapter. The UCS testing results will be used as follows:

- Red Lake testing: determination of effect of sample diameter on UCS results;
- Stillwater testing: verification of relationships determined in Chapter 4.1 and use test results in determining tensile strength based on UCS; and
- Macasssa testing: due to lab mix designs different from those of the mine site, relationships are not valid. Results will be used in subsequent in-situ testing section.

4.2.1.1 Red Lake test Results

The Red Lake observational test samples do not have the constitutive make up as those made at the Red Lake paste plant. Variation in water chemistry, cement content and curing conditions make for poor comparison. As such the samples will be used to determine the effect of sample size on UCS values. The purpose of this investigation is to determine if a size correction is appropriate for CPB UCS tests; no standard for test size exists. To determine this relationship, samples will be grouped depending on their age, cement content and sample diameter size. Median values from the data set will be used in determining the relationships in an effort to remove minimum and maximum values that could be attributed to testing error.

Data filtering on the data presented in Appendix 2 found that no conclusive trend was found between sample size and UCS material. It was thought that the UCS of the sample could
be scaled in a method similar to the ‘F’ factor of the Point Load Test (Ulusay and Hudson, 2007). The advantage to this method would be to normalize testing data between testing data sets within mines. The data presented does not support the existence of a scaling factor for sample strength based on sample diameter.

Yu (1995) recommends that the largest particle size should be 1/6 of the sample diameter. For paste backfill this is not an issue as the particle sizes are typically fine sand or smaller (Fall et al., 2007). Since no scaling relationship is required, the size of sample is unnecessary.

### 4.2.1.2 Stillwater test Results

The Stillwater tests were performed more for the purposes of determining the stress-strain property of the CPB; however, the UCS test values will be used to determine the validity of the relationship determined in Table 4-5. Table 4-26 shows the test values versus the modeled values.

<table>
<thead>
<tr>
<th>Sample Set</th>
<th>Cement Content (%)</th>
<th>Age (days)</th>
<th>Sample Name</th>
<th>Tested Strength (kPa)</th>
<th>Range of Modeled Strength (kPa)</th>
<th>Difference from Modeled Strength (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stillwater</td>
<td>12</td>
<td>45</td>
<td>35E8600_745pm</td>
<td>917</td>
<td>885 - 979</td>
<td>15</td>
</tr>
<tr>
<td>Stillwater</td>
<td>12</td>
<td>45</td>
<td>35E8600_545pm</td>
<td>1019</td>
<td>885 - 979</td>
<td>-86</td>
</tr>
<tr>
<td>Stillwater</td>
<td>12</td>
<td>45</td>
<td>35E8600_345pm</td>
<td>405</td>
<td>885 - 979</td>
<td>527</td>
</tr>
<tr>
<td>Stillwater</td>
<td>12</td>
<td>45</td>
<td>35E8600_245pm</td>
<td>824</td>
<td>885 - 979</td>
<td>108</td>
</tr>
<tr>
<td>Stillwater</td>
<td>12</td>
<td>45</td>
<td>35E8600_145pm</td>
<td>763</td>
<td>885 - 979</td>
<td>169</td>
</tr>
</tbody>
</table>

It can be seen that one test value is within the modeled range with one outlier from the data set; the outlier (35E8600_345pm) will be removed from any subsequent analysis. This suggests that the forecasting model predicted may need more data for more mature samples. Another possibility is that the lab test samples (45 days) are older than those that the relationships were determined upon. Testing of samples aged greater than 28 days at Stillwater mine could improve the accuracy of the modeled equations. It is recommended that Stillwater Mine test samples that are at the age of the average length of time it takes to mine a stope. This time is not known at present.
4.2.2 Tensile Strength Tests

Tensile strength is essential in determining the flexural strength of a beam (Mitchell, 1989a). Direct tensile strength is rarely performed due to the difficulty in securing the sample to the metal platens; to perform tensile testing, Brazilian Tensile testing is performed per ISRM-1979 (Ulusay and Hudson, 2007). Four tensile tests were performed on Stillwater samples obtained from site. Table 4-27 summarizes the results of the tensile testing, note a column is added to compare the tensile strength to the average UCS value of samples listed in Table 4-26.

Table 4-27: Results of tensile testing of Stillwater samples

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>Diameter</th>
<th>Thickness</th>
<th>Ratio</th>
<th>Peak Load</th>
<th>Tensile Strength</th>
<th>Compared to Average UCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>35E8600 4:45 pm</td>
<td>76.78</td>
<td>36.74</td>
<td>0.48</td>
<td>0.86</td>
<td>193</td>
<td>22%</td>
</tr>
<tr>
<td>35E8600 4:45 pm</td>
<td>76.78</td>
<td>32.47</td>
<td>0.42</td>
<td>0.52</td>
<td>133</td>
<td>15%</td>
</tr>
<tr>
<td>35E8600 4:45 pm</td>
<td>76.78</td>
<td>31.32</td>
<td>0.41</td>
<td>0.55</td>
<td>146</td>
<td>17%</td>
</tr>
<tr>
<td>35E8600 4:45 pm</td>
<td>76.78</td>
<td>28.14</td>
<td>0.37</td>
<td>0.51</td>
<td>152</td>
<td>17%</td>
</tr>
</tbody>
</table>

It can be seen that from the test data the 1/10th value proposed by Tesarik, Seymour, Martin and Jones (2007); Swan and Brummer (2001) and Pakalnis et al. (2005) is not valid in the case presented. The median values of average comparison of 18% show that the sill beams designs have an inherent conservative value for tensile strength used in calculation.

4.2.3 Stress-Strain and Post-peak Behaviour of Samples Under Axial Load

The typical stress-strain response of backfill, once hydrated, has been assumed to be Mohr-Coulomb behaviour with tensile strength equaling 1/10th and cohesion values of ¼ of the UCS as proposed by Brechtel et al. (1999), Caceres (2005); le Roux et al. (2005); and Pierce (1997). Strain-softening approaches have been discussed by Krauland and Stille (1993); Borgesson (1981); Roscoe et al. (1958); Swan and Board (1989); and Swan and Brummer (2001). Testing of samples to determine the stress-strain response is required for the appropriate constitutive behaviour that will be studied as part of this research.
To determine the stress-strain response two tests were performed: constant strain loading and hysteresis displacement test. These tests were performed on select Red Lake and Stillwater samples respectively. The data for both test sets will be presented separately.

For this research strain softening is referred to as behaviour where the shear resistance reduces with continuous development of plastic shear strains. This is represented graphically by a negative slope in stress-strain plot.

### 4.2.3.1 Red Lake Constant Strain Testing

As add-on to the investigation presented in Section 4.2.1.1, a select amount of tests were performed with strain-stress measurements; thirteen samples were tested with stress-strain responses. Figure 4-12, Figure 4-13, and Figure 4-14 show the results of the testing for (relative) low, mid and high strength samples tested. Solid lines represent axial strain measurements, dotted lines represent volumetric strain measurements.

Figure 4-12: Red Lake samples stress-strain paths (relative low strength)
Figure 4-13: Red Lake samples stress-strain paths (relative mid strength)

Figure 4-14: Red Lake samples stress-strain paths (relative high strength)
It can be seen that the typical response is a curvilinear stress path with a post-peak strain softening before the sample degrades completely. The vertical stress paths after the strain softening is due to full destruction of the sample and a trip in the recording software, these values are kept in the figures to show that the loss of strength after maximal strain is abrupt and goes to zero.

Young’s Modulus and Poisson’s ratio were calculated for each sample per ISRM standards (Ulusay and Hudson, 2007). It should be noted that these values are not representative of Red Lake paste on site. In addition, peak stress, peak axial strain, maximum residual axial strain and corresponding residual stress are presented in Table 4-28.

Table 4-28: Summary of Red Lake constant strain UCS tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>Peak Stress (kPa)</th>
<th>Peak Axial Strain ()</th>
<th>Young’s Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
<th>Maximum Axial Strain ()</th>
<th>Corresponding Axial Stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>86</td>
<td>0.019</td>
<td>8.8</td>
<td>0.085</td>
<td>0.025</td>
<td>38</td>
</tr>
<tr>
<td>2</td>
<td>54</td>
<td>0.012</td>
<td>5.2</td>
<td>0.641</td>
<td>0.012</td>
<td>53</td>
</tr>
<tr>
<td>3</td>
<td>102</td>
<td>0.024</td>
<td>4.5</td>
<td>0.309</td>
<td>0.026</td>
<td>96</td>
</tr>
<tr>
<td>4</td>
<td>57</td>
<td>0.015</td>
<td>3.9</td>
<td>0.145</td>
<td>0.018</td>
<td>49</td>
</tr>
<tr>
<td>5</td>
<td>214</td>
<td>0.011</td>
<td>35.6</td>
<td>0.040</td>
<td>0.014</td>
<td>193</td>
</tr>
<tr>
<td>6</td>
<td>257</td>
<td>0.009</td>
<td>40.4</td>
<td>0.115</td>
<td>0.015</td>
<td>182</td>
</tr>
<tr>
<td>7</td>
<td>253</td>
<td>0.015</td>
<td>67.7</td>
<td>0.045</td>
<td>0.018</td>
<td>195</td>
</tr>
<tr>
<td>8</td>
<td>237</td>
<td>0.016</td>
<td>30.2</td>
<td>0.025</td>
<td>0.033</td>
<td>163</td>
</tr>
<tr>
<td>9</td>
<td>184</td>
<td>0.017</td>
<td>93.2</td>
<td>0.008</td>
<td>0.029</td>
<td>116</td>
</tr>
<tr>
<td>10</td>
<td>343</td>
<td>0.015</td>
<td>80.9</td>
<td>0.013</td>
<td>0.019</td>
<td>305</td>
</tr>
<tr>
<td>11</td>
<td>562</td>
<td>0.002</td>
<td>792.1</td>
<td>0.034</td>
<td>0.006</td>
<td>484</td>
</tr>
<tr>
<td>12</td>
<td>369</td>
<td>0.004</td>
<td>29.0</td>
<td>0.356</td>
<td>0.011</td>
<td>177</td>
</tr>
<tr>
<td>13</td>
<td>271</td>
<td>0.012</td>
<td>26.7</td>
<td>0.105</td>
<td>0.013</td>
<td>246</td>
</tr>
</tbody>
</table>

It can be seen from the above figures and table that sample 11 is a clear outlier due to the high modulus readings due to an abnormal stress-strain plot; the calculation of the Young’s Modulus per Fairhurst and Hudson (1999) uses the slope of the stress-strain plot at 50% of ultimate stress. This shows a limitation of the Fairhurst and Hudson (1999) as, when applied mechanically, gives misleading values. Further Chapter 6 will that the assumption of a Mohr-Coulomb failure envelope (linear-elastic to failure; perfectly plastic, post-peak) is not valid. By using a Mohr-Coulomb failure envelope and the reported Young’s Modulus values, the peak
strain for the maximum stress will be under reported by 0.5%; over a typical 5 m wide stope. This accounts for failure at closure point 2.5 cm less than if a curvilinear stress-strain envelope, as demonstrated by the data set, is used.

The data presented above will be used in modeling the stress-strain response of backfill in subsequent chapters.

### 4.2.3.2 Stillwater Hysteresis Displacement Testing

Five Stillwater CPB samples were tested as per displacement profile outlined in Figure 3-2. Axial and circumferential strain measurements were recorded during testing with the anticipation that the results can be used to determine the constitutive behaviour of CPB. Three of the samples test results are suitable for analysis; two samples had erroneous readings due to slippage of extensometers due to observed early cracking of the sample during testing. The following figures show the performance of the paste backfill during hysteresis loading.

Figure 4-15: Hysteresis loading of Stillwater CPB sample (35E8600_145pm Sample)
Figure 4-16: Hysteresis loading of Stillwater CPB sample (35E8600_545pm Sample)

Solid lines (positive values) are axial strain measurement. Dotted lines (negative values) are volumetric strain.

Figure 4-17: Hysteresis loading of Stillwater CPB sample (35E8600_745pm Sample)

Solid lines (positive values) are axial strain measurement. Dotted lines (negative values) are volumetric strain.
The behaviour of the CPB at the peak of the loading steps is similar to those of the Red Lake samples. Again it is found that the loading pre-peak is curvilinear and the post-peak behaviour is not approximated by Mohr-Coulomb type stress-stain envelope. Table 4-29 summarizes pertinent data from the testing.

Table 4-29: Summary of Stillwater hysteresis loading tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>Peak Stress (kPa)</th>
<th>Peak Axial Strain ()</th>
<th>Young's Modulus (MPa)</th>
<th>Poisson's Ratio</th>
<th>Maximum Axial Strain ()</th>
<th>Corresponding Axial Stress (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>35E8600_145pm</td>
<td>760</td>
<td>0.0011</td>
<td>1080</td>
<td>0.081</td>
<td>0.0027</td>
<td>582</td>
</tr>
<tr>
<td>35E8600_545pm</td>
<td>1018</td>
<td>0.0029</td>
<td>408</td>
<td>0.379</td>
<td>0.0069</td>
<td>435</td>
</tr>
<tr>
<td>35E8600_745pm</td>
<td>915</td>
<td>0.0058</td>
<td>447</td>
<td>0.127</td>
<td>0.0093</td>
<td>774</td>
</tr>
</tbody>
</table>

At this point, from the data it can be concluded that a curvilinear envelope prior to peak stress exists and that a certain amount of strain softening occurs before a sudden drop in stress is experienced by a sample. Using the information presented in this subsection, analysis of the constitutive behaviour will be made to determine the correct approach to numerically modeling the CPB under unconfined axial load in a subsequent chapter.

4.3 In-situ Test Results and Analysis

In-situ testing of CPB is desired as it represents the field strength of the beam. Yu (1995) and Stone (1993) point out that the in-situ values of CRF should be less than the QA lab test values due to the effect of grain-size distribution; however, le Roux (2005) and Pierce (1997) suggest the strengths of the backfill is greater in-situ than in the lab. The research presented by Yilmaz et al. (2009) in which samples cured with an applied pressure, similar to those experienced in a long-hole stope, have higher strength than standard lab cured samples. From the literature available it would appear that higher strengths are achieved underground as opposed to the lab.

The issue with the available literature is that either experimental study or drilling of core at the production level is required: this is not conducive to the mining cycle. The following presents efforts to determine the in-situ strength of CPB through index testing and bulk sample
tests. The index relationships are established through Windsor Pin Method and the pneumatic pin method.

### 4.3.1 Windsor Pin Method

Windsor Pin testing was performed at the Red Lake Mine in 2004. This work was performed by Ms. Kathryn Clapp.

A strength relationship was experimentally established between the strength of material being used and the penetration depth of the embedded pin. Based on manufacture information (Windsor, 2004), a negative linear relationship between strength and penetration depth is expected.

The calibration method consisted of performing UCS tests and comparing those to the Windsor Pin test on the material. Surface UCS CPB samples were tested by Red Lake technical staff. Following the UCS test, seven Windsor Pin tests were done on each sample cylinder then measured with a micrometer. The results of the correlated UCS and Windsor Pin penetration depth are shown in Figure 4-18.

**Figure 4-18: Correlation of Windsor Pin penetration depth to UCS of Red Lake CPB**
The results fail to prove that a correlation exists between penetration depth and Red Lake CPB, the non-existence of a linear relationship is indicated by a Pearson-product moment calculation ($R^2$) value of 0.17.

After it was established that a relationship could not be proven, discussions with the manufacturer took place to determine the cause. The manufacturer indicated that the paste strengths below 2 MPa are either below or in the extreme lower range of what can be tested with the Windsor Pin instrument. This is an issue, as was determined from the investigation of testing databases, most CPB backfill falls between 0.5 MPa and 5 MPa strength.

Based on the results of the experimental calibration and the limitation of the Windsor Pin tester for low strengths, it can be concluded that Windsor Pin method is not suitable in providing an index relationship of in-situ CPB strength.

4.3.2 Pneumatic Pin Penetrometer Test

The shotcrete industry (Bracher, 2005) performs in-situ early strength testing of backfill using a Hilti gun test. The test performed by measuring the depth of embedment of a pneumatically charged stud projected into shotcrete. The depth of embedment is then correlated to a UCS value. The nature of the relationship is shown to a decaying function.

To determine if the Hilti gun test is applicable to CPB, two experiments were performed: one at the UBC NBK lab, the other at the Stillwater mine site.

4.3.2.1 NBK Lab Pin Penetrometer Tests

As part of the Macassa lab testing, CPB was mixed as per Table and set to cure in ore bins.

Hilti Tests were performed in 9 locations within each sample mix to determine penetration depth for each sample mix. After the penetration test, CPB samples were obtained using a core drill and subsequently UCS tested. The summary of the test results are found in Appendix 2.
The results of the test show that there is no real discernible trend within the tested CPB backfill. It was found that in all cases the depth of penetration exceeded the length of the penetration pin (75mm). According to Bracher (2005) this is not the standard case: pins should not be fully embedded. It should be noted that the lightest charged cartridges (Hilti white) were used as the explosive force for the pin penetration.

The testing at NBK showed that a considerable amount of scatter was found using the Hilti penetration pin test.

4.3.2.2 Stillwater Mine Pin Penetrometer Tests

Field tests of the penetration test were carried out at Stillwater Mine. Tests were performed on the back of CPB sill beams. The purpose of this testing was to determine if the penetration test was suitable in the field. In total 25 tests were performed, graphical representation of the data is shown in Figure 4-19: Hilti penetration test results at Stillwater. The issue with the testing is that the in-situ strength was not known as no coring of the back took place. As such, the 28 day strength and the modeled strength per Table 4-5 were used to give an indication of the strength in the back of the stopes.
The issues that were present in the UBC NBK tests are also present within the Stillwater database. There exists a large amount of scatter on what is thought to be something of uniform strength. Further, the pins were embedded within the paste.

It can be seen that index testing, by either the Windsor Pin or Penetration pin test was not successful in trials for the research. It is the opinion of the researcher that the surface of the paste is too soft to provide an adequate seat for the penetration test. This is an issue as the test is originally designed for shotcrete/concrete and not the comparatively soft CPB.

4.3.3 Unconfined Compressive Strength of Surface Samples vs. Underground Samples

Discussion on the validity of samples being tested at a surface laboratory in a controlled environment in comparison to CPB cured in underground environments is needed. Comparisons between surface cured samples and actual mining environment is discussed by Yilmaz et al. (2009). This research attempts to replicate field environments within the lab, not the true field
conditions. Stillwater Mine was concerned with the potential degradation of CPB during the transportation process and the environmental effects on curing conditions. As such, Stillwater began an investigation between surface sample strengths and underground strengths.

The paste backfill at Stillwater traverses the large strike length of the mine before being deposited in the stopes. There existed some concern regarding the difference in the paste strength between the paste plant and the underground stope. As such, Stillwater cast underground paste samples and paste plant samples to quantifying the difference between underground and paste plant properties. Hemlo’s David Bell Mine does both underground and plant backfill tests as well (Donovan et al., 2007).

Stillwater started collecting CPB samples both underground and in the paste plant on January 3, 2003 and continued with this process until March 2, 2009. After this date, testing only takes place underground.

The database included 86 comparative underground and paste plant tests; performing both underground and surface testing was not the normal procedure.

To compare underground and paste plant samples for the same stope (batch pour), unique values had to be assigned to paste samples based on the following:

- Date cast,
- Level,
- Stope,
- Cement content, and
- Age of cylinder at UCS test.

For stopes that had both underground and paste plant tests, the maximum, minimum and mean strengths for the samples were calculated and compared. In cases where the sample size for the underground or paste plant portion of the sample was greater than three (3) an additional weighted mean was calculated where the minimum and maximum were eliminated from a calculated average. This was done to reduce the effects of abnormal samples. The results are shown in Figure 4-20, where each data point signifies a stope that had comparable surface and paste plant samples.
It can be seen that a near one-to-one linear relationship exists with the paste plant samples being 1.42% stronger than the underground paste samples with a standard error of +/- 86 kPa. The Pearson Product Moment Correlation ($R^2$) for the data is 0.81 indicating a strong agreement. It should be noted that previous studies in Jordan et al. (2003) demonstrated that the underground paste samples were 20% stronger than the surface samples. The linear-regression shown in Figure 4-20 forces the regression through the origin. Jordan et al. (2003) performed the same study; however the data was not forced through the origin. This assumption by Jordan is misleading as it is not possible for a sample to have zero-strength on surface and strength underground or vice versa.

Based on the understandings that Stillwater cast the cylinders underground but cure the samples on surface, it can be concluded that there is no statistically significant difference between the samples prepared on surface to those prepared underground. That is to say, the strength of the material is not affected by the transport through the paste line system at Stillwater. This finding leads to the discussion that the gain in strength between underground
and surface strength is not caused by transportation, rather it is caused by the conditions. Investigation into cored strength of CPB is required.

### 4.3.4 Cored Paste Strength

The strength of cored paste samples is compared to the strength of the laboratory samples to determine the effect of curing conditions on paste strength. The cored samples were obtained from Red Lake. The samples were shipped to the UBC NBK rock lab where they were subsequently tested for their unconfined compressive strength.

Sixteen cored samples were tested. Unfortunately, no true comparison could be performed since the age, cement content and 28 day strength of the backfill pour was not provided by the Red Lake mine. The results of the cored strength tests found that the average strength of the backfill was 1200 kPa. The average tested values are below the 1500 kPa design strength at Red Lake.

Another data set existed for the Red Lake mine of cored samples tested by Lafarge Canada. Two paste blocks from underground were excavated and sent to Lafarge for UCS testing. In total sixteen samples were tested and found that the average paste strength was 4500 kPa. This is a three time strength gain compared to the design strength at Red Lake.

Although the in-situ values being less than the test values are counter to the findings of Yilmaz et al. (2009), it is not possible to draw any definitive conclusions without more information about the cored paste. Further testing of the cored strength of in-situ CPB vs. 28-day testing should be performed to determine the effect of curing conditions.

It would appear that from the investigation of in-situ methods, the correlation between moisture content and strength shown in Figure 4-7 and Figure 4-8 is the best option. The correlation should be performed in the laboratory and interpolation/extrapolation between the field moisture content and the in-situ strength can then be applied.
4.4 Chapter Summary

In summarizing the observational and experimental results, it was found that there exists a strong relationship between age of samples and strength of the samples, cement content and strength, and moisture content and strength. These relationships were shown to be statistically significant and can be used in estimating strength of CPB. However as shown in Table 4-30 and Table 4-31 relationships are site specific and cannot be applied from one mine to another. Relationships must be made for individual mines as part of the initial set up of CPB at a mine.

Table 4-30: Summary of strength of CPB based on age of sample

<table>
<thead>
<tr>
<th>Mine Site</th>
<th>Cement Content</th>
<th>Estimated Strength (kPa)</th>
<th>Standard Error (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red Lake</td>
<td>5%</td>
<td>139 ln (Age of Sample) -20</td>
<td>132</td>
</tr>
<tr>
<td>Red Lake</td>
<td>10%</td>
<td>354 ln (Age of Sample) -70</td>
<td>32</td>
</tr>
<tr>
<td>Red Lake</td>
<td>15%</td>
<td>596 ln (Age of Sample) -58</td>
<td>100</td>
</tr>
<tr>
<td>Macassa</td>
<td>3%</td>
<td>541 ln (Age of Sample) + 41</td>
<td>503</td>
</tr>
<tr>
<td>Macassa</td>
<td>5%</td>
<td>435 ln (Age of Sample) + 28</td>
<td>173</td>
</tr>
<tr>
<td>Macassa</td>
<td>7%</td>
<td>901 ln (Age of Sample) + 81</td>
<td>255</td>
</tr>
<tr>
<td>Macassa</td>
<td>10%</td>
<td>1,436 ln (Age of Sample) + 79</td>
<td>492</td>
</tr>
<tr>
<td>Stillwater</td>
<td>7%</td>
<td>99 ln (Age of Sample) + 122</td>
<td>57</td>
</tr>
<tr>
<td>Stillwater</td>
<td>10%</td>
<td>156 ln (Age of Sample) + 110</td>
<td>41</td>
</tr>
<tr>
<td>Stillwater</td>
<td>11%</td>
<td>360 ln (Age of Sample) + 29</td>
<td>152</td>
</tr>
<tr>
<td>Stillwater</td>
<td>12%</td>
<td>189 ln (Age of Sample) +212</td>
<td>47</td>
</tr>
</tbody>
</table>

Table 4-31: Strength of Sample based on cement content at certain ages

<table>
<thead>
<tr>
<th>Mine Site</th>
<th>Age of Sample</th>
<th>Estimated Strength (kPa)</th>
<th>Standard Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red Lake</td>
<td>3</td>
<td>48 (Cement %)-148</td>
<td>78</td>
</tr>
<tr>
<td>Red Lake</td>
<td>7</td>
<td>59 (Cement %)-240</td>
<td>44</td>
</tr>
<tr>
<td>Red Lake</td>
<td>28</td>
<td>155 (Cement %)-328</td>
<td>51</td>
</tr>
<tr>
<td>Macassa</td>
<td>14</td>
<td>412 (Cement %) -54</td>
<td>614</td>
</tr>
<tr>
<td>Macassa</td>
<td>28</td>
<td>448(Cement %)-199</td>
<td>652</td>
</tr>
<tr>
<td>Stillwater</td>
<td>3</td>
<td>44(Cement %)-61</td>
<td>52</td>
</tr>
<tr>
<td>Stillwater</td>
<td>7</td>
<td>55(Cement %)-14</td>
<td>134</td>
</tr>
<tr>
<td>Stillwater</td>
<td>14</td>
<td>76 (Cement %)-146</td>
<td>118</td>
</tr>
<tr>
<td>Stillwater</td>
<td>28</td>
<td>93 (Cement %)-140</td>
<td>144</td>
</tr>
</tbody>
</table>
From the experimental testing it was found that the strength of a sample was irrespective of the size of the sample: the variation in strength was the same no matter the size. Sample size of 5, 7.5, 10, 15 cm samples were tested (Appendix 2). The relative size of the particles of CPB in comparison to the test cylinder size is the reason for there being no strength difference between the different size of samples. What was most important of the experimental testing was the strain response of the CPB under uniaxial loading. It can be found that the material has a curvilinear strain path prior to failure followed by strain softening, followed by full disintegration of the sample. This finding will be used when determining the design strength of sill beams.

The experimental and observational approach proves that by studying the cemented paste backfill test results, efficiency in cement content, length of time required for design strength to be reached can be achieved. With the cement content and strength gain being site specific, guidelines in design will be written such that these values and relationships need to be tested and implemented to understand the strength of CPB sill beams. Further, the laboratory studies show that the stress path of the material is important as CPB can sustain larger amount of strain prior to failure as opposed to the standard Mohr-Coulomb failure. This will have a large impact on design guidelines as it implies the stiffness of the material is as important to the strength of the material when dealing with stopes with large closure.

The observational and experimental chapter aimed to address the constitutive behaviour of paste, determine relationships for estimating strength and measuring in-situ strength through cost-effective measures. To state simply, we now have an understanding of CPB strength and behaviour under axial load. These findings coupled with analytical, numerical and empirical observations will create the design guidelines.

**Chapter takeaways**

- CPB under axial load follows a hyperbolic axial strain path on way to peak load;
- Size of sample does not affect UCS strength;
- For Stillwater CPB, tension as a percentage of the UCS is between 15 and 22%;
- Moisture content, slump value and pulp density can be used as an index test for UCS strength; and
- Strength vs. age and cement content vs. age relationships are valid but are site specific for CPB.
5 UNDERHAND CEMENTED PASTE BACKFILL CUT-AND-FILL MINES

Mining underneath backfill is common in mines with diminishing pillars, dwindling reserves, high stress and weak ground (DeSouza et al., 2003). Operations that mine under backfill are summarized in Pakalnis et al. (2005). This chapter delves deeper into the types of mines that utilize UCF CPB mining. To do this a case study approach was adopted for the Red Lake, Lucky Friday, Stillwater and Kencana mines. These four mines were selected as they represent the range of mine conditions that are conducive to UCF mining. For each mine, a summary of the operation, the geotechnical setting, CPB designs, span widths, ground support and design guidelines are presented.

5.1 Red Lake

Red Lake mine is a narrow vein, high grade, competent rock with risk of rock burst that employs UCF CPB mining. Red Lake has been mining under paste since 2002. The summary of Red Lake mine is presented by Mah et al. (2003), Kumar (2003), and Hughes (2008).

Historically Red Lake has used various mining methods including mechanized cut and fill, double level captive cut and fill stoping, and long hole sill removal (Mah et al., 2003). Red Lake currently utilizes two mining methods: primarily mechanized cut and fill (“MCF”) and occasional long hole stoping. MCF mining commenced below the 31 level at Red Lake Mine in an effort to optimize mining the high-grade gold and mitigates the risks of the high stresses developed through mining a diminishing sill pillar.

UCF is utilized at Red Lake in an effort to extract ore under a weak or highly stressed back. A sill beam is prepped for the initial cut, and each successive cut is developed below with the construction of a new sill beam. Stope widths vary between 5 m and 10 m, however it is noted that in the larger spans only 7 m of the span consists of paste, with the larger spans consisting of rock/paste backs. Heights of stopes vary with maximums observed to be 5 m. The use of underhand stopes is planned to increase at Red Lake due to the focus of mining within the High Grade-Zone below the 38 Level of the mine (greater than 2100 m below surface).
Red Lake is planning on increasing production from the High-Grade-Zone (HGZ). The HGZ expansion will increase the tonnage obtained from underhand cut and fill mining as the ore body geometry is more conducive to UCF than the current method. Currently the mining method in the HGZ consists of development of overhand and underhand stopes along strike from a production level. Mining continues from the production level with both underhand and overhand stopes being developed and backfilled with cemented paste backfill (CPB) until the diminishing sill pillar between successive levels reaches a predetermined thickness. The thickness of the sill pillar is determined from in-situ stress, induced stress and monitored seismic activity. Once the sill pillar reaches the engineered thickness, a de-stress slot is blasted to create a low-stress mining environment, thereby reducing the risk of over-stressing and rock burst potential. Figure 5-1 demonstrates the development of the de-stress slot in sill pillars.

Figure 5-1: Schematic of de-stress slot

![Schematic of de-stress slot](image)

RLGM is currently studying optimizing the mining method in the HGZ. Due to change in the thickness (footwall to hangingwall), ore body dip and the strike length of the HGZ orebody, current practices listed above are not optimized to the development. Increase in underhand span widths of the CPB and the amount of stopes within the de-stress slot are to be reduced to optimize the mining in the HGZ.
The original CPB sill span is designed for 6 m wide headings based on design strength of 1.5 MPa backfill. Current discussions are that the maximum CPB sill span be increased between 7.5 m and 9 m.

With the change in the orebody dip and size in the lower levels of the mine, it is planned that de-stressing of the sill pillar will occur after further underhand mining progresses; resulting in the mining of stopes in higher than current induced stresses.

This mining in a high stress environment requires that the CPB perform in more challenging conditions. The ability to withstand closure and seismic events will need to be incorporated into the design of the backfill.

Typical RMR ratings for the rock are 65-75%. Stress gradients are represented in Table 5-1. Mining depths under paste vary from 2000 to 3000 m.

<table>
<thead>
<tr>
<th>Principal Stress</th>
<th>Direction</th>
<th>Gradient (MPa/m of depth)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_V$</td>
<td>Vertical</td>
<td>0.029</td>
</tr>
<tr>
<td>$\sigma_{H(min)}$</td>
<td>Parallel to strike of orebody</td>
<td>0.0276+ 3.64/depth</td>
</tr>
<tr>
<td>$\sigma_{H(Max)}$</td>
<td>Perpendicular to strike of orebody</td>
<td>0.042+8.18/depth</td>
</tr>
</tbody>
</table>

5.1.1 Backfill Strengths

The original design strength for Red Lake mine was 2.0 MPa. Modifications to the paste recipes and cement content have changed the design (target) of 28-day, UCF of surface cured CPB samples are now 1.5 MPa. Details on the Paste Backfill strength at Red Lake are provided within Chapter 4.1 and 4.2.

Entry times for backfill are dictated by time and not strength of fill. The timeline is based on strength vs. time gain, verified through laboratory testing. Table 5-2 summarizes the guidelines at Red Lake for exposing a vertical face, or for mining within/underneath the CPB.
Investigating Figure 4-3, the 7 day strength of 15% CPB at Red Lake is, on average, approximately 950 kPa.

Table 5-2: Guidelines for entry at Red Lake

<table>
<thead>
<tr>
<th>Cement content (%)</th>
<th>CPB location</th>
<th>Minimum time for exposure of vertical face</th>
<th>Minimum entry time for mining within/underneath</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Sidewall</td>
<td>48 Hr.</td>
<td>7 Days</td>
</tr>
<tr>
<td>9</td>
<td>Sidewall</td>
<td>24 Hr.</td>
<td>3 Day</td>
</tr>
<tr>
<td>12</td>
<td>Sidewall</td>
<td>12 Hr.</td>
<td>1 Day</td>
</tr>
<tr>
<td>12</td>
<td>Back (UCF)</td>
<td>N/A</td>
<td>18 days</td>
</tr>
<tr>
<td>15</td>
<td>Sidewall</td>
<td>12 Hr.</td>
<td>1 Day</td>
</tr>
<tr>
<td>15</td>
<td>Back (UCF)</td>
<td>N/A</td>
<td>7 Days</td>
</tr>
</tbody>
</table>

5.1.2 Span Width

With the use of mechanized cut and fill the widths of the stopes has a minimum limit prescribed by the machines. It was noted that the stope widths were at a minimum of 4.5 m. The maximum allowable span without special consideration of ground support under an all CPB back is 6.1 m, with vertical side walls.

With the expansion of the mine into the HGZ zone, the ore body is becoming more irregular in shape. This is resulting in larger spans than the current span widths at other areas of the mine. Currently, due to the shallow dip of the ore-body, the back in the UCF is composite between CPB and rock. The limit at Red Lake is 9.6 m with 6.1 m of CPB and 3.5 m of rock overhang.

5.1.3 Minimum Ground Support

The ground support for UCF mining at Red Lake is a two stage process: the installation of sill beams, followed by installation of ground support on advance.

The sill beam preparation consists of the use of stand-up rebar and shear paddles installed in the wall, shown in Figure 5-2. Rebar installed is 1.65 m long #6 rebar on a 1.2 m by 1.2 m pattern. Shear paddles are installed in the hangingwall of the stope to prevent against rotational
failure. The shear paddles consist of fully encapsulated 1.65 m long, #6 rebar, installed 0.9 m into sound rock, 75 mm wide straps are tied to the paddles. The bolts are installed 0.9 m above the working floor.

Figure 5-2: Sill mat preparation

The ground support on advance consists of 1.8 m long, Roc-set R39 Split Sets on 0.9 m x 0.9 m pattern. 6 gage welded wire mesh screen is installed in conjunction with the tendon support to contain small failures between the tendon supports. Within the wall and corners, and in the case of overhangs, support within the sound rock consists of fully encapsulated resin #6 rebar on a dice-five pattern spaced 1.5 m pattern. The length of the rebar bolt is dependent on the width of the span: for rock spans less than 1.5 m, 1.65 m length bolts are employed; rock spans greater than 1.5 m, 2.1 m length bolts are used. In high seismic zones the use of modified cone bolts augments the rebar support within the solid rock.

Red Lake has performed bond strength testing on the split sets within the CPB. The tests were performed such that the bolt hole was fully drilled, then over drilled such that the split set was embedded only within 0.6 m of the CPB. Performing tests in this fashion provide a usable
value for bond strength, and not the pull out value of the entire bolt. The average results of the pull out tests on six (6) SS 39 indicate bond strengths value of 1.36 tonnes/foot; or 4.46 tonnes per meter. The bond strengths within paste fill are comparative to those in hard rock (Pakalnis et al., 2007). It is likely the reason for this behaviour is the pushing of the split set into the paste, filling the annulus of the bolt and increasing its frictional resistance through increased confinement.

In addition to the friction bolt testing, Red Lake performed pull-out tests on resin-rebar bolts. In total eighteen (18) bolts were tested for bond-strength values. To perform the test, 1.65 m long, 20mm diameter, 400 MPa rebar bolt placed in a 1.2 m long, 33mm diameter borehole. 60 cm of Fasloc resin (3/4 30 min type) was placed at bottom of the borehole and the remainder of the borehole was filled within inert resin: the bolt was only grouted over 60 cm of its length. Figure 5-3 show the applied load vs. displacement of twelve of the eighteen pull tests (six pull test measured load only). It can be seen that uniformly that the bolt behave linearly over the first 1.5 mm before beginning to yield, two bolt pulled out completely during testing. Analysis of the testing showed average bond strength per 30 cm (approximately 1 foot) to be 4.85 tonnes; minimum and maximum values were 2.75 tones and 9.0 tonnes respectively.
5.1.4 Design Guidelines

The design guidelines for Red Lake were based on analytical work using Mitchell and Roettger (1989) beam mechanics and numerical modeling performed by Itasca Consulting Group. The research on the design guidelines was performed over two phases of design between 2000 and 2004. This work was done by outside consultants Itasca Consulting Group, Pakalnis & Associates and Wilson Blake. The findings below are based on the final consulting report prepared by Pakalnis & Associates (Pakalnis and Associates, 2000 & Pakalnis and Associates, 2004).

The initial analytical design was prepared by Pakalnis and Associates (2000) and the approach was based on Mitchell and Roettger (1989) research. Minimum strength requirements for various beam widths were analyzed based on a 3.1 m high stope height. It was found that rotational failure was critical to stability, due to the dip of the stope walls. 2 MPa (10% cement) CPB was suitable in preventing rotational stability in spans up to 4.6 m.
The use of the sill mat (stand-up rebar bolts) was based on empirical research of success at Lucky Friday. There is no analytical justification for the stand-up rebar bolts in archived reports between 2000 and 2012. It is thought that the sill mat design would contain cold joints prior to the installation of on-advance support.

The capacity of the supported paste to remain stable in a seismic event was assessed below by Itasca, but the empirical approach was determined by Dr. Wilson Blake:

“North American experience mining under paste fill indicates that back instability resulting from a rock burst is extremely rare. …… show that a 1.0 MPa strength paste fill is stable without support to spans over 20 ft. The fill support recommendations …… are, therefore, conservative. The fill support of chain-link on the floor, the shotcrete and the bolting through the shotcrete with Swellex will exceed the 8.1 kJ/m² dynamic loading resistance provided by the standard support in the overhand stopes”.


Initial numerical modeling performed by Itasca verified in-situ field measurements of earth pressure cells (EPC) placed along the HW/Paste/FW contacts. The numerical modeling was performed with Itasca’s FLAC2D (Itasca, 2012). Using the field measurements, attempts were made to synthesize the Young’s Modulus and Poisson’s Ratio of host rock and the CPB. Back analysis was used to vary the Young’s Modulus and the Poisson Ratio of the host rock to reflect the field stresses; whereas, the values used for the CPB were from Itasca’s in-house database. Table 5-3 summarizes the values used for the analysis.

Table 5-3: Material properties used in design guideline (Pakalnis and Associates, 2004)

<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s Modulus (GPa)</th>
<th>Poisson’s Ratio ()</th>
<th>Shear Modulus (GPa)</th>
<th>Bulk Modulus (GPa)</th>
<th>Density (kg/m3)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPB</td>
<td>0.9</td>
<td>0.3</td>
<td>0.346</td>
<td>0.75</td>
<td>2.06</td>
<td>Itasca Database</td>
</tr>
<tr>
<td>Host Rock</td>
<td>42</td>
<td>0.25</td>
<td>16.8</td>
<td>28</td>
<td>2.93</td>
<td>Back analysis</td>
</tr>
</tbody>
</table>

It was found from investigation that the purpose of the back analysis was to determine the host rock properties.

Once the back analysis was complete, numerical investigation was performed to determine the stability of the sill beam, and the lowest UCS necessary for a stable sill beam. The
first model was to determine the static and dynamic stability of mining of the 38-746-1 for the
first six undercuts. This stope was selected since it had the shallowest ore geometry (45°) and
would be designed with a 60° stope wall having a 3.4m cut height. The result of this stope
configuration would represent the ‘worst-case’ scenario of a 1.4m rock undercut and a 6.1m span
under paste, for an effective span of 7.5m. Rotational, flexural and compressive failures were
considered possible modes of failure for the numerical model. A parametric analysis was
performed with the following being analyzed:

- Effect of cohesion varying from 100kPa to 500kPa;
- A 0.3m(1ft) air gap with/without on top of fill;
- Effect of large convergence;
- Static/Dynamic influences employing 1.5/2Mn event; and
- Support under dynamic loading.

The results of the numerical modeling demonstrate that approximately 10 mm of closure
occurs for each lift; resulting in a cumulative closure of 82 mm after six cuts. Numerical
modelling was performed to determine the minimum amount of closure required for failure of
the sill beam was 320 mm. From this finding it was determined that “closure is not a concern as
actual deformations are much less than critical based upon observation, monitoring (albeit
limited) and modelled results” (Pakalnis and Associates, 2004). The presence of an air gap
between successive lifts was found to have no effect on stability.

The varying of the cohesion properties was performed to determine the required strength
of backfill. It was determined that for static analysis, a 150 kPa or greater cohesion satisfied
numerical stability; 250 kPa was required to satisfied stability criteria for a local seismic event
that results in a downward ppv of 800 mm/s @ 20Hz (Pakalnis and Associates, 2004, p.9).

It was determined from the analysis of ground support that the stand-up rebar provided
limited additional support. Further, although the horizontal rebar ‘paddle’ support provides an
additional 5 tonnes of resistance, it was found that they were not critical to overall stability. An
angled rebar that penetrated the CPB and rock within the hangingwall was found to provide
additional support to prevent sliding.

The overall findings and recommendation of the numerical analysis were as follows:
• Analytical design is suitable;
• Angled support is preferred but shear paddles will perform adequately;
• Effect of convergence is minimal giving overall strength of fill;
• Tight fill in the back is not essential;
• Critical design UCS was found to 1 MPa, in-situ strengths are much higher, and sill beam is stable as designed.

5.2 Lucky Friday

Lucky Friday was selected for a case study as it was one of the first mines to implement UCF mining with CPB for diminishing pillars in high stress weak rock environments.

Lucky Friday is a deep underground silver, lead, and zinc mine located in the Coeur d’Alene Mining District in northern Idaho. The mine has been in operation since 1942 producing 88 million ounces of silver as of 1988 (Noyes, Johnson, Lautenshcalger, 1988). In 1984 underhand cut and fill mining was adopted to counter the issue of rock bursts in diminishing pillars. Ramps were developed in the footwall and mobile equipment was introduced with this mining method switch. Currently all production is from underhand longwall stopes, nominally 170 m long (Hedley, 1993).

Stope cuts are typically between 3 and 3.7 m high and backfilled with CPB to within 0.6 m-1.0m of the back with ore width between 2 and 3 m wide.

Lucky Friday average rock mass is between 45 and 65% by RMR ratings. The stress gradient at the mine is shown in Table 5-4. Mining occurs at depth below 1500 meters, with recent development occurring 2300 m below surface (Hecla, 2012).

<table>
<thead>
<tr>
<th>Principal Stress</th>
<th>Direction (trend --&gt; Plunge)</th>
<th>Gradient (kPa/m of depth)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1$</td>
<td>320--&gt;13</td>
<td>70</td>
</tr>
<tr>
<td>$\sigma_2$</td>
<td>221 --&gt; 33</td>
<td>42</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>068--&gt;54</td>
<td>29</td>
</tr>
</tbody>
</table>
5.2.1 Backfill Strengths

The backfill at Lucky Friday has been tested by Hedley (1993); Williams et al. (2007). Williams reports that the Lucky Friday Mine uses a tailing paste fill with no free water, and 8–10% cement that develops a UCS strength of 2070 kPa after curing for seven days. Hedley (1993) reports that the paste has a bulk density of 1922 kg/m3; compressive strength of 2.74 MPa, and a deformation modulus of 1.37 GPa after 28 days.

5.2.2 Span Width

The span width at Lucky Friday is narrow in comparison to other mines. Mining widths are between 2 and 3 m wide. Based on layouts of stopes and access, intersection spans of 9 m are possible (Williams et al., 2001).

5.2.3 Minimum Ground Support

The Lucky Friday mine rarely installs rock bolts for ground support in the back of underhand stopes once the sill beam is undercut. The Lucky Friday fill preparation is shown in Figure 5-4. The 1.8 m Dywidag bolts are driven into the prep muck on the floor with a steel fence post type driver. Bolts are spaced no more than 1.2 m apart in any direction. Dywidag bolts are used because the protruding 0.3 m of bolt under the fill is often bent or hit by fly rock, which would ruin the threads on other types of bolts. To maintain good support in the crown of the cut below, the bolts are placed as close to the ribs as is practical. Plates and nuts are attached to the tops of the bolts and wire is attached to one wall and run along a row of bolts, wrapping around each bolt just under the plate, and attached to the other wall so that the bolts stay vertical as the fill is poured around them. The top plate anchors the bolt in the fill. This fill preparation is used for stopes up to 7.5 m in width (Blake, 2002).

Further support to the undercut is provided by installing shear paddles consisting of two rows of steel mats oriented horizontally along the competent wall and chain link along the weaker south wall. The horizontal paddles are anchored to the wall with 1.2 m split-set bolts and 15 x 15cm bearing plates spaced on 0.9 m centers. On occasion, the back of the stope will be spot bolted with additional split-sets, depending on the width of the stope and the bolt pattern used in the fill preparation (Williams et al., 2007).
For stopes that exceed 7.5m width, additional support is installed in the sill beam: 25 cm x 25 cm x 4.8 m long timbers are installed on 2.4 m centers. The timbers are normally placed in the center of the stope, and are never placed close to the walls to prevent stope closure squeeze on the timber. Holes are drilled in the timbers so that five Dywidag bolts can be inserted through them and plated. These timbers are tied down to prevent floating during the paste pour. As mining progress underneath, these timbers are not supported with vertical timber unless observed that to be taking load (Blake, 2002).

5.2.4 Design Guidelines

The analytical design of the Lucky Friday sill mat is not published. However, numerical modeling and extensive instrumentation of the sill mat has been published. The available literature does not present a justified case for the stand-up rebar sill mat, as opposed to supporting on advance.

However, Williams et al. (2001) and Whyatt, Williams and Blake (1995) discuss the suitability of backfill for mining at Lucky Friday. The design guidelines at Lucky Friday was to provide an engineered paste back that would reduce stresses directly at the back. It was hoped that the paste fill would “synthesize the rock mass,” replacing the rock with a less burst prone material. Consideration of beam mechanics was not discussed. Rather, the performance of the
fill was numerically modeled (Whyatt et al., 1995) with the amount of closure and stresses within the fill predicted. Williams et al. (2001) then performed monitoring of the stope to verify the behaviour predicted by the model. The design guidelines at Lucky Friday are based on an observational approach to engineering: model expected behaviour, analyze the results of model, instrumentation and monitoring of the stope, and verify model predictions to ensure stability.

5.3 Stillwater Nye Operation

Stillwater Mine was selected as a case study due to the weak rock mass, low-burst prone rock and high dilatant properties of the rock. Convergence of the stopes is the largest issue due to the large stope heights and dilative properties of the rock.

From Hughes et al. (2013, p. 1):

“Stillwater Mine (Montana) commenced mining Platinum Group Metals (PGMs) ore in the mid 1980’s. Several mining methods are employed which are dictated on the ore grade, layout and location of the ore. Underhand cut and Fill underneath cemented paste backfill (CPB) has been successfully used in the lower reaches of the mines since 2001.....”

“Approximately 80,000 to 90,000 Reef tons per year, or 10% of the total Reef tons, are mined under CPB at the Stillwater Mine. In the earlier days of underhand cut and fill production mining, expected productivity gains, reduced mining costs, and the anticipation of poor ground conditions were the primary drivers behind decisions made to mine under CPB. Significant productivity gains did not occur; therefore in recent years, decisions to mine under CPB have been made solely on the existence of poor ground conditions in the Reef orebody.

Stillwater Mine employs the Q System for evaluating ground conditions. When Q values fall below approximately 0.4 throughout a planned stoping block, the decision is made to mine the given stope block utilizing the underhand cut and fill methodology in most cases.

The underground workings at Stillwater Mine extend to the west of the Stillwater River Valley for nearly 6100 m and to the east 3000 m. The river valley is positioned at 1525 m elevation. Ten mine levels are positioned below the river valley, while nine mine levels are positioned above the river valley within the mountain to the west and east. Underhand cut and fill mining is primarily located off of levels below the river valley at depths from surface of 1000 m to 1700 m, and within lateral distances to the east and west of the river valley of 1500 m to 3000 m. These areas of the J-M Reef are in many locations composed of sheared dunites and other heavily faulted rock types. This feature, coupled with stress and strain fractured rock in some locations, creates the ground conditions with which underhand cut and fill methods are most suitable.
Long range plans at the Stillwater Mine include establishing new mining levels at elevations of 520 m and 430 m, as well as advancing existing levels to the east through the areas of the J-M Reef which have proven more fitting to underhand cut and fill mining methods. With this being said, it is expected that cut and fill mining under CPB will be utilized for the foreseeable future at Stillwater Mine. However, cut and fill mining under CPB is not expected to increase as a percentage of overall total Reef tons mined at Stillwater Mine.”

Ground stresses at the mine were measured by Johnson, Brady, MacLaughlin, Langston, and Kirstens (2003) and are summarized in Table 5-5.

<table>
<thead>
<tr>
<th>Principal Stress</th>
<th>Direction (trend --&gt; Plunge)</th>
<th>Gradient (kPa/m of depth)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1$</td>
<td>000--&gt; 00</td>
<td>57</td>
</tr>
<tr>
<td>$\sigma_2$</td>
<td>090--&gt; 00</td>
<td>45</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>000--&gt; 90</td>
<td>30</td>
</tr>
</tbody>
</table>

5.3.1 Backfill Strengths

A thorough Quality Assurance/Quality Control (QA/QC) program on backfill strength is currently performed at Stillwater. Since the commencement of underhand mining at Stillwater 17,540 UCS samples have been tested to insure that the minimum design strength of 1 MPa (145 psi) is achieved. The details of the backfill strength at Stillwater are presented in Sections 4.1 and 4.2.

5.3.2 Span Width

As part of this research, an empirical study was investigated to determine all the underhand span widths that have been mined at Stillwater. The span widths were measured from surveyed plans of the stope. Both the inscribed circle and largest perpendicular span were measured for each surveyed plan. In addition to the span widths, the 28-day strength from the database discussed in Chapter 4 were coupled with the span widths.
As indicated in Figure 5-6, it can be seen that the majority of the stopes mined at Stillwater are between 4 and 5 meters with respect to perpendicular span. It was noted during the
analysis, that the larger spans occur at intersections and, typically the wide perpendicular spans occur when HW or FW areas are slashed out for grade purposes.

Further investigation Figure 5-5 demonstrates that stable inscribed spans of 10 m are possible with backfill with strength of 1500 kPa. The spans at Stillwater include standard ground support discussed within. The conclusion that can be reached from Figure 5-5 is that it represents a stability chart for Stillwater based on strength of backfill as no failures occur within the reported data. However, the Factor of Safety of the points in the figure is not considered and the use of the chart should be applied with proper engineering judgment.

5.3.3 Minimum Ground Support

The ground support at Stillwater for UCF stope is reported by Jordan et al. (2003) and was observed during a site visit in August, 2012. The support is placed typically after the stope is under cut; no stand-up bolts are installed as of August 2012. To protect the CPB from blast damage, 30 to 45 cm of blast rock is placed on the floor prior to placement of CPB.

Previously, as stated by Jordan et al. (2003), Stillwater placed stand-up 1.8 m rebar, in the stope prior to placement of fill. The stand-up rebar is placed on 1.2 m across the width of the stope and 1.8 m along the length of the stope. In addition, channel iron (0.15 m high by 0.05 m wide) was placed across the stope every 1.8 m for spans greater than 4 m in width. These two ground support methods are not currently practiced at the mine. 11 gauge, welded wire mesh is used to contain small raveling failures between the frictional support tendons.

The current ground support within underhand CPB consists of 1.8 m long, 39 mm diameter split-sets on a 0.9 m by 0.9 m pattern. In intersections where the inscribed circle span exceeds 6 m or wider, the installed support consists of 2.4 m rebar (#7) bolts on a 0.9 m by 0.9 pattern; and 3.6 m (#7) rebar bolts on a 1.8 m by 1.8 m pattern augment the standard friction bolt support. In cases where there is concern with the strength of the backfill or when dictated by ground support a maximum of 50 mm of shotcrete is placed on over the CPB.
5.3.4 Design Guidelines

The design guidelines at Stillwater are referred to as the Kirsten design. The guidelines are described in detail by Jordan et al. (2003). The main principle of the design guidelines is to prevent flexural failure of the beam. This failure, referred to as tensile failure, is the most common and requiring the highest strength beam to prevent failure, as such it is used as the critical design criteria. Solid mechanic beam equations are used to solve for the necessary strength of the beam assuming 1/10th beam tensile strength to compressive strength ratio (Jordan et al., 2004, p. 8). An inherent conservatism is used in the design as the beam is assumed to be pin supported and not, as it the fact of actual conditions, a simply supported beam. Support is not considered in the critical design strength calculations. In addition to this conservative approach to the design equations, a Factor of Safety of 1.5 is used for design once the critical design. Equation 5-1 is the critical design equation for Stillwater Mine.

\[
\sigma_t = \frac{3 \gamma L^2}{4 \ t}
\]

where:

- \( \sigma_t \) = tensile strength (kPa)
- \( \gamma \) = material density (kN/m\(^3\))
- \( L \) = length of beam (m)
- \( t \) = height of beam (m)

5.4 Kencana

The Kencana mine is a weak rock mine with high wall convergence. CPB UCF was selected as the mining method as the CPB represents a more stable back than the host rock.

The Kencana mine (Gosowong Mine) is a high grade gold mine located on the Halmahera Island in Indonesia. The mine is owned by PT Nusa Halmahera Minerals, of which Newcrest mining has a 75% controlling interest. The mine is selected as a case study as Kencana is a weak rock mass environment with a large amount of ground closure present.
The Kencana ore body is a high grade, epithermal gold deposit 125 meters below the surface with a strike length of 400 m, dipping at 45° and an average orebody thickness of 12 m. The ore body contains 2,000,000 ounces of resources. The total mining rate is approximately 1,200 tonnes per day (Febrian et al., 2007).

UCF mining with CPB is used as the predominant mining method. Occasional long hole stopes are employed when fair to good ground conditions are present. The UCF mining method was selected due to the high recovery, HW control, the high selectivity of the method, and it limits the exposed span (Febrian et al. 2007).

In general, conditions associated with the orebody range from RMR values of 65 located in the footwall, to highly weather ore with RMR of 25. Ground stresses are low due to the proximity of the orebody to surface; stresses are summarized in Table 5-6.

<table>
<thead>
<tr>
<th>Principal Stress</th>
<th>Direction</th>
<th>Gradient (kPa/m of depth)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1$</td>
<td>Vertical</td>
<td>27</td>
</tr>
<tr>
<td>$\sigma_2$</td>
<td>Parallel to strike</td>
<td>9</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>Perpendicular to strike</td>
<td>9</td>
</tr>
</tbody>
</table>

5.4.1 Backfill Strengths

The backfill at Kencana is a made from a mined tuff that is processed at a screening plant and is mixed with between 10 and 16 percent cement depending on cure time requirements. The average strength of the material is 2.5 MPa. The tails at the mine are not conducive to paste backfill due to the highly weathered state of the ore body.

As part of this research, the QA/QC procedures for paste strength testing at Macassa mine were developed in conjunction with the technical services staff at the mine. The procedures for the testing are listed in Appendix 3.
5.4.2 Span Width

The span width at Kencana is governed by geology; where ore exists stopes will be slashed out or a parallel drift will be drifted alongside the cemented paste. Span widths under paste are between 4.5 and 7 m. Where dictated by ore span widths spans up to 13 m can be opened up with cable bolt support.

The issue with the spans at Kencana is due to the weak rock mass: the paste is more competent than the ore body. This causes issues in the support of the back as additional ground support is required in the back and sidewalls of the weak ore.

5.4.3 Minimum Ground Support

A considerable amount of ground support is employed at Kencana to maintain a stable back. Support during stope preparation and on-advance is incorporated into the ground control program.

As far as sill mat support at Kencana, the design is based on Lucky Friday. The stope preparation sill mat support consists of a 1.2 m x 1.2 m #6 rebar, standing vertical. Shear paddles in the hangingwall consisting of 2.4 m long bolts for 4.3 m lift heights and 3 m long bolts for 5 m lift heights and intersections. The shear paddles are to be located 1 m above the working floor and drilled at +10°. The embedment depth of the rebar is dictated by the critical depth of embedment based on bond-strength for the rock type (nominally 1.4 m).

For ground support in UCF stopes, nine distinct support classes were designed. Based on rock type, three types of ground support were as follows: Type I for RMR > 55%; Type II for RMR between 35 and 55%; and Type III for RMR between 25 and 35%. Once the support for the rock type has been established, the support is then differentiated based on the dip of the footwall: Support (A) for HW dip of 70°; Support (B) for HW of 90°; and Support (C) for HW dip of 45° (see Figure 5-7).
Table 5-7 shows the support in the rock in the undercut, the support in the CPB consists of the sill mat described above for Kencana. Wall support refers to the support in the wall of the filled stope.

Table 5-7: Kencana minimum ground support for walls in underhand support

<table>
<thead>
<tr>
<th></th>
<th>Support A</th>
<th></th>
<th>Support B</th>
<th></th>
<th>Support C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>HW Dip = 70°</td>
<td></td>
<td>HW Dip = 90°</td>
<td></td>
<td>HW Dip = 45°</td>
</tr>
<tr>
<td>Back</td>
<td>Wall</td>
<td>Back</td>
<td>Wall</td>
<td>Back</td>
<td>Wall</td>
</tr>
<tr>
<td>Bolt Type</td>
<td>SS 47</td>
<td>SS 47 (grouted)</td>
<td>SS 47</td>
<td>SS 47 (grouted)</td>
<td>SS 47</td>
</tr>
<tr>
<td>Pattern (m x m)</td>
<td>1.2 x 1.2</td>
<td>1.5 x 1.5</td>
<td>1.0 x 1.0</td>
<td>1.0 x 1.0</td>
<td>1.0 x 1.0</td>
</tr>
<tr>
<td>Bolt Length (m)</td>
<td>3.0</td>
<td>2.4</td>
<td>3.0</td>
<td>2.4</td>
<td>3.0</td>
</tr>
<tr>
<td>Welded Wire Mesh</td>
<td>No</td>
<td># 6 Gauge</td>
<td>No</td>
<td># 6 Gauge</td>
<td>No</td>
</tr>
<tr>
<td>Shotcrete thickness (mm)</td>
<td>50 (standard)</td>
<td>N/A</td>
<td>50 (Fibercrete)</td>
<td>75 (Fibercrete)</td>
<td>100 (Fibercrete)</td>
</tr>
</tbody>
</table>

Bond strength of the 47mm diameter split sets in the weak rock are as shown in Table 5-8. Split sets are not installed within CPB at Kencana: the sill mat is supported by stand-up rebar.
Table 5-8: Bond strength for 47 mm split sets at Kencana mine

<table>
<thead>
<tr>
<th>RMR</th>
<th>Ungrouted</th>
<th>Grouted</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 55%</td>
<td>4.1</td>
<td>7.2</td>
</tr>
<tr>
<td>35 - 55%</td>
<td>2.6</td>
<td>5.8</td>
</tr>
<tr>
<td>25 - 35%</td>
<td>1.5</td>
<td>4.4</td>
</tr>
</tbody>
</table>

5.4.4 Design Guidelines

The design of the sill mat at Kencana incorporated design approaches from Red Lake mine and numerical modeling specific to the mine site. The design guideline researched the stability of the sill beam and the host rock overhangs as the overhang was identified as a possible weak point of a beam.

The empirical design investigated the required strength of the sill beam based on beam theory mechanics (Obert and Duval, 1967) and span widths vs. UCS at other operations.

Figure 5-8 (Pakalnis and Associates, 2006) illustrates the empirical chart and the beam mechanics formulation for different beam thicknesses. Note, a factor of safety of 2.0 is applied to the sill beam curves. From the chart it was determined that the required strength of backfill for stability of a 6 m span (3.5 m high sill height) according to beam mechanics was 1.8 MPa. This was used as the initial design strength of the backfill.
Mitchell stability analysis (Mitchell and Roettger, 1989) was performed on the sill beams at Kencana, with both rotational and flexural failure considered. Rotational failure was an issue with the 45° dipping ore bodies (Support class C from Table 5-7). It was determined shear paddles were to be installed so they resist the moment caused by the rotational failure. A conservative approach was performed with the assumption that the hangingwall and footwall would provide no shear resistance to rotation (counter to Caceres, 2005). Assuming the bolts installed as shear paddles have a shear resistance of 8 tonnes (Pakalnis and Associates, 2006), taking a moment about the point of rotation, it was determined that the shear paddles anchors needed to be spaced 0.75 m along the HW. However, in practice these are placed at 1.2 m (offset of the stand-up bolts) due to considerations of shear strength of the footwall and hangingwall as per Mitchell analysis (Mitchell and Roettger, 1989).

The release of cold joints and the effect of flexural failure were incorporated into the design by way of the sill mat. It was determined that during flexural failure, the lower point of the beam would be in tension and would fail. If the lower portion of the beam was contained with tendon support, the flexural failure would not occur. To determine the required bolt length,
first the neutral point between compressive and tensile force within the beam was determined through beam theory, then the weight of the beam in tension was calculated per meter length. Once the weight was calculated, the spacing and capacity of the bolt to contain the material in tension is found. This results in a bolting pattern of 1.2 m x 1.2 m spacing would be suitable in containing the mass in tension for up to 6 m wide and 5 m high. The length of bolt required to be safely anchored within the mass under compression was determined to be \( \frac{1}{2} \) of the beam height plus 0.30 m, this bolt length provides the necessary anchorage within compression zone to contain the mass in tension.

Numerical modeling was performed to determine the stability of the sill based on width of the stope and the amount of closure within the stope. The model set-up and scenario is presented in Hughes et al. (2006). Figure 5-9 show the results of the model. It was found that the Mitchell stability for rotational failure was an over-estimate of required strength, and that the model reflected the findings of Caceres (2005). It should be noted that the required strength from modeling is presented with a Factor of Safety of 1.0.

Figure 5-9: Kencana numerical model results

![Kencana numerical model results](image)

The effect of closure was investigated for the stopes. Figure 5-10 plots the results of the difference between 0 mm and 10 mm side wall closure (20 mm total across stope). It can be seen
that the amount of closure affects the minimum strength of the material. It was found during analysis that closure failures are common for spans less than 4 m or when 20 mm of closure was experienced. In wide stopes (greater than 4 m) or where there is no closure flexural failure dominates. It was concluded that closure increase the compressive strain within the fill ultimately causing failure. This type of failure is verified by Krauland and Stille (1993) for stopes with large closures.

**Figure 5-10: Stability chart for 4.75 m high paste sill**

The main issue with the undercuts at Kencana was the rock/CPB hybrid span. Where there is a shallow ore body, and the requirement to mine the stopes at 60\(^\circ\), geometry dictates that a rock overhang is formed. The overhang causes issues in Kencana as it is assumed the host rock is weaker than that of the paste. To support the rock overhang, support needed to be designed to support the overhang. It was assumed that the overhang would fail from the top of the hangingwall in the bottom cut to the top of the hangingwall in the sill (weak rock mass; ½ span type failures). Limit equilibrium analysis were performed to determine the amount of both vertical (from the undercut) and horizontal support (added prior to backfill stope) would be required to support the overhang. To provide a suitable factor of safety, horizontal wall support needed to be installed prior to undercutting to prevent collapse of the overhang and then vertical
support installed after the undercut to support the sill beam. The bolting pattern for the rock overhangs are described in Table 5-7.

Kencana mine has been safely mining for eight years utilizing UCF CPB mining. The challenges of weak rock and high closure were addressed through large amount of support prior to placement of fill, the use of sill mat support, shotcrete and bolting on advance. The high cost associated with this design is feasible due to the high grade gold deposit.

5.5 KL Gold Macassa Mine

KL Gold Macassa mine is similar in design to Red Lake. The mine is in a high stress environment with competent ground: rock bursting is an issue. The use of UCF with CPB was used to provide a safe, low-rock burst potential back in stopes.

Historic mining in the Macassa claim has occurred on-and-off since the 1920’s. The production at the mine consisted of shrinkage stoping at shallow depth and transition to cut and fill methods using waste rock, or sand and gravel as backfill at depth (Hedley, 1992). Rockburst were common at the mine with increasing depth due in parts to the depth of working, structural features, dip of orebody, sequence of stoping operation and the rate of mining.

Mining widths varied between 1.5 and 15 m (Hedley, 1992, p. 256) with underhand cut-and-fill (traditional) and underhand bench and fill being employed. Long-hole stoping was employed at the mine until a 3.8 Mn event in April 1997 created a considerable damage to the No.3 Shaft (Blake and Hedley, 2003).

The use of fill was a necessity to the operation of mining at Macassa. Further, the elimination of diminishing pillars was required to mitigate the risk of rock bursting. It was found stiffening the fill reduced the energy released as a results of a rock burst (Hedley, 1992). This is a key finding in designing CPB to meet the imposed loads at the mine. Tight fill is required in all stopes.

Upon acquisition of the property in 2001 by KL Gold a review of mining methods occurred. To mitigate the risk posed by rock bursting, underhand cut and fill was initially planned. As mining progressed, it was found that the stress issues were in the hangingwall and
not the back. Macassa mine developed a mining method in which stresses are shed into the lower side of the hangingwall: a modified underhand bench and fill design. Higher-back stopes are mined and subsequent stopes are offset on the hangingwall side in the stress shadow. This allows for a non-CPB back and a low stress environment due to the shadowing of the stress.

With this mining method, it is rare that Macassa develops under CPB at present. Mining under CPB only occurs when access to stope is an issue and development is required through/under paste.

The mine stress gradient is provided in Table 5-9.

Table 5-9: Stress gradient at Macassa Mine (Hedley, 1992)

<table>
<thead>
<tr>
<th>Principal Stress</th>
<th>Direction</th>
<th>Gradient (kPa/m of depth)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_1$</td>
<td>Perpendicular to Strike</td>
<td>42</td>
</tr>
<tr>
<td>$\sigma_2$</td>
<td>Parallel to strike</td>
<td>30</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>Vertical</td>
<td>26</td>
</tr>
</tbody>
</table>

5.5.1 Backfill Strength

The backfill strength employed at Macassa Mine is discussed in detail in Section 5.5.2. Macassa technical service believes that the high strength of the backfill are due to the coarser fraction of mined esker, representing 30% of the non-binder constituents of the fill. Weatherwax (2008) provides an explanation on the coarse fraction increasing the strength of CPB.

5.5.2 Span Width

There are two types of underhand spans at Macassa: mechanized and traditional UCF. The traditional UCF fills are typically 2.1 m wide, with the mechanized UCF fills 3.6 m wide.

The issue with the span widths in the mechanized stopes is that they are typically developed as panel drifts. If tight fill is not achieved during the panel drift filling, larger than planned stopes occur due to gaps existing between the filled stope back and the fill. To reduce
the span when tight fill is not achieved in panel mining, span-interrupters are installed between the fill and the back.

The span interrupter consists of shotcrete sprayed between the back and the fill from the footwall side of the drift. The shotcrete ‘pillars’ are placed such that they are spaced along the length of the drift equal to the width of the drift. The widths of the pillars are 0.5 m and extend into the previously filled stope a minimum of 0.5 m.

In essence the span interrupter reduces the maximum span to that of the width of the originally planned stope. The issue with the span interrupter is that they are a reactionary ground support to poorly filled stopes: tight filling of stopes would eliminate the need of span interrupters.

5.5.3 Minimum Ground Support

There exist three ground support standards for the sill mat at Macassa: span width less than 2.1 m; multiple drifts (FW side); multiple drift (HW side).

Figure 5-11 shows the typical layout for the spans less than 2.1 m. Of note is that no stand-up bolts are installed prior to placing the backfill, only the ‘shear paddles’ are installed. After the sill is undercut, ground support consisting of 1.5 m long SS 39 friction bolts on a 1.8 m x 1.8 m pattern are drilled into the sill. No welded wire mesh is installed as ground support.
Figure 5-11: Sill mat support for stopes narrower than 2.1 m

Figure 5-12 shows the support when drifting beside CPB on the HW wall. There exists no ‘shear-paddles’ in the HW support, rather 2.1 m long, #6 rebar are placed on 0.9 m x 0.9 m centers no closer than 0.6 m to the sidewalls. Support on advance is 1.5 m long SS39 friction bolts with 1.8 m x 1.8 m spacing. No welded wire mesh is installed as part of sill mat construction or during advance.
Figure 5-12: Sill mat support multiple drift (FW side)

Figure 5-13 is the ground support used for large spans or when the stope is developed with backfill in the Footwall side. ‘Shear-paddles’ are installed in the hangingwall consisting of 1.5 m rebar spaced 0.9 m apart, 0.9 m above the floor. Rebar sill is prepped similar to that described above. Ground support on advance consists of the SS39 friction support on the pattern previously described.

Macassa performed friction bolt testing to determine the pull out strength, and bond strength of the support in the backfill. The purpose of the testing was to determine if tapered bolt ends or straight bolts were more effective, and the gain in strength from pushing bolts over the last 0.6 m of the hole depth rather than drilling the complete length. Table 5-10 presents the results from the testing. It can be seen pushing the bolt into the drill hole, drilling the hole to the length of the bolt or using a tapper end or straight end has no effect on the bond strength. Table 5-11 summarizes the results based on bolt type.
5.5.4 Design Guidelines

The design guidelines at Macassa are based entirely on Red Lake design guidelines. Macassa personnel were previous Red Lake employees and incorporated their experiences at Red Lake into designing the sill mat at Macassa.

The design guidelines for UCF mining at Macassa are a response to the requirement to control the ground by reducing the size of underground voids and mining under an engineered back as opposed to a high stress back in a burst prone environment.
Table 5-10: Macassa friction bolt pull out test result

<table>
<thead>
<tr>
<th>Test</th>
<th>Bolt type</th>
<th>Bolt length (m)</th>
<th>Hole Length (m)</th>
<th>Length bolt in borehole (m)</th>
<th>Load at slippage (tonnes)</th>
<th>Bond Strength (tonnes/m)</th>
<th>Bond Strength (tonnes/foot)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>FS39 Galv. taper end</td>
<td>1.65</td>
<td>1.47</td>
<td>0.18</td>
<td>1.36</td>
<td>1.23</td>
<td>0.37</td>
</tr>
<tr>
<td>2</td>
<td>FS39 Galv. taper end</td>
<td>1.65</td>
<td>1.32</td>
<td>0.33</td>
<td>6.80</td>
<td>6.17</td>
<td>1.85</td>
</tr>
<tr>
<td>3</td>
<td>FS39 Galv. taper end</td>
<td>1.65</td>
<td>1.42</td>
<td>0.23</td>
<td>6.58</td>
<td>5.97</td>
<td>1.79</td>
</tr>
<tr>
<td>4</td>
<td>FS39 Galv. taper end</td>
<td>1.65</td>
<td>1.63</td>
<td>0.02</td>
<td>0.91</td>
<td>0.82</td>
<td>0.25</td>
</tr>
<tr>
<td>5</td>
<td>FS39 Galv. taper end</td>
<td>1.65</td>
<td>1.63</td>
<td>0.02</td>
<td>0.91</td>
<td>0.82</td>
<td>0.25</td>
</tr>
<tr>
<td>6</td>
<td>FS39 Galv. taper end</td>
<td>1.65</td>
<td>1.63</td>
<td>0.02</td>
<td>1.36</td>
<td>1.23</td>
<td>0.37</td>
</tr>
<tr>
<td>7</td>
<td>FS39 Galv. taper end</td>
<td>1.65</td>
<td>1.32</td>
<td>0.33</td>
<td>6.80</td>
<td>6.17</td>
<td>1.85</td>
</tr>
<tr>
<td>8</td>
<td>FS39 Galv. taper end</td>
<td>1.65</td>
<td>1.29</td>
<td>0.36</td>
<td>6.36</td>
<td>5.76</td>
<td>1.73</td>
</tr>
<tr>
<td>9</td>
<td>FS39 Galv. taper end</td>
<td>1.65</td>
<td>1.32</td>
<td>0.33</td>
<td>6.80</td>
<td>6.17</td>
<td>1.85</td>
</tr>
<tr>
<td>10</td>
<td>FS39 Galv. taper end</td>
<td>1.65</td>
<td>1.42</td>
<td>0.23</td>
<td>6.58</td>
<td>5.97</td>
<td>1.79</td>
</tr>
<tr>
<td>11</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.04</td>
<td>0.61</td>
<td>4.99</td>
<td>4.53</td>
<td>1.36</td>
</tr>
<tr>
<td>12</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.16</td>
<td>0.49</td>
<td>3.63</td>
<td>3.29</td>
<td>0.99</td>
</tr>
<tr>
<td>13</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.04</td>
<td>0.61</td>
<td>4.08</td>
<td>3.70</td>
<td>1.11</td>
</tr>
<tr>
<td>14</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.06</td>
<td>0.59</td>
<td>2.27</td>
<td>2.06</td>
<td>0.62</td>
</tr>
<tr>
<td>15</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>0.89</td>
<td>0.76</td>
<td>5.62</td>
<td>5.10</td>
<td>1.53</td>
</tr>
<tr>
<td>16</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.47</td>
<td>0.18</td>
<td>0.91</td>
<td>0.82</td>
<td>0.25</td>
</tr>
<tr>
<td>17</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.42</td>
<td>0.23</td>
<td>0.91</td>
<td>0.82</td>
<td>0.25</td>
</tr>
<tr>
<td>18</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.16</td>
<td>0.49</td>
<td>4.99</td>
<td>4.53</td>
<td>1.36</td>
</tr>
<tr>
<td>19</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.11</td>
<td>0.54</td>
<td>3.63</td>
<td>3.29</td>
<td>0.99</td>
</tr>
<tr>
<td>20</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.21</td>
<td>0.44</td>
<td>4.08</td>
<td>3.70</td>
<td>1.11</td>
</tr>
<tr>
<td>21</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.09</td>
<td>0.56</td>
<td>2.27</td>
<td>2.06</td>
<td>0.62</td>
</tr>
<tr>
<td>22</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>0.89</td>
<td>0.76</td>
<td>5.62</td>
<td>5.10</td>
<td>1.53</td>
</tr>
<tr>
<td>23</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.34</td>
<td>0.31</td>
<td>6.35</td>
<td>5.76</td>
<td>1.73</td>
</tr>
<tr>
<td>24</td>
<td>FS39 Galv. without taper end</td>
<td>1.65</td>
<td>1.29</td>
<td>0.36</td>
<td>3.63</td>
<td>3.29</td>
<td>0.99</td>
</tr>
</tbody>
</table>

Table 5-11: Summary of pull testing by bolt type

<table>
<thead>
<tr>
<th>Bolt type</th>
<th>Average Bond Strength (tonnes/m)</th>
<th>Maximum Bond Strength (tonnes/m)</th>
<th>Minimum Bond Strength (tonnes/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FS39 Galv. taper end</td>
<td>3.74</td>
<td>6.17</td>
<td>0.82</td>
</tr>
<tr>
<td>FS39 Galv. without taper end</td>
<td>3.43</td>
<td>5.76</td>
<td>0.82</td>
</tr>
</tbody>
</table>

5.6 Summary of Case Histories

The case histories demonstrate that UCF CPB mining can be applied in challenging ground conditions, be it high stress, high closure, high-seismic zones or areas of weak rock mass. The advantage in UCF for the ground conditions is that the sill beam in the back mitigates the risk posed by the challenging ground. The type of support varies between mines, however, it is
shown that support, either before after under-cutting, is necessary. For all mines investigated the paste strength is greater than 1.0 MPa at 28 days; however, entry times to stopes are performed typically less than the 28 days strengths. Empirical evidence was presented in determining span width vs. strength of fill that can be used in future design. Figure 5-5 is a valuable tool for empirical design in showing the 28 day strength of the fill for the spans widths mined at Stillwater. Bond strength within paste is crucial in design of support systems and is listed for the operations studied; however, the bond strength is site specific and dependent on drilling techniques and bolt types.

Each of the case studies has a different design to the sill beams. However, each mine approached the engineering design incorporating observational, numerical and analytical design elements. The need to support the fill from surficial failures is incorporated in each design, as is addressing the failure of the beam from detachment from stope walls. Further, all mines take the control of their backfill as paramount to ensuring the stability of the fill.

**Chapter takeaways**

- UCF mining is successfully applied over a wide range of ground condition
- Ground support differs from site to site; and
- Cemented paste design strengths range between 1000 and 5000 kPa
6 Constitutive Behaviour of Cemented Paste Backfill

Cemented paste backfill (CPB) is used across a wide range of stress and rock mass environments. As such, its behaviour, or constitutive relationship, is necessary to properly reflect the stress-strain behaviour under load. This chapter will research the development of a constitutive relationship for CPB under axial load. By understanding the constitutive relationship of the material, a better understanding of sill beam can be gained.

The constitutive behaviour presented in literature centers around four models. Helinski (2008) present the Critical state line (CSL) model in describing the placement of CPB in a large stope. The Strain softening model (Krauland and Stille, 1993; Swan and Brummer, 2001; Hughes et al., 2006), investigates the post-peak behaviour of CPB. Bretchel et al. (1999); DeSouza and Dirige (2001); and Pierce (2001) present the Mohr-Coulomb approach. Duncan-Chang hyperbolic model (Duncan and Chang, 1970) investigates a non-linear elastic response to loading. This chapter is a discussion comparing these constitutive approaches applied to laboratory testing. The validity of the models as applied to axial loading of cemented paste backfill is the end goal.

6.1 Mohr-Coulomb Constitutive Model

The linear elastic, perfectly plastic constitutive model approach was first conceived by Coulomb (1776). The Mohr-Coulomb failure criteria for modelling backfill was undertaken by Bretchel et al. (1999); DeSouza and Dirige (2001); and Pierce (2001).

The Mohr-Coulomb constitutive model is a simple linear-elastic, perfectly plastic yield surface. Elastic strains are solely responsible for the gain in stress within Mohr-Coulomb relationships. The advantage of the Mohr-Coulomb for numerical modeling is that the following seven material properties are required as inputs: cohesion, friction angle, dilation angle, shear modulus, bulk modulus, tensile strength and unit weight. The shear modulus and bulk modulus are interchangeable with the Young’s Modulus and Poisson’s ratio. These material properties are obtained from common laboratory testing. Tri-axial testing or a combination of UCS, tensile testing and shear tests. The results from the laboratory tests are a direct input into the constitutive model.
The Mohr-Coulomb model follows a linear-elastic model until the material reaches a peak strength. Once peak strength is reached, yielding of the material occurs. Elastic stress increases in the yielding phase are countered by plastic strains. For granular material (CPB) the plastic dissipation of elastic stresses obeys a non-associated flow rule (Drucker and Prager, 1952). DeSouza and Dirige, 2001) suggest that CPB obeys the non-associate flow rule.

The following describes the Mohr-Coulomb equations per Itasca (2011) and Cowin (1973). The information is described in an effort to relate the constitutive relationship to the behaviour CPB under applied strains.

For Mohr Coulomb, the initial phases of loading in a non-failed state follow Hooke’s Law. In Hooke’s law the material follows a linear loading path in stress-strain space. All strains are elastic and recoverable. For a CPB beam, this is the initial loading path of the beam, and is not considered to be related to failure. Hooke’s Law is as follows:

\[
\begin{align*}
\Delta \sigma_1 &= \alpha_1 \Delta \varepsilon_1^e + \alpha_2 (\Delta \varepsilon_2^e + \Delta \varepsilon_3^e) \\
\Delta \sigma_2 &= \alpha_1 \Delta \varepsilon_2^e + \alpha_2 (\Delta \varepsilon_1^e + \Delta \varepsilon_3^e) \\
\Delta \sigma_3 &= \alpha_1 \Delta \varepsilon_3^e + \alpha_2 (\Delta \varepsilon_1^e + \Delta \varepsilon_2^e)
\end{align*}
\]

Equation 6-1

where:

\[
\begin{align*}
\alpha_1 &= \frac{E(1-v)}{(1-2v)(1+v)} \\
\alpha_2 &= \frac{Ev}{(1-2v)(1+v)}
\end{align*}
\]

Equation 6-2

Two failure envelopes exist in Mohr Coulomb relationships: material in compression (shear) and material in tension. These failure envelopes are analogous to a crushing failure (compression) and tensile failure per Jordan et al. (2003). The failure envelope for shear is defined as follows:

\[
f^* = \sigma_1 - \sigma_3 N \theta + 2c \sqrt{N \theta}
\]

Equation 6-3
The tensile failure envelope is given by the following function:

\[ f' = \sigma^t - \sigma_3 \]  \hspace{1cm} \textit{Equation 6-5}

Plastic potential flow (gs), (the amount of plastic deformation occurring on the yield surface) takes the following form:

\[ g^s = \sigma_1 - \sigma_3 N_\psi \]  \hspace{1cm} \textit{Equation 6-6}

where:

\[ N_\psi = \frac{1 + sin\psi}{1 - sin\psi} \]

\[ \psi = \text{angle of dilation} \]

The failure envelope is a stress-state limit that the material under strain cannot exceed. Once the material reaches the failure envelope the onset of yielding, or plastic deformations, occur. In terms of CPB beam loading, once the maximum stress is reached, the beam can be expected to undergo plastic deformations under a constant applied load: the beam is in a failed state. The plastic deformation takes the form of the following differential equation:

\[ \Delta \epsilon^p_i = \lambda_s \frac{\partial g^s}{\partial \sigma_i} \quad i = 1, 2, 3 \]  \hspace{1cm} \textit{Equation 6-7}

Allowing the \( \lambda_s \) term to be defined later (Itasca, 2011), integration of the potential function with respect to the principal stresses yields the following equations for principal plastic strains:

\[ \Delta \epsilon^p_1 = \lambda_s \]

\[ \Delta \epsilon^p_2 = 0 \]  \hspace{1cm} \textit{Equation 6-8}

\[ \Delta \epsilon^p_3 = -\lambda_s N_\psi \]

In terms of deformation model mechanics, the \( \lambda_s \) accounts for what is termed an elastic guess (Itasca, 2011). The elastic guess serves an incremental stress value that exceeds the failure
surface of the material; the elastic guess stress is transferred to irrecoverable plastic strains. This is the beginning stages of computing ‘plastic flow’: dissipating elastic stress increases with plastic deformations.

With the formula of the plastic strains known, the elastic guess needs to be defined to be able to model the Mohr Coulomb behaviour. By inputting the current stress state in the equation of the failure envelope and some re-arranging allows the elastic guess to be defined as following:

\[
\lambda_s = \frac{f(\sigma_1, \sigma_3)}{(\alpha_1 - \alpha_2 N \psi) - (\alpha_2 - \alpha_1 N \psi) N \theta}
\]

Equation 6-9

Manipulating Hooke’s law for elastic behaviour above, and the fact that total strain is a sum of elastic and plastic strains, the incremental stresses can be expressed as follows:

\[
\Delta \sigma_1 = \alpha_1 \Delta \varepsilon_1 + \alpha_2 (\Delta \varepsilon_2 + \Delta \varepsilon_3) - \lambda_2 (\alpha_1 - \alpha_2 N \psi)
\]

\[
\Delta \sigma_2 = \alpha_1 \Delta \varepsilon_2 + \alpha_2 (\Delta \varepsilon_1 + \Delta \varepsilon_3) - \lambda_2 (1 - N \theta)
\]

\[
\Delta \sigma_3 = \alpha_1 \Delta \varepsilon_3 + \alpha_2 (\Delta \varepsilon_2 + \Delta \varepsilon_3) - \lambda_2 (-\alpha_1 N \psi + \alpha_2)
\]

Equation 6-10

That is to say, once the material has yielded, the incremental stress is a portion of stress due to linear elastic strains and a countering plastic stress.

The elastic guess provides the amount of incremental stress to apply to a yielding surface, the updated stress state can be determined with the following equation, where the ‘C’ and ‘P’ superscripts indicate current and previous stress states:

\[
\sigma_i^C = \sigma_i^P + \Delta \sigma_i \quad For \ i = 1,2,3
\]

Equation 6-11

To determine the appropriateness of the Mohr-Coulomb constitutive behaviour to model the laboratory stress-strain curves, an algorithmic code was made to demonstrate the behaviour of a material under axial load. The algorithm was coded in Visual Basic (VBA) and simplified graphs are presented to determine the appropriateness of Mohr-Coulomb in modeling CPB behaviour under axial load. The model incorporated the equations above to determine the appropriate stresses for incremental strains. The results of the model were compared to the UCS tests performed in Chapter 4 for Red Lake and Stillwater samples. The Young’s modulus values reported are obtained at the 50% of peak stress (per Fairhurst and Hudson, 1999). Cohesion and
friction values were manipulated per Equation 6-3 to achieve a fit between the data and the model results. Comparisons between the Mohr-Coulomb model and the stress strain curves presented in Chapter 4 are presented in detail in Appendix 4; an example of the fit between the test data and Mohr-Coulomb model is shown in Figure 6-1.

![Figure 6-1: Red Lake Sample #12 Mohr-Coulomb fit](image)

A summary of the simulation for the Mohr-Coulomb fit is provided in Table 6-1. One result of the fit is that the cohesion values are typically 35% of the UCS values. These values were determined by investigation of curve-fit to the experimental data. However, an infinite combination of friction angles and cohesion will give the same failure strength. Although there are mathematically infinite combinations of friction angle and cohesion values to obtain the same failure strength, the boundary conditions are the material properties determined through laboratory testing. To determine the physical limits investigation into literature was performed. It was found that the friction angles used are similar to those presented by Aref et al. (1999) and Pierce (2001).
Table 6-1: Summary of Mohr Coulomb simulation

<table>
<thead>
<tr>
<th>Sample</th>
<th>Peak Stress (kPa)</th>
<th>Young's Modulus (MPa)</th>
<th>Peak Stress (kPa)</th>
<th>Young's Modulus (MPa)</th>
<th>Cohesion (kPa)</th>
<th>Friction Angle</th>
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<td>754</td>
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<td>20</td>
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<td>15</td>
</tr>
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<td>14</td>
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<td>20</td>
</tr>
<tr>
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<td>55</td>
<td>5.2</td>
<td>20</td>
<td>18</td>
</tr>
<tr>
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<td>4.5</td>
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<td>20</td>
</tr>
<tr>
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<td>4</td>
<td>57</td>
<td>3.9</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Red Lake Sample _5</td>
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<td>80</td>
<td>17</td>
</tr>
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<td>90</td>
<td>20</td>
</tr>
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</tr>
<tr>
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<tr>
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<td>27</td>
<td>271</td>
<td>106.7</td>
<td>104</td>
<td>15</td>
</tr>
</tbody>
</table>

The above analysis demonstrates that the Mohr-Coulomb model predicts failure at a lower strain than experienced in the lab. Another method in numerically modeling peak stress to corresponding axial strain is to use the secant modulus of the material at failure. The secant modulus values are not suggested by any ISRM or ASTM standard.

The Mohr-Coulomb model is restricted to a linear loading path to failure. This is in direct contrast to the laboratory samples that demonstrated a curvilinear failure loading path. Further, with the post-peak conditions being constant, the Mohr-Coulomb model cannot provide any strain softening approach to the material.

The Mohr-Coulomb model provides a suitable ‘first-pass’ approach to numerical synthesis of CPB. The advantages are that a small amount of material properties are required and it is common in modern software codes. However, the strain softening and curvilinear elastic loading is ignored in a Mohr-Coulomb model. The study presented above demonstrates that the Mohr-Coulomb constitutive model is a poor representation of CPB under axial loading.
6.2 Hyperbolic Elastic Model

The hyperbolic elastic constitutive model provides a curvilinear loading path for material under loading uniaxial and triaxial conditions. The hyperbolic elastic model was first suggested by Duncan and Chang (1970) to represent the behaviour of sands under tri-axial loading.

The hyperbolic model achieves a curvilinear loading path by reducing the elastic modulus given an applied axial stress to the material. In comparison, the lack of a curvilinear loading path is a weakness to the Mohr-Coulomb model. In general, the Hooke equations for linear elasticity can be modified for each strain increment as shown below in Equation 6-12 (after Rahnema, 2008).

\[
\begin{bmatrix}
\Delta \sigma_x \\
\Delta \sigma_y \\
\Delta \tau_{xy}
\end{bmatrix} =
\begin{bmatrix}
E_t \frac{(1-\nu)}{(1+\nu)(1-2\nu)} & \frac{E_t \nu}{(1+\nu)(1-2\nu)} & 0 \\
\frac{E_t \nu}{(1+\nu)(1-2\nu)} & E_t \frac{(1-\nu)}{(1+\nu)(1-2\nu)} & 0 \\
0 & 0 & \frac{E_t}{2(1+\nu)}
\end{bmatrix}
\begin{bmatrix}
\Delta \varepsilon_x \\
\Delta \varepsilon_y \\
\Delta \gamma_{xy}
\end{bmatrix}
\]

Equation 6-12

In Equation 6-12, \( E_t \) represents the instantaneous tangential modulus of the material. It was found by Kondner (1963) that clay and sand soils may be approximated by hyperbolae over a range of axial strains. Through testing of materials it was determined that soil, under compression never reaches the hyperbolae asymptote. Once discovered, it was suggested that the compressive strength of the soil be represented as a fraction of the asymptotic strength. This fraction is referred to as \( R_f \) and is defined as follows:

\[
R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{ULT}}
\]

Equation 6-13

where:

\[(\sigma_1 - \sigma_3)_{ULT} = \text{Asymtopic value of stress – strain curve} \]
The ultimate strength of material is defined as the slope of a transformed axis of the stress-strain plot of the material. A sample of the transformed plot is shown in Figure 6-2. Investigation of the failure strength (Equation 6-15) shows that the peak strength is defined by two material properties: friction angle and cohesion.

To replicate the hyperbolic elastic model of a material, the first step is to perform compressive strength testing of material with elastic modulus readings. This has been discussed to great extent within Chapter 4 through 6. After the tests are complete, the data can be plotted on a transformed stress strain plot with the y-axis equivalent to the axial strain divided by the deviatoric stress; x-axis, axial strain (Figure 6-2). The initial elastic modulus ($E_i$) and ultimate hyperbolic strength (asymptotic stress) can then be determined by defining the intercept and best-fit slope, respectively, of the plotted transformed data.

After the relevant values are determined from the testing data, the tangential elastic modulus at any stress point can be determined using the following equation:

$$E_t = (1 - R_f S)^2 \ E_i$$  \hspace{1cm} Equation 6-16
where:

\[ S = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f} \] \hspace{1cm} \textit{Equation 6-17}

For uniaxial compressive testing, the above equation 6-16 is suitable. Duncan and Chang (1970) suggest an equation for the tangential modulus under triaxial conditions.

Once the tangential modulus is known at a given stress point, the strain is easily determined from the following equation:

\[ \varepsilon_a = \frac{\sigma_a}{(1 - R_fS)^2 E_i} \] \hspace{1cm} \textit{Equation 6-18}

Duncan et al. (1980) provide the following caution in applying the hyperbolic model to constitutive modeling:

- The intermediate principal stress \( \sigma_2 \) is not accounted for;
- Results may be unreliable when extensive failure occurs;
- It does not consider the volume change due to changes in shear stress (shear dilatancy);
- Input parameters are not fundamental soil properties, but only empirical values for limited range of conditions; and
- The model is mainly intended for quasi-static analysis.

The issue with the above limitations is that in numerical simulation of CPB, a Duncan-Chang constitutive approach may lead to the material failing at unrealistic strains. This will be discussed later when considering strain-softening models.

To determine the suitability of a hyperbolic model for CPB, uniaxial stress-strain tests presented in Chapter 4 and 5 are analyzed. The initial step was to determine the hyperbolic asymptote and initial elastic modulus from the transferred plots. The determination of these properties was difficult for the Stillwater samples where hysteresis loading occurred: best fit lines were manually drawn through plots and values measured. Figure 6-2 is an example of the
transferred plot: the slope of the line is the reciprocal of asymptotic hyperbolae limit and the intercept is the reciprocal of the initial elastic modulus (Duncan and Chang, 1970).

Figure 6-2: Red Lake Sample #5 transformed plot

Once the initial values were determined, a Visual Basic script was used to calculate the stresses given an increase in strain within the material per Equation 6-17. The script calculated the tangential elastic modulus per Equation 6-16. The value of Rf for each sample was determined from a transferred plot as shown in Figure 6-2. To fit the data, the friction angle and cohesion were modified such that the calculated stress-strain path matched the laboratory results. First estimates of friction angles were those determined in the Mohr-Coulomb analysis. It is hoped that the result of the hyperbolic modeling provides a better fit of axial loading than the Mohr-Coulomb analysis.

Figure 6-3 through Figure 6-18 are graphical representations of the calculated hyperbolic model in comparison to laboratory data.
Figure 6-3: Stillwater Sample 35E8600_145pm hyperbolic curve fit

Figure 6-4: Stillwater Sample 35E8600_545pm hyperbolic curve fit
Figure 6-5: Stillwater Sample 35E8600_745pm hyperbolic curve fit

Sample: 35E8600_745pm / E50: 447 MPa / UCS: 915.21 kPa

Figure 6-6: Red Lake Sample #1 hyperbolic curve fit

Sample: Red Lake Sample _1 / E50: 9 MPa / UCS: 86.24 kPa
Figure 6-7: Red Lake Sample #2 hyperbolic curve fit

Figure 6-8: Red Lake Sample #3 hyperbolic curve fit
Figure 6-9: Red Lake Sample #4 hyperbolic curve fit

Figure 6-10: Red Lake Sample #5 hyperbolic curve fit
Figure 6-11: Red Lake Sample #6 hyperbolic curve fit

Sample: Red Lake Sample _6 / E50: 40 MPa / UCS: 256.62 kPa

Figure 6-12: Red Lake Sample #7 hyperbolic curve fit

Sample: Red Lake Sample _7 / E50: 68 MPa / UCS: 253.49 kPa
Figure 6-13: Red Lake Sample #8 hyperbolic curve fit

Figure 6-14: Red Lake Sample #9 hyperbolic curve fit
Figure 6-15: Red Lake Sample #10 hyperbolic curve fit

Sample: Red Lake Sample _10 /  E50: 81 MPa /  UCS: 343.43 kPa

Figure 6-16: Red Lake Sample #11 hyperbolic curve fit

Sample: Red Lake Sample _11 /  E50: 792 MPa /  UCS: 561.65 kPa
Figure 6-17: Red Lake Sample #12 hyperbolic curve fit

Sample: Red Lake Sample _12 / E50: 29 MPa / UCS: 368.69 kPa

Figure 6-18: Red Lake Sample #13 hyperbolic curve fit

Sample: Red Lake Sample _13 / E50: 27 MPa / UCS: 271.31 kPa
As seen in Figure 6-3 through Figure 6-18, the majority of the curve fits follow the axial stress/strain path of the laboratory tests. Stillwater Sample 35E8600_545pm and Red Lake Sample #1 are two notable exceptions where the modeled fit does not follow the stress-strain path towards failure. This is due to a poor approximation of the Ei value from the data plotted in the process shown in Figure 6-2. The hyperbolic model does not account well for the post-peak behaviour of the material. This is because the model is elastic and cannot account for plastic strain. The values used in the hyperbolic model are summarized in Table 6-2.

Table 6-2: Hyperbolic model fit parameters

<table>
<thead>
<tr>
<th>Sample</th>
<th>Peak Stress (kPa)</th>
<th>Young's Modulus [E50] (MPa)</th>
<th>Peak Stress (kPa)</th>
<th>Initial Elastic Modulus [Ei] (MPa)</th>
<th>Rf</th>
<th>Cohesion (kPa)</th>
<th>Friction Angle</th>
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</tbody>
</table>

In general Rf values are in agreement with those proposed by Duncan and Chang (1970), ranging between 0.7 and 0.93. The elastic modulus values from the data fit (Ei) appear to be appropriate and account for the curvilinear loading path (applied stress dependency) of CPB. It was found from the study that hyperbolic curve fit is more appropriate in modeling the pre-peak behaviour axial stress-strain path of the material.
The cohesion and friction angles used in Table 6-2 are not unique. Equation 6-15 has infinite solutions given a cohesion value and a failure strength: various cohesion and friction angle can be used to obtain same values of $R_f$. Figure 6-19 is a comparison of published friction angles vs. cohesion values for CPB. The values used in the hyperbolic model (summarized in Table 6-2) are plotted for comparative purposes. It can be seen that the cohesion and friction values used are well within the values obtained in literature.

Figure 6-19: Comparison of cohesion to friction angle values

![Friction Angle (°) vs. Cohesion (kPa)](image)

To determine the ratio of cohesion to the UCS for CPB, UCS values were calculated by Equation 6-15 and plotted against the published cohesion values as shown in Figure 6-20. It can be seen that a strong correlation exists between the UCS and cohesion ($R^2=0.97$). The slope of the line can be considered the ratio of UCS to cohesion. Figure 6-20 summarizes an important finding of this research. Current practice is to assume cohesion is 25% of the UCS of the material. In practice, limit equilibrium models are performed with this cohesion assumption. Once stability is obtained with the cohesion assumption, the required UCS is back calculated.
The findings shown in Figure 6-20, demonstrate that using a cohesion equivalent of 25% of UCS, an implicit conservatism is included in the design. The finding presented in Figure 6-20 demonstrates that the cohesion as a percentage of UCS should be 32%; this finding reduces the required UCS determined from limit equilibrium and numerical analysis by 22%. The implications of this are significant on two fronts: design stability and paste recipe design. The target strength for paste strength can be reduced if the studies by mine sites verify the cohesion vs. UCS relationship.

Figure 6-20: UCS vs. cohesion of published data

![Graph showing UCS vs. cohesion of published data](image)

Two issues remain with the hyperbolic elastic behaviour. First, the model forces the material to deform under load to infinity; there is no limit to the amount of stress or strain the material can achieve. This suggests that the material will never fail (i.e. assumed to be elastic), and is infinitely compressible. Secondly, and related to the first point, the model does yield. This is a large issue as inspection of the hysteresis loading of the Stillwater samples shows that the material experiences plastic deformations under axial loading and can sustain loads post-peak strength. However, it was observed during testing that the samples will ultimately fail at an ultimate strain and cannot take any applied load. Both of the plastic deformations and reduction in material strength post-peak can be accounted for by adopting piece-wise strain-softening/explicit loading path models.
6.3 Strain-softening Models

Strain-softening models account for post peak behaviours of materials and can account for a yield surface. Elegant strain-softening models relating to critical state line theory are presented by Roscoe, Schofield and Worth (1958) and by Helinski et al. (2010a) in relation to the placement of backfill. These models are based on material properties and the principle of constant volume of material at residual states. Explicit strain-softening models are presented by Krauland and Stille, 1993; Itasca, 2011; Hughes et al., 2006; and Swan and Brummer (2001). These studies defined the strain stress path by forcing the stress path over explicit pre-defined boundaries through piece-wise constraints.

Although inelegant, the explicit model is suitable for determining the post peak/ strain softening behaviour of CPB. It allows user-specified behaviour to be accounted for in the stress-strain response of the material: it is simple in its constitutive approach. The critical state line approach, discussed by Roscoe et al. (1958), is more suited for fully coupled soil models. The incorporation of critical state theory to loading of CPB during undercut is a project all to itself with limited return due to, firstly, the non-consolidating nature of CPB once hydrated, and the large amount of additional ‘engineering’ that accompanies UCF mining negates the elegant approach (ground support, large factor of safeties, the need for operational performance over theoretical benefits). Ultimately, the purpose of the strain-softening model is to account for post-peak behaviour of the material in numerical codes such that failure can occur by reducing the strength of the material to nil. This is a direct contrast to simple plastic models that allow infinitely compressible or infinite strength material.

Explicit strain-softening models as proposed by Andrieux et al. (2003); Swan and Brummer (2001) and Hughes et al. (2006) will be tested against the stress strain paths. Table 6-3 summarizes the strain-softening parameters used for the material in these studies. It should be noted that the relationships shown are piece-wise linear function that reduces the cohesion and/or friction angle from the ultimate strength of the material. Comment by Andrieux et al. (2003) indicates that these values were “based on previous Itasca projects” (p.4). From this dissertation it was found the published strain-softening values are not substantiated with laboratory testing. It should be noted that the values used by Hughes et al. (2006) were based on
replicating work of a previous consulting study. All the above references use strain-softening for incorporation of constitutive behaviour with Itasca’s FLAC software. Although implicit in reasoning, nowhere in the studies does it acknowledge that the strain-softening values are for post peak behaviour only: strain-softening values apply to plastic strains. Table 6-3 has updated the published relationships to explicitly state that the reduction is for plastic strain only.

Table 6-3: Published strain-softening relationship

<table>
<thead>
<tr>
<th>Source</th>
<th>Cohesion (kPa)</th>
<th>( \Phi )</th>
<th>Tensile (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Swan and Brummer (2001)</td>
<td>( \left{ \frac{c_i - \frac{c_i}{0.015} \varepsilon_a^p}{\varepsilon_a^p &lt; 0.015} \right} )</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Andrieux et al. (2003)</td>
<td>( \left{ \frac{3c_i}{0.06} \varepsilon_a^p \right} )</td>
<td>32</td>
<td>( \left{ \frac{3\sigma_t}{0.06} \varepsilon_a^p \right} )</td>
</tr>
<tr>
<td>Hughes et al. (2006)</td>
<td>( \left{ \frac{c_i}{0.01} \varepsilon_a^p \right} )</td>
<td>( \left{ \frac{\Phi_i + 10}{0.01} \varepsilon_a^p \right} )</td>
<td>( \left{ \frac{\sigma_t}{0.01} \varepsilon_a^p \right} )</td>
</tr>
</tbody>
</table>

\( c_i = \) initial cohesion; \( \sigma_t = \) initial tension; \( \phi_i = \) initial friction angle; \( \varepsilon_a^p = \) axial plastic strain

In addition to the relationships shown in Table 6-3, Hughes et al. (2006), published the following relationship for the elastic modulus based on the UCS value of CPB. Again, the reported values are published in an effort to validate a previous consulting study. The relationship as proposed is as follows:

\[
E = \begin{cases} 
0.15 \text{ GPa} & \text{for } 0 < \text{UCS} < 400 \text{ kPa} \\
-371600 \text{UCS}^2 + 1318 \text{UCS} - 274.4 & \text{for } 400 < \text{UCS} < 1750 \text{ kPa} 
\end{cases}
\]

Equation 6-19

A VBA script was prepared to model the behaviour of the proposed strain softening and is compared to the stress-strain results of Red Lake Sample #4, Red Lake Sample #8 and Red Lake sample #11; these samples were selected as examples of typical performance for samples with post-peak strain-softening behaviour, or non-associated flow. A graphical comparison of
these models is shown in Figure 6-21 through Figure 6-23. For strain-softening curve fitting, the value of the cohesion for all samples is taken as 25% of the UCS as Swan and Brummer (2001); Andrieux et al. (2003) and Hughes et al. (2006) suggest.

Figure 6-21: Red Lake Sample #4 strain softening comparison
Figure 6-22: Red Lake Sample #11 strain softening comparison

Figure 6-23: Red Lake Sample #11 strain softening comparison
The comparisons of the strain softening models to lab data demonstrate that the strain-softening models are not valid. The reason for the poor fit is the generic application of parameters and lack of testing data.

A strain-softening model can be constructed from work already presented within Chapter 4 of this thesis and the relationships shown in Table 6-3. Firstly, the loading path to the peak strain should take the form of the hyperbolic equation as shown in Equation 6-16. After the peak stress is reached, the sharp drop in stress (shown in Figure 6-21 through Figure 6-23) is easily eliminated by dividing the failure strength shown in Equation 6-13 by the Rf value in Equation 6-12. To determine the post peak behaviour, the slope of the line is determined between peak stress and the stress at maximum strain. It is argued that the friction angle is constant during testing; the drop in strength of the material is a function of the loss of cohesion. The drop in strength will be assigned solely to reducing the cohesion value.

To determine the strength drop, the peak cohesion values are reduced by a value equal to the slope of the line between maximum strength and maximum strain. Once maximum strain is reached, the cohesion value will reduce to zero to reflect the complete loss of strength. The complete loss of strength of the CPB was witnessed during material testing.

The proposed loading path for the CPB is shown below in equation 6-19 and 6-20. The term Δc is introduced to determine the strain-softening value cohesion with respect to the failure surface: the maximum stress is the axial stress value at time of complete sample failure. The values used for the Δc are obtained from laboratory testing of the CPB. The term \( \varepsilon_{pa} \) is the value of axial plastic strain, coded as post-peak strain values. Figure 6-24 shows a comparison between a lab sample and the proposed model.

\[
\Delta c(kPa) = \begin{cases} 
0 & \text{for } \varepsilon_a^p > \varepsilon_{failure} \\
\frac{1}{2} \left( 1 - \sin \phi \right) \left( \frac{\sigma_{max} - UCS}{\varepsilon_{failure} - \varepsilon_{peak}} \right) \varepsilon_{pa}^p & \text{for } 0 < \varepsilon_a^p < \varepsilon_{failure} \\
c_i & \text{for } 0 < \varepsilon_a^e < \varepsilon_{peak} 
\end{cases}
\]

*Equation 6-20*
It can be seen that the proposed model has a very good fit to the stress-strain path and accounts for the material properties. The parameters necessary for analysis are obtained from the same test procedure as standard elastic properties test per Fairhurst and Hudson (1999). The proposed model accounts for the hyperbolic elastic loading of the material as opposed to the existing linear approach up to the ultimate strength of the material. Once the peak strength is reached, the strain-softening values soften the cohesive values based on laboratory data, as opposed to generic application of unverified relationships. Further the model allows for an ultimate load leading to a complete failure of the material, as opposed to an infinitely compressible body as suggested by perfectly plastic models. The issue with the proposed model is that it is not elegant nor based on any material property for strain-softening behaviour, rather it is a coarse approach in fitting the laboratory behaviour. Due to the demonstrated fit of the model
with its piece wise approach to elastic loading and strain-softening post peak behaviour, the proposed model should be used for numerical modeling of axial loading of backfill.

The constitutive relationship analysis of this research is important for sill beam performance as it defines the correct stress-strain response of the beam under loading. It was found that the Mohr-Coulomb approach, typically used in industry, does not reflect the correct loading path to failure and poorly approximates the post-peak behaviour. Strain-softening accounts for post-peak behaviour, however the current practice in industry greatly underestimates the post-peak capacity of the sill beam. It was found that a hyperbolic elastic loading path to failure with a cohesion softening model properly reflects the behaviour of CPB under load. This is important to sill beam stability as it allows the beam to undergo large deformations prior to failure. Further, the investigation into constitutive model shows that CPB has a residual strength higher than what was currently applied in practice. This allows for the sill beam to, one, not fail catastrophically at peak stress and two, and gradually fail during the successive mining of cuts within a stope. Another large finding is that current practice over-estimates the required strength of the sill beam by 22%. This finding provides a larger factor of safety for currently design beams and can reduce costs associated with cement content for future designs. Understanding the constitutive behaviour of the beam will allow designers to reduce strength in beams and to allow for larger deformations before the beam fails.

6.4 Behaviour of Cemented Paste Backfill in Seismic Environments

This section investigates the strength of backfill in a seismic environment. The behaviour of backfill in seismic environments is discussed by Hedley (1992). Building on the research of Hedley, further study by Kaiser, McCreath and Tannant (1997) investigated the role of support with respect to seismicity. This section will use theories presented by both researchers to determine the ability of CPB to sustain seismic events. The analysis will determine the seismic energy the sill beam can sustain. The focus of backfill research in seismic environments is typically for reducing spans and the application of fill to provide confinement to pillars. This section attempts to identify the stability of CPB sill beams within the context of seismicity. This analysis is based on static tests and not dynamic testing. This section is presented in an attempt
to quantify observed behaviour based on available material. Further research to fully quantify the behaviour of CPB during seismic events is needed.

The ability of backfill to withstand seismic energy is based on the materials modulus resiliency or, alternatively, strain-energy density. The resiliency of a material is defined by the area under the elastic portions of a stress strain curve; the toughness of a material is the area under the full stress-strain curve. For design purposes, only the elastic portion of the stress-strain curve will be considered.

The strain-energy density (u) is defined by:

\[ U = \int_{0}^{\varepsilon_{\text{peak}}} \sigma d\varepsilon \]  

Equation 6-22

If the Young’s modulus of the material is used, the strain energy density is simple equation as follows:

\[ U = \frac{UCS^2}{2E} \]  

Equation 6-23

However, as discussed in this Chapter, the advantage of the hyperbolic model is that, typically, the failure strain is larger in comparison to the linear-stress strain path. For strain-energy density the hyperbolic model will provide a large capacity for the strain imposed by a seismic source. Integration of the hyperbolic model was performed by parts. The results of the strain-energy density integral calculations are found in Table 6-4.
Table 6-4: Strain energy of UCS tests

<table>
<thead>
<tr>
<th>Sample</th>
<th>Peak Stress (kPa)</th>
<th>Peak Axial Strain ()</th>
<th>Young's Modulus (MPa)</th>
<th>Initial Elastic Modulus [Ei] (MPa)</th>
<th>Rf</th>
<th>Strain energy by Young's Modulus (kJ/m$^3$)</th>
<th>Strain energy by hyperbolic stress-strain path (kJ/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>35E8600_145pm</td>
<td>760</td>
<td>0.0011</td>
<td>1080</td>
<td>4948</td>
<td>0.894</td>
<td>0.267</td>
<td>0.321</td>
</tr>
<tr>
<td>35E8600_545pm</td>
<td>1018</td>
<td>0.0029</td>
<td>408</td>
<td>558</td>
<td>0.926</td>
<td>1.270</td>
<td>2.160</td>
</tr>
<tr>
<td>35E8600_745pm</td>
<td>915</td>
<td>0.0058</td>
<td>447</td>
<td>2889</td>
<td>0.922</td>
<td>0.936</td>
<td>2.374</td>
</tr>
<tr>
<td>Red Lake Sample _ 1</td>
<td>86</td>
<td>0.019</td>
<td>9</td>
<td>24</td>
<td>0.896</td>
<td>0.411</td>
<td>0.592</td>
</tr>
<tr>
<td>Red Lake Sample _ 2</td>
<td>54</td>
<td>0.012</td>
<td>5</td>
<td>14</td>
<td>0.736</td>
<td>0.292</td>
<td>0.405</td>
</tr>
<tr>
<td>Red Lake Sample _ 3</td>
<td>102</td>
<td>0.024</td>
<td>4</td>
<td>14</td>
<td>0.699</td>
<td>1.301</td>
<td>1.913</td>
</tr>
<tr>
<td>Red Lake Sample _ 4</td>
<td>57</td>
<td>0.015</td>
<td>4</td>
<td>16</td>
<td>0.788</td>
<td>0.406</td>
<td>0.512</td>
</tr>
<tr>
<td>Red Lake Sample _ 5</td>
<td>214</td>
<td>0.011</td>
<td>36</td>
<td>129</td>
<td>0.845</td>
<td>0.636</td>
<td>2.565</td>
</tr>
<tr>
<td>Red Lake Sample _ 6</td>
<td>257</td>
<td>0.009</td>
<td>40</td>
<td>109</td>
<td>0.797</td>
<td>0.826</td>
<td>1.017</td>
</tr>
<tr>
<td>Red Lake Sample _ 7</td>
<td>253</td>
<td>0.015</td>
<td>68</td>
<td>231</td>
<td>0.938</td>
<td>0.471</td>
<td>1.491</td>
</tr>
<tr>
<td>Red Lake Sample _ 8</td>
<td>237</td>
<td>0.016</td>
<td>30</td>
<td>157</td>
<td>0.902</td>
<td>0.936</td>
<td>1.017</td>
</tr>
<tr>
<td>Red Lake Sample _ 9</td>
<td>184</td>
<td>0.017</td>
<td>93</td>
<td>436</td>
<td>0.992</td>
<td>0.182</td>
<td>1.354</td>
</tr>
<tr>
<td>Red Lake Sample _ 10</td>
<td>343</td>
<td>0.015</td>
<td>81</td>
<td>204</td>
<td>0.895</td>
<td>0.726</td>
<td>1.910</td>
</tr>
<tr>
<td>Red Lake Sample _ 11</td>
<td>562</td>
<td>0.002</td>
<td>792</td>
<td>2443</td>
<td>0.837</td>
<td>0.199</td>
<td>0.377</td>
</tr>
<tr>
<td>Red Lake Sample _ 12</td>
<td>369</td>
<td>0.004</td>
<td>29</td>
<td>1723</td>
<td>0.953</td>
<td>2.348</td>
<td>0.645</td>
</tr>
<tr>
<td>Red Lake Sample _ 13</td>
<td>271</td>
<td>0.012</td>
<td>27</td>
<td>109</td>
<td>0.851</td>
<td>1.360</td>
<td>1.118</td>
</tr>
</tbody>
</table>

The investigation shows that the hyperbolic model allows the CPB to absorb more strain energy. From the analysis presented in this Chapter it was seen that the hyperbolic model follows more closely stress-strain path of axial loaded samples as compared to the Young’s modulus strain path. With the calculated strain-energy of the material in compression, the next step is to determine the seismic source that would cause the CPB to fail due to exceeding its strain-energy.

Kaiser et al. (1997) found through study of monitored seismically active mines that the seismic energy is related to the scaled target distance of the event and the ground motion as follows:
\[
\log E_s \propto 2 \log R v
\]  
\textit{Equation 6-24}

where:

- \(E_s\) = Seismic Energy (MJ)
- \(R\) = scaled target distance (m)
- \(v\) = ground motion velocity (m/s)

Alternatively, if using the seismic moment of the event, the seismic energy is proportional to the following relationship is derived from Kaiser et al. (1997):

\[
\log E_s \propto 0.5 \log \left( \frac{7M_o^2}{16r_o^3} \right) - 1.16
\]  
\textit{Equation 6-25}

where:

- \(M_o\) = Seismic Moment (GN m)
- \(r_o\) = source radius (m)

So, after investigation of the axial tests, we can determine the strain energy density. This strain energy reflects the ability of the material to absorb energy by way of displacement within the elastic region of the material; it was found that the hyperbolic model allows for larger energy capacity. This energy absorption was related to seismic energy through study by Kaiser et al. (1997) and Hedley (1992). For CPB sill beams, the amount of seismic energy a sill beam can contain is based on the volume of the beam and the strain energy density. For designers this value of energy absorption can be the applied to site specific data for seismic moment and radius sources.

For example, consider a large event of 1.7 Nuttli magnitude event. The event occurs 15 m away from the CPB sill beam that measures 5 m wide x 4.5 m high, and runs for 10 m along the stope. The strain energy densities were the average for the Stillwater samples listed in Table 6-4. The seismic energy capacity for the given example can be determined in equations 6-21, 6-22 and 6-23; Table 6-5 list the results of the calculations.
Table 6-5: Seismic energy calculations

<table>
<thead>
<tr>
<th>Seismic Data</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Nuttli Magnitude</td>
<td>1.7</td>
<td></td>
</tr>
<tr>
<td>Seismic Moment</td>
<td>501.19</td>
<td>GN m</td>
</tr>
<tr>
<td>Radius to source</td>
<td>15</td>
<td>M</td>
</tr>
<tr>
<td>Seismic Energy</td>
<td>394.78</td>
<td>kJ</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sill beam</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Height</td>
<td>5</td>
<td>M</td>
</tr>
<tr>
<td>Width</td>
<td>5</td>
<td>M</td>
</tr>
<tr>
<td>Length</td>
<td>10</td>
<td>M</td>
</tr>
<tr>
<td>Volume</td>
<td>250</td>
<td>m(^3)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Strain Energy density</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>by Young's Modulus method</td>
<td>0.825</td>
<td>kJ/m(^3)</td>
</tr>
<tr>
<td>by Hyperbolic model method</td>
<td>1.618</td>
<td>kJ/m(^3)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Strain Energy of Beam</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>by Young's Modulus method</td>
<td>206.25</td>
<td>kJ</td>
</tr>
<tr>
<td>by Hyperbolic model method</td>
<td>404.5</td>
<td>kJ</td>
</tr>
</tbody>
</table>

It can be seen that the hyperbolic model is able to absorb the seismic energy source; however, the Young’s modulus method would fail under the 1.7 Nuttli magnitude event.

The strain energy of the backfill illustrates that the CPB has a significant capacity to sustain energy from a seismic event. It has been noted in Red Lake mine that the sill beam performs well in seismic events compared to the host rock. The above analysis explains the performance of the CPB in seismic environments.

### 6.5 Summary

The constitutive modeling chapter provides a new method in modeling the stress-strain response of CPB. It was found that the Mohr-Coulomb constitutive model does not reflect the axial loading path. A hyperbolic model (Duncan and Chang, 1970) model was found to suitably model the stress-strain path of the CPB under axial. Published strain softening relationships were too drastic in reducing the strength of CPB post-peak. A post-peak strain-softening model in which the cohesion is reduced was created to reflect experimental data.
Through the constitutive analysis it was found that a cohesion value of 32% is appropriate. This value is a 22% increase in suggested cohesion from the literature; the large cohesion value will positively affect the required UCS for stable sill beams.

The behaviour of CPB in mines with seismic events was investigated by comparing the strain-energy density of the material to the energy of seismic source. An example was provided to show the thought process: site-specific development is required.

The purpose of the constitutive model chapter is to understand how the material reacts to the imposed stresses. The findings will be applied to the stability of the sill beam once undercut and subject to imposed stresses and strains. The analytical and numerical modeling chapter will take the findings of the constitutive behaviour of CPB and apply to the sill beam.

**Chapter takeaways**

- For CPB, cohesion as a percentage of the UCS is between 30 and 35%;
- Mohr-Coulomb criteria is a poor approximation of CPB under axial load;
- CPB follows a hyperbolic stress-strain path to failure;
- Post peak stress-strain behaviour can be approximated by a linear decay of the cohesion values; and
- When using a hyperbolic stress-strain path, CPB has a large amount of strain energy that can be used to withstand seismic events.
7 ANALYTICAL MODEL

Building on the findings of the observational approach, the analytical analysis will attempt to quantify sill beam stability. The analytical analysis chapter will build equations to support findings of the observational chapter. The analytical equations will be developed from first principles or from published equations discussed in Chapter 2. The findings of this chapter will provide an understanding of analytical input properties and parameters. Most importantly, through the developed equations, this chapter will identify the critical failure modes for CPB sill beams.

7.1 Horizontal and Vertical Stresses Within Sill Beams

From work by Blight (1984); Mitchell, Olsen and Smith (1982); Handy (1985); Grice (2004); Aubertin et al. (2003); Pirapakaran and Sivakugan (2007); Fahey et al. (2009); and Singh, Shukla and Sivakugan (2011), an understanding of the behaviour of fill in long hole stopes is gained. The studies have some application to CPB sill beams, but have a limit to their applicability when the effect of cementation (material cohesion) is considered. A discussion on the development of stress within sill beams is presented below. Investigation into self-generated stress both vertical and the effect of closure is presented.

7.1.1 Vertical Stresses

The following equation was proposed by Blight (1984), and was built on the works of Reimbert and Reimbert (1976) and Terzaghi (1943). This equation is suitable in assessing vertical stress within the stope. Discussion on Blight’s model is discussed in Chapter 2. Blight (1984) proposes the following differential equation to assess vertical stress within stope:

\[ d\sigma_z = \left( \gamma \sin \beta - 2 \frac{\tau}{w} \right) \, dz \]

Equation 7-1

By integrating Equation 7-1 for the depth of the stope the following first order linear equation is used to calculate the lateral earth pressures:

\[ \sigma_z = \frac{w}{2K_o \tan \phi} \left( \gamma \sin \beta - \frac{2c}{w} \right) \left( 1 - e^{-\frac{2K_o \tan \phi \mu z}{w}} \right) \]

Equation 7-2
where:

\[ K_0 = \frac{c}{\gamma - K_o} \]

Similarly, the Terzaghi model considers cohesion in assessing vertical stresses and is defined as follows:

\[ \sigma_z = \frac{\gamma w - 2c}{2 \tan \phi} \left(1 - e^{-\frac{2K_o \tan \phi z}{w}}\right) \quad \text{Equation 7-3} \]

What can be seen by the Blight and Terzaghi approach is that the vertical stresses start as negative and attenuate as a negative function based on stope width, unit weight, cohesion, and friction angle. The negative stresses are in essence saying that arching does not occur in consolidated backfill, once fully drained and long conditions are assumed.

This is not to say that arching suggested in backfill by Mitchell, Olsen and Smith (1982); Handy (1985); Grice (2004); Aubertin et al. (2003); Pirapakaran and Sivakugan (2007); Fahey et al. (2009); Singh, Shukla and Sivakugan (2011) is to be ignored. The arching research is essential in the understanding and assessment of stresses during the placement and hydration of backfill or for unconsolidated fill. It is the researcher’s opinion that once the fill in a CPB is at long term conditions, the effect of arching are not significant.

To determine the vertical stress within the fill, the effective stress equation, per Terzaghi, Peck and Mesri (1996), is to be used:

\[ \sigma_z = \gamma z - u \quad \text{Equation 7-4} \]

where:

\[ u = \text{pore water pressure (kPa)} \]

A special case of vertical loads exists where catastrophic failure of the overlying sill beams occurs. The sequential failure of all CPB sill beams would cause an imposed surcharge vertical load. For the purposes of the analysis, the worst case scenario would occur: the material
would lose all cohesion and act as frictional material: arching of the failed material would then occur. Comparing the Blight (1984) and Terzaghi (1943) equations, the preference is to use the Terzaghi equation for this special case, as the state of the failed material (active, at reset, or passive) is not required. So, to determine the maximum surcharge load, an infinite height of fill is considered and is defined by Blight (1984) as follows:

\[ \sigma_{z(\text{max})} = \gamma h - \frac{\gamma w}{2\tan\phi} \]  

Equation 7-5

From the researcher’s experience at over a dozen UCF mines, the complete failure of consecutive sill beams has not occurred; no published literature of this catastrophic event exists to the researcher’s knowledge. The surcharge load is useful in the application of large cut-and-fill long hole stopes where unconsolidated material is placed above a prepared consolidated sill beam.

### 7.1.2 Horizontal Stresses

Since arching of the backfill is not present due to the consideration of cohesion within the Terzaghi and Blight analysis, the lateral earth pressure models suggested by Mitchell, Olsen and Smith (1982); Handy (1985); Aubertin et al. (2003); Pirapakaran and Sivakugan (2007); and Marcinysynchyn (1996) do not apply. The stresses generated within the fill will be caused by stope closure.

To determine stope closure, assumptions per Salamon (1968) will be considered. These stope closures were incorporated with backfill by Hedley (1992) but have the following restrictions:

- A single isolated stope is considered to be a two-dimensional thin slit;
- Backfill does alter the closure distribution within the stope;
- Closure is fully transferred to backfill; and
- Both the rock and backfill behaved as linear elastic materials.

The restrictions pose an issue as, Chapter 7 proved that CPB is not linear elastic, and further, in weak rock environments, the closure of stopes is due to yielding of the ground (see Chapter 5). The CPB non linearity can be accounted for at a later stage by incorporating the
loading of CPB by an elastic modulus that has a stress dependency. The plastic yielding of the stope walls will not be considered in the analytical models, it is an option more suited for numerical analysis.

The equation to determine the displacement at any point along a stope wall put forward by Salamon (1968) and applied to backfill by Hedley (1992) is as follows:

\[
w' = \left( \frac{4(1 - \nu^2)}{E'_t} \right) \left( 1 + \frac{\Delta H_c \sin \beta}{2D} \right) \left( \sqrt{\frac{H}{2}} - \Delta H_c^2 \right)
\]

Equation 7-6

where:

\[
\sigma_h = \frac{\nu D}{2} [(1 + K) + (1 - K) \cos 2\beta]
\]

\[w' = \text{amount of convergence (m)}\]

\[D = \text{depth below surface to center of stope (m)}\]

\[E'_t = \text{Elastic Modulus of rock (kPa)}\]

\[K = \text{ratio of the in-situ horizontal to vertical stress}\]

\[H = \text{Height of stope (m)}\]

\[\Delta H_c = \text{distance above or below the centerline of stope (m)}\]

To determine the amount of closure occurring over the height of a stope, the convergence equation needs to be integrated over the height of the fill with respect to the stope. For purposes of integration, the middle term of the convergence equation can be reduced to unity as it accounts for variation of stresses with depth and only applies to stopes with large heights. Of note, the elastic response of the stope walls that occurs during the mining of the CPB sill beam needs to be subtracted from the total convergence occurring when subsequent cuts are taken. To determine the average amount of convergence acting on a CPB sill beam the following equation is proposed:

\[
\Delta w_{\text{stope}} = \frac{2}{h_1 - h_2} \int_{h_1}^{h_2} w'_n \, d\Delta H_c - \frac{2}{h_1 - h_2} \int_{h_1}^{h_2} w'_0 \, d\Delta H_c
\]

Equation 7-7
where:

\begin{align*}
    h_1 &= \text{distance from top of CPB sill to centerline of stope} \\
    h_2 &= \text{distance from bottom of CPB sill to centerline of stope} \\
    w_n' &= \text{wall convergence for ‘n’ stope cuts} \\
    w_0' &= \text{original amount of wall convergence when mining CPB sill}
\end{align*}

The doubling of the convergence is to account for closure on both the footwall and hangingwall. Performing the integration in absence of accounting differential stress change along the stope, the total amount of convergence per CPB sill beam can be assessed as follows:

\[
\Delta w_{\text{stope}} = \frac{1}{h_1 - h_2} \left( 2(1 - \nu^2)\sigma_h \right) \left( \Delta H_c \left( \sqrt{\left( \frac{H}{2} \right)^2 - \Delta H^2_c} \right) - \left( \frac{H}{2} \right)^2 \tan^{-1} \left( \frac{\Delta H_c}{\sqrt{\left( \frac{H}{2} \right)^2 - \Delta H^2_c}} \right) \right)
\]

\[
- \Delta H'_c \left( \sqrt{\left( \frac{H'}{2} \right)^2 - \Delta H^2'_c} \right) - \left( \frac{H'}{2} \right)^2 \tan^{-1} \left( \frac{\Delta H'_c}{\sqrt{\left( \frac{H'}{2} \right)^2 - \Delta H^2'_c}} \right) \right|_{h_1}^{h_2}
\]

\text{Equation 7-8}

where:

\[H' = \text{Height of the stope at time of mining CPB sill beam}\]

A displacement profile of stope walls with arbitrary values is shown in Figure 7-1. From the assumptions of Hedley (1992), it can be seen that the CPB sill has no effect on the displacement profile; rather the CPB will take the displacement as applied stress.

Figure 7-1: Suggested stope wall displacement profile
Now that the profile of the convergence and the total amount of conversion per sill height is known, the stress within the stope can be calculated. For an initial pass, the following equation is proposed to determine the stress within the CPB sill beam:

$$\sigma_a = \frac{\Delta w_{stope}}{w} E_t$$  \hspace{1cm} \textit{Equation 7.9}

where:

$$\sigma_a = \text{axial stress within CPB sill (kPa)}$$
$$E_t = \text{tangential modulus of CPB (kPa)}$$

To be complete, the modulus should be of the form of the hyperbolic model as discussed in Chapter 6. With the introduction of the hyperbolic function the axial stress within the beam becomes:
The stress in the beam is paramount as the shear strength along the interface is a function of the applied stress. Further, as will be discussed in the failure modes, the amount of closure can cause a crushing failure of the sill beam.

7.2 Beam Equations

The beam theory approach to CPB sills has been explored by Stone (1993) and Jordan et al. (2003). The foundation of the theory is work performed by Euler-Bernoulli (Timoshenko and Gere, 1961). For beam mechanics, the type of beam is critical in determining the stresses and strains within. For CPB sill beams, conservatism is employed and the beam is considered simply support (Stone (1993), Jordan et. al (2003), Mitchell and Roettger (1989). However, due to the sill beam being affixed along its contact length, the fixed beam approach should be considered. A comparison between fixed beam and simple beam approach with respect to stress within CPB sill beams will be presented.

To note, with beam equations the tensile and flexural strength are required for analysis. The tensile strength will be assumed to be 0.2 of the compressive strength as determined from the testing presented in Chapter 4. The flexural strength will be assumed to be 13% of the compressive strength of CPB per Naik (2009).

For simple or fixed beams, the modes of failure are due to maximum deflection, shear, or bending forces.

7.2.1 Simple Beam

The simple beam analysis assumes that the end support cannot take a horizontal load or any moment loads; only a vertical reactionary force is provided by the abutment. The consideration of sill beams being simple beam supports poses problems: the simple beam hypothesis by Mitchell and Roettger (1989) contradicts the shear strength along the fixed support of the abutments presented by Mitchell and Roettger (1989)!

\[ \sigma_a = \frac{\Delta w_{stop}^e}{w} (1 - R_f S)^2 E_t \]  

\textit{Equation 7-10}
To determine the maximum flexural stress that can be supported by a simple CPB sill beam under self-load, the following equation is proposed (from Mott, 2002):

\[
\sigma_f = \frac{3}{4} \left( \frac{yw^2}{h} \right)
\]  \hspace{1cm} \text{Equation 7-11}

where:

- \(\sigma_f\) = flexural strength (kPa)
- \(h\) = height of sill beam (m)

The above flexural equation is identical to the tensile equation proposed by Jordan et al. (2003). This is likely due to an assumption by Jordan that tensile strength is the weakest part of compressive, flexural and tensile strengths.

In the absence of flexural testing values, the assumption by Jordan et al. (2003) that flexural strength will govern CPB sill beam stability during flexural failure will be used for analysis.

Figure 7-2 demonstrates that slender beams require the CPB to have a higher tensile/flexural strength; the graph has been normalized for unit weight. For a typical unit weight of paste backfill (20 kN/m\(^3\)), the required tensile values for 5 m wide by 5 m high is 75 kPa. These tensile values are less than the range of tensile tests performed in Chapter 4.

Research in this thesis has found that for a limited value of CPB testing, the ratio of tensile strength to UCS is approximately 1:5. Published literature suggests that the value should be \(1/10^{th}\) of the UCS (Stone, 1993; Tesarik et al., 2007; Hughes et al., 2006). It can be seen that the ratio has a large effect on the required UCS for flexural stability. The assumption that the tensile strength is \(1/10^{th}\) of the UCS introduces conservatism to the design. It should be noted that no Factor of Safety are applied to the flexural analyses; rather the use of the \(1/10^{th}\) factor for tensile strength is a pseudo Factor of Safety of 2.0 in comparison to the lab tests. The tensile strength required for a 5.0 m by 5.0 m stope with unit weight of 20 kN/m\(^3\) is 75 kPa; reasonable given the tested values in Chapter 4.
The Factor of Safety against flexural failure for a simple beam is as follows:

$$FS = \frac{4}{3} \left( \frac{\sigma_t h}{\gamma W^2} \right)$$  \hspace{1cm} Equation 7-12

The shear stress within the sill beam can be determined by the following formula:

$$\tau = \frac{\gamma hw}{2}$$  \hspace{1cm} Equation 7-13

The available shear strength within the beam is dictated by the Mohr-Coulomb failure envelope:

$$\tau = c + \sigma_n \tan \phi$$  \hspace{1cm} Equation 7-14
The friction is of two components, the normal friction that was shown above to be a function of the convergence of the stope, and the cohesion. Given that the analysis is for a simple beam, no convergence can be applied to the beam without making it statically indeterminate: no convergence will be applied. As such, in shear, the cohesion will provide the shear strength against shear failure. Figure 7-3 present a normalized plot of the required shear strength for shear stress stability.

![Figure 7-3: Normalized shear strength required for various sill beam heights](image)

It can be seen that a reciprocal relationship exists between the ratio of the height to width and the normalized required shear strength, as the beam becomes more slender, higher shear strength is required. The effect of the beam with respect to the percentage of cohesion with respect to the UCS has a large effect on the required UCS design strength. The common assumption that the cohesion is 25% of the UCS (Caceres, 2005; Tesarik et al., 2007) leads to an inherent Factor of Safety. The testing of both the tensile and cohesive strength of the material is
imperative to determine the Factor of Safety of the sill beam with respect to the UCS as is suggested by Stone (1993).

To determine the Factor of Safety against shear failure within a beam, the following equation is suggested:

\[
FS = 2 \left( c + \frac{\Delta w_{stope}}{w} E_t \tan(\phi) \right) \frac{\gamma h w}{y h w}
\]

Equation 7-15

Comparing shear failures to tensile failures, it appears that tensile failures are the limiting mode of failure for simple beam analysis.

7.2.2 Fixed Beam

The fixed beam analysis assumes that the abutments can take horizontal, vertical loads and moments about the support abutments. The principles of the beam formula are discussed by Mott (2003). The difference between a simple beam and a fixed beam in terms of a CPB sill beam is that the model incorporates the contact of a sill beam with the side walls.

The shear loads and shear strength equations are assessed in the same manner as the simple beam (Section 7.2.1).

The tensile loads for a fixed beam differ from the simple beam in that the moment distribution has both a positive and negative component. Further, the moments are less due to the restriction of the ends of the beam. Due to the lower moment within the beam, the tensile strength required for a stable fixed beam is less than that of a simple beam. The required tensile (flexural) strength for a fixed sill beam is determined by the following equation:

Fixed Beam Flexural Failure Equation:

\[
\sigma_f = \frac{1}{2} \left( \frac{\gamma w^2}{h} \right)
\]

Equation 7-16
Again, as was performed for the simple beam, it will be assumed per Jordan et al. (2003) that the tensile strength will be equal to the flexural strength. A normalized plot of the fixed beam analysis is shown in Figure 7-4.

Figure 7-4: Normalized plot of UCS vs. sill beam slenderness for fixed CPB sill beam

Comparing Figure 7-4 to Figure 7-2, it is observed that the fixed beam support requires a lower flexural strength value than that of the simple beam support. This is due to the fixed beam support reflecting the actual configuration of the CPB sill beam in the mine.

The Factor of Safety against fixed beam failure is as follows:

\[ FS = 2 \left( \frac{\sigma_{t} h}{\gamma W^2} \right) \]

Equation 7-17
7.2.3 Beam Displacement

Another method of assessing the strength of the beam stability is the amount of deflection at any point within a simple beam and comparing it to the maximum tolerable deflection. For the deflection at any point along the width of a simple support CPB beam and the maximum deflection are as follows:

$$\Delta y = \frac{-\gamma x}{12Eh^2}(w^3 - 2wx + x^3) \quad \text{Equation 7-18}$$

$$\Delta y_{\text{max}} = \frac{-5\gamma w^4}{384Eh^2} \quad \text{Equation 7-19}$$

where:

- $x =$ arbitrary distance along sill beam (m)
- $w =$ width of beam across stope (m)
- $E =$ tangential modulus of material (kPa)
- $h =$ height of sill beam (m)

Alternatively, the displacement at any point along a fixed sill beam is as follows:

$$\Delta y = \frac{-\gamma x^2}{24Eh^2}(w - x)^2 \quad \text{Equation 7-20}$$

$$\Delta y_{\text{max}} = \frac{-\gamma w^4}{384Eh^2} \quad \text{Equation 7-21}$$

A normalized plot of displacement comparing the fixed beam to the simple beam support is shown in Figure 7-5. From investigation of the figure, the fixed beam support limits the amount of vertical deflection within the beam, whereas large noticeable displacements are noticed in the simple beam displacement equation.

The displacement in spans was measured to 4.5 mm for a 5 m wide stope in cemented rock fill (CRF) by Tesarik et al. (2007). By inputting the data of the Tesarik Study (13.5 m
length, 4.5 m high beam, Modulus of Elasticity 1.31 GPa) the maximum calculated deflection for simple and fixed beams are 3.5 mm and 0.5 mm respectively. From this limited data, it would appear that the simple beam support reflect beams displacement. However, the Tesarik et al. (2007) model had a vertical cold joints on which measure displacement were found.

Figure 7-5: Normalized vertical displacement for sill beams

For failure of a beam due to vertical displacement, the maximum allowable deflection of a beam is commonly used in the civil engineering to determine serviceability of structures. ASCI (2006) recommend that the tolerable maximum deflection is 1/360 of the width for live loads and 1/240 for combined live and dead loads. For the purposes of the study the maximum tolerable deflection will be 1/240 of the width of the stope.

**Displacement Failure Criterion:**

\[ \Delta y_{max} > \frac{W}{240} \]  
*Equation 7-22*

From the above, the resulting Factor of Safety against displacement style failures is considered to be the following:
7.2.4 Crushing Failure

Beam crushing occurs due to the convergence of the stope. The resistance against failure is based on the stiffness and strength of the fill and the applied load due to convergence. Jordan et al. (2003) state that numerically this type of failure is possible; however they have not experienced it at Stillwater Mine. Further investigation into crushing was performed by Mitchell and Roettger (1989). In section 7.1.2, the amount of closure within a stope was calculated and can be applied to crushing failures. The issue with crushing failure is that it is highly dependent on the input variables of the stress field and the modulus of elasticity. With the large amount of parameters and inputs necessary a parametric plot of the crushing failure is not possible.

To determine the amount of convergence a stope can withstand before failure occurs can be defined as follows:

**Convergence Failure Criterion:**

\[
\Delta W_{\text{allowable}} = \frac{w \sigma_c}{E_t} \tag{Equation 7-24}
\]

where:

\[
E_t = \left[1 - \left(\frac{2c \cos \phi + 2 \sigma_3 \sin \phi}{1 - \sin \phi}\right)\left((\sigma_1 - \sigma_3)_{\text{ult}}\right)^{-1}\left(\frac{\sigma_c}{(\sigma_1 - \sigma_3)_f}\right)^2\right] E_i \tag{Equation 7-25}
\]

To determine the Factor of Safety against crushing failure, Equation 7-26 compares the amount of expected convergence due to the elastic response of the stope; the Factor of Safety equation is as follows:

\[
FS = \frac{W}{240 \Delta y_{\text{max}}} \tag{Equation 7-23}
\]
Sill beam failure can also occur due to stope wall movement and this is caused by beam failure as discussed by Jordan et al. (2003). Beam relaxation is the opposite of beam crushing and occurs when outward displacement of the stope walls occurs due to relaxation of the stresses surrounding the stope. The failure is due to the loss of normal force around the stope, and the beam will fail due to detachment similar to a sliding, rotational, or shear failure and not a failure type in its own right. The relaxation of stresses around a stope that is actively mining is not common, rather the relaxation is caused intentionally for distressing purposes, or due to mining of adjacent stopes once the initial stope is depleted. Relaxation of stope walls would not be a common occurrence in an actively mined area and is not being investigated as part of this study.

### 7.3 Mitchell Theory

Mitchell theory equations were developed through engineering first principles and observations of centrifuge models. The summarizing document of Mitchell’s work can be found in Mitchell and Roettger (1989). The work determined equations for caving, rotational, flexural, and sliding failures.

#### 7.3.1 Caving

The caving failure and its theory are discussed in further detail in the literature search of Chapter 3. A caving failure is a failure associated with poor quality fill as no outside factors are associated with the failure. It is strictly associated with unit weight and tensile strength of the material. Caving failure is independent of the surcharge loads or lateral confinement. Caving failures occur in stopes that have a high height to width ratio.

Mitchell and Roettger (1989) assume that caving failure would extend to a stable arch of a height that is half the width of the stope, and proposes that tensile failure will occur when the following condition is met:

\[
FS = \frac{\Delta w_{\text{allowable}}}{\Delta w_{\text{stope}}}
\]

Equation 7-26
where:

\[ \sigma_t = \text{tensile strength (kPa)} \]

The error, or conservative approach, in the above formula is that the resistance of the fill to the caved material is provided by a linear line across the length of the stope and not over the failure surface, assumed by Mitchell to be an arch equivalent to the perimeter of a half-circle diameter is that of the stope width. This is incorrect, as the failure will be resisted by the tensile strength over the rupture length, \( \frac{1}{2} \) of the perimeter of the circle; dropping of ‘\( \pi \)’ in the denominator is due to the consideration of both the rupture and failure surface being curvilinear. Updating Mitchell’s equation is the following caving failure formula:

*Caving Failure Criterion:*

\[ \gamma W \geq \frac{8}{\pi} \sigma_t \] \hspace{1cm} \text{Equation 7-27}

The Factor of Safety for caving failure is defined as follows:

\[ FS = \frac{\gamma W}{4\sigma_t} \] \hspace{1cm} \text{Equation 7-29}

### 7.3.2 Sliding

Sliding, also referred to as shear failure, sill beam shear failure is discussed in detail as part of the literature search of Chapter 3. The shear/sliding failure discussed by Mitchell and Roettger (1989) and Jordan et al. (2003) differs from that of the beam theory shear failure as the failure is along the contacts between the rock and CPB, for ease of reading shear failure along the interface will be referred to as sliding failure. To define sliding failure: sliding failure is the
downward vertical movement of a sill beam, as a unit, along the HW/CPB/FW contacts. Shear failure proposed by Jordan et al. (2003) is defined as follows:

\[ \tau = \frac{1}{2} \gamma W \]  \hspace{1cm} Equation 7-30

The one-half factor in the equation is an approximation for the coefficient of lateral earth pressure. No attempt to measure the shear strength is provided by Jordan et al. (2003); the approach presented contains numerous simplifications and ignores the properties of the hangingwall and backfill interface. Jordan et al. (2003) argued that since tensile failure dominates, the need to quantify shear strength is unnecessary, however for a complete sill beam design the shear strength of the material must be considered.

Mitchell and Roettger (1989) present that sliding failure will occur when the following inequality is satisfied:

\[ \gamma h > 2 \frac{\tau}{\sin \beta} \left( \frac{h}{W} \right) \]  \hspace{1cm} Equation 7-31

That is to say, failure occurs when the weight of the sill beam is greater than the shear strength of the material. The shear strength is a function of the cohesion and normal force applied across the interface. In a typical Mitchell analysis the normal force acting along the interfaces is assumed to be zero, and the mobilized shear strength is provided solely by the cohesion. Further per Caceres (2005); Hughes et al. (2006); Tesarik (2007); and Swan & Brummer (2001) the cohesion is assumed to be ¼ of the UCS of the material.

The issue in assigning these parameters is that the parameter values are a function of the interface and not of the rock or the CPB. Nasir and Fall (2008) present findings on the interface strength and suggest they take the form of 0.65 of the weaker of the rock or CPB. This relationship was presented for limited data, and should be thought of as a constant that can be defined by the designer. As shown in Chapter 5, the belief that the cohesion is ¼ of the UCS does not appear valid. Looking at taking the normal force to be equal to zero, section 7.1
provides a way of estimating the normal force by finding the incremental wall closure over the stope length. So there are three issues with the sliding failure proposed by Mitchell and Roettger: the interface values need to be defined, cohesion is not necessarily \( \frac{1}{4} \) of the UCS and the normal force across the face should be quantified. In addressing these shortcomings, it is proposed sliding will occur when the following inequality is satisfied:

\[
\gamma h > 2 \left( C_1 c + \frac{\Delta W_{\text{stope}}}{W} E_t \tan(C_2 \phi) \right) \left( \frac{h}{W} \right)
\]

Equation 7-32

where:

\( C_1 \) = cohesion interface multiplier (range between 0.25 and 1.0)
\( C_2 \) = friction interface multiplier (range between 0.25 and 1.0)

The Factor of Safety against sliding is assessed as follows:

\[
FS = \frac{2 \left( C_1 c + \frac{\Delta W_{\text{stope}}}{W} E_t \tan(C_2 \phi) \right)}{\gamma W \sin \beta}
\]

Equation 7-33

### 7.3.3 Flexural

Flexural failure is discussed in detail in Chapter 3. The flexural failure as presented by Mitchell and Roettger (1989) differs to beam theory in that it accounts for both the tensile and compressive strength in determining the stability of the CBP beam, whereas Beam Theory relies on solely on the flexural strength of the CPB.

Flexural failure occurs in stopes that have a comparatively small height to width ratio. The flexing of the beam causes the beam to fail in tension due to self-weight loading. Mitchell and Roettger (1989) proposed that flexural failure will occur using a simple fixed beam formula as follows:
**Mitchell Flexural Failure Criterion:**

\[
\left( \frac{W}{h} \right)^2 \geq \frac{2(\sigma_t + \sigma_c)}{\gamma h}
\]

*Equation 7-34*

This equation assumes that the sill beam is fixed to the sidewall and no movement is to occur, thereby implying that flexural failure will occur before shear failure. Mitchell and Roettger (1989) describe this beam as a simply supported beam and is a more conservative approach to a fixed supported beam as it requires greater strength to fail in flexure.

An issue with the flexural formula is that the cohesion and flexural strength are required. It is assumed that the tension is 1/10 of UCS per Tesarik et al. (2007); Bretchel et al. (1999); Swan and Brummer (2001) and Hughes et al. (2006). Chapter 5 and 6 show that this is not necessarily the case; however, if we assume these values to be true, a comparison between the Mitchell and Roettger (1989) (Equation 7-34) and Beam Theory per Timoshenko and Gere (1961) (Equations 7-12 and 7-16) can be performed. Figure 7-6 compares the compressive strength required for a stable sill beam based on the Mitchell and Roettger (1989) and Beam Theory equations for a nominal CPB with unit weight of 20 kN/m³ and 5 m high sill beam.

It can be seen that the Mitchell and Roettger’s equation parallels closely the fixed beam support. Since Mitchell and Roettger (1989) will fail at a lower strength than both beam theory equations, the beam theory equations will be used in the place of the Mitchell and Roettger (1989) flexural equations.
7.3.4 Rotational

Rotational failure occurs when a sill beam detaches from the hangingwall contact and rotates about a fixed pivot at the lower footwall contact. Details and discussion on rotational failure are provided in Chapter 3.

The limitation with the rotational failure is due to geometry: the sill beam will not be able to rotate out of the stope if the distance between the lower footwall and upper hangingwall is longer than the width of the stope from lower footwall to lower hangingwall. Figure 7-7 is a graphical representation of the geometric restraints of rotational failure. A consideration was allowed for cases where deformation or crushing of the sill beam equal to 5 cm is allowed for rotation to occur.
The rotational equation provided by Mitchell (1989a) is as follows:

\[ \sigma_v + \gamma h \geq \frac{\sigma_t h^2}{2W(W - h \cot \beta)\sin^2 \beta} \quad \text{Equation 7-35} \]

Caceres (2005) found that there is an inherent Factor of Safety of two in this equation. Further Caceres investigated rotational failure and found that the shear strength between the hangingwall and sill beam contact should be included in the analysis. Caceres presents rotational failure as follows:

Rotational Failure Criterion:

\[ \gamma h \geq \frac{\sigma_t h^2 + 2\alpha \tau h W \sin^2 \beta}{W(W - h \cot \beta)\sin^2 \beta} \quad \text{Equation 7-36} \]
where:

$$\alpha = \text{percentage of hangingwall in contact with backfill}$$

Using relationships for normal strength and applying those to shear strength using the coefficients determined for sliding failure, the Factor of Safety for against rotational failure is presented as follows:

$$FS = \frac{\sigma_t h + 2\alpha \left( C_1 c + \frac{\Delta W_{\text{stopo}}}{W} E_t \tan(C_2 \phi) \right) W \sin^2 \beta}{\gamma W(W - h \cot \beta) \sin^2 \beta} \quad \text{Equation 7-37}$$

### 7.4 Simulation of Analytical Equations

Equations for rotational, sliding, caving, flexural, maximum tolerable displacement, shear failure and crushing failure have been presented. What is not known is which is the critical failure method. Jordan et al. (2003) and Stone (1993) suggest that flexural failure dominate sill beam behaviour. Mitchell and Roettger (1989) suggest that rotational failure dominates sill beam behaviour, and Stone counters that rotational failure is only valid in certain stope geometries and this is supported within this chapter.

The issue with all the analyses presented in literature is that they do not account for the mining environment and are presented for homogeneous fill: material properties and ground stresses are considered constant. The only variable that changes in the analysis is the required UCS for the material with cohesion and tension equal to $\frac{1}{4}$ and $\frac{1}{10}$ of the UCS respectively, and the friction angle set to 30 degrees. This is not realistic of the variation in fill within a mine, the different mining environments that CPB UCF is applied to and the properties of the backfill. A more thorough analysis of the critical failure methods needs to be performed.

It is proposed that a multi-variable statistical analysis will identify the critical failure method based on analytical equations. A Monte-Carlo simulation will be performed with an assumption that the explicit variables have a uniform distribution: the variables will be constrained to a range determined from laboratory studies, field observations, and from published values.
Given the distribution of all variables, random values are generated within the distribution and used to compute the factor of safety. Some combination of the random values will yield the lowest factor of safety which then defines the critical failure mode. From the simulation, the most critical failure methods can be determined by their total number of occurrences over the simulation. Further analysis can be applied to the failure methods based on mine stresses, amount of closure, UCS of the material, size of stope, etc… For the analysis to be thorough, 40,000 simulations will be performed to determine the critical failure methods of CPB beams.

To perform the analysis, the variables and their ranges must be defined. Two types of variables exist: explicit and implicit. The explicit variables need to be defined, whereas implicit variables are determined based on the values of the explicit variables. A table of the explicit variables, the range of the variables and source of the variables is presented in Table 7-1. The implicit variables are computed for the explicit variables, the relationships for the explicit variables are shown in Table 7-2.

Assuming a uniform distribution, the values of the explicit variables are computed as follows:

\[ \text{value} = \min + r(\max - \min) \]

where:

\[ r = \text{random number uniformly distributed between 0 and 1} \]

For the case of applying a maximum surcharge to the analysis, if the random number generated is between 0.75 and 1.0, the surcharge will be applied. This is introduced as a bias towards assessing the beam stability over its own self-weight and not strictly the surcharge. For wall convergence, since a scatter of displacement values will come about due to the random variables, the convergence will be calculated based on only one CPB beam undercut by a new stope that has dimensions equal to the sill beam.
Table 7-1: Range of explicit variables for analytical simulation

<table>
<thead>
<tr>
<th>Explicit Variable</th>
<th>Symbol</th>
<th>Range</th>
<th>Units</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>CPB</em></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UCS</td>
<td>$\sigma_c$</td>
<td>500-6000</td>
<td>kPa</td>
<td>Chapter 4</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>$E_t$</td>
<td>10-1100</td>
<td>MPa</td>
<td>Chapter 5</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>$\phi$</td>
<td>10-40</td>
<td>$^\circ$</td>
<td>Chapter 6</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>$\gamma_p$</td>
<td>15-23</td>
<td>kN/m$^3$</td>
<td>Chapter 5</td>
</tr>
<tr>
<td>Failure Ratio</td>
<td>$R_f$</td>
<td>0.75-1.0</td>
<td></td>
<td>Chapter 6</td>
</tr>
<tr>
<td><em>Rock</em></td>
<td></td>
<td></td>
<td></td>
<td>Pakalnis and Associates (2011)</td>
</tr>
<tr>
<td>UCS</td>
<td>$\sigma_r$</td>
<td>10-150</td>
<td>MPa</td>
<td>Pakalnis and Associates (2011)</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>$E_r$</td>
<td>10-80.0</td>
<td>GPa</td>
<td>Pakalnis and Associates (2011)</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>$\gamma_R$</td>
<td>25-32</td>
<td>kN/m$^3$</td>
<td>Pakalnis and Associates (2011)</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>$\nu$</td>
<td>0.33</td>
<td></td>
<td>Pakalnis and Associates (2011)</td>
</tr>
<tr>
<td><strong>Stress Parameters</strong></td>
<td></td>
<td></td>
<td></td>
<td>Pakalnis and Associates (2011)</td>
</tr>
<tr>
<td>Mine Depth</td>
<td>$D$</td>
<td>400-3200</td>
<td>m</td>
<td>Chapter 5</td>
</tr>
<tr>
<td><strong>Stope Dimensions</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Height</td>
<td>$h$</td>
<td>2.5-8.0</td>
<td>m</td>
<td>Arbitrary</td>
</tr>
<tr>
<td>Width</td>
<td>$W$</td>
<td>3.0-10.0</td>
<td>m</td>
<td>Arbitrary</td>
</tr>
<tr>
<td>Dip</td>
<td>$\beta$</td>
<td>45-90</td>
<td>$^\circ$</td>
<td>Chapter 5</td>
</tr>
<tr>
<td><strong>Reduction Factors</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Interface Cohesion</td>
<td>$C_1$</td>
<td>0.5-1.0</td>
<td>( )</td>
<td>Nasir and Fall (2008)</td>
</tr>
<tr>
<td>Interface Friction Angle</td>
<td>$C_2$</td>
<td>0.5-1.0</td>
<td>( )</td>
<td>Nasir and Fall (2008)</td>
</tr>
<tr>
<td>Hangingwall Shear Strength</td>
<td>$\alpha$</td>
<td>0.0-1.0</td>
<td>( )</td>
<td>Full range variable</td>
</tr>
</tbody>
</table>
Table 7-2: Implicit variables for analytical simulation

<table>
<thead>
<tr>
<th>Implicit variable</th>
<th>Symbol</th>
<th>Relationship</th>
<th>Units</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CPB</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cohesion</td>
<td>$c$</td>
<td>$f(\phi, \sigma_c)$</td>
<td>kPa</td>
<td>Chapter 4</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>$\sigma_t$</td>
<td>0.1 - 0.35 ($\sigma_c$)</td>
<td>kPa</td>
<td>Chapter 6</td>
</tr>
<tr>
<td>Maximum Surcharge Load</td>
<td>$\sigma_z$</td>
<td>$\gamma_p W - \gamma_p W (2 \tan \phi)$</td>
<td></td>
<td>Terzaghi (1943)</td>
</tr>
<tr>
<td><strong>Stress Parameters</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical Stress</td>
<td>$\sigma_v$</td>
<td>$\gamma R D$</td>
<td>kPa</td>
<td>Arjang &amp; Herget (1997)</td>
</tr>
<tr>
<td>K Factor</td>
<td>$K$</td>
<td>$7.44 D^{0.198}$</td>
<td>()</td>
<td>Arjang &amp; Herget (1997)</td>
</tr>
<tr>
<td>Horizontal Stress</td>
<td>$\sigma_h$</td>
<td>$K \sigma_v$</td>
<td>kPa</td>
<td>Arjang &amp; Herget (1997)</td>
</tr>
</tbody>
</table>

Factor of safety equations and variable ranges were entered into a computer script that was created for this research. Forty thousand iterations of sill beam stability were calculated and sorted. A failed sill beam is considered to be a simulation in which the factor of safety was less than or equal to unity. Critical design equations for the sill beam that were analyzed are as follows: closure based on Young’s Modulus, closure based on hyperbolic model, rotational, sliding, beam shear, simple flexural failure, fixed flexural failure and caving. Equations used for calculating the factor of safety are those discussed in the above sections.

Two separate analyses are presented: one with closure associated with a linear stress-strain path to peak strength, the other utilizes the hyperbolic stress-strain path to peak failure.

For the linear stress-strain path, the simulation of 40,000 discrete inputs found that closure failure was the most critical limiting design criteria for stable beams, and for cases where the beam had a factor of safety below 1.0 (failure). Secondary limiting design and failure type was shear beam; tertiary limiting design and failure types were simple flexural type failures.
Figure 7-8 shows the distribution of the limiting design criteria for the Monte Carlo simulations; these are the distribution for stable sill beams. It can be seen that the strain in the sill beam due to closure is an issue that needs to be taken into account. Mitchell’s studies (1982, 1989a, 1989b with Roettger, 1989) fail to address the role of closure on sill beam stability. In cases of stable beam configurations, the closure stress is useful in that it mobilized the frictional component of the shear strength. In previous studies of sill beam stability this normal force was assumed to be zero; it can be seen that this is not a valid assumption as closure has a large effect on stability of a sill beam.

What occurs during closure failure is that the sill beam is compressed horizontally until compressive failure occurs. The difficulty is assessing this type of failure has been in the ability to quantify the amount of closure and normal forces applied on a sill beam. This research attempts to quantify the amount of closure expected on a sill beam and assumes that the closure will act fully on the sill beam in a uniform manner. In doing this, a normal force is applied to the sill beam increasing the frictional component of the shear strength to a limit. As discussed in Hughes et al. (2006) and Krauland and Stille (1993) there is a limit to the amount of closure a CPB sill beam can accommodate. It was found in these studies that narrow stopes with comparatively stiff CPB modulus fail. Investigation of the Monte Carlo analysis, for the purposes of engineering paste backfill, show that 95% of all Young’s Modulus closure failures can be prevented when the following inequality is satisfied:

\[
\frac{\sigma_c}{E_t} > 0.01W
\]  

*Equation 7-38*
When looking at sill beams that have failed (FS<1.0), as shown in Figure 7-8: Limiting design criteria for stable sill beams.
Figure 7-9, the closure failures still dominate. This is a very important finding as previous studies have neglected the role of closure on the stability of sill beams, and when incorporated into the simulation, closure failures dominate.

A deformable fill with high strength can prevent crushing failures. In designing backfill the inequality in Equation 7-38 should be satisfied if crushing failure is an issue.
Rotational failures occurred 1% of the time in stopes that had dips less than 65 degrees and height to width ratio less than 0.5; the Monte Carlo analysis of rotational failure is in agreement with Figure 7-7. Of note with the rotational analysis, all sill beams failed when a full surcharge is applied to the sill beam.

Flexural failures occurred 3% of the time in the Monte Carlo analysis. The flexural failures occurred for slender beams with height to width ratios less than 0.4 with widths greater than 7.25 m; UCS values ranged between 500 and 2000 kPa. Increasing the aspect ratio of the sill beam greatly reduces the probability of failure of a sill beam for flexural failure.

Shear failure within the sill beam occurred 1% of the time. Inspection of the criteria of shear failures shows that shear failures occur when the height to width ratio close to unity.
Chapter 6 found that CPB does not follow a linear stress-strain path. As such, a separate analysis as that presented above was considered with the difference being closure failure was associates with a hyperbolic stress-strain path. An alternative to the analysis above is now presented where the linear stress-strain closure failures are filtered out.

Without the linear closure, the probability of failure of sill beams given the inputs of Table 7-1 is 5%. In comparison, with the linear-stress strain closure failures, the probability of failure is 43%. The limit design criteria by percentage are shown in Figure 7-10.

Figure 7-10: Filtered Monte Carlo limit design results

It can be seen that the beam shear is the highest critical failure. Beam shear were limit criteria for sill beams that had a high height to width ratio. When comparing the limiting state to design to sill beams that have failed (Figure 7-11) it can be seen that beam shear is not critical with respect to failure.
In comparison to the full Monte Carlo analysis, only 2,316 of the 40,000 cases failed. 16,923 cases failed in the full analysis. Again, similar to the Monte Carlo analysis that considered linear stress-strain paths, crushing failures dominate. In the filtered case, the ‘hypo-closure’ dominates the cases that have a factor of safety less than 1.0. One finding from the filtered case is that simple flexural failure is found to be second most critical (25% of all failed cases. This finding supports the theory presented by Stone (1993) and Pakalnis et al. (2005).

The main takeaway from the filtered results, is that even with the hyperbolic closure model (one that tolerates more closure), the closure model still dominates the cases where sill beams have failed. This finding again counters the published results that both flexural (Stone, 1993) and rotational (Mitchell and Roettger, 1989) dominate sill beam stability. Closure must be taken into account for sill beam analysis.
From the Monte Carlo analysis the main findings are as follows:

- Closure is major failure mode over a range of mining environments;
- By having a CPB with a UCS/Elastic modulus ratio greater than 0.01 of the width of the stope eliminates 95% of all closure failures;
- Flexural failure dominates outside of closure for sill beams with a low height to width ratio; and
- Shear failures dominate for stopes with no closure and sill beams with a height to width ratio near unity.

Designs of sill beams should ensure that all failure methods are addressed through adequate fill properties or stope geometries that increase the factor of safety or eliminate the possibility of failure.

### 7.5 Effect of Ground Support on Sill Beam Stability

Ground support within a sill mat is of four forms:

- Pre-installed stand-up rebar bolts;
- Ground support installed after undercutting of sill;
- Pre-installed ground support in wall rock; and
- Surficial support after undercut of sill beam, typically in the form of welded wire mesh.

From the above the vertical installed tendons are to prevent the vertical release of paste backfill once undercut. The purpose of the rebar bolts it to provide confinement per Williams et al. (2001) and to contain cold joints. The idea of confinement will be addressed within this section; cold-joints will be addressed in a section unto itself. The ground support installed after the undercut of the sill is to contain possible wedge or slab failures.

In addition to the vertical tendons, the wall bolts or ‘shear paddles’ are part of the support system and their purpose is to resist shear and rotational movement of the sill beam. Angled rebar bolts can be installed upon undercutting as opposed to ‘shear paddles’ based on the design guidelines of individual mines.

The welded wire mesh contains material that could potentially release between the vertical tendons; or in mining terms “loose” material.
The support provided by the ground support elements will be investigated in how they relate to stability of the CPB sill beam. The ground-support role in providing confinement, shear strength, and containment of failed material will be investigated.

### 7.5.1 Stand-up Rebar Support

The inclusion of stand-up rebar bolts was an ad-hoc decision of underground miners at Lucky Friday in an effort to replace timber support or cable slings and welded wire mesh (Blake, 2013). The purpose of the rebar support was to contain the sill beam when undercut. The main advantage of the stand-up rebar bolts was the stope turnaround time that went from 3 weeks with cable sling/ welded wire mesh or timber lagging to 1.5 days with the stand-up rebar bolts (Blake, 2013).

Stand-up bolts are plated at the top and bottom of the bolt with the bolting plate of the same strength as the rebar. It was reported by mine operation that the bolt heads are rarely seen or able to be re-plated or used as screen anchorage once the beam is undercut. This is an operational issue as screen is difficult to install, and additional support must be added to allow for screen installation.

The stand-up rebar bolts can hold a capacity of the bolt themselves. For the standard #6 and #7 rebar used for stand-up bolt the ultimate tensile strength of the bolt is 18 and 24.5 tonnes respectively. The working-idea with the stand-up bolts is that they are to be installed passed the neutral horizontal-axis of the beam (centroid) such that the upper portion of the bolt is in compression and able to contain the potential loose material. Williams et al. (2007) argued that this would cause a ‘cone-of-compression’ within the sill beam that can approach the ultimate tensile strength of the bolt. Pells (2008) studies found that although the bolts do take tensile load, the amount of compression provided by a rock bolt is negligible

The role of the stand-up rebar is the support of the dead-weight wedge as provided by Pakalnis and Associates (2008). The rebar is to retain possible loose material and prevent differential displacement and separation within the beam.
7.5.2 Frictional Support

Split sets are used exclusively for post-undercut support of sill beams. Split sets are common, with no known cases of Swellex in CPB. The split set bolts rely on frictional forces around the surface area of the bolt. The frictional bolt rely on having significant anchorage past a failure surface, the anchorage capacity of the frictional bolt is referred to as the bond strength. The frictional support in CPB beam serves two purposes: anchorage of screen to the back and containment of the loose blocks within the sill beam.

The bond strength values for CPB should be determined per ASTM 4435 (ASTM, 2013d). The tests should be performed such that only a portion of the bolt is tested for bond strength such that the bolt does not yield. A summary of bond-strength values for CPB is provided in Chapter 5.

7.5.3 Liner Support

To the researcher’s knowledge, the use of liners in operation is restricted to welded wire mesh, or chain-link mesh (screen). Studies have been performed by Donovan et al. (2007) on the effectiveness of geofabrics as liners. From discussion with technical services at mine site, shotcrete is not used as a liner support due to difficulty in mobilizing shotcrete plants to stopes and the associated costs. Investigation of the support provided by welded wire mesh and chain-link and how it relates to CPB is provided.

The function and capacity of screen in ground support is discussed by Pakalnis and Ames (1983). The screen acts as a passive support and provides no true confinement to the sill beam. Pakalnis and Associates (2008) lists that the support capacity for 5 cm x 5 cm, 6 gauge welded wire mesh and 2.5 cm x 2.5 cm, 9 gauge, galvanized chain link support capacity (bagged strength) of 33 kN and 32kN respectively. That is to say that the chain-link will hold a bagged loose material to its capacity until the screen is past it loading limit.

For CPB, with a nominal unit weight of 20 kN/m$^3$ and a standard bolting pattern of 1.2 m x 1.2 m, the use of mesh in the ground support system, allows for containment of 1.1 m of detached material before failure of the screen occurs. This is significant in that, the screen can contain approximately 20% of the sill beam. In cases of cold-joint detachment, the use of screen
adds to the support capacity of the system of bolts, tensile strength of the material and shear strength along the edges of the cold-joint.

It must be stated that the use of mesh as support must be done in conjunction with ground support and that the weight of material held to the screen is to be added to the support of the vertical tendon supports.

7.5.4 Shear Supports

Shear support are tendon supports that are installed horizontal to sub-horizontal within the sill beam and anchored within the hangingwall and footwall. Shear paddles are installed in the hangingwall of the stope to prevent against rotational failure. The shear paddles consist of fully encapsulated 1.65 m long, #6 rebar, installed 0.9 m into sound rock, 75 mm wide straps are tied to the paddles. The bolts are installed 0.9 m above the working floor. The installation of shear-paddles is shown in Figure 7-12.

The shear paddles improve both the shear strength of the interface contacts and, in effect, reduce the exposed span of the CPB beam. To determine the shear strength provided by shear-paddles, it is assumed that the rebar has a shear value of 50% of the ultimate tensile strength (ASTM A615, 2013c). For the #6 rebar shear paddles described above on a 1.2 m linear spacing, the shear resistance along the contacts is increased by 80 kN per linear meter along the length of the stope.

The shear-paddles provide resistance to failure along the contact between the hangingwall and footwall. Macassa mine installs shear-paddles in cases where interface are weak or steeply dipping. It is recommended that shear-paddles are installed in stopes with poor adhesion between the contacts or in stopes that satisfy the rotational criteria determined in Section 7.3.4.
7.6 Discontinuities Within the Sill Beam

A cold-joint is defined as a discontinuous interface between the successive layers of a paste pour. It is assumed that the worst-case scenario for failure due to bending, buckling (closure) or tensile failure is a horizontal cold joint. Figure 7-13 shows the cold joint and the penetration of ground support past the discontinuity. An argument is made that cold joint in literature was presented by Nasir and Fall (2008) and Fall and Nasir (2010), in which direct shear tests were performed on laboratory samples of CPB and rock or CPB interfaces. The role of cold joints on sill beam stability has not been presented in literature.

If a cold-joint slab is considered as dead weight wedge, (Pakalnis, Brady, Hughes, Ouchi, Caceres and MacLaughlin 2007), the strength along the cold-joint interface will rely solely on the tensile strength of the material. The resistance to failure will be through the shear strength generated along the wall interfaces. Further, the effect of ground support will be investigated as part of the cold-joint analysis.
The first analysis for cold joint stability attempts to determine the required tensile strength for a continuous cold joint that extends across the full length of the stope. For the analysis, the failure surface looks at failure over a unit-length (nominal 1.2 m, equal to typical bolt spacing in UCF CPB). The equation to determine the required tensile strength is as follows:

\[ \sigma_t = \gamma h_c \]

*Equation 7-39*

where:

- \( \sigma_t \) = tensile strength of CPB (kPa)
- \( \gamma \) = unit weight of fill (kN/m\(^3\))
- \( h_c \) = height of cold joint (m)

The tensile strength required is independent of the width of the stope and is strictly controlled by the depth of the cold joint and the unit weight of the material. The value of tension of CPB cold-joints has not been assessed in any known study. It is suggested that a reduction factor be applied to the indirect tensile strength of the material as tested in Chapter 5. From the results present in Chapter 5, the tensile strength of CPB was found to be, on average, 20% of the
UCS of the material; the tensile strength of the cold-joint should be $\frac{2}{10}(UCS)(C_1)$ Where $C_1$ is a constant of unknown values, but suggested to be $\frac{1}{4}$ as a conservative value.

To assess the role of the shear strength in stabilizing the cold joints, a Mohr-Coulomb failure surface will be assessed. The values of cohesion will be set to 35% of the UCS and the friction angle will be $20^\circ$ as found in Chapter 6. The required mobilized shear strength to contain the cold joint block is as follows:

\[
\tau = \gamma W \quad \textit{Equation 7-40}
\]

\[
\tau = c + E_t \frac{\Delta W}{W} \tan \phi \quad \textit{Equation 7-41}
\]

where:

- $W =$ Width of stope (m)
- $c =$ cohesion (kPa)
- $E_t =$ Elastic Modulus of CPB (kPa)
- $\Delta W =$ stope closure (m)
- $\phi =$ friction angle of CPB

From the equation above, two factors play into the role of shear strength stabilizing a cold joint: the amount of cohesion required given no normal strength along the wall interface; and the amount of closure required given no cohesion along the wall interface.

The amount of cohesion required to stabilize the cold joint is simply the width of the stope multiplied by the unit weight of the material. Note, that if the assumption is made that both walls will provide shear strength (per Caceres, 2005) the required cohesion is reduced by half. Using results from Chapter 6, the cohesion can be estimated to be 35% of the UCS of the material. For cohesion along the side walls a reduction factor is to be applied to the intact value of cohesion. A reduction factor of 0.5 of the material cohesion is suggested based on work by Fall and Nasir (2010). For design, must consider that cohesion will act on both sidewalls and should be accounted in calculations. Although a factor of 0.5 is suggested, testing should be performed on interfaces to determine the actual reduction factor.
The amount of closure required to stabilize the cold joint is determined by the following equation:

\[
\Delta W = \frac{\gamma W^2}{E_t \tan \theta}
\]  

Equation 7-42

It can be seen that the relationship is inversely proportionate to the elastic modulus, and the squared power of the width of the stope. To be explicit: stiff fill and narrow stopes are preferred (to a limit, see Hughes et al., 2006).

The effect of the ground support is considered as per Pakalnis et al. (2007), in considering the support of the capacity of support element over the length of the sill beam. Now, a #6 Dywidag an ultimate tensile capacity of 185 kN and a SS39 split set bolt has a ultimate tensile capacity of 140 kN (Pakalnis et al., 2007): the bolt breaking will not be an issue; the issue is the critical bond strength (the depth of embedment of the bolt past the cold joint). The bond strength of the bolt past the cold joint will be assessed; but first, the required support pressure needed to stabilize the cold joint will be analyzed. Once the required pressure is known, the available support pressure provided by the support can be determined. For the purposes of the support pressure analysis it will be considered that the tensile strength along the cold joint and shear strength of the wall rock interface is nil. As found in Chapter 5, the spacing of support is typically 1.2 m x 1.2 m, this bolting pattern will be used in the analysis and all bolts are to be plated such that stripping failures are not possible. The required support pressure to contain cold joints is provided by the following:

\[\text{Required Support Pressure} = \gamma h_c\]

The available support pressure from the ground support system utilizing frictional support is provided by:

\[
\text{Available Support Pressure (kPa)} = \begin{cases} 
\frac{W}{B_s B_{sl}} (\Delta B_s (B_L - h_c)) & 0 < (\Delta B_s (B_L - h_c)) < S_{ult} \\
\frac{WS_{ult}}{B_s B_{sl}} & (\Delta B_s (B_L - h_c)) > S_{ult} \\
0 & (B_L < h_c)
\end{cases}
\]
where:

\[ B_{sw} = \text{Bolt spacing along width of stope (m)} \]
\[ B_{sl} = \text{Bolt spacing along length of stope (m)} \]
\[ \Delta B_s = \text{Bond Strength of bolt (kN/m)} \]
\[ B_l = \text{Bolt Length (m)} \]
\[ S_{ult} = \text{Ultimate strength of bolt (kN)} \]

Using suitable values of bond strength listed in Chapter 5 for SS 39 friction bolts (nominal 40 kN/m), the critical depth of embedment of the bolt based on a breaking strength of 140 kN for a SS39 bolt (Pakalnis et al., 2007) is 2.5 m. From Chapter 5, typically the bolt lengths of SS39 when employed are less than the critical bond strength: bond failure is an issue when using friction type bolts. Stand-up rebar bolts are plated at the collar and the terminus of the bolt (see Lucky Friday Design), the bolt plates are of equal strength to the rebar bolt in a proper design. For the purpose of design, when stand-up rebar bolts are used the breaking strength of the bolt/plate should be substituted for the bond strength values used in frictional support calculations.

A factor of safety analysis was performed using the available bond strength equations and the weight of the released cold joint block. A factor of safety (FS) of 1.0 was employed to determine the critical depth of the cold-joint to cause failure due to lack of support pressure; the analysis is presented in Table 7-3 and Figure 7-14.
Table 7-3: Critical depth of cold joint for bolting patterns (FS= 1.0, γ= 20kN/m$^3$)

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Bond Strength (kN/m)</th>
<th>Bolt Pattern (m x m)</th>
<th>Bolt Density (1/m$^2$)</th>
<th>Bolt Length (m)</th>
<th>Critical depth of cold joint (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS39</td>
<td>40</td>
<td>0.9 x 0.9</td>
<td>1.23</td>
<td>1.8</td>
<td>1.25</td>
</tr>
<tr>
<td>SS39</td>
<td>40</td>
<td>1.0 x 1.0</td>
<td>1.00</td>
<td>1.8</td>
<td>1.08</td>
</tr>
<tr>
<td>SS39</td>
<td>40</td>
<td>1.2 x 1.2</td>
<td>0.69</td>
<td>1.8</td>
<td>0.72</td>
</tr>
<tr>
<td>SS39</td>
<td>40</td>
<td>1.5 x 1.5</td>
<td>0.44</td>
<td>1.8</td>
<td>0.41</td>
</tr>
<tr>
<td>SS39</td>
<td>40</td>
<td>0.9 x 0.9</td>
<td>1.23</td>
<td>2.4</td>
<td>1.61</td>
</tr>
<tr>
<td>SS39</td>
<td>40</td>
<td>1.0 x 1.0</td>
<td>1.00</td>
<td>2.4</td>
<td>1.44</td>
</tr>
<tr>
<td>SS39</td>
<td>40</td>
<td>1.2 x 1.2</td>
<td>0.69</td>
<td>2.4</td>
<td>0.96</td>
</tr>
<tr>
<td>SS39</td>
<td>40</td>
<td>1.5 x 1.5</td>
<td>0.44</td>
<td>2.4</td>
<td>0.56</td>
</tr>
<tr>
<td>20mm Dywidag</td>
<td>180</td>
<td>0.9 x 0.9</td>
<td>1.23</td>
<td>1.8</td>
<td>1.80</td>
</tr>
<tr>
<td>20mm Dywidag</td>
<td>180</td>
<td>1.0 x 1.0</td>
<td>1.00</td>
<td>1.8</td>
<td>1.80</td>
</tr>
<tr>
<td>20mm Dywidag</td>
<td>180</td>
<td>1.2 x 1.2</td>
<td>0.69</td>
<td>1.8</td>
<td>1.80</td>
</tr>
<tr>
<td>20mm Dywidag</td>
<td>180</td>
<td>1.5 x 1.5</td>
<td>0.44</td>
<td>1.8</td>
<td>1.80</td>
</tr>
<tr>
<td>20mm Dywidag</td>
<td>180</td>
<td>0.9 x 0.9</td>
<td>1.23</td>
<td>2.4</td>
<td>2.40</td>
</tr>
<tr>
<td>20mm Dywidag</td>
<td>180</td>
<td>1.0 x 1.0</td>
<td>1.00</td>
<td>2.4</td>
<td>2.40</td>
</tr>
<tr>
<td>20mm Dywidag</td>
<td>180</td>
<td>1.2 x 1.2</td>
<td>0.69</td>
<td>2.4</td>
<td>2.40</td>
</tr>
<tr>
<td>20mm Dywidag</td>
<td>180</td>
<td>1.5 x 1.5</td>
<td>0.44</td>
<td>2.4</td>
<td>2.40</td>
</tr>
</tbody>
</table>

Figure 7-14: Critical depth of cold joints for bolting pattern (FS =1.0; γ= 20kN/m$^3$)

It can be seen that the use of 2.4 m long SS39 on a 1.0 x 1.0 m pattern is approximating the 1.8 m stand-up rebar bolts. It must be stated that without the application of cohesion or
tensile strength to the analysis, any cold joint with a depth exceeding the critical depth outlined in Table 7-3 would cause sill beam failure.

Consolidating the above analyses of tension, cohesion and ground support, the strength of the sill beam to resist cold joint release can be defined as follows:

\[
\text{Mobilized strength against cold joints per unit length (kN/m)} =
\begin{cases} 
\frac{W}{B_{sw}B_{sl}} (\Delta B_s (B_L - h_c) + \left( \frac{35}{100} C_1 \sigma_c + E_t \frac{\Delta W}{W} \tan \phi \right) h_c + \frac{2}{10} C_2 \sigma_c W) & \text{for } 0 < (\Delta B_s (B_L - h_c) < S_{ult} \\
\frac{W S_{ult}}{B_{sw}B_{sl}} + \left( \frac{35}{100} C_1 \sigma_c + E_t \frac{\Delta W}{W} \tan \phi \right) h_c + \frac{2}{10} C_2 \sigma_c W & \text{for } (\Delta B_s (B_L - h_c) > S_{ult} \\
\left( \frac{35}{100} C_1 \sigma_c + E_t \frac{\Delta W}{W} \tan \phi \right) h_c + \frac{2}{10} C_2 \sigma_c W & \text{for } (B_L < h_c)
\end{cases}
\]

Equation 7-44

where:

\( \sigma_c \) = Unconfined compressive strength of CPB (kPa)

\( C_1 \) = \frac{\text{Interface cohesion}}{\text{internal cohesion}}

\( C_2 \) = \frac{\text{Interface tensile strength}}{\text{internal tensile strength}}

To determine the mobilized strength against cold joint failure numerous tests need to be performed as part of a laboratory or field study. The research presented for cold joint stability is novel in that it analyses the stability of cold joints by incorporating ground support, tensile strength of interfaces, and mobilized shear strength of the sidewall/CPB interface. Further an analysis is presented with laboratory and field values incorporated. The values of the cohesion and tensile reduction factors have yet to be tested. It is suggested that lab testing similar to Nasir and Fall (2008) and Fall and Nasir (2010) be carried out to verify these values.

7.7 Chapter Summary

The analytical chapter investigated the stability of the sill beam through published equations and equations derived from first principles. Investigation into the role of ground support and the stability of discontinuities within the sill beam were analyzed.
Stresses within the sill beam were analyzed to calculate both self-generated and applied stress to the sill beam. It was found that self-generated stresses are largely insignificant for UCF CPB sill beams in comparison to those of long-hole stopes as the height of sill beam does not allow significant stresses to develop. The inclusion of cohesion into the equations presented by Blight (1984) and Terzaghi (1943) eliminates self-generated horizontal stresses within the sill beam. The effect of the fill being active, at-rest, or passive is inconsequential. It was found that stresses due to lateral closure of stope walls were the most significant.

This study developed methods to determine the amount of elastic closure applied to sill beams due to successive mining. The expected elastic closure was then transferred through the sill beam by ways of a linear stress-strain path and hyperbolic stress strain path derived in Chapter 6. This application allowed for quantification of failures due to excessive closure.

The inclusion of closure had a large effect on traditional sill beam stability approaches presented by Mitchell and Roettger (1989), Jordan et al. (2003) and Stone (1993). The traditional approaches ignored the effect of outside stresses; traditional methods allowed simple parametric studies. By incorporating outside stress influences, the number of variables increased and simple parametric assessments were not possible. To accommodate the analysis of sill beam stability with the added variables, a probability of failure analysis using a Monte Carlo Simulation was performed. Two separate analyses were performed: linear-elastic closure stresses and hyperbolic stress-strain paths. In both cases, closure stresses were found to be critical failure for sill beams with a factor of safety less than 1.0. The probability of failures for the two analyses was 43% and 5% for the linear elastic and hyperbolic stress-strain cases respectively. Closure stresses must be taken into account for a sill beam analysis. It was found that stability can be achieved in 95% of the closure sill beam failures by having a ratio of UCS to elastic modulus greater than 1%.

Ground support was shown to have an effect on rotational instability and the support of cold-joints within sill beam. From a numerical modeling standpoint, ground support was found to not affect closure type failures. From data gathered as part of Chapter 5, it was determined that given common bolting patterns stand-up rebar can support discontinuities to a height equal
to nominal height of the bolt. Frictional support, based on findings presented in Chapter 5 can support cold joints to depths shown in Figure 7-14.

The analytical assessment chapter built on the findings of Chapter 5 and 6 in determining sill beam stability. The findings of this chapter will be applied to numerical modeling techniques to understand the behaviours of sill beams over time during the mining cycle.

**Chapter takeaways**

- Closure of stope walls can be approximated by an elastic difference equation;
- Closure failure dominates CPB stability;
- Flexural dominates in stopes with low closure;
- Discontinuities in the sill beam are stabilized by sidewall cohesion, tensile strength along discontinuity and, to a limit, bolt capacity;
- From a design standpoint soft fill with high strength is desired.
8 NUMERICAL MODEL ANALYSIS

The purpose of using a numerical model is to reflect the behavior of sill beams for all geometries and stress conditions. Previous study by Caceres (2005) determined stability for CRF sill beams using beam theory equations. Modifications of this work will allow for the incorporation of CPB behaviour into the numerical analysis.

The numerical model verifies the analytical results. What separates the numerical analysis from an analytical approach is the ability to measure and monitor the development of strains and failed elements during the convergence of the stope.

The numerical model analysis will investigate the failure mechanics of the sill beam discussed in Chapter 7. Once the model can verify the failure modes, further investigation of sill beam performance in varying ground conditions will be studied. The sill beam performance study will provide an understanding of required strengths for the various sill beams.

The role of ground support in sill beam design will be investigated through numerical modeling. The performance of the ground support and its increase to the stability of sill beams will be investigated.

The numerical model Chapter is essential in answering the research questions presented. It is hoped that through the numerical analysis an understanding of the CPB response to ‘time-based’ loading will be understood. Further, the role of ground support will be defined through numerical means, verifying the findings of previous chapters. The ability of parametric study and ability to reflect various ground conditions allows the numerical model to be an essential tool in determining the required strength for various widths of sill beams. The findings of this chapter will assist the answering the primary research question: what is the required paste fill strength for an arbitrary sill beam width?

8.1 Model Construction

The analytical model found that closure is the dominant mode of failure in a high stress environment, while, flexural failure was found to be the dominant failure in low stress environments. Further, shearing failure occurred in configurations of stiff fill, weak hangingwall
and high height to width ratios. From these failure modes it was deduced that two models needed to be constructed: a simple beam model and a model to investigate the integration of a sill beam within a mine environment.

The constitutive behaviour and the material properties of CPB will be discussed within this section.

8.1.1 Beam Theory Model

The beam theory numerical model is simple in its approach. The steps for the sill beam model are as follows:

1) User defines CPB properties, sill width and height.
2) CPB placed in the model, allowed to settle under self-weight.
3) Undercut removed.
4) Convergence applied, if applicable, model cycled until equilibrium or unstable conditions are achieved.

A snapshot of the grid generation for a 7 m wide, 3.5 m sill beam with 85 degree hanging wall is shown in Figure 8-1

The grid and material properties are defined as user-input variables. Once the model equilibrates, the total convergence is applied incrementally over 10,000 calculation steps. Failure is defined as when a model is non-converging. For simple flexural failure analyses, the interface between the host rock and the CPB is turned to zero shear strength. The lowest grid point on hangingwall and footwall contacts is fixed in the X and Y direction: this is not a realistic assumption. The simple model will be used to verify failures of the analytical model for sill beam stability.
8.1.2 Mine Setting Model

The mine wide setting model was initially constructed to reflect a representative cross-section of an underhand cut and fill mining stope. The model was similar to the beam theory model only that four sill beams are incorporated in the model and a large external boundary representative of the host rock was constructed. After initial investigation, the mine-setting model offered no additional benefit to the simple beam model; and therefore external loading and stress can be explicitly applied to the beam model to reflect a mine setting.

8.1.3 Constitutive Behaviour and Model Properties

The model parameters are based on the ranges provided in Chapter 7. The range of values will be kept constant where possible and judgment of the researcher will be applied in determining the inputs for the model. In cases of parametric assessment, the researcher will choose appropriate properties to allow for comparable assessments.

A strain-softening approach was applied to the backfill discussed in Chapter 6. The constitutive model was created within the limitations of the FLAC code. Cohesive values were
stepped down in accordance with Chapter 6; but were set to be 0.65 of the original cohesion value. Angle of friction values were kept constant. Absent of data, the tensile strength was reduced according to the cohesive value per Caceres (2005); Swan and Brummer, (2001); and Hughes et al. (2006) in accordance with FLAC software in which the strain-softening properties are associated only with plastic strains.

8.2 Validation of Analytical Model

In order to build confidence in the numerical analysis, verification of the analytical model failures will be analyzed. The failures analyzed are from the Monte Carlo simulation (Chapter 7) in which the failure method was critical and had a factor of safety less than 1.0. No analyses of displacement or caving are investigated as they were not seen as critical.

8.2.1 Convergence

The convergence model is the most critical model from the analytical simulation performed in Chapter 7. The simple beam model will be run with the following properties to determine if failure occurs.

<table>
<thead>
<tr>
<th>Trial</th>
<th>UCS (kPa)</th>
<th>Friction Angle</th>
<th>Ratio of Initial Cohesion/UCS</th>
<th>Ratio of Tension/UCS</th>
<th>Emod (MPa)</th>
<th>Surcharge</th>
<th>Depth (m)</th>
<th>σv/σv</th>
<th>Height (m)</th>
<th>Width (m)</th>
<th>Stope Dip</th>
<th>Amount of Closure (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4380</td>
<td>27</td>
<td>0.31</td>
<td>0.32</td>
<td>633</td>
<td>No</td>
<td>2385</td>
<td>1.60</td>
<td>4.06</td>
<td>3.32</td>
<td>58</td>
<td>0.023</td>
</tr>
<tr>
<td>2</td>
<td>1841</td>
<td>13</td>
<td>0.4</td>
<td>0.34</td>
<td>633</td>
<td>No</td>
<td>2636</td>
<td>1.56</td>
<td>2.75</td>
<td>5.90</td>
<td>83</td>
<td>0.082</td>
</tr>
<tr>
<td>3</td>
<td>5501</td>
<td>27</td>
<td>0.31</td>
<td>0.15</td>
<td>870</td>
<td>No</td>
<td>3063</td>
<td>1.52</td>
<td>4.40</td>
<td>7.79</td>
<td>80</td>
<td>0.102</td>
</tr>
</tbody>
</table>

The output of the convergence model verification is found in Appendix 5. From investigation of the figures, the convergence models fail in a classic shear pattern, or ‘hour-glassing’ shear banding. From the analysis, the numerical model verifies the analytical failures. This verification builds confidence in the numerical model; the numerical model agrees with the previously published analytical equations.
8.2.2 Rotational

Rotational failures, according to the analytical model, occur in strong paste beams with low height to widths ratios with stope dipoles less than approximately 75 degrees. Three failed scenarios from the analytical model are selected to verify the numerical model’s ability to simulate rotational failure; the model analyses are shown in Table 8-2.

Table 8-2: Rotational failures for numerical model verification

<table>
<thead>
<tr>
<th>Trial</th>
<th>UCS (kPa)</th>
<th>Friction Angle</th>
<th>Ratio of Initial Cohesion/UCS</th>
<th>Ratio of Tension/UCS</th>
<th>Emod (MPa)</th>
<th>Surcharge</th>
<th>Depth (m)</th>
<th>σn/σv</th>
<th>Height (m)</th>
<th>Width (m)</th>
<th>Stope Dip</th>
<th>Amount of Closure (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1118</td>
<td>28</td>
<td>0.3</td>
<td>0.24</td>
<td>402</td>
<td>123.22</td>
<td>2643</td>
<td>1.56</td>
<td>2.57</td>
<td>8.88</td>
<td>63</td>
<td>0.017</td>
</tr>
<tr>
<td>2</td>
<td>1186</td>
<td>36</td>
<td>0.25</td>
<td>0.15</td>
<td>764</td>
<td>112.49</td>
<td>1171</td>
<td>1.84</td>
<td>3.45</td>
<td>9.46</td>
<td>69</td>
<td>0.007</td>
</tr>
<tr>
<td>3</td>
<td>2138</td>
<td>23</td>
<td>0.33</td>
<td>0.18</td>
<td>463</td>
<td>248.73</td>
<td>1645</td>
<td>1.72</td>
<td>3.92</td>
<td>9.66</td>
<td>63</td>
<td>0.037</td>
</tr>
</tbody>
</table>

Numerical model output of the rotational failure can be found in Appendix 5. Investigation of the figures shows that a tensile failure surface is developed between the lower footwall contact and upper footwall contact. This indicates that an active wedge is developed and the resistance is along a defined plane. Stabilization of the active block requires ground support to prevent movement along defined failure plane. It can be concluded that the numerical model can model rotational failure.

8.2.3 Flexural

Flexural failure occurs in stopes with low height to width ratios and low CPB UCS. Closure was found not to be of concern. Three flexural failures from the analytical model were selected to verify the numerical model; the model properties and parameters are shown in Table 8-3.
Table 8-3: Flexural failures for numerical model verification

<table>
<thead>
<tr>
<th>Trial</th>
<th>UCS (kPa)</th>
<th>Friction Angle</th>
<th>Ratio of Initial Cohesion/UCS</th>
<th>Ratio of Tension/UCS</th>
<th>Emod (MPa)</th>
<th>Surcharge</th>
<th>Depth (m)</th>
<th>$\sigma_t / \sigma_s$</th>
<th>Height (m)</th>
<th>Width (m)</th>
<th>Stope Dip</th>
<th>Amount of Closure (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>511</td>
<td>17</td>
<td>0.37</td>
<td>0.15</td>
<td>443</td>
<td>None</td>
<td>1820</td>
<td>1.68</td>
<td>2.56</td>
<td>9.11</td>
<td>49</td>
<td>0.018</td>
</tr>
<tr>
<td>2</td>
<td>965</td>
<td>22</td>
<td>0.34</td>
<td>0.12</td>
<td>258</td>
<td>189.02</td>
<td>1362</td>
<td>1.78</td>
<td>2.55</td>
<td>9.42</td>
<td>52</td>
<td>0.012</td>
</tr>
<tr>
<td>3</td>
<td>682</td>
<td>16</td>
<td>0.38</td>
<td>0.15</td>
<td>442</td>
<td>None</td>
<td>1247</td>
<td>1.81</td>
<td>2.56</td>
<td>9.78</td>
<td>80</td>
<td>0.03</td>
</tr>
</tbody>
</table>

The outputs from the numerical model are shown in Appendix 5. By investigation of the numerical model, a tensile failure develops vertically at the mid-point of the span. The large displacement near the bottom of the stope is caused by plastic flow of material at failure: this is a limitation of the model. Apart from a classical ‘flexural breakthrough’ type failure, the numerical model verifies failure for flexural failures.

8.2.4 Shear

Shear failures occurred in the analytical model in cases where the height to width ratio was near unity or greater; low values of Elastic Modulus and closure were also present. Representative failures were selected from the analytical simulation analysis, the failures properties and parameters listed in Table 8-4.

Table 8-4: Shear failures for numerical model verification

<table>
<thead>
<tr>
<th>Trial</th>
<th>UCS (kPa)</th>
<th>Friction Angle</th>
<th>Ratio of Initial Cohesion/UCS</th>
<th>Ratio of Tension/UCS</th>
<th>Emod (MPa)</th>
<th>Surcharge</th>
<th>Depth (m)</th>
<th>$\sigma_t / \sigma_s$</th>
<th>Height (m)</th>
<th>Width (m)</th>
<th>Stope Dip</th>
<th>Amount of Closure (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1210</td>
<td>32</td>
<td>0.28</td>
<td>0.2</td>
<td>105</td>
<td>No</td>
<td>931</td>
<td>1.92</td>
<td>7.02</td>
<td>9.95</td>
<td>77</td>
<td>0.014</td>
</tr>
<tr>
<td>2</td>
<td>863</td>
<td>30</td>
<td>0.29</td>
<td>0.27</td>
<td>411</td>
<td>No</td>
<td>515</td>
<td>2.16</td>
<td>7.84</td>
<td>9.11</td>
<td>80</td>
<td>0.01</td>
</tr>
<tr>
<td>3</td>
<td>971</td>
<td>24</td>
<td>0.32</td>
<td>0.26</td>
<td>133</td>
<td>No</td>
<td>465</td>
<td>2.21</td>
<td>6.39</td>
<td>7.7</td>
<td>52</td>
<td>0.008</td>
</tr>
</tbody>
</table>

Outputs of the numerical model can be found in Appendix 5 for shear failures. Investigation of the shear failures show that the entire stope block moved. The sliding of the block is more of a shear failure described by Mitchell and Roettger (1989) as opposed to shear failure described by Mott (2002). Further investigation of the shear failure shows that the maximum shear stress occurs at the abutment of the sill beam. In analytical models, shear failure is in absence of interface contacts and the beam fails in shear per beam theory. For numerical
models, the interface acts as a plane of weakness due to the development of shear stress within the beam. The incorporation of the interface promotes failure along the interface surface. This failure causes the block sliding. It can be concluded that the numerical model fails for shear failure; different failure mechanics take place due to the model set-up.

The constructed numerical model proved to reflect the results of the analytical models based on the models proposed by Stone (1993); Caceres (2005); Mitchell and Roettger (1989) and Jordan et al. (2003). The numerical model reflects the findings analytical model and has the advantage of being able to incorporate the material properties and parameters of the required input for a sill beam analysis.

8.3 Design Curve

A goal of this research was to develop a design curve for necessary CPB UCS strength for a given span width. The analytical model findings demonstrated that a large range of properties and parameters create numerous stable and unstable configurations. The large range of possible input parameters and their effect on stability do not allow for a general design curve: site specific design curves are necessary. However, a research questions is what is the necessary strength of paste backfill required for an arbitrary span? To answer this question given the non-unique solution, nine distinct design numerically derived curves will be constructed of UCS vs. Span width. The nine numerical models were created to best represent the ground conditions that are conducive to UCF mining. The design curves that will be created are similar to those of Pakalnis et al. (2005) and those of the Stillwater case study (Chapter 5). Discussions between the curves will be presented.

The nine distinct curves that will be modeled are to reflect the stress regimes (high, normal and low) and soft, medium and stiff CPB. To reflect the stress regimes, closure will be applied to the stope walls per convergence model. For the stiffness of the fill, the minimum, mean and maximum per range listed in Chapter 7 will be used. Table 8-5 summarizes the ranges used for the design curves.
Table 8-5: Scenarios for numerical design curves

<table>
<thead>
<tr>
<th>Trial</th>
<th>Elastic Modulus (MPa)</th>
<th>Amount of Closure (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>500</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>1100</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
<td>0.05</td>
</tr>
<tr>
<td>5</td>
<td>500</td>
<td>0.05</td>
</tr>
<tr>
<td>6</td>
<td>1100</td>
<td>0.05</td>
</tr>
<tr>
<td>7</td>
<td>10</td>
<td>0.15</td>
</tr>
<tr>
<td>8</td>
<td>500</td>
<td>0.15</td>
</tr>
<tr>
<td>9</td>
<td>1100</td>
<td>0.15</td>
</tr>
</tbody>
</table>

The issue with the numerical analysis is the possible range of variables that can be justified. To simplify the analysis the following properties and parameters will be kept constant. Justification of the values used is provided where possible.
Table 8-6: Numerical model properties and parameters

<table>
<thead>
<tr>
<th>Variable</th>
<th>Symbol</th>
<th>Range</th>
<th>Units</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPB</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UCS</td>
<td>$\sigma_c$</td>
<td>variable</td>
<td>kPa</td>
<td>Design control</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>$E_t$</td>
<td>10, 500,1100</td>
<td>MPa</td>
<td>Varied for analysis</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>$\varphi$</td>
<td>$f(\sigma_c, c)$</td>
<td>$^\circ$</td>
<td>Chapter 6</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>$\gamma_p$</td>
<td>18</td>
<td>kN/m$^3$</td>
<td>Chapter 5</td>
</tr>
<tr>
<td>Cohesion</td>
<td>$c$</td>
<td>0.35 UCS</td>
<td>kPa</td>
<td>Chapter 5</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>$\sigma_t$</td>
<td>0.2 ($\sigma_c$)</td>
<td>kPa</td>
<td>Chapter 5</td>
</tr>
<tr>
<td>Maximum Surcharge Load</td>
<td>$\sigma_z$</td>
<td>0.0 kN/m</td>
<td></td>
<td>Found not critical</td>
</tr>
<tr>
<td>Rock</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UCS</td>
<td>$\sigma_r$</td>
<td>125</td>
<td>MPa</td>
<td>R5, Pakalnis (2011)</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>$E_r$</td>
<td>80.0</td>
<td>GPa</td>
<td>Pakalnis (2011)</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>$\gamma_R$</td>
<td>28</td>
<td>kN/m$^3$</td>
<td>Pakalnis (2011)</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>$\nu$</td>
<td>0.33</td>
<td></td>
<td>Pakalnis (2011)</td>
</tr>
<tr>
<td>Stress Parameters</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical Stress</td>
<td>$\sigma_v$</td>
<td>$\gamma_R D$</td>
<td>kPa</td>
<td>Arjang &amp; Herget (1997)</td>
</tr>
<tr>
<td>K Factor</td>
<td>$K$</td>
<td>7.44D$^{0.198}$</td>
<td>( )</td>
<td>Arjang &amp; Herget (1997)</td>
</tr>
<tr>
<td>Horizontal Stress</td>
<td>$\sigma_h$</td>
<td>$K\sigma_v$</td>
<td>kPa</td>
<td>Arjang &amp; Herget (1997)</td>
</tr>
<tr>
<td>Mine Depth</td>
<td>$D$</td>
<td>-400, 800, 3000</td>
<td>m</td>
<td>Low, medium, high stress</td>
</tr>
<tr>
<td>Stope Dimensions</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Height</td>
<td>$h$</td>
<td>4.5</td>
<td>m</td>
<td>Typical observation (Chapter 5)</td>
</tr>
<tr>
<td>Width</td>
<td>$W$</td>
<td>variable</td>
<td>m</td>
<td>Design control</td>
</tr>
<tr>
<td>Dip</td>
<td>$\beta$</td>
<td>80</td>
<td>$^\circ$</td>
<td>Rotational failure possible</td>
</tr>
<tr>
<td>Reduction Factors</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Interface Cohesion</td>
<td>$C_1$</td>
<td>0.65</td>
<td></td>
<td>Nasir and Fall (2008)</td>
</tr>
<tr>
<td>Interface Friction Angle</td>
<td>$C_2$</td>
<td>0.65</td>
<td></td>
<td>Nasir and Fall (2008)</td>
</tr>
<tr>
<td>Hangingwall Shear Strength</td>
<td>$\alpha$</td>
<td>0.65</td>
<td></td>
<td>Nasir and Fall (2008)</td>
</tr>
</tbody>
</table>

The numerical design curves are created by determining the required strength for stability given an arbitrary width. The widths of the stope will vary between 2 m and 10 m in increments of 0.5 m. Material strengths will be varied by 250 kPa before a stable configuration is achieved; refinements of 25 kPa will be performed until a numerically stable configuration is achieved. In total 72 design points will be created for the design curves.
Figure 8-2 shows the required UCS for increasing span width. The shape of the curve is similar to those presented by Pakalnis et al. (2005) and Stone (1993). The sudden increase in required strength at the 7.5 m span width is due to the failure method changing from a flexural failure to a rotational failure. From the low stress analysis (no closure) it is concluded that the stability of the beam can be approximated using beam theory equations discussed in Chapter 7.
Transitioning from low stress to medium stress regime, Figure 8-3 shows the required UCS for increasing span widths. The shape of the curves is no longer that of beam theory equations (shown in Chapter 7) and follows, in theory, the findings of Hughes et al. (2006), and Krauland and Stille (1993). The effect of the 5 cm of confinement changes the failure modes: crushing failure dominates. The soft fill \((E=10 \text{ MPa})\) allows the material to accommodate the strain before reaching failure and is similar in trend to that of the low stress regime.

The fill with Elastic Modulus of 500 MPa vary significantly from the low stress regime. This fill experiences crushing failure between 4 m and 8 m. Between 2 m and 4 m, the fill fully fails due to crushing and is kept stable from high normal forces at the boundary. These large normal forces provide frictional support to shear strength; the stability of these failures is described in Silo Theory (Reimbert and Reimbert, 1976; and Mitchell, Olsen and Smith (1982). This stope is stable due to post-peak strength criteria. Although numerically stable, a fully failed shear strength mobilized stope is undesirable in the field. A fully mobilized paste column can only be considered marginally stable and would likely fail with any additional load or loss of confinement.
The stiff fill (1100 MPa) has undesirable performance in the medium stress regime. The fill has large enough stiffness that post-peak conditions are difficult to achieve. As such high strength fills are required between 2 and 8 m. Note, the analysis is capped at 6000 kPa as this represents the limit of CPB strength found in Chapter 4. At larger spans, the beam enters a rotational failure mode and the large displacements causes large stresses within the sill beam requiring stronger fill.

The findings of the medium stress analysis are as follows:

- Soft fill is desirable as the beam can accommodate large strain before entering post-peak conditions;
- Closure plays a large role on stability of stopes, supporting findings of Chapter 7;
- Sill beams can be numerically stable due to the shear strength of post peak conditions; and
- Stiff fills require high strengths to accommodate large strains.

The medium stress analysis demonstrates that closure cannot be ignored when performing analysis on sill beam stability. The high stress fill analysis (Figure 8-4) furthers this statement.

Figure 8-4: UCS vs. span width (high stress)
The high stress analysis demonstrates that soft fill is desirable for sill beam stability. The upwards trend between 4 m and 2 m is due to crushing failures. Flexural failures dominate for the soft fill between 4 and 10 m. Low strengths requirements for soft fills are reflective of the increased confinement and shear strength (eliminating rotational failure); flexural failure dominates at the wider width with high stress. The flexural failure is not present at wide widths in the low and medium stress regimes, as hangingwall detachment allows for rotational failure. The high closures does not allow for hangingwall detachment.

The medium stiffness fill curve can be separated into two portions: closure failures between 2 and 5 m, and rotational/flexural failures from 5 m onwards. The failure at 2 m is a complete failure of the material and the fill is kept stable due to post-peak shear strength behaviour. The medium stiffness fill behaves identical to the stiff fill in the 2m stope.

The stiff fill experiences complete failure and is supported by post-peak conditions between 2 and 6 m. Crushing failures dictate stability between 6 and 8 m; afterwards flexural/rotational failure dictates stability. The poor performance stiff fills in high stress areas underlines the importance of CPB stiffness.

The numerical design charts (Figure 8-2 through Figure 8-4) illustrate the importance on including closure in analyses. Soft fills are able to accommodate strains, whereas stiff fills tend to fail for similar closures/strains. For wide or low stress stopes, the failure modes follow Mitchell and Roettger (1989) analysis of sill beams. In narrow stopes, compressional failures govern stability when closure is applied. The numerical analysis findings are comparable to those of Krauland and Stille (1993). Comparison between observational, analytical and numerical will be discussed in the subsequent chapter.

8.4 Assessment of Stability for Sill Beam with Cold Joints

Cold joints were identified by operators as a large concern to sill beam stability. Backfill placement in the stope is a piece-wise operation: significant time between successive pours can occur. The interface between the placed backfill is considered by operations to be a weak point. The study of cold joints is discussed by Nasir and Fall (2008), but no analysis is performed with respect to stope stability of placed fill. Chapter 7 presents an analytical equation to assess the
stability of cold-joints. The analytical assessment will be tested against a numerical analysis presented below.

The numerical analysis consists of the simple beam model discussed above with a discrete discontinuity placed within the sill beam. The height of cold joint and the shear and tensile strengths of the discontinuity are user controlled variables. The numerical analysis of cold-joint stability investigates the effect of stope wall shear strength and the tensile strength of the cold joint. It is assumed that the worst-case scenario for cold joint is one that lies horizontal as no shear strength would be present on the discontinuity surface and the sidewall would provide no tensile strength. The numerical model will assess the limiting strength for the sidewalls and the discontinuity surface.

To assess the shear strength of the sidewalls, the model will be run to equilibrium with 10 cm of closure, the tensile strength of the discontinuity will be set to zero, and the stope sidewalls will be relaxed until the cold-joint beam fails. The modulus of the fill will be set arbitrarily to 50 MPa such that crushing failures do not occur, yet allow normal stresses to build in the fill. The UCS of the fill will be set 1 MPa to represent a standard fill per Red Lake and Stillwater specifications (Chapter 5). It was determined in Chapter 7 that cold-joint shear strength stability is governed by the width of the stope, verification of this relationship will occur. The shear strength will be analyzed independent of the tensile strength, and will be combined for a full analysis.

The tensile analysis will assess the required tensile strength to stabilize a horizontal cold joint. The model will be set up such that the sidewall shear strength will be zero; other variables will be kept similar to those described for the shear strength analysis. Chapter 7 demonstrated that the tensile strength of the cold joint is dependent on the tensile strength of the contact and the height of the cold joint. To test the cold joint model, the cold joint height is increased until the separation occurs.

The tensile strength and shear strength analysis is only valid if the contact materials are tested. Outside of that data, the assumption has to be made that the cold-joint is unstable mass supported only by ground support. The role of ground support in cold joint stability will
investigated in a subsequent section of this chapter. This analysis will limit itself to the role of shear and tensile strength in cold joint stability.

The first analysis for cold-joints investigates the required shear strength to support a cold-joint that extends across the width of the stope. The cold joint analysis investigated span widths between 4 and 10 m wide. Cold-joint heights were selected to be 0.1, 0.25, 0.4, 0.5 and 0.75 of the span widths. The numerical model was cycled for 5000 steps with an initial cohesion value of 1000 kPa along the interface, the cohesion value was reduced by five-percent and the cycle was repeated. Detachment from the interface occurred when 0.01 m of movement occurred along the interface. Once the movement was detected the cohesion was recorded. It should be stated that the values for cohesion along interface for the FLAC are the cohesive strength for the entire length of interface and not, as one would expect, the unit length value. The results of the analysis are found in Figure 8-5.

Figure 8-5 shows that the shear strength required to stabilize a cold joint follows a linear profile. Investigation into the analysis shows that the required shear strength is:

\[ \tau = \frac{\gamma W}{2} \]

*Equation 8-1*

Equation 8-1 is similar to the cold-joint shear analysis presented in Chapter 7. The denominator of two accounts for both the hangingwall and footwall; the issue in applying shear strength over both interfaces was not explicitly stated in Chapter 7, rather a conservative approach was taken to allow the user to specify if both side walls contribute shear strength. A couple points can be drawn from this analysis. Firstly, the numerical model reflects the first-principal approach to stabilizing cold-joints with cohesive forces. Secondly, the analysis only accounted for the cohesion properties of the interface. If friction and closure is applied, there will be an added component to the shear strength of the interfaces.
The tensile strength analysis was performed by removing all shear strength values from the sidewall interface (cohesion and friction angle equal to zero). Span widths and cold joint heights were the same as those performed for the shear strength analysis. The tensile strength of the horizontal cold-joint was set to 1 MPa and reduced in increments of 5 percent every 5000 model cycles. Fine tuning of the limiting tensile values was performed when warranted. Figure 8-6 graphs the results of the tensile strength analysis.
Figure 8-6 is not what was expected from the analytical analysis provided in Chapter 7. The analytical analysis found the tensile strength required to support a cold joint of arbitrary height is a linear function; the numerical model demonstrates a power function. The numerical investigation is in agreement with the analytical model in that the width of the stope is not a factor in tensile failure of a cold joint. The required tensile strength is the same regardless of the beam width. Investigation into the relationship of the required tensile strength from numerical analysis is:

$$\sigma_t = \frac{8\gamma h_c^2}{h}$$

*Equation 8-2*

The numerical analysis appears to have some model dependency; the appearance of the ratio of the height of the stope to the cold joint should have no effect on the required tensile strength. The reason for a unique relationship in the model is not known; yet, the relationship holds true when verified for other stope configurations. Whereas the shear strength along the
sidewalls approximated the analytical equation, numerical model tensile strength approximation will require scaling to match the analytical equation.

The numerical analysis of sill beams failing on the sill beam showed that cohesion and tensile strength can stabilize the beam. The issue is that the numerical model assumes full contact and the values reported are difficult to assess in the field. However, the numerical model verifies the findings of the analytical assessment and provides design values for stable sills.

8.5 Effect of Ground Support on Stability

Chapter 5 presented the ground support employed by mines utilizing UCF. It was observed that three bolting systems are employed at the mine; a fourth bolt system will be investigated as part of the analysis:

- Stand up re-bar bolts prior to placement of fill;
- Frictional support bolts on advance under CPB sill;
- Shear paddles; and
- Angled bolts connecting CPB sill to stope side walls.

The angled bolts will be investigated to determine if they can support the sill beam against rotational and flexural failure as per shear paddles.

The simple beam FLAC model will be employed to determine the loads acting on the bolts. In addition, the effect of the bolts on stability of the sill beam will be investigated. Bolting patterns will be determined per properties listed in Table 8-7.

Table 8-7: Bolt properties used in numerical analysis

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Bolt Length (m)</th>
<th>Spacing across drift (m)</th>
<th>Spacing down drift (m)</th>
<th>Bolt Strength (kN)</th>
<th>Bond Strength (kN/m)</th>
<th>Elastic Modulus (GPa)</th>
<th>Shear Modulus (GPa)</th>
<th>Area (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#7 Rebar</td>
<td>2.4</td>
<td>1.2</td>
<td>1.2</td>
<td>240</td>
<td>600</td>
<td>200</td>
<td>75</td>
<td>3.82E-04</td>
</tr>
<tr>
<td>SS 39 Rebar</td>
<td>2.4</td>
<td>1.2</td>
<td>1.2</td>
<td>140</td>
<td>37.5</td>
<td>200</td>
<td>75</td>
<td>1.19E-03</td>
</tr>
<tr>
<td>#7 Rebar shear paddle</td>
<td>1.8</td>
<td>1.2</td>
<td>1.2</td>
<td>240</td>
<td>600</td>
<td>200</td>
<td>75</td>
<td>3.82E-04</td>
</tr>
<tr>
<td>#7 Angled side wall bolts</td>
<td>1.8</td>
<td>1.2</td>
<td>1.2</td>
<td>240</td>
<td>600</td>
<td>200</td>
<td>75</td>
<td>3.82E-04</td>
</tr>
</tbody>
</table>
The investigation on the role of ground support in sill beam stability will consist of investigating each bolt type for each failure type for stable and unstable configurations as performed earlier in Chapter 7 & 8. The inputs for the failed and stable configurations will come from random sampling of the Monte Carlo analysis presented in Chapter 7. The anticipated results of this analysis are the effect of ground support on sill beam stability and the performance of the ground support during analysis. For cases where the ground support is installed post-undercut (SS39 and angled rebar), the numerical model was cycled 5000 steps prior to installation of support to account for actual conditions of settlement of the sill beam before installation of ground support. In total 32 analyses were performed to analyze the effect of ground support on sill beam stability. The model properties and results of the numerical analysis are presented in Appendix 6.

In all cases the numerical model demonstrated that installed ground support did not add to the stability of the sill beams. Sill beams that were unstable without ground support remained unstable when ground support (outlined in Table 8-7) was installed. Although the bolts are taking loads up to 200 kN, the ground support does not add stability to the sill beam. The only case where there was a noticeable difference in failure methods was for flexural failure: stand up bolts and split set reduced the displacement of the sill beam, but ultimately the excessive strain within the sill beam itself causes a numerically unstable configuration. Williams et al. (2001) discuss that rebar bolts provide an ellipse of confinement due to displacement of the sill beam; the ellipse of confinements was not noticed in the numerical model supporting the work of Pells (2008).

The ground support numerical model findings are as follows:

- Ground support employed in this study is not sufficient in preventing full sill beam collapse for unstable unsupported sill beams;
- The stand-up rebar and SS39 assist in reducing displacements;
- Installed ground support does take expected axial and shear loads;
- Ground support does not provide confinement within sill beam.

Inspection of the numerical model with ground support underlines the importance of the strength of the interfaces between the stope walls and sill beam. The loads supported by the
ground support are too large for the employed ground support to have an effect on sill beam stability.

The ground support investigation did demonstrate that the vertical supports do provide resistance against sill beam displacement. This finding will be tested against the support of cold-joints within a sill beam.

### 8.5.1 Numerical Model of Ground Support for Sill Beam Discontinuities

A numerical model was created as per Chapter 6.4 testing the stability of horizontal discontinuities with ground support. Stand up rebar bolts and SS39 friction stabilizers were investigated as ground support options. The discontinuity had no tensile strength and the side walls provided no shear strength: the ground support retained the entire load of the unstable mass. The findings of the numerical model analysis are shown in Table 8-8.

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Bolt Pattern (m x m)</th>
<th>Bolt Density (1/m²)</th>
<th>Bolt Length (m)</th>
<th>Numerical Model Critical depth of cold joint (m)</th>
<th>Analytical Model Critical depth of cold joint (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS39 Stand up 20mm Dywidag</td>
<td>1.2 x 1.0</td>
<td>0.84</td>
<td>1.8</td>
<td>1.125</td>
<td>0.85</td>
</tr>
<tr>
<td>SS39 Stand up 20mm Dywidag</td>
<td>1.2 x 1.0</td>
<td>0.84</td>
<td>1.8</td>
<td>1.30</td>
<td>1.20</td>
</tr>
<tr>
<td>Stand up</td>
<td>20mm Dywidag</td>
<td>1.2 x 1.0</td>
<td>0.84</td>
<td>2.4</td>
<td>1.75</td>
</tr>
</tbody>
</table>

The results of the numerical horizontal discontinuity analysis are similar to analytical model. The numerical model underestimated the critical depth for the stand-up rebar bolts and overestimates the SS39 bolts in comparison to the analytical methods. The numerical model poorly accounted for the actual configuration of the stand-up rebar bolts and is the cause of the difference between the two analyses. For the SS39 bolts, the likely difference is due to the consideration of horizontal confinement (ignored in analytical model) on the frictional bolts that causes the discrepancy in critical cold-joint depth.

The numerical analysis shows that the role of ground support is to retain detached blocks within the sill beam. Analysis found that the employed ground support outlined in Appendix 6
provides no additional stability to a failed sill beam. The advantage of ground support was to contain discontinuities within the sill beam. The critical depths were found to be in general agreement with the analytical analyses. This is no trivial finding. Ground supports purpose in CPB beam is to prevent the detachment of the beam; ground support improves miner safety. Another benefit of the ground support retaining loose material is that the beam dimensions remain intact.

8.6 Summary of Numerical Model Findings

The numerical model analysis purpose was to support or counter findings of the analytical and observational findings. First the model was calibrated to the analytical model for failed and stable conditions. The numerical model for rotational and sliding failures were highly dependent on the interface strength values, the interface values were not critical for the crushing and flexural failures. It was found that the numerical models reflect the failure mechanics proposed by Mitchell and Roettger (1989) and Krauland and Stille (1993).

Once confidence in the model was established, maximum spans for UCS design curves were established for low, medium and high stress conditions. The critical finding of the design curves was the required UCS for a stable configuration was dependent on the amount of closure and the modulus of elasticity. Crushing failure occurred in stiff fills with large amount of closure, the same stope configurations were stable for soft fills at low strengths. The main conclusion of the finding was that soft fills are desired as they are able to withstand closure stresses at low UCS values.

Numerical analysis of ground support found that the employed ground support (outlined in Appendix 6) provided no structural benefit to sill beam stability for unstable configurations. This statement can be misleading as it indicates that ground support serves no purpose; that is not the case. Rather, the main role of ground support for CPB sill beams is to contain cold-joint, and in turn improving miner safety. Ground fall is the largest risk posed to mining with paste backfill (Seymour, Clark, Tesarik and Stepan, 2013). Understanding the role that ground support plays in stabilizing cold-joints will assist in preventing ground fall. An Additional advantage of ground support was to keep the beam intact during redistribution of stresses and strain during the undercutting of the beam.
The numerical analysis agreed in general to the analytical assessment. Frictional bolts performance was improved in the numerical model in comparison to the analytical model. The incorporation of confining pressure in the numerical model is the cause of the increase support capacity. The ground support is more of a tactical measure to reduce ground fall, and in turn the safety to the miner operating below the fill.

The numerical model, once confidence was gained in calibration to analytical results proved useful in determining critical properties for design. The numerical model allowed for ‘time-generated’ strains and stresses to develop and dissipate due to a changing mine environment. The findings of the numerical model were very sensitive to the input properties. Model confidence was an important part of determining the suitability of the findings. It was not a simple process to determine the correct input and expected behaviour.

The numerical model solved no problems; it verified findings of analytical and observational techniques. The findings of the numerical model are in themselves meaningless without an attachment to the actual mine settings. Material properties, parameters and understanding of critical behaviours at the mine site are needed before the numerical model can be used as a tool. It was experienced during this study that erroneous behaviour of sill beams occurred when physically unrealistic input properties and parameters were inputted into the model. Without the experienced gained from site visits to mine sites and observations from field, the erroneous behaviour of the sill beam could go undetected.

The numerical model is a tool in the design process. It fits in with analytical and observational techniques as part of the process. Once observations are made in the field, or a proper understanding of the ground conditions are learned, the numerical model can be used to advance the design process. The main advantage found during the numerical study is that it allows for complex parameters and ground conditions to be simulated as opposed to calculated or justified through observation. The numerical analysis should be coupled with other techniques as part of the design of sill beams.
Chapter takeaways

- Numerical analysis found that ground support main purpose is to contain loose blocks within sill beam;
- A negative slope of strength vs. span width indicates a closure failures; and
- A positive slope of strength vs. span width indicate beam theory type failures.
9 DISCUSSION

Through the course of this research, three approaches to sill beam stability have been investigated: observational, analytical and numerical. This chapter links these analyses together in an effort to finalize the design guidelines. The discussion will be of the form of design curves for the Red Lake and Stillwater mine. The Stillwater and Red Lake mine were selected for this discussion because they have the most amount of relevant information for the study. A comparison to the numerical, analytical and observational methods is performed for these mine sites. The findings of this chapter will link in to the design guidelines for the underhand cut and fill CPB sill beams.

9.1 Red Lake Mine

The Red Lake mine is suitable for this discussion as numerous and varied data sources are available on the mine. Design guidelines from the original design, QA UCS data, laboratory testing, and site observations are available for input properties and parameters. The gathered data will be used for numerical and analytical analysis. The results of the analysis will be compared to current design practices and original design guidelines.

9.1.1 Model Input and Parameters

Five distinct models were analyzed for the Red Lake mine design curves: FLAC numerical model, analytical model with linear CPB stress strain path, analytical model with hyperbolic stress strain path, beam theory equations with test values, and values assumed from the literature.

The material properties and model parameters for the analysis are listed in Table 9-1. The justification of the values within the table has been provided in previous chapters of the thesis. No ground support was analyzed as Chapter 8 found the installed ground support did not have an effect on structural stability.
Table 9-1: Red Lake design curve model properties and parameters

<table>
<thead>
<tr>
<th>Variables</th>
<th>Symbol</th>
<th>Range</th>
<th>Units</th>
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<tbody>
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<td><strong>CPB</strong></td>
<td></td>
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<td></td>
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<tr>
<td>UCS</td>
<td>$\sigma_c$</td>
<td>Variable</td>
<td>kPa</td>
</tr>
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<td>Elastic Modulus</td>
<td>$E_t$</td>
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<td>MPa</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>$\varphi$</td>
<td>20</td>
<td>$^\circ$</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>$\gamma_p$</td>
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<td>kN/m$^3$</td>
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<tr>
<td><strong>Rock</strong></td>
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<td>MPa</td>
</tr>
<tr>
<td>Elastic Modulus</td>
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<td>GPa</td>
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<tr>
<td>Unit Weight</td>
<td>$\gamma_R$</td>
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<td>kN/m$^3$</td>
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<td>Poisson's Ratio</td>
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<td></td>
</tr>
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<td><strong>Stress Parameters</strong></td>
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<td></td>
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<tr>
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<td>$D$</td>
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<td>M</td>
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<td>$\sigma_H / \sigma_v$</td>
<td>$K$</td>
<td>1.48</td>
<td></td>
</tr>
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<td><strong>Stope Dimensions</strong></td>
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<td></td>
</tr>
<tr>
<td>Height</td>
<td>$h$</td>
<td>Variable</td>
<td>m</td>
</tr>
<tr>
<td>Width</td>
<td>$W$</td>
<td>4.5</td>
<td>m</td>
</tr>
<tr>
<td>Dip</td>
<td>$\beta$</td>
<td>70</td>
<td>$^\circ$</td>
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<tr>
<td><strong>Reduction Factors</strong></td>
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<td></td>
</tr>
<tr>
<td>Interface Cohesion</td>
<td>$C_1$</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>Interface Friction Angle</td>
<td>$C_2$</td>
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<tr>
<td>Hangingwall Shear Strength</td>
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<td><strong>Model Specific CPB Variables</strong></td>
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<td><strong>Hyperbolic Model</strong></td>
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<td><strong>FLAC, Hyperbolic, Linear &amp; Laboratory Value Beam</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cohesion</td>
<td>$c$</td>
<td>0.32 UCS</td>
<td>kPa</td>
</tr>
<tr>
<td>Tension</td>
<td>$\sigma_t$</td>
<td>0.22 UCS</td>
<td>kPa</td>
</tr>
<tr>
<td><strong>Beam Equations with standard assumptions</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cohesion</td>
<td>$c$</td>
<td>0.25 UCS</td>
<td>kPa</td>
</tr>
<tr>
<td>Tension</td>
<td>$\sigma_t$</td>
<td>0.10 UCS</td>
<td>kPa</td>
</tr>
</tbody>
</table>

The minimum UCS values for spans between two and ten meters were determined by satisfying either numerical or analytical stable conditions. Analytical stability was considered a factor of safety greater than 1.0, and numerical stability was considered to be a converging numerical model.
9.1.2 Model Results and Discussion

The results of stable paste strengths vs. span widths for the five models are presented in Figure 9-1. Stable configurations are those that plot above the design line curves. It needs to be stressed that the values plots correspond to a factor of safety of 1.0.

![Figure 9-1: Red Lake span design curve](image)

The plotted results of the analyses (Figure 9-1) are in line with the results of the analytical and numerical analyses presented in Chapter 7 and Chapter 8 respectively. The incorporation of closure in the FLAC, linear stress-strain, and hyperbolic stress-strain models results in higher required strengths for smaller closure. Beam theory models require increasing strength with increased widths; the required strengths for beam stability are low.

By comparing the models to the actual range of spans and paste strengths at Red Lake, the linear stress-strain model is not justifiable for design. The spans at Red Lake are stable at present; while the linear stress-strain model predicts failure for the current spans. The amount of
convergence (3.9 cm) and relatively stiff fill (0.9 GPa) produced high stresses within the beam, requiring high strengths. The need to quantify the external loading is underlined with this finding.

The beam equation models are stable for all configurations; however, these models do not account for external loading. The depth of mining, and stresses (Mah et al., 2003) at Red Lake are considerable. A model that does not account for any sidewall convergence should be looked at skeptically. A comparison between the beam theory models demonstrates that by performing tensile laboratory testing (nominal pricing $50 per test) the required strengths for stable sill beams can be reduced. The use of the approximation of tensile strengths being 1/10th of the UCS has large implications on costs. The binder requirements required for stronger paste are substantial in comparison to the cost of a tensile testing laboratory program that could justify reducing the required strengths.

The hyperbolic stress-strain path and FLAC models incorporate closure stresses in the model. The difference between the FLAC and hyperbolic model is likely due to the linear stress-strain path required for FLAC as compared to the hyperbolic model. However, both show closure failures dominate over the current design widths at Red Lake. The hyperbolic stress-strain model does predict stable configurations for the majority of the design spans at Red Lake; albeit with a factor of safety of 1.0. Refinement of the hyperbolic model could reduce stresses; the larger the Rf factor the larger strains that can be incorporated. Further, the modulus values are from Itasca studies and were not independently verified; a softer fill would reduce the required strengths. In addition to the above, the strengths reported are 28-day laboratory strengths. Field strengths are stronger than lab values as described in Chapter 5. The difference between field values and lab values is never explicitly stated, it is used by designers as an implicit factor of safety.

The closure models that reflect the closure appear to reflect the conditions at the mine site: strong fills are stable; however, screen is necessary to contain loose CPB fill. It is suggested, per Martin et al. (2009), that this is spalling of the fill due to stresses. This spalling would be expected at Red Lake for narrow spans, or for spans in high stress areas. The spalling in high stress areas was observed during site visits. This underlines the need for ground support
at Red Lake consisting of bolts and screen with CPB to contain loose block that can possibly detach from the beam. With closure, the need for support is not restricted to cold-joint detachment; it contains spalling from closure stresses.

The amount of closure predicted by the analytical and numerical models may not be representative of actual closure conditions at the mine. Red Lake strategic approach to mining is to reduce the stresses in the rock before mining. De-stress slots are common, as is mining ‘in the stress-shadow,’ where mining takes place in low stress conditions due to mine geometry. These strategic approaches may reduce the closure of the stope walls as the high-stress is mitigated. The amount of actual closure could be determined with the incorporation of the mine-wide stress model employed at the mine. This should be supported with actual closure measurements within the stopes. By knowing the expected closure, the stope widths, strengths and necessary ground support can be optimized. This is recommended for future study at Red Lake.

The design guidelines at Red Lake employing beam theory equations show that the operating spans and paste strength at Red Lake are stable. However, these equations should be treated with skepticism as they do not incorporate the significant stresses at the mine. The incorporation of stresses in the analysis demonstrated that closure stresses dominate failure over the range of spans at Red Lake. It is recommended that the closure of the stope walls be incorporated into the design of stopes at Red Lake.

9.2 Stillwater Mine

The Stillwater mine provided the largest amount of data for this research. The observational data from the sill width and UCS comparison will be used as empirical evidence for stable configuration. The results from the laboratory studies and database investigation are included in the discussion. A comparison to the design guidelines currently employed at Stillwater will be discussed.

9.2.1 Model Input and Parameters

The model input and parameters for the Stillwater are well understood through laboratory testing and mine-site observations. Table 9-2 summarizes the model inputs used for the study.
The host rock is listed as weak per Chapter 5; the CPB values were determined in Chapter 4 through 6.

Table 9-2: Stillwater design curve model inputs

<table>
<thead>
<tr>
<th>Variables</th>
<th>Symbol</th>
<th>Range</th>
<th>Units</th>
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<tbody>
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<td></td>
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</tr>
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<td>UCS</td>
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<td>MPa</td>
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<td>°</td>
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<tr>
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<td>m</td>
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<tr>
<td>Dip</td>
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<td>°</td>
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<td>Interface Friction Angle</td>
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<td>0.10 UCS</td>
<td>kPa</td>
</tr>
</tbody>
</table>
9.2.2 Model results and discussion

Three separate analyses were performed to model the behaviour of sill beams: linear elastic closure analytical; hyperbolic elastic analytical analysis and numerical model analysis.

The Stillwater design span is based on works by Jordan et al. (2003). The design guideline was based on tensile failure of the material per beam theory. The design guideline used an approximation of tensile strength to be 1/10 of the UCS. Testing of Stillwater CPB (Chapter 4 and 5) found that the tensile strength is 22% of the UCS. The tensile design model has been used successfully at Stillwater as the sill beams remain stable during the mining cycle as discussed in Chapter 5. However, empirical evidence of span widths vs. the UCS strength was collected as part of the study and does not agree with the design guidelines.

Figure 9-2 is a comparison between the tensile model proposed by Jordan et al. (2003) with both values of tensile strength (lab and assumed); the observation of UCS vs. Span width; and the results of analytical and numerical analysis. A stable configuration is represented by coordinates above a design line, unstable values are below (FS = 1.0).

Every data point shown for the inscribed span and perpendicular span widths are stable given installed ground support. The data plotted in Figure 9-2 finds that the tensile failure equations do approximate the actual conditions at the mine. Stable configurations are achievable at lower strengths than predicted by the tensile. The linear closure model is very poor fit to the data. The excessive closure at Stillwater would cause failure in the sill beam if a linear elastic model is applied. The FLAC model provides a stable design curve. However, it was observed during the analysis that the model did not properly transfer the closure to the sill beam if it was not explicitly stated. The hyperbolic model provides a stable design curve and accounts for the closure of the material. Cracking in the sill beam due to closure was observed during site visits to Stillwater mine: the effects of closure need to be accounted for. From inspection of the observational data compared to the numerical model, it is recommended that the hyperbolic closure model be applied to assess sill beam stability at Stillwater.
9.3 General discussion

Through this study, some question remain around how the research findings relate to the implementation at mine sites, and these require further discussion. The discussion presented here is simply that: discussion on the applications of the findings within an operating environment.

The main discussion point is the behaviour of the sill beam in the mine environment. The design curves up to present in literature (Hughes et al., 2006, Pakalnis et al., 2005, Stone, 1993) have showing positive trends with respect to required strength vs. span widths. When closure is incorporated, narrow stopes are problematic as the closure is independent of stope width. The narrow stopes allow for large axial strains, an in turn axial stresses develop. For design curves a negative strength vs. span width trend is indicative of closure failures. A positive trend is indicative of standard beam theory failures. This is to say, as shown in the hyperbolic model in
the Red Lake mine analysis (Figure 9-1), there exists a CPB strength minimum where the failure mode transitions from closure to beam theory failure. This local minimum is the design optimum. When designing CPB sill beams the CPB should be designed with this local minimum in mind: the fill should be able to accommodate closure stresses and be stable for beam theory failures. Softening the fill would reduce the effect of closure stresses. Increasing the strength of the fill has no effect on changing the design curves. By optimizing the strength and stiffness of the fill, binder reductions may be possible. The main take-away of this discussion is the stiffness of the fill as a critical design property.

There were issues in incorporating material stiffness into numerical codes. Elastic stiffness of CPB was poorly handled by numerical codes. Standard practice is to determine the modulus of elasticity by determining the slope of the stress-strain line at 50% of the UCS value (Fairhurst and Hudson, 1999). For CPB this has adverse effects as shown in Chapter 6. The Young’s modulus does not allow for the true strains to be incorporated. For numerical codes that do not allow, for what was determined to be correct hyperbolic stress-strain path, the secant modulus at peak stress should be used to determine the elastic stress-strain path of the model. This would approximate the closure stresses that are experienced by the CPB laboratory samples. The best practice would be the incorporation of curvilinear elastic stress-strain paths. In the absence of the incorporation of curvilinear models, the secant modulus at failure should be used as an approximation of the elastic stress-strain behaviour of CPB.

Staying on the theme of numerical codes, discussion needs to take place on their validity. A large amount of time was spent in validating the numerical model. The ease in which unrealistic configuration was possible was alarming. Considerable amount of time was required before the model was judged representative of actual behaviour understood from observational and analytical study. Further, numerical models required ‘tweaking’ to satisfy certain requirements: numerical manipulation was required to satisfy constitutive behaviours. Additionally, the numerical model provided no conclusions on to itself. A more thorough understanding of the development of failure mechanisms were gained with numerical models. However, nothing was discovered that was not previously known with the numerical analysis. The numerical model was used only to strengthen observational and analytical techniques. This
argument is presented to ask the following question: is numerical analysis the answer to CPB sill beam stability?

From this research, it was found that the properties and behaviour of sill beam stability are not well understood. Tensile, cohesive, friction angle and interface strength values are typically based on assumptions or selected from published literature (where it was originally assumed!). This is problematic. In terms of operational budget, money is spent on analysis with suspect inputs. This research showed that cohesive values were under estimated by 22% and tensile values under estimated by 100%. These assumptions lead to higher strengths that are addressed through additional binder content. Potvin et al. (2005) state that binder is the largest cost associated with consolidated fill. So, in designing sill beams, is the money better spent on numerical modeling, or on laboratory testing? From this study, the researcher determines that cost saving are better achieved by understanding of the material properties. Once the material behaviours are known, more realistic behaviours of sill beam can be achieved through analysis.

The results of beam theory equations for stable sill beams are open to judgment due to their low strength requirements. A comparison to the standard vertical fill model by Mitchell et al. (1982) is warranted based on the findings of this research. The Mitchell model investigates the strength of exposed vertical faces through two methods: maximum vertical stress and ‘active-wedge’ stability. Without going into detail, the results of these analyses are typically low strengths requirements. In the researcher’s experience these models are widely used in industry. Now, this model is widely accepted and is put into practice. Few question the validity of the Mitchell vertical exposed wall. The equations find a stable number and a factor of safety of 1.2-1.5 are applied to the values. The values are understood since the loads are easily understood. This is not the case for the stresses and strains of a horizontal beam.

For UCF mining, beam theory (Jordan et al., 2003) or Mitchell equations (Mitchell and Roettger, 1989) are used for design in industry. These equations are applied with numerous assumptions and the results are treated with skepticism: factor of safety of 2.0 or greater are common. It is argued that engineering judgment has prevailed in the design of sill beams as described by Peck (1962). Beam theory equations provided low values for stable sill beam and these values were verified with numerical models (Hughes et al., 2006) but were never put into
practice. Stronger paste fills were prescribed as recommendations. The equations did not match the judgment of the designer. Whereas the vertical model were easy to understand, the horizontal beams were more difficult to contextualize.

This research demonstrates that skepticism was warranted: the low values did not seem correct. Beam theory equations were absent of closure and crushing failure was considered in select studies (Krauland and Stille, 1993 for the best case). The updated equations and findings of this study bring the sill beam from the thought of a simply supported beam to an engineered material in contact with its surrounding environment.

I believe the reason for my skepticism of the equations of beam theories is based on the experience of my research supervisor and his insistence in the importance of field study. In the field, I have observed fill that would classify as R1 per ISRM classification (Ulsay and Hudson, 2007). This fill has strength but is very delicate. It will crumble under pressure and completely disintegrate: weak fill has little stiffness and no post-peak strength. In terms of weak fill supporting a beam: it will not be stable. The delicate nature of the fill will not allow for ground support to be installed and would not be stable for any additional loading. Weak fill although theoretically stable is not stable in practice: in this case judgment overrides the theoretical solutions. The skepticism of the beam theory was justified by inspection of the material in the field.

Although this research focused on cemented paste backfill, UCF mining is common with cemented rock fill. Cemented rock fill has high strength and is typically in the 4-7 MPa range (Pakalnis et al., 2007). The higher strengths are based on a few different reasons. The primary reason is the downscaling of UCS values (Stone, 1993; Yu, 1995) to account for inclusion of larger particles sizes. Additionally, the fill has high stiffness in comparison to paste backfill (ranging between 1 and 4 GPa per Yu, 1995), and the CRF is commonly used in weak rock environments (Rai et al., 2013; Knockler, 2008). With the high closure in the weak rock environments and the high modulus, it follows that high strengths are required. The relationship of stiff fills requiring high stress is supported by the findings of this research.

Chapter 8 demonstrated that ground support had little to no effect on structural sill stability. Again, this finding needs engineering judgment. The sill beam is not homogenous; it
has to be assumed that discontinuities exist within the fill. Since the behaviours of the discontinuities are not known, the strength of the discontinuities should be considered to be zero. The installed ground support should ‘stitch’ the beam together. The purpose of the ground support is to keep the sill beam intact.

The effect of stand-up rebar bolts in comparison to support on advance had no advantage in the analysis. The preference of the two systems is up to the operation. The following bolt support appears to have the highest benefit from this research: angled re-bar bolts with steel strapping along the length of the stope and vertical support. The advantage to the support is that it can be installed on-advance. Further, this ground support system would replace shear-paddles and stand up bolts improving support turnaround time. Vertical support should be the form of rebar as they are able to contain cold joints to a depth approximately equal to its length. This ground support assumes that welded wire mesh is installed to contain spelled loose backfill. The ground support recommendations are based on discussion with operations and through the researcher’s experience and observations in operating environments.

This research relied on observations from the field to validate results from numerical and analytical analysis. The observations were observed from the researcher’s perspective. There was no instrumentation as part of the study and this is considered problematic. Discussion is currently taking place with Red Lake mine to monitor a stope during the undercut process. Once this work is performed it can be compared to findings of this study.

The instrumentation of stopes is not a priority of operations. The study by Donovan et al. (2007) is a well-studied stope but this type of study is not common. The researcher’s discussions with operations indicated that instrumentation is regarded as expensive, it interrupts productivity, and the results are neither accurate nor precise. This is an issue as designs are not validated by field behaviour. This is an important point that needs to be rectified: instrumentation should be part of any sill beam design.

The discussion chapter investigated sill beam designs based on the findings of the numerical, analytical and observational chapters. Design curves were constructed for Stillwater and Red Lake. Both the design curves found that the hyperbolic model approximates the actual
conditions, albeit with a Factor of Safety of 1.0. The hyperbolic model incorporates closure into
the model and provides reasonable values based on engineering judgment.

Discussions were had from questions that arose from the research that needed to be
addressed. This included the role of numerical modeling in design, the need for proper model
inputs and suggestions on ground support.

The discussion chapter coupled with the analytical, experimental, observational and
numerical chapter has built a case for constructing design guidelines for sill beams. These
guidelines can be used for initial design and feasibility of UCF with CPB mining.

**Chapter takeaways**

- Traditional design guidelines, with tensile strength 10% of the UCS, poorly reflect
the empirical evidence
- Hyperbolic analytical closure model has good agreement with the Stillwater Mine
case history; and
- The importance of determining CPB properties cannot be overstate.
10 DESIGN GUIDELINES

This chapter incorporates the findings of this research in a succinct design guideline. This design guideline is intended for operations that are planning or refining underhand cut-and-fill mining with cemented paste backfill. The guideline will discuss necessary properties and parameters for design, suggested testing methods, QA/QC guidelines, design considerations and equations, ground support details, and suggested monitoring and instrumentation.

Where helpful, references are provided that would assist the designer. Complete details of reference are found in the bibliography.

10.1 Required Design Inputs

The confidence of analytical and numerical design is built on the model inputs. Verification in the field and engineering judgment furthers this confidence. Empirical relationships are useful when properly vetted. No empirical relationships were found to be valid for CPB during the course of this study. The commonly used tension and cohesion approximations did not hold up to investigation. Table 10-1 lists the required inputs for analytical and numerical design. The list incorporates both sets of parameters for linear and hyperbolic stress-strain paths for closure analyses.

The establishment of the properties and parameters can be through laboratory testing or through published relationships where justified. Details on the suggested laboratory testing for the design parameters are described in the following section.

10.2 Suggested Laboratory Testing

The suggested laboratory testing for cemented paste backfill should yield the required design parameters with the least amount of testing. It is suggested that UCS testing, tri-axial, tensile, interface testing and shear testing be performed to obtain proper design inputs. A minimum of twelve tests per paste mix design is recommended such that an understanding of the variation
Where possible the tests should be performed on samples from the operation paste plant or prepared in a manner as similar as possible. Again, the objective is to have design inputs that are reflective of the CPB placed underground. Initially, quarterly updates of the laboratory testing should take place. Once confidence is built, the verification of laboratory testing can be performed annually.

Table 10-1: Design inputs for sill beam designs

<table>
<thead>
<tr>
<th>Design Inputs</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CPB</strong></td>
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<tr>
<td>UCS</td>
<td>$\sigma_c$</td>
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<tr>
<td>Elastic Modulus</td>
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<tr>
<td>Horizontal Stress</td>
<td>$\sigma_h$</td>
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10.2.1 Unconfined Compressive Strength Testing

For design purposes the UCS testing should be done in conjunction with strain testing. The stiffness of fill is critical to the stability of sill beams in converging environments, and understanding the full stress-strain curve is important for designers. It is recommended that strain-rate testing be performed at a rate of 0.002 percent strain per minute. The test should be performed past the peak strength and up to full sample failure.

The purpose of the UCS testing with strain readings is to obtain the following properties and behaviours:

- Young’s modulus,
- Secant modulus,
- Modulus of resiliency (strain-energy),
- UCS strength,
- Poisson’s ratio,
- Shear and bulk modulus,
- Failure ratio,
- Initial elastic modulus, and
- Post peak behaviour.

Twenty tests should be performed to determine a representative behaviour. The two highest and two lowest modulus values should be eliminated before general trends are determined.

For reference on strain-rate testing refer to Chapter 4 and to Fairhurst and Hudson (1999).

10.2.2 Tri-axial Testing

Tri-axial testing is valuable for design parameters as the behaviour under confinement can be obtained as well as a range of design parameters. Tri-axial testing is difficult to perform on CPB as the strengths of the material is too weak for rock tri-axial testing setups and too strong for soil tri-axial testing setups. It is likely that the testing will need to be performed at research facilities.

Strain-based testing should be performed for tri-axial testing as it gives an indication of failure prior to material disintegration. This indication allows one sample to be tested over a
range of increasing confining values. In total ten separate failure curves should be obtained for CPB samples tested at 0, 150, 250, and 500 kPa confinement.

From the tri-axial testing the following design inputs can be obtained:

- Cohesion,
- Internal friction angle,
- Behaviour under confinement, and
- Failure envelope.

Tri-axial testing with cemented paste backfill is discussed in Pierce (1997).

10.2.3 Tensile Testing

Tensile testing is an important parameter as tensile failure dominates the beam theory equations failure. Tensile testing can be performed directly or indirectly (Brazilian). The Brazilian tensile tests were performed as part of the study and provided suitable results. All tensile testing should be paired with UCS testing of the same or as similar as possible samples to determine the ratio between tensile and UCS values.

10.2.4 Interface Shear Testing

Interface shear testing has been performed by Fall and Nasir (2010) and Nasir and Fall (2008). Outside of these studies, the behaviour of interface values is not known. It is suggested that direct shear testing be performed within the confines ASTM D5607 (ASTM, 2008c). It is recommended that interface samples be prepared as follows:

- Trim 10 cm diameter 20 cm high (4”x8”) paste cylinder to 10 cm high;
- Pour CPB to depth of ½ of paste cylinder;
- Allow sample to cure for 7 days;
- Pour remaining sample;
- Prepare sample per ASTM D5607 sample;
- Ensure sample remains intact to test breaking strength of interface.

Preparing samples in this manner will allow for interface strengths and discontinuity friction angles. The same procedures can be performed with rock core and CPB interfaces.
The design inputs that can be achieved from interface testing are as follows:

- Interface friction angle, and
- Interpolated interface cohesion.

10.2.5 Unit Weight Testing

The unit weight is integral in determining CPB sill weights and maximum surcharge loads. The testing for the unit weight is performed by the water displacement method described in ASTM C127 (ASTM, 2012c). The relevant design input from the unit weight test is the following: CPB unit weight (density)

The testing for all the above should be performed prior to design if possible. The values should be updated during the mine-life to account for variation in binder, mill tails and source water that may occur due to changes in supplier or mineralogy.

10.3 Quality Assurance and Quality Control Guidelines

Once the design for the CPB sill beam is complete, the beam should be stable provided all design recommendations are achieved. The issue with stability and backfill is the variation of fill strengths (Yu, 1995; Rai et al., 2013). Operations assess the variation of fill through two common UCS and slump tests. These are typically performed on surface and are used as quality control of the plant: the results are rarely incorporated into tactical measures for sill beams.

10.3.1 Surface Testing

The quality control and assurance of paste backfill was discussed in Chapter 4 and Chapter 5. Typically UCS tests are performed at 7, 14 and 28 day on paste batches to determine strength. Slump tests are performed to determine pumpability. The details of the backfill testing are discussed by Hughes et al. (2013) and in the ASTM Standards C31; C39; and C143. In addition Appendix 3 list suggested methods for performing UCS backfill testing.

The UCS loading rate is a question that is often asked from industry. At this time, there is no standard for loading rate. For the purposes of establishing a standard, UCS backfill tests should take two minutes per test. The loading rate should be determined from the following formula:
For typical paste strengths, this corresponds to a loading between 10 and 20 kN/min.

Slump testing should be performed per ASTM C143 (ASTM, 2012b). For better precision, values should be measured in millimeters as opposed to \( \frac{1}{4} \) inches.

Study of the Macassa backfill showed that a pulp density measurement with a Marcy scale provided useful information with regards to quality control of paste mixture. Further, the Marcy scale values were found to have a negative linear relationship with the slump values. The Marcy scale test is simple and can easily be incorporated into backfill sample preparation. To have another level of quality control, pulp density measurements should be included in surface QA/QC of CPB.

CPB strength is affected by the particle grain size. In CPB, the larger the grain size the higher the strength. This relationship was proven as part of study on the Macassa tails. Regular gradation testing of the non-cement binder portion of CPB provides insight into differentiation of stresses. For CPB, standard sieve tests do not provide the entire particle size distribution due to the high percentage of fines. The fines portion of the material requires hydrometer testing such that a particle size distribution can be obtained. The procedure for standard sieve and hydrometer gradation curves are found is ASTM D422 (ASTM, 2007). The gradation of tails portion of the CPB should be performed weekly or if variation of fill strength is suspected.

A relationship between moisture content and UCS was established in this research. This relationship is useful in determining in-situ strengths. To establish the relationship between moisture content and UCS, the moisture content should be measured for every UCS backfill sample. The moisture content should be performed per ASTM D1558 (ASTM, 2010d). Once the moisture content vs. UCS relationship is established it can be used for in-situ paste strengths.

### 10.3.2 In-situ Testing

In-situ testing of CPB was found to be difficult and have poor repeatability for tests. Chapter 4 describes the in-situ testing methods and their suitability for CPB. It was determined
that moisture content of the CPB has a strong correlation with the UCS. It is recommended that once the moisture content vs. UCS relationship is developed that in-situ strengths for underground bulk samples are determined through the moisture content relationship.

10.3.3 Database Management

Chapter 5 of this research illustrates that although large data exists for UCF operations, the collected data is not fully utilized. Typically, the purpose of building the database for UCF operations is to ensure that the paste plant is producing the right mixture and strengths on surface. This data is not integrated into mining planning. Typically the results are reviewed on a monthly basis and investigated as a measure of paste mixture; the results of the tests are not considered with respect to sill beam stability.

Forecasting of CPB paste strength can be used to improve scheduling. Once a strength vs. age of sample relationship is obtained, predictions on paste strength can be made based on 7 day strengths. By knowing predicting strengths of backfill, stopes can be mined when the sill reaches specified design strength. Depending on the results of the 7 day test this can improve the availability to mine the next undercut. In addition, safety can be improved by delaying the mining of a sill if the 7 day strengths are below designs.

The database of stope widths, paste strengths and age of backfill at undercut is a useful tool in design. Every stope that is mined for UCF should have relevant information gathered. This can be used to determine the empirical performance of UCF mining. The empirical performance can then be applied to future stopes in determine the likely stability of the sill beams.

10.4 Design Considerations

The structural stability of sill beams is the critical design consideration. The failure of sill beam is by rotational, flexural, shear and, most critically closure failure methods. The ability of the sill beam to withstand seismic energy also needs to be considered.

This section summarizes the equations from the analytical section to determine the minimum strengths required for a stable sill beam. The method suggested is to perform the
individual equations for failure methods with input parameters determined from lab testing and mine parametric inputs for specific span widths. The next step is to alter the UCS until a minimum factor of safety is reached for the most critical failure method. Below is a summary of the factor of safety equations for closure, rotational, flexural, and shear failure.

**Closure failure**

Closure failures are the most critical failure given the UCF CPB mining environments. To analyze closure failure, first the stope closure needs to be quantified. Three methods are available: observational, numerical and mechanical. Observational approach to closure is the preferred method as it is a direct measurement of the closure of the stope. Numerical approach to closure measurements uses the expected closure of stope walls given a mine wide stress model. Once determined, the numerical and observational closure values would be placed directly into the closure equation.

Outside of direct or numerical methods to quantify closure, the following equation was developed to assess elastic closure (Chapter 7):

\[
\Delta w_{stope} = \frac{1}{h_1 - h_2} \left( \frac{2(1 - v^2) \sigma_h}{E'_t} \right) \left( \Delta H_c \left( \sqrt{\left( \frac{H'}{2} \right)^2 - \Delta H_c^2} \right) - \left( \frac{H'}{2} \right)^2 \tan^{-1} \left( \frac{\Delta H_c}{\sqrt{\left( \frac{H'}{2} \right)^2 - \Delta H_c^2}} \right) \right) - \Delta H_c' \left( \sqrt{\left( \frac{H'}{2} \right)^2 - \Delta H_c'^2} \right) - \left( \frac{H'}{2} \right)^2 \tan^{-1} \left( \frac{\Delta H_c'}{\sqrt{\left( \frac{H'}{2} \right)^2 - \Delta H_c'^2}} \right) \right)
\]

where:

\[
\sigma_h = \frac{\gamma D}{2} \left[ (1 + K) + (1 - K) \cos 2\beta \right]
\]
w’ = amount of convergence (m)
D = depth below surface to center of stope (m)
$E'$=Elastic Modulus of rock (kPa)
K = ratio of the in-situ horizontal to vertical stress
H= Height of stope (m)
$\Delta H_c$= distance above or below the centerline of stope (m)
$H'$= Height of the stope at time of mining CPB sill beam
$h_1$ = distance from top of CPB sill to centerline of stope
$h_2$ = distance from bottom of CPB sill to centerline of stope
$w_n'$ = wall convergence for ‘n’ stope cuts
$w_o'$= original amount of wall convergence when mining CPB sill

Once the amount of closure is quantified, the closure strains need to be related to stresses within the sill beam. The stress-strain path of closure is discussed in Chapter 6 through 8. However, there are three possible calculations for the allowable closure in stope: secant modulus, Young’s modulus, and hyperbolic cumulative closure. The researcher recommends that the cumulative hyperbolic closure be used (per Chapter 7); however, in absence of lab data, the secant modulus closure calculation provides the amount of closure allowed at failure. To calculate the factor of safety, first calculate the allowable closure based on selected method as follows:

$$\Delta w_{allowable} = \frac{w \sigma_c}{E} \quad \text{Equation 10-2}$$

Following that, determine the factor of safety against closure as follows:

$$FS = \frac{\Delta w_{allowable}}{\Delta w_{stope}} \quad \text{Equation 10-3}$$

The required compressive strength ($\sigma_c$) is varied until a suitable factor of safety is reached for the span width. Note, in an elastic analysis, the amount closure is independent of the stope width (Hoek and Brown, 1980). It was found through analytical investigation that in mines with high closure or narrow stopes that closure failures were most common. If these issues are present at the mine, further study on the closure behaviour of the stopes and stress-strain behaviour is recommended.
Indications of closure failures in the field are ‘bagging’ of CPB in welded wire mesh, and conjugate failures in the beam. The conjugate failures are cracks in the beam near the HW and FW contacts.

**Rotational Failure**

Rotational failure will not occur if the following geometric inequality is satisfied:

\[
 w < \sqrt{h^2 + \left(\frac{w - h}{\tan \beta}\right)^2}
\]

*Equation 10-4*

In stopes that have a high height to width ratio, or steeply dipping walls, rotational failure is not of large concern.

\[
 FS = \frac{\sigma_t h + 2\alpha \left( C_1 \frac{\Delta w_{stope}}{w} + E_t \tan(C_2 \varnothing) \right) W \sin^2 \beta}{\gamma W (W - h \cot \beta) \sin^2 \beta}
\]

*Equation 10-5*

where:

- \( \alpha \) = percentage of hangingwall in contact with backfill.
- \( C_1 \) = cohesion interface multiplier (range between 0.25 and 1.0)
- \( C_2 \) = friction interface multiplier (range between 0.25 and 1.0)
- \( \sigma_c \) = tensile strength of CPB

Rotational failure differs from other failures as it can be easily eliminated by altering the geometry of the stope.

**Flexural failure**

Flexural failures are common in stopes with a low height to width ratio. The failure is based on beam theory and failure is caused by tensile induced cracks at mid span due to excessive beam deflection. The design equation for beam failure is as follows:
Flexural failure can be prevented with the use of high strength paste backfill, or lightweight fill. Stope geometries have little effect due to the squared term associated with the width of the stope affecting the factor of safety.

**Shear Failure**

Shear failures are common in stopes with poor wall contact. Failure occurs as full beam displacement along the interfaces. Shear failure can be prevented by improving interface contact strength and support connecting the beam to the side walls. Factor of safety for shear failure is as follows:

\[
FS = \frac{2\left(C_1 c + \frac{\Delta w_{stope}}{W} E_t \tan(C_2 \varnothing)\right)}{\gamma W \sin\beta}
\]

Equation 10-7

The above is the analytical method in determining the factor of safety against failure for each failure type. To construct design curves the minimum strength for the critical failure should be established over the proposed mine span widths. The determined values (with appropriate factor of safety) become the guideline for design.

UCF mining is selected in areas of high stress as the engineered back mitigates the risk of rock bursts in seismically active areas. The ability of CPB to sustain seismic energy can be quantified based on the material’s seismic energy density. The seismic energy density can be determined from integrating the stress-strain plot. The following equation establishes the strain energy capacity of CPB:

\[
U = \int_{\epsilon_o}^{\epsilon_{peak}} \sigma_a d\epsilon
\]

Equation 10-8
In terms of the strain energy density with respect to the factor of safety of the beam in a seismic event, study of the mine system is required. The scaled distance and design magnitude of an event need to be determined for the analysis. These values would be obtained from a seismic study of the mine-plan. The following equation is proposed to determine the stability of a CPB sill beam in a seismic event:

\[ FS = \frac{\int_{\varepsilon_0}^{\varepsilon_{peak}} \sigma_a d\varepsilon w h L}{E_s} \]  

\[ \text{Equation 10-9} \]

where:

\[ \log E_s \propto 0.5 \log \left( \frac{7M_0^2}{16r_0^3} \right) - 1.16 \]

\[ L = \text{length of stope} \]

The CPB should have enough strain-energy density to withstand the seismic environment of the mine. Stronger fill and softer stiffness are two methods to increase the strain-density.

### 10.5 Ground Support

Ground support for CPB sill beams is of two broad categories: support installed prior to placement of fill and support on advance under the sill beam. Bolt types are either grouted rebar or frictional support bolts (split-sets). For design purposes Table 10-2 is a summary of bond strengths for frictional support.

**Table 10-2: Bond strength for friction support bolts in CPB**

<table>
<thead>
<tr>
<th>Bolt type</th>
<th>Average Bond Strength (tonnes/m)</th>
<th>Maximum Bond Strength (tonnes/m)</th>
<th>Minimum Bond Strength (tonnes/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FS39 Galv. taper end</td>
<td>3.74</td>
<td>6.17</td>
<td>0.82</td>
</tr>
<tr>
<td>FS39 Galv. without taper end</td>
<td>3.43</td>
<td>5.76</td>
<td>0.82</td>
</tr>
</tbody>
</table>

For stand up rebar bolts placed prior to pour in the sill, the bond strength is equal to the breaking strength of the bolts, due to the plating of the bolt at the head and toe of the bolt.
Cold-joint support provided by the ground support can be assessed with the following equation:

\[
\text{Available Support Pressure (kPa)} = \begin{cases} 
\frac{W}{B_{sw}B_{st}} (\Delta B_s (B_L - h_c)) & 0 < (\Delta B_s (B_L - h_c)) < S_{ult} \\
\frac{WS_{ult}}{B_{sw}B_{st}} & (\Delta B_s (B_L - h_c)) > S_{ult} \\
0 & (B_L < h_c)
\end{cases}
\]

where:

- \(B_{sw}\) = Bolt spacing along width of stope (m)
- \(B_{st}\) = Bolt spacing along length of stope (m)
- \(\Delta B_s\) = Bond Strength of bolt (kN/m)
- \(B_L\) = Bolt Length (m)
- \(S_{ult}\) = Ultimate strength of bolt (kN)

Numerical analysis found that structural beam stability was not improved by the employed ground support. Rather, the role of the ground support in the numerical model was to support detached blocks and to ensure the beam remains intact and performs as designed. Caution should be applied to this finding as Williams et al. (2007) noted that ground support measured loads within fill. In the numerical model, the difference between material stiffness of the fill and the ground support had the effect that the support did not interact with the fill. Further research is required on this matter.

The investigation of the preference between ground support on advance and ground support before placement of fill (sill mat) found there was no preference either way. The decision of one method vs. the other is strictly operational. The ground support should provide sufficient capacity to support detached beams and ensure that the wall contacts remain intact. It is recommended that welded wire mesh be installed to contain CPB that detaches between ground support.
10.6 Monitoring and Instrumentation

Monitoring and instrumentation are essential in determining design is performed as prescribed. The critical issues with sill beam stability are stope closure and sill beam displacement. For the CPB, the critical issues are two-fold: what is the minimum required strength, and are these strengths obtained in-situ? The presence of cold joints should be identified and their depth recorded. Ground support should be installed to standards and the support should perform as designed. The monitoring and instrumentation program should ensure that all these are addressed and re- incorporate into the design process.

Sill beam closure and displacement can be determined through active and passive measurements. Passive measurements from survey crews underground can measure the wall displacement directly below the sill beam and vertical movement at the center of the beam. These measurements can then be compared to the original design. Active measurement through instrumentation is possible. Closure readings have been performed by Williams et al. (2001) with respect to sill beam. Williams et al. (2001) measured displacement between two fixed points on the sidewall of a lower cut with a tape extensometer. It would be beneficial to install the closure meter within the fill. Discussions with mine operations indicate access is an issue for instrumentation programs. Access issues are understandable, but the importance of instrumentation should not be dismissed. If instrumentation is not possible due to access issues, then passive measurements with total station equipment should be performed.

Determining loads within CPB has been measured by Hughes (2008); Williams et al. (2001) and Donovan et al. (2007). These studies measured CPB stresses by means of vibrating wire earth pressure cells (EPC). If EPC are to be measured they should be placed both in the vertical and horizontal direction and compared to the predicted values from numerical and analytical results.

Cold-joints are an issue for UCF mining. Investigation of cold joint height can be performed with the use of an electrical video scope along a drilled ground support hole. Investigation should be performed for every backfill pour to determine the existence of cold joints. However, the best way to eliminate cold joints is to pour full height stopes where possible.
Ground support measurements should be performed to determine the bond strength of frictional and grouted support per ASTM D4435 (ASTM, 2013d). For frictional support, the bond strength should be measured over a distance of 0.5 m; alterations to the borehole may be necessary. This provides a discrete interval for bond strength values as opposed to dividing pullout load over the entire length of the bolt. Ground support bond strength tests serve the purpose of validating assumptions of design and providing properties for future design work.

The design considerations for UCF CPB mining can be summarized as follows:

- Establish the behaviour of the CPB that will be used at the mine;
- Develop QA/QC guidelines to determine that design strengths are being achieved;
- Design sill beam to stable for spans and mine environment;
- Monitor and observe the behaviour of sill beams during the mining process; and
- Refine design.

These design guidelines are general in nature. Through this study it was found that sill beam stability is determined by site specific factors. The three factors found to be critical in determining sill beam stability are mine stresses, CPB stiffness and CPB strength. The purpose of the guidelines is to provide the mining engineer with a general methodology outline on designing stable sill beams.
11 CONCLUSIONS

This research achieved the objectives of the original hypotheses: that CPB sill beam design can be improved through a rigorous engineering study. The CPB sill beam design was improved after rigorous engineering study. The research of the historical design of sill beams, experimental and observational studies and in depth analysis of sill beam behaviour and stability increased the knowledge of the elements within the sill beam design process. Operations can now make more informed and educated decisions around the use of materials, and stope geometries that all contribute to sill beam design. This will benefit the operations with a more efficient mining method; and in-turn, the risk to the miner is mitigated by having a better understanding of sill beam design.

By reviewing the literature it was found that previous studies do not provide a full understanding of the design elements of sill beams. Stability methods did not account for the mining environment: closure was typically omitted. This shortcoming was fixed with a method of manually calculating the closure and applying its stope failure. Published stability equations differed in their approach and critical failures varied depending on the study. This research consolidated the stability equations and made refinements where necessary. The constitutive behaviour of CPB in literature was not consistent: different studies used different constitutive relationships. This research investigated the complete post-peak stress strain response of CPB under axial load. It was found that, two streams of study were needed: one stream to study the stability of sill beams; the other the behaviour of CPB.

In investigating the behaviour of CPB, experimental testing was performed on samples at the UBC NBK rock lab. The stress-strain behaviour was determined with constant strain and cyclical loading. It was determined that the stress-strain behaviour of CPB followed a hyperbolic path per Duncan and Chang (1970) up to peak stress, the post-peak behaviour is strain softening than can be approximated by a linear decay of the cohesion of the material. Lab testing found that failure ratio (Rf) between 0.78 and 0.85 are suitable for CPB. The constitutive model studies in this research had a very good agreement with the test data. The researched constitutive code properly reflected the full stress-strain response of axial loaded CPB.
In addition to the constitutive model for the CPB, the experimental program provided insight into the cohesion and tensile strength of the backfill. Backed up by published values, it was found that the cohesive value is approximately 32 percent of the UCS; tensile strength values were found to be 20 percent of the UCS. This has large implications for sill beam stability as design strengths were based on an assumption that the cohesion and tensile strength are 25 and 10 percent of the UCS, respectively. The values found in this research represent an increase in stability of 28 and 50 percent respectively for a given UCS. The method of the UCS tests was also investigated as part of this research. It was concluded that the sample size had no effect on the strength of the sample. Verification of in-situ methods was also performed as part of the experimental process.

The in-situ strength investigation was conducted to determine if a simple test could be performed to determine the CPB strength within the beam. The Windsor pin method, and pneumatic pin method test, suitable for concrete and shotcrete, did not provide a statistically valid relationship. It was found through investigation that the moisture content of CPB can be related to the UCS through a logarithmically decaying relationship. The relationship was found to be site specific. The moisture content – UCS relationship answered the research question of determining a simple in-situ strength approximation. The in-situ strength relationship should be developed by mine sites and incorporated into their QA/QC procedures.

QA/QC databases of three mines were investigated to determine if any statistically valid relationships exists between the data. Strength vs. cement content and strength vs. age relationship were verified for all three mine sites, supporting the work of Bartlett and Macgregor (1994), Shrestha (2008), Williams et al. (2007), Pakalnis et al. (2005), De Souza et al. (2003) and Annor, Tarr, and Fynn (2003). All the relationships that were found were site-specific, underlining the importance of the need to determine strength relationships for individual mines. Further investigation of the UCS of backfill had a negative linear relationship with the slump and pulp density values. Ultimately, performing QA/QC at the mine site is useful as it ensure that design strengths for stable sill beams.

Sill beam stability was investigated as part of this research based on the studies of Mitchell and Roettger (1989); Stone (1993); Krauland and Stille (1993); Caceres (2005); and
Jordan et al. (2003). The first step was to investigate the stability equations. It was found that the stability analysis lacked a method of assessing crushing failure: a crushing failure equation was developed. Once the stability equations were finalized, it was found a parametric study would be difficult with all the model inputs. To rectify this issue, a probabilistic analysis (Monte Carlo) was performed with a range of inputs determined from laboratory, field observation and published values. The Monte Carlo analysis found that closure failure was the most critical followed by flexural and shear failure. It was found that stability can be achieved in 95% of the closure sill beam failures by having a ratio of UCS to elastic modulus greater than 1%. Whereas, flexural and shear failures can be eliminated in beams by having a height to width ratio greater than 0.9. Rotational stability was uncommon as an equation determined within this research found that the conditions for failure are rare. The analytical investigation led the ground work for detailed numerical study.

The numerical study verified the analytical results, but failed to provide any new findings. Parametric studies were performed in low, medium and high stress ground conditions for three different CPB mix designs. It was found that soft fill requires the lowest strength in all three stress conditions. The effect of ground support was investigated with numerical and analytical models. It was found the ground support was useful in supporting possible loose blocks within the sill beam, but did not provide any additional structural support to the sill beam. The benefit to the numerical modeling was the understanding it provided the researcher in understanding the initiation and propagation of sill beam failure. The numerical model provided another tool in the design guidelines.

Part of the design guidelines was to observe operations utilizing UCF mining. The observations were summarized in a case study format. It was found that the application of the mining method varied between operations. The common theme was that all CPB sills had ground support; however the timing of the installation was not standard. Some mines prepared the ground support in the stope prior to the placement of the fill, other operations installed ground support in advance. The findings of this study found no one advantage from one method over the other, rather the decision is strictly a consideration for operational scheduling.
The operations provided span widths and design and lab strength of the CPB. This data was used to augment the empirical knowledge of possible spans based on given paste strengths. In extending this thought, the span widths vs. 28 day CPB strength history of Stillwater was constructed. With a historical database of stable sill beam span, a solid footing for empirical database was developed for the sill beam stability guidelines.

Design guidelines were constructed with existing equations, equations created in this study and numerical analysis. It was found that the analytical hyperbolic closure model represents the Stillwater history. In investigating the Red Lake mine, analytical hyperbolic closure and numerical model best represents the stable conditions at Red Lake. No unique strength vs. span curve can exist for the various mine environments that are conducive to the underhand cut and fill mining method.

The results from the design discussion were amalgamated with the results of the laboratory studies and observations in the field in determining the design guidelines for cemented paste backfill sill beams in underhand cut and fill mining.

Building on previous research work, the purpose of this study was to augment the understanding of the behaviour of CPB and design of sill beams. Through the research approach and results, the project was successful in answering the research question posed by mine operators. In the end, a design guideline was provided for CPB sill beams that can be used to create or augment the sill beam design in underhand cut and fill operations. It is hoped that the design guideline will be applied to extend the reserve life of mines or increase the economic feasible of unmined reserves.
12 RECOMMENDATIONS FOR FUTURE WORK

This research sought to forward the knowledge of underground mining through the development of a comprehensive design guideline for sill beams. Literature, experimental and observational studies guided the research questions. Due to research constraints, there are aspects of the original research questions that may be addressed through future work. This chapter identifies areas where future work is recommended.

The primary remaining issue was identified as the need to properly define the behaviour and properties of CPB. To date, there is not an extensive published testing database for CPB. A database that lists modulus of elasticity, tensile and cohesive strength values would assist in feasibility stage of designing UCF sill beams. The published research is site specific and lacks the depth of information of more comprehensive studies. It is recommended that a more comprehensive study would include, testing across paste backfill sites to develop a database of testing values would assist in the design of future sill beams. Testing, at a minimum, should focus on stope convergence, vertical displacement, stress within the backfill and ground support loads.

The constitutive equation proposed in this study basis its curve fitting from test data. The fitted data was then related to existing constitutive models. The constitutive model created is a piece-wise function. As such, it is recommended that the development of a more elegant CPB constitutive model be explored. To complete this, studies into the adequacy of cap-yielding soil models or strain-softening should be performed. The study should investigate the behaviour in tri-axial conditions and could dove tail with the CPB testing database.

The experience from testing at mines sites is that UCS testing is not performed to a universal standard. It is recommended, that a sector wide study be performed to develop standards for required testing for CPB. The goal of this study would be to standardize slump, grain size analysis, moisture content, UCS procedures, sample size, flexural testing, curing conditions, and maturity indexes for CPB. The standardization of testing would allow for comparison between mine sites and ensure proper QA/QC standards.
A purpose of this study was to analyze the behaviour of sill beams. The study relied heavily on analytical and numerical techniques. There exists little in the way of published studies on the performance of backfill once a sill beam is undercut. Observations of the deformation, stress, ground support loads and convergence are priorities for monitoring. Once these properties are more fully understood, site-specific relationships can optimize the mining cycle. Further, the knowledge learned from a more thorough understanding of these properties and the performance of backfill will allow other mine sites a better understanding when designing future sill beams.

This research found that numerical models were limited in the ability to assist in UCF sill beam designs. From the analytical model, the convergence is critical to sill beam stability. The ability to analyze convergence stress is well suited to FDM or FEM models. Another critical failure mode is detachment of sill beams: this is well suited to DEM analysis. With contrasting needs for numerical code, a more complete model is necessary. In determining stability of sill beams the use of FEM/DEM code would provide full analysis. FEM or DEM models need simplification and assumptions for successful convergences. The use of advanced modeling is inconsequential without the understanding of the material behaviour. It is recommended, that a thorough comparison between observational/empirical approach and advanced numerical code for CPB is conducted.

Seismicity was addressed in this research by investigating the energy absorption capabilities of CPB. The energy absorption study only considered static stress–strain tests. The behaviour of CPB during seismic loading is a worthwhile study. Following on Hedley (1992), study of the dissipation of seismic waves within CPB and the contribution of CPB in stope stability is recommended. To do this, it is suggested that a geophone be placed in CPB stopes in a seismically active and monitored mine. Once data has been collected it can be corroborated with the findings of this research.

"No theory can be considered satisfactory until it has been adequately checked by actual observations"

-Ralph B. Peck (1962)

The above rings true. The models presented in this research and recommendations need to be observed in the field before they can be truly validated. Monitoring a CPB sill beam during
an undercut is the next step of this research. The monitoring data can then be compared to the prescribed analytical and numerical data, amendments then made to the models; and subsequent alterations to initial guidelines. The designing, observing, re-designing would complete an engineered solution.
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APPENDICIES
APPENDIX 1

Cemented paste backfill composition information
In total, five CPB tests data sets are included for analysis as part of this research. The five sets CPB samples were of three forms: samples made in the UBC NBK rock mechanics laboratory, samples cast at the Stillwater Paste plant; and samples cast at the Macassa Fill plant. A description of the five sets is presented with respect to the origin of the samples, date range of tests, cement content, grain size, mineralogy, sample preparation, slump and pulp density, size of the samples, and age of samples at time of testing.

The following lists, where available, the sample preparation, the mineralogy, and the particle size distributions for the test samples

A1.1 Sample Preparation

A1.1.1 UBC/Red Lake cemented paste samples

Eight (8) five-gallon buckets of tails were shipped to NBK. The samples were sent at natural moisture content. ATSM C 192 (ASTM, 2007) was used as a reference and guideline for sample preparation. Paste mixes were made at 6%, 8%, 10% and 12% binder content. Normal Portland cement was selected for the binder. Water was added to initiate the hydration process at a water content equal to 25 percent (by weight). Slump tests were performed per ASTM C143 (ASTM, 2012b) before samples were cast in their molds.

The cemented paste recipes for the Red Lake backfill testing are listed in Table A1.1. The recipes were constructed such that the pulp density was 78% solids by weight.
Table A1.1: Red Lake backfill recipes

<table>
<thead>
<tr>
<th>Blend #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Binder</td>
<td>Type II Portland Cement</td>
<td>Type II Portland Cement</td>
<td>Type II Portland Cement</td>
<td>Type II Portland Cement</td>
</tr>
<tr>
<td>Tailings Blend</td>
<td>100.00%</td>
<td>100.00%</td>
<td>100.00%</td>
<td>100.00%</td>
</tr>
<tr>
<td>Tailings (dry)</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Sand (wet)</td>
<td>6.00%</td>
<td>8.00%</td>
<td>10.00%</td>
<td>12.00%</td>
</tr>
<tr>
<td>Binder Content</td>
<td>6.00%</td>
<td>8.00%</td>
<td>10.00%</td>
<td>12.00%</td>
</tr>
<tr>
<td>Binder Total (kg)</td>
<td>3.19</td>
<td>4.35</td>
<td>5.56</td>
<td>6.82</td>
</tr>
</tbody>
</table>

**Batch Information**

| Tailings Dry Weight (kg) | 50.00 | 50.00 | 50.00 | 50.00 |
| Sand Dry Weight (kg) | 0.00  | 0.00  | 0.00  | 0.00  |
| Tailings Blend Moisture | 0.00% | 0.00% | 0.00% | 0.00% |
| Tailings Moisture | 0.00% | 0.00% | 0.00% | 0.00% |
| Sand Moisture | 0.00% | 0.00% | 0.00% | 0.00% |
| Water (as moisture) (kg) | 0.00  | 0.00  | 0.00  | 0.00  |
| Water (added) (kg) | 15.00 | 15.33 | 15.66 | 16.00 |
| Total Water (kg) | 15.00 | 15.33 | 15.66 | 16.00 |
| Pulp Density (%) | 78.0  | 78.0  | 78.0  | 78.0  |

A total of sixty-nine (69) cylinders were cast; sample diameter, percent cement content and age of tests are listed in Appendix 3. Cylinders were cast such that a set of 7, 14 and 28 day existed for each individual cylinder size and percent cement. In cases when additional batch material was available, any remaining backfill was made in 2 inch sample containers. An assortment of test samples are shown in Figure A1.I.

Once the paste was placed into the cylinder, the samples were covered in absorbent fabric and wetted down to control humidity. During curing the samples were kept at room temperature, measured to be 19°C.

It was found that leaving paste in the cylinders for the full curing duration was undesirable, especially for lower strength samples. The samples were soft, making testing difficult due to the failure of the samples before applied load. In an attempt to mitigate the issue, the day before testing, the samples were cut out of the cylinders and placed in a lime bath overnight, as per ASTM C31 (ASTM, 2012a). Placing specimens in the bath allowed for the outer shell of the sample to harden, improving testing and handling. It was determined that not all samples were suitable for testing due to samples being below 1.8 height to width ratio.
A1.1.2 Macassa Mine

The backfill at Macassa is considered a blended paste backfill, or a high density sand fill. The backfill solid aggregate consists of 70% esker sand and 30% mine tailings. The backfill binder content varies at the mine depending on the demands of the underground mining environment; backfill binders of 3, 5, 7 and 10 percent are used at the mine. The binder at the mine is a standard Type 10 Portland Cement. These blends were followed as closely as possible when the samples were made at UBC NBK, such that the values at Macassa could be replicated. However, due to the need for the testing to have a large array of UCS values, alterations were made to the original recipe used at Macassa.

ASTM C 192 (ASTM, 2007) was used as a reference and guideline for sample preparation. Paste mixes were made according to practices at Macassa Mine with binder contents of 5, 6 and 8%; however, in an effort to have a range of constituents, the blend of sand and tails was changed such that it was not representative of the mine. Table A1.II summarizes the mix design for the Macassa tails.
The Macassa paste samples were difficult to mix as the fine tails did have an observed low permeability. As such, the saturation of the samples was very difficult and long mix times in a cement mixer and the breaking down of large ‘baled’ material was required. The slump on the fully mixed backfill was 7.5 cm; much lower than the slump of the fill at Macassa Mine.

The three mixed tails batches were placed in separate mineral ore container. The containers were kept at room temperature and the moisture was controlled with the placing of wet burlap bags over the samples to ensure moisture was available during the hydration process.

Table A1.2: Macassa CPB paste recipe

<table>
<thead>
<tr>
<th>Blend #</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Binder</strong></td>
<td>Type II Portland Cement</td>
<td>Type II Portland Cement</td>
<td>Type II Portland Cement</td>
</tr>
<tr>
<td>Tailings Blend</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tailings (dry)</td>
<td>100.00%</td>
<td>100.00%</td>
<td>100.00%</td>
</tr>
<tr>
<td>Sand (wet)</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td><strong>Binder Content</strong></td>
<td>8.00%</td>
<td>7.00%</td>
<td>5.00%</td>
</tr>
<tr>
<td>Binder Total (kg)</td>
<td>6.52</td>
<td>5.65</td>
<td>3.95</td>
</tr>
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<td><strong>Batch Information</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tailings Dry Weight (kg)</td>
<td>75.00</td>
<td>37.50</td>
<td>22.50</td>
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<td>Sand Dry Weight (kg)</td>
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<tr>
<td>Tailings Blend Moisture</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Tailings Moisture</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Sand Moisture</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>Water (as moisture) (kg)</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Water (added) (kg)</td>
<td>15.00</td>
<td>15.00</td>
<td>15.00</td>
</tr>
<tr>
<td>Total Water (kg)</td>
<td>15.00</td>
<td>15.00</td>
<td>15.00</td>
</tr>
<tr>
<td>Pulp Density (%)</td>
<td>84.5%</td>
<td>84.3%</td>
<td>84.0%</td>
</tr>
</tbody>
</table>

A1.1.3 Stillwater Mine

As the dataset was provided by the mine, the following understanding from the sample preparation is from discussion with mine personnel. Samples are prepared in samples that are 76 mm in diameter and 152 mm plastic samples conforming to ASTM C31 (ASTM, 2012a). Originally, samples were prepared every hour on surface and collected at the end of the paste.
line underground every hour as well. As discussed by Hughes et al. (2013) this practice is no longer performed. Twenty-four hours after casting, the samples are sent to SK Geotechnical labs for independent testing. Three (3), seven (7) and twenty-eight (28) day ages test are performed on samples. The unique aspect of Stillwater test procedures is that Stillwater dictates stope re-entry time on the strength of the backfill samples. That is, once the required design strength is verified during the age tests, the subsequent underhand stope is then scheduled for mining.

Stillwater prepares the CPB with standard Type I-II Portland Cement at 73% solids (Jordan et al., 2003); historical cement contents are between 7 and 18 percent cement.

A1.2 Mineralogy

A1.2.1 Red Lake

A quantitative phase analysis of the tailings material from Red Lake mine was done using X-Ray powder diffraction data and the Rietveld method. The tests were performed at UBC Earth and Ocean Sciences. Details of the testing method are described by Wilson, Raudsepp and Dipple (2006). Results of the XRD diffraction are summarized in Table A1.3.

<table>
<thead>
<tr>
<th>Mineral</th>
<th>Ideal Formula</th>
<th>(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>SiO₂</td>
<td>27.2</td>
</tr>
<tr>
<td>Clinochlore</td>
<td>(Mg,Fe²⁺)₃Al(Si₃Al)O₁₀(OH)₈</td>
<td>5.1</td>
</tr>
<tr>
<td>Biotite</td>
<td>K(Mg,Fe³⁺)₂₃AlSi₅O₁₀(OH)₂</td>
<td>5.3</td>
</tr>
<tr>
<td>Muscovite</td>
<td>KAl₃(AlSi₅O₁₀)(OH)₂</td>
<td>2.3</td>
</tr>
<tr>
<td>Actinolite</td>
<td>Ca₃(Mg,Fe²⁺)₂₃Si₂O₄(OH)₂</td>
<td>7.9</td>
</tr>
<tr>
<td>Cummingtonite</td>
<td>Mg₇Si₆O₂₂(OH)₂</td>
<td>9.1</td>
</tr>
<tr>
<td>Plagioclase</td>
<td>NaAlSiO₈ – CaAl₁₂Si₂O₈</td>
<td>13.3</td>
</tr>
<tr>
<td>Calcite</td>
<td>CaCO₃</td>
<td>2.8</td>
</tr>
<tr>
<td>Dolomite</td>
<td>CaMg(CO₃)</td>
<td>25.8</td>
</tr>
<tr>
<td>Grossular</td>
<td>Ca₃Al₂(SiO₄)₃</td>
<td>1.2</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>100</td>
</tr>
</tbody>
</table>

This mineralogy is consistent with the Red Lake Greenstone Belt as described by Kumar (2003).
A1.2.2 Macassa

No quantitative or qualitative analysis of the mineralogy of the tails and sand was performed on the Macassa samples. Kerrich and Watson (1984) summarize the Macassa ore bearing deposit as an Archean Lode deposit. A summary of the geochemical analysis of the ore bearing mineralization is shown in Table A1.4; the values presented are based on the mean of 20 samples prepared by Kerrich and Watson (1984). It can be seen that the majority of the rock is a silicate with secondary Alumina-silicate composition.

Table A1.4: Average of geochemical analysis of Macassa ore
(after Kerrich and Watson, 1984); n = 20

<table>
<thead>
<tr>
<th>Mineral</th>
<th>Mean</th>
<th>St. Dev</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO2</td>
<td>56.3</td>
<td>6.2</td>
</tr>
<tr>
<td>TiO2</td>
<td>0.9</td>
<td>0.4</td>
</tr>
<tr>
<td>Al2O3</td>
<td>14.6</td>
<td>1.5</td>
</tr>
<tr>
<td>Fe2O3</td>
<td>3.8</td>
<td>2.2</td>
</tr>
<tr>
<td>FeO</td>
<td>4.0</td>
<td>0.9</td>
</tr>
<tr>
<td>MnO</td>
<td>0.1</td>
<td>0.0</td>
</tr>
<tr>
<td>MgO</td>
<td>3.8</td>
<td>2.2</td>
</tr>
<tr>
<td>CaO</td>
<td>4.8</td>
<td>2.2</td>
</tr>
<tr>
<td>K2O</td>
<td>4.7</td>
<td>1.4</td>
</tr>
<tr>
<td>Na2O</td>
<td>3.7</td>
<td>1.1</td>
</tr>
<tr>
<td>P2O5</td>
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<td>0.2</td>
</tr>
<tr>
<td>LOI</td>
<td>4.1</td>
<td>1.7</td>
</tr>
<tr>
<td>Total</td>
<td>101.1</td>
<td>0.8</td>
</tr>
</tbody>
</table>

A1.2.3 Stillwater

The information provided by the mine site did not contain any information regarding the mineral composition of the tails. No study was performed on the composition of the Stillwater Tails.
A1.3 Particle Size Distribution

A1.3.1 Red Lake Mine

Seven tests were performed with the Malvern Mastersizer; the particle size distribution curve for Red Lake Mine’s tailings is graphically shown in Figure A1.2. The coefficient of uniformity, coefficient of curvature, and percent of samples finer than 20 microns is 12; 1.3; and 37% respectively.

Figure A1.2: Results of Malvern Mastersizer analysis on Red Lake samples

A1.3.2 Macassa Mine

Particle size analysis was performed by personnel at Macassa Mine. The results of the particle size analysis will be used to determine a relationship between particle size and the properties of the cemented backfill. The particle size analysis was conducted in accordance with ASTM C-136 (ASTM, 2006).
Recently the stockpile of the esker sand has been screened to determine the particle size gradation. Figure A1.3 shows the particle size distribution for twenty-eight (28) test samples performed between April 8, 2011 and June 17, 2011. Table A1.5 summarizes the testing based on the D90, D50 and D10 particle sizes.

Figure A1.3: Particle size distribution for twenty eight samples of backfill sand

![Particle size distribution](image)

Table A1.5: Summary of particle size analysis based on twenty eight samples

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Average</th>
<th>Max</th>
<th>Min</th>
<th>St. Dev</th>
</tr>
</thead>
<tbody>
<tr>
<td>D90</td>
<td>1.179</td>
<td>1.773</td>
<td>0.946</td>
<td>0.262</td>
</tr>
<tr>
<td>D50</td>
<td>0.501</td>
<td>0.555</td>
<td>0.459</td>
<td>0.027</td>
</tr>
<tr>
<td>D10</td>
<td>0.224</td>
<td>0.252</td>
<td>0.183</td>
<td>0.015</td>
</tr>
</tbody>
</table>

Table A1.5 shows that the D90 criteria are widely ranging and likely due to the presence of random oversize within the sand mix. The average value is nearer to the minimum showing there is positive 'skewness', or the more random values occur towards the higher end of D90 particle dimensions. The D50 and D10 show a low degree of variability with low standard
deviations in comparison to the D90. This indicates that there is little deviation for the finer fraction of the sand.

A1.3.3 Stillwater Mine

As the database was provided as-is by the mine - the testing had been completed - the particle size distribution was not part of the available data. Further, as the samples that were provided were cemented, it was not possible to perform a particle seize analysis.
APPENDIX 2

Summary of experimental testing
Table A2.1: List of prepared Red Lake CPB samples by cement, age of test, and nominal diameter

<table>
<thead>
<tr>
<th>% Cement</th>
<th>Age of Test Samples (Days)</th>
<th>Nominal Diameter (in)</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>7</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>7</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
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<table>
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<th>% Cement</th>
<th>Age of Test Samples (Days)</th>
<th>Nominal Diameter (in)</th>
<th>Quantity</th>
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Table A2.2: Summary of results of experimental lab UCS testing (continued)

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Table A2.2: Summary of results of experimental lab UCS testing (continued)

| Sample Set | Cement Content (%) | Age (days) | Sample Name       | Diameter (mm) | Height (mm) | Area (mm²) | Load (kN) | Strength (kPa) |
|------------|---------------------|------------|-------------------|---------------|-------------|------------|-----------|----------------|---------------|
| Stillwater | 10                  | 45         | 35E8600_745pm     | 76.86         | 124.67      | 4,640      | 4255      | 917            |
| Stillwater | 10                  | 45         | 35E8600_545pm     | 77.05         | 134.93      | 4,663      | 4749      | 1019           |
| Stillwater | 10                  | 45         | 35E8600_345pm     | 76.84         | 133.26      | 4,637      | 1878      | 405            |
| Stillwater | 10                  | 45         | 35E8600_245pm     | 77.04         | 137.04      | 4,661      | 3843      | 824            |
| Stillwater | 10                  | 45         | 35E8600_145pm     | 76.75         | 136.04      | 4,626      | 3529      | 763            |
| Macassa    | 8                   | 28         | Pure Paste 1      | 39.01         | 68.58       | 1,195      | 899       | 752            |
| Macassa    | 8                   | 28         | Pure Paste 2      | 37.55         | 78.8        | 1,107      | 886       | 801            |
| Macassa    | 8                   | 28         | Pure Paste 3      | 37.29         | 62.12       | 1,092      | 678       | 621            |
| Macassa    | 7                   | 28         | 50 Sand:50 Paste #1 | 33.01     | 61.88       | 856        | 1306      | 1526           |
| Macassa    | 7                   | 28         | 50 Sand:50 Paste #2 | 35.35     | 70.12       | 981        | 1519      | 1549           |
| Macassa    | 7                   | 28         | 50 Sand:50 Paste #5 | 36.37     | 55.14       | 1,039      | 1224      | 1178           |
| Macassa    | 7                   | 28         | 50 Sand:50 Paste #6 | 36.61     | 63.91       | 1,053      | 1535      | 1458           |
| Macassa    | 5                   | 28         | 70 Sand:30 Paste #4 | 37.33     | 62.44       | 1,094      | 493       | 451            |
| Macassa    | 5                   | 28         | 70 Sand:30 Paste #6 | 35.55     | 60.58       | 993        | 518       | 522            |
Table A2.3: Summary of results of penetration test at NBK Labs

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Table A2.3: Summary of results of penetration test at NBK Labs (continued)

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APPENDIX 3

Cemented Paste Backfill Procedures
This document covers the quality control procedures of paste backfill at Kirkland Lake Gold Inc.’s Macassa mine. Detailed descriptions for the practice and test methods for paste backfill sampling, the making of test cylinders, and the testing procedures of cured backfill cylinders are listed.

The purpose of the quality control is to verify that the paste backfill is entering the underground mine as designed with respect to slump and strength. The procedures are in place to ensure that testing can be done in a reproducible manner, allowing for comparison between different backfill batches.

This document details only the procedures for testing and does not discuss in detail safety standards that should be obeyed during the testing. This document is to be read in conjunction with the appropriate safety manuals. In the event of a discrepancy between the safety standards and these test procedures, the safety standards are to supersede the testing procedure.

Definitions

Batch:

*Production of a quantity of backfill per mix, based on weight and mix design, i.e. tons.*

Mix:

*The designated name for the backfill based on the amount of sand, tails, water and, binder used in mixing. The mix is abbreviated by the cement content, i.e. 3%, 7%, 10%*

Load:

*The amount of material, by weight, within a concrete-mixer truck. Assumed to be 14 tons. 1 Load = 14 tons.*

For example, if in one day 560 tons of 7% backfill are delivered underground using 40 cement truck loads and 400 tons of 3% backfill are delivered underground using 29 cement truck loads. Two mixes were used: 3% and 7%. One batch was produced for each mix; Two batches total for day. The 3% batch was 400 tons; the 7% batch was 560 tons. The 960 tons were delivered in 69 loads.

Test cylinder:

*A cylinder that has been prepared for Ultimate Compressive Strength testing (UCS). Typical cylinders are 4” in diameter by 8” in height with a plastic cap. Test cylinders should be labelled to identify the designated location of the backfill, the mix, the date of casting and the load of which the sample was taken.*
Test cylinder set:

Three cylinders that are prepared from the same batch and load. Sets should be prepared at the same time and have a sequence.

For example, a cylinder labelled 5317.37MCF/10-May-31/7%/14/B would refer to a 7% backfill batch mix prepared for 5317.37 MCF stope on May 31, 2010, the sample was taken from the 14 truck load and it is the second of three cylinders from the set.

Sampling Procedures

Proper sampling allows for representative samples to be obtained to determine the compliance and quality requirements of backfill.

- Sample ID should be of the following form: Stope/Date/Paste Recipe/Load. As an example: 3427.30PCF#17/10-Jul-9/7%/12 would describe a backfill batch destined for 3427.30 PCF #17 Stope, prepared on July 9, 2010 with 7% cement by weight taken from the 12th truck of the batch.
- A minimum of one backfill test cylinder set should be prepared (3 cylinders per set) for each backfill batch mix. The test cylinder set should be prepared approximately in the middle of the batch.
- In cases where the batch mix is over 500 tons, two sets of backfill test cylinders should be prepared. The first set of cylinders should be prepared at the 1/3 mark of the batch and the 2nd set of cylinders should be prepared at 2/3 mark of the batch.
- Samples should not be taken within the initial or final 10% of the backfill poured per cement truck. Again, a representative sample is the goal of the sampling.
- The time between sampling for backfill, performing the slump test and final preparation of test cylinders should not exceed 15 minutes.
- Sampling should be performed either at the dispatch plant or at the hopper at the paste borehole intake.
- Samples should be obtained by placing a bucket or shovel at the discharge of the chute of the concrete truck.
- The sample should not contain any oversized material (1/2” aggregate or binded lumps of backfill).

Materials Required:

- Large Bucket, shovel, hand scoops and safety equipment as required.

Standard Practice for Making and Curing Backfill Test Cylinders

The backfill cylinders are to be used for testing the unconfined compressive strength (UCS) of the backfill. Without proper preparation of the test cylinder the UCS value of the material is meaningless. It is imperative that the test cylinder be prepared properly to ensure proper strength results and to reduce testing costs due to poor samples.

Samples should be stored in a covered, humid environment with a constant temperature of 23 °C +/- 2 °C. The storage area should be level with samples being place upright at all times. Below are the recommended procedures:
Three cylinders should be prepared for each batch poured.
Cylinders should be new for each test procedure and holes should be drilled in the base of the cylinder to allow free drainage.
Sample ID should be of the following form with individual identification for each sample by a letter (A, B or C): **Stope/Date/Paste Mix/Load/(A,B,C)**. As an example: **3427.30PCF#17/10-Jul-9/7%/12/A**
Sample ID should be placed both on the lid and side of the cylinder
Samples should be prepared in two equal lifts with rodding occurring between the lifts.
Rod each layer 25 times uniformly over the cross section with the rounded end of the rod.
After rodding, tamp the outside of the cylinder with the rod 10 times to remove any trapped air pockets within a sample.
Ensure that the backfill reaches the top of the cylinder. It is best practice to over fill the cylinder, performing the rodding and then level of the remaining backfill by knifing the rod over the top of the cylinder.
Clean the edges of the cylinder with a damp sponge to ensure that the cap will seal properly.
Place the sample in storage box, ensuring that the sample remains level and that the cylinder is not squeezed.
Figure 1 illustrates the common problems associated with cylinder preparation

**Materials needed:**

Three new cylinders with lids, damp sponge, hemispherical rod and permanent marker.

**Figure A3.1: Common Issues with Sample Cylinders**
Ultimate Compressive Strength Test Procedures

The testing method should follow ASTM C39/C39M-09a with the following alterations:

- One sample per test cylinder set should be tested at 14 days +/- 1 day, while the remaining two should be tested at 28 days +/- 1 day.
- Alteration Paragraph 6.2 - Record the perpendicularity of the sample with respect to the vertical axis to the nearest tenth of a degree. Test should be performed on all samples regardless of the perpendicularity.
- Alteration 6.3 - Record the individual sample heights and diameter to the nearest one-tenth of a millimetre using calibrated digital measuring callipers. Three measurements of each should be taken and the average of both the length and diameter should be used in the calculations.
- Alteration 6.4 - Record the weight of the sample as per
- Alteration 7.1 - Samples should be removed from their plastic cylinders prior to breaking
- Alteration 7.5 - The loading rate of the sample shall be done in one steady rate from a nominal seat load. The loading rate should be set to 7.5 kN/min.
- 7.6 - Record the type of failure as per Figure 2 of standards.
- Perform a moisture content of the failed sample as per ASTM D2216-05

At a minimum the following should be recorded for each test cylinder:

- Stope Location
- Mix Design
- Date Cast
- Date Tested
- Sample Weight (kg)
- Sample Height (mm)
- Sample Diameter (mm)
- Measure of the planarity taken from the highest point
- Peak Load (kN)
- Compressive Strength (MPa and PSI)
- Moisture Content of Failed Sample

The results of the testing should be entered within a database that allows for review. The database should have all information updated weekly. Monthly reports should be prepared that detail the variation of strength over time and note any changes in the paste recipes.
APPENDIX 4

Mohr-Coulomb model fit of axial stress-strain test data
Figure A4.1: Stillwater Sample 35E8600_745pm Mohr-Coulomb fit

Figure A4.2: Stillwater Sample 35E8600_145pm Mohr-Coulomb fit
Figure A4.3: Stillwater Sample 35E8600_545pm Mohr-Coulomb fit

- Modeled Results: Elastic Modulus: 408 MPa, UCS: 1016.52 kPa, Friction Angle: 15°, Cohesion: 390 kPa
- Lab Test: 35E8600_545pm: Elastic Modulus: 408 MPa, UCS: 1017.80 kPa

Figure A4.4: Red Lake Sample #1 Mohr-Coulomb fit

- Modeled Results: Elastic Modulus: 5 MPa, UCS: 85.69 kPa, Friction Angle: 20°, Cohesion: 30 kPa
- Lab Test: 35E8600_545pm: Elastic Modulus: 408 MPa, UCS: 1017.80 kPa
Figure A4.5: Red Lake Sample #2 Mohr-Coulomb fit

- Modeled Results / Elastic Modulus: 5 MPa / UCS: 55.06 kPa / Friction Angle: 18 / Cohesion: 20 kPa

Figure A4.6: Red Lake Sample #3 Mohr-Coulomb fit

- Modeled Results / Elastic Modulus: 4 MPa / UCS: 99.97 kPa / Friction Angle: 20 / Cohesion: 35 kPa
Figure A4.7: Red Lake Sample #4 Mohr-Coulomb fit

Figure A4.8: Red Lake Sample #5 Mohr-Coulomb fit
Figure A4.9: Red Lake Sample #6 Mohr-Coulomb fit

Differential Stress (kPa) vs. Axial Strain

- Modeled Results
- Elastic Modulus: 40 MPa
- UCS: 257.07 kPa
- Friction Angle: 20°
- Cohesion: 90 kPa

Figure A4.10: Red Lake Sample #7 Mohr-Coulomb fit

Differential Stress (kPa) vs. Axial Strain

- Modeled Results
- Elastic Modulus: 68 MPa
- UCS: 253.17 kPa
- Friction Angle: 21°
- Cohesion: 87 kPa
Figure A4.11: Red Lake Sample #8 Mohr-Coulomb fit

Differential Stress (kPa) vs. Axial Strain

Modeled Results / Elastic Modulus: 30 MPa / UCS: 237.21 kPa / Friction Angle: 22 / Cohesion: 80 kPa

Figure A4.12: Red Lake Sample #9 Mohr-Coulomb fit

Differential Stress (kPa) vs. Axial Strain

Modeled Results / Elastic Modulus: 93 MPa / UCS: 185.66 kPa / Friction Angle: 20 / Cohesion: 65 kPa
Figure A4.13; Red Lake Sample #10 Mohr-Coulomb fit

Figure A4.14; Red Lake Sample #11 Mohr-Coulomb fit
Figure A4.15: Red Lake Sample #12 Mohr-Coulomb fit

- Modeled Results
- Elastic Modulus: 116 MPa
- UCS: 371.32 kPa
- Friction Angle: 20
- Cohesion: 130 kPa

Figure A4.16: Red Lake Sample #13 Mohr-Coulomb fit

- Modeled Results
- Elastic Modulus: 107 MPa
- UCS: 271.07 kPa
- Friction Angle: 15
- Cohesion: 104 kPa
APPENDIX 5

Numerical model outputs
Figure A.5.1: FLAC output: convergence failure (Trail #1)
Figure A.5.2: FLAC output-convergence failure (Trail #2)

Figure A.5.3: FLAC output-convergence failure (Trail #3)
Figure A.5.2: FLAC output- shear failure (Trail #1)

Figure A.5.3: FLAC output- shear failure (Trail #2)
Figure A.5.4: FLAC output - shear failure (Trail #3)

Figure A.5.5: FLAC output - rotational failure (Trail #1)
Figure A.5.6: FLAC output - rotational failure (Trail #2)

Figure A.5.7: FLAC output - rotational failure (Trail #3)
Figure A.5.8: FLAC output- flexural failure (Trial #1)

Figure A.5.9: FLAC output- flexural failure (Trial #2)
Figure A.5.10: FLAC output - flexural failure (Trail #3)
APPENDIX 6

Ground Support Numerical Model Outputs
Table A6.1 – Ground support model parameters

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<tr>
<th>Case</th>
<th>UCS (kPa)</th>
<th>Friction Angle</th>
<th>Ratio of Cohesion/UCS</th>
<th>E (MPa)</th>
<th>Surcharge (kN/m)</th>
<th>Depth (m)</th>
<th>Height (m)</th>
<th>Width (m)</th>
<th>Stoppe Dis</th>
<th>Amount of Closure (m)</th>
<th>Analytical Factor of Safety</th>
<th>Failure Type</th>
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Figure A6.1 – Case 1: Failed Closure Model with Shear paddles
Figure A6.2 – Case 1: Failed Closure Model with angled rebar bolts

Figure A6.3 – Case 1: Failed Closure Model with stand-up rebar bolts
Figure A6.4 – Case 3: Failed (shear) beam with stand-up rebar bolts

Figure A6.5 – Case 3: Failed (shear) beam with shear paddles
Figure A6.6 – Case 3: Failed (shear) beam with angled bolts

Figure A6.7 – Case 3: Failed (shear) beam with split sets
Figure A6.8 – Case 5: Failed (rotational) beam with stand-up rebar

Figure A6.9 – Case 5: Failed (rotational) beam with shear paddles
Figure A6.10 – Case 5: Failed (rotational) beam with angled rebar

Figure A6.11 – Case 5: Failed (rotational) beam with split sets
Figure A6.12 – Case 7: Failed (flexural) beam with stand-up rebar

Figure A6.13 – Case 7: Failed (flexural) beam with shear paddles
Figure A6.14 – Case 7: Failed (flexural) beam with angled bolts

Figure A6.15 – Case 7: Failed (flexural) beam with split sets