PERFORMANCE OF PASTE FILL FENCES
AT RED LAKE MINE

by

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ABSTRACT

Advancements in technology in mining have allowed previously unfeasible ore bodies to be developed. Paste backfill is one technological advancement that has allowed for the development of high-grade, low tonne production when employing the cut and fill mining method. Goldcorp Inc.'s Red Lake Mine currently utilizes this method and is the site for the study of this thesis.

Paste backfill (paste) is defined as a mine backfill material that consists of eighty-five percent solids by weight and does not bleed water when placed often consisting of between two and fifteen percent Portland cement by weight. A paste barricade or paste fill fence is a constructed barricade whose purpose is to retain backfill within a mined out stope. The construction of the barricade varies with different operations, for Red Lake Mine the barricade consists of an anchored rebar skeleton covered with an adequate thickness of shotcrete.

The majority of the applicable barricade research focuses on hydraulic fill barricades in open stope mining. The barricade pressures in these instances are much larger than those experienced in paste backfill barricades. As such, the current paste loading theory is based on material with a different loading mechanism. Although some research is currently underway, the majority of the barricade research is based on brick barricades and not the shotcrete, rebar skeleton as used at Red Lake.

Catastrophic failures of barricades can occur without an understanding of the loading mechanisms. Based on the catastrophic risk, this thesis provides an investigation into the behaviour of the paste backfill and paste barricades at Red Lake Mine in order to provide a safe, cost effective design of paste barricades.

This thesis develops an understanding of paste loading mechanisms and barricade capacity derived from a field study of nine instrumented fill fences at Red Lake Mine. Eight of the fences were instrumented to monitor the reaction strain in the fence and the applied pressures during standard production paste pours, the ninth fence was a controlled destructive test that determined the ultimate capacity of the fence. The data for these tests were gathered in real time and was subsequently reduced to assist in analysis. Yield Line Theory, Rankine Theory, strain induced stress, stress vs. strain analysis and numerical modeling were used to develop an understanding of the paste loading mechanisms and the capacity of the paste fill barricades. The analysis determined that the paste backfill behaves as a Rankine-like soil in the initial stages of loading with an average coefficient of lateral earth pressure, Ka, of 0.56. The destructive test
determined that the yielding stress of a paste barricade is approximately 100 kPa. Further findings from the research determined that the rate of placement of paste does effect the loads applied to the fence and that the largest pressures exerted on the fill fence occur when paste lines were flushed with water at the end of the pour.

This thesis provides an understanding of the paste loading and fill fence interaction with respect to failure. Based on this research the Red Lake Mine should be able to increase production without increasing risk to mine personnel by quantifying the overall loading and strengths of the fill barricade.
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GLOSSARY

Aquacrete: A proprietary gypsum based shotcrete.

Barricade: A constructed wall whose purpose is to retain backfill within a mined-out stope. Can be constructed of timber, concrete, brick and shotcrete.

Binder: Material used to cement particles together, provides true cohesion within placed fill.

Bulkhead: A constructed barrier that separates sections of a mine, no external loads applied. Typically used for ventilation purposes.

Fibercrete: A type of shotcrete that provides additional tensile and shear strength.

Fill Fence: Specific type of barricade consisting of an anchored rebar skeleton and shotcrete. Barricade type used at Red Lake Mine.

Friction Angle: Maximum angle of a stable slope of a material determined by friction, cohesion and the shapes of its particles.

Load: A measured applied force. Measured in tonnes, kN or MN.

Loading Mechanics: Manner in which stress is applied to an element, ex. hydrostatic, self-weight, Rankine soil. Rate, material properties and stress conditions are factors the loading mechanics.

Modulus of Elasticity: Mathematical description of an object tendency to be deformed elastically when a force is applied to it. Defined as the slope of the stress-strain curve in the elastic deformation region.

Mohr-Coulomb: Mathematical model that relates the shear strength to the stress of a material element, equation: \( \tau = c + \sigma \tan(\theta) \). Materials behaving according to the theory are referred to as Mohr-Coulomb material.

Paste: High density thickened tailings with binder. Used in underground mine fill.

Poisson's Ratio: Ratio of the amount of lateral strain to axial strain.

Pressure: Force per unit area. Measured in kPa, MPa or tonnes/m².

Rankine Soil: A soil that behaves according to Rankine's Lateral Earth Pressure Theory that states that horizontal pressure relate to vertical pressure through a constant "K" (\( K = \sigma' / \sigma_v \)).
Shotcrete: Concrete that is pneumatically sprayed onto a surface. Consists of Portland Cement, blended aggregates, accelerator and silica fume. When dried has shear strength of 2 MPa.

Stope: Mine excavation that its purpose is to remove the ore.

Strain: Total deformation of an element in terms of the element's original length. Units are dimensionless, referred to in terms of ε and με (micro strain).

Stress: Internal resistance offered by a unit area of a material from which a member is made to an externally applied load. Measured in kPa, MPa or tonnes/m^2.
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1 INTRODUCTION

The purpose of this thesis is to investigate the loading mechanisms of paste and determine the capacity of paste barricades during loading in order to provide a complete understanding of paste barricades at Goldcorp Inc. Red Lake Mine. The current paste barricade employed at the Red Lake Mine is based on assumed loading mechanisms and barricade ultimate loads. Three questions are to be answered by this research:

- What are the loading mechanisms of paste against barricades?
- What is the capacity of the paste barricades at Red Lake?
- Based on the loading mechanism of the paste, do the fill fences pose a risk of failure?

Paste is defined as a mine backfill material that consists of eighty-five percent solids by weight, does not bleed water when in place, and consists of between one and ten percent Portland cement by weight. (Potvin et al., 2005)

A paste barricade, or interchangeably a paste fill fence, is a constructed barricade whose purpose is to retain backfill within a mined-out stope. The construction of the barricade varies with different operations. At Red Lake Mine, the barricade consists of an anchored rebar skeleton covered with an adequate thickness of shotcrete. Backfill rates are limited by the capacity of the paste plant and fence design. Understanding the loading mechanisms of fill barricades will allow for an increase in production by maximizing the pour rate from the paste plant.

This thesis aims to provide a reference for paste barricade design in underhand cut and fill. The majority of the applicable barricade research focuses on hydraulic fill barricades in open stope mining. The barricade pressures in these instances are much larger than those experienced in paste backfill barricades. As such, the current paste loading theory is based on material with a different loading mechanism. Although research by Revell and Sainsbury (2007a & 2007b) is currently underway, the majority of the paste barricade research is based on brick barricades and not the shotcrete, rebar skeleton used at the Red Lake Mine.
Catastrophic failures of barricades can occur without an understanding of the loading mechanisms; Australia has recorded fatalities due to barricade failure (Grice, 1998). This thesis provides an investigation into the behaviour of the paste and paste barricades at the Red Lake Mine in order to provide a safe, cost effective design of paste barricades, and so to minimize risk.

This research consists of the analysis of nine field tests of instrumented paste barricades at Red Lake. Data gathering for this thesis was carried out by Goldcorp ground control personnel and research was performed by co-op students Ali Rana and Josh Clelland, by UBC graduate student Kathryn Dehn, and by the author Paul Hughes. The author made three site visits to the Red Lake Mine site in August 2006, January 2007 and August 2007. On-site work consisted of refining existing data into a more manageable form, as well as designing and implementing a destructive fill fence test. Further contributions were the theoretical, analytical and numerical analyses of all nine fences, which form the majority of this thesis.

This thesis consists of nine chapters that include a description of the Red Lake Mine operation, a literature study of current research, results from an instrumentation program and subsequent analysis.

Chapter 2 details the local geology of the mine, the discovery of the high-grade zones at Red Lake Mine and the mining methods necessary to extract the ore.

Chapter 3 focuses on the current research on different types of barricades, numerical analysis of loading and fence performance and a summary of reported barricade failures of barricades, their consequences and their implications in future design.

Red Lake Mine carried out an instrumentation program of nine fill fences over a three-year period. Chapter 4 discusses the rationale of the program, the instrumentation design and the layout of instruments for each individual barricade. Chapter 5 reports the data gathered from the fences.

Analysis of the recorded data is presented in Chapters 6 and 7, and is used to determine the behaviour of both the paste and barricades during loading. The analysis consists of theoretical, analytical and numerical modeling approaches. The numerical analysis uses finite element software to determine the loading characteristics of the paste and the barricade’s
response to the loading. The conclusions summarize the thesis findings and provide recommendations for developing a safe working environment for the use of paste barricades.

The contribution of this thesis to the operations of Red Lake Mine is that, by understanding the strength of their paste barricades, operational modifications can be made that provide both economic and safety benefits. The economic benefits are: understanding the interaction between paste and paste barricades and fence capacities can improve production. The increase in production can be realized by increasing fill pour rate, ultimately leading to continuous paste pours, which would allow more rapid backfill times for stope completion. A potential safety benefit is that by increasing the fill rate, discontinuities between successive paste pours would be minimized; these discontinuities create groundfall hazards in underhand cut and fill stopes. A further safety benefit is that by understanding the capacity of paste barricades, the risk of designing a backfill stope that would exceed a barricade’s capacity can be largely eliminated.

The anticipated contributions of this thesis are novel methods of determining paste barricade capacity, understanding of the loading mechanism of paste (as currently this has not been researched), and defining of the limitations of instrumentation used to monitor paste barricades. This thesis, based on a comprehensive review of the literature, presents the first instrumented, destructive fill fence pull test. This is critical because it determines the ultimate capacity of the fence and how it relates to the loads that are applied to the fence during paste pours. Further, the loading mechanism for paste against a barricade is not known, or at least has not been previously published. This thesis presents the relationship between fill height and lateral pressures based on instrumentation from field studies of paste pours. This relationship is critical in the design of barricades and the numerical modeling of paste. Since nine barricades were instrumented, various observations of instrumentation performance were noted. This thesis presents a thorough analysis of instrumentation used and should provide guidance to any further research in instrumenting paste barricades. This thesis aims to provide a valuable academic contribution and should serve as a reference to mine operators in the design of safe and economical future barricades.
2 RED LAKE MINE SITE

The Red Lake region of Ontario is has had mines continuously operating since the early 1900’s (Twomey, 2002). The Red Lake Region is located within the eponymous Red Lake Greenstone Belt that has undergone a significant amount of structural deformation and intrusions contributing to the gold bearing rocks in the region that form the ore body of Red Lake Mine (Kumar, 2003).

The Red Lake Mine extracts ore from a world-class gold deposit. With Goldcorp’s defining of the High Grade Zone at depth in 1994 (Twomey, 2002), Red Lake Mine had to develop novel methods to mine the ore in a cost-efficient manner. Through the development of mechanized overhand cut and fill, underhand cut and fill and pillar extraction, the mine has become a leader in high-grade, low tonnage mining. With the selection of the mining methods, paste backfill technology was employed at the mine to provide a safe engineered backfill that can be used as a structural element in mine sequencing.

Geographically, Red Lake Mine is located in western Ontario within the township of Balmertown, Ontario. The nearest cities to the mine are Kenora and Dryden, Ontario (Figure 2.1).
2.1 Red Lake Mining History

The Red Lake District is home to a world-class gold deposit that has yielded more than 20 million ounces of gold over its 70-year history (Twomey, 2002). Historical mines in the region have included the Cochenour, Wilanour, Mackenzie Red Lake, Gold Eagle, Howey, Goldshore, and Wilmar mines.

Mining on the Red Lake property dates back to 1946, with production commencing in December of 1948. The mine at this time was named the Arthur White mine and was owned and operated by Dickenson Mines Ltd. The mine consisted as a narrow ore body deposit with grades of 17 grams/tonne. As mining progressed deeper, the ore characterization changed into a disseminated sulfide ore with grades of 10 grams/tonne (Twomey, 2002, p. 19).

Goldcorp purchased the mine from Dickenson Mines Ltd. in 1989 and executed an exploration program to find and develop new resources (Twomey, 2002). Based on results from initial exploration holes from 1989 to 1991, Goldcorp set out to test the extents of the new mineralization. In 1994, a significant discovery was made with the discovery of a zone with a grade of 311 grams/tonne over 2.3m; this was the initial find known as the High Grade Zone.
Recent audits of the HGZ show that it contains an average grade of 70.3 grams per tonne within 1.85 million tonnes (Twomey, 2002, p.20).

With the discovery of the HGZ, the focus of the mine switched to a high grade-low tonnage operation that utilized mechanized cut and fill methods and paste backfill. With mining methods optimized for the ore body, Goldcorp currently has a unit cost of $238 per ounce of gold mined. (Goldcorp, 2007)

Goldcorp Inc.’s Red Lake Mine acquired the Placer Dome’s Campbell Mine in 2006. With these two mines joined, the combined resources are 23.5 million ounces of gold, with an average grade of 22.3 grams per tonne (g/t) (Goldcorp, 2007).

2.2 Regional Geology

The Red Lake Mine is situated in the Red Lake Greenstone Belt within the Uchi Subprovince, an Archean granite-greenstone terrain made up of sedimentary, volcanic and syn-volcanic intrusions (Twomey, 2002). Figure 2.2 shows the regional geology of the Red Lake Mine. The age of these rocks is approximately 290 million years.

The greenstone belt, in which the Red Lake Mine is situated, is composed primarily of several distinct assemblages. Those relevant to the mine are the Balmer and Bruce Channel Assemblages (Kumar, 2003).

The Balmer Assemblage consists primarily of tholeiite and komatiite lava flows that are between 2,992,000 and 2,959,000 years in age (Twomey, 2002). The Balmer assemblage is host to the Red Lake Mine and is the most favorable in terms of gold in the region.
The Balmer assemblage is unconformably overlain by the Bruce Channel, which was formed 2,894 million years old (Kumar, 2003). The Bruce Channel is composed of metasedimentary rocks and is folded around the Balmer assemblage near the Red Lake Mine (Twomey, 2002).

The rock in the region has undergone metamorphisms to a greenschist facies with contact amphibolite metamorphism present around the aureole of the batholith intrusions; examples of these are visible at the Red Lake Mine (Andrews and Hagon, 1996).

Deformation at Red Lake occurred during two major structural events. The first episode (D1) took place after the late volcanisms (2,740 million years old) of the region and produced an associated fold (F1) and foliation (S1). This deformation caused the large-scale northeast trending folds of the region. The second deformation event (D2) corresponds to a:

“north-east / south-west compression responsible for the development of a penetrative southeast trending schistosity (S2), that is axial planar to a steeply east plunging fold (F2)” (Twomey, 2002).
This deformational event is responsible for the Dickenson and Campbell faulting which is an essential geological structure for Red Lake Mine (Kumar, 2003).

### 2.3 Local Geology

The mine is situated within the hanging wall side of the southeast limb in a fold (anticline) of the Red Lake greenstone belt (Kumar, 2003). The stratigraphy of the mine site consists of a host mafic unit consisting of pillowed or massive basalts that are banded to strongly foliated. Intercalated with the mafic rocks are highly carbonated and altered units varying from rhyolite breccia to talc-chlorite schist to carbonized metamorphic rocks. Above the package of the mafics is a mix of felsic flows, pyroclastic, clastic and chemical sedimentary rocks. (Kumar, 2003)

Figure 2.3 shows a cross section of the geology based on the condemnation hole for the recently completed Shaft #3. It should be noted that this is a simplified geology of the mine. As an example, Twomey (2001) argues that the Andesite within the mine is misidentified and that it is a variously altered, Fe-Tholeiite. However, for engineering purposes the rocks have been simplified to ease identification.

A series of post mineralization intrusions have occurred consisting of quartz feldspar porphyries, metadiabase, serpentinite/periodite and lamprophyres dykes that are apparent throughout the underground workings.
Figure 2.3: Cross Section of Geology Based on Condemnation Hole for Shaft #3, after Pakalnis (2003)
### 2.4 Mine Production

The Red Lake Mine complex produced 592,000 ounces and 616,000 ounces of gold in 2006 and 2005 respectively. Revenues for these periods were $360,800,000 and $362,000,000. The tonnes milled in 2006 were 768,000, grading 28 grams/tonne and in 2005 were 675,000 tonnes with an average grade of 38 grams/tonne. The mining costs associated with mining at Red Lake are $195 per tonne and $132 per tonne for 2006 and 2005 respectively. (Goldcorp, 2007) The difference in grade and cost per tonne are most likely due to the acquisition of the Campbell Mine from Placer Dome Inc., which is a lower grade, higher tonnage producing mine.

Red Lake currently mines between 550 and 700 tonnes per day with a mill capacity of 800 tonnes per day. With current plant upgrades and Shaft #3 coming on line, it is anticipated that 1200 tonnes per day will be processed from the underground workings. (Goldcorp, 2007)

### 2.5 Mining Practices at Red Lake

Historically Red Lake has used various mining methods including mechanized cut and fill, double level captive cut and fill stoping, and longhole sill removal (Mah, 2003). Due to structural instabilities or bursting associated with mining at Red Lake mine, they currently utilize two mining methods: primarily mechanized cut and fill ("MCF") and occasional long hole stoping. MCF commenced below the 31 level at Red Lake Mine due in part to both high-grade gold and the high stresses developed through mining.

The selection of mining methods is based on the following criteria as presented by Mah et al. (2003):

- selectively mine highly variable, high grade zones by allowing geology control of each ore blast,
- reduce dilution in order to optimize the limited hoisting capabilities,
- reduce seismic activity by minimizing span, eliminating sill pillars, and better control of the mining sequence,
- provide the available mining faces to ensure 650-700 tonnes per day,
- maximize ore recovery; and
- develop methods that would aid in mining at future depths.
Based on the above list, the following methods are currently used at Red Lake Mine: overhand cut and fill, underhand cut and fill, and sill pillar recovery. Table 2.1 shows the employed mining methods for the period between 2002 and 2004.

Table 2.1: Mining Methods in practice between 2002 and 2004 at Red Lake (Mah et al., 2003)

<table>
<thead>
<tr>
<th>Mining Method</th>
<th>2002</th>
<th>2003</th>
<th>2004</th>
</tr>
</thead>
<tbody>
<tr>
<td>overhand cut and fill</td>
<td>100%</td>
<td>70%</td>
<td>60%</td>
</tr>
<tr>
<td>underhand cut and fill</td>
<td>0%</td>
<td>25%</td>
<td>25%</td>
</tr>
<tr>
<td>sill pillar recovery</td>
<td>0%</td>
<td>5%</td>
<td>15%</td>
</tr>
</tbody>
</table>

2.5.1 Overhand Cut and Fill

Mechanized overhand cut and fill ("MOCF") is the most widely used mining method employed at Red Lake. The first stopes mined with this method are the 31-1 Sub, 32 Sub, 34 Level, 36-2 Sub and 37 Levels.

MOCF is a low tonnage mining method that requires the use of jumbo drilling to advance horizontal stopes. Typically, a MOCF stope has three to six stopes per level. Figure 2.4 illustrates the layout of a MOCF level. Note that attack ramps are driven to a maximum of +/- 20% as to minimize wear on equipment and to minimize the development between successive lifts. Attack ramps are backfilled with waste rock. As one stope is being mined, typically another is being filled ensuring a continuous mining strategy.
Red Lake uses captive overhand cut and fill stoping in areas that are remote from mine infrastructure and where development in waste would be excessive (Mah et al., 2003). Access is gained from the top by a 2.1 m x 2.1 m drift or from below with the use of a ladder. Captive overhand cut and fill stoping represents a very small amount of the total tonnages at the mine.

2.5.2 Underhand Cut and Fill

Mechanized underhand cut and fill ("UCF") is utilized at Red Lake in an effort to extract ore under a weak or highly stressed back. UCF commences with the construction of the sill mat in the first cuts of the stope, this is described in Chapter 2.5.4. Once the sill mat is created, the next successive cut is developed immediately below. 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. Heights of stopes vary with maximum observed to be 4 m.

The advantages of the UCF method are:

- the back is engineered to provide a safe working environment in high stress areas,
- wall dilution is low in comparison to MOCF; and
- seismic risk is reduced (Mah et al., 2003).
2.5.3 Sill Pillar Recovery

Sill pillar recovery is utilized at Red Lake where induced stress in the pillar exceeds the safe level of 0.6 times the Unconfined Compressive Strength, UCS, of the rock (Kumar, 2003). The method involves an initial step of blasting a destress slot to reduce the induced stress in the pillar. Once the destressed pillar is in a relaxed mode, underhand cut and fill mining progresses to extract the remaining ore. Underhand cut and fill is used as it is a method of mitigating the hazardous ground conditions in the stope due to the reduction of induced stress.

The destress slot reduces the stress within the pillar, Figure 2.5 demonstrates this mining method. Prior to blasting the destress slot, there is an induced principal stress within the pillar that meets the criteria of bursting. Once the destress slot is blasted, the principal stress has dropped to a safe level, and the pillar can be mined by man-entry methods (Pakalnis, 2003).

![Image of destress slot](image)

Figure 2.5: Destress Slot after Pakalnis (2003)
2.5.4 Paste Practices

Red Lake mine incorporated paste backfill technology into their mining practice in 2000. Golder's Paste Technology Inc. designed the backfill plant and it came into production in October of 2000 (Golder, 2000). The plant was designed to provide the required backfill volume using the mine tailings and not be the critical path to production.

The paste is mixed on surface and pumped underground through a gravity fed system. The paste delivery system consist of a main trunk of 12.2 cm diameter boreholes down to 30 level, then switching to a Schedule 80 A53 Grade B pipe, and, closer to the stopes, a combination of 12.2 cm Schedule 40 A53 Grade B pipe and 15 cm DR9 HDPE pipe. The total length of the paste system is approximately 2500m. Average velocities within the backfill system have been measured to be 1.83 m/s (Mah et al., 2003). Considering that the batch plant can produce 80 tonnes per hour of paste with average unit weight of two tonnes/m$^3$, the average pouring rate at the stope is 40 m$^3$/hr. The quantity of material prepared by the plant is referred to as a batch, calculated to be 10 tonnes of paste.

The paste recipe at Red Lake is between 5-10% Portland cement by dry weight (Mah et al., 2003). Typically, the higher cement contents are used for underhand cut and fill sill mats and mucking floors, as it is less susceptible to wear. The lower cement content is used in the overhand cut and fill stopes as the strength of the paste is not crucial for ground support. The definition of percentage cement varies for different mines, as there is some inconsistency with including the weight of water in the calculation of cement content. Due to this inconsistency, caution must be used when comparing the percentage of cement for paste on a mine to mine basis. The percent of cement for the paste is based on the following equation (Golder, 2000):

$$weight \text{ of cement} = \frac{\text{dry weight of tailings}}{(1 - \text{cement \%})} - \text{dry weight of tailings}$$

Eqn. 2.1

Solving for cement \%:

$$cement \% = 1 - \frac{\text{dry weight of tailings}}{\text{weight of cement + weight of dry tailings}}$$

Eqn. 2.2
The unconfined compressive strength ("UCS") of paste at Red Lake Mine is 2MPa. This strength is from paste tested at surface; through observations of cured paste in stopes there appears to be a discrepancy between the surface tests and the cured paste in stopes.

The particle sizes used in the paste are fine tailings from the processing plant. The gradation of the paste in the tailings is shown in Figure 2.6.

Underground, stopes are prepared for paste pours in advance of placement through the preparation of the paste barricade and the preparation of engineered floor for UCF stopes.

UCF stope preparation consists of placement of polyethylene on the ground followed by the placement of stand up rebar with plates attached to both ends and the installation of shear ‘paddles’. The rebar acts as support for the immediate back while the shear ‘paddles’ are designed to prevent rotational failures of the paste. Both supports can be seen in Figure 2.7.
Gradations range between 3 and 100 microns.

Figure 2.6: Tailings Gradation Curve (after Golder 2002)

Figure 2.7: Sill Mat Preparation
Once the stope has been prepared, the construction of the fill fence is started. The following is from Goldcorp’s “Red Lake Mine Organization Manual Procedure: Pastefill Fence Construction” (2002):

- rebar must be anchored 0.6 m to the surrounding solid ground on walls and floor when built on waste or constructed on top of paste,
- holes in rock must be filled with resin and 2.1 m, #6 rebar must be spun into hole,
- fence to be constructed with #6 rebar on a 0.6 m by 0.6 m grid (Figure 2.8),
- rebar connections must overlap a minimum of 45 cm and be secured with a minimum of three areas with #9 galvanized wire,
- secure ‘bed springs’ with tie wraps inside of fence designed height  (Figure 2.8),
- apply minimum of 10 cm of shotcrete; shotcrete should be applied to surrounding area a minimum of 0.6 m (Figure 2.9),
- allow shotcrete to cure for twenty four hours prior to placement of paste,
- if necessary, 10 cm paste pipe to be installed through fence to provide an outlet for the paste line flush,
- when placing paste, no pour is to exceed a maximum height of 1.5 m, and
- when placing successive lifts allow 24 hr set time before placement of second lift.
Figure 2.8: Fence Rebar Skeleton and 'Bed Springs'

Figure 2.9: Completed Shotcrete Fill Fence
In cases where the span is greater than 6 m or if engineering department dictate, the fence is arched to ensure that the fence remains largely in compression with respect to paste loading. A schematic of an arched fence is shown in Figure 2.10. The maximum arch typically is 0.3 m over the entire length of the fence as this appears to be the limit of arching within the rebar.

![Figure 2.10: Schematic Plan View of Arched Fence](image)

An important note regarding paste pours at Red Lake is that when the pour is complete the lines are flushed with water to clear out the lines, preventing future blockages. The lines are flushed to their terminus within the stope and the excess water typically bleeds out through the fill fence.
3 CURRENT PASTE AND PASTE BARRICADE PRACTICES

Paste backfill is used in many mines around the world and in large parts of Canadian mines. Some of the Canadian mines using paste are Breakwater’s Myra Falls, B.C., Falconbridge’s Kidd Creek, ON, Lupin, NT, Teck Cominco’s Louvicourt, QC, and Goldcorp’s Campbell Mine, ON. The following is an overview of current paste and paste bulkhead practices from the literature review.

The purpose of the bulkhead is to retain the paste during placement within mined out stopes. As the understanding of loading on bulkheads evolved, so has the design of bulkheads with various materials being used in construction. Paste bulkheads are constructed in various ways including concrete design, timber frame design, sprayed fibercrete bulkheads and cable sling/burlap construction.

Failures of bulkheads have occurred with catastrophic consequences. Such reported failures will be investigated and how the findings can be applied to fill barricades at Red Lake will be discussed. Studies of failures have included numerical modeling, these are discussed in detail below.

3.1 Paste Practice

Paste backfill was developed in response to a need to dispose of the fine tailings in underground operations. Initially hydraulic fill consisting of fine tailings was developed for this purpose. However problems with hydraulic backfill were the settling of the particles within the slurry, the ponding of water above the settled particles and the low hydraulic conductivities. These problems led to lengthy backfill placement as the water needed to percolate before subsequent lifts could be placed. 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 as it allows for faster placement and ultimately greater mine production.
Potvin et al. (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.

Potvin et al. (2005) provide an excellent overview of paste practices and applications as summarized below:

South Africa began using paste in the late 1980’s as the result of low recoveries from processing of material suitable for use in hydraulic backfill. A problem with paste in South Africa was that the long, deep, and relatively flat ore bodies made pumping paste difficult. This made delivery of paste very expensive as the system needed booster pumps and high pipe pressures were necessary to transport the paste. Based on research to resolve this problem, it was found that the coarser particles within the paste reduced pipeline frictional losses.

Lucky Friday was the first mine in North America to implement paste backfill in an effort to manage high ground stresses. The backfill plant at Lucky Friday consists of a concrete batching plant that produced five tonnes of backfill, which is pumped through a vertical pipeline. The paste delivery within the mine workings is gravity fed and rarely required additional head to deliver paste.

The preparation for filling at Lucky Friday is very similar to that of Red Lake in that the underhand paste sill is constructed by placing welded wire mesh on the floor of the stope and installing ‘stand-up’ rebar to prevent ground falls due to cold-joints when undercut.

Australia initially had little success with paste backfill. Material properties did not meet specifications and either switching to another backfill method or major modifications to design was necessary. However, today Australia is considered one of the leaders in paste development and research is conducted in many mines using paste backfill, including Cannington, Mount Isa and Junction Mine.
In Canada, INCO was the first company to implement paste at the Garson Mine. While the ultimate plan was to have a stiffer backfill to assist in mining, the major benefits were with regards to increase in production. Other mines currently uses paste are: Chimo, Lupin, Louvicourt, Marathon, Campbell and Red Lake. (Potvin et al., 2005)

3.2 Bulkheads

Paste Bulkheads are constructed in various ways including concrete design, timber frame design, shotcrete bulkheads, and cable sling/burlap construction. In all cases, the purpose of the bulkhead is to retain the backfill during placement.

3.2.1 Concrete Bulkhead

Concrete bulkhead construction has taken place since 1970’s as a method of containing hydraulic backfill. Although not typically used at common paste operations, concrete bulkheads are found in literature by Mitchell et al. (1975) and Bridges (2003).

The construction of the bulkhead tends to be a one-meter wide, keyed in concrete block with mousetrap drains that allow excess pore water to dissipate (Mitchell et al., 1975). Mitchell et al. (1975) argue that since pressures are not measured during cemented hydraulic backfill pours, bulkhead tends to be over-designed. To support this, an instrumented concrete bulkhead test was carried out at Fox Mine, Manitoba. Based on preliminary calculations, a horizontal stress of 560 kPa was expected at the base of a 45 m high stope; however, this does not take into account arching or cementation of the backfill that could occur. (Mitchell et al., 1975)

Two concrete bulkheads were instrumented with ‘rubber sandwich cells’ (Mitchell et al., 1975) to determine the pressures build up behind the bulkhead. The bulkheads were instrumented approximately 90 m below desired fill level in the stope on both the hanging wall and footwall drives to determine if the pressure varied between the two positions.

The test showed that after 50 days the loads against the bulkhead were either attenuated or reduced with a maximum measured pressure of 100 kPa (Mitchell et al., 1975). The most likely reason for this reduction is, as the authors suggest in their initial hypothesis, the arching and the cementation of the backfill. This is likely since the backfill tends to demonstrate shear strength behaviour once pore water pressures dissipate.
Based on these findings, the authors of the study recommended that timber bulkheads be constructed instead of the concrete bulkheads, as these were over designed.

From the literature, concrete bulkheads are seldom used as backfill retention systems. However, the design is still used in hydraulic plug design. Below are some observations of concrete bulkheads:

Advantages:
- Able to withstand large pressure, and
- low permeability

Disadvantages:
- difficult construction,
- expensive,
- require specialized labour, and
- tend to be over designed for typical backfill operations.

3.2.2 Timber Bulkhead

The timber bulkhead was used as a free draining design for cemented hydraulic fills. Smith and Mitchell (1982) go into the detail of the design of the timber bulkheads in the paper in which they describe proper backfill techniques. The design of the bulkhead consists of vertical timbers (12.7 x 30.5 cm), 1.3 cm apart, with bearing support provided by laminated beams bolted to the drift floor and back. A free draining fabric is attached to the stope side of the fence and sealed off with cement. (Mitchell, 1982)

The authors suggest that this design construction would be able to withstand pressures of 95.8 kPa before yielding. In addition, the design is suitable for headings not exceeding 4.3m, as the applied pressures exceed the capacity of the timber (Smith & Mitchell, 1982).
From the readings, the following are observations of timber bulkheads:

Advantages:
- easy to assemble,
- made from common mine supplies,
- can be reusable, and
- free draining,

Disadvantages:
- variety in quality of wood affects strength,
- difficulties in transporting material to stope,
- quality control/quality assurance issues,
- labour intensive, and
- proper sealing is necessary in order for piping channels not to develop.

3.2.3 Brick Barricade

Berndt (2007) provides a thorough summary of the design, construction and benefits of the brick barricade. The brick barricade became common practice through the 1980s when concrete bricks replaced traditional wooden barricades. An initial problem with the brick construction during hydraulic fill placement was the use of non-permeable materials, resulting in poor drainage and the buildup of large pore pressure behind the barricade. Permeable bricks reduced bulkhead pressures and allowed for faster placement of backfill. Typically, brick barricade fences are constructed three deep, in an arch, due to the increase in strength, as shown in Figure 3.1.

The brick barricade bulkhead has been tested in three full-scale destructive tests at Mt. Isa; destructive failure strength of 746 kPa was obtained in one test. Two other tests resulted in loads of 460 kPa and 220 kPa before failure. Neither of these fences failed violently rather they commenced leaking once full water height was obtained behind the fence (Grice, 1989). It should be noted though that subsequent analytical modeling by Duffield (2003) determined a failure strength of 427 kPa for permeable concrete brick barricades. This lower value of failure in comparison to the full-scale field test load is most likely due to the numerous assumptions made in simplifying the brick barricade in the model.
Figure 3.1: Construction of Brick Barricade after Berndt (2007)

Through the literature research, it seems that Australia is foremost in researching and using this barricade for backfill retention. Below are observations from the readings regarding the brick barricade.

Advantages:

- industry standard in Australia,
- common construction method,
- no build up of pore pressure, free draining, and
- arch design allows for stronger fence with small change in design layout,

Disadvantages:

- long construction time, the need to prepare the floor, sidewalls and back to allow for proper sealing of the wall,
- weak in shear (Smith & Mitchell, 1982),
- porous bricks must be designed so as not allow the seepage of particles into the media, and
- bricks are prone to variability (Kivakugan, 2004)
3.2.4 Cable Sling Bulkheads

The cable sling barricade design is currently used at the Campbell Mine and has historically been used in the South Deep Gold Mine region of South Africa (Bridges, 2003). Very few case studies are available regarding the use of cable sling burlap fences in industry for backfill retention. However, their design has been covered extensively by Lang (2000) with respect to the design of the bulkheads for fill retention in longhole stopes.

Typical construction, as shown in Figure 3.2, consists of horizontal and vertical cables bolted into the surrounding rock, the cables are covered with burlap and the paste is subsequently placed behind the fence. Detailed construction of the fence is described by Lang (2000). Below is a summary of his construction method:

- drill off equally spaced 4 cm diameter holes on each side of the bulkhead location,
- holes to be drilled at 30° to the face,
- construct an arch with placed timber posts along bulkhead footprint,
- sling cables over bulkhead footprint, and
- place screen to retain fill, typically chain link is used as primary screen with a layer of fabrene placed over the chain link.

![Figure 3.2: Cable Fence, after Bridges (2003)](image)

The design of the fence is based on the tension in the cables induced by the paste loading. After determining the load imposed by the paste, the fence capacity is determined by the following equations described by Lang (2000, p. 5-11):

\[ T = \frac{t}{\cos \theta} \]

Eqn. 3.1
\[ T = \frac{w s^2}{8d} \]  \hspace{1cm} \text{Eqn. 3.2}

\[ d = s \tan \theta / 4 \]  \hspace{1cm} \text{Eqn. 3.3}

Where:

\begin{itemize}
  \item \( d \) = deflection of cable at center span
  \item \( t \) = component of rope tension normal to load
  \item \( T \) = maximum rope tension
  \item \( w \) = weight of load per unit of horizontal length of span
  \item \( s \) = horizontal distance between supports
  \item \( \theta \) = angle between the horizontal and a tangent to rope curve at support
\end{itemize}

Eqn. 3.1 determines the total tension within the fence, the number of cables needed for design is based on the total force exerted on the cables divided by the tensile capacity of a single cable.

Below are the observations from the readings on cable sling bulkheads:

Advantages:
\begin{itemize}
  \item easy installation,
  \item limited amount of material used in construction, and
  \item design is based on engineering practice,
\end{itemize}

Disadvantages:
\begin{itemize}
  \item containing fill behind screen difficult,
  \item cables must move a considerable distance to activate strength,
  \item free water tends to leak out of the bottom creating hazards in headings, and
  \item construction must be followed closely to ensure cables will behave as designed during loading
\end{itemize}

3.3 Bulkhead Failures

Yumlu and Guresci (2007), Revell and Sainsbury (2007a; 2007b) and Grice (1998) discuss bulkhead failures. Due to the catastrophic consequences of bulkhead failure, the majority of the published papers do not include mine locations, but the description of the
failures is comprehensive. Further to the published failures, a fill fence failure at the Red Lake Mine is investigated.

Yumlu and Guresci (2007) discuss failure of bulkheads at Inmet’s Cayeli Mine in Turkey. Cayeli is a fully mechanized, longhole open and blind mine post backfilling with an average paste cement content of 7%, with design strength of 1.0 MPa. Stope dimensions are typically between 7 and 10 m wide, between 15 and 30 m long and between 15 m and 20 m high (Yumlu and Guresci, 2007, p. 1). The mine uses a planar, shotcrete reinforced barricade similar to those employed at Red Lake.

Cayeli Mine has recorded five barricade failures between 2003 and 2004 (Yumlu and Guresci, 2007). All failures occurred in blind stopes, where access is limited to a single manway entry. Two of the five were associated with wall failure in the stope and as such were considered rock mechanics issues and not failures due the paste filling process. Of the other failures, the first is discussed in detail below, with general comments regarding the other two following.

The first fence failure at Cayeli mine occurred in December of 2003, with a violent failure that resulted in severe damage to equipment and mine infrastructure; no injuries were reported. The failed reinforced shotcrete barricade was constructed as per mine standards, and paste placement commenced with the pouring of the 7% cement paste plug for the blind stope at a standard fill rate of 0.43 m/hr in height. After two days of curing, the paste pour commenced with the filling of the remainder of the stope at a rate of 0.62 m/hr in height (Yumlu and Guresci, 2007, p. 2). Failure occurred during the final stages or tight filling portion of the stope.

It was found that failure occurred due to overfilling of the stope, as backpressure in the pipeline was observed at the operation. It was also noted that since no breather hole was used in the pour that there was no place for the excess paste and air pressure to escape and thus large barricade pressures started to accumulate, ultimately leading to failure. (Yumlu and Guresci, 2007)

The other two failures, although with geometric and construction differences, both occurred due to overfilling of the stopes. From the failures, Yumlu and Guresci (2007) observed the following problems were common:

- fast filling rate,
continuous filling or poor plug curing times,
overfilling during final tight fill of stope,
lack of adequate breather holes did not let excess paste flow out of stope,
lack of fill management and fill monitoring controls, and
inadequate fill barricade design.

As a result, the following recommendations were made (Yumlu and Guresci, 2007):

- Reduce fill rate to 0.35 m/hr,
- Fill the paste plug 2 m above the brow (7 m total in height),
- Increase plug curing time from 2 days to 7, and
- Continuously pour the remainder of the stope above the plug omitting a third stage of tight filling.

Revell and Sainsbury (2007a) discuss five failures of paste bulkheads in Australian mines. Due to the sensitive nature of the failures, no companies or specific sites were discussed. Two mine operations are discussed: Mine A and Mine B.

Mine A is a pillarless long-hole mining operation with stope dimensions 30-60 m high and 20 m square in plan view. The bulkheads are fibercrete, 5 m wide by 5 m high, and between 250 and 350 mm thick with no structural rebar, no shear pins and no breather holes. The bulkheads are designed to withstand 200 kPa of load. Paste plugs are poured to a height of 7 m and cured so that the influence of the second pour on the barricade is reduced (Revell and Sainsbury, 2007a).

The first failure at Mine A was caused by wall sloughing of the sidewalls into the uncured paste plug resulting in the failure of the fence and the flow of paste into the mine workings. The fill fences were designed for static load and not for dynamic load imposed by the wall sloughing. Since the mine knew of the potential of wall slough in the stope, it was decided that the stope should be filled as quickly as possible to reduce any interruption of the mine cycle due to groundfall. Unfortunately, the increase in pour rate, left a large amount of uncured paste in the stope, which, when the walls sloughed, caused a catastrophic failure. Further study of this failure would have been useful as it could determine the dynamic loading capabilities of the
fence, however the authors determined that the failure was rock mechanic related and did not investigate further.

After the failure, the following steps were taken to reduce the possibility of another failure (Revell and Sainsbury, 2007a):

- Create a clearly signed non-entry zones around the paste pour and place a waste pile a nominal distance away from the stope to reduce the energy of a ‘paste wave’,
- Arch the fence, and
- Identify the risk of wall sloughing before filling.

The second and third failures at Mine A were also due to wall slough. The difference with failures two and three were that the fill fences were wider than standard design (8m). It should be noted that no personnel were injured or equipment damaged due to the ‘no-man’ zone surrounding the fill barricade. In addition, a recommendation was made when fill fences are large in span or near a cross-cut, shear pins should be drilled into the back and floor.

Mine B is an uphole, open stope mine with the ore 0.5 m thick on an angle of 60°. The paste cycle in the mine is the critical path of the operation and as such, paste is exposed or mined adjacent to within two days of curing. The paste barricades are an arched, non-structurally reinforced sprayed Aquacrete bulkhead, nominally 4 m wide and 4 m high, with thickness between 150 and 400 mm. Aquacrete is a gypsum product that is applied in a similar manner to fibercrete; it is allowed 48 hours to set before placement of paste commences. The design strength of the bulkheads is 246 kPa. (Revell and Sainsbury, 2007a, p. 7)

The first failure occurred when the paste plug was estimated between 6.5 m and 7 m above the floor. The following caused failure (Revell and Sainsbury, 2007a, p. 7):

- design thickness was 300 mm but was found to be on average 200 mm thick with the lower part of the barricade 150 mm thick,
- inflow was observed in the stope, and
- design was based on Aquacrete strength of 10 MPa, testing revealed material strength varied between 5 and 11 MPa.
After this first failure, rigorous testing of the Aquacrete was performed, elimination of water in the stopes was carried out before filling and systems were installed to ensure a fence of adequate thickness was constructed.

The second failure at Mine B occurred during the plug pour when the paste was at a height of 6m above the floor. The following caused failure (Revell and Sainsbury, 2007a, p. 9):

- paste plug failed to reach the top of the bulkhead, exposing the bulkhead to higher than design loads upon pouring of remainder of paste,
- paste plug failed to reach design height due to paste being held up on higher levels, and
- Aquacrete was designed to be 400 mm thick, however was found to be 300mm thick in failed barricades.

Following the failure, paste heights were monitored throughout the paste pours using instrumentation. In addition, supervisors were required to sign off on the fill heights before the placement of the next lift of paste. A final recommendation was made to replace Aquacrete with shotcrete due to the uncertainty of the Aquacrete strength (Revell and Sainsbury, 2007a).

Grice (1998) discusses in detail two bulkhead failures in Australian mining operations. Both barricades were brick barricades designed to retain hydraulic fill. The failures occurred during the placement of backfill and both were piping failures due to the ponding of water above the backfill. Piping type failures like this are uncommon in paste operations due to the absence of free water. It was found that raising the solids content to over 70% could eliminate the possibility of piping failure.

During a site visit by the author, a paste barricade failure occurred at Red Lake on January 19, 2007. This, to the author’s knowledge, was the first known paste barricade failure to occur at Red Lake. Failure occurred during the placement of paste in a blind stope, Figure 3.3 and Figure 3.4 are after failure photos taken on January 20, 2007. From inspection of the failure, the paste in the stope had reached the top of the fence before failure. Paste was observed to have flowed approximately 50 m away from the barricade. Further inspection showed that the side and floor were connected to the rock with shear pins. Operators reported hearing loud ‘hissing’ noises prior to failure. It was observed by the author that the overlap of the rebar for the horizontal bars tended be located at the same point in the fence causing an inherent
weakness in the fence. It is believed, upon discussion with mine personnel, that the failure was most likely caused by inadequate ‘breather’ pipe in the stope that caused bulkhead pressures to increase and the fence failed along the inherent weakness of the fence.

Figure 3.3: 34 Stope Fill Fence Failure

Figure 3.4: 34 Stope Fill Fence Failure
3.4 Numerical Modeling and Instrumentation

Numerical modeling and instrumentation of barricades has been studied by, amongst others, Yumlu and Guresci (2007), Mitchell, Smith and Libby (1975), Dehn et al. (2007), Grice (1989), Revell and Sainsbury (2007b) and by AMEC (2003).

Apart from Revell and Sainsbury (2007b), the available papers deal with the field scale testing of fill barricades for various types of backfill. Mitchell and Smith (1975) and Grice (1989) were discussed in previous sections, and below is a summary of their testing:

Mitchell and Smith (1975) determined, with rubber sandwich pressure cells, that a cement bulkhead measured a maximum lateral earth pressure of 100 kPa. Greater loads were not experienced due to arching of backfill.

Grice (1989) presented the findings of three brick bulkhead tests, two of which failed due to leakage and did not sustain maximum pressure before failure. The fence that reached maximum loads failed under a pressure of 746 kPa. Instrumentation used for the test consisted of pressure transducers and a data acquisition system; further information was not disclosed.

The work by Dehn et al. (2007) is the basis of this research, and as such will be discussed in later chapters. In summary, the work consists of the measuring of real-time stress and strain of instrumented barricades at Red Lake Mine.

Yumlu and Guresci (2007) discuss the instrumentation program for fill fences Mine at Cayeli Underground Mine in Northern Turkey. The instrumentation program was initiated as part of the response to three barricade failures discussed earlier.

The program consisted of three instrumented fill fences and was constructed in a similar manner to the shotcrete fences at Red Lake Mine. The instrumentation for each fence consisted of earth pressure cells to measure pressures within paste, contact pressure cells to measure loads on barricades and piezometers set up to measure pore pressures within the paste.

The results of the paste pour lateral pressures are presented in Figure 3.5. Their findings show a maximum lateral earth pressure on the barricade was approximately 100 kPa. The instrumentation also showed that at the commencement of the paste placement, paste loading against the fence was similar to self-weight, or hydrostatic. Pressures were reduced over time due to pore water pressure dissipation and curing of the paste.
Based on the instrumentation program the following recommendations were made (Yumlu and Guresci, 2007, p. 9):

- Reduce fill placement rate from 0.43 m/hr to 0.35 m/hr, and
- Fill and cure the paste plug to a height of 7 m (2 m above fill fence).

The instrumented fill fences provide a site specific understanding of the loading mechanisms of backfill on barricades. Revell and Sainsbury (2007b) modeled a paste barricade with the finite element model, FLAC3D, which allows understanding of paste loading in a computer environment and allows for multi-site interpretation.

The barricades were modeled as a shotcrete barricade with no rebar supporting elements. The model was designed to simulate two separate fill barricades: a 5 m by 5 m square geometry and a 5 m by 5 m horseshoe shaped model. The constitutive model used in the analysis was a strain softening Mohr-Coulomb failure envelope with tension and cohesion reducing from 100% at zero plastic strain to 0% at critical plastic strain.

The model was calibrated to bulkhead theories currently used in design, either the yield line solution or those put forward by Beer (1986). It should be noted that the shape of the fence determines which of the two theories is more applicable. (Revell and Sainsbury, 2007b)

The model was tested against a bulkhead failure at an unnamed mine. From back analysis, it was assumed that paste loaded the fence in a hydrostatic manner, and that with a
paste fill height of reportedly 7m, the imposed load on the barricade would be approximately 132 kPa (Revell and Sainsbury, 2007b, p. 9). From the model, it was found that failure occurred at 130 kPa within the model and failure mechanisms were visually similar to those of the mine failure (Revell and Sainsbury, 2007b, p. 9). In comparison the studies at Red Lake found that during the initial stages of loading, paste does not act in a hydrostatic manner. As such, this numerical model would not be suitable for the fill fences at Red Lake.

Revell and Sainsbury (2007b) conclude that the numerical model provides a useful tool in the design of bulkheads but should be used in conjunction with field testing, back analysis and engineering judgment. Numerical modeling should not be used as a stand alone design tool. This is aligned with general engineering practice in which modeling is used in conjunction with observations, empirical approached and laboratory studies.

AMEC engineering was hired by Red Lake to carry out a numerical model of the shotcrete barricade employed at the mine. The report (AMEC, 2003) discusses the results of the modeling of four different fence geometries using SAP2000plus, a widely used structural engineering software (Computer & Structures Inc., 2000):

- Flat fence, 7.5 cm thick
- Arched fence, 7.5 cm thick
- Flat fence, 10 cm thick
- Arched fence, 10 cm thick

It was found from the model that the 10 cm thick, flat fence currently used at Red Lake, given the assumed lateral loading conditions, is suitable for loading for spans up to 4.5 m; a 10 cm arched fence is necessary for larger spans. This assumption of lateral loading is critical throughout the design and recommendations were based on this assumed loading. No quantification of the lateral loading was mentioned in the report. It was found from the model that the maximum allowable pressure for the flat fence was 3.35 kPa, where for the arched fence the maximum allowable pressure was 5.7 kPa.

### 3.5 Loading Mechanism of Paste

The relationship between the horizontal loads and the vertical loads during paste pouring is essential to understand the fill fence and paste interactions. Knowing the horizontal loads for
a given paste height will give engineers the ability to better refine the design of fill fences. The
theory of lateral earth pressures is described in Rankine Theory (Terzaghi et al., 1996), the
relating of Rankine Theory to backfill has been proposed by Mitchell et al. (1981), Revell and
Sainsbury (2007a), Marcinyszyn, Pakalnis, Dunbar and Vongpasial (1997) and Dehn et al.,
Pakalnis, Corey (2007).

Rankine Theory relates the horizontal pressure to the vertical pressure based on a
coefficient which determines whether the soil is active, passive or at rest (Terzaghi et al., 1996).
The soil is considered homogenous, well drained, and cohesionless and importantly has a degree
of internal friction. In Rankine Theory, a soil is considered active when it is applying a force to
an external structure, as is the case with paste loading a fill fence. Figure 3.6 shows the pressure
distribution of the paste against a fill fence using Rankine Theory.

The Rankine Theory provides the following formula to estimate the active lateral earth
Pressure:

\[ \sigma_h = K_a \sigma_v \]  \hspace{1cm} \text{Eqn. 3.4}

Where:
\[ \sigma_v = \gamma H \]
\[ K_a = \frac{(1 - \sin \Phi)}{(1 + \sin \Phi)} \]  \hspace{1cm} \text{Eqn. 3.5}

\( \Phi \) = internal angle of friction
\( \gamma \) = unit weight of soil
\( \gamma = 18.63 \text{ kN/m}^3 \) (Dehn et al., 2007)
\( H \) = height of soil (m)
Mitchell et al.'s (1981) research find the value of Ka was determined to be unity, where the horizontal earth pressures are equivalent to the vertical pressures. This was further supported by Revell and Sainsbury (2007a) where, for back calculating their bulkhead failures, it was assumed that the loading is hydraulic, implying Ka would equal one. Marcinyszyn et al. (1999) shows that backfill behaves either in passive or at rest mode after placement and consolidation, as such the values of the coefficient of earth pressure are greater than one. This implies that the horizontal loads would be equal to or greater than the vertical pressures.

Dehn et al. (2007) studies at Red Lake Mine provide values of coefficient for active earth pressures during paste backfill. Values of Ka are reported as 0.63 for five percent cement and 0.51 for ten percent cement. These results would suggest that the paste has some internal friction and a Ka value of one is not accurate. Further evidence of the internal friction is provided in Figure 3.7, where the material is free standing. This would indicate that using Ka values of one or assuming hydrostatic loading conditions is unfounded. It should be noted that this picture is of a low slump paste.
Understanding the loading mechanism of paste provides a measure of maximum pressures exerted on the fences during paste pours. This assists the design of fill fences as the applied loads are known and can be related to the strengths of the barricade.
Nine paste barricades were instrumented to determine the loads and stresses applied to the fence during loading. Eight of the fences were instrumented to record stress and strains during planned paste pours, the other fence was instrumented to record the stress and strains during a controlled destructive fill fence test. The destructive test was required to understand the failure mechanism of the fence and to provide interpretation of paste instrumentation results with respect to fence capacity. Table 4.1 summarizes the instrumented paste barricades.

The instrumentation used for the monitoring was a selection of vibrating wire instruments that were logged during the placement of the paste. The instruments used in the fence were earth pressure cells, load cells, shotcrete embedded strain gauges, rebar strain gauges and tilt meters. The mechanics of the instrumentations are discussed with remarks concerning their performance during testing.
Table 4.1: Fill Fence Inventory

<table>
<thead>
<tr>
<th>Fence I.D.</th>
<th>Date of Testing</th>
<th>Earth Pressure Cells</th>
<th>Strain Gauges</th>
<th>Measured Fill Height</th>
<th>Tested By</th>
</tr>
</thead>
<tbody>
<tr>
<td>32-826-8</td>
<td>August 7-September 11, 2003</td>
<td>5 (4 horizontal &amp; 1 Vertical)</td>
<td>None</td>
<td>No</td>
<td>Josh Clelland</td>
</tr>
<tr>
<td>37-746-2</td>
<td>November 28-December 4, 2003</td>
<td>1</td>
<td>3</td>
<td>No</td>
<td>Josh Clelland</td>
</tr>
<tr>
<td>34-806-4st</td>
<td>December 20-24, 2003</td>
<td>2 (vertical and horizontal)</td>
<td>1</td>
<td>No</td>
<td>Josh Clelland</td>
</tr>
<tr>
<td>31-806-3</td>
<td>July 9-13, 2004</td>
<td>1</td>
<td>3</td>
<td>Yes</td>
<td>Ali Rana</td>
</tr>
<tr>
<td>36-746-1</td>
<td>August 2-12, 2004</td>
<td>2</td>
<td>3</td>
<td>Yes</td>
<td>Ali Rana</td>
</tr>
<tr>
<td>34-806-1e</td>
<td>August 24-29, 2004</td>
<td>1</td>
<td>3</td>
<td>Yes</td>
<td>Ali Rana</td>
</tr>
<tr>
<td>34-786-14a</td>
<td>July 6-7, 2005</td>
<td>2</td>
<td>3 rebar 3 concrete</td>
<td>Yes</td>
<td>Kathryn Clapp</td>
</tr>
<tr>
<td>34-786-14b</td>
<td>July 21, 2005</td>
<td>2</td>
<td>3 rebar 3 concrete</td>
<td>No</td>
<td>Kathryn Clapp</td>
</tr>
<tr>
<td>Destructive Test</td>
<td>23-Jan-07</td>
<td>2</td>
<td>3 rebar 3 concrete</td>
<td>N/A</td>
<td>Paul Hughes</td>
</tr>
</tbody>
</table>

4.1 Instrumentation

The purpose of the instrumentations was to quantify the stresses and the strains imposed on the fence during loading. Monitoring consisted of readings taken during past pours during normal mine operations. Vibrating wire instruments were attached to the fence at critical points on the fence to maximize the recorded data. In total five types of instruments were used during the monitoring: earth pressure cells, embedded concrete strain gauges, rebar strain gauges, load cells and tiltmeters.

Red Lake mine engineering staff decided upon instruments initial locations. The locations were attempting to find differences in lateral pressures with height and the difference in strains at various points on the barricade. As the testing program continued minor
modifications were made to the instrument locations, however large modifications were not made to allow for comparison between different tests.

4.1.1 Geokon 4800 Vibrating Wire Earth Pressure Cells

Pressure against the fill fence and pressure due to the self-weight of the paste were measured using vibrating wire earth pressure cells ("EPC") purchased from Geokon Inc. Figure 4.1 and Figure 4.2 show the EPC installed on the destructive test fence and for a paste pour respectively.

Figure 4.1: EPC Configuration on Loading Plate for Destructive Fill Fence Test
The EPC instrument consists of two thin stainless steel plates that are welded together at their edges and kept separate by a thin layer of hydraulic fluid. A steel pipe connects the hydraulic fluid with a pressure transducer. When pressure is applied to the steel plates, it causes a corresponding increase in the pressure. Due to the pressure change in the pipe, a change in frequency of the vibrating wires in the pressure transducers occurs. This change in frequency corresponds to a measured increase in pressure (Geokon, 2003). The equation used to convert pressure transducer readings to pressure is shown in Eqn. 4.1 (Geokon, 2003, p. 19). Note: pressure readings are in forms of 'Digits'.

\[ P_{\text{corrected}} = [(R_0 - R_I) \times G] + [(T_I - T_0) \times K] \]

Eqn 4.1

Where:

- \( R_I \): Current Pressure Reading
- \( R_0 \): Zero Strain Reading
- \( G \): Gauge Factor (MPa/Digit)
- \( T_I \): Current Temperature (°C)
- \( T_0 \): Initial Temperature (°C)
- \( K \): Thermal Factor (MPa/°C)
EPCs, as recommended by AMEC (2003), were mounted to 1 cm plywood boards with a thin layer of grout to reduce the chance of point loading within the EPC. EPC units were mounted directly to the fence to ensure that the load being measured was between paste and the wall.

Tesarik et al. (2006) found that the EPCs are sensitive to temperature changes and that the manufacturer’s compensation does not properly account for the differences. According to the study, if the proper correction of the cells is not applied, the readings not only are in error but also, in some cases, demonstrate a reverse in trend of actual behaviour in the stope. At the low pressures recorded by the instruments during the Red Lake testing program (ex. 40 kPa), these corrections tend to be comparatively large and require investigation before being used in this study.

Tesarik et al. (2006) argue that using the manufacturer’s correction factor accounts for only a small percentage of the temperature effects on EPCs. This is because the supplier’s temperature correction is compensating for the effect of temperature on the vibrating wire only (Geokon, 2003) and not on the entire system of the EPC. Tesarik reasons that both the metallic cover and oil within the EPC will be affected by temperature increase, and provides the following three methods to calculate proper stresses:

- theoretical correction, in which the modulus of the surrounding media and expansion of the liquid within the cell is taken into account,
- polynomial equations fit to temperature-induced stress versus temperature readings for EPCs with little to no load, and
- average correction factor obtained from the slope of the linear regression form from the stress versus temperature plots during non-mining activities.

In all cases where the above corrections were applied, the final corrected stress was closer to the benchmark values obtained from calculating the overburden weight of paste on a vertical EPC. These readings required prior set up of instrumentation, and are difficult to do after the fact.

For the purposes of the EPCs used at Red Lake, the only applicable correction proposed by Tesarik et al. (2006) would be the theoretical correction, as the other two rely on long term
loading interactions, i.e. months to years, that require non-mining periods or background readings of instruments prior to loading, neither of which were possible with the instrumentation program. Tesarik et al. (2006) theoretical equation was applied to fence 36-746-1’s EPC-1 (§) to determine the effectiveness of the calculation. The theoretical equation is applied to eqn. 4.1 in the following manner Tesarik et al. (2006, p. 4):

\[
P_{\text{corrected}} = [(R_0 - R_1) \times G] + [(T_1 - T_0) \times C] \quad \text{Eqn 4.2}
\]

Where:

\[
R_1 = \text{Current Pressure Reading}
\]

\[
R_0 = \text{Zero Strain Reading}
\]

\[
G = \text{Gauge Factor (MPa/Digit)}
\]

\[
T_1 = \text{Current Temperature (°C)}
\]

\[
T_0 = \text{Initial Temperature (°C)}
\]

\[
C = \text{Theoretical Temperature Correction Factor (MPa/°C)}
\]  
\hspace{1cm} = 1.5 \times E \times K \times D \times R
\]

\[
E = \text{elastic modulus of deformation of paste (MPa)}
\]  
\hspace{1cm} = 2,264.238 \times 10^6 \text{ Pa (Tesarik et al., 2006, p.4)}
\]

\[
K = \text{coeff. of thermal expansion of liquid film encased in earth pressure cell (1/°C)}
\]  
\hspace{1cm} = 700 \times 10^{-6} / ^{°}\text{C (Tesarik et al., 2006, p.4)}
\]

\[
D = \text{Thickness of liquid film in EPC (mm)}
\]  
\hspace{1cm} = 0.51 \text{ mm (Geokon, 2003, p. 7)}
\]

\[
R = \text{Radius of earth pressure cell (mm)}
\]  
\hspace{1cm} = 114.3 \text{ mm (Geokon, 2003, p. 7)}
\]
The theoretical thermal correction equation yields a correction factor (C) of $9.22 \times 10^{-3}$ as opposed to Geokon’s thermal correction factor (K) for EPC 1 of $8.14 \times 10^{-4}$. Figure 4.3 demonstrates that this correction translates to a 10 fold increase in the pressure. As will be discussed in Chapter 6, a realistic value of pressure would be approximately 25 to 50 kPa for the paste heights for this particular fence. The theoretical thermal correction is not suitable for the loads experienced in the instrumentation program and the manufacturer’s correction will be used throughout the analysis. The reason is that this correction provides lateral earth pressures that are unreasonable given the paste height.

The EPC performed best of all instruments installed in the fences. No units failed or gave erroneous readings. A limitation experienced with the EPCs was that in dynamic testing the cells did not respond rapidly to the changing loads, as was the case in the destructive fill fence test.
4.1.2 Geokon 4200 Vibrating Wire Embedment Strain Gauges

Strain within the shotcrete was measured with embedded strain gauges purchased from Geokon Inc. Figure 4.4 shows the instrument installed on a fill fence.

![Image](image_url)

Figure 4.4: Embedded Concrete Strain Gauge Installed on Destructive Fill Fence Test

The strain gauges work on the vibrating wire principle, where a length of steel wire is tensioned between the bulbous terminals of the strain gauge that are embedded within a material, in this case, shotcrete. Deformation of the material in which the instrument is anchored results in movement between the two terminals, causing a change in tension within the steel wire. The wire is then plucked with an electromagnetic charge and the resonant frequency of vibration of the wire is recorded through an electromagnetic coil. In addition, a thermostat within the instruments measures temperature fluctuations that can account for changes in strain readings.

The data recorded for the strain gauges is in units of micro strain. In the process of making the strain gauges, the method of clamping the vibrating wire causes tension in the wire, and so a correction is applied to compensate for all instruments constructed with this measure tension (Geokon, 1996). This correction is known as a batch factor. It should be noted that two sets of strain gauges were used in the testing, therefore only two batch factors were used in testing: pre 2005 and post 2005.
Large changes in temperature can have a significant effect on the strain gauge. To compensate a thermal correction is applied to the readings (Geokon, 1996). For this study the large temperature changes are most likely associated with the curing process of concrete.

One issue that arises with this temperature increase is that the coefficients of expansion for steel and concrete are different. With a temperature increase, the vibrating wire itself will elongate and thus reduce in tension for a given load, indicating a compressive strain within the embedded material. This elongation of the vibrating wire is offset by a corresponding stretching of the wire due to the expansion of concrete with a temperature increase. Once this temperature correction, shown in Eqn. 4.3, is applied to the strain readings, Geokon (1996, p. 12) defines the strain as the ‘Load Related Strain’. However, Geokon defines the strain imposed on the concrete due to temperature decrease as the ‘actual strain’. The ‘actual strain’ defines the effect of temperature on the concrete only, and not on the steel gauge, therefore for the purpose of the analysis the ‘load related strain’ will be used. The calculation to determine the load related strain at any point during the testing is the following (Geokon, 1996, p. 12)

$$\mu e = B (R_0 - R_1) + (C_1 - C_2)(T_1 - T_0) \quad \text{Eqn. 4.3}$$

Where:

- $R_1 =$ Current Strain Reading ($\mu e$)
- $R_0 =$ Zero Strain Reading ($\mu e$)
- $B =$ Batch Factor
- $C_1 =$ Coefficient of expansion of steel ($\mu e/^{\circ}C$)
  $= 12.0$ (Geokon, 1996)
- $C_2 =$ Coefficient of expansion of shotcrete ($\mu e/^{\circ}C$)
  $= 12.2$ (Dehn et al., 2007)
- $T_1 =$ Current Temperature ($^{\circ}C$)
- $T_0 =$ Initial Temperature ($^{\circ}C$)

Thermal corrections were not applied to the instrumented destructive fill fence tests, as the temperature within the test stope was constant at $27^{\circ}C$. 


One of the embedded strain gauges used in testing appears to have failed or gave erroneous readings. This instrument referred to as SG1 installed on fence 34-806-1, give inconsistent readings with large fluctuations. One limitation noted during use of the instruments was that they tended to fluctuate between -1 and +1 units when using a manual reader.

To compensate for the fluctuation in the readouts during the destructive fill fence test, the mean value of the values recorded was used. It should be noted that the automatic data logger records the instantaneous value of the instrument every two minutes, therefore no corrections for fluctuating readings could be applied. Since typical strain increases were recorded between one to three units, there exists a chance that the reading could be in error by as much as 100%.

4.1.3 Geokon 4111A Vibrating Wire Rebar Strain Gauges

Strain within the rebar skeleton of the fill fence was measured using Geokon 4111A Vibrating Wire Rebar Strain Gauges for fences 34-786-14a, 34-786-14b and the destructive fill fence. Figure 4.5 shows the rebar strain gauge attached to an instrumented fill fence. The rebar strain gauges are normally used for monitoring of strain within concrete piers and beams in civil construction, however their use in this project was applicable.
The rebar strain gauge mechanism is similar to that of the Geokon 4200 Vibrating Wire 'dumb bell' strain gauge, with the only notable difference being that it measures strain within the rebar element rather than the within shotcrete.

The rebar strain is measured in dimensionless numbers that correspond to the frequency of the plucked vibrating wire. The measurements are related to strain with the following equation (Geokon, 2004, p. 10):

\[
\mu \varepsilon_{\text{corrected}} = [(R1 - R0) \times G] + [(T1 - T0) \times (C1 - C2)] \quad \text{Eqn 4.4}
\]

Where:
- \( R1 \) = Current Strain Reading (units)
- \( R0 \) = Zero Strain Reading (units)
- \( G \) = Gauge Factor (\( \mu \varepsilon/\text{units} \))
- \( T1 \) = Current Temperature (°C)
- \( T0 \) = Initial Temperature (°C)
- \( C1 \) = Thermal coefficient of expansion of steel (\( \mu \varepsilon/°C \))
- \( C0 \) = Thermal coefficient of expansion of concrete (\( \mu \varepsilon/°C \))
An issue with the rebar strain gauges is that the direction in which it installed will determine if the strains recorded are in tension or compression. The direction needed to be recorded in order to maintain consistent readings throughout the testing program. This is essential, as the tensile strength of rebar is much less than the compressional strength and will fail at lower strains.

Another issue was that, during the monitoring of fences 34-786-14a and 34-786-14b, the rebar strain gauge was coupled, or attached, to the rebar skeleton, as shown in Figure 4.6. This tends to stiffen the rebar skeleton and so not to reflect the behaviour of the actual rebar but rather to reflect the behaviour of a stiffer rebar skeleton. This configuration was not used for the destructive fill fence test; shows the arrangement of the rebar strain gauges for this test. The strain gauge should installed as a structural element of the fence and not attached to the rebar skeleton as this reflects the construction process used for standard fences.
4.1.4 Geokon 4900 Vibrating Wire Load Cell

A load cell was used to determine the lateral load placed on the fence during the destructive fill fence test. Figure 4.7 shows the configuration of the load cell during testing.

Figure 4.7: Load Cell Configuration for Destructive Fill Fence

The load cell measures applied loads through three vibrating wire gauges spaced evenly around the annulus of the device, as shown in Figure 4.8. The three gauges can determine if
distributed load is being applied across the load cell. Values are averaged from the three gauges to determine the total load on the cell.

Figure 4.8: Schematic of Vibrating Wire Layout for Load Cell

Loads are measured as dimensionless numbers and converted to loads with the following equation. The adjustment for temperatures has been found to be 1.5 digits per Centigrade degree as determined from Geokon testing (Geokon, 2005, p. 15). For the destructive fill fence test, temperatures in the stope were constant and thus temperature correction factors were not used. Eqn. 4.5 determines the measured load applied on the load cell (Geokon, 2005, p. 15)

\[
\text{Load (tonnes)} = \left( L_1 - L_0 \right) \times G - 1.5 (T_0 - T_1)
\]

Where:

- \( L_1 \) = Current Load Reading (units)
- \( L_0 \) = Zero Load Reading (units)
- \( G \) = Gauge Factor (tonne/units)
- \( T_1 \) = Current Temperature (°C)
- \( T_0 \) = Initial Temperature (°C)

Eqn. 4.5

The load cell performed with minimal problems during the destructive fill fence test. One problem identified was the difference between the load provided by the gauge on the hydraulic jack and the load cell. The plot of the difference between the two readings over time is shown in Figure 4.9. Although readings did not differ greatly, discrepancy between the two
readings can be accounted for by the slack in the testing system that would cause jumps in the load cell. This was observed to have occurred during testing on several occasions and did not adversely affect the testing. The load cell values will be used in further analysis since it is more precise and accurate in comparison to the pressure gauge on the hydraulic jack.

![Figure 4.9: Destructive Fill Fence Test - Load Cell and Jack Pressure vs. Time](image)

**4.1.5 Geokon 6350 Vibrating Wire Tiltmeters**

Vibrating tiltmeters were used for fences 34-786-14a and 34-786-14b. Figure 4.10 shows the installation of the tiltmeters. The purpose of the tiltmeters was to measure the upper and lower deflection with respect to the vertical.
The tiltmeters mechanism of operation is based on a pendulous mass inside a steel casing. The mass inside the sensor, under the force of gravity, sways as the instrument is moved, a vibrating wire resists the mass’ motion, and this resistance in the vibrating wire corresponds to a change in the angle with respect to vertical through the following equation (Geokon, 2006):

\[
\Delta \theta = [(R1 - R0) \times G] + T
\]

*Eqn 4.6*

Where:

- R1 = Current Reading
- R0 = Initial Reading
- G = Calibration Factor (degrees/ digits)
- T = Temperature correction factor same as discussed in Chapter 4.1.1

Results of the tiltmeters are included in the following Chapter; however, the results from the tiltmeters were not useful in the analysis, as they did not show fence displacement. Electronic displacement sensors or of a total station would be more applicable to measure the
displacement during pouring of paste would be more beneficial to the analysis. As such, the use of tiltmeters illustrates that the fence underwent some deformation, yet its value is not quantifiable; displacement measurements would be more useful as they give the true deformation. Future tests should investigate the displacements of the fence to determine the movement of the fence under pressure.

4.2 Fill Fences

The following discusses the instrumentation, time-line and information gathered for each fence.

4.2.1 Fill Fence- 32-826-8

Fill Fence 32-826-8 was the first fence to be monitored during a paste pour. Josh Clelland, a Laurentian University Co-Op Student employed by Red Lake Mine, carried out the instrumentation and monitoring of the fill fence during pouring, with assistance from Red Lake Mine engineering staff. The testing of the fence occurred during the period of August 7 to September 11, 2003.

The fence was constructed in two parts for a total height of 3.5 m; the first section was 2.1 m high and contained all of the instrumentation, for production purposes another 1.4 m in height was added on to the fence. The span of the fence was not reported in any documents.

The purpose of the instrumentation was to determine the horizontal loads being applied to the fence and how they relate to the vertical loads. This was achieved by four load cells installed in a manner similar to that of Figure 4.1. In addition, one load cell was installed horizontally in order to measure the vertical pressure during the paste pour. EPCs were installed at heights of 0.25 m, 0.85 m, 1.45 m and 2.05 m above the floor as shown in Figure 4.11.
Another issue with the monitoring of this fence was that the height of the paste during paste pouring was not measured. As such, no relationship between height of paste and horizontal load can be made. From the data, the only information from this fence that can be used is the determination of horizontal loads with respect to vertical to determine a Rankine ‘K’ factor.

4.2.2 Fill Fence - 37-746-2

The second fence was monitored in 37-746-2 HW zone between November 28, 2003 and December 3, 2003 by Laurentian University co-op student Josh Clelland. The fence dimensions are 3.9 m high by 3.6 m wide, as shown in Figure 4.12. Instrumentation layout can also be seen in the figure. In total, three shotcrete embedded strain gauges and one earth pressure cell was used in the monitoring program.

The instrumentation measured over a total of 4.5 m of paste pour as noted by Mr. Clelland. With the known height of the paste, the pressures on the fence can be used to determine the properties of the paste during pouring. The strain gauge information will be useful in determining stress-strain relationships for the fence.
4.2.3 Fill Fence- 34-806-4

Fill Fence 34-806-4 was monitored during paste pour between December 20 and 24, 2003 by co-op student Josh Clelland. As shown in Figure 4.13, the fence measures 3.6 m high by 5.35 m wide. Two EPCs were installed at the fence, one vertical, one horizontal and two shotcrete embedded strain gauges were installed, one near the center of the fence, one near the perimeter of the fence.

The strain gauge data can be used to determine the differences in strain between the center and perimeter of the fence. A horizontal to vertical stress ratio can be obtained from the vertical and horizontal earth pressure cell data.
4.2.4 Fill Fence- 31-806-3

Fill Fence 31-806-3 was monitored by UBC Co-Op student Ali Rana during a paste pour between July 9 and 13, 2004. The fence dimensions, as shown in Figure 4.14, are 4.25 m wide by 4.25 m high. The three strain gauge array as well as one horizontally mounted earth pressure cell were used in the instrumentation as shown in Figure 4.14. It is intended that the earth pressure cell should measure the amount of load on the fence while the strain gauges determine the difference in loading with the concrete in the vertical vs. horizontal direction and the difference between the center of the fence and the perimeter.

Fill heights were estimated based on amount of paste used in the stope; the fill was poured in three shifts and had final heights of 1.5m, 2.7 m and 4.25m. Twelve hours of cure time occurred between pouring of successive lifts, this would provide time for the paste to cure and develop cohesive properties.

Since the fill heights were known, the use of Rankine's lateral earth pressures can be used to determine the friction angle of soil.
4.2.5 Fill Fence- 36-746-1

Fill fence 36-746-1 was instrumented by Co-op student Ali Rana for a paste pour that took place between August 2 and 12, 2004. The fence tested was the largest of the eight instrumented paste pour fences, measuring 5.35 m high by 12.25 m wide, total dimensions and instrumentation locations are shown in Figure 4.15.

Two earth pressure cells and three strain gauges were used. A vertical strain gauge was place directly in front of an earth pressure cell in an effort to determine a stress strain relationship for the fence during paste loading.

Figure 4.14: 31-806-3- Fence Dimensions and Instrument Layout
Paste heights were estimated based on the volume of batches processed at the paste plant and the dimensions of the stope. Heights were estimated after every batch poured.

4.2.6 Fill Fence 34-806-1

Fill fence 34-806-1 was monitored by Ali Rana during paste placement of paste between August 24 and 29, 2004. The fence dimensions are 6.9 m wide by 3.7 m high, with three strain gauges and one EPC, as shown in Figure 4.16.

Paste heights were measured in the same manner as 31-806-3 and 36-746-1, with reading estimated after each paste pour.
4.2.7 Fill Fence- 34-786-14a

Fill fence 34-786-14a was instrumented by UBC PhD student Kathryn Dehn as part of a larger paste fill research program. Dehn et al. (2007) discusses the instrumentation program of both this fence and 34-786-14b. It should be noted that the fill fence research program became more focused with the following being key to the research:

- determining the lateral loads at the fence/paste contact,
- determining the amount of strain in rebar and shotcrete, and
- recording deformation of fence to correlate with strain.

The key difference between these and previously tested fences is the investigation of strain within both the shotcrete and the rebar and analyzing them separately.

Fill Fence 34-786-14a was poured between July 7 and 7, 2005 with two lifts of paste reaching the height of 1.2m on the first and 2.3 for the second, heights were measured through infrared cameras. The fence dimension was approximately 3.4 m in height by 4.8 m wide. Figure 4.17 shows the approximate locations of the instruments.
4.8m

Figure 4.17: 34-786-14a Instrument Layout

4.2.8 Fill Fence- 34-786-14b

Fill fence 34-786-14b was monitored on July 21, 2005 by Kathryn Dehn. The fence dimensions were 3.85 m in height by 3.5 m in width and it was instrumented in a similar fashion as fill fence 34-786-14a. Figure 4.18 shows the approximate instrument locations.

The filling of the paste for this fence was expedited by the fact that waste rock was placed in the stope to reduce the volume of paste required (Dehn et al., 2007). No mention is made of the amount or the location of the waste rock with respect to fill fence. The paste was placed to a height of 1.5 m with 10% cement. The remaining height of the stope was filled with 5% cement.
4.2.9 Fill Fence- Destructive Test

This fill fence is the only fence tested by the author of the study. The purpose of the destructive fill fence test was to determine the breaking strength of the Red Lake fill fence and to quantify the behaviour of the paste fence with respect to failure. Fill fence construction timelines are a function of production and therefore the program in place required a generic, instrumented fence implemented by ground control personnel at Red Lake Mine.

The fill fence test occurred on January 21, 2007 and construction and placement of instruments for the fence took place between January 9 and January 20, 2007. Contractors undertook the fence construction with assistance from Paul Hughes and Ray Wilkins of Red Lake Mine ground control department. The rebar portion of the fence was constructed, instruments installed and fence dimensions measured during one shift. The fence was built to standard specifications as outlined by Goldcorp (2002) with the exception that the rebar was not
anchored into the floor. This was done so that the fence would represent the worst-case scenario of fence loading. The fence dimensions were nominally 8 m wide by 2.6 m high.

The shotcrete for the fence was the limiting factor in performing this test in a timely manner. The shotcrete had to be scheduled around production and was delayed by a week. During the shotcrete application, a safety bay constructed of shotcrete was also created so that should catastrophic failure occur, operators would be safe. Once the fence was shotcreted, the installation of steel I-beams was carried out to creating a testing platform. This was installed during a single shift.

The following testing apparatus needed to be constructed for the test, drawings of these are shown in Figure 4.19, Figure 4.20 and Figure 4.21:

- 1 m x 1m loading plate,
- Cable system to keep loading plate in position, and
- 25 mm rebar bar, 2.5 m long.

The loading plate was built on surface, based on drawings provided to the contractors by Red Lake Mine engineering department. The plate was constructed as designed and performed as intended. The cable system was designed to prevent the loading plate from applying load to the fence prior to loading. Two eyelets were drilled into the back and a cable winch was rigged to support the plate. This allowed the plate to move freely prior to loading. A 25 mm steel bar and various plates were rigged up to the I-beams to allow the hydraulic ram to push against the bar placing load on the fence. These plates and bars were supplied and constructed by contractors.
The fill fence was monitored with instruments during the testing to determine the stress strain response of the fence during applied load. The instruments consisted of strain gauges,
earth pressure cells and a load cell as described in Chapter 4.1. The layout of the instruments is shown photographed in Figure 4.21 and schematically drawn in Figure 4.22.

Figure 4.21: Photograph of Instrument Layout for Destructive Fill Fence Test
A data logger was problematic throughout the test program. A fully charged data logger was brought underground; this failed to log the instruments correctly. The unit was brought up to surface, recharged and was tested on surface and successfully logged five instruments. The unit was brought underground and failed to log a cycle of instruments.

It was then decided that instruments would be read using Geokon Digital Readout Units GK-401 and GK-403. Using these units lengthened the testing time, as each instrument had to be read manually.

Loads were applied to the fence and time was taken to allow the fence to equilibrate the slack in the system. Once the equilibrium was reached, instruments were read and the load was further increased. Test loads of 0.5, 1, 1.5, 2, 3, 4, 5, 7, 9, 11 and 15 tonnes with respect to the hydraulic gauge were applied to the fence. After the 15 tonne load, a 17 tonnes load was placed on the fence to see if failure could be initiated, no instrument reading were taken. Failure did not occur during testing.

Figure 4.22: Destructive Fill Fence Instrument Layout

A data logger was problematic throughout the test program. A fully charged data logger was brought underground; this failed to log the instruments correctly. The unit was brought up to surface, recharged and was tested on surface and successfully logged five instruments. The unit was brought underground and failed to log a cycle of instruments.

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The test did not demonstrate any noticeable cracks during testing. At the 11 tonne mark, a superficial piece of shotcrete 4 x 4 x 1cm broke away from the fence and travelled 2 m across the drift. Noticeable deformation of the fence was evident but no signs of distress were obvious. An outline of the loading plate was apparent due to the deformation around the area, but no other signs of distress were visible. It was estimated that the fence had undergone 7cm of horizontal deformation based on hydraulic jack piston displacement, however no measurement were taken to verify these estimates since no personnel were allowed in front of the fence.

After the testing on January 21, 2006, clean up of the test site commenced the next day. In an effort to fail the fence Ray Wilkins loaded the fence from zero load to approximately 15 tonnes and the rebar skeleton was pulled out from the shotcrete fence, ultimately failing the fence. Caution should be applied in considering this the ultimate failure load, as the fence had been loaded to 17 tonnes and ultimate failure did not occur.
5 RESULTS

The following Chapter describes the results of the instrumented fill fence and explains the trends and values reported. The Chapter presents the recorded values vs. time for all fences. This chapter presents all the data that will be subsequently used in analysis to determine the loading mechanism of the paste and the ultimate capacity of the fence.

Results presented for Fill Fences 32-826-8, 37-746-2, 34-806-4, 31-806-3, 36-746-1 and 34-806-1 data were recorded with a Campbell Scientific Data Logger. The Data Logger is a 16-channel system that reads instruments on a pre-set interval. For the purposes of the fill fence monitoring the interval was set for two minutes.

Due to the large amount of data recorded for analysis purpose, a filter was applied to the data so that a reading every thirty minutes was reported. This made no difference to trends as compared to the use of all data readings. The only advantage with using more data is that spikes occur during testing. However, the spikes appear random and not influential to the analysis. These spikes may be a result of background noise caused by mechanical/electrical influences such as pumps, blasting or communications, among other mining activities.

After the data logger was used for the above mentioned fences, it ceased to work as a battery problem was apparent. The remaining three fences' recordings were taken manually with a Geokon 403 readout unit. The manual readings did not provide the near continuous readings and were subject to operator error in comparison to the data logger.

The following is a chronological presentation of the recorded fill fence data.

5.1 Fill Fence- 32-826-8

Figure 5.1 show the data collected for five earth pressure cells in Fill Fence 32-826-8 over the course of 35 days. The EPCs show two upward trends, one commencing at the start on August 8, 2003 and peaking between August 10 and 12th, the other starting at August 27 and finishing around September 4.
Figure 5.1: Fence 32-826-8- Pressure vs. Time

Records passed on from Josh Clelland show that the first rise between August 8 and the 17th is due to the pouring of an estimated 4.4 m of paste. The second rise in the EPCs 0.85m and 0.25 above the floor from August 27 is not explained in the records and is not reflected in the other EPCs. The EPC 1.45 m above fluctuates between positive and negative values, providing no useful data, most likely due to the relatively low horizontal loads.

The temperatures for the EPCs, shown in Figure 5.2, increase as the curing starts and decrease as the curing process ceases. The temperature readings in the EPCs 0.55 m and 0.25 m above the floor show an upward trend around August 27 and may explain the uptrend in the load reading at this time seen in Figure 5.1. It should be noted that considering the proximity of the vertical EPC to that of EPC 0.25m, the change in the temperature readings and stress readings around this time are abnormal.
### 5.2 Fill Fence- 37-746-2

The details for fill fence 37-746-2 are mentioned in Chapter 4.2.2. This fill fence was the second to be instrumented and the details for the test are on record, however the results of the test are not.

No discussion, presentation or analysis of the data was included with the information provided to UBC on the fence. From this, it can only be deduced that the data for this fence was not downloaded, or was erased or misplaced.

It should be noted that no heights were properly recorded during the placement of the paste and very little analysis would have been possible for this fence.

### 5.3 Fill Fence- 34-806-4

Figure 5.3 shows the EPC pressure and temperature vs. time for Fill Fence 34-806-4. The vertical EPC shows a large spike around December 21, this would correspond to a paste
pour and line flush with water. The EPC shows that the paste continues to cure until a large spike at December 22 most likely due to a final water flush of the line.

The large drop in pressure for the vertical EPC on December 22 is very abnormal and does not follow any known trend of curing of concrete. The recording of the horizontal instrument ceased on December 22. There is no mention in the records of why this occurred. If it was a case of damage to the instrument it could explain the behaviour of the vertical EPC as well, however this is only conjecture.

![Graph](image_url)

**Figure 5.3: Fill Fence 34-806-4 Pressure vs. Time**

Fill fence 34-806-4 was the first fence to report the data for strain during the pouring of paste. From Figure 5.4, it can be seen that during the initial placement of paste, the fence undergoes very little strain. The strain increases around December 21 to a maximum of 2000 με before decreasing. The strain gauge on the side of the fence ceases to record at the same moment as the horizontal EPC. This is most likely due to a failure of the data logger. The
middle strain gauge performs properly throughout the test and apart from a spike around December 22, read between 500-600 με after placement of paste.

Figure 5.4: Fill Fence 34-806-4 Shotcrete Strain vs. Time

5.4 Fill Fence- 31-806-3

Instrumentation for Fill Fence 31-806-3 consisted of one EPC and three strain gauges. The data was recorded properly and all instruments performed as intended.

Figure 5.5 shows the plot of the EPC. There exists three rises in data that corresponds to the placement of three paste pours. The first paste pour consist of 75 batches that commenced on July 9 and raised the height of fill to approximately 1.5 m in the stope. The decrease in the loads is most likely due to the curing of the paste. The July 10 night shift poured 55 batches of paste in the stope raising the paste height to approximately 2.7 m. The final paste pour commenced on the day shift of July 11 and consisted of 70 batches of paste, raising the total height of the paste to approximately 4.2 m. From the graph, it can be seen that the paste began to cure during pours 1 and 2. The sharp rises near the end of the first and second pour are most likely due to the flushing of the paste line with water, although there is no record of this
occurring. The maximum pressure recorded by the EPC is a value of 0.019 MPa during the final pour on July 12.

![Figure 5.5 Fill Fence 31-806-3 Pressure vs. Time](image)

Figure 5.5 Fill Fence 31-806-3 Pressure vs. Time

The strain gauges for Fence 31-806-3 (Figure 5.6) correlate well with the applied load on the fence (Figure 5.5). The strain commences with the placement of the first pour. SG 1 shows a steady positive increase with the placement of the paste and after the placement stops, the strain reduces, most likely due to the development of shear strength within the curing paste.

SG 2 and 3 show an initial negative trend in the strain indicating that the elements of the fence are undergoing tensile loading. However, during the second and third paste pours, the strain in these units begins to show a positive increase similar to that found in SG1. SG1 and SG2 show similar trends but SG1 has larger gains and commences at a value of strain nearer to zero. This is most likely due to the location of the instrument with respect to the center of the fence.
The initial negative decrease in all SGs prior to the placement of the paste could be due to the curing of the shotcrete reducing the strain in the material. However, this is speculation and needs further investigation. The maximum strain measured is 124 με recorded by SG1.

Figure 5.6: Fill Fence 31-806-3 Strain vs. Time

5.5 Fill Fence- 36-746-1

Instrumentation for Fill fence 36746-1 consisted of two EPCs and three strain gauges embedded in the concrete. EPC 1 commenced recording on the night shift of August 2, 2004 with the pouring of the 32 batches reaching a height of 0.9 m. EPC began readings on August 7 as the height of the fill reached the instrument’s height of 2.5 m.

The filling sequence for the fence consists of five pours that reached estimated heights of 0.9 m, 1.7 m, 2.5 m, 3.4 m and 3.7 m.

Figure 5.7 shows a spike around August 5 for EPC1 that is associated with a water flush at the end of paste pour #1. Similar spikes are seen around August 10 on EPC 1 and 2, inspection of the records conclude that these are associated with water flushes at the end of the fourth pour.
The maximum pressures recorded by the EPCs are 0.0250 MPa and 0.018 MPa for EPC 1 and 2 respectively.

Figure 5.7: Fill Fence 36-746-1 Pressure vs. Time

Figure 5.8 shows an initial negative reading for SG 1 indicating tensile strain in the shotcrete before a generally positive increase in the strain readings with a maximum peak reading of 75 με. This peak reading is seen in all three strain gauges and, through inspection of the records, is most likely associated with the flushing of the paste line prior to the placement of the third batch of paste. Figure 5.8 also shows a negative trend for SG 2 and 3, reaching a maximum negative value of -220 με and -180 με respectively, before a positive trend commences around August 6th. The difference in strain from the lowest value to the highest for SG 2 and SG 3 is 50 με and 100 με respectively, similar to the range of strains that exist for SG 1.

SG 1 and SG 2 cease to work on August 7. This is most likely due to the commencement of measurement of EPC 2 that requires two data logger channels (pressure and temperature). However since there is no record of the reason for the cessation of readings, this is only
speculative. Without these readings, it is difficult to assess the behaviour of the fence throughout the testing.

Figure 5.8: Fill Fence 36-746-1 Shotcrete Strain vs. Time

5.6 Fill Fence- 34-806-1

Fence 34-806-1 consisted of six pours over the length of 5 days, reaching a total height of 4.25 m. Instrumentation for the fence consisted of one EPC and three strain gauges that performed as designed during the pouring of the paste.

Figure 5.9 shows the plot of the stress measured by the EPC vs. time on the primary axis and fill height vs. time on the secondary axis. The trend of the readings from the earth pressure cell follows closely the rise of paste within the stope. The only area where the trends of the stress and fill height do not agree is late on August 25th, after the placement of the first lift. As has been the case in prior fills, this is most likely associated with the flushing of the paste lines after the pour.

The EPC reaches a maximum value of 0.04 MPa at the end of the placement of paste on August 29, 2004.
Figure 5.9: Fill Fence 34-806-1 Pressure and Fill Height vs. Time

The behaviour of the three strain gauges is plotted in Figure 5.10. SG 1 shows dramatic fluctuations in readings during the length of the paste placement. As mentioned in Chapter 4, this strain gauge malfunctioned during the testing.

SG 2 and SG 3 follow a similar negative trend until August 28, 2004 when SG2 begins a positive increase to a final value of zero. The maximum value recorded by the strain gauge was measured on SG 1 on August 28, 2004 when the gauge measure a strain value of -375με.
Figure 5.10: Fill Fence 34-806-1 Shotcrete Strain vs. Time

5.7 Fill Fence- 34-786-14a

Instrumentation for Fence 34-786-14a consisted of two EPCs, three rebar strain gauges, three shotcrete strain gauges and four tiltmeters. All instruments performed as designed during the three measured lifts of paste that reached a height of 1.7 m.

Figure 5.11 shows the measured EPC pressure vs. time with the paste height indicated by labeled vertical lines. As shown, the measured pressure increase corresponds well with the height of paste. ‘EPC @ 1.8 m’ commences reading pressures at 13:39 and most likely corresponds with the fill being at the same height as the instrument. As expected the lower EPC, ‘EPC @ 0.6 m’ measured the highest pressure near the end of the pour with a maximum value of 0.013 MPa.
Figure 5.11: Fill Fence 34-786-1-14a Pressure vs. Time

Figure 5.12 and Figure 5.13 show the measured strain in the rebar and shotcrete respectively during the placement of paste in the stope. As can be seen, the rebar strain gauges “Center Horizontal @ 0.6 m” and “Center Vertical @ 1.2 m” show an increase in strain with the height of paste in the stope. “Center Horizontal @ 1.8m” does not show any increase in strain during the placement of the paste and it is most likely that strains did not develop in the upper parts of the fence since the paste did not reach the height of the instrument. The maximum measured strain was in “Center Vertical @ 1.2 m” which had a measured strain value of approximately 25 με.
Figure 5.12: Fill Fence 34-786-1-14a Rebar Strain vs. Time

The shotcrete strain gauges show a similar pattern in strain to the rebar strain gauges with the strains increasing with the height of paste in the stope. Strain gauge “Center Horizontal @ 1.8 m” does not show any significant strain measurements and, much like the rebar strain gauge at 1.8 m, the low readings are because the paste height is below that of the instrument. The maximum recorded strain was approximately 32 με recorded by “Center Vertical @ 1.2 m” strain gauge at the end of the paste pour.
Figure 5.13: Fill Fence 34-786-1-14a Shotcrete Strain vs. Time

The results from the tiltmeters are shown in Figure 5.14. The tiltmeters measure deflection of the fill fence during the pouring of paste. A measured maximum deflection of 0.05 degrees in the lower portion of the fence was recorded, that would result in a deflection at mid-fence (fence height 3.4 m) of 1.5 mm.

One conclusion that is made from the loading is that the upper portion of the fence tilts back and the lower part of the fence tilts forwards. This is agreement with loading mechanism proposed by Revell and Sainsbury (2007b) and AMEC (2003), where it was anticipated that the fence would bulge in the near middle of the fence.
Fill Fence 34-786-14b was loaded in a continuous pour that reached an assumed total height of approximately 3.5 m. The initial paste was poured to a height of 0.6 m and subsequently flushed with two batches of water prior to the placement of the second pour. It was during this second water flush that the peak pressures were recorded in EPC 1 and EPC 2 (Figure 5.15). As subsequent placement of the paste proceeded however, a downward trend occurred in the pressures; this is unusual behaviour, as this was not observed in any previous fence. This reduction in pressure either could be due to the curing of the paste or possibly because waste rock in the stope acted as a baffle for the paste and did not allow it to load the fence in the same way as would happen in a stope without waste rock in it. The maximum load recorded by the lower EPC was 0.019 MPa.
Figure 5.15: Fill Fence 34-786-1-14b Pressure vs. Time

Figure 5.16 and Figure 5.17 show the strain in the rebar and the shotcrete respectively. The maximum strain recorded in the rebar was by the vertical strain gauge and measured 130 με during the flushing of the paste lines. As can be seen in Figure 5.16, the strain in the rebar begins to lessen after the initial flush of the paste lines. It can be seen that the rebar strain gauges all respond in similar fashion. However, the maximum values are different in all cases due to the relative stiffness of the materials.
Figure 5.17 shows the strain readings of the shotcrete during the paste pouring. It should be noted that the strain gauge “Center Horizontal @ 1.8 m” did not record any significant data and was reported as a faulty gauge. The trends of the shotcrete strain gauges correspond well with those of the rebar strain gauges, as increases occur at the same time in both cases. Further, these increases in strain also correspond with an increase in load recorded in the earth pressure cells. The maximum strain recorded was by the “Center Vertical @ 1.2 m” strain gauge, which recorded a value of 180 με.
The results of the tiltmeters are shown in Figure 5.18. As can be seen, very little deflection was measured during the loading of the fence. This is similar to the deflection measured for 34-786-14a.

The maximum deflection occurred in the lower part of the fence with an angle of 0.20° at the initial stages of loading. For this fence height (3.85 m) this would correspond to a mid fence horizontal displacement of 6.7 mm normal to the fence.
5.9 Fill Fence- Destructive Test

The results of the destructive fill fence test are integral to assessing the capacity of the fill fences under paste loading. It is critical to assess the loading mechanisms and the capacity of the fence and compare those to the results measured for the previous fences. The loads applied to the fence during this test were higher than those of the previous paste fill fences. This was done to determine at what load the fence would fail. Because of its importance, this test had the largest amount of instruments and the load applied to the fence needed to be measured accurately.

In order to have the most accurate reading possible for the fill fence, a load cell, hydraulic jack gauges and two EPCs were used to monitor the load during testing. Figure 5.19 shows the converted pressures of the load cell and hydraulic jack readings and the pressures recorded by the EPCs. The converted pressures for the load cell and hydraulic jack are simply the measured loads divided by the area of the loading plate.
As can be seen, there appear to be some discrepancies between the measured loads. The load cell and hydraulic cell match closely, whereas the EPCs tend not to follow the applied loads. The EPCs are most likely experiencing point loading, and the discrepancy between the hydraulic jack and load cell is due to the load cell being able to account for the load settling due to the removal of slack in the system.

The hydraulic jack and load cell agree with each other, having a correlation of 1.033 with an r-squared value of 0.89 when compared against each other. For further analysis, the load cell values will be used, as this is a calibrated and accurate instrument.

The EPCs do not agree with each other, or with the hydraulic jack or load cell. The loading of the fence was intended to be uniform, but the EPCs show that the majority of the load was concentrated on the lower part of the fence. The correlation between the earth pressure cells has a value of 0.07 with an r-squared value of 0.24, indicating that the loading was anything but symmetrical. It is considered odd that the value of the lower earth pressure cell exceeds the amount of energy input into the system. This would be possible if the EPCs were
point loaded, with loads being concentrated on the EPC and not on the surrounding plate. This asymmetrical loading can possibly be accounted for by:

- point loading of EPC,
- loading plate not parallel to fence during load application,
- loading plate not tight to fill fence, and
- defective loading cell.

Of the above, it would seem that the loading plate not being parallel to the fence, causing point loading in the lower cell would be the most likely explanation. As the EPCs provide poor results, they will not be used for further analysis.

The strains in the destructive fill fence are shown in Figure 5.20. As the test proceeds and higher loads are placed on the fence, the strain increases quite drastically, with the most noticeable increase showing on the horizontal top rebar strain gauge. Strain increases in the rebar are larger than those in the shotcrete panel. As the steel is stiffer, an increase in strain as compared to the same increase in shotcrete would be caused by a larger load on the steel, therefore from this test we can see that the steel is carrying most of the load during the test.
5.10 Summary of Observations

The maximum pressure and strains measured during the instrumentation are shown in Table 5.1. It should be noted that the majority of the maximum increases in stress and strains were measured during the flushing of the paste pour lines. As such, an alternative to flushing the paste lines within the backfilled stope should be investigated as this event causes the most dramatic rises in pressures and strains in the stope.
Table 5.1: Summary of Maximum Pressures and Strains on Fence

<table>
<thead>
<tr>
<th>Fence</th>
<th>Height (m)</th>
<th>Width (m)</th>
<th>Maximum Pressure (kPa)</th>
<th>Maximum Strain in Concrete (µε)</th>
<th>Maximum Strain in Rebar (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td>32-826-8</td>
<td>4.50</td>
<td>N/A</td>
<td>50</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>37-746-2</td>
<td>3.90</td>
<td>3.60</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>34-806-4</td>
<td>3.60</td>
<td>5.35</td>
<td>22.5</td>
<td>2000</td>
<td>N/A</td>
</tr>
<tr>
<td>31-806-3</td>
<td>4.25</td>
<td>4.25</td>
<td>20</td>
<td>125</td>
<td>N/A</td>
</tr>
<tr>
<td>36-746-1</td>
<td>5.25</td>
<td>12.25</td>
<td>25</td>
<td>-220</td>
<td>N/A</td>
</tr>
<tr>
<td>34-806-1</td>
<td>3.70</td>
<td>6.90</td>
<td>40</td>
<td>-375</td>
<td>N/A</td>
</tr>
<tr>
<td>34-786-14a</td>
<td>3.40</td>
<td>4.80</td>
<td>14</td>
<td>35</td>
<td>25</td>
</tr>
<tr>
<td>34-786-14b</td>
<td>3.85</td>
<td>3.50</td>
<td>19</td>
<td>180</td>
<td>130</td>
</tr>
<tr>
<td>Destructive Test</td>
<td>2.60</td>
<td>8.00</td>
<td>127</td>
<td>-50</td>
<td>1100</td>
</tr>
</tbody>
</table>

The maximum strains in the 34-786-14a and b fences were recorded within the vertically aligned instruments, with the strains being the largest in the shotcrete. However, the largest strains measured in the destructive test were in the upper horizontal rebar and the shotcrete gauges did not measure large amounts of strain. The difference between the strain measurements for these different fences is most likely due to the different mechanisms of loading.

The majority of the measured strains within the shotcrete were negative values, indicating an unloading or tensile related strain. This was unexpected as it was thought that the loading in the shotcrete would be compressive loading, resulting in positive strain. One explanation for this behaviour could be that the strain gauges underwent large values of strain opposite to the direction of loading during the curing of the concrete and as the loading of the fence progressed, the strain gauges began to ‘unload’ resulting in negative values. Another explanation is that the depth of embedment of the instruments within the shotcrete could affect the values, as the front of the fence will generally be in compression and the back of the fence would be in tension. However, this is one hypothesis and there is not enough information to clearly state why this behaviour is occurring.
6 ANALYSIS

In this Chapter, classical approaches of stress and strain analyses will be used to calibrate field results in order to understand the behaviour of the fence and paste mechanism during loading. This is not a trivial task as the interactions between the system parts are complex and the results are limited to the accuracy and precision of the instruments used in the analysis.

The analysis of the fence is carried out in two manners: analytically and numerically. Each will be discussed in separate chapters. These steps are aligned with rock mechanics design in which multiple approaches are employed to arrive at a calibrated response with subsequent implementation into future design.

The analytical analysis is divided into two sections: theory based analysis and field analysis based on the results of the testing. By comparing the field behaviour with theoretical design, an attempt to determine a relationship between previously published results and the analysis of the field study at Red Lake. Through these analyses, an understanding of the performance of the fence under paste loads can be compared to the failure strength of the fence.

The theoretical analysis is based on work done in Australia by Beer (1986), where theory of concrete walls is applied to bulkhead design. The field analysis draws its conclusion from interpreting the data presented in Chapter 5. The data is interpreted to determine the loading mechanism of the paste, and the stresses induced by strain within the elements. The stress vs. strain plots are interpreted to determine if the fences are being loaded to the point of yielding.

6.1 Theoretical Analysis

As mentioned in Chapter 3, with multiple catastrophic failures of bulkheads in Australia, a substantial amount of research has been conducted there on bulkhead design. Current theoretical analysis applied in Australia mines is based on research by Beer (1986) and the Timoshenko and Young yield line theory depending on the shape of the fence (Revell and Sainsbury, 2007b).

The yield line theory proposed by Beer assumes that a square concrete slab will fail in tension along the diagonal lines along the bulkhead/rock interface. Design is based on the following equation (Revell and Sainsbury, 2007b, p. 3):
Equation 6.1

\[ w_p = \frac{24m_p}{b^2} \]

Where:

\[ w_p = \text{Wall Pressure at failure} \]
\[ m_p = \text{Plastic Moment} \]

Equation 6.2

\[ m_p = \sigma_c \frac{h^2}{8} \]

\[ \sigma_c = \text{Compressive Strength of Material (MPa)} \]
\[ b = \text{height and width of square fence (m)} \]
\[ h = \text{thickness of fence (m)} \]

Applying this theory to the fences tested, Table 6.1 summarizes the capacity at failure using equations 6.1 and 6.2. It should be noted that the fences were not square and that the thickness of the fences was assumed 10 cm thick based on the design guidelines used by Goldcorp (2002) and recommended by AMEC (2003). Further, the strength of the shotcrete was 30 MPa (Revell, 2007). For cases where the fence dimensions were not square, the smaller of the two dimensions will be used for a conservative answer.

Table 6.1: Fence Capacity Based on Beer (1986)

<table>
<thead>
<tr>
<th>Fence</th>
<th>Height (m)</th>
<th>Width (m)</th>
<th>b (m)</th>
<th>h (m)</th>
<th>( \sigma_c ) (MPa)</th>
<th>( m_p ) (Mn/m)</th>
<th>( w_p ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>32-826-8</td>
<td>4.50</td>
<td>N/A</td>
<td>4.5</td>
<td>0.10</td>
<td>30</td>
<td>0.039</td>
<td>45.9</td>
</tr>
<tr>
<td>37-746-2</td>
<td>3.90</td>
<td>3.60</td>
<td>3.6</td>
<td>0.10</td>
<td>30</td>
<td>0.039</td>
<td>71.7</td>
</tr>
<tr>
<td>34-806-4</td>
<td>3.60</td>
<td>5.35</td>
<td>3.6</td>
<td>0.10</td>
<td>30</td>
<td>0.039</td>
<td>71.7</td>
</tr>
<tr>
<td>31-806-3</td>
<td>4.25</td>
<td>4.25</td>
<td>4.25</td>
<td>0.10</td>
<td>30</td>
<td>0.039</td>
<td>51.4</td>
</tr>
<tr>
<td>36-746-1</td>
<td>5.25</td>
<td>12.25</td>
<td>5.25</td>
<td>0.10</td>
<td>30</td>
<td>0.039</td>
<td>33.7</td>
</tr>
<tr>
<td>34-806-1</td>
<td>3.70</td>
<td>6.90</td>
<td>3.7</td>
<td>0.10</td>
<td>30</td>
<td>0.039</td>
<td>67.9</td>
</tr>
<tr>
<td>34-786-14a</td>
<td>3.40</td>
<td>4.80</td>
<td>3.4</td>
<td>0.10</td>
<td>30</td>
<td>0.039</td>
<td>80.4</td>
</tr>
<tr>
<td>34-786-14b</td>
<td>3.85</td>
<td>3.50</td>
<td>3.5</td>
<td>0.10</td>
<td>30</td>
<td>0.039</td>
<td>75.8</td>
</tr>
<tr>
<td>Destructive Test</td>
<td>2.60</td>
<td>8.00</td>
<td>2.6</td>
<td>0.10</td>
<td>30</td>
<td>0.039</td>
<td>137.4</td>
</tr>
</tbody>
</table>

Table 6.1 illustrates that the smaller the size of the fence (‘b’ factor), the greater the overall strength of the fence. The results from the above analysis then can be compared to those
of the maximum pressure exerted on the fence (Table 5.1). Table 6.2 demonstrates the percent of the total capacity of the fence mobilized by the maximum pressures recorded in the field.

Table 6.2: Percent Total Capacity of Fence Mobilized during Testing

<table>
<thead>
<tr>
<th>Fence</th>
<th>Maximum Pressure Measured (kPa)</th>
<th>Total Capacity ( w_p ) (kPa)</th>
<th>% Of Total Capacity Mobilized</th>
</tr>
</thead>
<tbody>
<tr>
<td>32-826-8</td>
<td>50</td>
<td>45.9</td>
<td>109%</td>
</tr>
<tr>
<td>37-746-2</td>
<td>N/A</td>
<td>71.7</td>
<td>N/A</td>
</tr>
<tr>
<td>34-806-4</td>
<td>22.5</td>
<td>71.7</td>
<td>31%</td>
</tr>
<tr>
<td>31-806-3</td>
<td>20</td>
<td>51.4</td>
<td>39%</td>
</tr>
<tr>
<td>36-746-1</td>
<td>25</td>
<td>33.7</td>
<td>74%</td>
</tr>
<tr>
<td>34-806-1</td>
<td>40</td>
<td>67.9</td>
<td>59%</td>
</tr>
<tr>
<td>34-786-14a</td>
<td>14</td>
<td>80.4</td>
<td>17%</td>
</tr>
<tr>
<td>34-786-14b</td>
<td>19</td>
<td>75.8</td>
<td>25%</td>
</tr>
<tr>
<td>Destructive Test</td>
<td>127</td>
<td>137.4</td>
<td>92%</td>
</tr>
<tr>
<td>Destructive Test (^1)</td>
<td>170</td>
<td>137.4</td>
<td>124%</td>
</tr>
</tbody>
</table>

Note:\(^1\) Load of 170 kPa was placed on fence but no instrument readings were taken.

Based on the above table it can be seen that Beer’s (1986) analysis determines that Fence 32-826-8 and the Destructive Test should have failed. Although neither of the fences failed during testing, the Destructive Test ultimately failed after a re-application of the load at a reported value of 150 kPa.

Limitations with Beer’s analysis are that it considers the fence to be homogenous (better suited to a shotcrete only fence), square and supported on all four sides. This is not the case with Red Lake’s fences as they are a composite shotcrete and rebar fence, typically wider than they are high and the majority of the fences are not secured along the top of the fence. However, as Beer’s analysis did predict failure in the destructive test there appears to be some merit to its use as part of the overall design.
6.2 Field Analysis

The field analysis will investigate the loading mechanisms of paste, the stress in the shotcrete and rebar as determined by the measured strain and the stress-strain response of selected fences during loading.

6.2.1 Loading Mechanism of Paste

The loading mechanism of paste was determined through study of the instrumented paste pours. The load mechanism analysis is to determine how much of lateral load is applied against a fence for a given height of uniform paste. As such, the ratio of assumed vertical load vs. measured horizontal loads was investigated and the paste was considered a Rankine Soil during this loading of the fence. This analysis assumes that the soil is still in saturated form and has not commenced hydrating.

Fill heights were necessary to determine the friction angle of the paste during initial placement. This was an issue as not all the fences recorded actual or inferred height of paste during pouring. Once the fill heights were known, they can be compared to the lateral earth pressure recorded by EPCs to determine if a Rankine linear relationship exists. The fences that fit these criteria were:

- 36-746-1,
- 34-806-1,
- 34-786-14a, and
- 31-806-3.

Using Figure 5.6, Figure 5.7, Figure 5.9 and Figure 5.11, average lateral earth pressures for indicated measured heights of paste were selected from the graph. These points were plotted as shown in Figure 6.1, to determine the correlation between the data. As expected, there appears to be a linear trend in the data with a slope ratio of approximately 9 horizontal: 1 vertical and an r-squared value of 0.79 demonstrating a good relationship.
Since the fill height is in general agreement with the lateral earth pressure, Rankine Theory can be used to determine the coefficient of active lateral earth pressure (Ka).

By knowing the depth of fill and the lateral earth pressures placed upon the EPCs for the known fill height we can determine the coefficient of lateral earth pressure and in turn the friction angle for the paste during placement. Table 6.3, Table 6.4, Table 6.5, and Table 6.6, show the individual friction calculation for the known lateral earth pressures and depths of paste.

The effect of the rate of paste placement was analyzed as shown in Figure 6.2. The friction angle does not seem to be dependent upon fill rate as the coefficient of determination ($R^2$) is low and that the slope of the trend line is nearly horizontal. This figure supports the findings of Figure 6.1, as there is a strong agreement between the data of fill height and lateral earth pressure without incorporating fill rate. It should be noted that this is limited to four data points. Further study is required to define this relationship.
Poor agreement between fill rate and friction angle. Slope of trend line is near horizontal indicating that friction angle is not related to fill rate.

\[ R^2 = 0.024 \]

Figure 6.2: Friction Angle vs. Fill Rate

Since no useful data was collected during the paste pour regarding percent of cement used, it is considered for the analysis that the paste will behave in the same way regardless of the percent of cement. Fence 36-746-1 was poured with two paste batches with separate percentages of cement. On August 7, 2004, the paste changed from 5% cement to 10% cement (reference Figure 5.7). As can be seen from the figure no sudden changes are evident and based on Table 6.6, the resulting friction angles for instruments embedded in the separate paste pours are 21.9° and 10.4° for the 5% and 10% pastes respectively. These findings are in sharp contrast to those of Dehn et al. (2007), where in Fill Fence 34-786-1-14a, the friction angles for the 5% and 10% were determined to be 13° and 19°. The analyses show the resultant friction angles counteract each other based on the percent cement in the paste.

As such, for the purposes of numerical modeling and subsequent analysis the percentage of cement and fill rate will be considered not to have an effect on the friction angle of the paste during placement.
Table 6.3: Friction Angle Determination for Fill Fence 34-786-14a

<table>
<thead>
<tr>
<th>Recorded height of fill above Floor (m)</th>
<th>34-786-14a (Elev 0.6m)</th>
<th>Fill Depth above instrument (m)</th>
<th>Average Pressure Reading (kPa)</th>
<th>Ka Value based on readings</th>
<th>Friction Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>N/A</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0.70</td>
<td>0.10</td>
<td>1.00</td>
<td>0.54</td>
<td>17.5</td>
<td></td>
</tr>
<tr>
<td>1.2</td>
<td>0.60</td>
<td>6.50</td>
<td>0.58</td>
<td>15.3</td>
<td></td>
</tr>
<tr>
<td>1.3</td>
<td>0.70</td>
<td>8.15</td>
<td>0.62</td>
<td>13.3</td>
<td></td>
</tr>
<tr>
<td>1.7</td>
<td>1.10</td>
<td>11.00</td>
<td>0.54</td>
<td>17.5</td>
<td></td>
</tr>
<tr>
<td><strong>Average Values</strong></td>
<td></td>
<td></td>
<td><strong>0.57</strong></td>
<td><strong>15.9</strong></td>
<td></td>
</tr>
</tbody>
</table>

Table 6.4: Friction Angle Determination for Fill Fence 31-806-3

<table>
<thead>
<tr>
<th>Recorded height of fill above Floor (m)</th>
<th>31-806-3 (Elev 0.8m)</th>
<th>Fill Depth above instrument (m)</th>
<th>Average Pressure Reading (kPa)</th>
<th>Ka Value based on readings</th>
<th>Friction Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>N/A</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.52</td>
<td>0.72</td>
<td>7.00</td>
<td>0.52</td>
<td>18.5</td>
<td></td>
</tr>
<tr>
<td>2.74</td>
<td>1.94</td>
<td>17.00</td>
<td>0.47</td>
<td>21.2</td>
<td></td>
</tr>
<tr>
<td>4.27</td>
<td>3.47</td>
<td>20.00</td>
<td>0.31</td>
<td>31.8</td>
<td></td>
</tr>
<tr>
<td><strong>Average Values</strong></td>
<td></td>
<td></td>
<td><strong>0.43</strong></td>
<td><strong>23.8</strong></td>
<td></td>
</tr>
</tbody>
</table>

Table 6.5: Friction Angle Determination for Fill Fence 34-806-1

<table>
<thead>
<tr>
<th>Recorded height of fill above Floor (m)</th>
<th>34-806-1 (Elev 0.60m)</th>
<th>Fill Depth above instrument (m)</th>
<th>Average Pressure Reading (kPa)</th>
<th>Ka Value based on readings</th>
<th>Friction Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>N/A</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>0.46</td>
<td>N/A</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1.65</td>
<td>1.05</td>
<td>12.00</td>
<td>0.62</td>
<td>13.75</td>
<td></td>
</tr>
<tr>
<td>2.56</td>
<td>1.96</td>
<td>22.00</td>
<td>0.60</td>
<td>14.37</td>
<td></td>
</tr>
<tr>
<td>3.26</td>
<td>2.66</td>
<td>32.00</td>
<td>0.65</td>
<td>12.45</td>
<td></td>
</tr>
<tr>
<td>3.51</td>
<td>2.91</td>
<td>36.00</td>
<td>0.67</td>
<td>11.60</td>
<td></td>
</tr>
<tr>
<td><strong>Average Values</strong></td>
<td></td>
<td></td>
<td><strong>0.63</strong></td>
<td><strong>13.0</strong></td>
<td></td>
</tr>
</tbody>
</table>
Table 6.6: Friction Angle Determination for Fill Fence 36-746-1

<table>
<thead>
<tr>
<th>Recorded height of fill above Floor (m)</th>
<th>36-746-1 (Elev 0.55m)</th>
<th>36-746-1 (Elev 2.6m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fill Depth above instrument (m)</td>
<td>Average Pressure Reading (kPa)</td>
</tr>
<tr>
<td>0</td>
<td>N/A</td>
<td>-</td>
</tr>
<tr>
<td>0.91</td>
<td>0.36</td>
<td>3.50</td>
</tr>
<tr>
<td>1.67</td>
<td>1.12</td>
<td>9.66</td>
</tr>
<tr>
<td>2.5</td>
<td>1.95</td>
<td>17.24</td>
</tr>
<tr>
<td>3.4</td>
<td>2.85</td>
<td>22.07</td>
</tr>
<tr>
<td>3.72</td>
<td>3.17</td>
<td>25.00</td>
</tr>
<tr>
<td>Average Values</td>
<td>0.46</td>
<td>21.9</td>
</tr>
</tbody>
</table>
Further to the fences analyzed above, Fill Fence 32-826-8 can be used to determine the coefficient of active lateral earth pressure. However, since paste depth were not recorded, as was the case with the previous analyzed fences, the vertical EPC and horizontal EPC located 0.25 m above the floor can be used. The slope of the line of the horizontal vs. vertical stress in Figure 6.3 is the coefficient of active soil pressure value. As can be seen this has a value of 0.55 and a $R^2$ value of 0.92 indicating a very good correlation. The deviation from the best fit line at the 0.05 MPa point is most likely due to the curing and the development of self cohesion within the paste. This $K_a$ value is in general agreement with those determined from the measured paste height analysis discussed previously. This value is a direct measurement of horizontal load vs. vertical load, whereas the previous values were indirect measurements based on measured fill heights.

The paste internal angle of friction was determined to be on average 17 degrees (Table 6.7). There has been no separation of paste based on the fill rate and the percent of cement in paste in the analysis as there seems to be a general agreement between horizontal and vertical
loads recorded in the field as shown in Figure 6.1. Further, it was assumed that the paste had not undergone any hydration during the measurements and as such had no cohesion value.

### Table 6.7: Summary of Friction Angle Analysis

<table>
<thead>
<tr>
<th>Fill Fence</th>
<th>Instrument Height (m)</th>
<th>Ka Value</th>
<th>Friction Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>36-746-1</td>
<td>0.55</td>
<td>0.46</td>
<td>21.9</td>
</tr>
<tr>
<td>34-806-1</td>
<td>2.6</td>
<td>0.69</td>
<td>10.4</td>
</tr>
<tr>
<td>34-786-14a</td>
<td>0.6</td>
<td>0.57</td>
<td>15.9</td>
</tr>
<tr>
<td>31-806-3</td>
<td>0.8</td>
<td>0.57</td>
<td>15.9</td>
</tr>
<tr>
<td>32-826-8</td>
<td>0.25</td>
<td>0.55</td>
<td>16.9</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>0.56</td>
<td>16.99</td>
</tr>
</tbody>
</table>

The analysis of lateral earth pressure vs. fill height or vertical pressure shows that there exists a strong relationship between the two selected variables during initial stages of paste pours. This value was defined as the coefficient of active lateral earth pressure, $K_a$, and was determined to be 0.56 and did not seem to be affected by percent of cement in the paste or the fill rate. This value of $K_a$ demonstrates that the loading of the fence is not hydrostatic, indicating that the paste is self supporting during loading contrary to findings published by Marcinynshyn et al. (1999), Mitchell (1991) and Revell and Sainsbury (2007a).

### 6.2.2 Induced Stress Due to Strain

The following equation calculates the stresses within the elements due to the measured strains:

$$\sigma = E\varepsilon \quad \text{Eqn. 6.3}$$

Where:

- $\sigma = \text{Stress}$
- $E = \text{Modulus of Elasticity}$
- $\varepsilon = \text{Strain}$

The tensile strength and modulus of elasticity for the #6 rebar are 425 MPa and 200,000 MPa respectively, while the tensile strength and modulus of elasticity for shotcrete are 5.3 MPa and 20,000 MPa (Dehn et al., 2007).

This relationship can be used on the fences that had strain gauges installed, that is all except fences 32-828-6 and 37-746-2 as these two were the first fences tested and no strain...
gauges were installed. The results for the stress due to strain can be seen on the secondary axis in selected figures in Chapter 5. Table 6.8 is a summary of the maximum stresses recorded, showing corresponding figures for each applicable fence.

Table 6.8: Summary of Maximum Strain Related Stresses in Fence

<table>
<thead>
<tr>
<th>Fill Fence</th>
<th>Figure</th>
<th>Instrument Type</th>
<th>Maximum Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>34-806-4</td>
<td>5-4</td>
<td>Shotcrete SG</td>
<td>42</td>
</tr>
<tr>
<td>31-806-3</td>
<td>5-6</td>
<td>Shotcrete SG</td>
<td>2.5</td>
</tr>
<tr>
<td>36-746-1</td>
<td>5-8</td>
<td>Shotcrete SG</td>
<td>-4.4</td>
</tr>
<tr>
<td>34-806-1</td>
<td>5-10</td>
<td>Shotcrete SG</td>
<td>-7.25</td>
</tr>
<tr>
<td>34-786-1-14a</td>
<td>5-12</td>
<td>Rebar SG</td>
<td>4.5</td>
</tr>
<tr>
<td>5-13</td>
<td>Shotcrete SG</td>
<td></td>
<td>0.65</td>
</tr>
<tr>
<td>5-16</td>
<td>Rebar SG</td>
<td></td>
<td>26</td>
</tr>
<tr>
<td>34-786-1-14b</td>
<td>5-17</td>
<td>Shotcrete SG</td>
<td>3.75</td>
</tr>
<tr>
<td></td>
<td>Rebar SG</td>
<td></td>
<td>220</td>
</tr>
<tr>
<td>Destructive Test</td>
<td>5-20</td>
<td>Shotcrete SG</td>
<td>-1.01</td>
</tr>
</tbody>
</table>

From the maximum stresses, we can determine the amount of total tensile capacity of the material that is mobilized, summarized in Table 6.9.

Table 6.9: Summary of Mobilized Tensile Strength

<table>
<thead>
<tr>
<th>Fill Fence</th>
<th>Material</th>
<th>% of σt</th>
</tr>
</thead>
<tbody>
<tr>
<td>34-806-4</td>
<td>Shotcrete</td>
<td>792%</td>
</tr>
<tr>
<td>31-806-3</td>
<td>Shotcrete</td>
<td>47%</td>
</tr>
<tr>
<td>36-746-1</td>
<td>Shotcrete</td>
<td>83%</td>
</tr>
<tr>
<td>34-806-1</td>
<td>Shotcrete</td>
<td>137%</td>
</tr>
<tr>
<td>34-786-1-14a</td>
<td>Rebar</td>
<td>1%</td>
</tr>
<tr>
<td></td>
<td>Shotcrete</td>
<td>12%</td>
</tr>
<tr>
<td>34-786-1-14b</td>
<td>Rebar</td>
<td>6%</td>
</tr>
<tr>
<td></td>
<td>Shotcrete</td>
<td>71%</td>
</tr>
<tr>
<td>Destructive Test</td>
<td>Rebar</td>
<td>52%</td>
</tr>
<tr>
<td></td>
<td>Shotcrete</td>
<td>19%</td>
</tr>
</tbody>
</table>

From Table 6.8, it can be seen that two gauges exceed the tensile capacity of the material. The shotcrete gauges in 34-806-4 and 34-806-1 have exceeded the tensile capacity of their material and would represent a failed element. A further three strain gauges, (Rebar Destructive test, shotcrete 36-746-1, shotcrete 34-786-1-14b) recorded readings that were above 50% and may be undergoing plastic strain, but further study is needed.
It is interesting to note that from the theoretical analysis in Chapter 6.1, none of the fences was predicted to fail based on theory. However, this analysis shows that two fences were considered to have failed elements.

This analysis only considers the elastic strain and does not incorporate plastic strains. As such, the stresses predicted by the analysis may not reflect the true behaviour should plastic deformations occur. Further, the shotcrete strain gauges in all fences except the destructive test are installed onto the rebar and could be influenced by the movement of the rebar rather than the shotcrete, and demonstrate these setups.

The stress induced by the strain analysis shows that the strains in the fence tend to vary between compressive and tensile strain. No failures were reported in any of the cases where failure was predicted by tensile stresses exceeding capacity of the fence. As such, this analysis does not provide any useful information.

6.2.3 Stress-Strain Behaviour Analysis

The stress-strain behaviour analysis of the fence is to determine if the fence undergoes any plastic strain during imposed loads. The plastic strain is defined as the deformation of a material that is irrecoverable when the applied load is removed (Mott, 2002). As such, the elastic portion of the stress-strain behaviour tends to be linear, whereas the plastic deformation can occur under constant load. When a material enters a plastic strain dominant behaviour it is said to be yielding and the stress vs. strain curve begins to flatten out. (Mott, 2002)

To determine the stress vs. strain behaviour, fences with known fill heights and instrument locations will be analyzed to determine the imposed paste loads and how they translate to fence strain. The fences that can be used for this analysis are:

- 31-806-3,
- 36-746-1,
- 34-806-1,
- 34-786-14a, and
- Destructive Test.

To carry out the stress vs. strain analysis, an assumption has to be made that the strain in the strain gauge is directly related to the imposed load of the paste and not due to bending moments of the fence. Further, a fill rate has to be assumed to determine the height of paste in
the stope. In the case of fence 34-786-14a, the fence was poured at a constant rate and the fill rate is based on the total height of the paste divided by the time taken to fill the stope. For the other paste fences, the total height was divided by the time to reach the fill height, however the paste pour was not continuous and therefore the assumed fill heights do not necessarily match the height of paste in the stope. From this, the paste fill height and in turn, imposed stress can be determined for every timed instrument reading.

The calculated imposed stress readings are then plotted against the recorded stress values. It should be noted that since instruments are located above the floor of the stope, only values where the assumed fill height is above the instrument would be used. For the destructive fill fence test, the stress vs. strain plots are directly related to the imposed loads from the loading system and the recorded strains during testing.

The following analyses investigates the stress-strain behaviour of the fences and attempts to understand the capacity of the fences by determining if they show yielding behaviour.

6.2.3.1 Fill Fence 31-806-3

Fill fence 31-806-3 instrument locations and dimensions are discussed in Chapter 4 and strain values are shown in Chapter 5.

Records show the paste pour commenced on July 9, 2004 at 2:30 pm and finished at a height of 4.25 m on July 11, 2004 at 7:30 p.m. Therefore, the overall paste fill rate for this stope is 0.08 m/hr assuming the pour was continuous. Figure 6.4 shows the assumed stress vs. measured strain for the fence. As can be seen, the trends of the plots are not linear; however, they do not show any apparent yielding.
General low stress and near linear trend imply that fence is not yielding

Figure 6.4: Fill Fence 31-806-3 Stress vs. Strain

6.2.3.2 Fill Fence 36-746-1

Instrument location and strain plots for 36-746-1 are found in Chapter 4 and 5 respectively.

Paste pouring began on August 4, 2004 at 10:35 pm and finished at August 5, 2004 at 10:30 pm at a height of 2.5 m; if the pour were assumed continuous, this would result in fill rate of 0.10 m/hr. A long period of time elapsed before the remainder of the stope was filled to a total height of 3.8 m. As such only the first pour will be analyzed.

Figure 6.5 shows the assumed imposed stress vs. measured strain for the concrete strain gauges. It can be seen that the behaviour is near linear and that no yielding is occurring.
Linear Trend shown in all strain gauges indicating no yielding occurring.

Figure 6.5: Fill Fence 36-746-1 Stress vs. Strain

6.2.3.3 34-806-1

Instrument locations and measured strain for 34-806-1 are discussed in Chapter 4 and 5.

Records for the paste pouring state that placement began on August 25, 2004 at 9:35 am and stopped on August 28, 2004 at 1:00 pm at a total paste height of 4.25 m. The fill rate for this stope, assuming continuous pouring, is 0.06 m/hr.

Figure 6.6 shows the assumed induced stress vs. measured strain for the fence. SG1 has unreliable readings that fluctuate widely, and should be ignored. SG 2 and SG 3 show that the general trends of the plots are linear, indicating that no yielding of the elements is occurring.
Strain is negative indicating tensile strain

SG1 random reading but readings are still relatively low compared to largest measured strains

SGs trends are near linear indicating no yielding occurring

Figure 6.6: Fill Fence 34-806-1 Stress vs. Strain

6.2.3.4 Destructive Fill Fence Test

The stress vs. strain plot for the destructive fill fence test directly relate to the applied stress measured by the load cell to the measured strain, as opposed to the assumed stresses in the other cases.

As can be seen in Figure 6.7, the stress vs. strain plot shows the yielding of the rebar elements around the 100 kPa load. From there, the slight increase in stress causes a large non-linear increase in strain. This is due to plastic deformation within the rebar. It should be noted that the concrete strain gauges are not undergoing plastic strain at these loads. It can be concluded that for the destructive fill fence test, the rebar membranes are taking the majority of the applied loads, thereby undergoing plastic deformations. With a yielding value of 100 kPa and a Ka value of 0.56, the Rankine Theory implies that the fence, as constructed, could withstand a paste height of 9.5m.
6.2.4 Summary of Results for Stress Strain Analysis

Figure 6.8 shows a combination plot of all the stress vs. strain measurements with the exception of Fence 36-746-1/SG1, as this gauge's readings are questionable. It can be seen that only the rebar strain gauges in the destructive fill fence yielded during testing and that the strain gauges within the shotcrete elements failed to yield under loading. Further, the highest loads occurred in the destructive fill test, and based on this analysis, it can be concluded that below loads of approximately 100 kPa fill fences do not exhibit yielding behaviour.
Figure 6.8: All Fences: Stress vs. Strain

- Below 40 kPa: recorded stress and strain within instrumented paste pour fences
- 100 kPa: yielding of fence

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7 NUMERICAL MODELLING

7.1 Introduction

Numerical modeling was utilized to couple the analysis of the paste pour with the analysis of the fill fence pull test (destructive test) and to further develop the understanding of the loading mechanism of paste and the reaction of the fence. In total three models were constructed: a model to determine the parameters of the paste during pouring, a model to simulate the pull test and a coupled model of the paste and fill fence to determine loads on the fence during pours.

The purpose of the numerical model is to understand the loading mechanism of paste and to reproduce the applied loads of the paste in the field test. This process involved numerous trials that varied the model parameters so that they reflect the field results; these parameters included the friction angle, unconfined compressive strength, Young’s Modulus and Poisson’s ratio. Once these parameters are known they can be combined with the fill fence model to form a coupled numerical model.

The fill fence model was developed to approximate the behaviour of the fill fence during the destructive test. The model is constructed to represent a composite of shotcrete and rebar. The material parameters were varied to reflect failure induced as recorded by the destructive fill fence test. Although not a true representation of the fill fence, it is the goal to have the numerical model induce failure at the same loads as the field test. Once complete, the fill fence model is to be coupled with the paste model to determine the capacity of the fence under the paste loads.

The purpose of the coupled model is to compare the failure loads imposed by the destructive fill fence test and the paste loading model to see if the paste loads can lead to a fill fence failure.

Since a coupled model was needed, it was difficult in selecting a model package that would incorporate both.

7.1.1 Model Software Selection

Various software programs were considered for use in modeling the paste and fence behaviour. The software selected would need to have the following characteristics: non-linear behaviour, time-stepped behaviour, industry acceptance and ease of use.
HCItasca’s FLAC2D and FLAC3D (Itasca, 2003) were used as initial models. FLAC is a continuum code that is well suited for time-stepped problems. The code can formulate large strains and displacements and non-linear behaviour. FLAC2D and FLAC3D have been used successfully to model paste and bulkheads by Hughes et al. (2006), Revell (2007) amongst others.

HCItasca’s code was not selected for modeling since a parametric study would need to take place to determine the numerical parameters of both the paste and the fill fence; this type of modeling with the FLAC code is tedious and requires considerable computer resources.

AMEC used SAP2000 (Computer & Structures Inc., 2000), a software package developed at University of California at Berkley, to study the paste fill fence loading at Red. SAP2000 is 3D software that is used in civil engineering for bridge and building design. As far as the criteria set out for software selection, SAP2000 is able to perform time-staged calculations and accommodate non-linear behaviour. However, SAP2000 is used predominantly in the civil industry and after a literature search, was found not be in use in the underground mining field. Another issue with SAP2000 was that the software was expensive and beyond the scope of the study.

Rocscience’s Phase$^2$ (2007) was selected for modeling both the paste and fill fence models. Phase$^2$ is an elastic-plastic finite element stress model that incorporates time-staged properties. Phase$^2$ is used throughout the mining industry and extensively in rock mechanics; publications by Hoek (2003), Curran (2003) and Martin (1999) show the effectiveness of Phase$^2$ in modeling underground environment. Phase$^2$ is easy to use, fast computing modeling software that allows for quick modeling results, ideal for parametric studies.

7.2 Paste Loading Model

A Phase$^2$ model was developed to represent numerically the paste loading mechanism discussed in Chapter 6. The model was constructed with a 4m high paste fill fence, with 16 stages of 0.25 m paste lifts.

7.2.1 Model Parameters

Three materials were used in the model: Host rock, paste and a fence composite.
7.2.1.1 Host Rock

The host rock was modeled to be similar in composition to the host rocks at Red Lake Mine. However, as only body forces were used in the analysis, the properties of the host rock were inconsequential to the model results. Suitable values based on results from testing performed by Mah (1995) were used as properties for the rock, as shown in Table 7.1. Since no direct shear tests were performed on the rock, an assumed friction angle was used, based on the geology and published values by Barton (1973). The rock was designated as an isotropic, elastic material with a Mohr-Coulomb failure criterion. It should be noted that the Mohr-Coulomb failure criterion is considered a plastic model, but the Phase\(^2\) software (Rocscience, 2007) allows the user to input the failure criterion to define the elastic portion of the failure surface.

Table 7.1: Host Rock Model Properties

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Unit Weight (MN/m(^3))</th>
<th>UCS (MPa)</th>
<th>Friction Angle</th>
<th>Young’s Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basalt</td>
<td>0.027</td>
<td>105</td>
<td>30°</td>
<td>50000</td>
<td>0.3</td>
</tr>
</tbody>
</table>

7.2.1.2 Paste Parameters

The properties and parameters for the paste were derived through a parametric model study to reflect the behaviour of the paste recorded in the field during the paste pour instrumentation. The properties and parameters necessary for the Phase\(^2\) model are unit weight, tensile strength, cohesion, friction angle, Young’s modulus and Poisson’s ratio.

The unit weight of the material was determined by Dehn et al. (2007) in which the paste at Red Lake was determined to be 18.63 kN/m\(^3\). This was input directly into the model as the unit weight for the paste and kept constant throughout the parametric study of the elastic properties.

The tensile strength and cohesion was based on the unconfined compressive strength value (UCS) of the paste. A relationship for the tensile and cohesive strength of based on Mohr-Coulomb theory is approximately one tenth and one quarter of the UCS respectively (Caceres, 2005). The unconfined compressive strength of the paste was determined from testing performed by Golder (2002). Golder tested six samples (three 6 day samples and three 28 day samples); the average 28 day strength of the samples was 1.96 MPa. The design strength of the
paste is 2.0 MPa, as this strength is proved through testing; this value was used in the model design. With a UCS value of 2.0 MPa, the tensile and cohesive values for the paste were 0.2 MPa and 0.5 MPa respectively.

The friction angle for paste was discussed in Chapter 6.2.1. Based on the analysis presented in that chapter, Table 6.7 shows that the average friction angle based on the data from all fences is $17^\circ$, this value was used as a constant in the Phase$^2$ model for the paste friction angle.

### 7.2.1.3 Paste Elastic Properties

The Young’s Modulus and Poisson’s ratio for the paste was determined through a parametric study of a Phase$^2$ Model. The model was to replicate the best-fit line of recorded paste fill shown in Figure 6.1. As the unit weight, friction angle and compressive strength of the paste are constant for the model to replicate the findings, only the Young’s Modulus and Poisson’s ratio can be changed.

The numerical model was set up as shown in the schematic in Figure 7.1 and within the model in Figure 7.2. The fill fence height is four meters high and 0.1 m wide. For this part of the model the fence was constructed to be indestructible so that the influence of the fence was negligible. The model was set up to approximate the loading of a fence in stages. Sixteen- 0.25 m lifts were used to represent the recommended loading rate as prescribed by AMEC (2003). Since the material is fully saturated upon placement, a modeled piezometric surface was implemented at the height of the paste in each lift. Figure 7.1 and Figure 7.2 do not show the piezometric levels for clarity purposes.
Figure 7.1: Schematic of Phase 2 Paste Model

Figure 7.2: Close up of Phase 2 Paste Model
As mentioned above, the purpose of the model was to represent the best-fit loading of the paste as shown in Figure 6.1. To achieve this, material queries were set up 0.5 m, 1 m and 10 m away from the fence and compared to the best-fit line in the above figure. Only the Young’s Modulus and Poisson’s ratio were altered while the remaining properties were kept constant. Error! Reference source not found. shows the combination of the parameters used in the parametric study; values changed are shown in bold. Figure 7.3 shows the results of the lateral earth pressure vs. fill height for the parametric studies, note: (a), (b) and (c) relate to the distance of the earth pressures away from the wall: 0.5 m, 1m and 10 m respectively.

Table 7.2: Parameter Variation for Phase2 Parametric Study

<table>
<thead>
<tr>
<th>Parameter</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesion (MPa)</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
<td>17</td>
</tr>
<tr>
<td>Unit Weight (kN/m3)</td>
<td>18.63</td>
<td>18.63</td>
<td>18.63</td>
<td>18.63</td>
<td>18.63</td>
<td>18.63</td>
<td>18.63</td>
<td>18.63</td>
<td>18.63</td>
</tr>
<tr>
<td>Tension (MPa)</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Modulus of Elasticity (MPa)</td>
<td>500</td>
<td>500</td>
<td>500</td>
<td>500</td>
<td>250</td>
<td>500</td>
<td>500</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.3</td>
<td>0.4</td>
<td>0.45</td>
<td>0.4</td>
<td>0.4</td>
<td>0.35</td>
<td>0.38</td>
<td>0.33</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Figure 7.3 shows the results of the material queries for all trials and their comparison to the field trial values. From the parametric study, it was found that the results near the fence, Trials “a” and “b”, were generating values that did not realistically match real world behaviour. This unusual behaviour involved negative values being recorded 0.25 m away from positive values. Since this behaviour was not recorded in the field, it would seem that the fill fence in this model influences the paste, as such the material query ‘c’ for all trials was used to fit the data. The most likely reason for the unusual behaviour near the fence is the discretization of the model. Phase2 uses an automatic discretization, which for this model generate large triangle nodes throughout the paste and small triangle mesh within the fence. The large triangle mesh caused the model to behave unusual near the smaller mesh of the fence.
Figure 7.4 shows the model output for the queries used in the analysis. It can be seen that the queries are set to record stress values at 0.25 m intervals. The model shows the horizontal earth pressure, or in the case of the paste loading the minor principal stress ($\sigma_3$). It can be seen from the query and contours shown that the hypothesis of the fill fence influencing the paste during modeling is valid. It should be noted that this behaviour was not noted in any of the instrumented fill fences. An explanation for the behaviour of the model near the fence as shown in and is the discretization and meshing of the model. Because the paste material is modeled as a 0.25 m x 30 m unit, the three-node triangles that Phase$^2$ utilize are relatively narrow compared to their overall length and are poor at transferring stresses at their boundary. (Rocscience, 2007)
Stress distribution 10 m away from fence matches measured stress distribution in field tests.

Model discretization is likely reason for differences.

Figure 7.4: Phase 2 Model Query

From Figure 7.3, it can be seen that only the ‘c’ trials provide a trend that is comparable to the best fit of the field tests. Figure 7.5 shows only the ‘c’ trial results, and from this it can be seen that Trials 6c, 8c and 9c best correlate with the field trials. Table 7.3 shows a statistical analysis of the model calibration. It can be seen that trial 9c matches the best fit data of the field tests concerning the slope, r-squared correlation and correlation to the best fit lines.

Looking at trial 9 in Error! Reference source not found., it corresponds to a Young’s modulus of 500 MPa and a Poisson’s Ratio of 0.34 for the paste. The Young’s modulus is similar to the values reported for eight paste tests performed by UBC in which the average modulus was determined to be 640 MPa. However, the Poisson’s ratio values do not agree as UBC reports a value of 0.12, but the discrepancy is most likely the nature of the paste being placed in viscous state and the lab tests being undertaken on cured samples.
Trial 9c shows best fit with field values.

Figure 7.5: Parametric Study of Paste Pour- 'c' trials

Table 7.3: Statistical Analysis of Model Calibration

<table>
<thead>
<tr>
<th>Trial</th>
<th>Field Test</th>
<th>6c</th>
<th>7c</th>
<th>9c</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope of Line</td>
<td>0.0094</td>
<td>0.0101</td>
<td>0.0115</td>
<td>0.0096</td>
</tr>
<tr>
<td>R-Squared Deviation</td>
<td>1</td>
<td>0.9997</td>
<td>0.9996</td>
<td>0.9997</td>
</tr>
<tr>
<td>Correlation to Field Test</td>
<td>1</td>
<td>0.9998</td>
<td>0.9998</td>
<td>0.9998</td>
</tr>
</tbody>
</table>

Table 7.4: Paste Parameters and Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight (kN/m$^3$)</th>
<th>UCS (MPa)</th>
<th>Friction Angle</th>
<th>Young’s Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Paste</td>
<td>18.63</td>
<td>2.0</td>
<td>17°</td>
<td>500</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Table 7.4 list the values used to calibrate the Phase$^2$ model to the results recorded in the field. Assumptions and interpretation used in deciding the paste parameters are as follows:
• Due to discretization, model results near the fence were invalid
• Effective stress was used to determine the paste loading
• Paste has a Mohr-Coulomb failure envelope and is Rankine soil during loading
• Effects of paste hydration was ignored
• Paste recipe was ignored in analysis

7.3 Fill Fence Pull Test Model

The destructive fill fence test was modeled in Phase\(^2\) to be able to be coupled with the paste loading model. The destructive fill fence model was calibrated against the field pull test of the fill fence.

The model was constructed as a 0.1 m wide by 2.4 m high fill fence similar to the construction of the field test. Loads were applied in similar manner to those of the field; that is, loads were applied over a 1 m area as a distributed load equivalent to those applied in the field. In the end, three loads were selected: 7.5 tonne/m\(^2\), 10 tonne/m\(^2\) and 11 tonne/m\(^2\). This is to calibrate the model so that it is stable under the 7.5 tonne load, commencing yielding at 10 tonne and yielding fully under an 11 tonne load as shown in the field test ().

The model was set up as shown in Figure 7.6., so that the fence would experience body forces only and thus far field stresses are ignored. The fence is defined as a Mohr-Coulomb material and the load is acting in a uniform matter over a 1 m length, 0.5 m above the floor, as was the case with the field test.
A parametric study was undertaken to calibrate the model to the behaviour recorded in the field. Since only the elastic portion of the fence was modeled, there was no analysis of the plastic parameters. The elastic modulus and Poisson’s ration were kept constant at 20 GPa and 0.15 respectively.

As mentioned above, the fence underwent three stages of loading, with the design that the fence would begin to yield under a load of 100 kPa and fully yield under a load of 110 kPa:

Stage 2: 75 kPa,
Stage 3: 100 kPa, and
Stage 4: 110 kPa.

The parametric study varied the value of the cohesion and tensile strength of the fence. The results of the parametric study are shown in Table 7.5.
Table 7.5: Calibration of Destructive Fill Fence Test

<table>
<thead>
<tr>
<th>Trial</th>
<th>Friction Angle (°)</th>
<th>Tensile Strength (kPa)</th>
<th>Cohesive Strength (kPa)</th>
<th>Stage 3 Yield</th>
<th>Stage 4 Yield</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>35</td>
<td>8500</td>
<td>21250</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>2</td>
<td>35</td>
<td>9000</td>
<td>22500</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>3</td>
<td>35</td>
<td>8750</td>
<td>21875</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>4</td>
<td>35</td>
<td>8600</td>
<td>21500</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>5</td>
<td>35</td>
<td>8700</td>
<td>21750</td>
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<td>Yes</td>
</tr>
<tr>
<td>6</td>
<td>35</td>
<td><strong>8725</strong></td>
<td><strong>21812.5</strong></td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>7</td>
<td>35</td>
<td>8740</td>
<td>21850</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>8</td>
<td>35</td>
<td>8730</td>
<td>21825</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Using the relationship of tensile strength being one tenth of the unconfined compressive strength, the ‘unconfined compressive value for the fence’ is 87.25 MPa, with the individual values for the fence being 30 MPa for shotcrete (UCS) and 250 MPa for #6 rebar (Tensile Yield Strength). As can be seen, the modeled strength of the fence is closer to that of the shotcrete than the steel.

Figure 7.7 shows the maximum strain in the fence during testing was 250 με at 100 kPa. This load and corresponding strain was not measured in any of the field tests, yet the fence fails at the same applied load as the destructive fill fence test.
The pull test model assumes that the fence will fail at the loads imposed by the fill fence test, but does generate the same stress strain path. Further, it assumes that the transition from elastic strain to plastic strain is determined to be the failure point, which was determined in the destructive fill fence test to be 100 kPa.

### 7.4 Coupled Model

The results from the paste model and the destructive fence model were combined to determine the capacity of the fence under paste loading. The coupled model was constructed in the same manner as the paste model, with all the parameters and properties being held constant and the fence parameters are those determined in Chapter 7.3.

Figure 7.8 and Figure 7.9 show the results of the model for stresses and strains respectively. The stress model shows that the maximum principal stress and the minimum principal stress values are 0.57 MPa and 0.021 MPa within the fence. This maximum value is considerably less than the 1.6 MPa at failure recorded during the pull test.
The strains shown in Figure 7.9 demonstrate that the maximum strains recorded during the coupled model are 15 με. This value of strain compared to the 250 με at failure during the pull test shows that under normal conditions the model demonstrates that the fill fence is adequately designed with respect to paste loading.
7.5 Summary of Model Analysis

The results of the modeling demonstrate that the paste behaved as a Rankine type soil before curing and that the model agrees in general with the values recorded in the field. The destructive pull test model was calibrated against the field test; the model yielded under the same values as those in the field. However, the pull test model was not a completely calibrated model, as the measured strains did not agree with those predicted by the model. This was a limitation in using the 2-D Phase$^2$ model to represent field behaviour. Further, as Phase$^2$ is a plane strain model, it does not incorporate the 3-D effects of paste loading.

When the models were coupled, it was determined that the numerically derived model does not generate enough strains based on imposed loads to cause failures for a 4 m high fence. The results of the model are one branch of the analysis that, combined with theoretical analyses, loading mechanisms of paste and stress strain behaviour, allows for an understanding of the performance of the barricade under paste loads that can ultimately be used in the safe and economical design of fill fences.
8 CONCLUSION

At the outset of this thesis, three questions needed to be addressed:

- What are the loading mechanisms of paste against barricades?

- What is the capacity of the paste fill fences at Red Lake?

- Based on the loading mechanism of the paste, do the fill fences pose a risk of failure?

These questions were answered in the research covered in the thesis and results of the research have been implemented at the Red Lake Mine for future paste pours. Since the Red Lake Mine operates at great depths under high stress, novel mining methods were developed to mine the ore body. As part of the mining method, paste backfill technology including paste and paste barricades were implemented. The current practices associated with paste, barricades were studied, and how they relate to the systems used at Red Lake. It was found from the literature search that few studies deal with paste barricade loading. Nine full size fill fences were instrumented to determine both the loading mechanisms of paste and barricade response. Analysis of the field trials was carried out using analytical and numerical methods to develop an understanding of paste loading and the capacity of the fence. The analysis was able to provide a response to the objectives of this research.

The initial loading mechanisms of the paste, where pressures are the greatest, were found to follow Rankine soil like behaviour. Figure 6.1 shows that there exists a relationship between lateral earth pressures and measured fill heights. It was determined from this relationship that the coefficient of lateral earth pressure, $K_a$, was 0.56. This value was further proven by analysis shown in Figure 6.3, where the horizontal and vertical loads of a fill fence were measured. From two different methods, the results for the coefficient of lateral earth pressure were determined to be the same. Further investigation found that the loading rate of the paste had no effect on value of $K_a$; the results of this are shown in Figure 6.2.

The capacity of the fill fence was determined from the destructive fill fence test. The destructive fill fence test was a novel approach to determining the ultimate capacity of the fence. Although ultimate failure was not reached, shows that the fence began to yield at 100 kPa or 10 tonnes of load. It was determined that in the destructive fill fence test, the yielding material and
the majority of the loads were distributed within the rebar skeleton. With a yielding value of 100 kPa and a Ka value 0.56, using Rankine Theory, the fence as constructed could withstand a paste fill height of 9.5m. These high pressures were never measured in any of the eight other instrumented fill fences.

During the eight instrumented paste pours the maximum recorded pressure was 50 kPa with an average of 26 kPa. The yielding load of the fill fence was 100 kPa. As such, under normal conditions the fill fences at Red Lake would appear to be designed for paste retention. The maximum pressures were recorded during the paste line flush at either the beginning or end of a paste pour. Failure does occur at Red Lake, as and show. The research suggests that these failures did not occur due to paste loading, but likely another factor such as poor construction or inadequate ventilation. show the stress vs. strain behaviour of all fences tested and it can be seen, on a strain based analysis, the only fence that yielded was the destructive fill fence. All other strain values are below the yielding value. This conclusion was supported by the numerical modeling, shown in Figure 7.8 and Figure 7.9. These modeled values were to approximate the behaviour of the paste loads applied to the fill fence. From this thesis it can be determined that the paste pouring used during testing did not pose a risk to mine personnel based on the yielding value of the fence determined in the destructive fill fence test.

From the research the following recommendations to the mine can be made:

- Fill pressures during backfill do not differ based on fill rate;
- Fill fence construction is suitable to the applied loads measured during the testing. No alterations to fence construction are necessary;
- Maximum pressures were recorded during paste line flushes. In order to reduce loads on fence, line flushes should be done outside of the backfilled stopes; and
- Continuous pouring is advised for underhand cut and fill stopes as it will eliminate hazards associated with ground fall due to cold joints.

The author's contribution to the existing state of the knowledge concerning paste and paste barricades is in the understanding of paste loading during initial placement together with the development of a destructive test for rebar/shotcrete fill fences and the determination of the ultimate capacity of the rebar/shotcrete fill fence. It was found that the assumption that paste acts in a hydrostatic manner was invalid and that paste has internal friction during placement
and behaves according to the Rankine Theory. The destructive fill fence test was a crucial part of the thesis as it provides a quantified value of the failure load of the rebar/shotcrete fence that can be used in design of fill barricades. Without the physical value, the results of the paste pour instruments would be meaningless, as it would not relate those values to the capacity of the fence. It is recommended that the destructive fill fence test be carried out at all operations to commence building a database of fence strengths. The destructive fill fence test demonstrated that the majority of the strain was occurring within the rebar and that the failure was initiated in the rebar element. A yielding value of 100 kPa was determined for the fill fence through this research. This parameter has not been reported in prior literature.

This thesis provides an understanding of the paste loading, fill fence interaction with respect to failure.
9 RECOMMENDATIONS FOR FURTHER WORK

This thesis aimed to provide a thorough treatise on the interaction of paste with shotcrete fill fences at Red Lake Mine. From the analysis presented, design guidelines were developed to assist Red Lake Mine in increasing their production due to faster turn-around times for stope backfilling. However, during the research certain items emerged that require future consideration. Issues that were apparent were limitations in the instrumentation program, variations in fence construction, safety issues during paste pouring, as well as the influence of fill rate and paste constituents on loading and design of numerical models.

It was apparent that the instruments used in the analysis require some modifications to be better suited for this type of monitoring. The instruments used during the testing were coarse in comparison to the small loads imposed on the fence. With a maximum measured paste pressure of 40 kPa, the earth pressure cells’ maximum measurement of 2000 kPa is considered too large to accurately measure the imposed loads. Further, the strain gauges measured both tension and compressive strains. However, since their location with respect the depth of the fence was unknown it became difficult to make a confident conclusion on the strain behaviour of the fence. In future tests, the location of the strain gauges with respect to the depth of embedment within the shotcrete should be measured. This would allow a better understanding and a proper differentiation between compressive and tensile induced strain. In addition, strain gauges should be installed in the center of the panel as opposed to near the rebar, to reduce the influence of the rebar skeleton on the concrete strain gauges. In general, with the use of instruments for future fill fence monitoring, proper planning and implementation of the program needs be undertaken to ensure that instruments used are able to measure the range of loads and that instruments are monitoring the intended phenomena.

The majority of the study focused on a certain type of fence construction. It would be worthy of future study to determine the capacity of various other fences used at the Red Lake Mine including arched fences and cable-sling fences employed at the adjacent Campbell property. Further testing and a thorough analysis of fence designs should be carried out to determine the benefits of tying in the back and the floor to the fill fence with embedded rebar. These tests could either be of performance of the fence during paste pours or as a destructive fill fence test depending on the need of the design. In addition to fence construction, monitoring of paste pours should be carried out to determine the relation of the percentage of cement within
the paste and the loading rate of placement to the coefficient of lateral earth pressure. The purpose of the recommended future work is to determine a better understanding of the paste barricades and to limit the hazards associated with the paste placement.

As far as numerical modeling is concerned, the behaviour of paste was accurately measured within the thesis. However, the numerical model only approximated the behaviour of the fill fence; a more complex numerical model is necessary to develop an understanding of the interaction between the rebar and shotcrete under the paste loads.

Studies in Australia by Revell and Sainsbury (2007a) identified interesting passive solutions to reduce the risk associated with barricade failures. These included rock berms situated a distance away from the barricade, which in the event of a failure would reduce the momentum of paste limiting the surge of paste flow into the mine workings. Another approach was creating no-man entry zones around a paste barricade to limit the risk of worker injury within the stope. Both mitigation measures should be investigated to their application at Red Lake Mine.

Further testing of fill fences is warranted to understand the variables discussed previously. The goal of future research should be to have a better understanding of the workplace in an effort to improve safety and mine cycle time.
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