GEOTECHNICAL STUDY ON THE FEASIBILITY OF USING BACKFILL IN STEEP DIP AREAS OF KONKOLA COPPER MINES (ZAMBIA)

By

Dzimunya Nevaid

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The University of Zambia

School of Mines

Mining Engineering Department

Lusaka

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Declaration

I, Dzimunya Nevaid, do hereby declare with all honest that the work presented in this dissertation is my original research work and that all quotes and citation of other authors work have been duly referenced and acknowledged. I also declare that no part of this dissertation has been presented or published in pursuit of another degree in this University or any other academic institution.

I, therefore, declare that this dissertation was written according to the rules and regulations governing the award of Master of Mineral Sciences (M.Min.Sc) degree of the University of Zambia.

Signature of author……………………………………………………………..

Date………………………………………………………………………

Certificate of Approval

This dissertation by Dzimunya Nevaid has been approved as fulfilling the requirements for the award of the Degree of the Master of Mineral Sciences by the University of Zambia.

	Name:	Signature:	Date:
Supervisor	…………………	……………….	……………
Internal Examiner (1)	…………………	……………….	…………….
Internal Examiner (2)	…………………	……………….	…………….
Internal Examiner (3)	…………………	……………….	…………….

Dedication

To my beloved family and my future wife all this effort was for you.

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Abstract

Backfill is described as waste material that is used to refill the underground voids left during mining for technical or safety purposes. Backfilling has several applications underground and the use of backfill has increasingly become a fundamental part in the entire underground mining activities all over the world. This has greatly been accelerated by the need to increase ore production as well as maintaining regional stability around mine operations. The ever increasing depth of mining operations has largely increased the use of backfill material in underground operations to allow efficient and safe extraction of the mineral deposits. The need for Konkola Copper Mines (KCM) to develop additional production areas further below its current operations has resulted in the sinking of No 4 shaft to a depth of 1500m below surface. Mining at increased depth may result into increased levels of induced stresses hence this may negatively impact on the stability of the stopes and ultimately can lead to interrupted production. The purpose of this research was to evaluate geotechnically the behaviour of the current and future stopes at increased depth assuming no backfilling is performed and to determine the depth at which backfill may be needed. Numerical modelling using Phase2 computer software and Mathew's stability graph method was used for the purpose of evaluating, geotechnically, the response of the stopes at different mining depths. Geotechnical investigations was done through core logging, underground structural mapping and laboratory rock tests to determine input parameters into the two evaluation techniques. Numerical modelling and empirical analysis have both indicated that stopes are stable but around 1250mL and below there are chances of increased and excessive instability problems which can lead to increased dilution and risk of collapse. Suggested improvements to reduce costs and improve copper ore production include the increasing of stope strike length from 20m to 30m on 950mL – 1050mL. Stopes are stable at this new length and the option improved overall ore production in the Bancroft area of the mine by 7.14% and a possible reduction in development costs on slot raises and draw-point cross-cuts by 28.6%. Mining with Cemented Hydraulic Fill (CHF) from 1250mL and below within the steep dipping areas (Bancroft areas) has proved to be very economic as gross revenue after an economic analysis increased by 15.5% as compared to leaving substantial amount of ore in stabilising pillars. With CHF, dilution can also be reduced although not quantified in this research.

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Abbreviations Acronyms

Abbreviation	Description
KCM	Konkola Copper Mines
KDMP	Konkola Deeps Mining Project
CHF	Cemented Hydraulic Fill
CAF	Cemented Aggregate Fill
PF	Paste Fill
HF	Hydraulic Fill
FWQ	Footwall Quartzite
FWSST	Footwall Sandstone
AGSST	Argillaceous Sandstone
PC	Porous Conglomerate
FWC	Footwall Conglomerate
HWQ	Hanging wall Quartzite
SLOS	Sublevel Open Stoping
LHOS	Longhole Open Stoping
PPCF	Post Pillar Cut and Fill
MOCB	Modified Overcut Bench
LOM	Life of Mine
HR	Hydraulic Radius
NGI	Norwegian Geotechnical Institute
SRF	Stress Reduction Factor
GSI	Geological Strength Index
RQD	Rock Quality Designation

RMR	Rock Mass Rating
UCS	Uniaxial Compressive Strength
CMS	Cavity Monitoring System
DTM	Digital Terrain Model
CP	Copper Piloting
FEM	Finite Element Method
FDM	Finite Difference Method
BEM	Boundary Element Method
DEM	Discrete Element Method
Mtpa	Million tonnes per annum
EBITDA	Earnings before Income Tax, Depreciation and Amortization

List of Symbols

Symbol	Description
γ	Unit Weight
k	Horizontal to vertical stress ratio
N'	Modified stability number
S	Shape factor of hydraulic radius
J_n	Joint set number
J_r	Joint roughness number
J_a	Joint alteration number
J_w	Joint water reduction factor
v	Poisson's ratio
E	Young modulus
σ_{ci}	Uniaxial compressive strength
σ_1	Major Principal stress
σ_2	Intermediate principal stress
σ_3	Minor principal stress
σ_h	Horizontal stress
σ_v	Vertical stress
σ_n	Normal Stress
E_h	Average deformation modulus of the upper part of the earth crust

CHAPTER 1 INTRODUCTION

1.1 Preamble

The goal of mining is to extract valuable minerals from the earth's crust and this process widely results in the formation of voids both underground and on the surface. Filling of these voids is usually done and the main function of the fill material in mines is to assist in controlling the stability of mine related voids (Potvin, et al., 2005). The use of different types of mine fill and their functions and ultimately the engineering requirements are closely related to the mining methods and mining sequence. Generally mine fill is applicable to artificially supported mining methods and the main functions of fill in artificially supported methods includes ensuring long term regional stability, limiting excavation exposure and also waste disposal.

Many factors, such as rock mass condition, stope geometry, in-situ stress condition, blasting, stope exposure time and geological structures influence stope stability and dilution (Jucheng, 2004)**.** Since backfilling of stopes can help in arresting instability, it is vital to assess the stability of mine stopes and openings underground to identify if backfilling is necessary. In underground mining operations, the mined-out spaces need to be backfilled to maintain the stability of surrounding rock mass and increase the ore recovery (Liung & Mamadou, 2015)**.** This research will mainly consider the effect of rock mass condition, stope geometry and in-situ stress on stability of the stopes to aid on backfilling of the mining voids.

1.2 Description and location of research site

KCM is a copper mine situated in Chililabombwe, nearly 25 km north of Chingola. KCM is currently exploiting the Chililabombwe orebody by underground operations. Figure 1.1 shows the mine location in relation to the entire map of Zambia.

Figure 1.1 Location of Konkola Copper Mines (KCM) on the Zambia map

1.2.1 Geology and geotechnical conditions

The orebody at Konkola takes the form of an anticline and it has a strike length of nearly 11km. On the other hand, the dip of the orebody varies between 15˚ and 65˚ and the average dip is 25˚ at No.3 shaft and 55˚ at No.1 shaft production areas. The thickness of the orebody varies between 3m-12m and the average is approximately 7m-8m. The mining operations are currently at 950m depth at No.1 shaft and 590m at No.3 shaft and another shaft, No.4, reaching to 1500m has been sunk to get access to mineralisation at greater depth.

All mine developments at No.1 Shaft are mined within five main rock formations. These are, Footwall Quartzite (FWQ), Argillaceous Sandstone (AGSST), Porous Conglomerate (PC), Footwall Sandstone (FWSST), Footwall Conglomerate (FWC), Oreshale and the Hanging wall Quartzite (HWQ).

The ore shale units at Konkola are blocky and well-bedded, oftenly with significant clay infill in the defects therefore making the orebody relatively soft as compared to the host rock. The lowest unit of the orebody, Unit A, is generally extremely oxidised and it grades from very weak rock to clay. In other areas of the orebody, there is a koalinized band around 2m above the geological hanging wall contact. This feature is assumed to adversely affect the hanging wall stability.

Structurally, the ore body is divided into five units. At the bottom is the finely banded dolomitic, calcareous sandstone, in places highly weathered to a brown micaceous clay, ‘A’ Unit varying in width from 0.3m to 1.0m and rock mass rating of 21-40. In some places the rock mass rating is less than 20. Above the ‘A’ Unit is the 1.0-1.5m thickly bedded, ‘B’ Unit. The ‘C’ and ‘D’ Units comprising inter-bedded strong siltstone and dolomite bands, lie below the strong siliceous ‘E’ Unit within which the Assay hanging wall usually lies. The hanging wall formation consists of quartzite and dolomitic sandstone bands, which in some places completely koalinized, resulting into poor hanging wall condition.

There are also three main aquifers at Konkola and also the geo-hydrological setting at the mine results in high inflows of water into the mine operation pumping nearly 300 000m^3 of water daily. These three aquifers are namely:

- ❖ Footwall Quartzite Aquifer
- ❖ Footwall Aquifer
- ❖ Hanging wall Aquifer

1.3 Background

KCM currently produces approximately 2.0 Mtpa of ore from the three existing underground operating units namely No. 1 shaft, No. 3 shaft and No. 4 shaft. Other operating units of the mine include a number of ventilation shafts, pipe shafts and also the Konkola West and East concentrators.

The mine is in the process of developing other additional areas of production and undertaking key projects to extend the life of the mine and to allow an increase in copper production that is aimed at producing 7.5 Mtpa over the coming five years. Among these projects is the Konkola Deeps Mining Project (KDMP) which is targeted at expanding copper ore production by accessing the ore body that lies further down underneath what the current operations are currently mining. This then involved the sinking of the No. 4 shaft which reaches a depth of 1500m and the commissioning of the 6.0 Mtpa concentrator to enhance mining output and recovery.

KCM has therefore engaged several consultants during the past decade in order to undertake several work to meet its targets and plans of ramping up production. Examples include AMC consultants which was engaged to undertake a backfill strategy study for the KDMP project at Konkola mine and also Paterson & Cooke for the design and piping requirements for the KDMP underground hydraulic backfill reticulation systems. The ramp-up for production at Konkola therefore required expansion of the underground production areas since the current concentrator and No. 4 hoisting shaft both have the design capacities that can meet the target production.

Due to the shape and outline of the orebody, the mine uses a number of mining methods. These methods include:

- Sublevel open stoping (SLOS)
- Long-hole open-stoping (LHOS)
- Post pillar cut and fill (PPCF)
- Modified over-cut benches (MOCB)
- Continuous retreat up dip etc.

However, the conclusions and opinions from the consultant work was for the notion that the mine should adopt mining with backfill in order to break up the resource into several mining blocks so as to maximise resource extraction by removing the need for stability pillars. This is because, currently, the mining operations at KCM have reached critical depths shown by the ever increasing induced mining stresses posing challenging ground conditions that require stabilisation. Reports from different departments have shown that recovery has been low due to dilution and therefore the consulting companies have strongly advised KCM to carry out a mine-wide systematic backfill operation into stoping activities.

This research was therefore initiated by KCM to establish the need of backfilling in the steep dipping areas and also depending on the mining method used to clarify what would be the behaviour of the stopes assuming that the mine does not carry out any backfilling in these areas of the underground operations. It is the aim of this proposed research also to show at what point, in terms of depth, the stopes will pose a risk to the surrounding mining activities if not backfilled. Therefore, it is the scope of this research work to ascertain the need for backfilling in the steep dipping areas of the mine.

1.4 Statement of the problem

The KDMP project at the Konkola mine will require operations to extend further to greater depths (1500m below surface) and hence the need for an extensive use of systematic backfill of the stopes. Konkola has been currently facing poor ground conditions that ultimately make the mining process challenging and dangerous. This ground conditions will therefore affect stable stope spans, sequence of mining and ground support practises. However, ore recovery will also be reduced from around 80% by prompting the mine to leave larger rib pillars (> 5m) of ore for stabilisation of the stopes as depth of mining operations increases.

This research was therefore initiated by KCM to establish the necessity of backfilling in the steep dipping areas and also depending on the mining method used to clarify what would be the behaviour of the hanging wall, footwall and pillars assuming that the mine does not carry out any backfilling.

1.5 Study objectives

In order to accomplish this research the following objectives are to be achieved:

Main objective

The principal objective of this study was to perform a detailed geotechnical study to determine if backfilling is necessary in the steep dipping areas of Kongola Copper Mine (KCM).

Sub-objectives

The corresponding sub-objectives of this study are to:

1. Model the stress distributions and displacement around the mine openings using numerical methods (Phase2).
2. Evaluate the zone of stability of the individual stopes at different depths using the empirical approach (Mathew Stability graph).
3. Compare the outcomes of the two methods and make recommendations.
4. Optimise ore extraction by selecting possible backfill material types that can be used at the mine in case modelling and empirical analysis prove the need to backfill the stopes.

1.6 Research questions

In order to accomplish this research work, the following research questions were born in mind:

1. Are the stopes in the steep dipping areas stable without any filling
2. Is there any long term effects if the stopes are not backfilled?
3. If backfilling is needed, what are the estimated quantities, strengths, and at what point in terms of mining depth should the filling be recommended?

4. Is sequencing of stopes mining an option?

1.7 Methodology

In order to achieve the above set objectives, the methodology used followed a systematic way of reviewing literature as well as collecting data from the mine that was used in the numerical modelling and Mathew's stability graph method. Stopes from different levels (950mL, 1050mL, 1150mL, 1250mL and 1350mL) were modelled using Phase2 computer software to analyse the stresses and displacements within the stopes as depth of mining increased. The behaviour of these stopes was used to make conclusions concerning the approximate depths of instability as mining depth increased.

On the other hand, the Mathew's stability graph method was used to determine the different zones of stability that these stopes at different mining depths can plot on. In this method, the stability number was determined following the procedures described by (Potvin, 1988). This stability number was plotted against the hydraulic radius of each stope on the Stability graph. Each zone of stability for the individual stopes was noted and discussed for the purpose of drawing conclusions.

Laboratory testing for uniaxial compressive strength of intact rock samples as well as core logging and structural mapping was used to evaluate the parameters that were needed in the evaluation techniques.

The outcomes of the two techniques was compared to each other in order that the approximate depth of instability can be suggested. Backfilling of the stopes located in the unstable zones was recommended on the bases of the outcomes of these evaluation methods and the backfill material suggested was that already analysed by KCM. The full methodology used in this research if discussed in detail under Chapter 3.

1.8 Significance of the study

A better understanding of the stope behaviour (hanging wall, footwall and pillars) will definitely go a long way in the ultimate allocation of the backfill resources and also it helps the mine to minimise the possible unnecessary cost of backfilling. However, the outcome can also contribute to the proper stabilization of the underground openings. More importantly, the research will facilitate the execution of the proposed primary/secondary ore extraction system which will obviously improve the copper ore production as the number of stabilising pillars will be reduced or the size of the pillars reduced hence helping the mine to achieve its target of ramping up production.

Since production is to be performed at greater depths where high induced stresses will be experienced, modelling of the stopes to determine depth at which backfilling should commence will contribute to efficiency of mining operations and it will also permit the accessing of high grade of copper ore which from exploration is deeper than the current levels of operation. The need to ascertain backfilling can also contribute towards proper waste management hence better environmental management efforts as little waste will find its way to the Lubengele tailings Dam.

1.9 Scope of study

The research work is focussing on the backfill needs for the current and future mining methods in the steep dipping areas of KCM. Steep dipping areas for this study is the part of the ore body exceeding 45 degrees (dip angle). These areas include Bancroft deeps, Bancroft central and Bancroft north. Stopes modelled are those from the Bancroft deeps which is from 0mS to 1000mS. The deeps are also selected because it is this portion of the Konkola deposit with the steepest outline of the orebody. However, the effect of stresses was much anticipated from this section because it has the greatest depth so far on the Konkola mine (1040mL).

The study of the backfill needs will include stope dimensions, backfill strength and timing requirements, exposure and geotechnical characteristics of the rock.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

This Chapter serves to highlight most of the relevant information related to this study. The contents in this section is a condensation of many theories by different authors, scholars and researchers internationally and as well as those at the mine. These theories have been used, in this Chapter, as basis to explain the processes and practices involved in stope stability analysis and backfilling of mine voids in underground operations.

2.2 Backfill and its relevance

In underground mine operations, backfill is generally described as waste material that is used to refill the underground voids left during mining for mining technical or mining safety purposes. Backfill is applied in order to prevent fires and explosions, to improve mine ventilation, to improve long-term strata stability of the rock, to reduce subsidence effects at the surface, as well as for economic and environmental factors associated with waste disposal. Underground Mining with backfill applications enables mining companies to achieve many of these goals. The technology of backfilling enables a wide range of engineering solutions for particular mine sites and their unique sets of problems and opportunities. Therefore, carefully engineered and properly operated backfill system can significantly improve the quality of a mining operation.

Normally when backfill material is used as engineering material, it must be of sufficient strength to be later exposed when the pillars are exploited as ore especially in tall vertical faces of the stopes. So there is need to better understand the fill material and properties of the materials that can be used to produce the fill material. Backfill material generally has a lower strength than the surrounding rock mass and therefore backfill material is mostly used to reduce the relaxation of the rock mass so that the rock will be able to carry further load (Mutawa, 2011). Researches has shown that backfill material permits arching within itself and this can be exploited to improve the utilisation of the mining reserve and hence improve the overall economics of the mining operations.

Mine fill is generally applicable to artificially supported mining methods (Potvin, et al, 2005). Within these methods, a number of backfill applications has been used to satisfy several engineering goals. Therefore in the artificially supported methods, as are mostly employed at Konkola mine, the main function of fill material is explained in the following subtopics.

2.2.1 Ensuring long-term regional stability

The stability of underground openings is dependent on several variables such as overall ground conditions, spans and time. With this in consideration, it is likely that large excavations that are indefinitely left open after mining have an increased risk of collapsing with time. There have been a number of cases where fill was not applied and the remaining crown pillars have failed up to the surface with a variety of consequences (Potvin, 2005).

Mine fill can therefore be used to mitigate this risk. The fill material acts as a bulking agent and its prime purpose is thus to occupy the mining void. The rock mass failure process is therefore arrested in case the stope or excavation becomes unstable and the loosening material is kept in place. This is accomplished by preserving some of the confining forces in within the rock mass, which usually increase as the distance away from the excavation boundary increases. The increased confinement can therefore promote stability in two ways:

1. In the jointed rockmass it prevents the opening of the discontinuities and joints. This allows the friction along these planes of weakness to stabilize and as a result the shear strength of the rockmass is maintained and the propagation of the failure is arrested.
2. It limits the amount of stope wall convergence which will ultimately have a positive impact on the regional stability of the mine.

Generally there is need to make careful considerations in selecting the fill material taking into account whether the fill material will be exposed to other voids or not. Other issues such as the potential for chemical reactions with the surrounding environment and the potential for liquefaction in case of an unforeseen collapse breaking into the fill material must also be considered in the process of selecting bulk filling material and whether a binder material will be needed.

2.2.2 Limiting excavation exposure

In a given set of ground and operating conditions, underground openings that are smaller than a certain critical dimension will remain stable for the duration of the mining operations. Fill material is therefore used to limit the exposure of stope walls and also the back of the underground excavation.

Orebodies are generally divided into stopes or lifts so that they can be sequentially extracted and then filled. For extraction operations progressing upwards, the fill material is usually used as a working platform for the stope above and alternatively when mining operations are progressing downwards, the fill material is used as a replacement roof. This is normally applicable where the ground conditions in the back of the stope are regarded as unsafe and the option of leaving crown pillars demarcating stopes is dismissed.

In order to limit the stope exposure, a variety of strategies can be adopted and these involve the use of uncemented fill and others cemented fill. The use of cemented fill to limit the excavation exposure can lead to productive as well as flexible sequences especially when exploiting large deposits with reasonable grade. In this operation the entire orebody can be divided into stopes and stopes can be in pre-production (drilling) while others can be in post-production (filling). In this application, the essential engineering property of the fill is its ability to stand after being exposed to the voids opened when adjacent stopes are excavated for example 200m vertical fill walls have been successfully exposed in Mount Isa Mines (Potvin, 2005).

The in situ stress regime is generally high in deep mining operations and under these conditions the sequencing of stope extraction can be used as one of the main strategic control measures for controlling the effect of mine induced stress concentrations. This phenomenon can therefore be an essential tool if critically analysed as the current mining operations at KCM has now reached critical depths as evidenced by the increasing mining-induced stresses that are believed to be creating challenging ground conditions at the mine.

Experience from mining at greater depth has indicated that leaving significant size pillars is the best method of minimising areas of high stress concentrations and therefore lowering the potential for rockburst and rock fall related problems (Potvin, 2005). To demonstrate this phenomenon, numerical modelling and stress analysis softwares can be used.

In a deep and steep tabular orebody, for example, ideal mining sequences will have no pillars and can involve starting with a single stope at the centre and then progressing to the adjacent stopes in a manner that will allow the whole excavation to expand outwards in and inverted "V" outline. Possibly, the initial stope could be situated at either corners of the orebody, progressing the mining by extracting the adjacent stopes diagonally and this can also result in the elimination of in situ pillars and hence increasing on resource utilisation.

2.2.3 Waste disposal

The cost associated with constructing and maintaining infrastructure of surface storage for mine waste have significantly increased in recent years. This is mainly a result of strict environmental standards and mine closure requirements that are transforming the economics of mine waste storage and disposal. Therefore considering the necessity of filling mine voids in increasingly perceived as an environmentally friendly vehicle as well as a cost cutting measure to permanent disposal of mine waste.

2.3 Open stope mining practices

There are several mining methods that can be used for the extraction of underground deposits and among them is open stoping which has been used in many mines around the world. Open stoping has become one of the most popular method in underground mining because it is safe by virtue of design and relatively cost efficient. The method is very development intensive, although the cost of development is finally compensated by the fact that much of it is done in ore. Efficient use of large-scale blasting makes open stoping method one of the lowest-cost underground mining methods available (Hartman, 1992). Several variations of open stoping have evolved due to modern technology such as improved drilling and blasting techniques and new mining equipment.

2.3.1 Definition of open stope mining

Open stoping mining is a technique that utilises gravity for flow of broken ore to the stope bottom. It is generally applicable to steeply dipping orebodies and the stopes should have a dip greater than the angle of repose of the broken ore material. Sublevel stoping is illustrated in Figure 2.1 and generally there are three characteristic features that differentiate open stope mining from the other underground methods:

1. It is a non-entry method i.e after production has commenced, the open stope does not need mine workers to get access into it.
2. Until final stope dimensions are obtained, the stopes are kept open. Broken ore is removed as stope extraction progresses. Backfill material for stabilisation is only provided after the whole stope is exploited (delayed backfill).
3. The open stopes are designed to maintain stability as opposed to caving methods.

The method is not selective and therefore the orebody outline must be considerably regular. However, a minimum orebody width of 5m is considerable to avoid excessive dilution from wall damage caused by drillhole deviations or blasting vibrations. In addition, fair to good rock mass strength is required in the stope back and walls since there is no major support system employed during the mining process. Therefore the more competent the host rock mass, the larger the open stopes that can be operated, and the more efficient the operation will be.

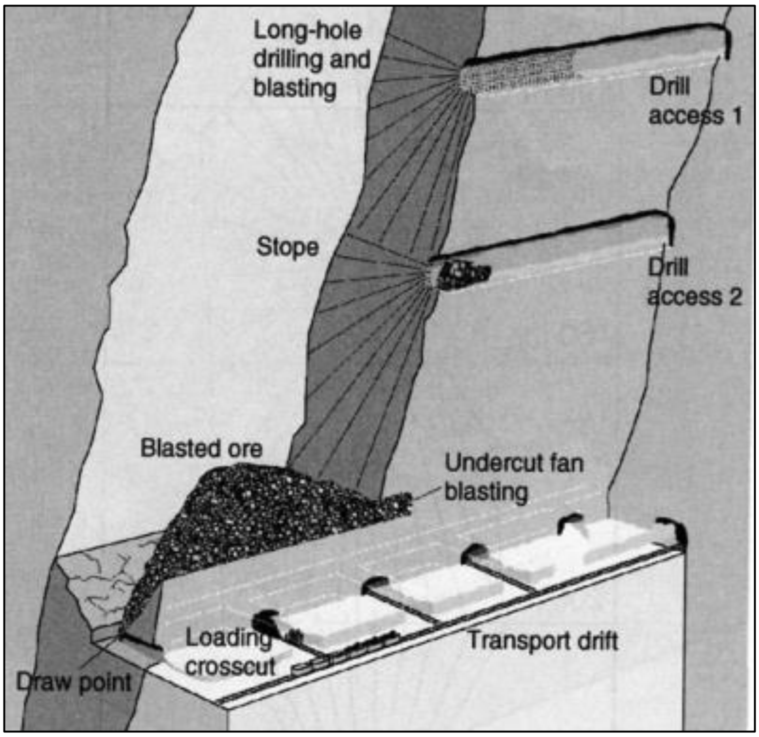


Figure 2.1 Sublevel open stoping

2.3.2 Classification of open stope mining methods

A suggested classification includes the following characteristics in order to group the open stope mining methods (Potvin, 1988):

- Mining direction (transverse and longitudinal)
- Use of pillars and mine backfill
- Drill hore diameter (blasthole or longhole stoping)

These specifications have been selected to group open stope mining methods, because they firmly decide the stope development and preparation needed, the sequence of ore extraction and method of retreat.

2.3.2.1 Mining direction

Direction of mining or extraction is the first characteristic that is used in the classification of open stoping methods. In longitudinal ore extraction, the direction of retreat and production headings are driven along the strike of the ore body. It is usually applicable where the orebody is narrow. In case of direction of retreat being vertical as in crater retreat, the longest horizontal axis is used to describe the direction of mining. This type of variant generally is faster and less expensive to bring into production as it requires less development and the major drawback is the ability of the stope backs to be self-supporting.

However, for wider or thicker orebodies, the transverse open stoping mining is used with the direction of retreat and production headings being driven across the orebody from footwall to hanging wall. The stopes have their longest horizontal axis across the orebody. Transverse mining is usually practiced on wider orebodies with a minimum width approximately 15m to make the variant more efficient.

2.3.2.2 Use of pillars and mine backfill

In relation to use of pillars and mine backfill, the open stoping methods have four options described as follows:

- Full lens mining without pillars or mine backfill,
- Permanent mine pillars that will not be mined are left with no backfill (this will reduce extraction ratio),
- Pillar recovery using cemented backfill,
- Avoiding mine pillars by employing stope sequencing and backfill operations.

Full lens mining is applicable where the orebody thickness is relatively small (ore lenses) and its desirable open stoping conditions exist when the host rock is strong enough to exclude the necessity of pillars or backfill. Potvin, 1988 plotted the wall rock strength (NGI, Q index) against the wall rock area from several cases and provided a rough criterion for the feasibility of the full lens stoping method.

In most cases, full lens stoping is not applicable and therefore stabilising pillars are left during the ore extraction process. These pillars can be recovered at a later stage but this whole process sorely depends on the ore grade and value and where the ore value in the pillars is justifiably high then use of backfill to recover the pillars is reasonable. Permanent mine pillars are left if ore value is relatively low and maximum extraction ratios of approximately 75% to 80% are realised (Potvin, 1988).

On the other hand, in high stressed ground conditions where risks of rock bursting are high and pillar recoveries expensive, sequencing of stope extraction and backfill operations can be used to exclude pillars. Stope filling can immediately commence after mining is finished and the stope directly adjacent to the filled stope is mined as soon as the fill material has set. The success of this process is dependent on the effectiveness of the filling cycle and another disadvantage of it is lower initial rate of return.

2.3.2.3 Drillhole diameter (blasthole and longhole stoping)

For production blasting, initially all the open stoping methods used small drillhole diameters (51 to 64 mm). However, the large diameter drill holes generally used in surface operations was introduced in underground production drilling. Blasthole open stoping is highly mechanised than the small diameter longhole stoping and therefore it is more cost effective and productive. On the other hand, blasthole open stoping is generally less selective and thus requires larger and more extensive development before actual production.

2.3.3 Open stope mining and stope sequencing

One of the limiting factors influencing the design of an underground opening is the maximum unsupported span that a rockmass can withstand without failure. This failure may occur usually as a function of either movement along discontinuities, or through a combination of intact rock failures and geological discontinuities. In most orebodies amenable to open stoping mining methods, the volume that may be safely excavated with stable excavation walls is many times smaller than the orebody itself (Villaescusa, 2003). As a result, a series of individual stopes should be excavated to accomplish full orebody extraction.

One of the most vitals tools that a mine design and planning engineer has for managing the overall behaviour of a rockmass is the extraction sequence of the stopes contained within a given block of an orebody. Therefore extraction sequencing of the stopes is fundamental to safely and economically achieve production requirements throughout a stoping life.

In situations where mine fill is not used, the main strategy is stope sequencing such that early over stressing of permanent pillars is prevented. However, there is a number of extraction strategies that can be followed to optimize pillar recovery in case mine fill is used. In general a stoping sequence is influenced by factors such as ore grade requirements, operational technicalities which includes existing development, backfill availability and induced stress considerations. A technically sound strategy is to avoid creating blocks of highly stressed rock mass within an orebody. This can be accomplished by retreating stopes systematically from the centre towards the orebody abutment as opposed to creating mine pillars that are located within central orebody zones. This is done by taking into account the stress re-distributions, production tonnages requirements and access constraints.

In general, an overall stope extraction sequence is influenced by the nature of the orebody in question (Villaescusa, 2003). There are many variants of stope sequencing and one example is pillarless, centre-out sequencing which was proposed to eliminate the need for secondary stopes (Morrison, 1995). In this variant, there is a slow rate of convergence of the host rocks as extraction from small stopes proceeds towards the orebody abutments from the centre as illustrated in Figure 2.2. The sequencing method is said to minimise the magnitude of any local seismic events as well as reduce the amount of released seismic energy.

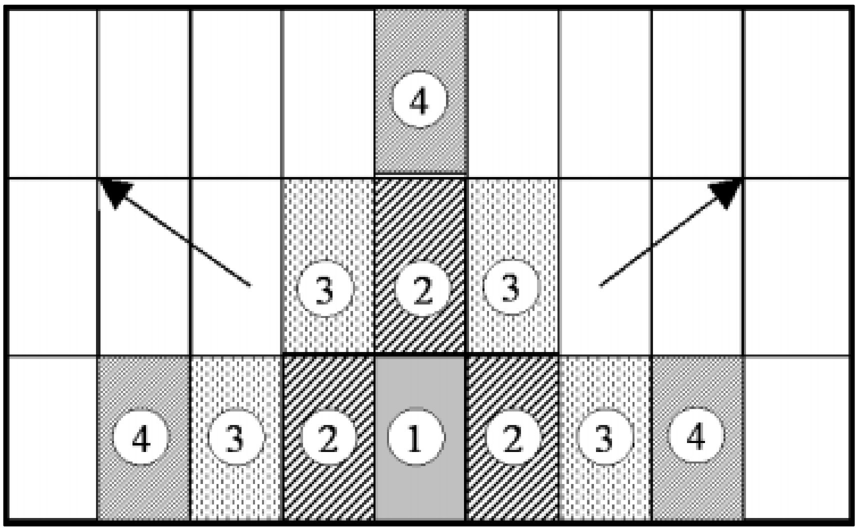


Figure 2.2 Pillarless mining sequence, centre out extraction (Morrison, 1995)

The most common stope extraction and sequencing procedures can be divided into two categories which are; methods that use backfill and those that do not use any backfilling.

2.3.3.1 Open stoping with no backfill

As previously discussed under Section 2.3.2.2, it has been clearly noted that the most simple and economic open stope extraction method is the full lens mining. This is mainly attributed to the exclusion of mining pillars during the extraction process. Generally this method is applicable to relatively thin orebodies or isolated lens provided the country rock mass quality can allow for stope walls to be self-supporting. Another variation of this approach is the open stope retreat method. In this approach there are no pillars or backfill required (Figure 2.3) and generally the method of retreat is underhand (top to bottom) instead of overhand (bottom to top).

When the orebody thickness is too large to be extracted by means of a single stope, permanent pillars are left. This is usually viable in cases where the ore value does not justify the use of mine fill. In this case permanent stope access entries are located within the stabilising pillars and also the stopes extraction advances towards the pillars. This is indicated in Figure 2.4. The design engineers should try by all means to minimise the pillar width in order to maximise the orebody extraction ratio but at the same time maintaining mine stability.

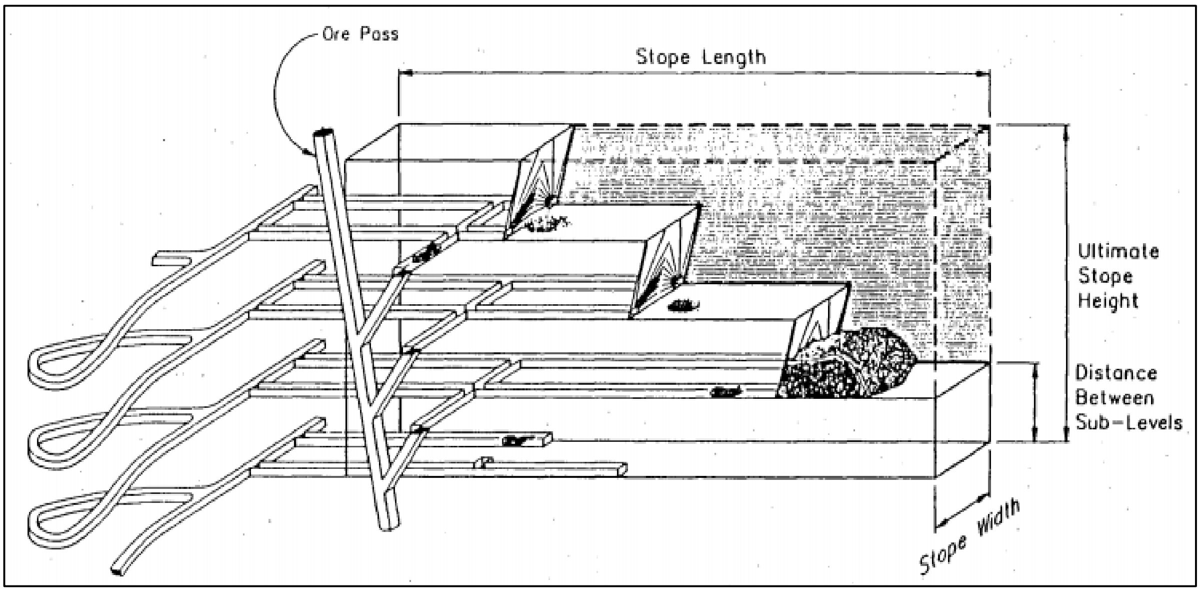


Figure 2.3 Idealised isometric drawing of sub-level retreat open stoping method (after Potvin, 1988)

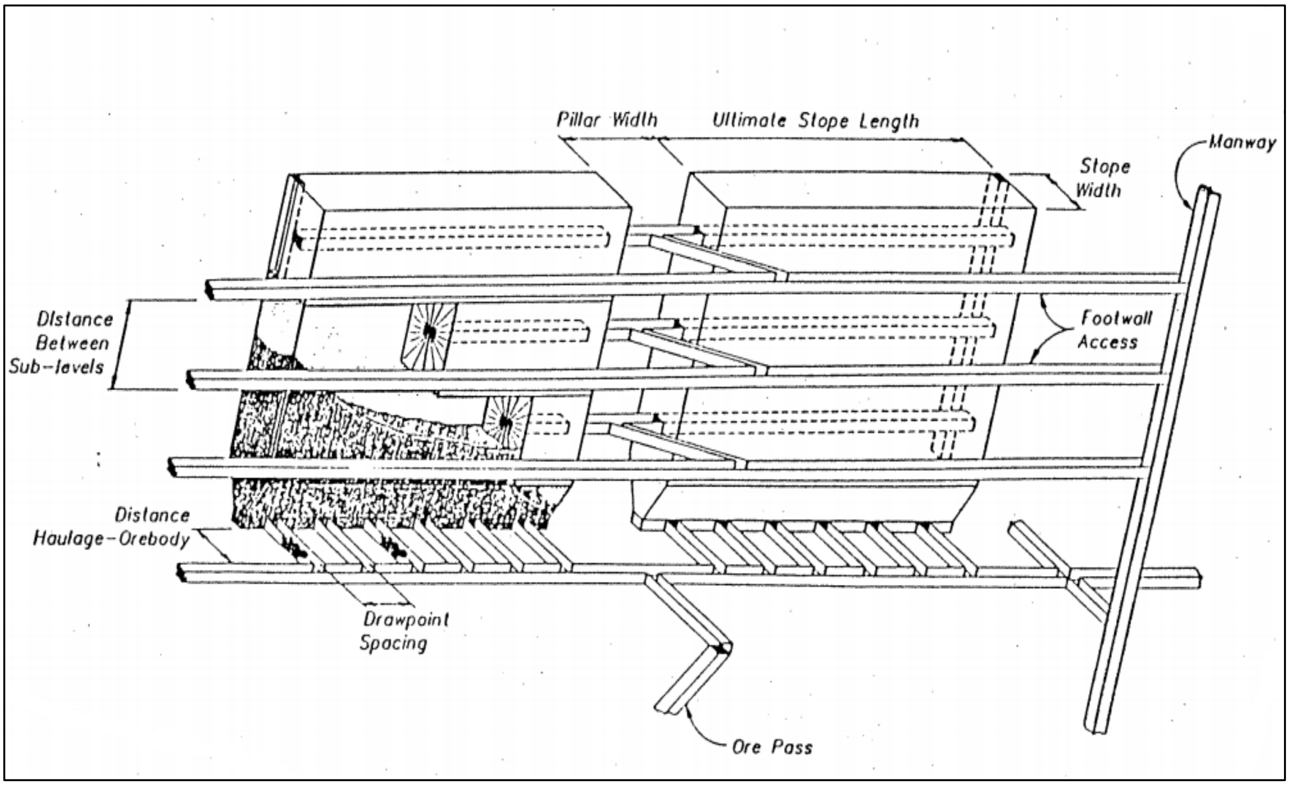


Figure 2.4 Idealised drawing of the longitudinal open stop with permanent pillars (after Potvin, 1988)

2.3.3.2 Open stoping with backfill

When ore extraction involves backfilling, the stoping activity and sequencing becomes a part of a systematic strategy to allow for optimum extraction of the ore body secondary and tertiary stopes. These secondary and primary stopes are a result of temporary pillars left when primary stopes are mined out. Primary stopes are therefore mined against rock walls. Secondary stopes are normally mined against one or more cemented fill walls and then tertiary stopes are encompassed by backfilled stopes. Usually primary and secondary stopes require cemented backfill, while uncemented fill is normally used to fill the tertiary stopes.

The commonly used mining sequence is called the “leap frog” and it alternates between extracting a stope and leaving the next stope as a temporary pillar as shown in Figure 2.5. In this approach, the order of stope mining is variable and is dependent on factors such as drill scheduling, development as well as availability of backfill material. Mining can be localised in one level to minimise drill equipment movement provided there is adequate number of stopes otherwise primary and tertiary stoping are simultaneously done on several levels.

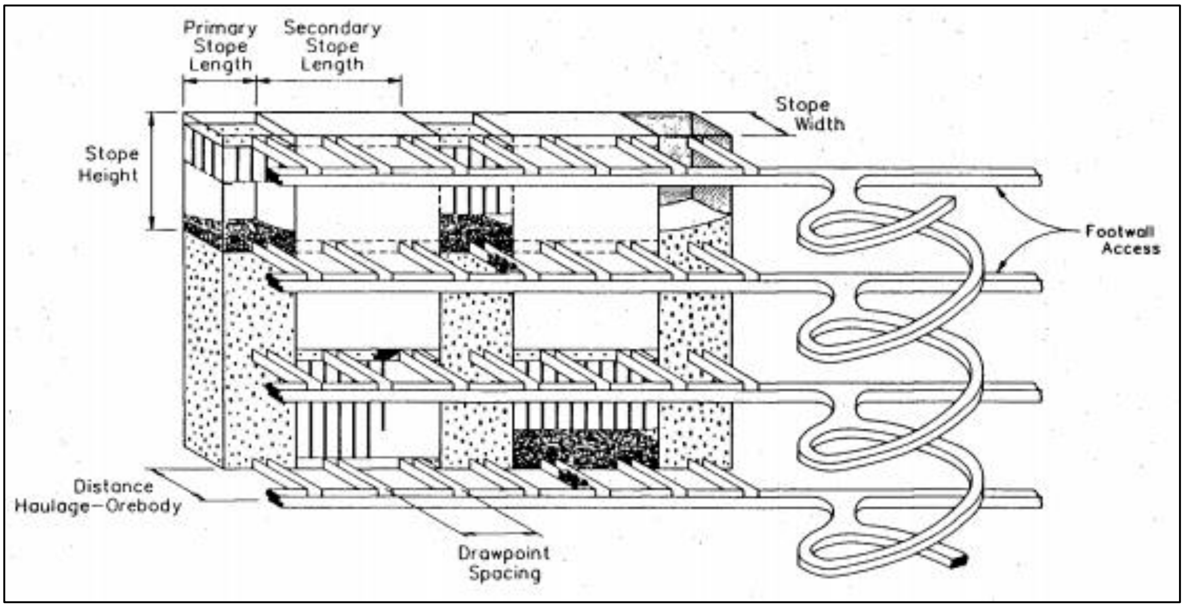


Figure 2.5 Idealised open stoping method with small primary stopes and large secondary stopes (after Potvin, 1988)

2.3.3.3 Stope and fill mining

In this variant, the principle is to mine and backfill adjacent stopes consecutively in such a pattern that no pillars are formed. A mining method utilizing a stope and fill method and a longitudinal open stoping is illustrated in Figure 2.6. All the stopes, excluding the first one, are mined against one backfill wall.

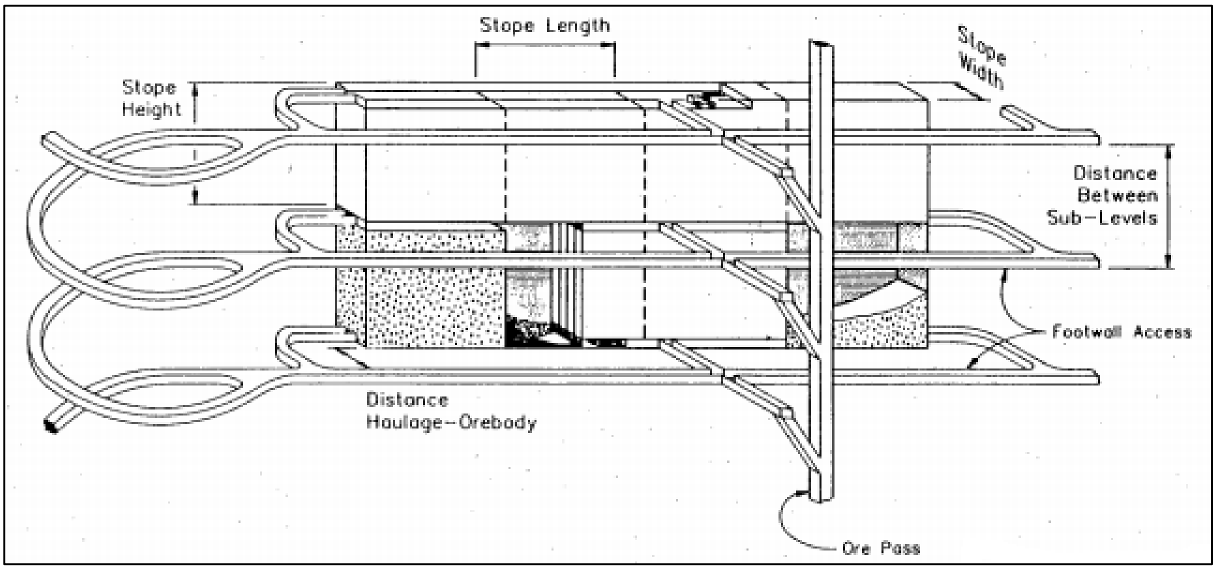


Figure 2.6 Idealised open stope mining method using stope and fill sequence of extraction (after Potvin, 1988)

This variant is sometimes useful in achieving total orebody recovery in highly stressed ground conditions although it has a limitation of losing the high rate of return of the primary stoping. Stoping can also commence from both ends of the ore body on one level at a time going towards the centre. This creates two production faces and therefore minimises the amount of development but has got a major drawback of causing stress concentrations problems in the central stopes. This can be avoided by retreating from one end of the ore body to the other. Another option implies starting a set of stopes at the centre of the block and then proceed towards both abutments. This will therefore keep two production faces and prevent stress build up although it is associated with creation of access problems and long tramming distances.

2.4 Backfill requirements and analysis for mining methods

Generally the backfill material properties and placement method is often depended on the mining sequence adopted and can thus be classified into two groups, that is cyclic and delayed backfilling methods. The stope geometry can be regarded as a key parameter in the determination of appropriate backfill strength requirements because research and in-situ monitoring has shown that stress and deformation conditions developed within backfill resulting from self-weight effects related chiefly to the exposed heights of the backfill material.

In describing the mining backfill technique, it is of relevant importance to understand the following terms below:

1. Mining object – these are usually smaller units (stopes or pillars) in which the ore deposit is cut into.
2. Mining life – It is a period of time starting from the beginning to the end of a particular mining operation (stope life, pillar life etc.)
3. Backfill purpose – This is the prime reason why the backfill material is employed in the mining operation. Normally backfill material is used for either of the following reasons (pillar recovery, working platform, ground support, waste disposal etc.
4. Mining geometry – The dimensions of the mining objects which are described by the length, height and width, and location of stopes relative to both the surface operations and other workings.

Delayed backfill is generally applied for situations where the rock mass characteristics are equally stable and efficient production is required. On the other hand, cyclic backfill is used as an operation cycle platform for mining equipment or mining may occur below, beside or through the backfill. Therefore, the cyclic one is used in situations where the rock conditions are relatively poor and a low dilution operation is needed. Examples of mining techniques related to cyclic backfill are overhand cut and fill, underhand cut and fill and post pillar cut and fill while those for delayed backfill are shrinkage stoping, sublevel stoping, vertical crater retreat, room and pillar, longwall mining block caving and blasthole stoping.

The specifications of the different mining techniques includes the following parameters (Masniyom, 2009):

- Rock condition- hanging wall, footwall and orebody.
- Mining object (stope or pillar)
- Backfill purpose
- Mining method – mining method and its classification
- Mining geometry (length, width and height of mining object)
- Operation period or Life of mining object
- Primary stress (major, minor and intermediate principal stresses)
- Mining operation productivity

2.4.1 Geomechanics of mine backfill

There are several backfill material properties. These includes UCS, unit weight, permeability, shear strength characteristics, elastic modulus, void ratio, liquefaction potential, particle size distribution, mineralogical composition and chemical reactions, Atterberg limits and slump characteristics. However, there are two main mine backfill mechanical properties that are generally regarded for industrial applications. These two properties are identified as:

- Compressive strength
- Permeability

The main reason or relevance of the compressive strength in mine backfill material is to ascertain whether the backfill material will fail due to self-weight in a backfilled stope or not. The compressive strength also determines the maximum height that the backfill can stand in a stope. Several researchers including (Grice. A. G, 1989; Annor. A. B, 1999; Belem, et al, 2000) have reported a required compressive strength of 1MPa after a curing period of 28 days. Generally, the required compressive strength to be used to fill a stope is chiefly related to the underground mining method used and mine backfill application in the underground mines. For example, (Hassani & Archibald, 1998) put forward a 5-7 MPa compressive strength of 28-day curing time for delayed backfill with pillar recovery. However, they also suggested that a strength of generally less than 1 MPa of 28-day curing is required for cut and fill mining.

Mutawa (2011) also carried out a geotechnical investigation on the available mine waste material at KCM, Zambia with a bid to develop an appropriate backfill material. The compressive strength of the backfill material was targeted to achieve a strength of 1 MPa for safe and economic ore production over a curing period of not less than 28 days. This strength also was to be coupled with suitable drainage characteristics in order for the material to resist failure due to self-weight as well as facilitating ore pillar recovery.

On the other hand permeability is important in determining how quickly transport water will drain from the backfill material in the stope. It is the main parameter that regulates the stope dewatering capability and stope filling rate. The universally accepted permeability rate in the mining industry is 100mm/hr (Hassani & Archibald, 1998).

2.4.2 Backfilling in the underground mining industry

Backfill is used in the mining industry for several reasons which include ground stabilisation, to extend the life of mine, lowering cost of mine closure (environment) and for purposes of extracting ore from adjacent blocks of ore/stopes etc. In the mining industry, there is a variety of different types of filling techniques used. These include hydraulic fill (HF), cemented hydraulic fill (CHF), paste fill (PF), rock fill (RF), cemented aggregate fill (CAF) etc.

2.4.2.1 Hydraulic fill (HF)

Hydraulic fill is the name given to a class of mine fill material types that can be delivered as high density slurry via pipelines and boreholes to the specific underground workings. The name hydraulic fill originates from the water-borne delivery method. This type of fill is usually prepared by dewatering and desliming tailings waste from mineral processing streams and has the following characteristics:

- The maximum particle size is usually less than 1mm, and generally most of the fines are removed to make sure that not more than 10% by weight of < 10µm are retained. This is vital to ensure proper fill permeability.
- Slurries are prepared at densities of between 40-50% (solids by volume).
- The slurry transport system should be turbulent and heterogeneous at average velocities greater than the critical settling velocity of the fill material.
- In situ permeability for HF is in the range of 10^{-5} to 10^{-6} m/s. excess water used to transport the fill material must drain out of the fill by vertical gravity drainage through the fill solids, decantation and engineered drainage facilities.
- Placed HF has porosities of 50% (void ratio = 1) but values as low as 30% have been recorded.

HF has found its application in the mining industry for a long time and also remains the most commonly used mine fill type. The preparation and delivery cost are relatively low as the dewatering and desliming processes can be performed using simple hydro cyclone technology. HF can be placed with or without cement and however the uncemented hydraulic fill becomes one of the cheapest bulk fill type available.

HF is used in a number of different mining applications underground. However, in each mining method, there are certain specific fill characteristics that are used to enable better or improved ground conditions, safe working conditions and increased ore recovery. The mining methods in which HF can find its application include cut-and-fill, drift-and-fill, post-pillar cut-and-fill, bench stoping and sublevel open-stoping.

HF can be cheaply produced from a variety of materials, ranging from natural surface sands deposits to the finely ground waste material from mineral processing operations. However, differences in particle shape, size distributions and mineral composition can influence transport, placement, drainage and performance properties of the fill material.

The fill material is prepared and handled through different equipment such as hydro cyclones, spiral and rake classifiers, drum filters, elutriation tanks for removal of fines from the slurry, storage tanks and pachucas and finally through delivery systems from preparation sites to stopes.

However, fill material needs to be contained in the stopes to retain the fill solids while permitting the excess transport water to drain out of the stopes. The retaining wall are called barricade and these structures must have the capacity to withstand the maximum anticipated lateral pressure from the HF. The wall must also be more permeable than the fill itself or must be equipped with adequate drainage facilities that will permit the passage of the drainage water. There are a number of design issues that must be accounted for during the design and construction of HF barricades. These are dimensions and shape of the wall, structural design of the wall, condition of the rock on the interface of the barricade, the quality of material and construction method used and most importantly the distance of the barricade wall from the brow of the stope. (Kuganathan, 2002) Clearly demonstrated that barricades built closer to the brow are subjected to higher loading conditions. On the other hand, the research proved that drainage is reduced with longer distances from the brow.

Despite the relatively low cost of using HF, strict controls are required to guarantee the safe operation of the fill systems relating to permeability, drainage, placement rates, design and construction of retaining barricades. Fatalities have resulted from some inrush accidents where HF has undergone liquefaction related failures (Torlach, 2000).

Advantages of HF

- Uses the wet tailings directly from the milling process which significantly reduces the amount of the tailings dams required on the mining lease.
- Can be used effectively if there is no mining next to or under the stope being filled.
- Uses a waste product.
- Short waiting time if firing a stope above. However, this chiefly depends on the type of drainage system available in the stope being filled as well as the amount of very fine sand material still used in the fill.

Disadvantages of HF

- The plant needs to be relatively close to the mill.

- ❖ The amount of H.F. produced depends on the mill's throughput & the type of raw ore being treated.
- ❖ A good drainage systems are required within the stope to enable efficiency and reduce accidents.
- ❖ Medium cost due to the continual maintenance associated with cyclones, pumps and the removal of the waste water and fines. The drilling of holes between the surface and the drives underground, installation of the underground pipelines to the top of the stope being filled and the building of bulkheads at the draw points.
- ❖ Excess water created needs to be pumped out of the mine.
- ❖ Can contaminates the ore in case it is used next to or over another orebody/stope.
- ❖ Relatively high risk of a bulkhead failure due to pressure caused by the water in the fill if the correct procedures are not properly followed.

2.4.2.2 Cemented hydraulic fill (CHF)

This is basically hydraulic fill that is provided with a binder (cement) to improve its strength characteristics and to allow the fill walls to be exposed by future mining activities. Cement binders must be added to HF to provide cohesive strength in any situation where the fill material requires additional shear strength due to exposure from pillar mining or from the possibility of resaturation and liquefaction risk.

Advantages of CHF

- ❖ Uses the wet tailings directly from the milling process which reduces the size of the tailings dams needed on the mining lease.
- ❖ Uses a waste product.
- ❖ Uses basic hydraulic fill and then adds cement/binder to the mix.
- ❖ Can be effectively used in case there is mining activities adjacent to or under the stope being filled.

Disadvantages of CHF

- ❖ The plant needs to be relatively close to the mill.
- ❖ The amount of H.F. produced depends on the mill's throughput & the type of raw ore being treated.
- ❖ A good drainage systems are required within the stope to enable efficiency and reduce accidents.
- ❖ Cemented Hydraulic fill needs to be pumped into the stope via pipelines.

- ❖ Excess water created needs to be pumped out of the mine.
- ❖ Relatively high risk of a bulkhead failure due to pressure caused by the water in the fill if the correct procedures are not properly followed.
- ❖ High cost as a result of the cement/binder content required, continual maintenance associated with, cyclones, pumps and the removal of the waste water and fines. Additional cost emanates from the drilling of holes between the surface and the drives underground, installation of the underground pipe works to the top of the stope to be filled as well as the building of retaining bulkheads at the draw points.
- ❖ Depending on the cement/binder content in the mix it could take up to 3 months for the fill to cure sufficiently before mining can be permitted on the stope adjacent to or under it.

2.4.2.3 Paste fill (PF)

Paste fill is defined as dense non draining slurry made from single or combination of several suitable solid engineered materials produced to toothpaste consistency. Paste can also be regarded as a non-segregating slurry, meaning that it has negligible excess water when stationary and remains fundamentally as a homogeneous single phase product. From underground hard rock mines, the density of paste fills is generally $75\%_{cw}$ and $85\%_{cw}$ (solids by weight) depending on particle size distribution and specific gravity of the solids.

Mill tailings are the largest waste stream from most underground stoping operations. Over the past 100 years, tailings have progressively become finer to increase metallurgical recoveries due to developments in mill processing. As a result, the finer nature of the tailings results in either reduced recoveries of HF from the tailings or unacceptable drainage properties of the resultant fill material.

Paste fill was initially developed to defeat this challenge by utilising the full stream of relatively fine tailings as an efficient and safe underground mine fill. Paste is now an extensively accepted alternative means of mine fill because over the past years, the preparation and distribution technology have quickly evolved.

Paste fill is engineered in order to meet the following criteria:

- ❖ Flow properties (rheology) – delivery of paste fill from surface to underground via pipes and boreholes at the highest practical pulp density.

- Strength properties – paste material should remain stable when exposed to static or dynamic loading conditions faced by the mining operation. This includes short term during local mining activities or long term stability against regional seismicity events.

Demands from mining methods

The requirements on PF imposed by the mining methods can be broadly divided into two areas namely the fill performance and the rate at which the fill can be placed. There are two key performance requirements for paste fill which are; firstly, flow characteristics to transport the paste to the stope and, secondly, for the paste to achieve the desired demands on the fill stability. The mining method being used will have a major influence on the filling method to be selected and therefore filling should be considered an integral part of the mining operation, rather than a post mining activity.

Rate of filling demands on the fill material are usually closely related to the time interval between completion of ore extraction in the stope to be filled, and beginning of adjacent stoping (Potvin, et al 2005). The stages of stope filling include; stope preparation, filling the stope and the curing time of the fill.

Supply of materials

The basic requirement of paste fill is:

- Tailings – major component of the fill. The tailings characteristics determine whether a paste can be produced.
- Water – generally most of the water present in the tailings stream is usually removed in the paste fill plant.
- Binder – its purpose is to increase the strength of the fill. The type of binder and strength requirements of the fill determines the rate of binder addition.
- Coarse fraction – a coarse material can be introduced to the tailings paste in order to improve the particle distribution to that of a well graded material. Typical coarse material used are sands or crushed rock (Li, et al., 2003). Addition of the coarse material results in greater fill density as well as enhanced water cement ratio.

Retention

After the completion of production from the stopes, barricades should also be erected in draw points in order to retain the paste fill. The barricades must be of sufficient strength to retain the paste fill and should also allow for drainage of any seepage water that may occur. Actual loads of paste fill on barricades was measured by (Revell & Bloss, 2000) using earth pressure cells in the fill and a boundary cell within the barricade.

Examples of PF applications

The crude mine located in Germany was the first mine to utilise paste fill (Potvin, et al 2005). Considerable effort was also applied by South African gold operations in the late 1980s in order to use PF as an alternative to tailings HF. The significant problem facing the utilisation of PF in South Africa is a result of extreme mining depths and relatively flat ore deposits, which imply long horizontal transfer distances for the fill mass. This condition requires booster pumps as high pressures are experienced in the pipelines thus leading to increased operating costs and making the system more difficult to operate.

Other operations in the USA and Canada also utilises paste fill. In Australia there are also several operations but the first PF system was installed at the Elura Mine (New South Wales) in the 1980s (Barrett, 2000). The system was not successful due to the behaviour of the material used and thus it was converted into HF system.

Advantages of PF

- Utilises a waste product that could otherwise be an environmental issue.
- Has a short curing time, 2 to 4 weeks. This is dependent on the size of the stope, the solids content and the binder content used.
- Very little waste water is produced underground since all of the water in the paste material is utilised during the curing process.
- Relatively low risk of a bulkhead failure due to the lack of pressure caused by the water in the fill if the correct procedures are strictly adhered to.
- In most cases PF does not need pumping as it is gravity transported via pipelines for deep mines. To obtain the required continuous flow rate in shallow mines, a positive displacement pump may be required.
- Environmental issues such as rain does not have much of an influence on the operation of the PF plants.

Disadvantages of PF

- PF processing plant needs to be relatively close to the mill.

- Higher costs due to the extra equipment required and the associated maintenance, the cement/binder content required, installation of pipelines, drilling of holes and building of bulkhead walls at the stope draw points.
- Pumps the tailings through banks of cyclones to remove most of the water and very fine material and then dries the tailings over a vacuum drying conveyor, then has cement/binder and water added to the mix.

2.4.2.4 Rock fill (RF)

Rock fill is one of the techniques used in filling underground mine openings. The fill material comprises of underground generated waste rock, overburden rocks removed by nearby surface mining operations, quarry produced rock fill, natural river gravel or various smelter slags produced from furnaces.

Rock fill in its natural state a loose and granular medium that cannot form vertical walls if exposed to an adjacent opening. The angle of repose of RF material ranges between 35^0 and 55^0. Therefore RF in its simplest use is dumped into mine openings in order to fill them. Further deterioration of the surrounding rock wall is delayed or arrested by filling the mine voids using RF. Sometimes the development waste is dumped into mine voids to avoid having to hoist the waste to the surface for purposes of disposal.

Advantages of RF

- This type of mine fill can be effectively utilised provided there is no mining adjacent to or under the stope being filled.
- There is no waiting time once the stope is filled if firing a stope from above is to be done.
- This type of fill uses the waste rock from the underground workings of the mine to fill the void which reduces the amount of the waste to be stockpiled on the surface.
- Low cost fill and can actually save money as the waste rock would otherwise need to be transported to the surface and a separate storage space would need to be set up.

Disadvantages of RF

- There is higher chance of ore contamination if used adjacent to or over another orebody/stope.
- Difficult to completely fill the stope as a result of the natural rill angle of the rock once tipped.

2.4.2.5 Cemented aggregate fill (CAF)

The CAF utilises the cemented hydraulic fill and then adds aggregate at some point in the system. It can also be a system which makes use of a batching plant to mix some dry tailings from a tailings dam, aggregate, binder/cement and water.

Review of some important aspects of CAF

There is a substantial amount of literature reviewed in relation to in situ and laboratory testing as well as evaluation of CAF performance. Experience from various authors and researchers in fill design and in situ testing has indicated that there are certain vital factors that influence the in stope performance of CAF as summarised in Table 2.1.

Table 2.1 Factors affecting the performance of the placed CAF

1. Constituent materials			
Cement	**Water**	**Admixtures**	**Aggregate**
Type Quantity	Quantity Quality	-	Maximum size, shape, grading, coarse : fine ratio, surface texture, absorption
2. Preparation			
Batch process	Continuous process	**Binder dispersion**: Active mixing, Passive mixing	
3. Placement			
Batch process	Continuous process		

Other aspects of CAF that needs to be fully evaluated and understood in order to operate a quality product include:

- ❖ **Particle size distribution** – the size distribution of the aggregate is an important parameter in determining the strength and placement properties if the CAF. Research indicates that fill masses with low density (high void ratio) generally are prone to segregation challenges and poor blast damage performance.

- **Fill placement** – observations on fill exposures indicates that during placement a degree of segregation is unavoidable.
- **Fill quality test work** – It is important that a fill quality test work program is maintained in order to guarantee that the CAF is produced to consistent standards that satisfies required demands. The following elements should be considered in such a test program: aggregate quality, particle size distribution of aggregate, aggregate moisture content, binder strength, slump tests, CAF reference cylinder strength tests, sands and tailings.
- **In situ assessment of CAF-** this is done in order to analyse and understand the in situ characteristics of the fill. Exposure mapping, coring, borehole pressure meter and video monitoring can be used for in situ assessments.

Advantages of CAF

- Utilises the wet tailings directly from the milling process which reduces the size of the surface tailings dams required on the mining lease.
- Utilises waste rock from the underground development workings of the mine which reduces the amount of the waste stockpile on the surface.
- Fills the stope quicker since two waste products are being used simultaneously.

Disadvantages of CAF

Refer to CHF disadvantages plus:

- Wears out the pipeline very quickly if the aggregate is added into the pipeline.
- Correct ratios must be maintained in order to satisfy the required strength.
- Depending on the cement/binder content in the mix it could take up to 3 months for the fill to cure sufficiently before mining can be allowed on the stope next to or under it

2.4.3 Hazards, risks and environmental aspects of mine fill

Uncertainty is one of the distinguishing aspects of underground mining. Mine fill material, although very useful can pose serious environmental, safety and healthy challenges to personnel and equipment if not successfully implemented. Because of these uncertainties, the application of risk management approaches may prove to be very useful in order to minimise the possible negative consequences of using mine fill. The risk management program should include; risk identification, risk analysis, risk evaluation, risk treatment, monitoring and reviewing as well as recording the risk management process.

The environmental aspects related to use of mine fill are considered from a positive aspect, in cases where fill has been used successfully, and on a negative aspect. Heavy metal contamination and Acid rock drainage (ARD) are the major challenges related to mine fill use. Some other fill mixtures if not properly designed can undergo chemical reactions which can result in the release of toxic gases e.g. hydrogen sulphide. This type of gas can pose death risks if the permissible levels underground are exceeded (Potvin, et al, 2005).

Hazards associated with mine fill underground include the following:

- Barricade failure
- Excessive water ponding from top of the placed fill
- Abrasive wear of fill lines
- Blocked fill pipes and or boreholes
- Liquefaction of fill
- Extensive stope wall collapse in an unfilled or partially filled stope and
- Vehicles coming in contact with fill pipelines

2.4.4 Current research and emerging technology on mine backfill

Researchers are constantly carrying out some research work related to mine fill in order to improve on system efficiencies, reduce cost and improve on safety healthy and environmental concerns (Krishna, 2011). Much work is being done in paste fill as its use has increased worldwide in response to a growing realisation that, in spite of the initial costs, paste fill is generally the most effective fill material when total costs are considered.

To date, most research into paste fill has been mainly focussing on the rheology aspects of the material in a bid to effectively design suitable pumping and distribution systems (McGuinness & Cooke, 2011). Other work has been seen into cemented backfill with Paterson & Cooke completing a study review on strength and flow behaviour test and feasibility design on cemented paste fill for Anglo Platinum between 2011 and 2012.

Other technologies are being developed within; mine backfill systems and design, slurry pipelines design, thickened and paste slurry transportation systems and slurry dewatering and thickener operations (Paterson & Cooke, 2016).

There is also much emphasis on reducing the cost of binder material and much cheaper and readily available possible binder by products from furnaces are being tested. Other waste materials are finding way into paste fill to improve on waste disposal and cost reduction (Spearing, et al., 2010).

2.5 Stope design and factors influencing stability

All underground mining operations create openings in different forms, mostly called stopes. These stopes can lead to instability problems for surface structures or further underground mining activities. The failure of a rock mass around such underground openings depend upon the in situ stress level and also on the characteristics of the rock mass. Any design of underground excavation requires systematic and careful approach, in order to determine rock mass properties and behaviour of rock around the opening. Rock mass quality, in combination with mechanical processes in the rock mass, especially stresses, are key factors in defining of stability (Svrkota, et al., 2015). Main influential factors to stability of underground openings are shown in Figure 2.7.

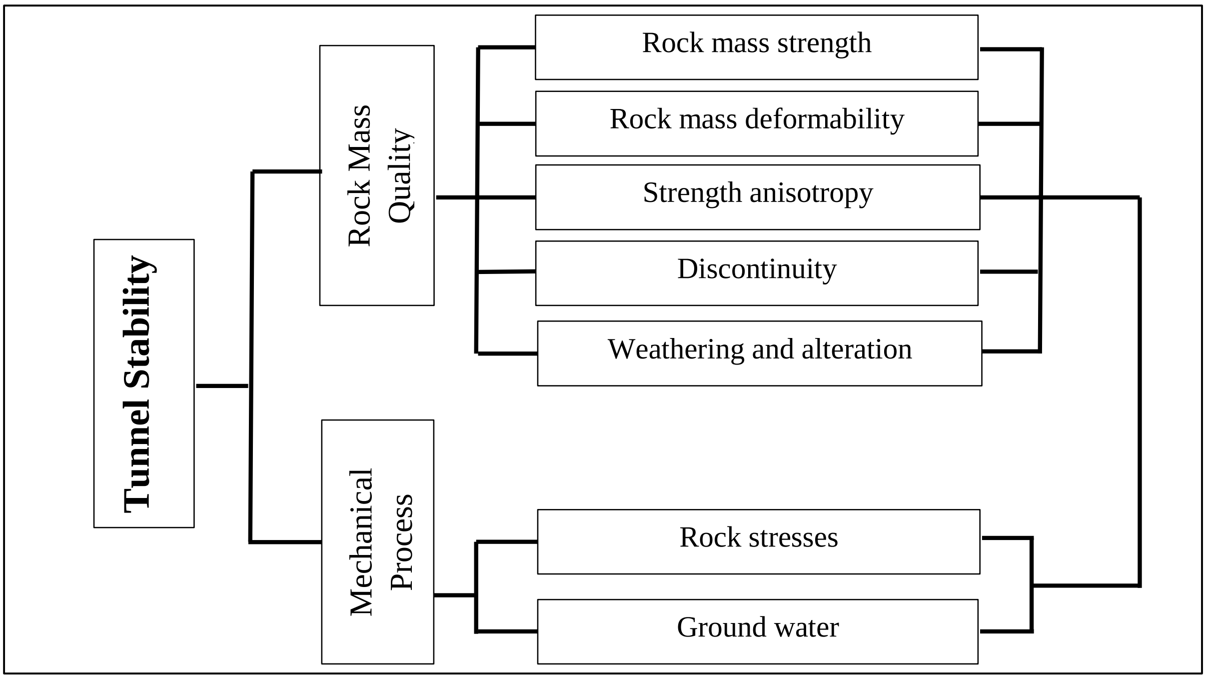


Figure 2.7 Influential factors to stability of underground openings

2.6 Stope design and stability before backfilling

The stability of stopes before backfilling is of much significance due to the possibility of failure in early or mid-backfilling as well as start of major failure in half-filled stopes. The Mathews stability graph method is widely used as a common stope design tool. The original version of this method for open stope design was initially introduced for mining operations at depths of less than 1,000m (Mathews, et al., 1981). Several other researchers have also gathered information from different locations of various mining depths and rock mass conditions to improve the method and assess its validity (Potvin, 1988).

The current version of this method is based on the analysis of over 350 case histories gathered from underground mines in Canada and it accounts for the fundamental factors affecting open stope design. Information in relation to the rock mass strength and structure, the stresses around the opening, size, shape and orientation of the stope is used to decide whether the stope will be stable without support, stable with support, or unstable even if supported.

2.6.1 The Stability Graph method

Mathews et al, 1981 introduced an empirical method or approach for the prediction of stability of open stopes in deep mining environments (below 1000m depth). Basically this method was an extension of the Norwegian Geotechnical Institute (NGI) rock mass classification system and had the capacity to recognise:

I. Structural failure in stopes,
II. Stress induced failure in open stopes,
III. And a combination of both structural failure and stress controlled failure.

The design procedure for the method depends upon the determination of two important parameters, N', the modified stability number which signifies the ability of the rock mass to stand up under a given stress condition, and S, the hydraulic radius or shape factor which accounts for the size and shape of the stope. The stability number indicates the effect of geotechnical factors constituting a major influence on the stope stability. A high stability number indicates a stable ground condition and the opposite is true for a lower number.

A plot between the stability number, N' and the hydraulic radius, S was therefore derived by plotting the two parameters on a semi-log graph as shown in Figure 2.8. Therefore the stability of a stope plane being analysed can then be evaluated according to where it plots with respect to the following three regions of the graph according to Potvin, 1988:

- Stable zone;
- Potentially unstable zone (where there is possibility of major failure); and
- And potentially caving zone.

In within these three zones, there are some regions separated by transition areas and were defined based on 26 different case studies from three mines (2 Canadian and 1 Australian) and other 29 cases from literature (Potvin, 1988). With the graph and a constant N'-value the hydraulic radius can be manipulated to maximize on stope dimensions while at the same time maintaining a stable zone of the stability graph.

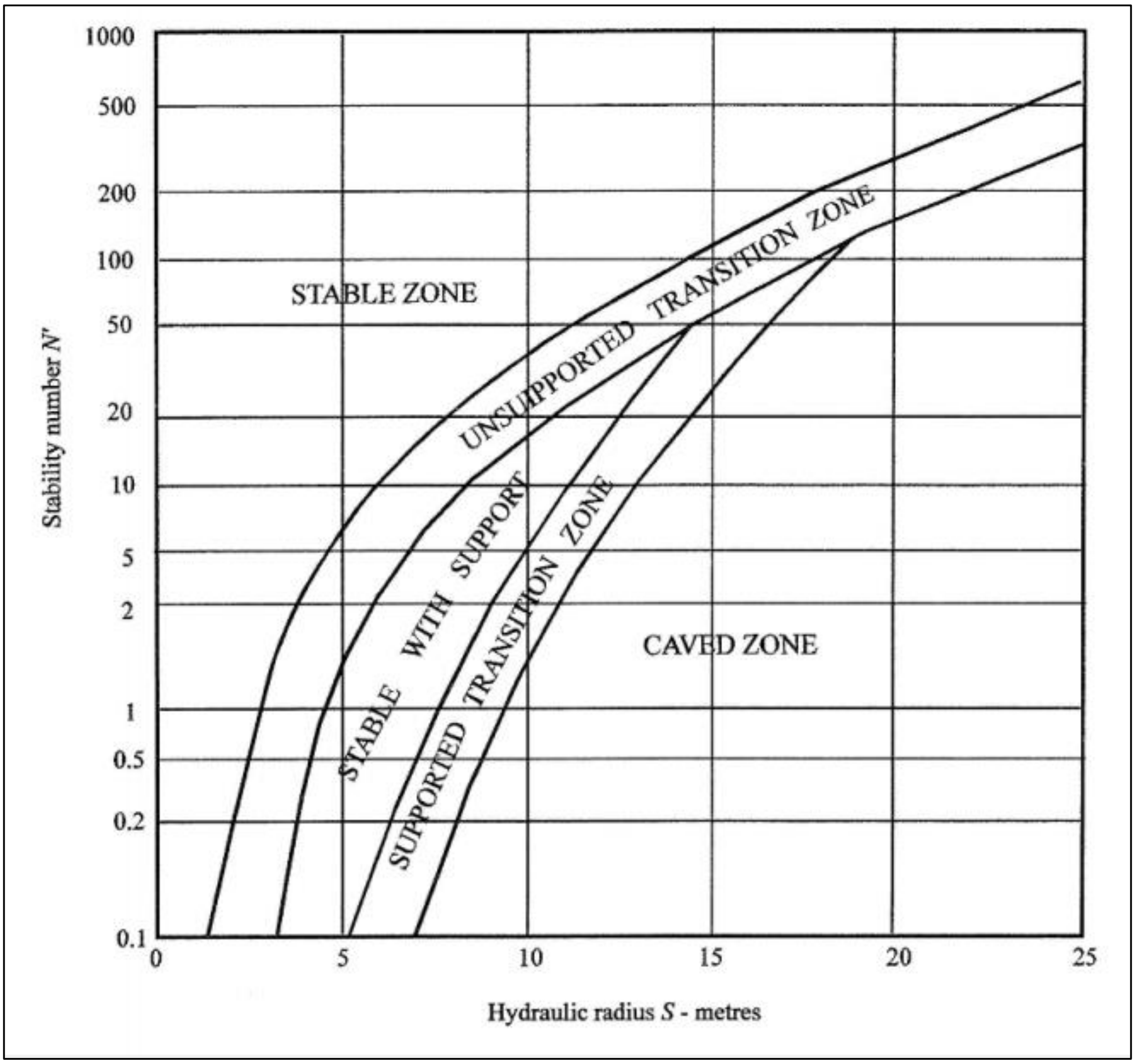


Figure 2.8 Stability graph showing zones of stable ground, caving ground and ground requiring support (After Potvin (1988), modified by Nickson (1992))

2.6.1.1 The stability number

The following procedure indicates how the modified stability number is calculated:

$$N' = Q' * A * B * C \qquad (2.1)$$

Where Q' is the modified Q Tunnelling Quality Index,

A is the rock stress factor,

B is the joint orientation adjustment factor, and

C is the gravity adjustment factor,

The modified Tunnelling Quality Index, Q', is determined from the outcomes of structural mapping of the rock mass in exactly the same manner as the standard Rock Tunnelling Quality Index, Q, classification system proposed by Barton et al (1974) of the Norwegian Geotechnical Institute (NGI), except that the stress reduction factor (SRF) is set to 1.0. This is because the SRF suggested in the NGI classification is based on case studies on tunnelling and it was seen to be inappropriate to effectively represent the effect of stresses in open stopes. The system has not been applied in conditions with significant groundwater, so the joint water reduction factor J_w is commonly 1.0 therefore:

$$Q' = \frac{RQD}{J_n} * \frac{J_r}{J_a} * J_w \tag{2.2}$$

A: The rock stress factor represents the stresses acting on the free stope surfaces at depth. It replaces the SRF in the original Q classification system to more correctly represent the influence of stresses acting on the exposed surfaces of the open stopes. This factor is calculated from the ratio between the unconfined compressive strength of the intact rock, σ_{ci} and the stress acting parallel to the exposed face of the stope under consideration. The intact rock strength can easily be obtained from laboratory testing of the rock core samples or from field estimates.

On the other hand, the induced compressive stress, σ_1, is determined from numerical modelling for specific conditions in each project (Mathews et al, 1981) or is estimated from stress distributions such as those in Hoek and Brown, (1980) using measured or the in-situ stress estimates.

The factor, *A* is therefore determined from the ratio σ_{ci}/σ_1 on the opening boundary and for:

$$\sigma_{ci}/\sigma_1 < 2 : A = 0.1 \tag{2.3}$$

$$2 < \frac{\sigma_{ci}}{\sigma_1} < 10 : A = 0.1125\left(\frac{\sigma_{ci}}{\sigma_1}\right) - 0.125 \tag{2.4}$$

$$\sigma_{ci}/\sigma_1 > 10 \ : A = 1.0 \tag{2.5}$$

B: The joint orientation adjustment factor considers the influence of orientation of critical geological structures intersecting the open stope surface and it ranges from 0.2 to 1.0. Factor B will reflect favourable or unfavourable cases depending on the relative orientation of these geological structures in relation to the investigated stope plane. The parameter B can be determined from Figure 2.9 which shows parameters for the calculations of the stability number.

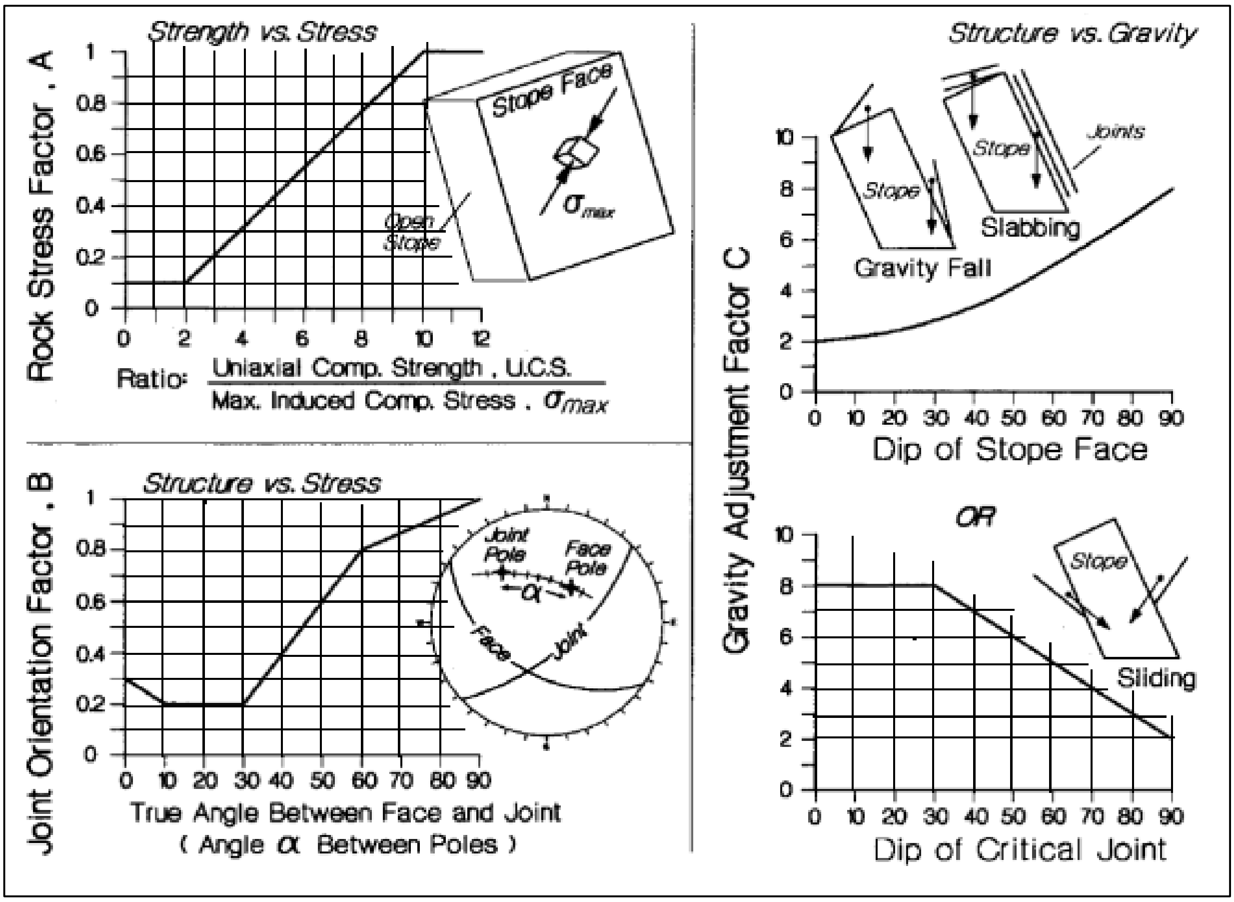


Figure 2.9 Parameters for calculation of the stability number (after Potvin (1988)

C: Factor C is a stope surface orientation factor. It is an adjustment for the influence of gravity of the stability of the stope face. Failure can basically occur from the roof of the stope by gravity controlled failure or, from the stope sides, by sliding or slabbing. Determination of C can either use the dip of the stope face or the dip of the critical geological structure (joint) on the analysed stope face. Mathews et al 1981, proposed that a vertical stope wall is approximately eight times as stable as a horizontal roof since in non-entry stopes some minor instability can be allowed. Therefore, C can also be obtained from Figure 2.9 or calculated from Equation 2.6 since Potvin (1988) suggested that both slabbing and gravity induced failure rely on the inclination of the stope surface:

$$FACTOR\ C = 8 - 7cosine(angle\ of\ stope\ plane\ inclination) \qquad (2.6)$$

In general factor C outlines the increasing potential of stope instability as the surface becomes more horizontal.

2.6.1.2 The Hydraulic Radius, HR or Shape factor, *S*

The shape factor, or hydraulic radius (HR) of the open stope under analysis needed for application in the Stability Graph can be calculated from the following simple relationship:

$$S = \frac{Cross\ sectional\ area\ of\ stope\ surface\ analysed}{Perimeter\ of\ stope\ surface\ analysed} \tag{2.7}$$

2.6.1.3 Discussion of the stability graph method

Potvin, (1988) concluded that the Mathews stability graph method offers a strong capacity for the design of open stopes although he modified the graph so that it can represent the transition zone between stable and caving more accurately. However, the parameters included in the method are based on an existing rock mass classification system, but they are adjusted in order to be more representative to mining conditions.

The combination of the four parameters used (*Q'*, *A*, *B*, *C*) permits the prediction of the stope instability that can result from high stresses, gravity, structural failure or weak rock mass, or any combination and interaction of these different parameters. The method is therefore relatively easy to use in practical situations and can be successfully used by design engineers on different sites because the parameters are well developed and graphically illustrated. The method is useful since it addresses the most important parameters influencing the stability of mine stopes (Potvin, 1988).

2.7 Rockmass properties

Reliable estimates of the rockmass strength and deformational characteristics are needed for almost every form of analysis used for the design of underground openings rock slopes and foundations. The Hoek and Brown failure criterion (Hoek and Brown, 1980) and the Mohr Coulomb failure criterion are the two most widely used and accepted failure criteria in rock engineering design. The Mohr Coulomb failure criterion was initially developed for the evaluation of intact rock failure and it is usually graphically represented as a straight line, with a tensile cut-off.

On the other hand, Hoek and Brown, (1980) introduced a method to estimate the strength of jointed rock masses. The method was based upon an assessment of the interlocking of rock blocks as well as the condition of the contact surfaces between these rock blocks. However, this method was modified over the years in a bid to meet the requirements of different users who were applying it to problems that were not taken into account during the formulation of the original criterion. For the above conditions and applications of these two criterion, the Hoek and Brown criterion will be used in numerical modelling.

Further changes to the method where necessary when the method was to be applied to very poor quality rock masses (Hoek, et al., 1992). This also eventually led to the development of a new rockmass classification called the Geological Strength Index, GSI (Hoek, Kaiser & Bawden, 1995), (Marinos & Hoek, 2001). Further changes were also made in 2002 to make the criterion applicable in numerical models and to update the methods for estimating the Mohr Coulomb parameters (Hoek, Carranza-Torres & Corkum, 2002).

The term rock mass can be defined as the combination of rock material together with the three-dimensional structure of discontinuities (joint orientation, joint properties and joint set geometry (spacing, length, position etc.)).

2.7.1 Generalised Hoek-Brown criterion

The generalised Hoek-Brown failure criterion for the jointed rock masses is represented by:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}[m_b \frac{\sigma'_3}{\sigma_{ci}} + s]^a \tag{2.8}$$

where σ'_1 and σ'_3 are the maximum and minimum effective stresses at failure,

m_b is the value of Hoek-Brown constant m for the rock mass,

σ_{ci} is the uniaxial compressive strength of the intact rock pieces, and

s and a are constants which depend on the rock mass characteristics.

Three properties of the rock mass should be estimated in order to use the Hoek-Brown criterion to estimate the strength and deformability of the jointed rock masses. These three are:

- Uniaxial compressive strength σ_{ci} of the intact rock pieces
- Value of the Hoek-Brown constant m_i for the intact rocks, and
- Value of the GSI of the rock mass.

For poor quality rock masses, the modified Hoek-Brown failure criterion is more appropriate. This other version is obtained by setting s=0 in Equation 2.8. However, for good to reasonable quality rock masses with considerably tight interlocking between rock blocks, the constant a = 0.5 and therefore reducing Equation 2.8 to the original Hoek-Brown failure criterion.

2.7.1.1 Intact rock properties

Intact rock generally refers to the unfractured rock blocks that exists between the structural discontinuities. For the intact rocks that constitute the rock mass, the constant s=1 and m_b= m_i and the failure criterion equation reduces to:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}[m_i \frac{\sigma'_3}{\sigma_{ci}} + s]^{0.5} \quad (2.9)$$

For a given rock, the relationship between the principal stresses at failure can be defined by two constants, the UCS σ_{ci} and another constant m_i. The constant m_i is said to be dependent on the mineralogy, grain size as well as the composition of the intact rock. Wherever possible, the values of these two constants should be determined from a set of triaxial experiments on intact rock using statistical analysis of the results from the tests on carefully prepared rock core samples. In the case that tests results are not available, the constant m_i and σ_{ci} can be determined from tabulated data provided by Hoek, Kaiser & Bawden (1995). Appendix 2 indicates the estimates of m_i. However, these calculations and many other calculations regarding the Hoek-Brown criterion can also be easily performed using the program RocLab that was developed by Rocscience.

The range of values of the minor principal stress (σ'_3) over which these test should be performed is critical in coming up with reliable values of the constants and (Hoek and Brown, 1980) used a range of $0 < \sigma'_3 < 0.5\sigma_{ci}$. In order to be consistent, it is fundamental that the same range be used in any laboratory triaxial tests on intact core samples and Hoek and Brown suggested that a minimum of five data points be included in the process.

The laboratory tests also should be performed at a moisture content as close as possible to the one which occur in the field. This is because many rocks exhibit a considerable decrease in strength with increasing moisture content and tests on samples, which have been left to dry in core sheds for several months should be avoided as they can give misleading values of the intact rock strength.

In order to estimate the values of the parameters m_b, s and a, Hoek, Kaiser & Bawden (1995) suggested the following relationships.

In general, for undisturbed or interlocking rock masses (GSI>25)

$$m_b = m_i e^{\frac{GSI-100}{28}}, \quad (2.10)$$

$$s = e^{\frac{GSI-100}{9}}, \tag{2.11}$$

$$a = 0.5 \tag{2.12}$$

For disturbed rock masses or rock masses or a very poor quality (GSI<25)

$$s = 0, \tag{2.13}$$

$$a = 0.65 - \frac{GSI}{200}, \tag{2.14}$$

Where GSI is the Geological Strength Index

Hoek, Kaiser & Bawden (1995) developed the following relationships between GSI and the RMR developed by Bieniawski (1976, 1989).

For $RMR_{76'}$>18:

$$GSI = RMR_{76'} \tag{2.15}$$

For $RMR_{89'}$>23:

$$GSI = RMR_{89'} - 5 \tag{2.16}$$

These is also an updated version of the Hoek-Brown criterion which includes the disturbance factor, D that results from blasting. This factor led to the modifications of the m_b, s and a, parameters but the equations derived prior to this will be used for the estimation of these parameters in the numerical modelling.

2.7.2 Geological strength Index

The strength of a jointed rock mass rely on the characteristics of the intact rock pieces as well as on the freedom of these individual pieces to slide and rotate under various stress conditions. The degree of freedom is controlled by the geometrical shape of the individual rock pieces and the condition of the discontinuity surfaces separating the rock pieces. Much stronger rock mass results from angular rock pieces with clean, rough discontinuity surfaces as compared to the rock mass which is controlled by rounded particles surrounded by altered and weathered material.

The GSI was introduced by Hoek (1994) and Hoek, Kaiser & Bawden (1995) make available for use a number which, when combined with the intact rock characteristics, can be utilized for estimating the reduction in the rock mass strength for different geological set ups. The method is illustrated in Appendix 1 for the block rock conditions.

During the initial years of the use of the GSI system in rock engineering, the value of GSI was approximated directly from RMR values. However, this correlation has proved to be unreliable, especially for poor quality rock masses and for rocks with lithological peculiarities that cannot be accommodated in the RMR classification. On the other hand, it is therefore recommended that GSI should be estimated directly by means of the charts presented in Appendix 1 and not from the RMR classification as was previously suggested.

2.8 In-situ and induced-mining stresses

The stress conditions existent in the rock mass before an excavation are referred to as the in-situ stresses, virgin stresses or initial stresses. Creating an underground excavation will change the stress conditions in the rock mass surrounding the opening and a new set of stresses are induced in the rock surrounding the excavation. The final stress state will be a result of the initial stress conditions and the stresses induced by the excavation.

An understanding of the magnitudes and orientations of these in-situ and induced stresses is an extremely important element of underground excavation design. This is because, in many cases, the strength of the rock is exceeded by these stresses and the resulting instability can have serious consequences on the behaviour of the mine openings (Hoek, et al., 1995). The stability of an underground excavation will therefore depend on the rock's ability to sustain failure induced by the stresses around the opening (Larsen Vestad, 2014).

2.8.1 In-situ stresses

To understand the concept of in-situ stresses, an element of rock at a depth of 1000m below the surface is considered. The weight of the overlying column of rock resting on this element of rock is simply the product of the unit weight of the overlying strata (usually 2.7 tonnes/m^3 or 0.027 MN/m^3) and the vertical depth. This vertical stress can be simply represented by the following relationship;

$$\sigma_v = \gamma z \tag{2.17}$$

Where σ_v is the vertical stress

γ is the unit weight of the overlying strata

z is the depth of element from the surface

Hoek et al, 1995 concluded that measurements of the vertical stress at several mining and civil engineering sites across the world confirm that this relationship of vertical stress and depth is valid as shown in Figure 2.10, although there is significant scatter in the measurements from these engineering sites.

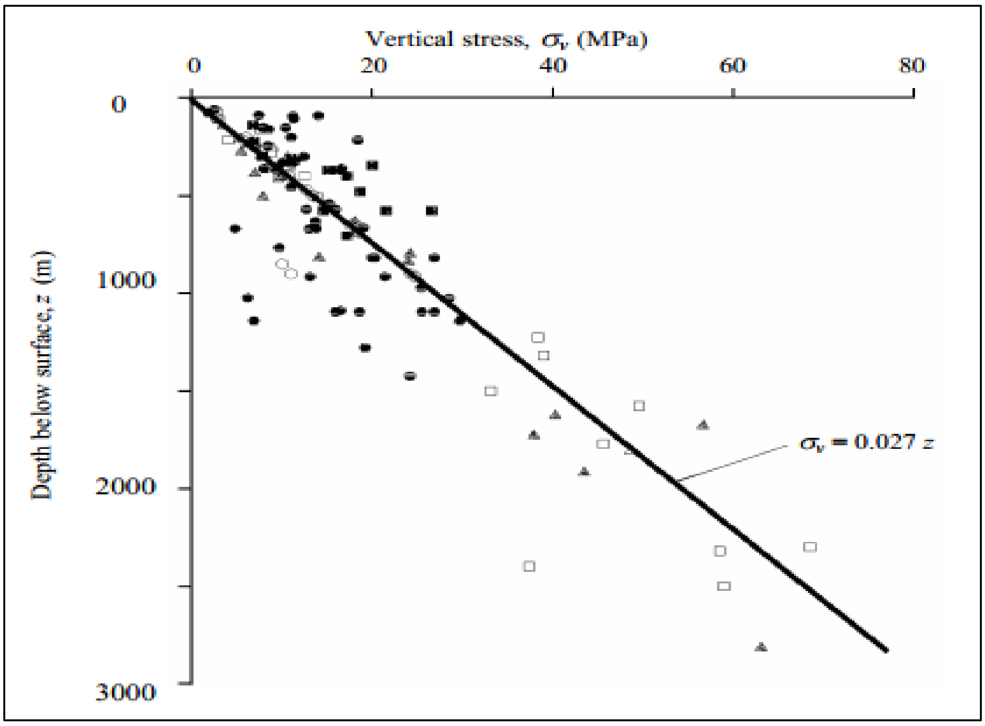


Figure 2.10 Vertical stress measurements from civil and mining engineering sites across the world, after Hoek and Brown, 1978)

On the other hand, the horizontal stress acting on the rock element at a depth z below the surface are quite difficult to estimate as compared to the vertical stresses. The ratio of the average horizontal stress to the vertical stress is usually denoted by k and therefore;

$$\sigma_h = k\sigma_v = k\gamma z \qquad (2.18)$$

For gravitationally loaded rockmass where no lateral strain was permitted during the formation of the overlying strata, the value of the parameter k is independent of the depth from the surface and is given by $k = v/(1-v)$, where v is the Poisson's ratio of the given rockmass (Terzaghi & Richart, 1952). However, this relationship was widely used but it is now seldom used. Horizontal stress measurements at civil and mining engineering sites tends to highlight that the ratio k is usually high at shallow depths and that it reduces at greater depths. (Sheorey, 1994) developed an elasto-static thermal stress model of the earth. This model considers curvature of the crust and variation of elastic constants, density and thermal expansion coefficients through the crust and mantle. This model provided a much simplified expression which can be easily used to estimate the horizontal to vertical stress ratio k (Hoek, et al., 1995) as is illustrated in equation 2.19.

$$k = 0.25 + 7E_h[0.001 + \frac{1}{z}] \quad (2.19)$$

Where z is the depth from surface (m) and E_h (GPa) is the average deformation modulus of the upper part of the earth's crust measured in a horizontal direction.

2.8.2 Analysis of Induced stresses

In the event that an underground excavation is made into a stressed rockmass, the in-situ stresses in the vicinity of the new opening are re-distributed. A clear illustration of the stresses induced in the rock surrounding a horizontal borehole as explained by Hoek et al, 2000 describes this phenomenon. This example is shown in Figure 2.11. Before the excavation is made, the in-situ stresses σ_v, σ_{h1} and σ_{h2} are uniformly distributed in the slice of the rock under consideration.

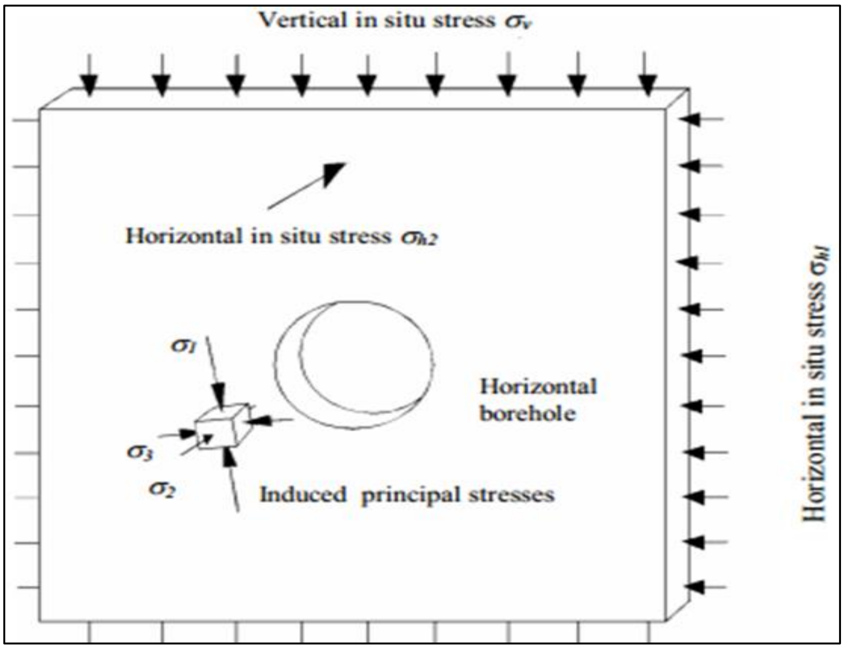


Figure 2.11 Illustration of principal stresses induced in an element of rock close to an underground borehole

When the rock within the borehole is excavated, the stresses in the immediate surroundings of the borehole are changed and new stresses are induced (σ_1, σ_2 and σ_3). These three principal stresses are numbered such that σ_1 is the largest and σ_3 is the smallest (algebraically) and the convention used in rock mechanics is that compressive stresses are always positive (Hoek, et al., 1995). These three principal stresses are mutually perpendicular but they may be inclined to the direction of the applied in-situ stresses.

Hoek et al, (2000) indicated that the redistribution of these stresses is concentrated in the rock very close to the excavation and also at a distance of around three times the radius from the centre of the borehole, the disturbances to the in-situ stresses is negligible.

Since in-situ stress is the most dominant influential factor in estimating the risk of failure in underground openings, risk estimation should be expressed through the ratio of stresses to rockmass strength, or vice versa. According to researches done by Martin, C et al in 2003, a decision tree diagram was proposed as shown in Figure 2.12. Other work by researchers concluded that ratio of uniaxial compressive strength(σc_i) to the vertical stress (σ_v) can be useful for defining stability in weak rocks. Their conclusion was that squeezing in weak rock will appear if $\sigma c_i/\sigma_v > 2$.

Hoek and Brown (1980) introduced a stress to strength ratio ($\sigma_1/\sigma c_i$) or stability index for hard rock, where σ_1 is major principal stress. Their conclusion was that value of stability index ranges from 0.1 to 0.5. For values bellow 0.15, rock mass is stable, between 0.15 and 0.35 is referred to occurrence of minor instability and severe instability for values over 0.4, with heavy support required.

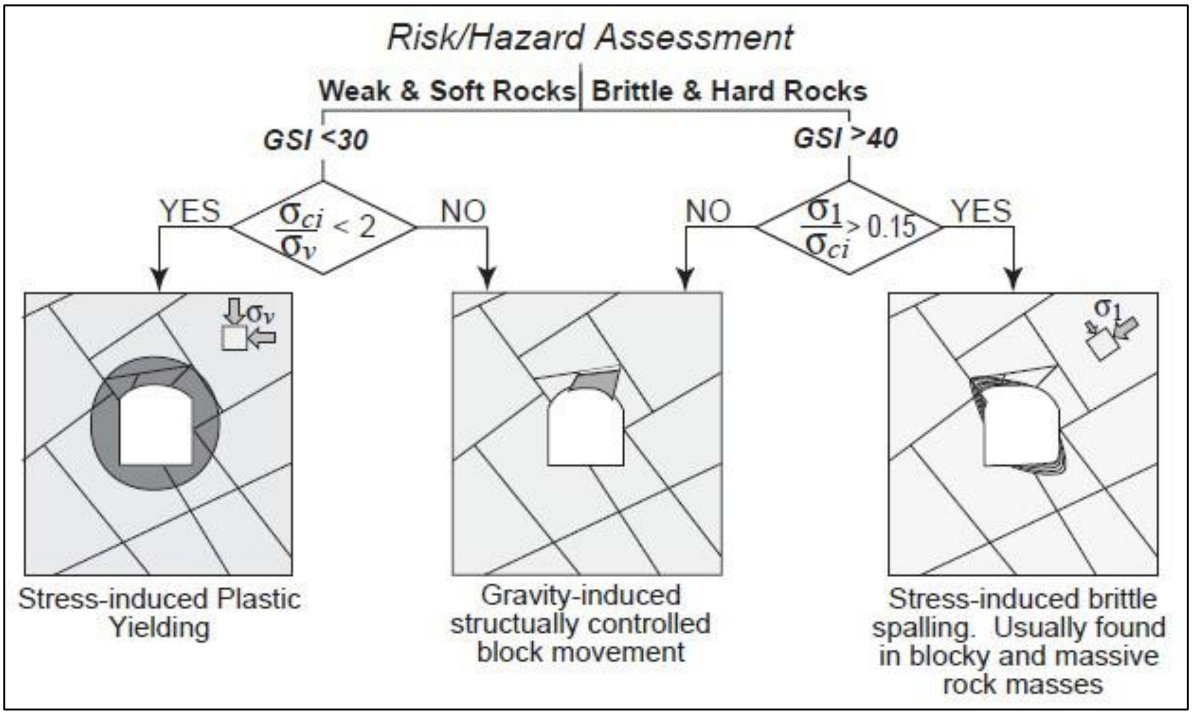


Figure 2.12 Decision tree diagram to estimate risk of failure in underground excavations (after Martin. C et al, 2003)

2.9 Numerical Modelling

Numerical modelling techniques enable some approximate solutions to problems which alternatively cannot be solved by conventional methods, such as complex geometry, material anisotropy, non-linear behaviour, in situ stresses. Numerical evaluations allows for modelling of material deformations and failure, modelling of pore pressures, creep deformation, dynamic loading, assessing effects of parameter variations etc. However, there are some limitations to numerical modelling e.g. input parameters defining the models are not usually measured and availability of these data is usually of low quality. However, analysis must be performed by experienced users with good modelling practise and also user must be aware of boundary effects, meshing errors, computer hardware capacity and time restrictions.

There are basically many types of numerical models constituting of different degrees of complexities. Some models are restricted two dimensional (2D) analysis, assuming plane strain or plain stress conditions, while other analysis can handle three dimensional (3D) geometries. Numerical models are usually better suited to analyse problems with specific behaviour and characteristics because of the mathematical concepts used in the development of these models.

Two different approaches can be assumed when modelling the behaviour of rock mass. These approaches are the continuum approach and the discontinuum approach. There is also another category which is the hybrid modelling. The choice of these methods is determined by many specific problems, mainly on the problem scale and fracture system geometry of the rock mass. The continuum approach basically assumes the rock mass under analysis to be a continuous medium with little or no significant geological discontinuities. On the other hand, the discontinuum approach assumes the rock mass to be composed of an assemblage of rock blocks that can slide and rotate against each other.

Although there are different approaches to numerical modelling, the approach adopted in all these methods is to separate the problem domain into small physical and mathematical components and then combine the influence of these components to estimate the behaviour of the entire system. The series of these mathematical equations are then solved approximately during this process.

2.9.1 Continuum Approach

In this approach, the assumption implies that at all points in a problem domain, the materials cannot be forced apart or broken into individual pieces. All material points originally in the neighbourhood of a particular point in the problem domain remain localised in the same neighbourhood throughout the deformation process. In relation to the assumed response of the medium being loaded, different kinds of rock mass properties can be assigned by way of constitutive equations for the material to the region under investigation. The properties that can be modelled comprise the following constitutive behaviours; linear elastic, non-linear elastic, elasto plastic, linear viscoelastic, anisotropic, elasto-visco-plastic, dilatant, stochastic and thermal-dependant (Bieniawski, 1984). According to Potvin, (1988) linear and non-linear elastic media is usually assumed in hard rock mining analysis.

In the rock mass where the excavation is created, the effect of stress distribution in that medium will be evaluated at discrete points inside this medium or at the excavation boundaries. This is achieved by the use of differential equations of equilibrium together with equations of strain compatibility. Therefore, depending on the mathematical method used, these continuum models can be categorized into two groups. These groups are the differential methods which approximate the solution for the whole domain and the integral methods which approximate the solution at the problem boundary only.

2.9.1.1 Differential methods

In this category, the entire region under analysis is divided into a mesh of elements characterised by various shapes and surface areas. Under differential methods there is finite element methods (FEM) and finite difference methods (FDM) (Potvin, 1988).

Finite element methods (FEM)

In this case the continuum is approximated as a series of discrete elements that are connected to each other only at precise shared points known as nodes. The load originally applied to the region is thus transferred along this network of elements and the transmission of these forces from one element to the other is completely defined by the interaction taking place at the nodes of these elements. The behaviour of each element is described independently using exact differential equations and thus the global behaviour of the material is represented by combining all individual elements. As a result the problem is then analysed as a group of nodal forces and displacements for a discretised region.

FEM is possibly the most versatile of all the methods and is basically capable of producing the most reliable results even in intricate geo-mining conditions and is capable of simulating non-linear elastic, plastic and heterogeneous material behaviours (Potvin, 1988). To obtain a more accurate solution, the mesh should be as fine as possible and constructed with smaller elements.

During the modelling process, a far field boundary of the region must be arbitrarily defined because the medium is not assumed infinite. However, the stress conditions in this far field may not be completely satisfied and this may result in some inaccuracies in the solution of the model. Other limitations of this method is its complexity in problem formulation, requirements of longer computer times and large memory capacities.

Finite difference methods (FDM)

The continuum is represented by a combination of discrete grid points at which displacements, velocities and accelerations are determined. The displacement field is computed by approximating the differential equations for the system as a set of difference equations (central, forward or backward) that are solved discretely at each grid point. The differential equations are approximated through the use of difference equations. FDM is best suited in solving dynamic problems and is rarely used in static problems.

2.9.1.2 Integral methods

In the integral method, only the contour of the excavation inside the region is to be discretised. This result in a reduction of the size of the problem by a significant magnitude as well as greatly simplifying the input requirements and therefore making the method useful in solving complex three dimensional problems. Variation of this method is the boundary element method (BEM) and pseudo-three dimensional displacement discontinuity model (Potvin, 1988).

Boundary element method (BEM)

The name is derived from the fact that the user "discretizes" or divides into individual elements the boundaries of the problem geometry only. These boundaries include excavation surfaces, joint surfaces, material interfaces and the free surfaces of shallow problems. In BEM, the boundary of the excavation is divided into linear (2D models) or surface elements (3D models). The effect of the stresses on the boundary is calculated from one element to the other using boundary integral equations. The stresses acting at all points throughout the remaining medium inside the region can thus be approximated from the boundary solutions.

BEM is usually useful when the rock mass material is homogeneous and isotropic and the method assumes the medium to be infinite or semi-infinite. The method is also capable of dividing the region into homogeneous sub-regions that are then assigned with different linear material properties. This particular capability of the method is useful in modelling mining problems where the hanging wall, footwall and ore have different rock characteristics.

Advantages of this method is that it is faster and simpler. Its limitations are attributed to the fact that the method is not powerful enough to provide sufficient room for complex geometry and excessive differences in the rock mass properties (Brown, 1987).

Pseudo-three dimensional displacement discontinuity model

This method is commonly applied to mining problems and the orebody is discretised into a grid of 2D squares elements. The third dimension of these elements is the width of the orebody which must be relatively small compared to the size of the problem so that the model can produce an accurate solution.

2.9.2 Discontinuum Approach

When the rock mass structure is relatively small or large as compared to the material structure a continuum approach is justified. In intermediate situations the behaviour tends to assume the characteristics of a discontinuum. In this scenario, the use of an approach of analysis that sufficiently models the load deformation responses of the separate discontinuities and allows for respective specific features of discontinuum patterns (Stewart & Brown, 1984). Discontinuum models have important features in that their deformation characteristics can either be rotational, extensional or sliding depending on the orientation, dip and the stiffness of the individual discontinuities. On the other hand, the interlocking behaviour of the blocks or the individual failure of the blocks can result from load displacement characteristics.

An example of the discontinuum approach is discrete element method (DEM). This method for modelling a discontinuum is relatively different compared with FEM, BEM, and FDM, and focuses mainly on applications in the fields of fractured or particulate geological media. The idea of DEM is to typify the fractured material as a collection of blocks formed by linked fractures in the problem domain, and solve the equations of motion of these blocks through continuous detection and treatment of connections between the blocks. The blocks can be deformable or rigid be with FEM or FDM discretizations. The DEM was described by (Brown, 1987) as a method that employs a dynamic relaxation procedure to solve Newton's laws of motion in order to decide the forces between, together with the displacements of, units during the progressive, large scale deformations of the discontinua.

DEM is desirably suited to modelling of materials of both large scale geological structures including faults, dykes and highly fractured assemblages of rock blocks. e.g. UDEC, 3 DEC.

2.9.3 Summary

Numerical modelling is a very promising and effective tool in understanding the rock mass response subjected to complex loading conditions. Efficient use of this tool for reliable design and fixing of strata management problems requires a thorough knowledge of the modelling theory, scope and limitations. Figure 2.13 is a diagrammatic representation of the cases under which the continuum and discontinuum approaches are implemented during interpretation, design and prediction of different kinds of rock mass problems.

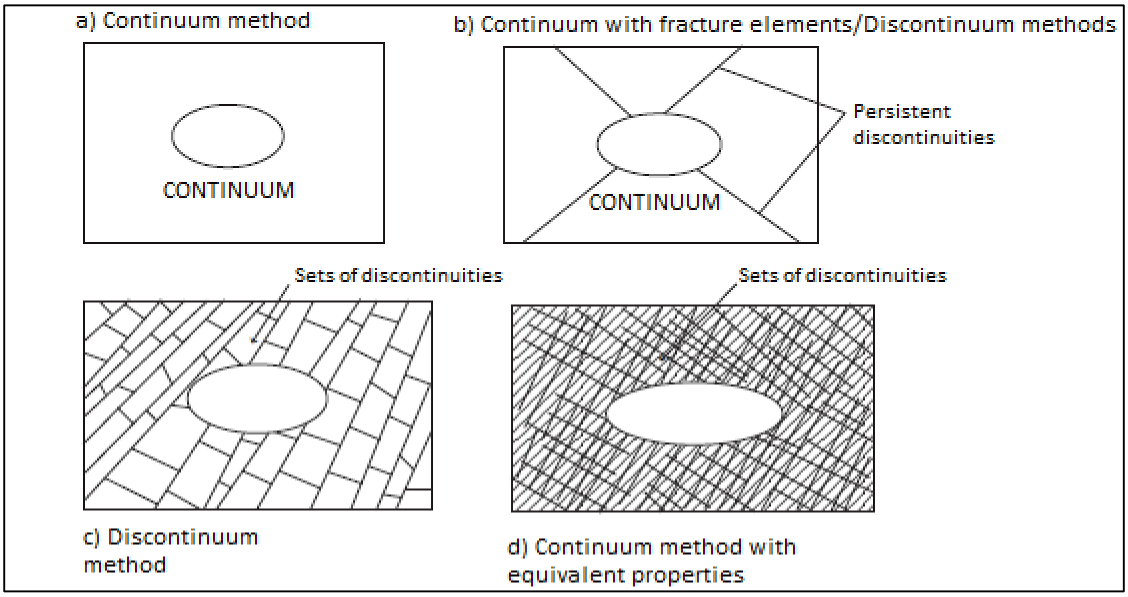


Figure 2.13 Diagrammatic representation of continuum and discontinuum approaches for different kinds of rock mass problems.

CHAPTER 3 RESEARCH METHODOLOGY

3.1 Introduction

This Chapter serves to support the research and helps in highlighting practical means of collecting all the relevant information that is necessary to evaluate the outcome of this research work. The data collection instruments used and the data analysis techniques are also highlighted in this section.

3.2 Research site

In order to properly accomplish the goals set in this research work, several site visits were undertaken to KCM in Chililabombwe. This was done for the prime and obvious reason to familiarise with the operations of the mine as well as to perform several data collection activities necessary to fulfil the goals of this research work. Several departments of the mine were consulted that are directly linked to the data and information needed in this research and among these departments are Mining, Geotech, Geology, Planning and Mineral Processing. Other specific visits were also done to the backfill plants, old hydraulic fill plant (East plant) and new hydraulic fill plant (West plant) at No. 1 Shaft, hydraulic fill tank at No. 3 shaft and waste rock crushing and grinding plant (WCGP) located at No. 3 shaft. The visits to these units was to enable an appreciation of the capacities of these plants in a bid to later make suitable recommendations on backfill material in case the research concludes that backfilling of stopes is vital in the steep dipping areas.

Underground visits were the most important ones because in the underground operations is where much of the data to input in the computer model was collected.

3.3 Desktop studies

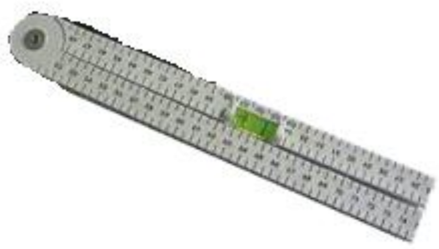
Detailed and extensive literature review was done in order to gather information related to this research work. Information from the Konkola mine reports from different departments was reviewed in order to have a feel of the background to the backfill requirements and also to get other useful information to use in the model. Other information from accredited journals, publications, textbooks and the internet was thoroughly reviewed to get the latest backfill systems, stope design and stabilisation practices from research work and other operating mines.

3.4 Data collection and instrumentation

According to (Hoek, 1986), data collection is the backbone of any practical rock mechanics analysis such as this study. Therefore it is very vital to collect as much information as possible in relation to the available and notable features of the discontinuity.

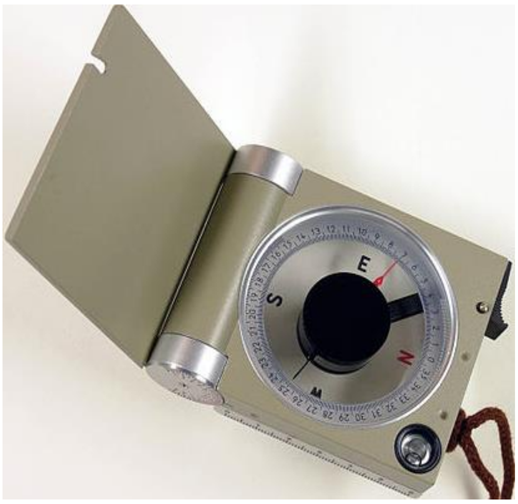
Data collection instruments therefore refer to the devices used to collect data needed to address the research questions and objectives. Instruments used in this research included mapping techniques for example scanline for measuring the joint pattern of the rock using a geological compass (Clar compass) indicated in Figure 3.1. UCS apparatus was also used to obtain data to be used in numerical modelling and the empirical method for data analysis requirements. For this purpose, a simple point load testing machine was utilised. A geological hammer was also used to estimate the UCS of the different rock formations. Core logging equipment such as measuring tape, pen knife and clinometer rule were used to characterise the different rock formations.

Data collection followed a systematic procedure in which the research site under study was visited. However, through the discussed instruments, measurements of the required rock strengths and other rock parameters required in the analysis was done.



(c) Clinometer rule

(a) Point load testing machine



(b) Clar compass

Figure 3.1 Data collection instruments

3.4.1 Data collection techniques

Generally there are only two sources of data, which is primary data and secondary data. Primary data is normally collected using surveys (questionnaire, interview, and standardized scales/instruments), experiments or direct observations. Secondary data collection may be undertaken by collecting information from a diverse source of documents or electronically stored information. Various techniques are available and the list below includes the different data collection techniques employed in the study for the collection of in-situ data:

- Using available information
- Core logging
- Structural mapping (scanline surveying)

- ❖ Laboratory testing

3.5 Laboratory testing of rock samples

In order to develop a proper model for the backfill requirements and timing at KCM, rock tests such as UCS were conducted on the rock samples from the steep dipping areas of Konkola underground mine. Rock materials to be used included that taken from the footwall, hanging wall and ore body.

3.5.1 Uniaxial Compressive Strength (UCS) tests

Uniaxial compressive strength was conducted in accordance with the International Society of Rock Mechanics (ISRM) standard as input parameters for numerical analysis. Samples were collected from core logs and then subjected to compressive testing using a simple point load testing machine. Based on these laboratory tests, the compressive strength (σc_i) and other rock deformational parameters were also obtained. These parameters include, Young modulus (E) and Poisson's ratio (v). And because the standard compression testing machine was not available, the parameters were estimated from RocLab (Rocscience). All these rock parameters were needed as input parameters in the numerical analysis performed using PHASE 2 computer softwares.

3.6 Software application and purpose

In order to perform the numerical modelling, computer software programs including PHASE 2 (Rocscience) and RocLab (Rocscience) were utilised. PHASE 2 was used to model the stopes at different mining levels and the purpose of RocLab was to estimate the Hoek-Brown strength parameters needed as input parameters into PHASE 2.

3.6.1 Numerical modelling procedure and model parameters

Stopes at different depths were modelled and analysed using the program PHASE 2 (Rocscience, 2002). The parameters needed to effectively formulate and run a stope model to analyse the stress magnitudes and distributions, displacements and issues of stability include the in-situ stress regime, UCS of the intact rock, Hoek-Brown strength parameters, rock mass deformability properties and stope geometries.

The modelling was done on different levels in the steep dipping areas. The levels were selected on the bases of the forecasted planning schedules which are to reach 1350mL as per Life of Mine plans (LOMP). Stopes cross sections along the dip of the orebody were modelled.

Different depths were selected to show the effect of the stresses on the stope stability as the extractions progresses downwards.

The models were developed, computed and analysed for levels starting from 950mL, 1050mL, 1150mL, 1250mL and finally 1350mL. The effects of stress with respect to depth was then analysed for these distinct levels using sigma contours, strength factors and displacement graphs from the numerical software.

3.6.1.1 Hoek-Brown constants

In order to determine the values of the parameters m_b, s and a, the relationships developed by Hoek et al (1995) described under Section 2.7 were utilised. These relationships depend on the RMR of the rock mass or GSI and therefore core logging was done at the Konkola Geotechnical Department to determine the RMR of the different rock masses in the steep dipping areas. The process of determining the RMR was done under the direct supervision of experienced geotechnical personnel at Konkola in order to obtained more accurate results. In order to get more generalised values, these parameters were also estimated from a similar set of calculations using the program RocLab.

The procedure for core logging was conducted on cores from Bancroft North. The exact location of the drill core was 1020mL, 468mN and all the joints and discontinuities within the different rock formations was carefully analysed to obtain correct joint conditions parameters to be used in determining the modified *Q'* for the Stability graph analysis. The RQD of the different formations were determined using a tape measure to sum all the core pieces greater than 100mm. The cumulative length was evaluated as a percentage of the total core run for the specific rock formation to give the RQD of the rock mass.

Measurement of the RQD on drill cores was done using tape measure and part of the Copper Piloting 813 (CP 813) drill cores are shown in Figure 3.2 and the whole core logging sheet is illustrated in Appendix 4.

Figure 3.2 Drill cores and core logging of CP 813 drill cores

3.6.1.2 Stope dimensions measurements

Stope dimensions are a critical parameter that is crucial in the formulation and running of the computer software model that is necessary in making informed conclusions on the behaviour of the stopes underground. KCM does not have the cavity monitoring system (CMS) which is a tool that can accurately measure the stope geometries and therefore the stope dimensions were obtained from design standards from the mining methods used in the steep dipping areas.

Therefore, in order to obtain a more accurate representation of the stope sizes and inclinations, Digital Terrain Models (DTM) obtained from the Mine Planning department were utilized. These DTMs are solids outlining the stopes and thus vertical cross sections were cut using Surpac to come up with strings showing the accurate cross sections to be used in the numerical modelling. Cross sections were cut from 200mS and 700mS and it was assumed that the stopes will follow this similar cross sections as depth of mining increases because solids beneath 1040mL were not available. However, for the sake of model simplification, the stopes were assumed to be even although in reality it is not so but the difference is of minimum significance.

3.7 Empirical analysis and calculations

This empirical approach was also selected to be included in the research because making conclusion on the stope behaviour based only on the Numerical Modelling using Phase2 cannot be generalised. However, numerical modelling and empirical design constitute the two broad general approaches for assessing underground stability in rock. Thus this approach was found useful as it has been used in several stope designs. Therefore, in order to decide whether a stope is stable there is several information that should be determined to be used in the evaluation of the stopes using the Stability Graph Method described in Section 2.6.1. As a result, the stability of the opening is detected by the rock mass strength and structure, the stresses around the opening, size, shape and orientation of the stope.

3.7.1 The Stability Number

The stability number can be calculated using the following four parameters:

- Modified *Q* Tunnelling Quality Index,
- Rock stress factor,
- Joint orientation adjustment factor, and
- The gravity adjustment factor,

3.7.1.1 Modified Tunnelling Quality index, Q'

This parameter can be determined from the outcomes of structural mapping of the rock mass in exactly the same manner as the standard Q index developed by Barton et al (1974). RQD of the different rock mass formations was determined from the core logs already sampled from the underground operations. This was performed at the Konkola Copper mine core shade at No.1 Shaft.

The other parameters needed for evaluation of Q were also evaluated with the help of geotechnicians at Konkola through structural mapping in underground operations and core logging of cores sampled from 1020mL in the Bancroft Deeps. These include the RQD, joint set number (J_n), joint roughness number (J_r), joint alteration number (J_a) and the joint water reduction factor (J_w) which are needed to calculate the parameter *Q'* as indicated by Equation 2.2. From the core shade, all the joint characteristics were described and this data was then used to obtain ratings from tables developed by Barton et al. This was aided by underground scanline surveying to also obtain accurate joint orientations and the conditions on these joint sets.

In the evaluation of *Q'*, the stress reduction factor was omitted as the influence of the stress will be catered for under the rock stress factor parameter.

3.7.1.2 Rock stress factor

This factor is calculated from the ratio between the unconfined compressive strength of the intact rock, σ_{ci} and the stress acting parallel to the exposed face of the stope under consideration. Since the stope face spans across different types of rock formations, an average of this strength values was used. Individual UCS values were obtained from test described under Section 3.7.1.1. The induced compressive stress, σ_1 was determined using numerical modelling for the specific conditions using the program PHASE2. The induced stresses as observed from the different models of the 950mL, 1050mL, 1150mL, 1250mL and 1350mL were compared to the UCS of the hanging wall quartzite and footwall conglomerate to obtain the rock stress factor.

3.7.1.3 Joint orientation adjustment factor

The joint orientation adjustment factor considers the influence of orientation of critical geological structures intersecting the open stope surface. The critical joint orientation was obtained from core logging and structural mapping. Scanline surveying was done underground and this procedure is very useful since it can provide all discontinuity features of concern and it provided first hand in-situ data required for the purpose of stability analysis. A three dimensional scanline survey technique was used in order to eliminate biases resulting from favoured orientation of the joints. The scanlines were therefore placed along the draw point cross-cut as well as along the trough drive which is mined within the ore body. The measuring tape was fixed to the rock face at waist height using small lengths of wire attached to the lower end of the support wire mesh. The wires were properly spaced (approx. 3m apart) along the tape in order to keep it taut and as straight as possible.

Figure 3.3 is a plan view illustration of the scanline position used to reduce the bias of favoured orientations of the discontinuities crossing the tape measure. The trough drive is mined inside the Oreshale (orebody) while the draw point cross-cut cuts across the following formations; PC, FWSST, FWC and Oreshale.

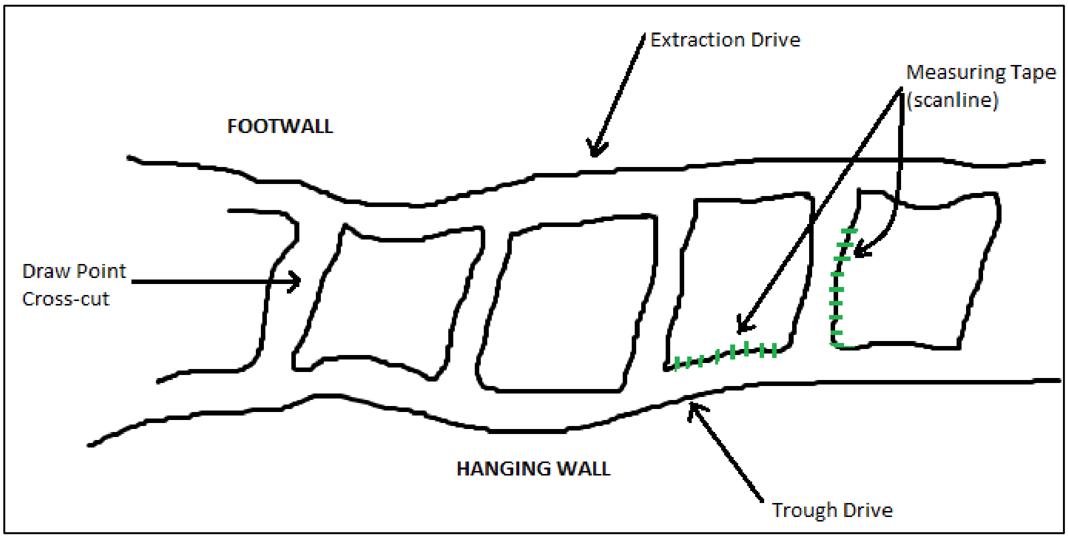


Figure 3.3 Plan view illustration of scanline location during underground structural mapping

The orientation of the discontinuities intersected with the tape was determined using a Clar compass. The reading was recorded by placing the folding lid of the compass against the joint plane under consideration and the body was levelled using the target bubble. The orientation, dip direction and dip, of the joints crossing the tape measure was recorded alongside their distance along the scanline. The conditions of the joints in relation to the type, persistence, type of filling, roughness, degree of weathering, ground water, etc was also recorded on the mapping log sheet.

The stope face dip was obtained from design plans and relating this to the orientation of the oreshale bedding. The true angle between the face and joint poles was then determined using stereographic nets (equal area polar nets) and the results was then used to evaluate the joint orientation adjustment factor as illustrated under Section 2.6.1.1.

3.7.1.4 The gravity adjustment factor

This parameter is dependent on the orientation of the stope face. To determine the stope face orientation, measurements of the stope geometries available from designs of the stopes for the steep dipping areas were used. The dip of the stope face was therefore obtained from plans prepared as per mine designs.

3.7.2 The Hydraulic radius, S

The value of the shape factor was determine from the dimensions of the stopes from available stope design plans. Therefore the different values of this parameter for specific locations was thus calculated from the simple relationship of dividing the stope surface cross sectional area by the perimeter of the same surface.

3.7.3 Stability Evaluation of stopes at different depths

A standard stability graph developed by Potvin, 1988 was then used to plot the different combinations of the stability number, N' and the hydraulic radius, S. The stability number for each stope at specific depth was then calculated from Equation 2.1 using the parameters described above as well as the respective hydraulic radius for the particular conditions. These plots on the standard graph were then used to observe the particular category or zone under which the individual stopes could follow as the depth from the surface increases. The results from this empirical approach was therefore used to make detailed comparisons with the analysis that resulted from the numerical modelling process.

3.8 Data analysis instruments and procedures

After the collection of data, it was then analysed and organised to produce useful and meaningful information. An elasto-plastic finite element program or a numerical code based on finite element method called PHASE2 (RocScience, 2002) was used for calculating and modelling stresses, and displacements around the underground openings. The computer package was used to analyse and interpret the visual graphics models of stress distributions around the excavations taking heed of the distances and interactions between the openings. Graphs and tables were also used for easy interpretation of the resulting information.

Results from the numerical modelling and the stability graph method was then compared to each other to produce a more generalised conclusion of the behaviour of the stopes in the steep dipping areas.

3.9 Summary

The whole methodology process undertaken to make conclusions and recommendations for this geotechnical study is summarized in the chat presented in Figure 3.4.

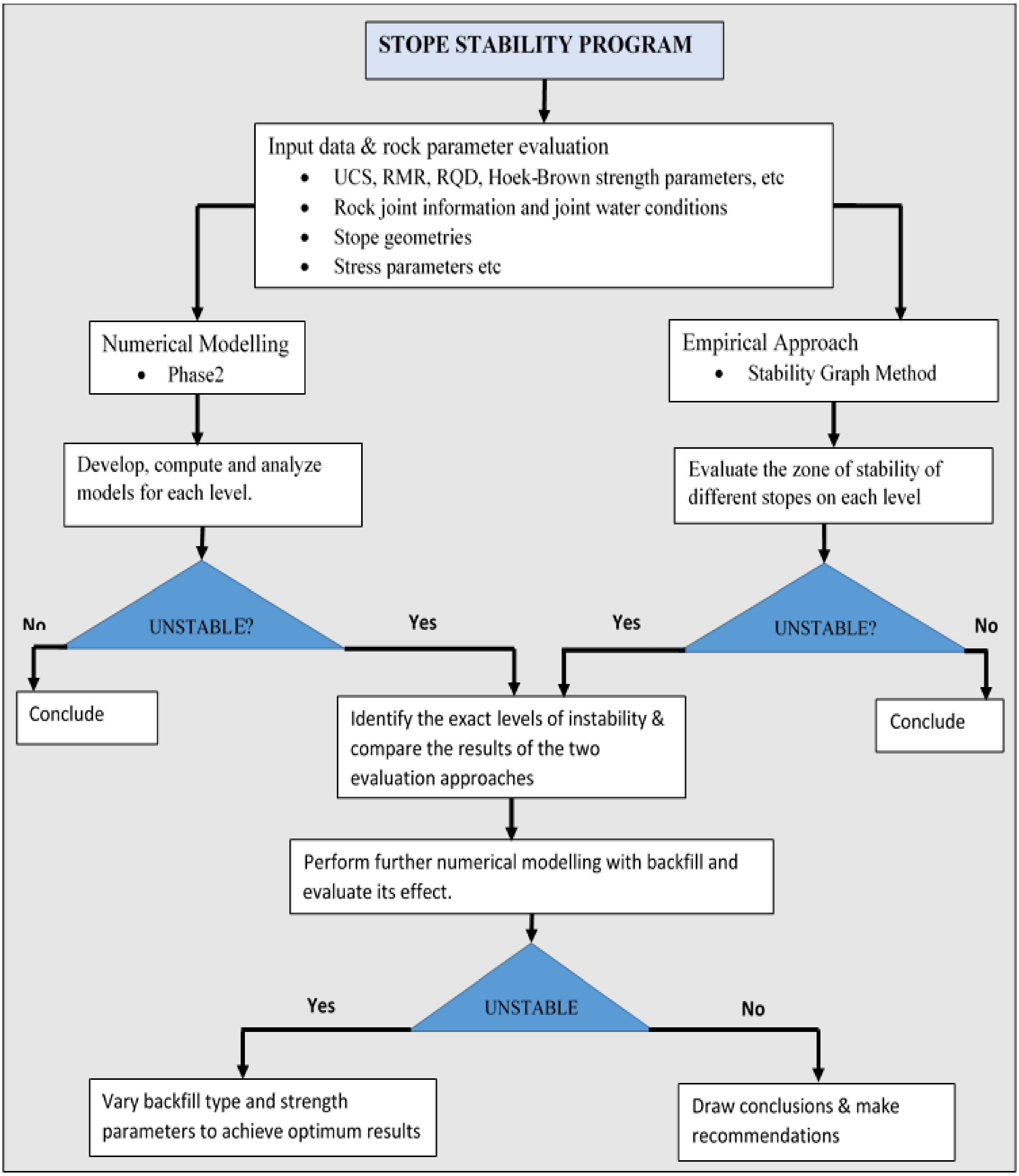


Figure 3.4 Flow chart showing summary of methodology

CHAPTER 4 DATA COLLECTION & ANALYSIS

4.1 Introduction

This Chapter describes the various laboratory and field tests work that were performed at Konkola Mine. These activities were done on the available core logs and underground draw point cross cuts and trough drives to evaluate the necessary input data to be used in the numerical model and empirical methods utilized for evaluating the stope performance. However, other secondary data from Konkola mine reports was analyzed and evaluated in order to establish a more generalized database for the input parameters into the two evaluation methods.

4.2 Input Data for numerical modelling

4.2.1 Stope geometry and mine design assessment

The modelling of stopes using Phase2 numerical software was performed in the areas indicated in the diagrams below. Figure 4.1 is a schedule indicating the entire longitudinal Konkola mine deposit with the zones showing the extent to which the reserves are developed.

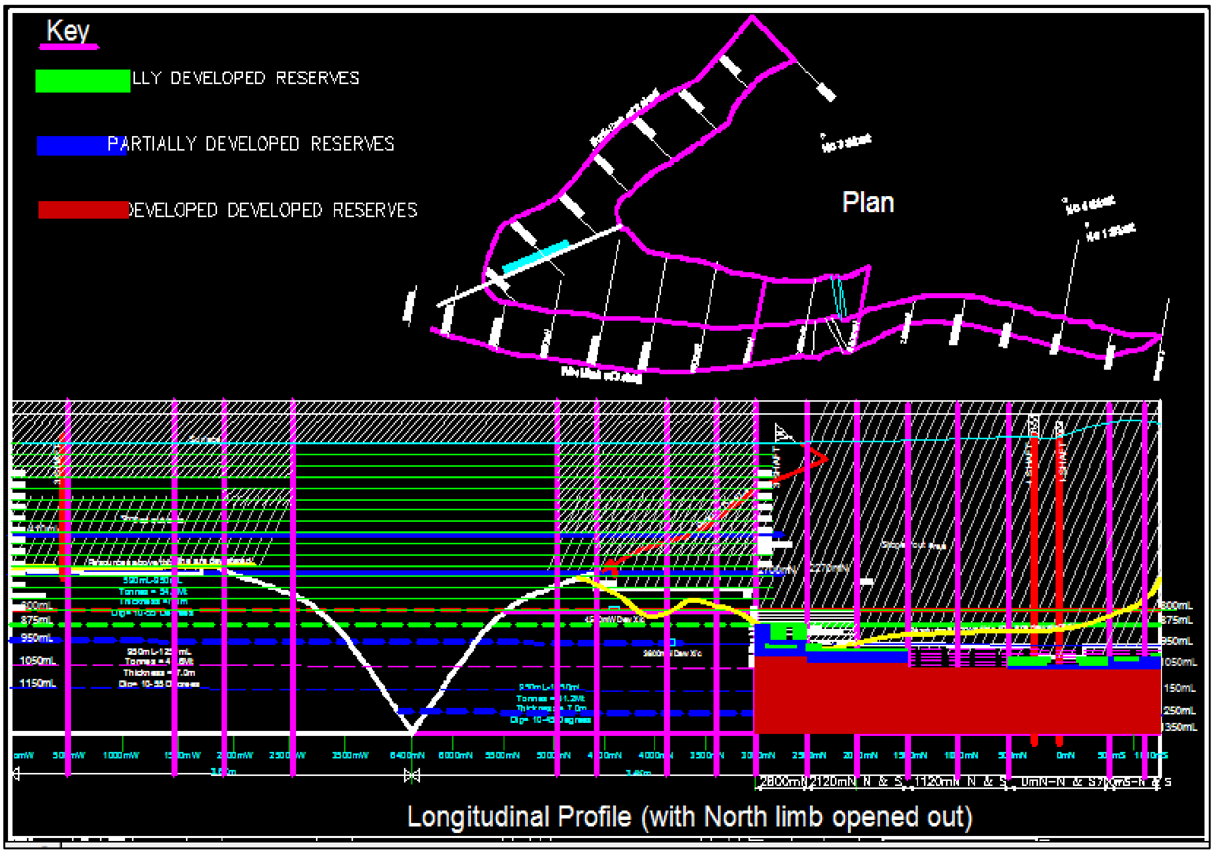


Figure 4.1 Longitudinal outline of the KCM deposit showing extent to which reserves are developed (KCM Life of Mine reports)

The stopes selected for the purpose of this study are those from the steep dipping areas. From Figure 4.2 the area indicated as Bancroft was chosen for the purpose of this study because it is within this portion of the mine where the ore body is steeply dipping with dips ranging from 40-70^0. Stopes from this area were modelled and analysed for the purpose of drawing conclusions on whether backfilling is necessary. However, Figure 4.2 is an outline of the Konkola deposit in plan view.

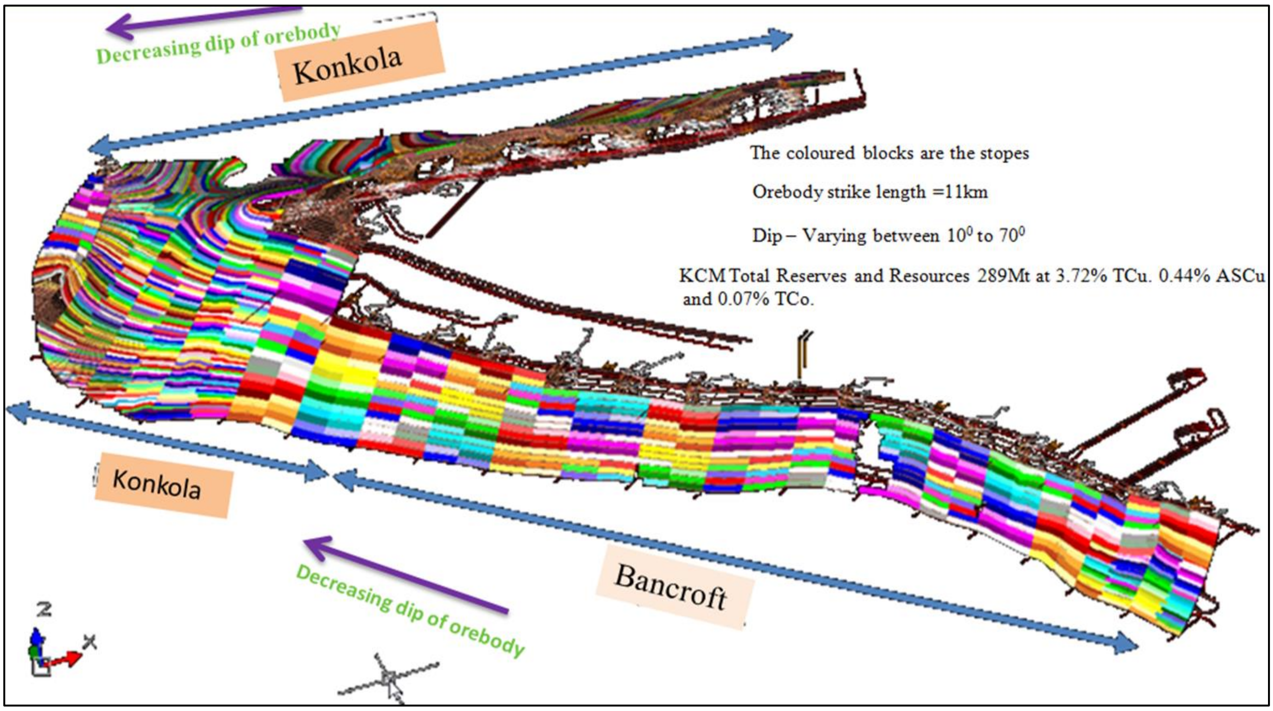


Figure 4.2 Plan view of KCM deposit showing the Bancroft area (KCM reports, 2016)

However, to clearly have a picture of the longitudinal section view of the mine developments, Figure 4.3 is an extraction from Figure 4.1. The diagram also is a clear indication of the extent, in terms of depth, to which the Konkola Mine is planning to extend its operations. It is seen that scheduling of stopes is planned to reach 1350 mL and it is to this depth that the modelling and evaluation of the stress distributions around these stopes was done.

Modelling of the stopes using PHASE2 was done on the cross section of the orebody. Surpac designs from the planning department showing stopes from 950mL to 1040mL were used in order to get an accurate representation of the cross sectional profile of the stopes in these areas.

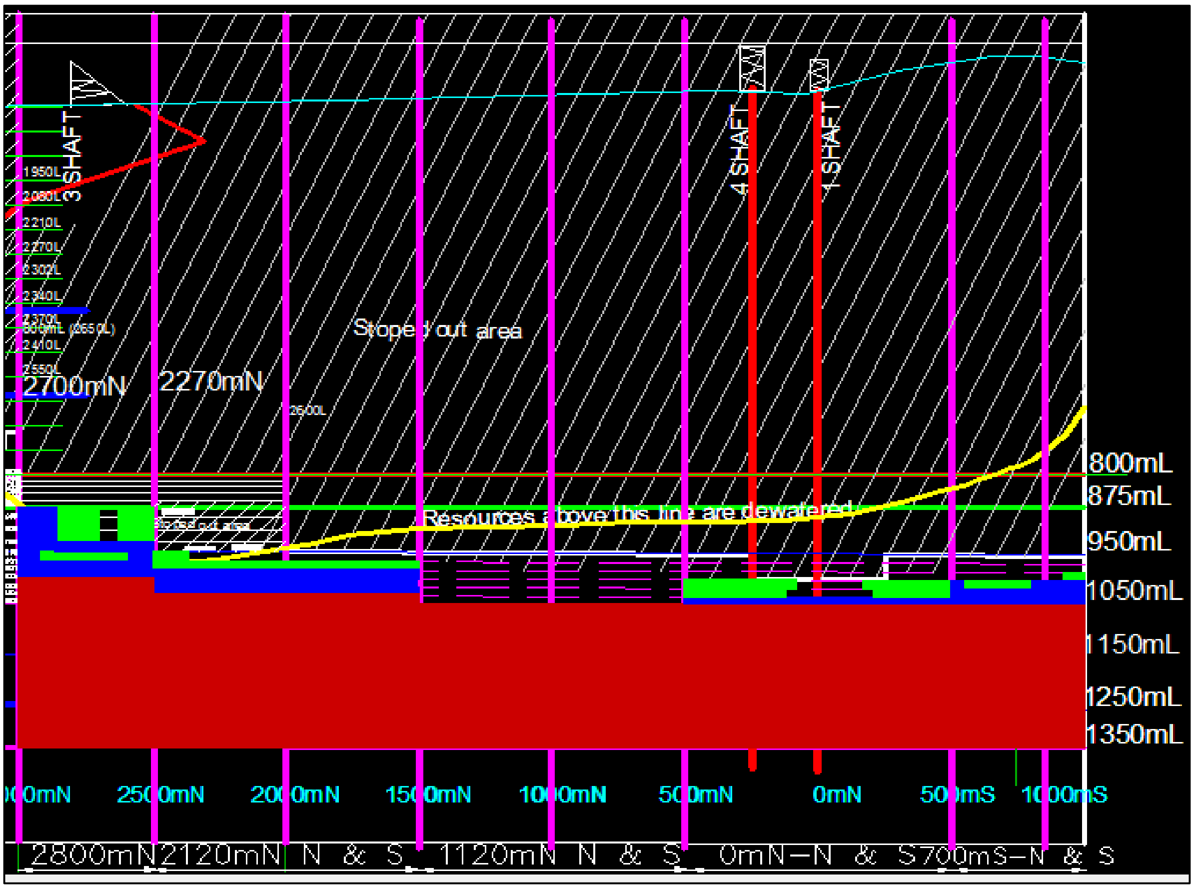


Figure 4.3 Illustration of the extend of depth of current LOM plan

Stopes analyzed in this research are those located in the steep dipping areas of Konkola mine. The research therefore focused on stopes from the Bancroft Deeps and Bancroft North underground mine areas. Table 4.1 is an indication of the details of the stope designs obtained from comparing the design mine plans for the longhole open stoping method used in the steep dipping areas of Konkola and the cross sections from Surpac.

Table 4.1 Design stope dimensions

			Stope dimensions		
Dip (^{0})	**Ore shale width(m)**	**Level height(m)**	**Height (m)**	**Width (m)**	**Length, along strike(m)**
55	6 (7)	20	20	8	20

Figure 4.4 is an illustration of the cross sections from Surpac and it was taken into account that for the modelling process, the stopes were assumed to be even. Measurements from inquiries

performed using Surpac on these sections indicated that most of the stopes are sloping at 50^0 to 60^0 with thicknesses ranging from 6-8m. However, intervals between the levels was averaging between 18-22m.

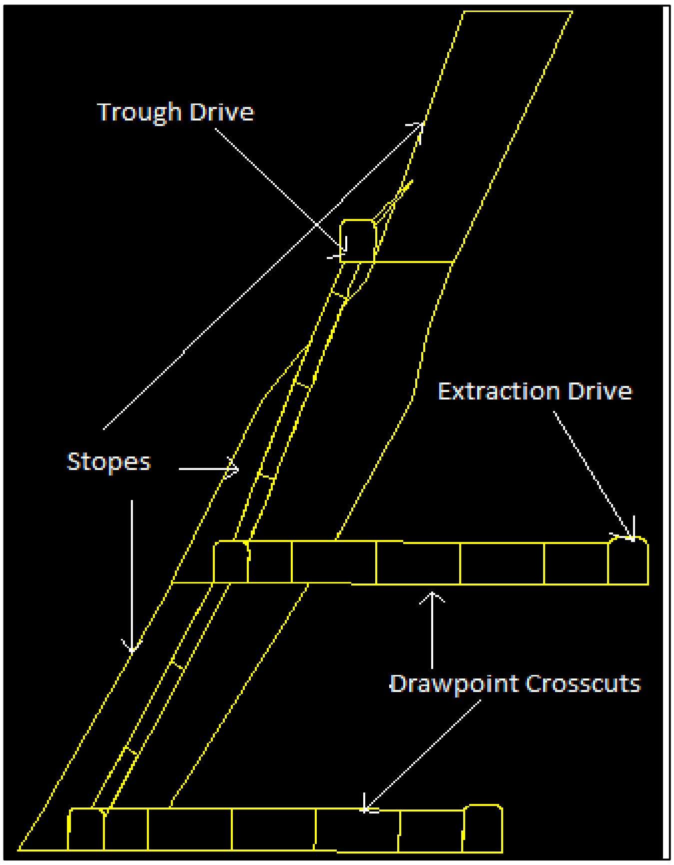


Figure 4.4 Surpac string cross section showing vertical outline of stopes

4.2.2 Rockmass quality

The orebody consists of various inherent geological properties of the rock mass and stresses that result in different variations of ground conditions over the whole mine thus affecting the stability of the different types of underground openings including the stopes. The mineralized zone is stratform and distributed over a wide area of nearly 7m width and approximately 11km along strike.

The ore shale deposit is hosted within a series of rock formations. Table 4.2 is a summary of the descriptions of these different rock formations that was provided by the Konkola mine geotechnical and geology departments.

The other rock mass parameters and Hoek-Brown strength parameters needed for the purpose of modelling was analyzed using the program RocLab. The input parameters used in RocLab software were all estimated from the built-in charts and tables in within the software framework and these are based on the rock type and the geological conditions at Konkola. This excludes the UCS of the intact rock as it is available from previously tested rock samples from all the geological formations as indicated in Table 4.2.

For the purpose of finding the thicknesses of the different rock formations needed as input data into the Phase2 model, data from core logging and underground structural mapping was used. Reliable estimates of the thickness of the formations was therefore obtained by combining these widths with those obtained from geological reports. Therefore, the average thicknesses of the formations are also summarized in the Table 4.2.

Table 4.2 Descriptions of the rock mass formations at KCM (KCM reports, 2015)

Formation	**General Descriptions**	**RMR Range**	**UCS (MPa) Range**	**Avg width (m)**
FWQ	Very strong, few joints, fresh to slightly weathered, competent ground.	80-90	148-551	320-400
AGSST	Hard with numerous rough and smooth joints, competent rock, intercepted in some portions with weak kaolin bands.	61-70	80-200	45-70
PC	Poorly to fairly grained, moderately consolidated, moderately jointed, slightly to moderately weathered.	35-50	12-240	5-15
FWSST	Hard with numerous rough and smooth joints, competent rock.	61-70	61-478	10-15
FWC	Coarse grained, poorly consolidated, moderately jointed, and slight to moderately weathered.	45-70	14-353	5-12
Oreshale	Hard, moderately jointed, thin to thickly bedding with kaolin infill in some portions.	25-50	20-399	5-20
HWQ	Formation of quartzite and dolomitic sandstone bands, which in some places completely koalinized, giving poor hanging wall condition.	45-75	59-307	30-150

Table 4.3 is a summary of the rock mass strength parameters estimates obtained from RocLab program that were used for the purpose of modelling the stopes. The factor D is dependent upon the degree of disturbance as a result of blast damage and stress relaxation. The factor varies from 0 (representing undisturbed in situ rock masses) to 1 (for very disturbed rock masses). Guidelines for the selection of factor D are described under Appendix 3.

The GSI of the rock mass was also estimated from Appendix 1. Values of UCS were average estimates obtained from UCS tests, Konkola mine reports and those normally used by consulting companies in other analyses.

Table 4.3 is a summary of rock formations that are close to the Oreshale because it is in these close proximities where the effects of stoping are to be seen.

Table 4.3 Rock mass parameters

Hoek Brown Classification	**HWQ**	**Oreshale**	**FWC**	**FWSST**	**PC**	**AGSST**
UCS-Intact (MPa)	170	110	130	175	140	135
GSI	55	35	55	60	50	50
D	0	0	0	0	0	0
m_i	20	6	21	17	21	17
m_b	4.009	0.589	4.210	4.074	3.521	2.851
s	0.0067	0.0007	0.0067	0.0117	0.0039	0.0039
a	0.504	0.516	0.504	0.503	0.506	0.506
E (MPa)	26000	2500	18500	25000	15000	11400
v	0.2	0.2	0.2	0.2	0.2	0.2

The RocLab screenshots for the estimation of all these rock mass parameters are shown in Appendices 6 to 9.

4.2.3 Stress Field

The in-situ stress field is an important parameter in numerical modelling. According to KCM reports, very limited in-situ measurements of stress have been carried out on the Zambian Copperbelt mines. The stress field used for modelling in this research was obtained from the in-situ measurements that were done at Konkola mine in 2001 by Rock Mechanics

Technologies (RTM). Three measurements have been done at Konkola Copper Mines. One was at No.1 Shaft and two were at No 3 Shaft. According to the Konkola reports, this stress measurement process was performed using CSIRO HI cells at these three different locations of the mine.

The conclusions from the measurement reports are that:

- ❖ No 1 Shaft (950mL/2700mN) was in an unmined area and therefore, the test results gave in-situ stresses; and
- ❖ The two sites at No 3 Shaft (590mL/275mW and 590mL/2160mW) where close to the stoped out areas and thus gave high induced stresses since the test results were affected by mining.

The test results were analyzed by making comparisons with measurements from other nearby mines to provide a guideline for using them in all future rock mechanics design projects and modelling work at Konkola mine.

The stress measurement data, despite not being perfect in all aspects, provided useful parameter, which can be useful for numerical modelling and stability analysis. Konkola mine therefore made some following recommendations when working out any rock mechanics design:

- ❖ Vertical stress - stress due to overburden assuming 0.026 MN/m^3 + 4.0 MPa* (* this accounts for an additional 150m of overburden on top of the depth from surface under analysis.
- ❖ Horizontal stress, Maximum – 0.85*Vertical stress and Minimum – 0.60*Vertical stress.

Table 4.4 is a summary of the measured principal stresses at the three locations at Konkola mine. The values used in the numerical modelling are those from No 1 Shaft site. This is because the Bancroft areas of the mine are serviced by this shaft.

Table 4.4 Principal stresses at the three measured locations

Location (test No)	950mL/2700mN(3-1)		590mL/725mW(1-1)		590mL/2160mW(2-1)	
Stresses	Value-MPa	Dip/Dir	Value-MPa	Dip/Dir	Value-MPa	Dip/Dir
Major – Sigma 1	39.4	51/121	28.5	24/40	18.7	78/234
Inter – Sigma 2	17.7	17/09	22.1	18/302	7.9	00/144
Minor – Sigma 3	15.4	34/267	7.2	59/179	5.8	12/54
Bedding plane	46/215		35/355		20/295	

4.3 Stability Graph Factors

4.3.1 Modified *Q* Tunnelling Quality Index

This parameter is determined from the outcomes of structural mapping in a similar manner as the Q classification system proposed by Barton et al. Cores from steep dipping areas were logged from the Geotech core shade at No.1 Shaft. The different rock classes at No.1 Shaft were individually analyzed to come up with the joint set information and characteristics.

In order to obtain the number of joint sets for the different rock mass formations, all the individual joint orientations (dip and dip direction) were measured and recorded. This data was then plotted as poles on an equal area polar nets. The position of the mean pole for all joints in the different formations was then estimated to come up with the number of joint sets. The orientation of the discontinuities measured from 300mS, 2500mN and 2700mN locations is summarized in Table 4.5.

Table 4.5 Summary of measured joint orientations

Rock Formation	**Joint orientation (format- dip/dip direction)**
Oreshale	51/060;45/080;60/075;35/045;35/060;55/050;55/045;45/075;59/087;77/073;70/060;75/070;75/032;80/334;75/320;80/298;70/312;82/305; 71/221;50/225;60/235;55/250;45/240
FWC	45/053;65/245;40/045;55/244;54/238;52/224;38/050;
FWSST	40/070;35/075;45/080;45/070;55/065;40/085;35/070;85/300;85/250;70/350;72/255;85/295;80/290;72/120;73/287;35/105;30/040;76/065;70/060;80/065;82/070;70/065;32/250;60/035;

Apart from logging, structural mapping underground was performed to evaluate the characteristics of the rock mass. Konkola mine is amongst the wettest mines in the world pumping approximately 300 000m^3/day, but this doesn't directly give the worst rating for the joint water reduction factor (J_w). The hanging wall aquifer accounts for over 35% of the mine water while the footwall aquifer accounts for 40%. All this water is efficiently drained from underground operations and the footwall aquifer is porous making the process quiet viable. Therefore, the joint water reduction factor was noted to be mostly of medium inflow with occasional outwash of joint infill. This was determined during underground face mapping and most joints are dump with minor to medium inflow trickling from the joints.

The RQD was easily determined from the cores by dividing the core pieces greater than 100mm by the recovery (m) as recorded in the core logging sheet. These values were then related to those available in the geotechnical reports for Konkola mine for a more generalized outcome.

Joint set number (J_n), Joint roughness (J_r) and joint alteration number (J_a) were estimated from core logs and underground face mapping. The other joint descriptions are summarized in Table 4.6.

Table 4.6 Summary of joint descriptions

Formation	Joint condition description
HWQ	Moderately weathered joints, kaolin infill around the Deeps. Smooth, slight undulating and multi wavy joints. Medium to widely spaced joints (0.3-1.5m to >1.5m). Two joint sets and other minor joints.
Oreshale	Slight undulating and sometimes straight large scale expression. The small scale expression is usually smooth undulating and smooth planar with no joint alteration or rather fresh with slight surface stains and sand filled. Extremely closely spaced to medium spaced (0.02-0.6m). Three joint sets and several other minor sets.
FWC	Slight to moderately weathered joints, medium spaced (0.2-0.6m), rough, stepped and irregular joint surfaces with non-softening sand infill. Two joint sets and other minor sets.
FWSST	Multi wavy and slight undulating joints with rough planner and rough undulating joint surfaces. Close to medium spaced joints (0.06-0.2m to 0.2-0.6m) which are slightly weathered and surface staining. Three joint sets plus random.

The ratings of the different joint parameters were then obtained from tables developed by Barton et al, (1974) for classification of individual joint parameters, Appendix 5.

From these activities, the rock mass joint information is summarized in Table 4.7. The table only includes those formations that are close to the oreshale from where stoping is performed and ratings giving the minimum strength conditions were taken for the purpose of the empirical analysis.

Table 4.7 Rock mass rating and joints parameters

Rock Formation	RQD	J_n	J_r	J_a	J_w	Q'
HWQ	80	6	2.0	2.0	0.66	8.80
Oreshale	52	12	1.0	1.0	0.66	2.86
FWC	65	6	3.0	2.0	0.66	10.73
FWSST	85	12	3.0	2.0	0.66	7.01

4.3.2 Rock stress factor A

The factor A was determined by utilizing the PHASE2 stope modelling results for the stopes analyzed on the five different levels (950mL, 1050mL, 1150mL, 1250mL and 1350mL). The maximum induced compressive stress, σ_1 on the hanging wall side of all the stopes from numerical modelling was observed. This value of σ_1 for the independent levels was then divided into the UCS of the surrounding rock mass on the hanging wall side in order to obtain the ratio that was then used to evaluate the value of factor A for the individual levels by simple interpolation utilising the graph represented in Figure 2.9 under Section 2.6.1.1. The results of the calculations of rock stress factor are summarised in Table 4.8.

Table 4.8 Calculation of stress factor A

Level (mL)	Avg induced stress, σ_1 (MPa)	UCS, (MPa)	UCS/σ_1	Factor A
950	30	170	5.7	0.53
1050	35	170	4.9	0.43
1150	37	170	4.6	0.40
1250	39	170	4.4	0.35
1350	42	170	4.0	0.32

4.3.3 Joint orientation adjustment factor B

Results of joint orientations measured from structural mapping were used together with the orientation of the oreshale in order to determine the true angle between these two planes. The orientation of the oreshale from Bancroft deeps to Bancroft North (700mS to 2700mN) has a dip and dip direction of 45^0-60^0 and 222^0-246^0 respectively (Konkola Geology Department, 2016). An average orientation for this analysis was estimated to be $55^0/246^0$ (dip/dip direction). On the other hand structural mapping results indicated the orientation of the critical joint set is near vertical with (70-85^0)/ (300-350^0) dip and dip direction respectively. Therefore the angle between these planes was driven from sterionets.

The angle between the stope face plane and the critical joint was found to be 76^0 using stereographic projection. The factor B was thus easily evaluated from the graphs developed by Potvin, 1988 which are discussed under Section 2.6.1.1. This angle then resulted in a factor B of 0.9 from simple extrapolations from the graph developed by Potvin.

4.3.4 Gravity Adjustment Factor C

This factor basically accounts for the influence gravity has on the type of stope face failure. The failure mechanism of the stope faces is expected to be predominantly sliding because according to Konkola mine reports, instability is said to normally result from two intersecting orthogonal joint sets plus another third near vertical joint set with (70-85^0)/ (300-350^0) orientation. This then resulted in factor C of 2.5 to 4 taking the dip of the critical joint set as 70-85^0. This value of factor C was easily extrapolated from the standard graphs discussed under Section 2.6.1.1.

4.3.5 Hydraulic radius

The Hydraulic Radius (HR) of each stope surface along the strike was calculated from the design stope dimensions from the design wireframes. The HR is only applicable to rectangular stopes and therefore it was very easy and convenient to evaluate this parameter as the stopes under consideration are rectangular. However, HR is constant for all the stopes analyzed as the dimensions for the stopes is constant at different levels of the mine within the Bancroft deeps and Bancroft North areas.

Stope dimensions are given in Table 4.1 and from these values the HR was evaluated as follows:

$$S = \frac{Cross\ sectional\ area\ of\ stope\ surface\ analysed}{Perimeter\ of\ stope\ surface\ analysed} = \frac{(20*20)}{(2*20)+(2*20)} = \mathbf{5.00m}$$

The input parameters necessary for the stability graph method are therefore summarised in Table 4.9. These parameters were then plotted on the graph to evaluate the zone under which the stopes at different depths follow under in relation to the prevailing induced stresses acting around those stopes. The calculation for the stability number is fully described under Section 2.6.1.

Table 4.9 Summary of the stability graph parameters

Depth/Stope Level (m)	***Q'* (HWQ)**	**Factor, *A***	**Factor, *B***	**Factor, *C***	**Stability number, *N'***	**Hydraulic radius, *S* (m)**
950	8.80	0.53	0.9	2.5	10.5	5
1050	8.80	0.43	0.9	2.5	8.5	5
1150	8.80	0.40	0.9	2.5	7.8	5
1250	8.80	0.35	0.9	2.5	6.9	5
1350	8.80	0.32	0.9	2.5	6.3	5

4.4 Summary

The relevant input parameters needed for the two evaluation techniques (numerical modelling and stability graph method) was thoroughly discussed and evaluated in this Chapter. For numerical modelling, parameters such as rock mass quality, stress field, stope geometries and mine design assessment was summarized. On the other hand, stability graph parameters evaluated included modified *Q* tunneling quality index, rock stress factor, joint orientation adjustment factor, gravity adjustment factor and the hydraulic radius of the stope face hanging wall.

CHAPTER 5 DISCUSSION OF RESULTS

5.1 Introduction

The main objective of this research project at Konkola mine was to perform a detailed geotechnical study on backfill requirements on stopes in the steep dipping areas of the mine. In order to achieve this goal, two evaluation methods (numerical modelling and empirical analysis) have been chosen to evaluate the behaviour of the stopes as mining depth increases. It is the idea of this project also to ascertain the depth at which the stopes are likely to be unstable hence the need to stabilize them with backfill for safe and increased economic copper ore production.

This chapter describes, discuss and compares the outcomes from the two evaluation methods. It was from these analysis that conclusions and recommendations were drawn.

5.2 Numerical modelling analysis

The software package PHASE2 was used to model the stopes at different levels. Modelling was done on stopes from 950mL, 1050mL, 1150mL, 1250mL and 1350mL to determine the induced stresses and displacements around these stopes as the depth increases. The reason why the analysis started at 950mL was simply because the other levels above are already stoped out.

A single model was created for all the five levels. Induced stresses, strength factors and displacements results for the different levels was analyzed and discussed to make detailed predictions of levels that may pose a risk to the stoping activities as a result of instability.

5.2.1 Induced stresses analysis

Figure 5.1 is an indication of the stope model in the "Interpret" mode of PHASE2 interface. The sub-vertical lines in the model indicate the distinct rock formations at KCM. The different stopes at each level are included as stages in this model although they are not visible from the diagram. Gravity loading was used for the stress input since stress measurement are only for a single location at No 1 shaft of KCM. Figure 5.1 clearly indicates the increase of vertical in-situ stresses as the depth from the surface increases as expected. The depth below surface is denoted by the y-axis (vertical axis).

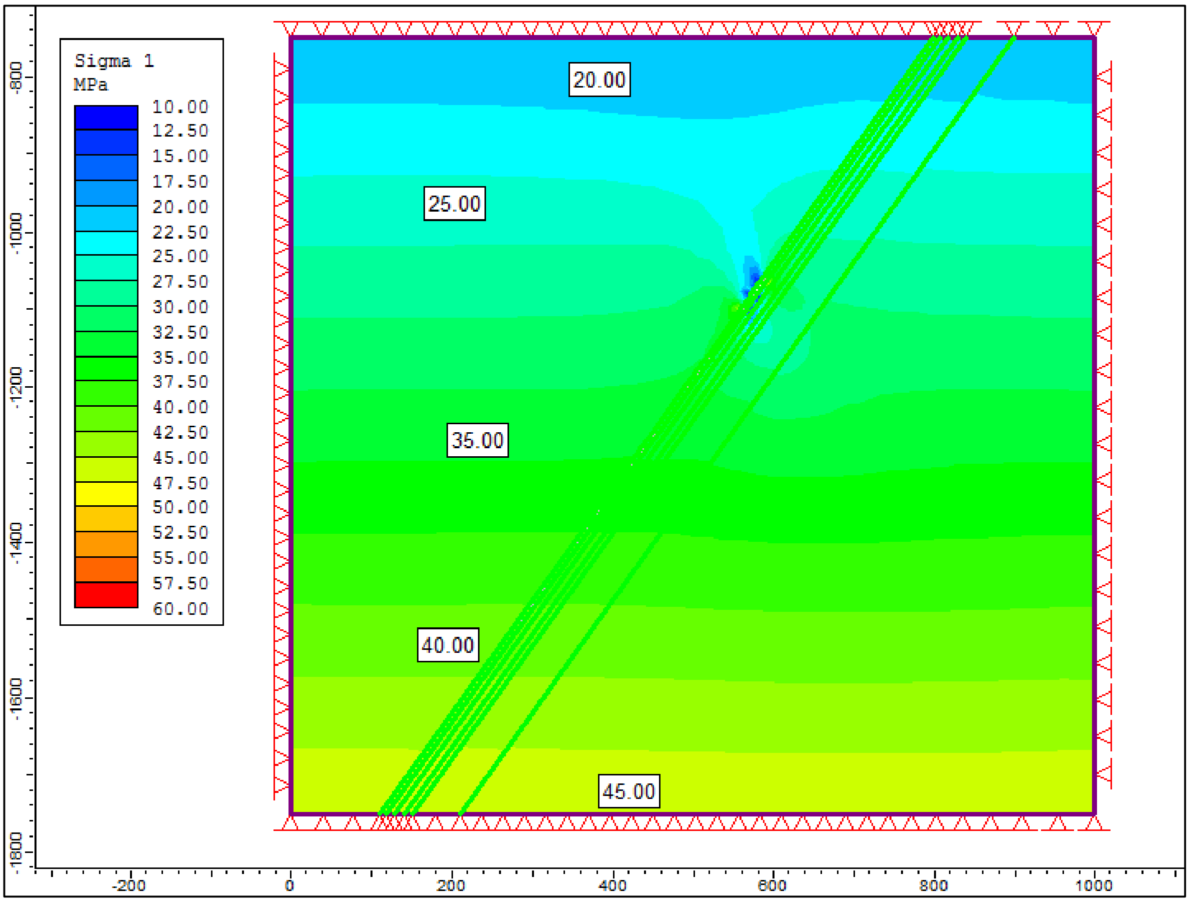


Figure 5.1 Phase2 interpret model showing the increasing stresses with depth (Phase 2)

Figure 5.2 to 5.6 are contour plots of the maximum compressive stress (sigma 1) induced around the stopes.

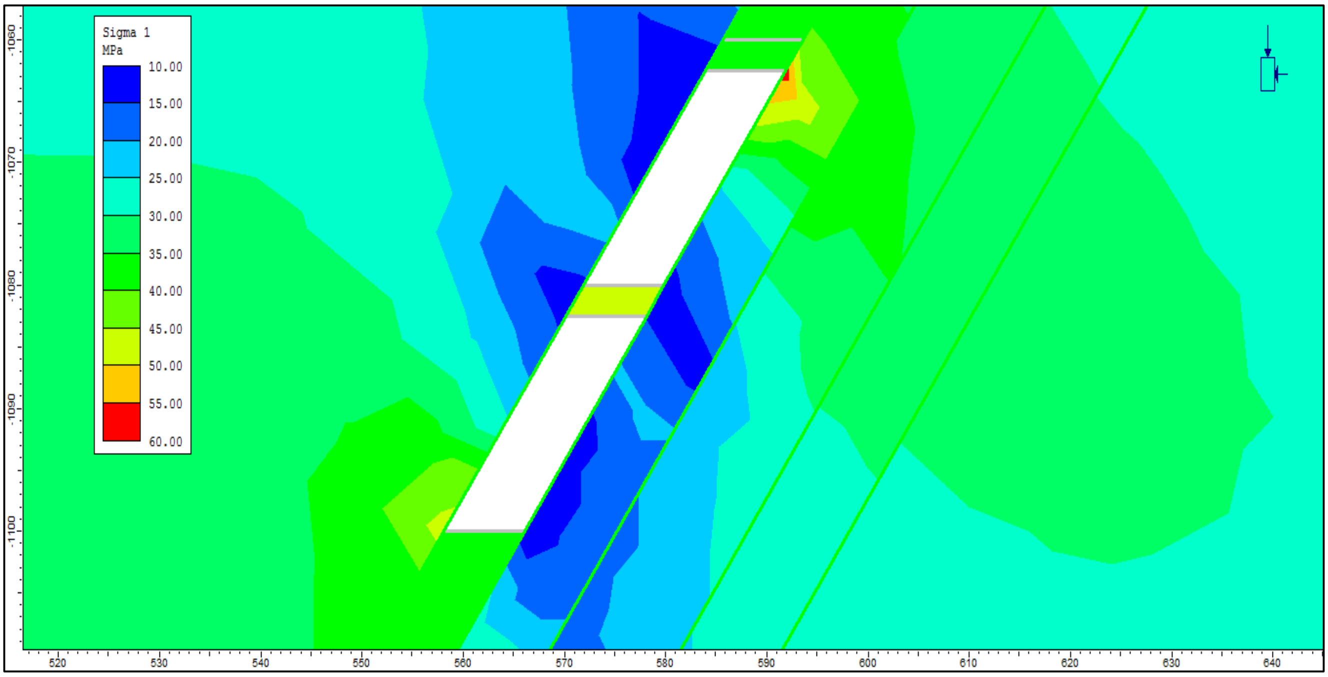


Figure 5.2 Induced stress (Sigma 1) contours for 950mL (Phase 2)

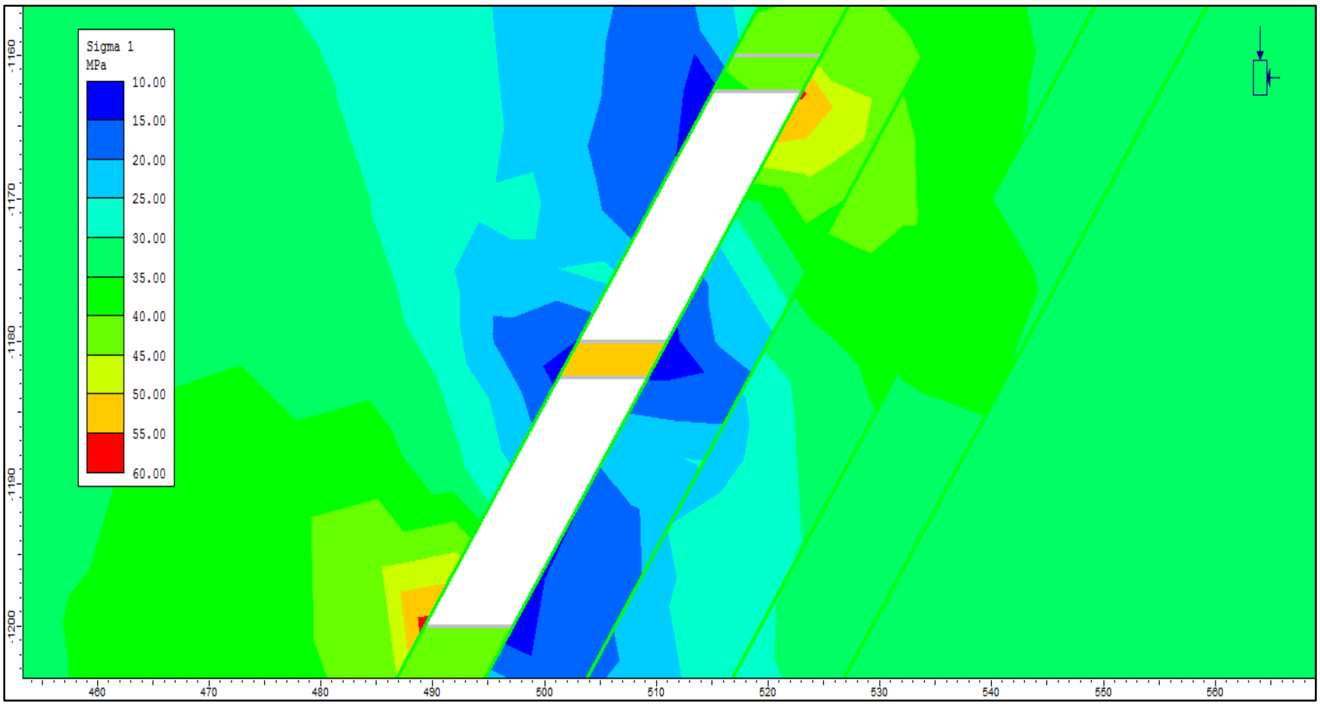


Figure 5.3 Induced stress (Sigma 1) contours for 1050mL (Phase 2)

The diagrams indicate that there are high stresses around the corners (abutments) of the stopes. This is because for open stopes, stresses concentration normally occur around the pillars, hanging wall abutments, and footwall abutments (Jucheng Wang, 2004). And if these stress concentrations exceeds the rock mass strength failure will occur.

Stope hanging walls are usually in a state of relaxation or a low compressive or tensile induced stress. So rather than stress induced failure on the hanging wall, these low compressive stresses or close to tensile stresses tend to loosen the rock mass thus making the hanging wall more prone or susceptible to gravity induced failures.

The contours for sigma 1 are all showing a similar trend for all the levels (Figure 5.2 to 5.6). The areas of low compressive stresses marked by the dark blue contours indicate a degree of relaxation and it is from these areas where the strength factor contours (Figure 5.10 to 5.14) are showing lower strength values hence implying that they are zones of instability.

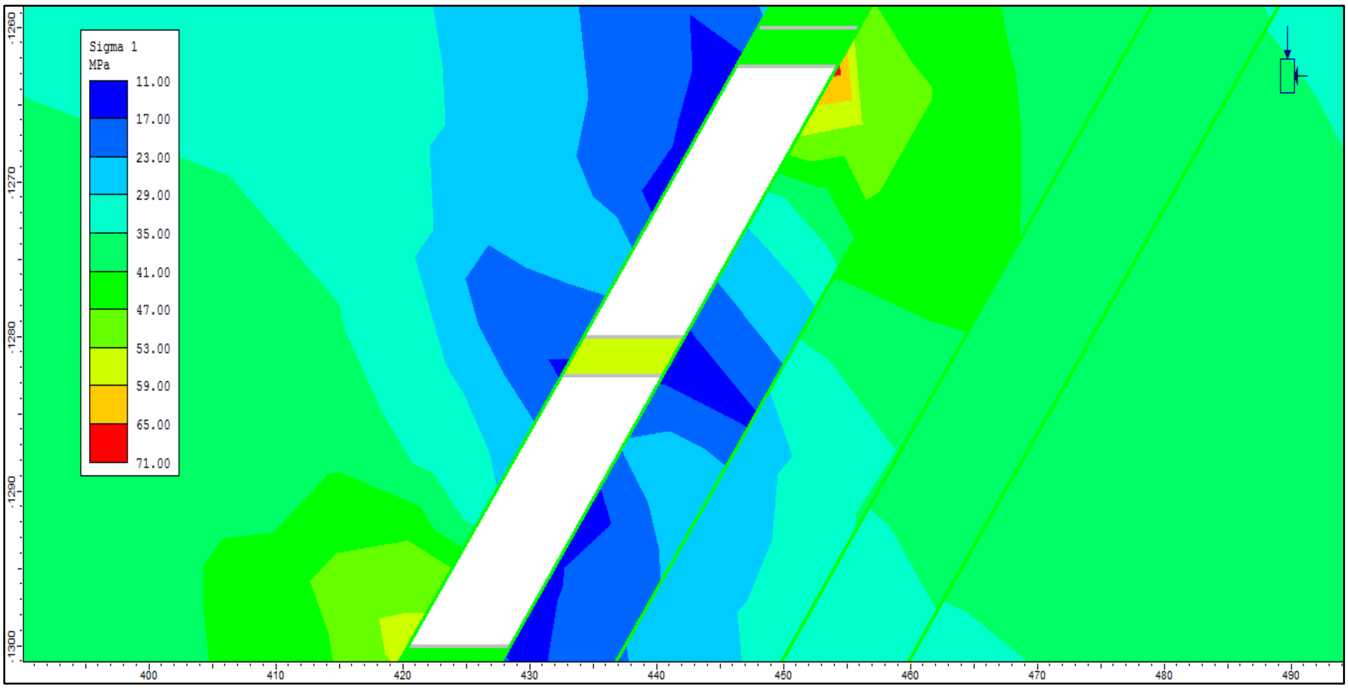


Figure 5.4 Induced stress (Sigma 1) contours for 1150mL (Phase 2)

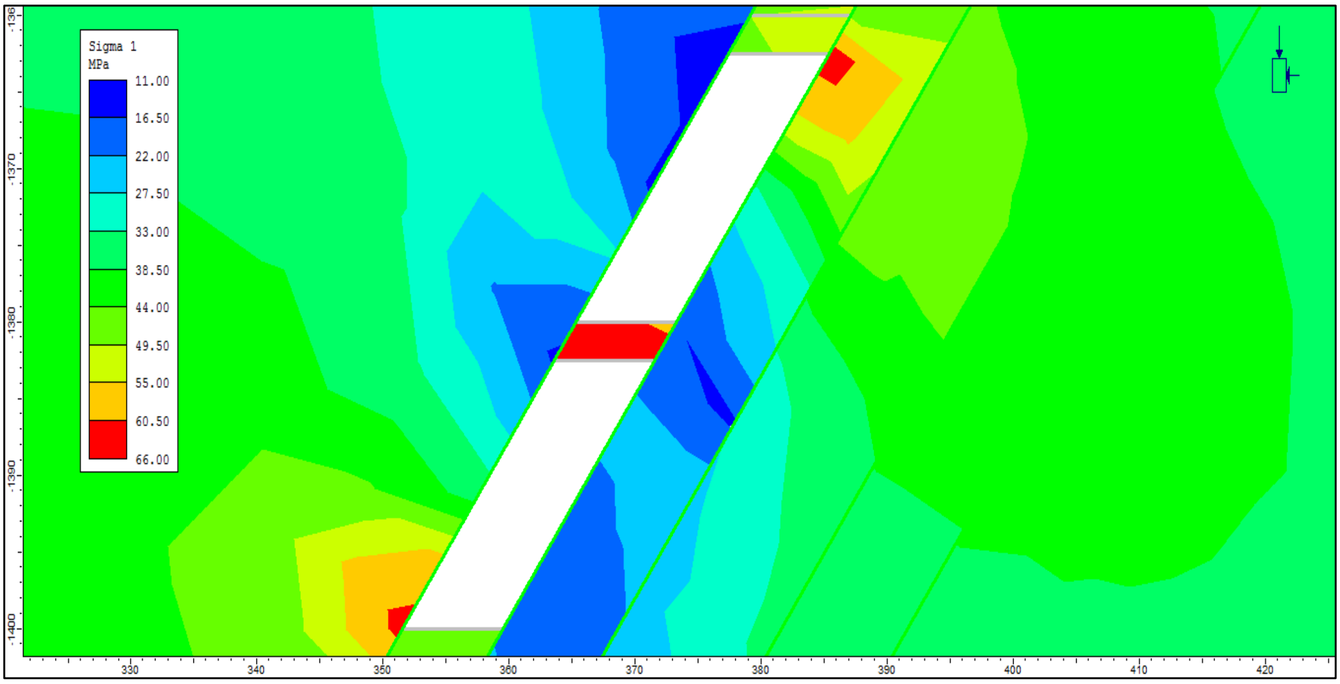


Figure 5.5 Induced stress (Sigma 1) contours for 1250mL (Phase 2)

Sigma 1 contours for 1250ml and 1350 mL (Figure 5.5 and 5.6 respectively) are indicating very high induced stresses around the crown pillar. Therefore, it is clear that around these regions of high concentration of induced stresses, there is a likelihood of compressive stress driven failures. These areas are the pillars and also the hanging wall abutments. Such kind of failures are most likely to be experienced from 1250mL to 1350mL and further down in case KCM plans to further mine beyond 1350mL.

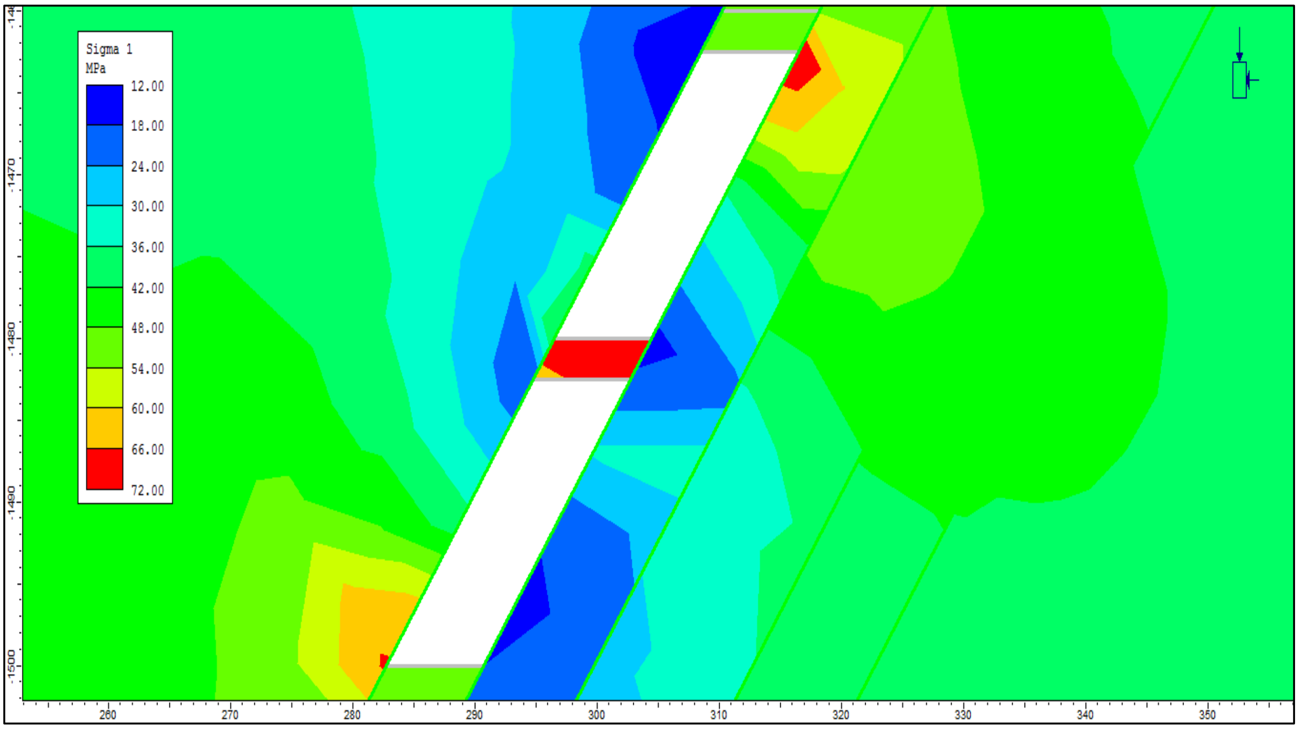


Figure 5.6 Induced stress (Sigma 1) contours for 1350mL (Phase 2)

5.2.2 Maximum total displacement analysis

Total displacement around the stope boundaries was also analyzed by selecting total displacement option from the data list in the toolbar. This is an important option as it gives a picture of how the stope walls are responding with an anticipated increase in the induced stresses as mining progresses further down to greater depths.

From the contours indicated in Figure 5.7, much of the displacement is experienced in the hanging wall side of the stopes. This also confirms to the geotechnical reports of Konkola mine which reports minor spallings of individual rock pieces from the hanging wall side of the stopes. The total displacement also does not give a picture of actual depth of instability but it is important in adding more understanding of stope behaviour and how backfilling material can help in arresting this inward movement of the stope walls.

Contour plots from the software are plotted on the graphics as the Total Displacement option is toggled on. The status bar will then indicate the maximum displacements for the entire stope at each level. Figure 5.7 is an illustration to indicate that much of the displacement is occurring at excavation walls of the hanging wall side of the stopes as shown by the contours.

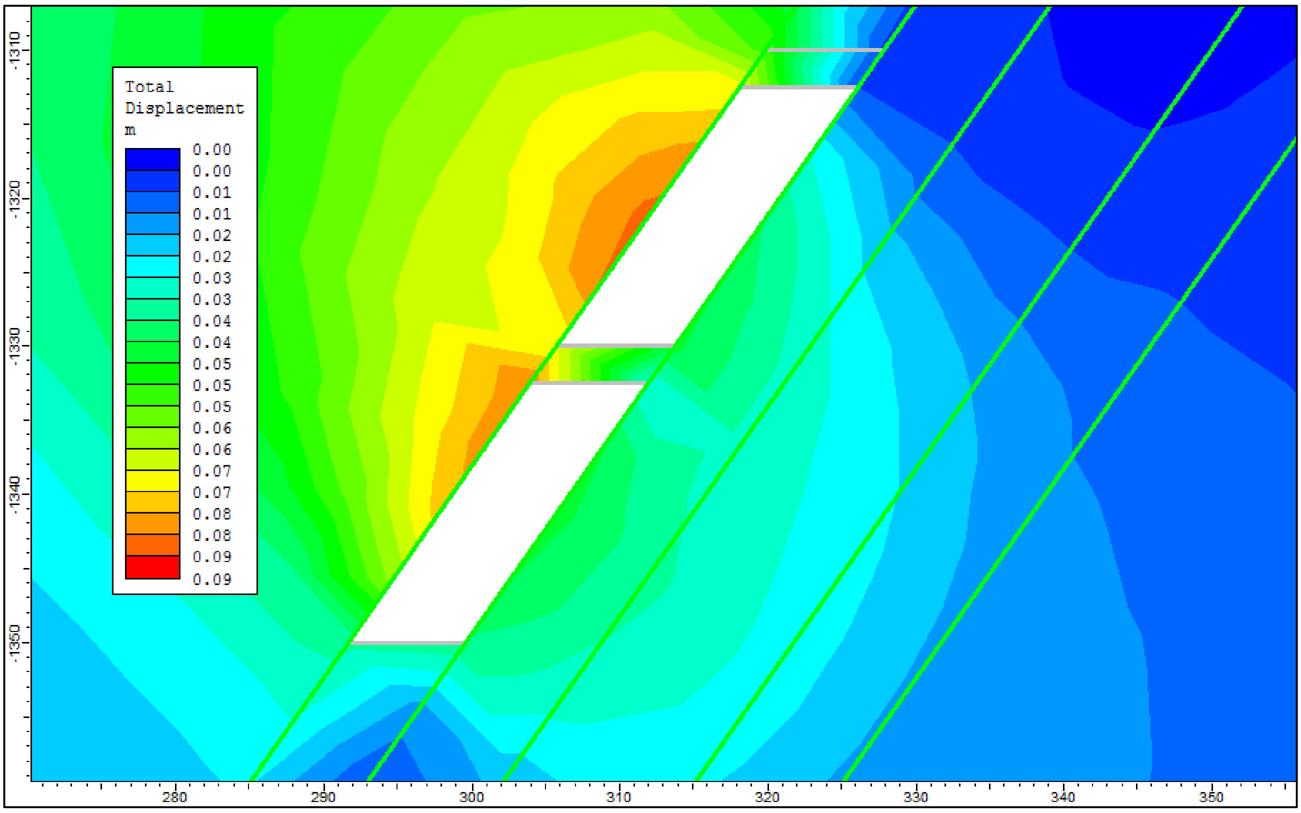


Figure 5.7 Maximum total displacement contours at 1350mL (Phase 2)

The maximum total displacements for all the stopes at different levels measured from PHASE 2 models was recorded. Table 5.1 is an indication of mining level and the corresponding maximum stope displacement. The deformed boundaries from PHASE 2 graphically illustrated the increasing inward movement of the excavation boundaries as depth increases. This also supports the trend shown in Table 5.1 of a linear increase in total displacement on stope walls as the depth of mining also increases.

Table 5.1 Maximum total displacement of stopes

Depth (mL)	**Maximum total displacement (m)**
950	0.055
1050	0.062
1150	0.070
1250	0.076
1350	0.083

Figure 5.8 is also an outline to indicate that the stope wall displacements increases as the depth of mining increases. This is also an indication that the induced stresses around the stope walls have got an impact on the stability of the stopes as operations increases from 950mL to 1350mL.

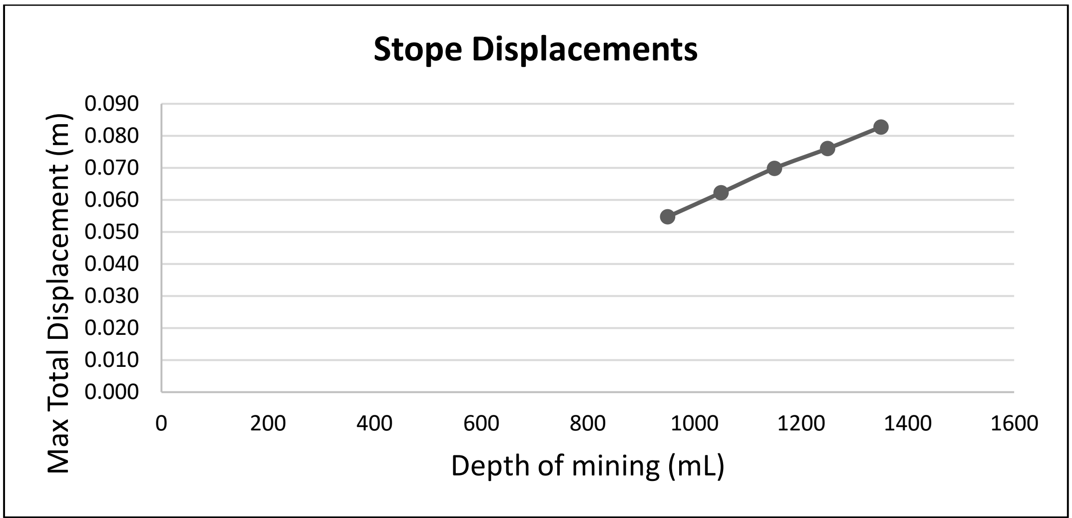


Figure 5.8 Maximum total stope displacement (Phase 2)

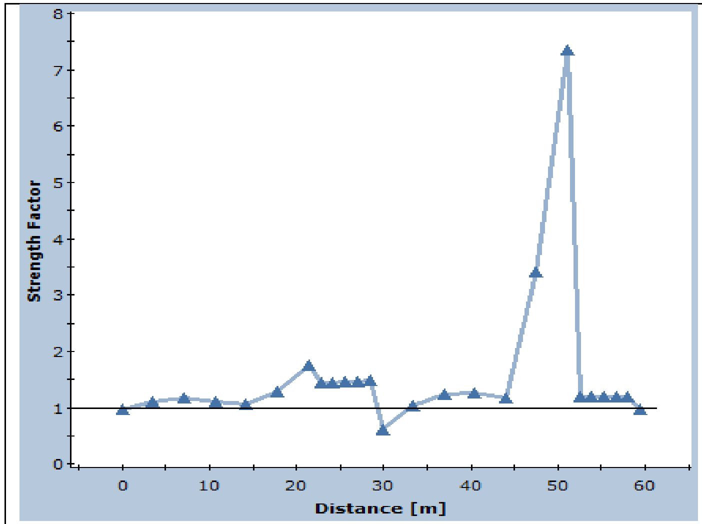
5.2.3 Strength Factor Analysis

The strength factor option in Phase2 represents the ratio of the available rock mass strength to the induced stresses at a given point. Since this model is an elastic analysis, all of the rock mass contained within the contours below 1 will fail if not supported. On the contrary, if it is a plastic analysis, all the rock mass contained within the contours marked 1, will have strength factors less than 1 and will fail if left unsupported. This is because in plastic analysis, the strength factor cannot go below one and when failure (yielding) occurs, the strength factor is by definition equal to 1. In elastic analysis, the strength factor can go below one, as an arbitrary or hypothetical measure of overstress.

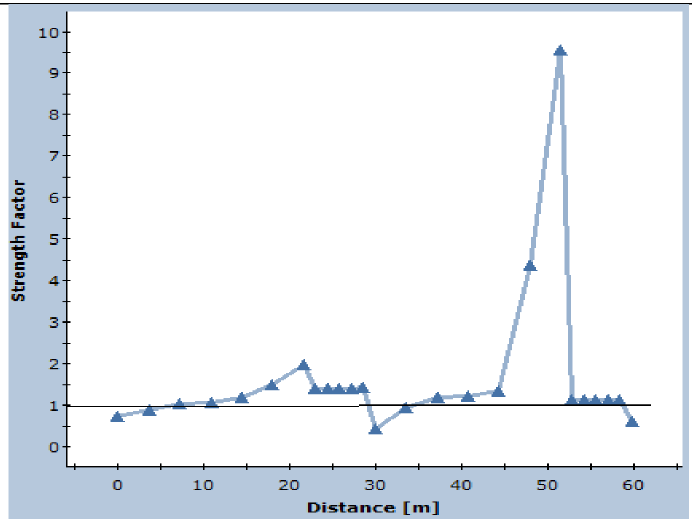
The majority of the strength factors around the stope peripheries are greater than one. This implies that the stopes are stable up to 1350mL under the current stope dimensions of 20m along strike, 20m height and 8-12m orebody thickness. The values of the strength factor, however are decreasing or converging towards unity as the depth increases from 950mL to 1350mL. This explains the fact that induced stresses are increasing downwards and thus the rock mass has more stresses to withstand with the increasing depth of ore extraction.

Figure 5.9 is a clear indication that much of the strength factor values are above unity. However, the trend of the decreasing strength factor value is not very clear as other lower levels has values greater than those of the upper levels on corresponding points along the stope walls.

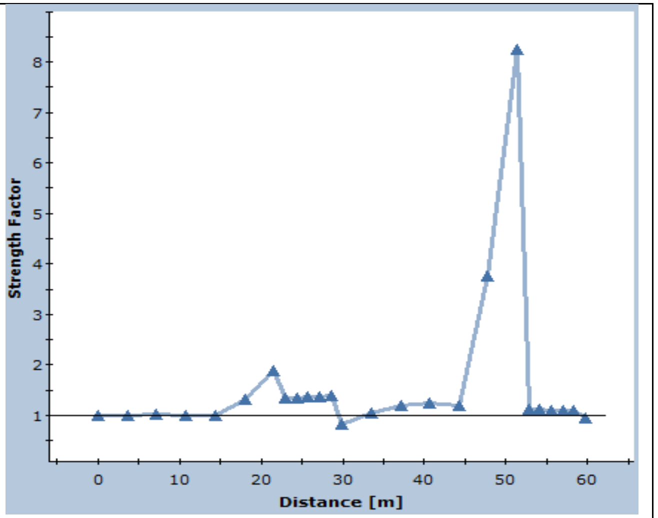
From Figure 5.9, the last flat portion of each line graph corresponding to approximately 53-60m distance along the horizontal axis represents the area of the crown pillar. The graphs shows that the strength factor around the crown pillars is also converging to one as the levels increase from 950mL to 1350mL. However the strength factor graphs are showing almost a similar trend of induced stresses around the stopes.



(a) 950mL



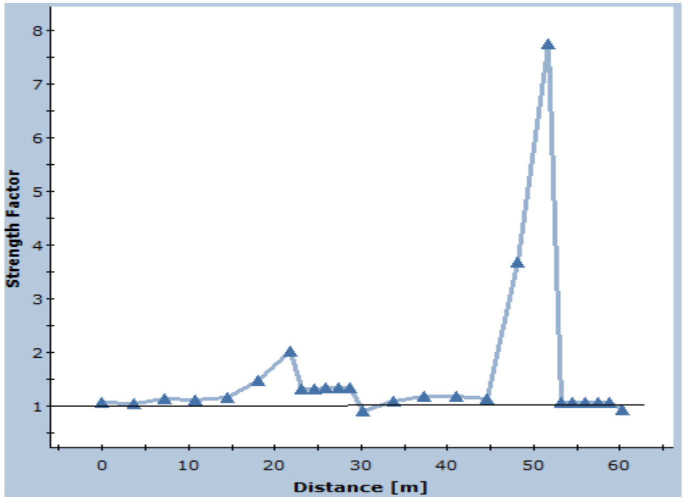
(b) 1050mL



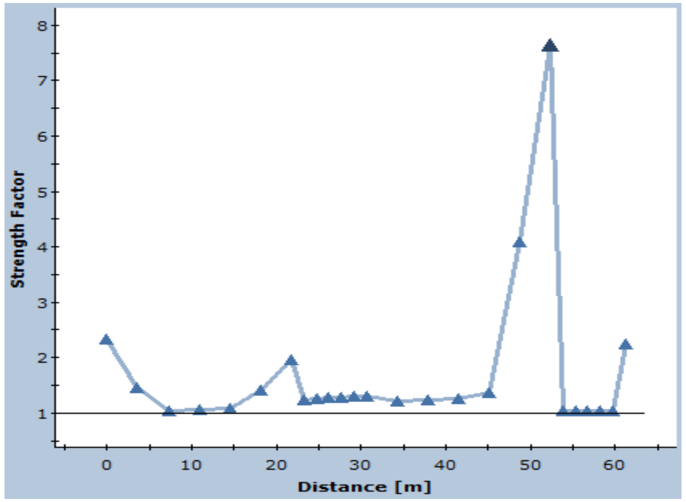
(c) 1150mL

Note:

The horizontal axis of each graph represents the distance right round the modelled stope cross sectional perimeter.



(d) 1250mL



(e) 1350mL

Figure 5.9 Strength Factor queries (Phase 2)

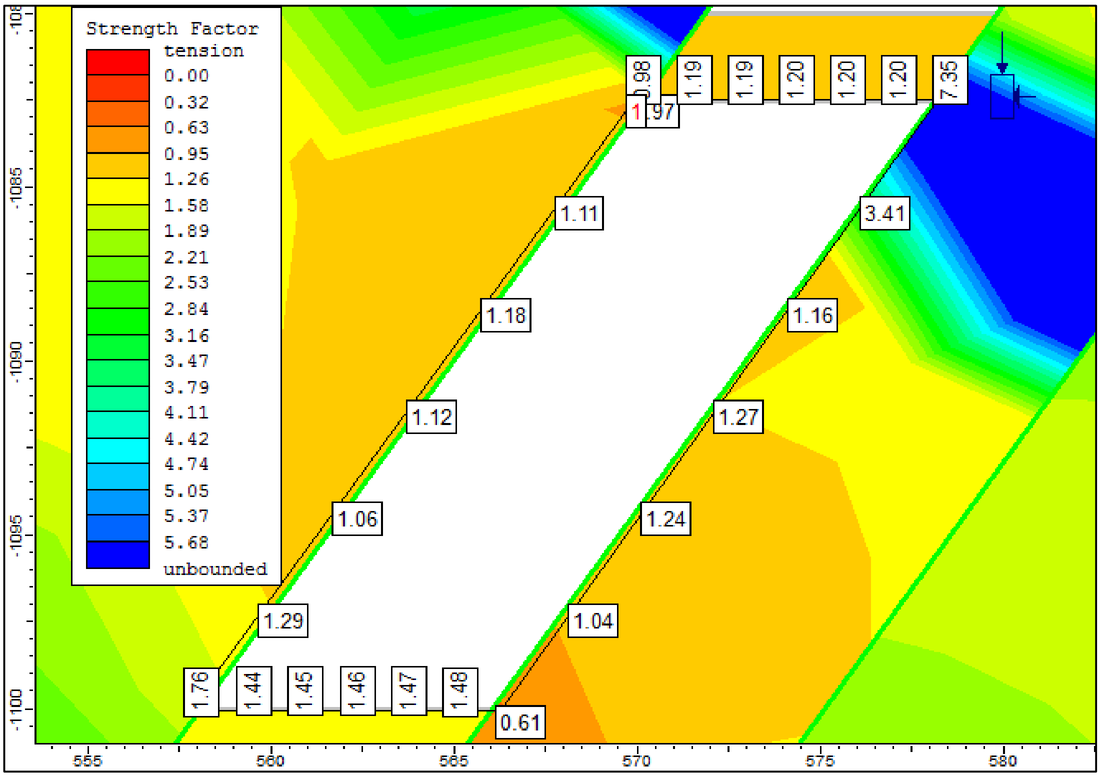


Figure 5.10 Strength factor contours for 950mL (Phase 2)

Figure 5.10-5.14 is an outline of the strength factor contours of the stopes for the different levels from 950mL to 1350mL. The contours also shows that the stopes are stable at the current design stope spans of 20m strike length, 20m height and 8m orebody thickness. Values of strength factors are converging to 1 in a more clear way along the crown pillar and stope floors as indicated by the diagrams. This is also an indication of high induced stresses in these areas of the stopes.

Areas that are showing slight instabilities from these figures are those close to the upper left corners and lower right corners of the stopes. These areas indicate values less than one and this means that there is a degree of failure associated with such areas of the stope. This is also explained by what the mine is currently experiencing around its current levels of operation (1040mL) where there are no challenges of ground collapse within the stopes. Only some minor spalling and dilution of the ore are the only draw backs.

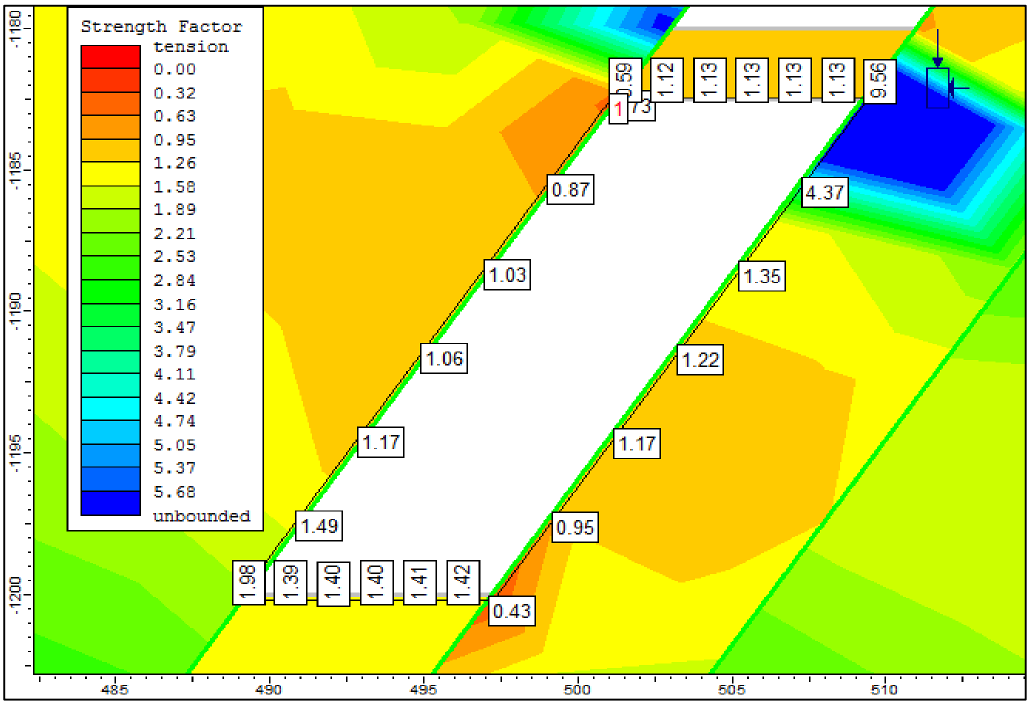


Figure 5.11 Strength factor contours for 1050mL (Phase 2)

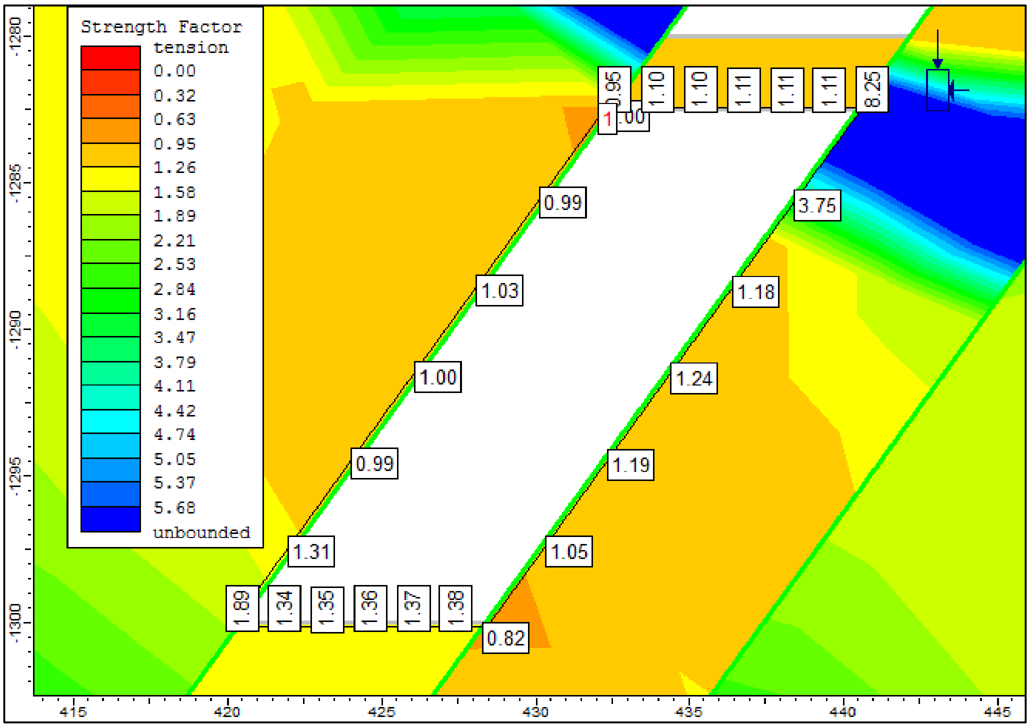


Figure 5.12 Strength factor contours for 1150mL (Phase 2)

However, another point to note from Figures 5.10-5.14 of the strength factor contours is that the areas close to the crown pillars has very high values. This also explains the stabilizing effect that the crown pillar has on the overall stability of the stopes since they act as abutments and thus they are high stress regions.

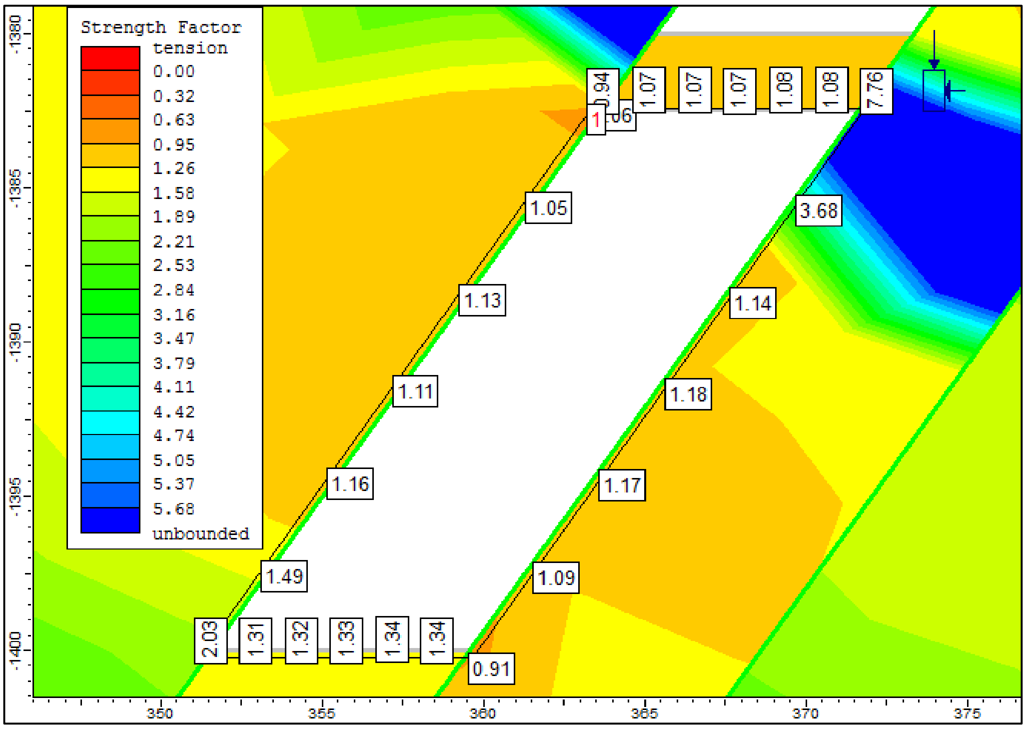


Figure 5.13 Strength factor contours for 1250mL (Phase 2)

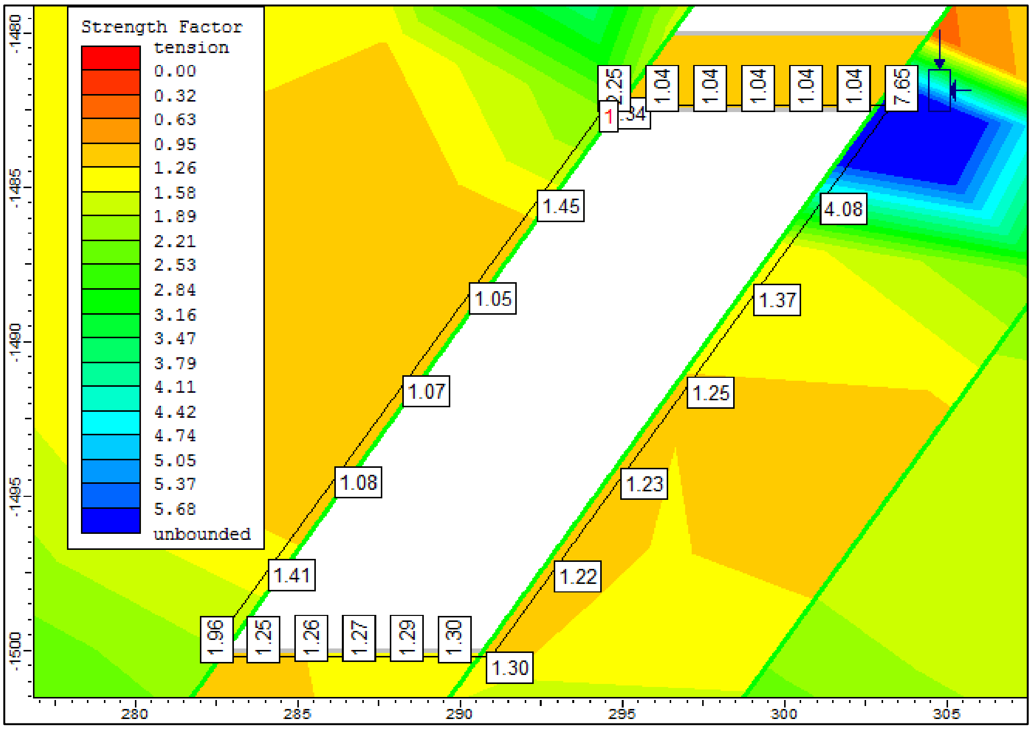


Figure 5.14 Strength factor contours for 1350mL (Phase 2)

The crown pillars at 1250mL to 1350mL are highly stressed and it is also shown from Figure 5.14 by a lower strength factor (1.04) that is almost equal to 1. This may imply an immediate collapse of this pillar when stoping activities are being done at these levels and thus posing a risk to the mucking crew. Figure 5.15 is showing the linear decrease of the strength factor with depth. However, it is the design of KCM to allow the crown pillar to cave in and self-fill the open stopes but this caving should take place after the whole broken ore has been mucked. So with this low value there is a chance of collapse before mucking is completed.

Figure 5.15 is an illustration of the decrease in the strength factor as the depth of operations increases.

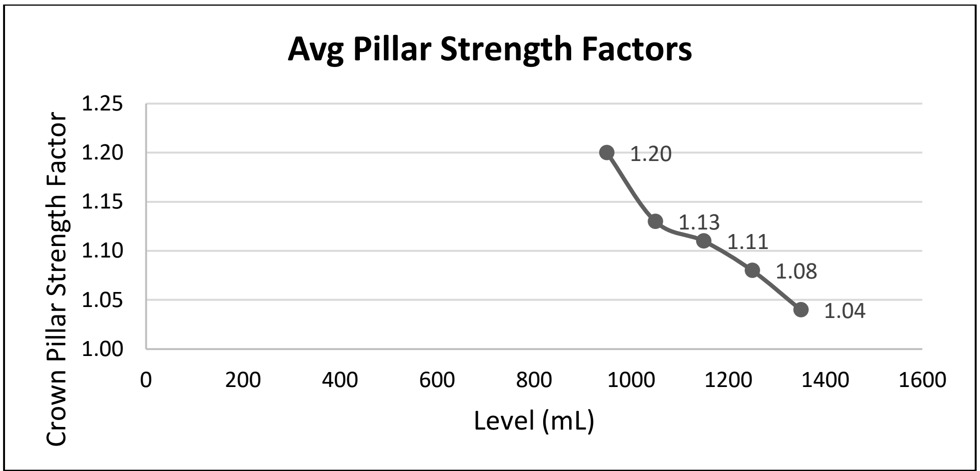


Figure 5.15 Strength factor variation with increasing mining depth

5.3 Stability Graph Analysis

Empirical analysis was used also to make informed conclusions on the behaviour of the stopes as mining activities progresses further downwards. The details of the determination of the evaluation parameters that are needed to calculate the stability number, *N'* and the hydraulic radius, *S* are discussed in Chapter 4 section 4.3.

The use of two different methods (numerical modelling and empirical analysis) has an advantage of improving the quality of the research work as the weaknesses of one method can be outweighed by the strengths of another hence creating a balance.

The stability number and the hydraulic radius values for each of the different levels under analysis (950mL, 1050mL, 1150mL, 1250mL and 1350mL) are summarised in the Table 5.2.

Table 5.2 Summary of stability graph parameters

Level (mL)	Stability Number, *N'*	Hydraulic Radius, *S*
950	10.5	5
1050	8.5	5
1150	7.8	5
1250	6.9	5
1350	6.3	5

These values in Table 5.2 were then plotted on the Mathews Stability graph. This plot is illustrated in Figure 5.16. As can be noted from the diagram the stopes from 950mL to 1150mL are stable under the current stope spans of 20m strike length, 20m height and 8-12m orebody thickness. This is because the stopes on these levels are seated in the stable region of the stope stability graph method.

Figure 5.16 is also showing that starting from 1250mL going down stopes are likely to start showing signs of instability under the current dimensions of 20m length along strike, 20m height and 8-12m orebody thickness. This is because the stopes are in the unsupported transition zone which is an intermediate zone between stable and stable with support regions. In this region stopes exhibit both characteristics of being stable as well as showing signs of instability. However, there is no expected sudden collapse of stopes around this depth since their location is closer to the stable region on the graph as opposed to the stopes being closer the stable with support region.

It is also the case with numerical modelling as zones of high stresses were notable around 1250mL to 1350mL as shown in Figures 5.5 and 5.6. In these levels, the crown pillars and hanging wall abutments are highly stressed and it was also clear from the numerical model that around these levels there are chances of compressive stress driven failures.

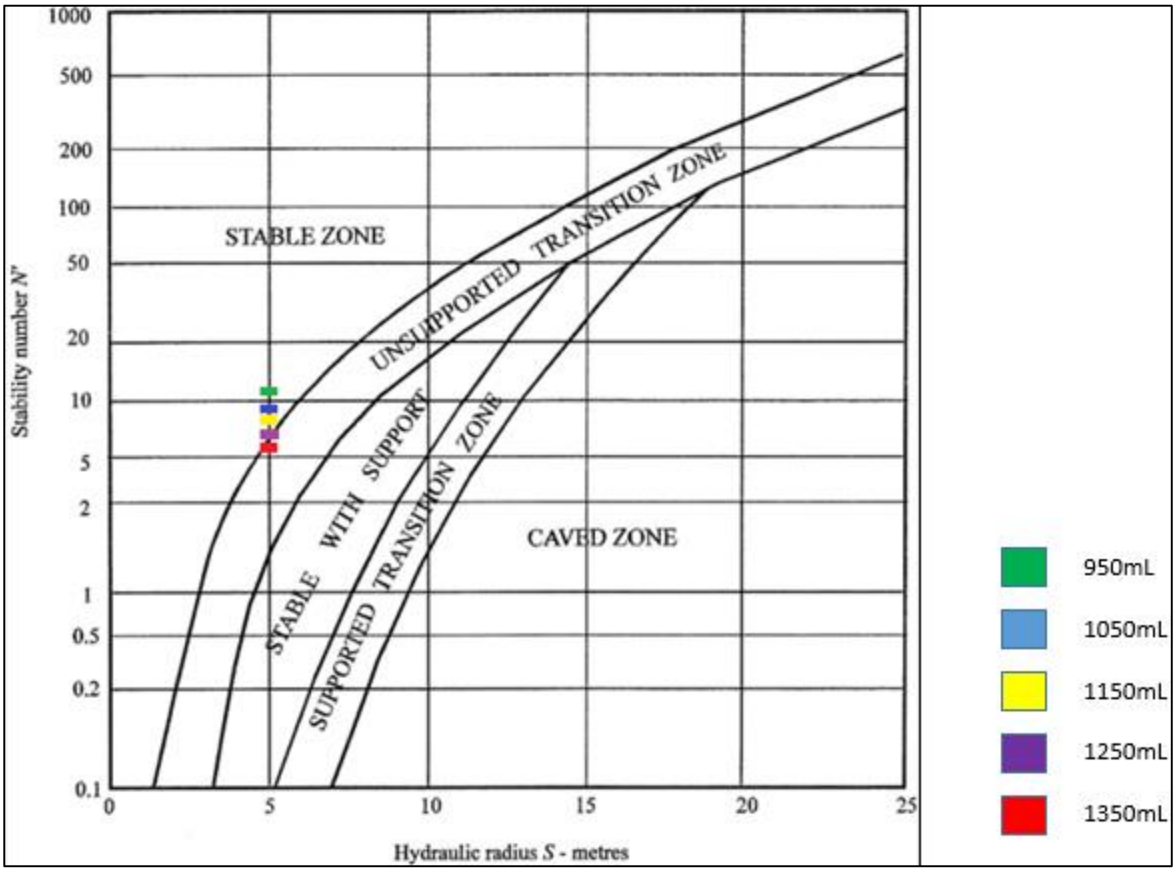


Figure 5.16 Stability graph plots of the stopes on differents levels

There is a possibility of increasing the stope span without backfilling to a hydraulic radius of around 6 for the stopes at 950mL to 1050mL. This hydraulic radius can allow opening up stopes of 30m strike length while maintaining the same stope height of 20m. An additional 10m of stoping length has an advantage of reducing the number of stabilizing pillars. The number of cut out raises and draw-point crosscuts is also reduced hence reducing development cost. This is because current stoping operations are leaving 5m rib pillars after every 20m of stoping and hence a lot of valuable ore is being left in the stopes as stabilizing pillars.

Table 5.3 is a stope productivity comparison between the 30m and 20m strike length stopes to evaluate the benefits of optimizing the stope dimensions. The comparison excludes the other developments such as trough drives and extraction drives since these are constant for both stope lengths of 20m and 30m along strike. A total mining length on a single level of 175m along strike was assumed to simplify the comparison.

Table 5.3 Stope productivity analysis for 950mL and 1050mL

Design parameter	Units	Quantity	
		20m Strike length Stope for 950mL	**30m Strike length Stope for 950mL**
Density in-situ ore	t/m^3	2.7	2.7
Density broken ore (approx.)	t/m^3	2.2	2.2
Design stope height	m	20	20
Design stope width	m	8	8
Design stope length	m	20	30
Design stope volume	m^3	3200	4800
No of stopes/175m strike length	-	7	5
No of rib pillars/175m	-	7	5
Stope tonnes/175m strike length	t	60480	64800
Ore volume in rib pillars	m^3	5600	4000
Draw point x-cuts/stope	-	2	2
Total draw point x-cuts/175m	-	14	10
Slot raises/175m	-	7	5

From the calculations performed in Table 5.3 it is clear that the overall ore production of the entire 950mL stopes will increase by **7.14%** when the 30m strike length of stopes is used. The current 20m length is conservative and hence it is costing the mine in terms of development as more slot raises and draw point cross-cuts are needed as compared to the 30m strike length. Therefore development costs on both slot raises and draw point cross cuts will be reduced by **28.6%** for the entire 950mL in the Bancroft area in case the 30m stope length is used.

It is also practical to use the 30m stope strike length for the entire 1050mL in order to optimize stope dimensions in relation to a specific level. This is possible because stopes in the 1050mL are also stable with a possibility of being moved closer to the unsupported transition zone. Currently there are no instability related problems at 1040mL and therefore operating these stopes at 30m strike length can add value to the current operations of the mine.

5.3.1 Economic Benefits of using Cemented Fill in the Bancroft Areas

As a result of the risk instability that can be experienced beyond 1250mL, the use of mining with backfilling was suggested. In order to ascertain the feasibility of this option, a preliminary cost analysis between use of backfill with no pillars against mining using the current designs of 20m strike length, 20m height and 8m orebody thickness was performed. The following options were considered:

Option A

Introduce hydraulic cemented fill and mine the stopes without leaving stabilizing ore rib pillars. The specification of the backfill material needed for the primary/secondary extraction system of KCM have been analyzed in a research by Mutawa (2011). In this study, geotechnical investigations on the available mine waste materials at Konkola Copper Mine (KCM) in Chililabombwe, Zambia, were performed to develop a suitable backfill material for safe and economic ore production. The material needed to have a strength of 1 MPa, as spelt out by Konkola mine management and suitable drainage characteristics to resist failure due to self-weight in a backfilled stope and therefore facilitate ore pillar recovery.

Option B

Maintain the stopes on HR equal to 5 but reduce stope span when instability problems become excessive. It is reasonable to take this risk because the stopes at 1250mL and 1350mL plotted close to the stable region of the Mathews stability graph.

The cost analysis was done over a distance of 175m on orebody strike while assuming operations to be on a single level. This was done in order to simplify the analysis. Table 5.4 is a comparison of the production performance of the two options A and B.

Table 5.4 Stope productivity comparison for Option A and Option B

Design Parameter	**Units**	**Option A**	**Option B**
Design Stope Length	m	20	20
Design Stope Height	m	20	20
Design Stope Width	m	8	8
No. Stopes/175m Strike Length		8.75	7
Total Stope volume	m^3	28000	22400
Total Tonnes	t	75600	60480
Dilution & Recovery			
Dilution	%	15	15
Mining Recovery	%	95	95
Diluted Tonnes	t	86940	69552
Total Tonnes Mucked	t	82593	66074
Volume to be backfilled	m^3	28000	-

The ore production unit costs used in this analysis were obtained from the KCM Life of Mine (LOM) plan. The costs indicated that materials handling and backfill contribute to a significant percentage of the mine total cost. The cemented backfill material used in the analysis represents approximately 16% of the total mining costs and this was based on an estimated unit cost of US\$21/m^3 of cemented backfill. This cost covers the placement as well as the backfill reticulation costs as stipulated in the LOM report. However, the LOM report planned for a copper price of US\$5753 per tonne of metal and US\$5700 was used for this analysis. Table 5.5 is an illustration of the economic cost analysis.

Table 5.5 Economic cost analysis for Option A and Option B

Production		**Unit**		**Option A**	**Option B**
Ore Delivered to Mill		Tonne		82,593	66,074
Mill Recovery		%		90	90
Total Contained Cu (@ 2.5% Cut off grade)		Tonne		1,858	1,487
Gross Revenue				**10,592,552**	**8,473,991**
Department	**Cost Centre**	**Unit**	**Rate**		
Mining	Supervision & Control	US$/t Ore	1.93	159,404	127,523
	Development	US$/t Ore	1.53	101,093	101,093
	Production	US$/t Ore	10.1	834,189	667,347
	Materials Handling	US$/t Ore	8.12	670,655	536,521
	Backfill	US$/t Ore	7.2	594,670	-
	Mine Services	US$/t Ore	4.82	398,098	318,477
Gen & Admin	General & Administration	US$/t Ore	6.06	500,514	400,408
Milling	Concentrator	US$/t Ore	4.52	373,320	298,654
Total Cost				(3,631,944)	(2,450,024)
EBITDA				6,960,608	6,023,967

(**Note**: EBITDA is Earnings before Income Tax, Depreciation and Amortization)

The use of hydraulic cemented fill and extracting the ore without leaving stabilizing ore rib pillars (Option A) is more profitable than maintaining the stopes on HR equal to 5 but reducing the stope span when instability problems become excessive (Option B). The economic cost analysis in Table 5.5 is a clear indication of this conclusion because mining with backfill has proved to be more profitable.

However, Option A results in copper ore production increasing by **25.0%** hence a step forward towards the mine plan of ramping up production to 7.5Mtpa from the current 1.7Mtpa. This is as a result of the extraction ratio change from 80% to 100% since in Option A no pillars are being left. The total income over the 175m strike length for a single level of stoping is also showing US$6,960,608 for Option A against US$6,023,967 for Option B. Projecting this gain over the entire operations of the Bancroft area of KCM will mean an increase in EBITDA of approximately **15.5%**.

5.4 Summary

The results regarding to stope stability analysis from numerical modelling and stability graph method was discussed in this Chapter. In numerical modelling, induced stresses, maximum total displacements and strength factors around stopes on different levels was analyzed. However, the different zones of stability of the stopes were plotted on the stability graph. From the results of these two techniques, conclusions in relation to actual levels of instability of the stopes as mining depth increases was then drawn.

CHAPTER 6 CONCLUSIONS & RECOMMENDATIONS

6.1 Conclusions

The main objective of the research project was to perform a detailed geotechnical study to determine if backfilling of stopes is necessary in the steep dipping areas of KCM. According to the outcomes of this research work, backfilling is necessary from 1250mL proceeding downwards. However, the sub-objectives that were also addressed by this research are summarised as follows and have been achieved in this study.

Sub-objectives:

1. To model the stress distributions and displacement around the mine openings using numerical methods (Phase2).
2. To evaluate the zone of stability of the individual stopes at different depths using the empirical approach (Mathew Stability graph).
3. To compare the outcomes of the two methods and make recommendations.
4. To optimise ore extraction by selecting possible backfill material types that can be used at the mine in case modelling and empirical analysis prove the need to use backfill.

The conclusions drawn from this research work have been established on the basis of geotechnical studies and economic analysis that are connected to the backfill requirements for these areas in KCM underground operations. The geotechnical behaviour of the stopes was evaluated and quantified based on established 2D computer numerical modelling using PHASE 2 as well as empirical analysis using the Mathews stability graph method. However, the use of advanced 3D modelling softwares such as MAP3D can further refine the results of this research work.

The following conclusions, corresponding to the above sub-objectives, were drawn based on the analysis of the two evaluation technique:

1. **a)** Using the current stope dimensions of 20m strike length, 20m height and 8-12m orebody thickness, the PHASE2 analysis indicated that the majority of the stopes are stable from 950mL to 1350mL with values of Strength Factors greater than 1.

   b) Numerical modelling also indicated that the stopes from 1250mL and beyond are likely to experience compressive stress driven failures around the crown pillars and the hanging wall abutments as a result of high induced stresses.

c) There is also displacements within the stopes especially on the hanging wall side. Displacements increased as depth of ore extraction also increased with a maximum of 0.083m noted for stopes around 1350mL.

2. **a)** Plotting of the specific stability numbers against HR on the Mathews Stability Graph indicated that from 950mL to 1150mL, stopes are in the stable zone.

   b) Stopes on 1250mL plotted on the boundary between stable zone and unsupported transition zone and 1350mL stopes plotted within the unsupported transition zone and hence there is a chance of these stopes to start showing signs of instability.

   c) Increasing the HR of 950mL and 1050mL stopes to 6 (allowing opening up stopes of 30m strike length while maintaining the same stope height of 20m) improved overall ore production, in these levels within the Bancroft area, by **7.14%**. This also reduced the development costs on both slot raises and drawpoint cross-cuts by **28.6%**.

3. Both evaluation techniques indicated that chances of instability within the stopes is likely to start when depth from surface of mining operations is around 1250mL under the current stope dimensions of 20m strike length, 20m height and 8-12m orebody thickness.

4. **a)** From the economic analysis, stopes at 1250mL and below, the use of hydraulic cemented fill and extracting the ore without leaving stabilizing ore rib pillars (Option A) is more profitable than maintaining the stopes on HR equal to 5 but reducing the stope span when instability problems become excessive (Option B).

   b) Option A increased copper ore production by **25.0%** in the Bancroft area hence a step forward towards the mine plan of ramping up production to 7.5Mtpa from the current 1.7Mtpa.

   c) The backfilling option however increased EBITDA by approximately **15.5%** within the Bancroft areas of KCM from the results of the economic cost analysis. The analysis clearly indicated that despite an increase in the mining cost due to additional cost of backfilling, the increased recovery of the resource and possible reduced dilution pays for this extra cost.

6.2 Recommendations

The following actions to be undertaken have been recommended in order to maximize on stoping operations and hence optimized ore extraction:

1. There is need to undertake a detailed economic based feasibility study on backfilling in order to fine-tune these preliminary findings and thus detail the actions needed to maximize ore production.
2. Mining with backfill has proved to be more viable than to leave substantial ore tonnages tied in stabilizing pillars hence there is need to plan for backfilling infrastructure facilities and training of backfill personnel to operate the existing cemented backfill system.
3. For the time being, stope strike length should be increased from the current 20m to 30m for 950mL to 1050mL to improve on ore production since these levels are already being exploited.

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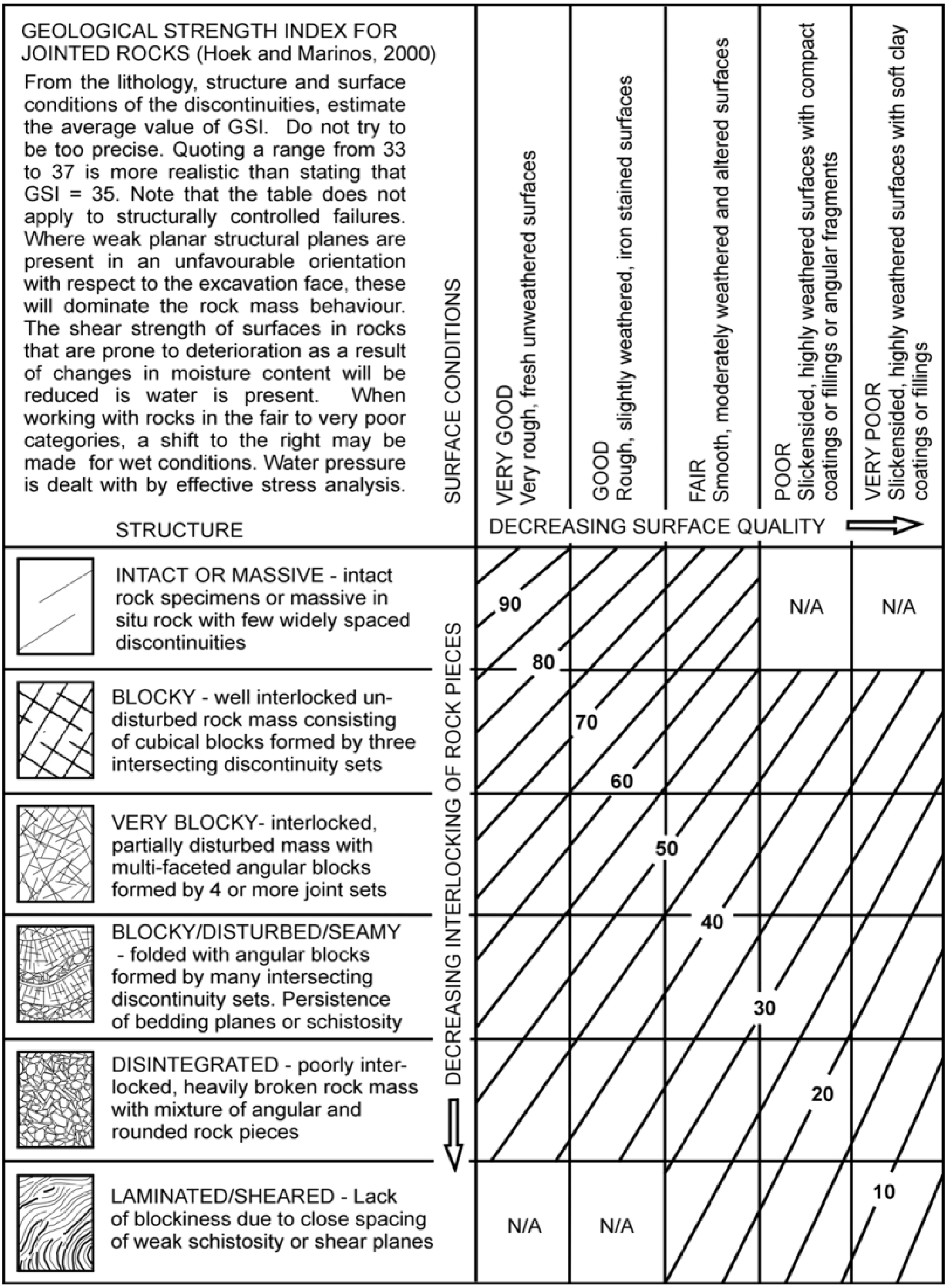
APPENDICES

Appendix 1 Characterisation of blocky rock masses on the basis of interlocking and joint conditions (After Hoek and Marinos, 2000)



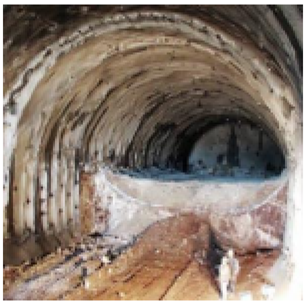
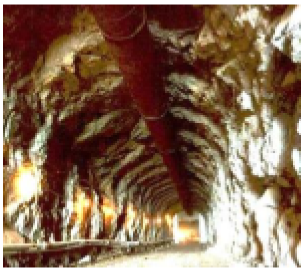
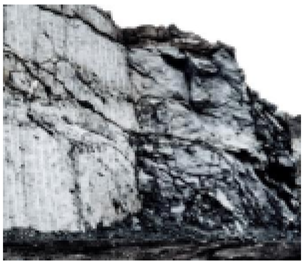
Appendix 2 Values of the constant mi for intact rock, by rock group. Note that values in parenthesis are estimates (Based on Deere, 1968 and Palmstrom and Singh, 2001)

Rock type	Class	Group	Texture: Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates* (21 ± 3) Breccias (19 ± 5)	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6		
	Foliated**		Gneiss 28 ± 5	Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Diorite 25 ± 5 Granodiorite (29 ± 3)			
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	Obsidian (19 ± 3)
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

* Conglomerates and breccias may present a wide range of m_i values depending on the nature of the cementing material and the degree of cementation, so they may range from values similar to sandstone to values used for fine grained sediments.

**These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

Appendix 3 Guidelines for estimating disturbance factor D (After Hoek, Carranza-Tores and Corkum, 2002)

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	D = 0
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	D = 0 D = 0.5 No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	D = 0.8
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	D = 0.7 Good blasting D = 1.0 Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	D = 1.0 Production blasting D = 0.7 Mechanical excavation

Appendix 4 Core log sheet for CP 813 core log

MINE: KONKOLA
LOCATION:
INCLINATION:
AZIMUTH:
Logged By: M. Goma & N. Dzimvnya
Date Logged: 04/10/16

HOLE COLLAR
X:
Y:
Z:

Drilling Date:
Drill Rig:
Drill Contractor:
Hole Size: NQ

DRILL HOLE NAME: CP 813

CP 813 1020 mL 4168 mN
1020 mL 4168 mN

Rock Description	Depth scale (m)	Run From (m)	Run To (m)	Recovery m	Recovery %	Solid m	Solid %	RQD m>100 mm	RQD %	IRS MPa	No of Discont	FF/m	Spacing of Discs	Discon Type	Depth of Discs	No. of fractures	Dip wrt Core Axis	Large Scale Express A	Small Scale Express B	J/Wall Alteration C	Joint Filling D	Fill Type	Fill Thick	Weather Grade	No of joint sets	Geotechnical Notes	RMR	Q (NGI)	RQD
AGS510 – 2.7																													
SW, Slightly partly	·	0	0.75	0.75		0.46		0.5		R4	4	4/0.75		J		4													
R4 – R5, fine,	0													J1			25	–	–	–	–	–	–	–	–	CJ (closed joint)			
grained, greyish-	0.1												0.06	J2			25	C	RS	W	NSF	QTZ	<1mm	2	2	SW			
purplish to dark	0.4													J3			10	MW	RU	W	NSF	QTZ	1	2	1	SW			
weathered portions	0.75													J4			50	C	RU	N	–	–	–	1	1	UW			
	0	0.75	2.7	1.95		1.8		1.8		R5	0	–	–	–	–	3	–	–	–	–	–	–	–	–	–	CJ			
PC 2.7 – 13.80		2.7	4.0	1.22		0.64		0.94		R5	2	2/1.22	1.11	J1	2.77	2	40	C	RS	N	NSF	QTZ	1	1	1	Rubble zone			
Pinkish, Greyish,														J2	3.96		60	MW	RS	W	TL	FEO	1	3	1				
Vugged and pitted in																													
some portions, Fresh		4.0	10.54	6.42		0.28		4.60		R5	7	7/6.42	0.9	J1	4.02	7	20	SU	RU	N	NSF	QTZ	1	1	2	SW			
rock with slightly														J2	4.87		50	MW	RS	N	NSG	QTZ	1	1	1				
weathered joints.														J3	5.80		25	SU	RU	N	"	"	1	1					
														J4	6.33		20	SU	RU	N	"	"	1	1					
														J5	8.38		25	MW	RS	W	GL	QTZ	2	2	2				
														J6	9.10		–			–		–	–	–		CJ			
														J7	9.76		15	MW	RU	W	NSF	FEO							
Discs, Pinkish, Fresh																													
Weathered bands &		10.54	13.83	3.20		1.93		2.82		R5	2	2/3.20	0.6	J1	11.35	2	90	SU	SU	N	NSF	QTZ	1	1	2	Rubble zone			
Rubble material.														J2	12.03		45	S	RP	W	NSM	FEO	1	2					
FWSST																													
Brownish, greenish,		13.83	21.00	6.99		5.75		6.20		R5	11	10/6.99	0.70	J1	14.12	10	25	S	RP	W	NSF	FEO	1	2	1				
Slightly weathered,														J1	14.24		20	S	RP	N	NSF	QTZ	1	1	1				
Vugged in some portions														J3	15.12		35	S	RP	W		FEO	2	2	1				
														J4	15.40			–	–	–	–	–	–	–		Closed joint			
														J5	15.56		50	MW	RS	W	GL		2	3	1				
														J6	15.85			–	–	–	–	–	–	–		Closed joint			
														J7	16.08		60	MW	RU	N	NSF		1	1	1				
														J8	16.45			–	–	–	–	–	–	–		CJ			
														J9	17.36		20	SU	SU	W	NSF	FEO	1	2	2				
														J10	17.43		–	–	–	–	–	–	–	–		CJ			
		21.00	30.70	9.6		9.37		8.85		R5	3	3/9.6	1.06	J1	23.35		20	SU	SU	N	NSC	QTZ	2	1	1				
														J2	24.60		80	–	–	–	–	–	–	–	1	CJ			
														J3	26.59		–	–	–	–	–	–	–	–	–	CJ			
FWC		30.70	33.8	3.1		2.78		2.27		R5	3	3/3.1	0.06	J1	33.48		–												
Pinkish, Slightly														J2	33.51		–												
weathered														J3	33.67		–												
ORESHALE		33.8	42.00	8.2		8.08		4.29		R4	11	11/8.2	0.75	JB	34.10	11	15	S	SP	N					1				
Dark grey fine														J	34.80		35	SU	RU	N	NSF	QTZ	2	1					
grained, bedded														J	35.10		35	S	SP	N	NSF	QTZ	2	1	2				
														J	38.3		40	SU	SU	N	N	N	N	1					
														J	39.0		31	S	RP	N	NSF	QTZ	2	1					
														J	40.10		35	SU	RS	N	NSF	QTZ	1	1					
														J	40.40		35	C	SU	N	NSF	N	1	1					
														J	40.6		35	SU	SU	N	NSF	N	1	1					
														J	40.7		40	S	SP	N	NSF	N	1	1					
														JB	40.92		40	SU	SU	N	NSF	N	1	1	2				
														JB	41.23		35	SU	SU	N	NSF	–	1	1					

MINE: KONKOLA
LOCATION:
INCLINATION: ______________
AZIMUTH: ______________
Logged By: SVEN MKONDE & NEYAID DZIMUNYA
Date Logged: 06/10/2016

HOLE COLLAR
X: ________
Y: ________
Z: ________

DRILL HOLE NAME CP813 1020 mL 468 m

Drilling Date: ____________
Drill Rig: ____________
Drill Contractor: ____________
Hole Size: NQ____________

Geotechnical Zones		Run		Recovery		Solid		RQD		IRS	DISCONTINUITY DATA							JOINT CONDITION								GEOTECHNICAL NOTES	CHARACTERISATION		
Rock Description	Depth scale (m)	From (m)	To (m)	m	%	m	%	m>100 mm	%	MPa	No of Discont	FF/m	Spacing of Discs	Discon. Type	Depth of Discs	No. of fractures	Dip wrt. Core Axis	Large Scale Express A	Small Scale Express B	J/Wall Alteration C	Joint Filling D	Fill Type	Fill Thick	Weather Grade	No of joint sets		RMR	Q (NGI)	RQD
HKIQ		42.00	49.5	7.28		6.89		4.73		R5	10	10/7.28	0.68	J1	42.55	10	35	MW	SU	N	NSF	QTz	1	1					
Dark to light														J2	42.80		35	–	–	–	–	–	–	–		CJ			
grey, fine														J3	43.70		30	S	SP	N	NSF	–	1	1					
grained.														J4	43.74		30	SU	SU	N	NSF	–	1	1					
														J5	44.27		–	–	–	–	–	–	–	–	–	CJ			
														J6	45.45		30	SU	SM	N	NSF	–	1	1					
														J7	46.15		–	–	–	–	–	–	–	–	–	CJ			
														J8	46.38		30	MW	SU	N	NSF	–	1	1					
														J9	47.70		35	SU	SM	N	SSF	–	1	1					
														J10	48.60		35	C	SU	N	NSF	–	1	1					

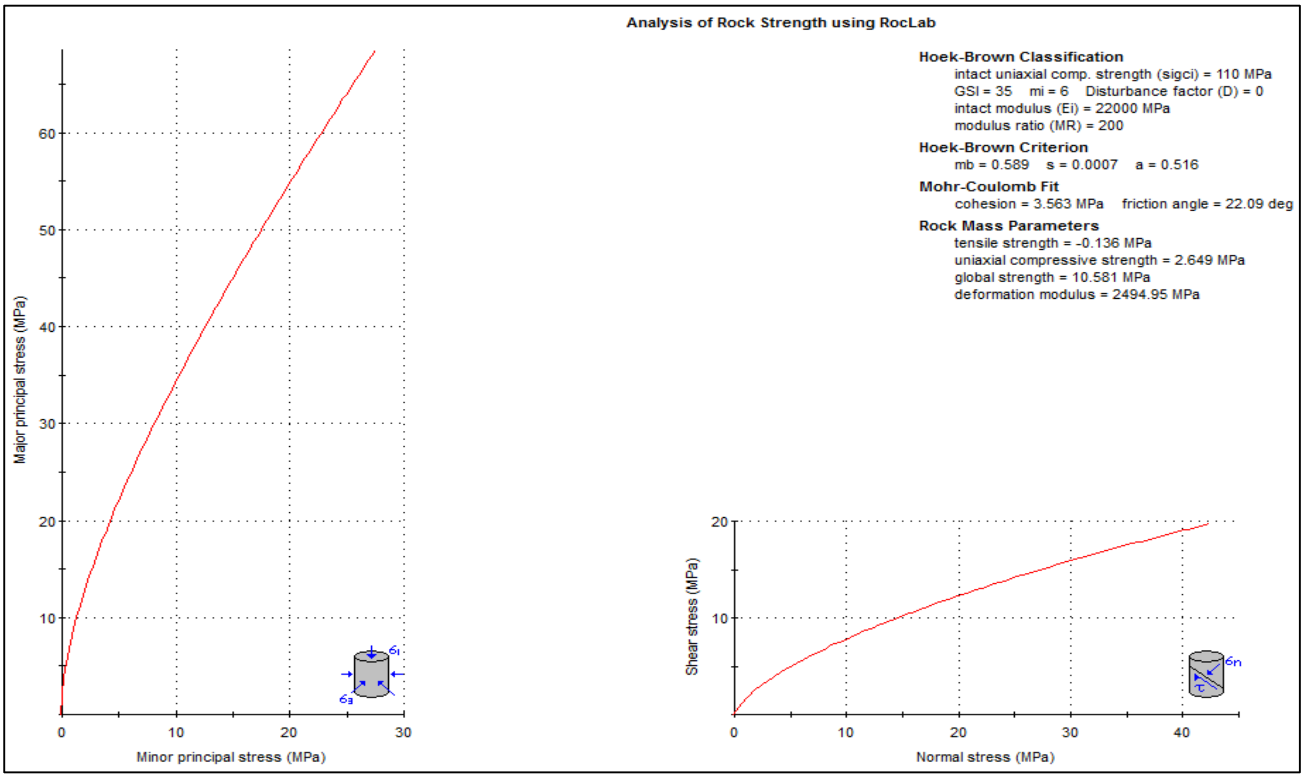
Appendix 5 Classification of individual parameters used in the Tunnelling Quality Index Q system (After Barton et al, 1974)

DESCRIPTION	VALUE	NOTES
1. ROCK QUALITY DESIGNATION	***RQD***	
A. Very poor	0 - 25	1. Where *RQD* is reported or measured as ≤ 10 (including 0),
B. Poor	25 - 50	a nominal value of 10 is used to evaluate *Q*.
C. Fair	50 - 75	
D. Good	75 - 90	2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently
E. Excellent	90 - 100	accurate.
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use ($3.0 \times J_n$)
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	2. For portals use ($2.0 \times J_n$)
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact		
b. Rock wall contact before 10 cm shear		
A. Discontinuous joints	4	
B. Rough and irregular, undulating	3	
C. Smooth undulating	2	
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is
E. Rough or irregular, planar	1.5	greater than 3 m.
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	2. J_r = 0.5 can be used for planar, slickensided joints having
c. No rock wall contact when sheared		lineations, provided that the lineations are oriented for
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	minimum strength.
J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)	

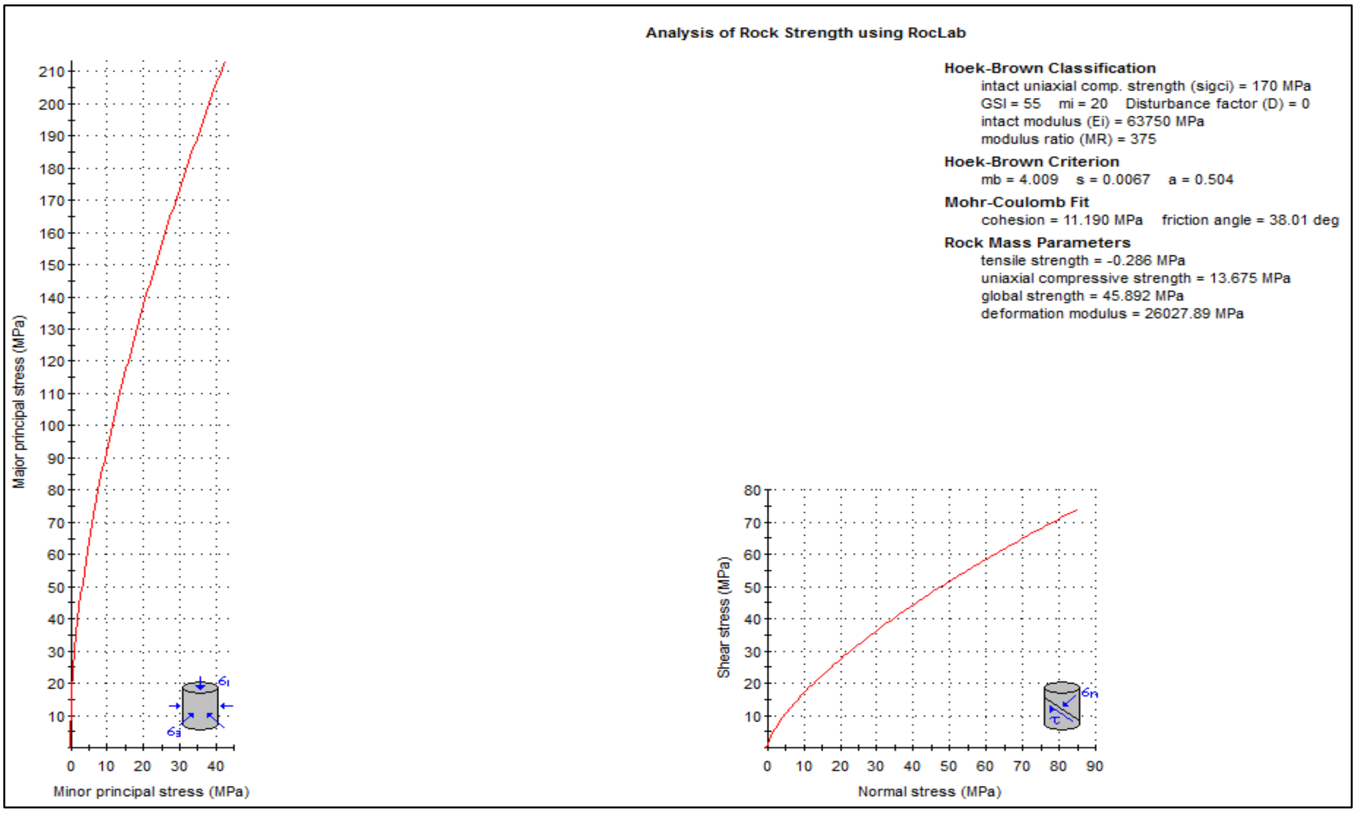
DESCRIPTION	VALUE	ϕr degrees (approx.)	NOTES
4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
a. Rock wall contact			
A. Tightly healed, hard, non-softening, impermeable filling	0.75		1. Values of ϕr, the residual friction angle, are intended as an approximate guide
B. Unaltered joint walls, surface staining only	1.0	25 - 35	to the mineralogical properties of the
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25 - 30	alteration products, if present.
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	20 - 25	
E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less in thickness)	4.0	8 - 16	

DESCRIPTION	VALUE	NOTES	
4, JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
b. Rock wall contact before 10 cm shear			
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24	
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16	
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0 - 12.0	6 - 12	
c. No rock wall contact when sheared			
K. Zones or bands of disintegrated or crushed	6.0		
L. rock and clay (see G, H and J for clay	8.0		
M. conditions)	8.0 - 12.0	6 - 24	
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0		
O. Thick continuous zones or bands of clay	10.0 - 13.0		
P. & R. (see G.H and J for clay conditions)	6.0 - 24.0		
5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	
6. STRESS REDUCTION FACTOR		*SRF*	
a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated			
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)		10.0	1. Reduce these values o the relevant shear zones only influence but f *SRF* by 25 - 50% if do not intersect the excavation .
B. Single weakness zones containing clay, or chemically distegrated rock (excavation depth < 50 m)		5.0	
C. Single weakness zones containing clay, or chemically distegrated rock (excavation depth > 50 m)		2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)		7.5	
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)		5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)		2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)		5.0	

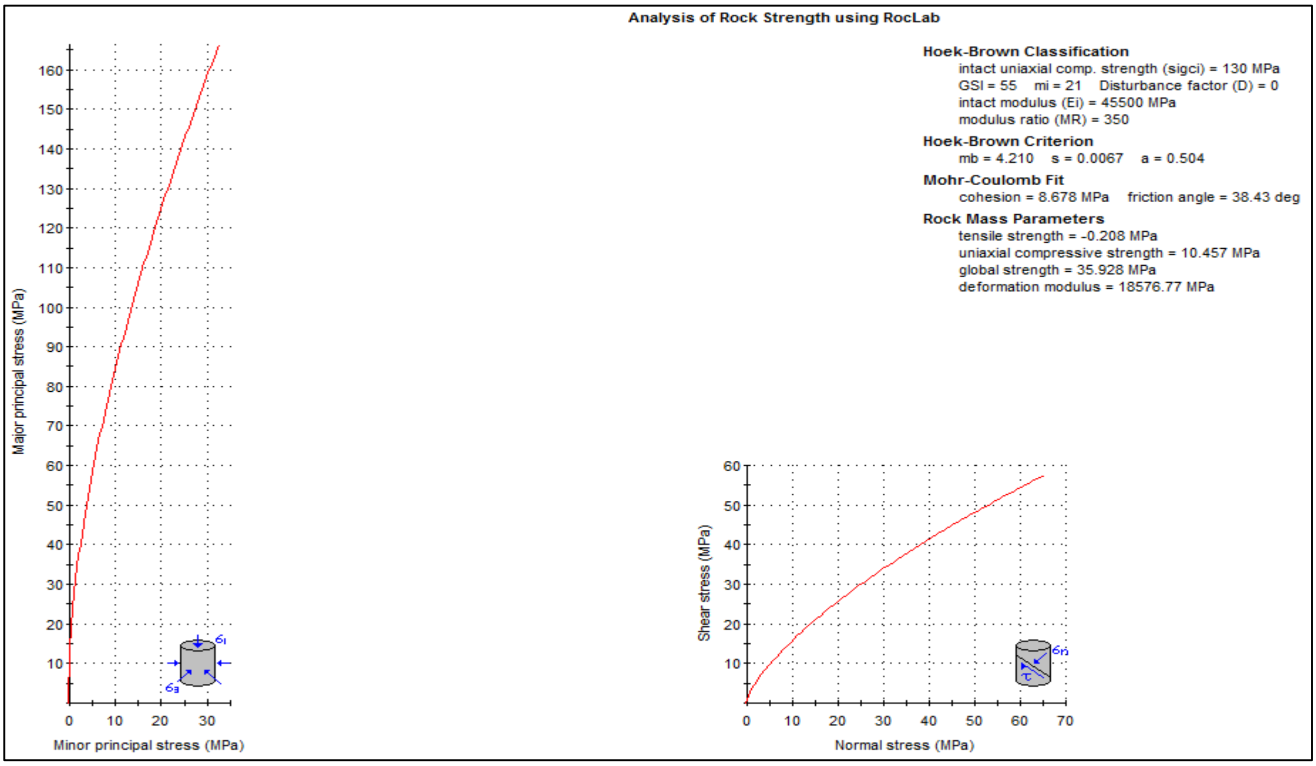
Appendix 6 RocLab estimates for Oreshale



Appendix 7 RocLab estimates for Hanging wall Quartzite (HWQ)



Appendix 8 RocLab estimates for Footwall Conglomerate (FWC)



Appendix 9 RocLab estimates for Footwall Sandstone (FWSST)