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School of Mining Engineering

MSc (Eng) Research Report

Influence of pit wall stability on underground planning and design when transitioning from open pit to sublevel caving

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A research report submitted to the Faculty of Engineering and the Built Environment, University of the Witwatersrand, Johannesburg, in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

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DECLARATION

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ABSTRACT

This research sought to address the influence posed by the pit wall stability and instability on underground planning and design when transitioning from open pit to underground. Conventionally, empirical methods are used and they sometimes lack consideration of rock mass behaviour, groundwater effects, structures as well as geological considerations. This can potentially result in massive failures of pit slopes and subsequent loss of infrastructure, excavations, loss of machinery and human lives. It was against this background that this research sought to reduce mining exposure to the above mentioned hazards. In line with the aims and objectives of the study, this research investigated stress changes around the pit slopes with progression of mining and also the influence of geological and geotechnical conditions on mine planning. This was done so as to determine the zone of geotechnical influence from which planning of the underground mine would be done. Elastic 3D numerical modelling approach was used to determine the expected underground back break and its influence on the underground structure, pit slopes as well as the primary access. Different Factor of Safety shells were modelled, so that the corresponding zone of influence for each Factor of Safety could be correlated to the mine design. The results suggested that a Factor of Safety of two was ideal for this research for underground infrastructure to be outside the zone of geotechnical influence from start to finish of mining the first slice until the last fourth slice of the sublevel caving. This approach yield better projections of rock mass and slope behaviour since it considers a broad range of parameters that include rock mass strength properties, geology, geo-mechanical parameters, groundwater and rock behaviour.

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LIST OF ABBREVIATIONS

3D	Three dimensional
BTS	Brazilian Tensile Strength
c	Cohesion
DME	Department of Minerals and Energy
EDM	Electronic Distance Measurement
FLAC	Fast Lagrangian Analysis of Continua
FoS	Factor of Safety
H-B	Hoek-Brown
GPa	Giga Pascal
GRT	Granite
GSI	Geological Strength Index
Ja	Joint Alteration Number
Jn	Joint Set Number
Jr	Joint Roughness Number
Jw	Joint Water
KIM	Kimberlite pipe
kN	kilo Newton
LGRT	leached granite
m	Metre
MK	Macrocrystic kimberlite
MPa	Mega Pascal
MRMR	Mining Rock Mass Rating
P1	Pit 1
P2	Pit 2
PMC	Palabora Mining Corporation
RQD	Rock Quality Designation
SRF	Stress Reduction Factor
T	Transitional Kimberlite

TK	Tuffisitic Kimberlite
TKB	Tuffisitic Kimberlite Breccia
TCS	Triaxial Compression Strength
UCS	Uniaxial Compressive Strength
UDEC	Universal Distinct Element Code
ρ	Density
σ	Normal stress
τ	Shear strength
ϕ	Friction angle

1 INTRODUCTION

For orebodies that extend beyond the economic depth of an open pit, there is an opportunity to go underground to further exploit the reserve provided the economics allow. Mining engineers prepare and plan for the new challenge of transitioning from open pit operations to underground operations. The planning and design of the underground mine has to be within the confines of safety and economics so as to profitably extract the ore. Furthermore, detailed attention has to be given to the stability of the pit wall and how it will influence the position of the portal and subsequent excavations, that is, primary and secondary developments that will enable access to the orebody, as well as service infrastructure. This research seeks to investigate how the stability of the pit wall bears an effect on underground infrastructure such as positioning of the portal, tunnel accesses, ventilation shafts and pumping stations when transitioning from open pit to sublevel caving. Design of the mine was done using Datamine software, whereas stability models and zones of influence of pit wall displacement was run in FLAC3D.

1.1 Problem Statement

The positioning of underground infrastructure when transitioning from open pit to underground operations is a difficult exercise which involves balancing of geotechnical aspects with mine planning and design, all feeding into the economics of the project. Poor planning and inadequate consideration of geotechnical properties can have devastating consequences such as uncontrolled backbreak, failure of pit walls, loss of lives, loss of equipment, excessive dilution and even loss of the mine. This research will thus investigate the influence of pit wall stability on underground planning during transition from open pit to sublevel caving operation. The report will further seek to derive a numerical analysis aided design for the positioning of underground infrastructure outside the zone of pit wall instability and geotechnical influence, which is a study area on the boundary of two disciplines, namely rock engineering and mine planning.

1.2 Justification of the Research

As a result of the probable geotechnical risks on the infrastructure when transitioning from open pit to underground, there was a need to undertake this study because of the following:

- The presence of stress concentration around the pit wall and the reduced confinement posed a stability risk
- Positioning of the primary accesses, ventilation shafts, workshops, refuge bays and pump chambers had to be properly planned to be in stable ground conditions
- For the first and second mining cuts, the in-pit haul roads had to be utilised for out of the mine transport and as such failure of pit wall needed to be avoided to ensure access roads are not lost
- To identify a breakaway position for the primary access of the mine (declines) close to the pit bottom where stable conditions were anticipated
- To come up with a mining sequence that would ensure maximum extraction is achieved without the hindrance of excessive slope failures of the pit wall as the depth of mining increased
- There was no crown pillar to be left in the transition zone at the pit bottom hence the effect of water to safety had to be investigated

1.3 Research Aim

The aims of this research are to:

- Determine suitable siting of underground infrastructure taking into consideration the stress changes around the pit slopes and stope walls as mining takes place
- Identify slope stability risks that are encountered during transition of open pit to underground mines
- Investigate the influence of geological and geotechnical conditions on mine planning

1.4 Objectives of the Research

The main objective of this research is to determine the zone of stable ground conditions from which planning of the underground mining will be done during transition from open pit to underground. In order to fully address the aim, the following focus areas stemming from the main objective will be looked at:

- To have a mine plan and design that avoids loss of access at the in-pit portals and prevents flooding of the open pit and underground operations.
- To investigate how the zone of mining influence changes with increasing mining depth
- To have an analysis of numerical modelling of the expected underground back-break and its influence on the underground structure, pit slopes and the primary access (declines)

1.5 Research Assumptions

The author assumes that sound mining engineering design and conduct was applied from the beginning of the open pit project until the pit reached the economic pit limit. Consequently, the subsequent design from the pit bottom to underground is validated and based on this assumption.

1.6 Structure and Contents of the Research

Chapter 1 contains the introduction, definition of the research statement, research objectives and the contents of the research. Chapter 2 focuses on the literature review in areas concerned with the research. In addition, Chapter 2 provides a summary of relevant published work in books, journals and conference papers particularly in the subject of the transition process of open pit to underground, and factors that have to be considered. The evaluation of the mechanisms of failure for pit walls, description of the problem area, geology, pit layout, slope angles and groundwater condition are also discussed under Chapter 2. The methodology used to conduct the research is detailed in Chapter 3. Chapter 4 discusses relevant case studies for this research. Results and analysis of the case studies are discussed in Chapter 5, including numerical modelling. A relevant Factor of Safety for mine design and planning is also discussed. The report ends with Chapter 6, which presents the conclusions and recommendations. A summary of the research outline is shown in Table 1:

Table 1: Structure and contents of the research

Chapter 1	• Introduction • Problems statement • Aim and objectives
Chapter 2	• Literature review
Chapter 3	• Research Methodology
Chapter 4	• Case studies
Chapter 5	• Results and analysis
Chapter 6	• Conclusions • Recommendations

2 LITERATURE REVIEW

2.1 Introduction

The open pit to underground mining transition requires sound knowledge and application of engineering practice to ensure a safe and economic mining operation. The availability of a considerable and mineable ore reserve footprint below the economic pit limit presents several transition challenges to the mining operation. As highlighted by Olavarría, et al. (2006), major challenges that are encountered during transition are:

- Consideration of the geotechnical aspects that have an influence on mine planning and design
- Positioning of both surface and underground infrastructure
- Fulfilment of the project deadlines and the realisation of the set project and deadline targets

Of the three challenges above, this research will focus mainly on the first two, which are; how the geotechnical aspects influence mine planning and design and also determining the positioning of the underground infrastructure. Attention is also given to both slope and underground stability. Adequate consideration of geotechnical aspects with respect to planning and designing is also emphasized by DME (1999) by taking note of the following ground control measures:

- Consideration for geological structures and their influence on wall stability
- Taking note of the shear strength of the rocks and the geological structure
- Analysis of groundwater, drainage patterns and dewatering strategies and their subsequent influence on the pit wall and underground stability over time
- Design of rock reinforcements where necessary
- Analysing the stability of the pit wall by looking at the geometry of the pit and the sequence of mining

- Monitoring the pit wall and stability conditions as mining takes place

The dimension, shape, orientation and position of the mine infrastructure can be adjusted and delineated accordingly after consideration of geotechnical input so as to come up with an integrated mine plan.

2.2 Factors that Influence Mine Stability

The structural integrity, stability and safety of a mine should be adequately considered in mine design so as to minimize probable losses that can result if not properly managed. Mine stability is affected by several geotechnical aspects that will be discussed in the following subsections. During the transition of open pit to underground; an assessment of the stress conditions, evaluation of the geological discontinuities, groundwater, lithology and alteration is important as these are factors that affect slope failure, that is, the stability of the mine. The essence of this section was to build an improved understanding through detailing how the geotechnical aspects have an influence on the stability of the pit and underground mine during transition.

2.2.1 Structural Geology

When a slope or excavation is made in a rock mass, its stability is influenced by the presence of geological structures such as joints, dykes, faults, bedding planes and foliation (Hoek, et al., 2000). These structures are briefly described by Wyllie & Mah (2004) and are listed below:

- Fault - this a discontinuity where there has been movement or displacement in the rock mass
- Bedding - this forms as a result of parallel deposition, forming contacts at each bedding plane
- Joints - these are discontinuity where there has been no displacement in the rock mass
- Foliation - this is formed as a result of parallel layering in metamorphic rocks, such as in schists or mica
- Dyke - a magmatic or sedimentary intrusion that cuts across a contiguous mass of rock or rock strata

Due to the significance of geological structures in the behaviour of rock mass, their identification and mapping should be thorough so as to incorporate the information during planning and design. In particular the stability of pit slopes is sensitive to discontinuity properties and orientations. In most cases, the discontinuities have an effect on the nature and mechanism of failure that will result when the shear strength is exceeded by the shear stress exerted on the discontinuity of the rock mass. For example, when two joints obliquely intersect one another along a line (apparent dip) and the driving downward forces exerted by the wedge on the contact surface exceed the shear strength, sliding results and the wedge fails (Hoek & Bray, 1981). The characteristics of the geological discontinuities such as the orientation, roughness, dip and persistence are of great importance in stability analysis and are illustrated in Figure 1.

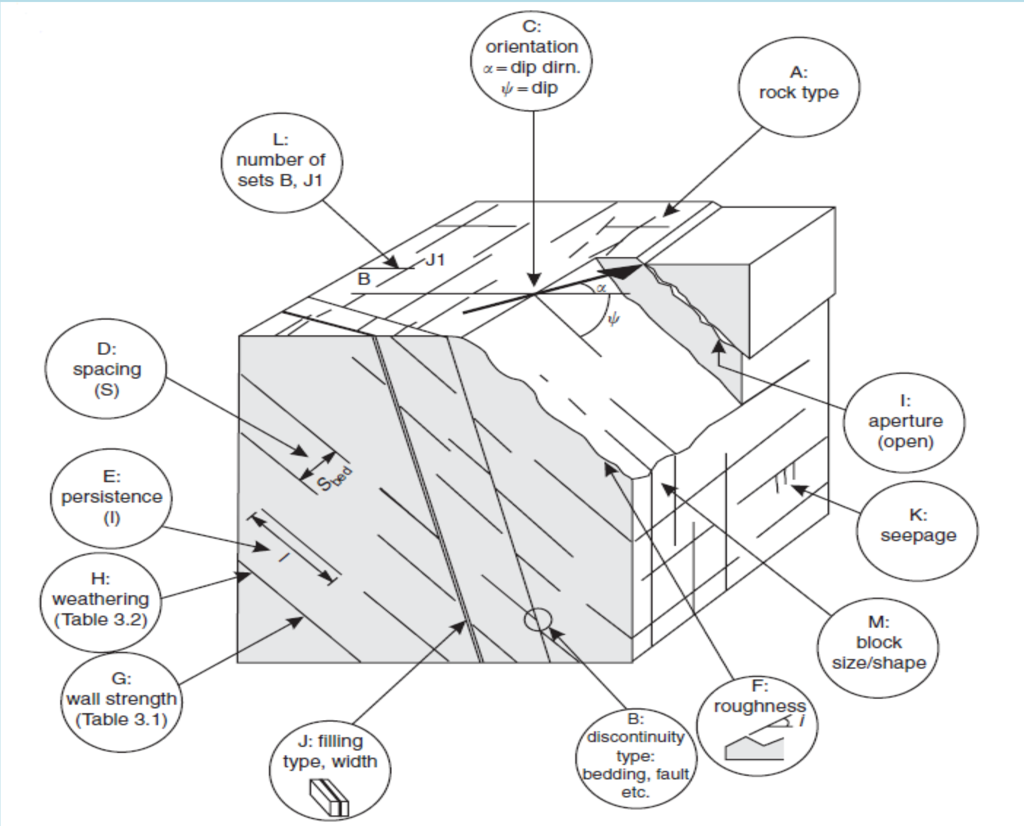


Figure 1: Geological discontinuities and characteristics (Wyllie & Mah, 2004)

The different failure mechanisms in rock engineering are a result of different conditions in terms of discontinuity orientation, persistence and spacing. Variability of these properties give rise to different block sizes, joint frequency and behaviour of the rock mass.

According to Wyllie & Mah (2004), discontinuity component that has a bigger influence on stability is orientation; however, the effect of spacing and persistence cannot be understated. Orientation is defined by dip of the discontinuity, which is the angle measured below the horizontal and the dip direction, which is measured clockwise from the north. Strike is perpendicular to the dip direction of the discontinuity. The pictorial representation of strike, dip and dip direction is illustrated in Figure 2.

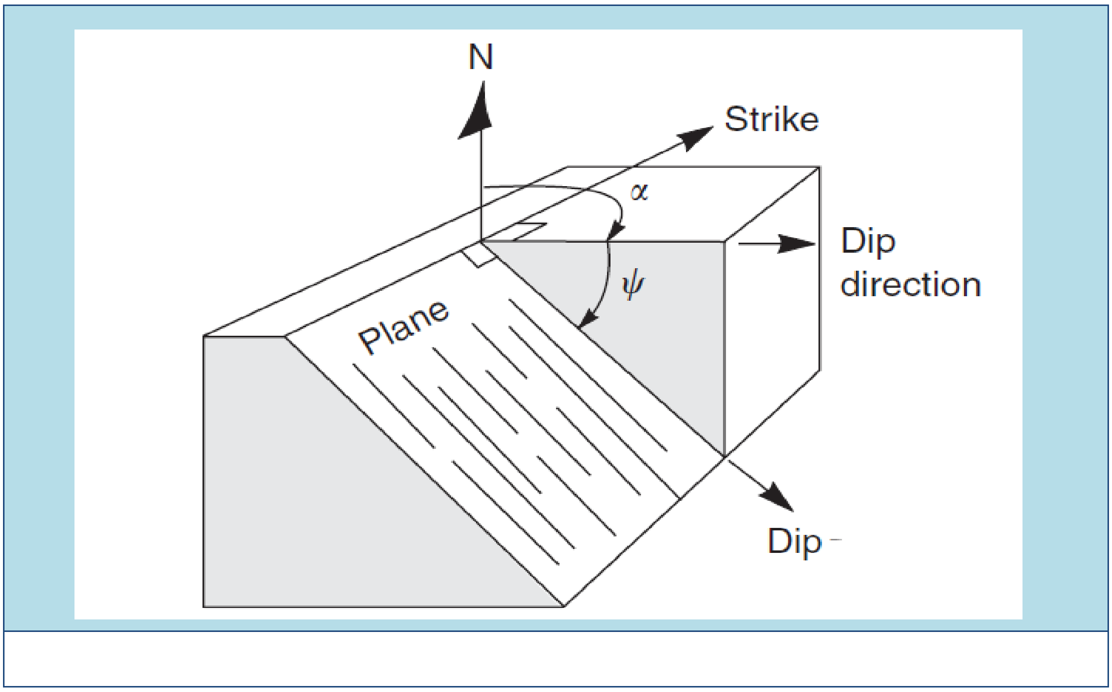
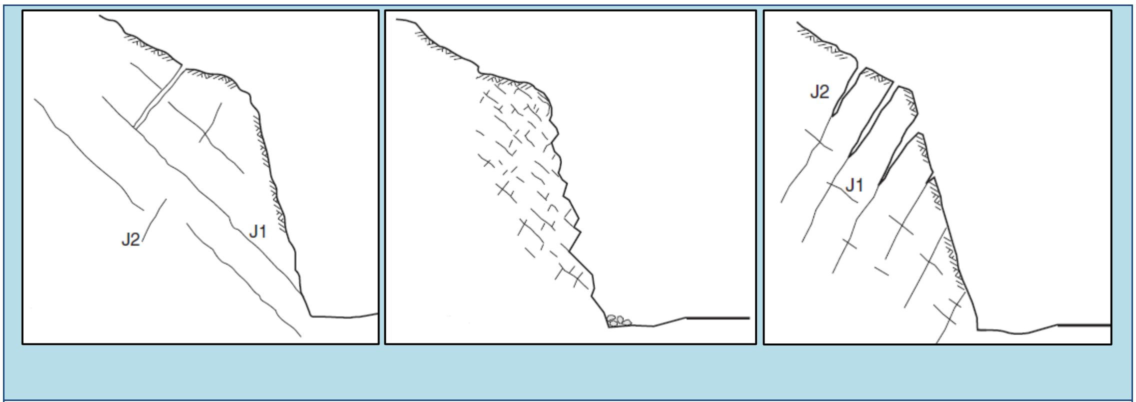


Figure 2: Illustration of orientation of discontinuities after Giani (1992)

In terms of quantification of the length of the discontinuity, that is ‘persistence’, Wyllie & Mah (2004) highlighted that the degree of persistence also has an effect on the slope stability. Hoek & Bray (1981) said that joints that dip persistently out of the face potentially form unstable blocks that slide along the discontinuity surface. Short and low persistence joints were suggested to form dislodging and ravelling of small blocks, whilst toppling can result if the joints are dipping into the face as illustrated in Figure 3. The spacing of joints determines the size and magnitude of the failure mechanisms. Wyllie & Mah (2004) noted that the classification of spacing ranges from wide (greater than 2m) to very narrow (less than 6mm) for each established set of joints. Furthermore, joint spacing has a direct influence on the rock mass rating in terms of strength and therefore also affect the stability of slopes or underground mines.



Unstable Sliding Blocks	Ravelling	Toppling

Figure 3: Effect of persistence on failure mechanism (Wyllie & Mah, 2004)

The local geological setting of a mine is also of interest as structures such as large structures and faults can separate open pit and underground workings into different domains or ground control districts. This would require an investigation and analysis of each zone differently since the rock mass quality may be different for the subdivided zones (Hoek, et al., 2000).

2.2.2 Groundwater

The influence of groundwater on stability is vital in mine planning and design, for if neglected can lead to inadequate design which can have disastrous effects. As highlighted by Styles (2009), groundwater reduces the effective normal shear stress through the uplift pressure exerted in the opposite direction. In slope stability, Wyllie & Mah (2004) summarised the challenges that groundwater presents and why its presence is unfavourable for stability as follows:

- The upward forces exerted by water pressure on the surface of discontinuities reduce the shear strength thus inducing failure

- Water and moisture in some rock masses speeds up the rate of weathering thus reducing the effective shear strength
- The volume changes as a result of temperature variations (freezing and thawing) of water can cause widening and wedging of fissures, as well as blockage, thereby a build-up of water pressure due to confinement and obstruction to flowing
- Water can cause loosening of rock masses through washing away of weak infilling

The inclusion of groundwater in design requires sufficient data collection of the closest estimates of water pressures within the rock mass. This can be done by either determining the hydraulic conductivity of the site or through the use of measurement instrumentation such as piezometers (Bell, 1992). Hydraulic conductivity gives a relationship of the flow rate of the groundwater through the rock mass to the pressure gradient that is offered (Wyllie & Mah, 2004).

Another interesting parameter that relates to groundwater pressure is the porosity of the rock mass, which is simply the ratio of the volumetric void of the rock mass to the total volume of the rock. Low porosity rock mass thus effectively has lower water pressure due to small discharge capacity.

In the analysis of groundwater in slopes, flow nets are usually used to represent the flow of groundwater. A flow net consists of flow lines, equipotential lines and a reference datum from which measurements are referenced (Venkatramaiah, 1993). A flow line as the name suggests represents the trajectory that water takes when flowing through the saturated rock whist equipotential lines represents points which can raise water to the same vertical level, that is, the same head (Venkatramaiah, 1993). An illustration of a flow net is shown in Figure 4.

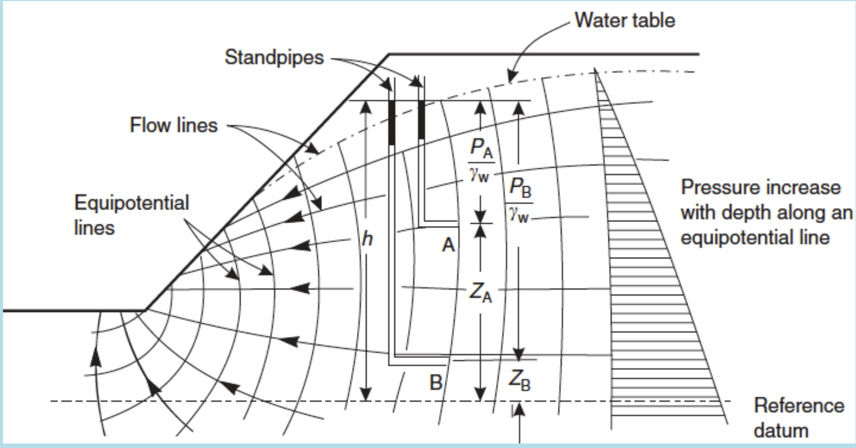
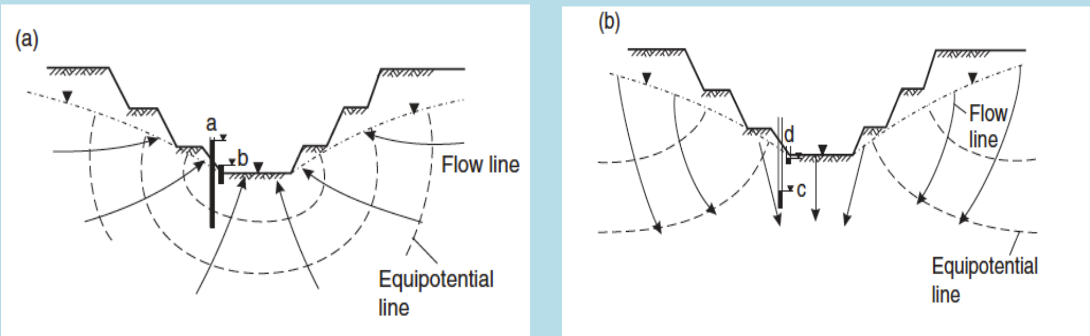


Figure 4: Typical slope flow net (Wyllie & Mah, 2004)

The importance of groundwater analysis through flow nets is vital when also transitioning from open pit to underground. As emphasized by Wyllie & Mah (2004), an understanding of the distribution of water pressures in slopes and subsequently the pit floor is required, so as to be able to identify whether water flow is towards or away from the pit. In cases where the pit is located in an area where the net saturated flow of groundwater is away from the pit, low pressure is generated below the pit bottom and in contrast if the pit is located in an area where groundwater flow is towards the pit, water pressure builds up below the floor of the pit (Wyllie & Mah, 2004). This distribution of groundwater pressure with respect to location of the pit in terms of recharge or discharge is shown in Figure 5.



a) Pit in recharge

b) Pit in discharge

Figure 5: Recharge and discharge groundwater conditions (Wyllie & Mah, 2004)

The correlation of groundwater and geological structures is important in slope stability analysis. In areas where there are low persistence joints which do not reach or extend to the skin of the excavation, water pressure develops. This is in contrast to the high persistence joints which allow water to drain out, thus reducing water pressure within the excavation (Giani, 1992). Porosity is also another groundwater condition that is related to the geological settings, and thus stability. The degree of porosity has an effect on the pore pressure in that jointed low-porous rock mass creates opportunity for water to occupy the joints thus increasing the water pressure. This porosity phenomenon was highlighted by Wyllie & Mah (2004) who referenced rock failures on steep slope faces occurring after heavy rainfalls. The failures are attributed to increased water pressure as water is channelled through joints due to low porosity.

2.2.3 Rock Mass Classification

The geotechnical condition and strength of the rock mass also has a significant effect on the stability of slopes. The input parameters for rock mass classification systems to determine the strength of the rock mass are dependent on the classification system used, but most systems share the same fundamentals. The geotechnical values assigned to rating parameters are weighted empirically based on relative impact to the rock mass. Rock mass can be classified quickly and visually through experience using the Geological Strength Index (GSI); and also through calculation from Barton’s Q-System and Laubscher’s Mining Rock Mass Rating. The GSI system is a quick on-the-field visual classification method that takes into account field observations in terms of the geology, lithology, the visible structures and persistence of the joints on the rock mass outcrop (Marinos, et al., 2005). GSI is easy to use, economical and useful for initial analysis before a detailed classification method is used. The in-field observations are then correlated to the GSI chart illustrated in Figure 6.

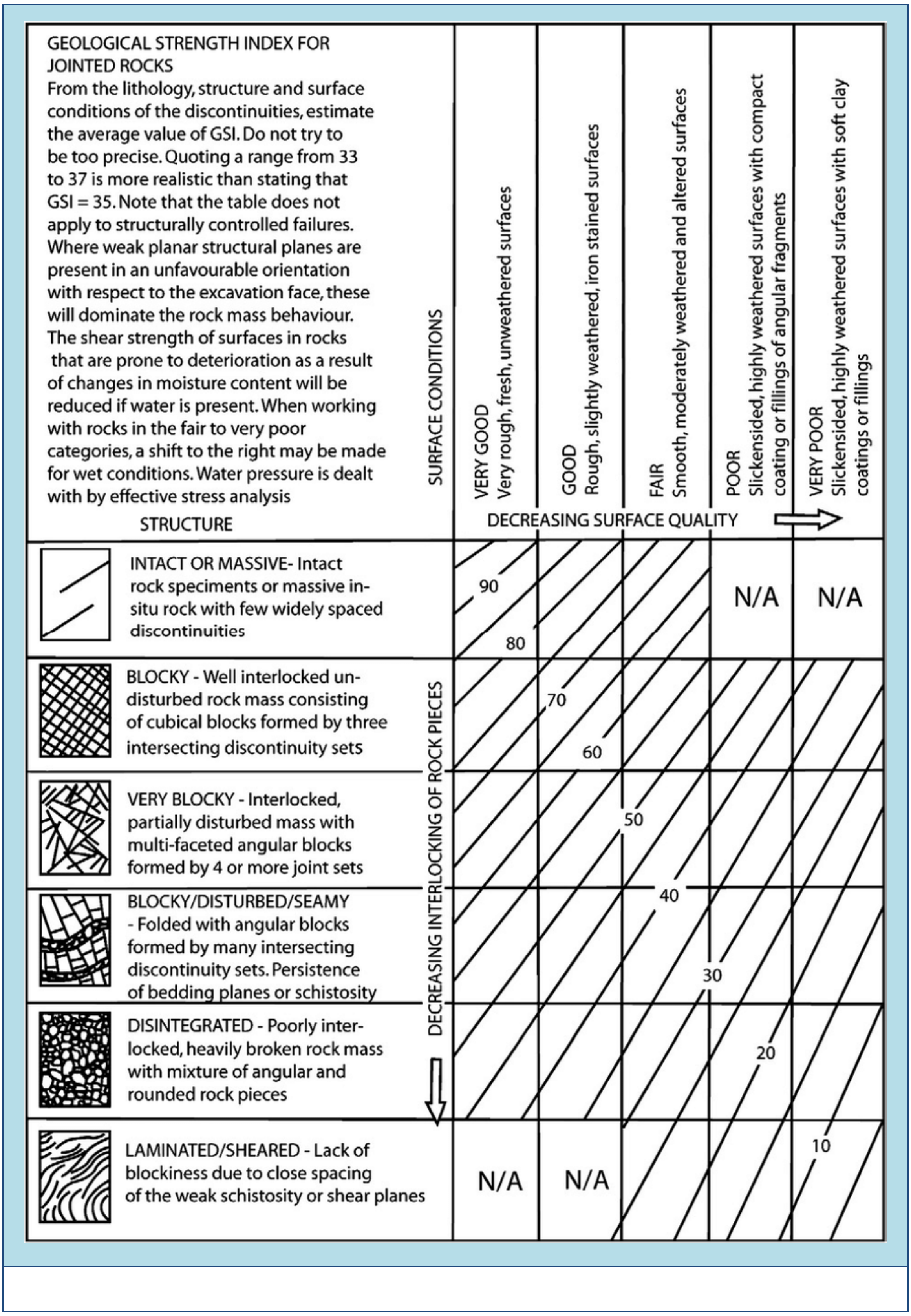


Figure 6: GSI Chart (Marinos, et al., 2005)

Another classification system that is used in rock mass classification is Barton's Q-system and is largely handy for underground tunnels, drives and haulages during the open pit to underground transition. As part of the evaluation of the rock mass, the Q-system was first published by Barton, et al. (1974). It was developed with the aim of being a helpful tool for evaluating the need for support in tunnels and rock caverns. As described by Barton, et al. (1974), the rock mass quality using the Q-system is given by:

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF} \quad \text{......} \quad \text{Equation 1,}$$

where $\frac{RQD}{Jn}$ = a measure of rock block size

$\frac{Jr}{Ja}$ = a measure of joint shear strength

$\frac{Jw}{SRF}$ = a measure of confining stress

The six input parameters provided by Barton et al (1974) in Q-system are described as follows:

- **RQD - Rock Quality Designation -** RQD is the percentage of the sum of intact core pieces longer than 100 mm in the total length of core. Alternatively, it can be obtained from inspection of exposed rock surfaces through the determination of the number of unhealed joint planes per m^3 of rock to give a volumetric joint count called J_v (Palmstrom & Broch, 2006).
- **J_n - Joint Set Number -** This corresponds with the number of joint sets present in a mapped rock mass locality or project area.
- **J_r - Joint Roughness number -** This refers to the texture of joints, both small scale and large scale.
- **J_a - Joint Alteration number -** This is the rating that incorporates weathering, coating, thickness and nature of gouge infill present in joints.
- **J_w - Joint water -** This rating allows for water pressure on the joint walls as well as the potential for the softening and outwash of joint gouge.
- **SRF - Stress Reduction Factor -** This is a measure of rock stresses and loosening, squeezing and swelling loads in rocks.

The Mining Rock Mass Rating (MRMR) was derived from the in-situ Rock Mass Rating (RMR). to The classification was customised to specific mining environments. The inclusion of the adjustment parameters such as weathering, mining induced stresses, joint orientation and blasting effects made an important contribution to mine planning and design as it provides acceptable guidelines, though detailed investigations have to be made for each of the input parameters (Laubscher, 1990). In the assessment of rock strength using RMR, the following five parameters have to be taken into consideration from the RMR parameter classification tables as shown in Appendix A (Laubscher, 1990) :

- Intact Rock Strength - this is the unconfined uniaxial compressive strength (UCS) of the rock
- Joint Spacing - this is the separation distance between joints and discontinuities
- Rock Quality Designation - RQD is the percentage of intact core pieces longer than 100 mm in the total length of core
- Fracture Frequency (FF/m) - this is a measure of joints or discontinuities that are cutting across a sampling line. The less they are, the stronger the rock mass
- Joint Condition and Water - this involves the evaluation of the state of the joint in terms of surface properties, joint alteration, filling, expression and water.

The derived Rock Mass Rating is then adjusted cumulatively with percentages from weathering, joint orientation, mining induced stresses and blasting effects. The importance of the rock mass ratings in defining the strength of the rock mass in which mining will be carried out lies in the correlation of the strength to the stability/instability of excavations (Laubscher, 1990) and estimation of support requirements. In slope stability, Terbrugge (2015) presented an empirical correlation between adjusted mining rock mass rating and recommended slope angles, in support of Laubscher (1990). The empirical correlation showing how important rock strength is on stability is illustrated in Table 2.

Table 2: Mining Rock Mass Rating and Slope Angles (Terbrugge, 2015)

Adjusted MRMR Rating	100	90	80	70	60	50	40	30	20	10	0
Overall Slope Angle (°)	>75	75	70	65	60	55	50	45	40	35	<35

2.2.4 Geometry

One of the critical aspects that affects open pit stability is the pit geometry. Depending on the competence of the structural domains of the open pit, different slope angles, inter-ramp angles and widths may be adopted for design and stability purposes. In competent rock, the pit slope angles may be steeper as compared to weak rock. Slope design is influenced by the geotechnical conditions, geological setting of the region, depth of the pit and groundwater conditions (Read & Stacey, 2009). Slope geometry characteristics that are pertinent in slope stability are illustrated in Figure 7.

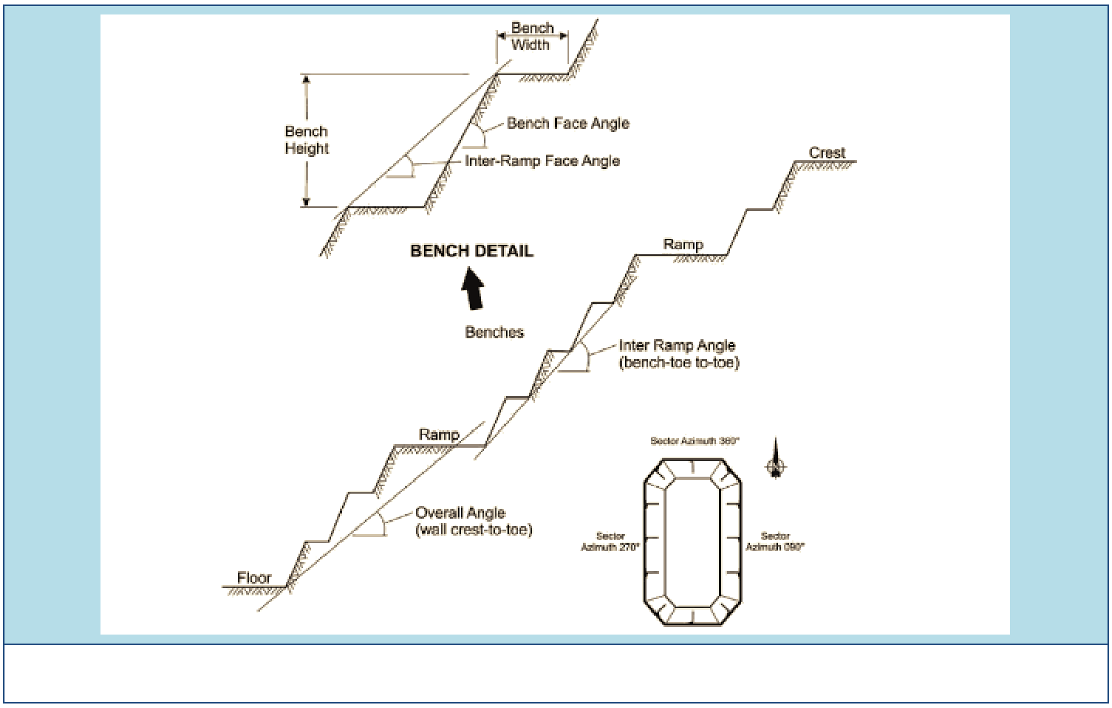


Figure 7: Slope geometry in open pit (Read & Stacey, 2009)

The curvature of a slope will have an effect on the stability of the slope and plays a part in the determination of safe slope angles. The curvature concept is based on pit slope plan analysis. Hoek, et al. (2000) said that the lateral stresses and confinement provided by the rock mass on the sides of concave slopes aid in the slope stability. The magnitude of these lateral stresses on the stability of the slope can be seen in the factors of safety for concave and convex planar curvatures in Figure 8.

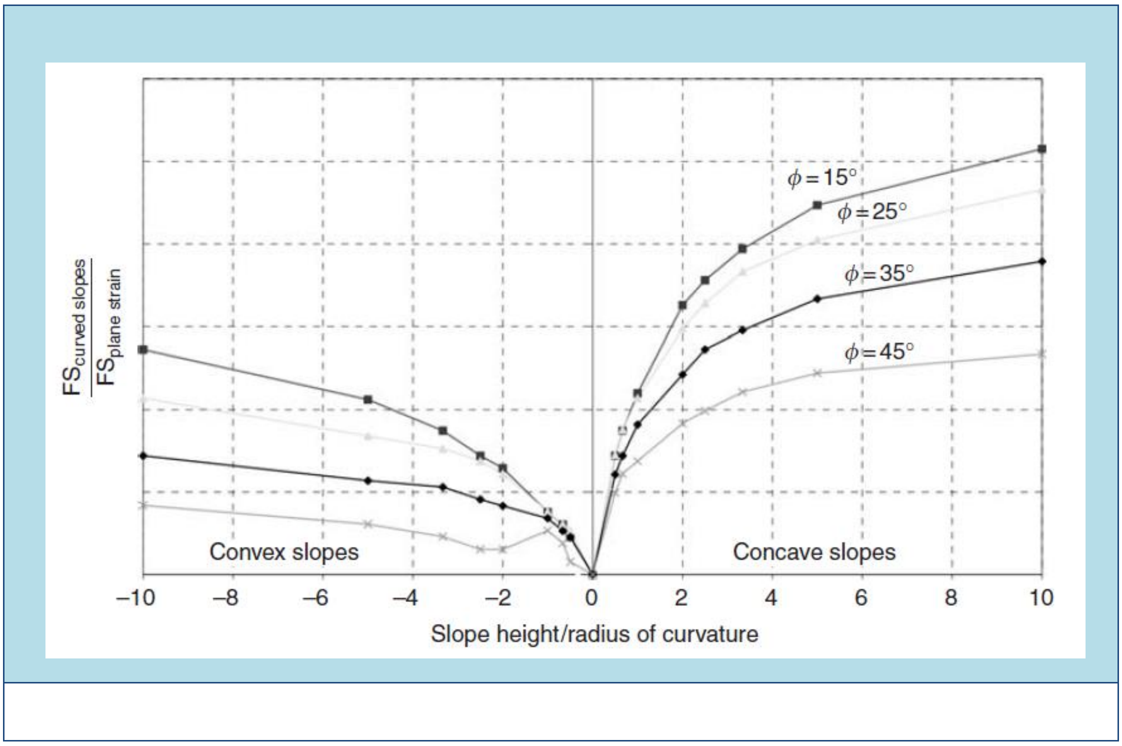


Figure 8: Relationship between curvature and Factor of Safety (Wyllie & Mah, 2004)

From Figure 8, it can be seen that for the slope height to radius of curvature, concave slopes have a better Factor of Safety. The rate of increase of Factor of Safety of the concave slope is more than that of convex, due to the lateral confinement stresses as described before.

2.2.5 Alteration

Zone of altered rock can have an effect on stability due to different properties of the rock mass strength. The alteration effect depends on the strength alteration material, for example, Wyllie & Mah (2004) explained that low strength alteration infills such as montmorillonite have an adverse effect on the stability as opposed to high strength alterations with high strength materials such as quartz. Alteration therefore plays a significant role in the overall rock competency and the rock mass condition.

2.2.6 Stress Conditions

Before mining is done, the rock mass is under virgin stress, that is, pre-mining stresses will be inherent in the rock. The subsequent mining of an area causes a disturbance that results in stress re-distribution around the excavated area. It follows that the action of the vertical stress exerted by the overburden and the horizontal stress around the excavation has an effect on stability. Of importance is the behaviour of the rock mass to the induced stress. As highlighted by Duncan, et al. (2014), when an excavation is made, the stresses that were carried by the removed rock mass are redistributed and the result will be increase in stress in other areas such as abutments of the stope. When this happens, there may be increased deformation, displacement and probable failure depending on the shear deformation along the planes of weakness (Duncan, et al., 2014). It thus becomes critical to carry out stress measurements and do numerical modelling and analysis to understand and predict instability and how stress changes with mining.

2.2.7 Weathering

The deterioration of contact surfaces at discontinuities as a result of weathering cause a reduction in roughness and shear strength of joints. Wyllie & Mah (2004) defined weathering in two forms; that is decomposition and disintegration. Decomposition occurs when a chemical reaction affects alterations on the chemical nature of the rock such as when minerals in rock are oxidised, reduced, hydrated or carbonated. Disintegration occurs when the external environment induces cracking, splitting and disintegration of the rock mass such as during swelling and freeze and thaw action by water. The effect of weathering is that it decreases the shear strength of the rock mass as a result of the reduction of strength of the intact rock (Mohammed, 1997). The different degrees of weathering are highlighted by Wyllie & Mah (2004) in Table 3.

Table 3: Weathering classification (Wyllie & Mah, 2004)

Term	Description	Grade
Fresh	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.	I
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition.	II
Moderatiely Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.	III
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.	IV
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

2.2.8 Blasting

In mining, blasting involves the use of explosives for rock breakage in a controlled manner as far as reasonably practicable. When the explosives are detonated, they are converted instantaneously into high temperature gases which when confined into a blast hole results in very high pressures in the range of 1-5 GPa, that are exerted against blast hole walls (Wyllie & Mah, 2004). This energy is transmitted into the surrounding rock mass in the form of shock wave, which travels at a velocity of detonation speed of around 6900 m/s. This energy thus results in:

- Crushing around the blast hole
- Relative radial motion
- Fracturing of the rock
- Rupturing and in flight collisions

The high-pressure gases produced by detonation contribute to the movement of fractured rock in the direction of least resistance, that is, the free face. The shock energy breakage mechanism is a result of the strain or compressive pulse from the detonating explosive charge (Wyllie & Mah, 2004). The gases produced penetrate and force their way into the crushed zone and flow into the cracks and proximal discontinuities thereby wedging and opening up the radial cracks and natural discontinuities within reach (Wyllie & Mah, 2004). The purpose of blasting is therefore to fragment the rock to the required size distribution optimal for conveyance to the crusher and plant. As a result of the rate of reaction and quantity of energy produced in blasting, not all the energy can be channelled and directed for rock breaking and movement of the muck. Some of the energy from blasting together with the compressive wave strain and the subsequent blast vibrations cause blast damages to the remaining in-situ rock behind the face thus causing instability. In order to minimize the blast damages resulting from blasting, controlled blasting on the last row before the final wall or face is practised. Controlled blasting includes pre splitting and post splitting, depending on whether the last rows are fired first or last. Smooth wall blasting where the perimeter holes are loaded with decoupled charge can also be useful in minimizing the damage caused by blasting.

In slope stability analysis, the quality of blasting is of importance as it has an adjustment bearing on the Mining Rock Mass Rating classification system. The adjustment of rock quality as a result of excavation technique is shown in Table 4.

Table 4: Adjustment factor for blasting (Hustrulid & Bullock, 2001)

Excavation technique	Adjustment (%)
Boring	100
Smooth wall blasting	97
Good conventional blasting	94
Poor Blasting	80

Poor quality blasting can cause excessive vibrations and dynamic loading of slopes as well as leaving cracks and bad hangings on the face. These cracks are susceptible to water ingress thus reducing the shear strength of the discontinuities as a result of water pressure (Hustrulid & Bullock, 2001). This will increase risk of slope failure in the open pit through rock-falls, wedge failure, creeping of the slope and decreased stability of slope benches and ramps.

2.3 Identification of Stable or Unstable Areas

An effective and active geotechnical monitoring system is vital in the identification of stable or unstable areas in/around the pit and also the open pit to underground mining zone. In addition to the visual inspections around the mining area, more detailed techniques that can be used in the investigation of unstable areas will be discussed under this section. Owing to the unexpected behaviour of the rock mass, it is important to identify stable and unstable areas around the pit to counter-manage the identified hazards. Slope monitoring and management thus becomes a vital and necessary exercise, which enables the geotechnical team to be up to date with the slope condition in terms of stability. Deterioration, displacement and groundwater can all be monitored and will be discussed in detail under this section. Early and timely detection of unstable areas assist in saving life, equipment and property of the mine that could potentially be lost if a catastrophic and unforeseen failure occurs. As suggested by Wyllie & Mah (2004), it is critically important to have an understanding of parameters to monitor so that correct selection of the measuring instrumentation can be done. Kliche (2009), said that the objectives of a slope monitoring program are:

- Manage risk through a safe operation that protects against losses in terms of workforce and equipment
- To forewarn instability thus allowing adjustment and modification of the mine plan so as to minimize impact of the instability
- To collect useful geotechnical information in order to analyse slope failure mechanisms and design remedial action plans of the rock slope

2.3.1 Geotechnical Mapping

This is done on the benches, pit walls and underground, paying attention to the orientation of the main joint sets, the persistence and the joint frequency. This process provides quantitative and qualitative data for assessments and evaluation of the pit wall stability (Knight Piesold Consulting, 2012). When pre-set benchmarks are exceeded in terms of movement, some monitoring systems activate a connected alarm to raise awareness to the management or geotechnical team (Wyllie & Mah, 2004).

2.3.2 Crack Mapping

In most cases, movement of a slope is characterised by tension cracks at the crest or face of the slope. Consequently, crack mapping becomes an important method of measuring the cracks developed by the tensile stresses in the slope.

Crack mapping involves installation of a pair of reference points on opposite sides of the crack. This arrangement of references on either side of the tension crack makes it possible to measure any change in distance between them. An illustration of the crack monitors is shown in Figure 9.

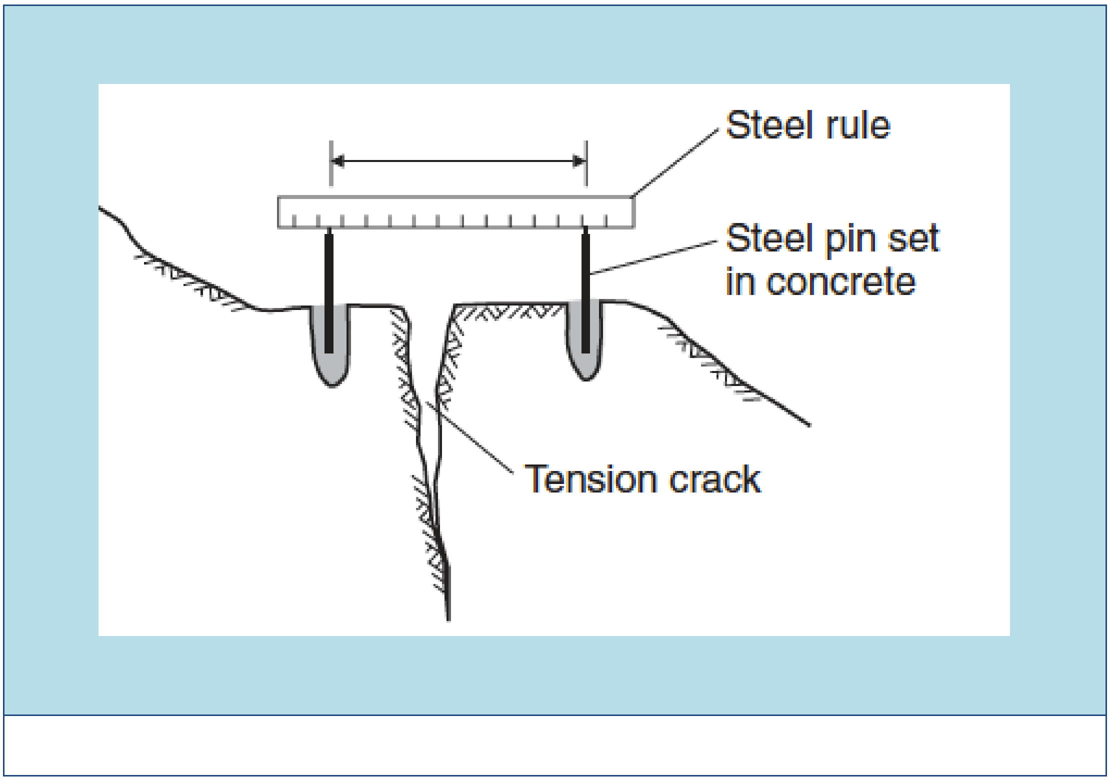


Figure 9: Use of pins in crack mapping (Wyllie & Mah, 2004)

When there is a series of cracks within the slope and there is need to measure the cumulative movement, a wire extensometer connected to an alarm system through a trip switch can be used. The arrangement will consist of a measurement station from which a tensioned wire will be connected between the station and the reference. When movement occurs and the wire extends beyond the calibrated limit, the trip switch on the measurement station is triggered. For slope monitoring to be effective with crack mapping, it is often safe and ideal for the measurement or reference point of the system to be sited on stable ground (Wyllie & Mah, 2004).

2.3.3 Surveying

Survey monitoring is a useful approach in slope monitoring and identification of unstable zones around the pit. Jooste & Cawood (2006) said that survey monitoring provides an opportunity to reduce risk on the operation and safety is thereby improved. Survey monitoring ranges from visual inspection to technologically and sophisticated automated measuring of displacement or movement on the faces. The frequency of visual inspections vary from mine to mine but is usually commensurate with the level of activity and is mostly done after blasting or after heavy rains (Jooste & Cawood, 2006). Usually, geotechnical zones that are classified as low risk areas are visually inspected. High risk areas are systematically monitored using survey equipment, and this is done jointly by the survey and rock engineering departments (Jooste & Cawood, 2006). The frequency of measuring, density of prisms and accuracy is determined by the degree of risk as well as the rate of slope movement. A case study of monitoring frequency for Venetia mine was done by Jooste & Cawood (2006) and is summarised in Table 5.

Table 5: Monitoring frequency against rate of movement (Jooste & Cawood, 2006)

Movement size (mm/day)	Monitoring frequency
0 to 2	once/month
2 to 5	once/week
5 to 10	once/2 days
10 to 50	once/ day
>50	continuous observation

Principles of survey monitoring

The survey system usually used in slope monitoring is the electronic distance measurement (EDM). Wyllie & Mah (2004) said that for a survey system to be effective, the following basic components are required, and are also shown in Figure 10:

- Reference points are required to be set up on stable ground viewable from the instrument stations
- Instrument stations are also set up on stable ground preferably in a triangular set-up
- Prisms are set up systematically around the pit

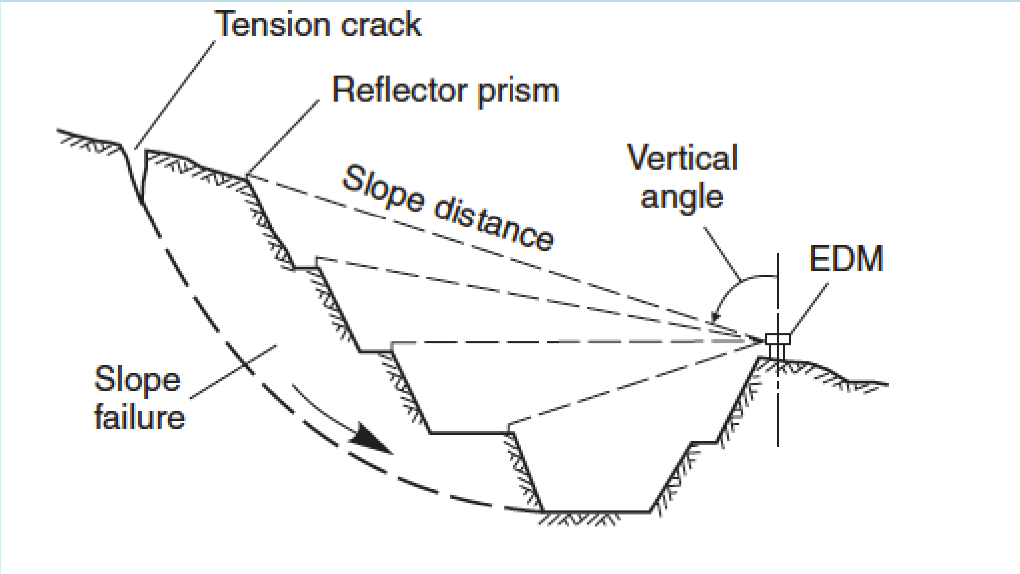


Figure 10: Displacement measurement (Wyllie & Mah, 2004)

For improved precision and accurate measurement, it is important that the measuring orientation is set up such that it is approximately in the direction of movement and slope displacement. A typical survey set up is illustrated in Figure 11.

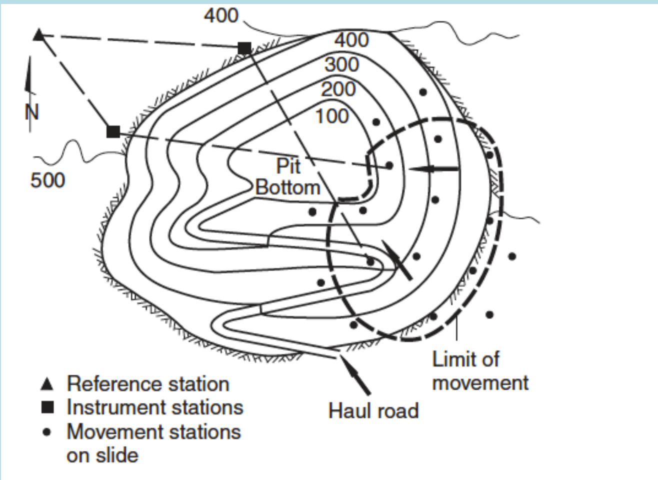


Figure 11: Instrument and measuring stations (Wyllie & Mah, 2004)

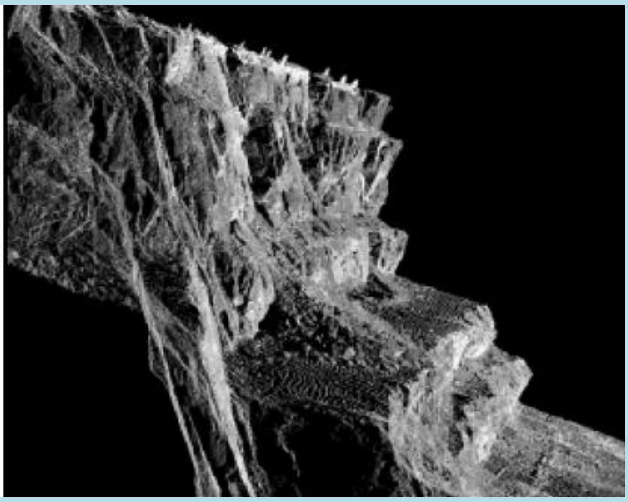
Displacement is therefore determined through change in coordinates of the reference prisms on the high walls either vertically or in the horizontal plane or both. The survey department collects the data and the rock-engineering department analyses it to determine areas with potential failure and high risk.

2.3.4 Laser Image Monitoring

This is an advanced monitoring system that involves the use of laser scanners with cameras for slope monitoring (Little, 2005). A Site Monitor software is used in conjunction with the laser and it allows important input information such as:

- User defined monitoring points
- Frequency of monitoring
- Grouping of points

With laser monitoring, certain reference points are identified and their exact coordinates programmed into the laser machine, thus acting as "laser" prisms. Data from laser monitoring shows as a point in three dimensional mode cloud and can be processed into a contour map and manipulated to suit the requirements of the user. A typical point cloud image processed from a laser scan of a slope is shown in Figure 12.



Laser installation	Laser point cloud

Figure 12: Laser scan monitoring machine and resultant point scan (Little, 2005)

As noted by Wyllie & Mah (2004), a series of laser scans from a reference point allows the position of slope face and highwall to be compared between successive scans thereby determining the location, size and orientation of displacement. Details of movement is gathered by comparing point by point of successive scans over time. Unstable areas can therefore be identified allowing through progressive monitoring.

2.3.5 Piezometer

An understanding of the groundwater pressure within a rock mass is important during evaluation of groundwater conditions in slopes and their effect on slope stability. As noted by Kliche (2009), piezometers are an important tool in measuring the groundwater pressure in slopes. These measurements are made in pre-determined locations that are representative of the area of interest. Piezometers measure groundwater pressure through conversion pressure to voltage (Kliche, 2009). They are inserted in boreholes that are correctly sealed, thus determining only the distribution of water pressure below the phreatic surface of water (Kliche, 2009). The measurement from piezometers is only valid for the immediate surroundings, that is, the point location to which it is installed (Wyllie & Mah, 2004). Wyllie & Mah (2004) highlighted key points that should be taken into consideration for good groundwater pressure measurements as follows:

- Orientation of the piezometer borehole should intersect rock mass discontinuities where water flows
- Fault, shear and fractured zones should be considered when sighting the piezometer
- Geology plays a vital role in the number of piezometers to be installed, for example, a sedimentary rock may require different piezometer completion zones for the successive rock strata differing in hydraulic conductivity.

Piezometers come in different types and characteristics depending on the permeability test that is to be carried out, and the conditions to which groundwater pressure is to be determined (Kliche, 2009). The general classification of piezometers in presented in Table 6:

Table 6: Broad Classes of piezometers (Kliche, 2009)

Broad Classification	Relative Volume Demand*	Readout Equipment	Major Limitations	Major Advantages
Open tube (standpipe)	High	Water level finder	Longer time lag in most rock types. Tube must be straight for whole length Difficulties likely in small-diameter tubes in small-diameter tubes if water levels are significantly below 30 m or if dip is less than 45°.	Cheap; simple to read
Closed tube (hydraulic)	Medium to low Usually	Bourdon gauge or mercury manometer	Requires readout location not significantly above lowest water level; therefore, not suitable for general borehole use.	Relatively cheap
Mechanical diaphragm (pneumatic or hydraulic)	Low to very low	Pressure transducer	Hydraulic types require for periodic "de-airing" of monitoring system.	Excellent longterm stability; can be made very small; simple to install
Electrical	Negligible	Electronic readout	Relatively expensive, especially for great cable lengths. Some zero drift is possible. Certain types may be susceptible to blast damage.	Ideal for remote monitoring; simple installation

The data that is collected from piezometers is regressed and correlated with pressures derived from qualitative analysis of conductivity, permeability and geological properties of the rock mass (Kliche, 2009). Continuous monitoring of groundwater conditions with piezometers enables useful tracking of groundwater pressures and any variations that might arise due to seasonal differences (summer, winter and rainy seasons), effectiveness of drainage systems and also the level of mining activity (Kliche, 2009).

Depending on the information gathered from the piezometers, if the groundwater pressure is high in the slope then there might be need to dewater the slope, that is, removing water in the rock mass behind the rock slope. By draining water from the slope, slope stability is increased due to a reduction in pore pressure, increased shear strength and reduced shear stress within the rock mass. Therefore in terms of monitoring groundwater, piezometers are useful tools.

2.3.6 Slope Stability Radar Movements

For failures that occur quickly, the slope stability radar monitoring system becomes handy and useful. The ability of radar monitoring to cover a large area over a given time gives it the advantage and edge to provide early warning through an alarm system for evacuation purposes (Little, 2005). Little (2005) said that, the slope stability radar used at Sandsloot highwall has the capacity to scan ten thousand square metres in sixty seconds. Cameras are also fixed to the radar dish, and they serve to create a mosaic of the scan area through conversion of the photographs (Little, 2005). Little (2005) elaborated on slope stability radar functions, highlighting the ability of the system to measure millimetre accuracy movements through differential interferometry on the pit walls. This is achieved through comparison of radar mosaic scans from successive phases (Little, 2005). The results are then represented in the graphic user interface in a two-dimension image which uses a colour coded key to note the extent of movement which might have occurred in the slope. Audible warning sirens are linked to the system to warn people and the geotechnical department in case movement has exceeded the tolerable limits that are pre-set in the system. This equipment is critical in risk management in terms of loss prevention, both of human life and equipment, since the timely warning gives people time to respond.

2.3.7 Data Interpretation of Slope Monitoring

Slope monitoring is an important way to determine the rate of slope movement over time, thereby identifying stable and unstable areas. This feeds both into the safety and economics of the mining operation (Wyllie & Mah, 2004) and also assist to determine the mechanism of failure so as to come up with slope stabilising measures and remedial action plans. Slope monitoring is usually interpreted in terms of time movement and velocity plots. This interpretation technique monitors movement of the slope against time (velocity) as well as the rate of change of velocity (acceleration) of the slope. An illustration of a velocity plot and failure is shown in Figure 13.

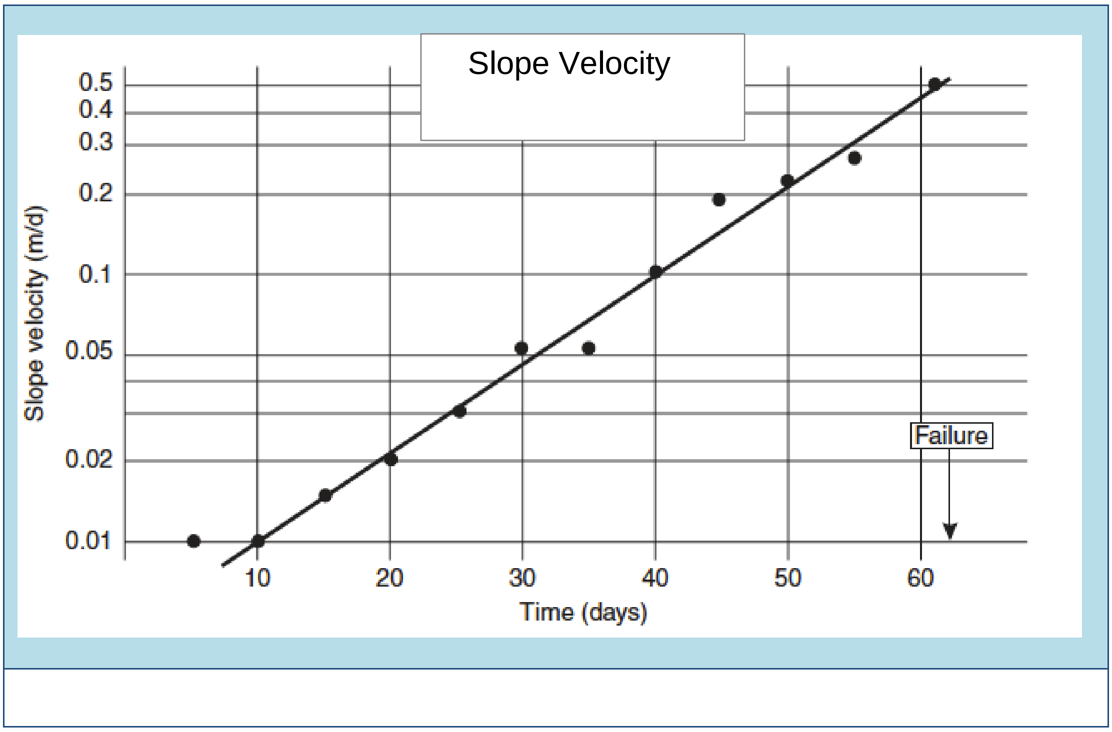


Figure 13: Slope velocity and failure after (Wyllie & Munn, 1979)

Through these plots, times of failure can be extrapolated from the measurements, thereby allowing redesign and planning to be re-adjusted or if need be, evacuation can be effected. These plots also enable the geotechnical team to assess the sensitivity of the slope movement and stability to mining activities (Wyllie & Mah, 2004) and make recommendations on the way forward.

2.4 Mechanisms of Slope Failure

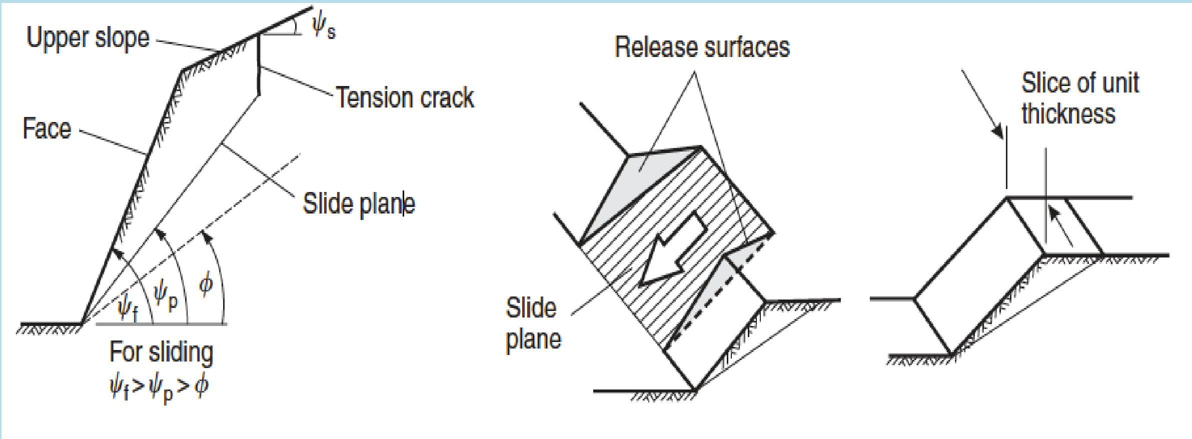
2.4.1 Plane failure

Plane failure is useful when illustrating the effect of shear strength and groundwater conditions on slope stability. Favourable conditions that have to be satisfied for plane failure to occur are provided by Wyllie & Mah (2004):

- The sliding plane of the block or rock must strike parallel or almost parallel (≈20°)
- The dip of the planar discontinuity must be less than the dip of the slope face hence the sliding plane must daylight into the slope face
- The dip of the sliding plane must be greater than the angle of friction of the plane

- The upper end of the sliding surface must cut the upper slope or should terminate in the tension crack
- Weak bedding plane from which parting can easily take place
- The lateral boundaries of the sliding surface must provide low enough resistance to the sliding block

The conditions necessary for planar failure are illustrated in Figure 14.



ψ_f: Slope face angle

ψ_p: Dip of the sliding plane

ϕ: Friction angle

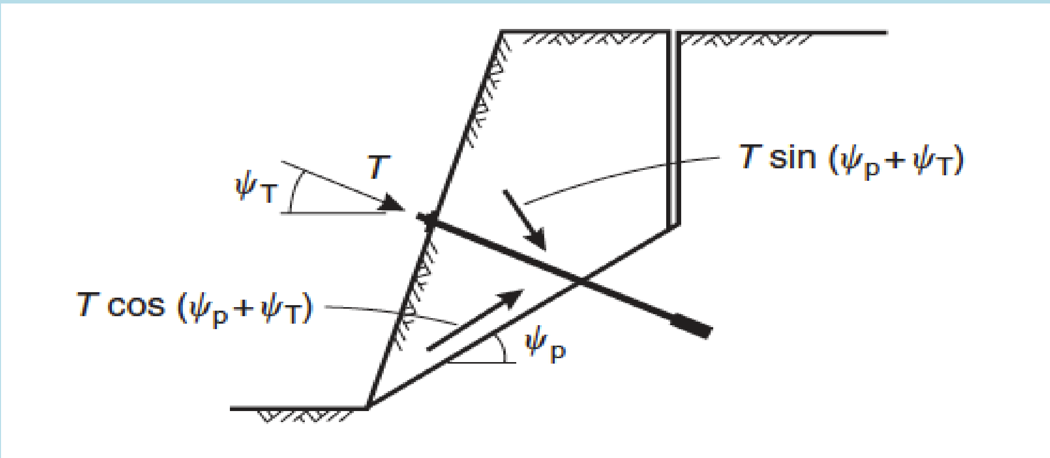
Figure 14: Slope geometry for plane failure (Wyllie & Mah, 2004)

Wyllie & Mah (2004) noted critical assumptions that must be taken into consideration for sliding surfaces and tension cracks. These assumptions are that the sliding surface and tension crack strike parallel to the slope; are vertically inclined and can allow water to fill the crack. Tension cracks are formed as a result of cumulative shear displacements in the rock mass thereby suggesting possibility of slope instability. Another important assumption in plane failure analysis is the absence of moments that would result in toppling or rotation of the sliding block. Consequently, plane failure occurs exclusively by sliding. This important point is supported by Wyllie & Mah (2004) by noting that the weight exerted by the sliding block, the resultant uplift force as a result of water pressure and the force of the water pressure in the tension crack are all applied at the centre of the sliding block. It is important to take into account the shear strength of the sliding plane, that is, cohesion and friction angle, thereby considering the rock mass a Mohr- Coulomb material (Wyllie & Mah, 2004). The shear strength equation is given by:

$\tau = c + \sigma \tan \phi$: ………………………………………………………. Equation 2,

where: τ is shear strength, c is cohesion, σ is normal stress and φ is friction angle.

In slope stability, Factor of Safety is an important rock engineering parameter and in terms of plane failure it is given as the ratio of resistive forces of sliding block movement to the driving forces acting on the block (Wyllie & Mah, 2004). When the Factor of Safety does not satisfy the design safety requirements of the slope, considerations in terms of changing the slope angle, reinforcement of the slope with mechanical end anchors or fully grouted tendons and construction of toe buttress should be made (Wyllie & Mah, 2004). Reinforcement of a slope that can undergo plane failure is illustrated in Figure 15.



ψ_T: Inclination of anchor below the horizontal
ψ_p: Dip of the sliding plane
ϕ: Friction angle
T: Tension in the anchor

Figure 15: Mechanical end anchor for slope reinforcement (Wyllie & Mah, 2004)

The mechanism by which the anchor improves the Factor of Safety is through increase in the normal force (Equation 2), thus increasing the shear resistance to sliding (Wyllie & Mah, 2004).

$T \sin(\psi_p + \psi_T)$…………………………………………………………....Equation 3,

where ψ_p is dip of the sliding plane, ϕ is friction angle and T is tension in the anchor.

The anchor also provides an upward shearing force that lessens the downward sliding (driving) forces. A buttress also uses the mechanism of providing resistance to shear movement by offering confinement to the sliding block as illustrated in Figure 16.

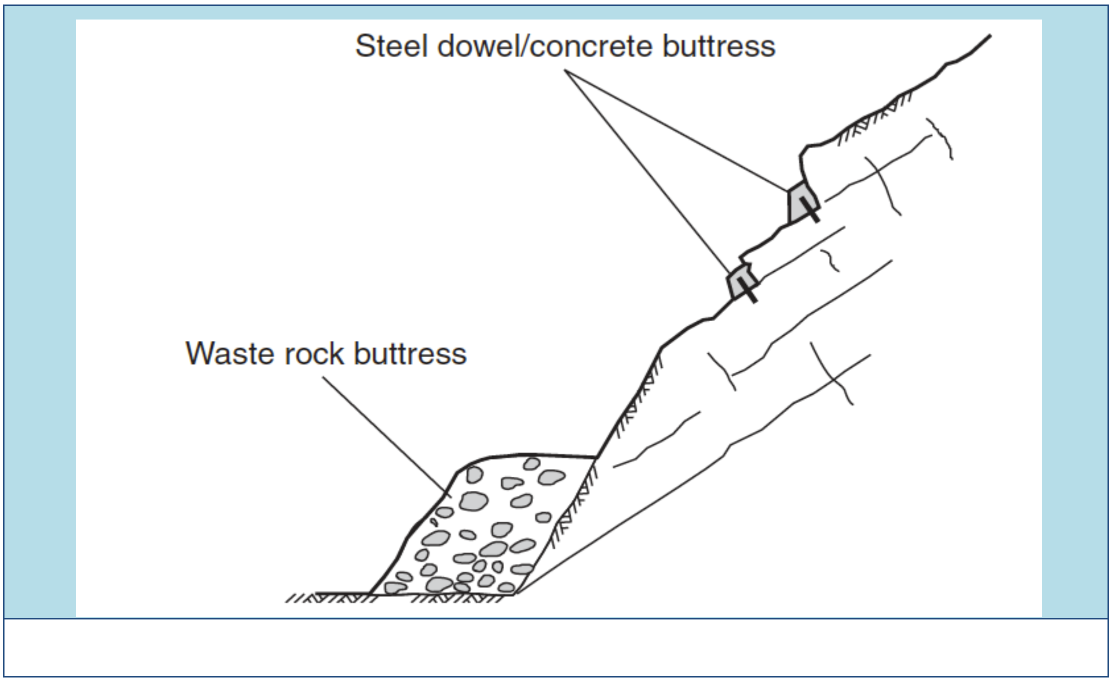


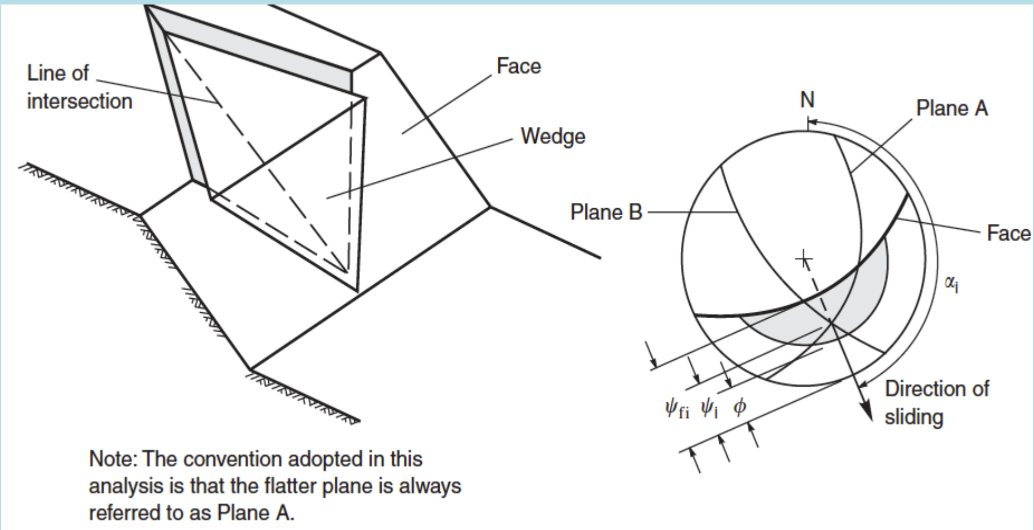
Figure 16: Buttress reinforcement of a slope (Wyllie & Mah, 2004)

2.4.2 Wedge Failure

Wedge failure occurs when joints or discontinuities are striking in an oblique orientation to the face of the slope such that the resultant wedge that is formed slides along the line of intersection (Hoek & Bray, 1981). The conditions that were noted by Wyllie & Mah (2004) for wedge failure are illustrated in Figure 17 and are stated as follows:

- Two planes obliquely cut across the slope face and intersect in a line

- The plunge of the intersecting line must be less than the dip of the face and at the same time more than the average friction angle of the two sliding planes
- The intersection line must dip in the direction of the slope face for sliding of the wedge to occur



$\psi fi > \psi i > \varphi$: ψf -dip of the face

ψi -plunge of the line of intersection

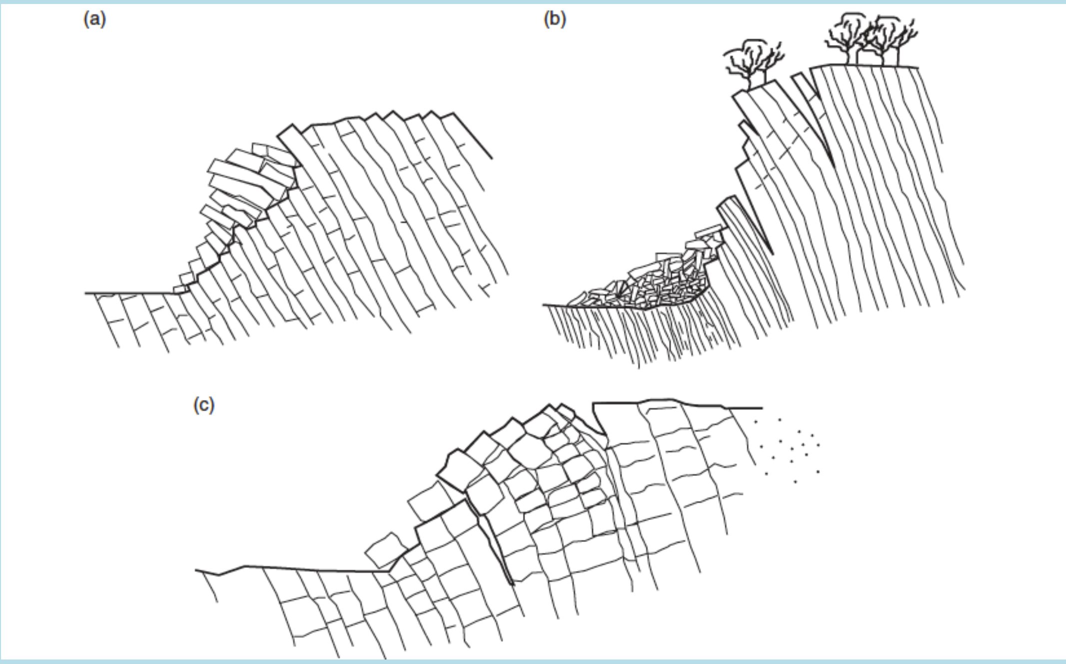
φ -average friction angle

Figure 17: Geometry and stereoplot of wedge failure (Wyllie & Mah, 2004)

The assessment and process of identifying potentially unstable wedges is done through kinematic analysis. Figure 17 also shows a stereoplot that shows the orientation of the intersection line, that is, the direction of sliding of the wedge. The line of intersection on the stereoplot is drawn from the centre and passes through where plane A and plane B meet. For sliding to occur, the point of intersection should be within the shaded region, that is, the friction circle. The Factor of Safety of wedges can be derived from design charts, which consider friction as the only input of shear strength. If cohesion, friction angle and the different strengths of the different planes are to be considered in detail, Factor of Safety can be derived from equations.

2.4.3 Toppling Failure

Toppling failure involves the rotation of rock blocks that are fixed at the base or bottom part that excludes sliding as in the case of wedge, plane and circular failure (Wyllie & Mah, 2004). When carrying out slope stability analysis, it is vital to carry out kinematic analysis of the rock mass for identification of the right failure conditions. Toppling failure often results when blocks without confinement have their centre of mass far from the base were they are fixed, hence causing rotation (Wyllie & Mah, 2004). In most toppling failures, wide tension cracks are often observed at the top hence it is a useful indicator during site investigations. Depending on the structural geology and the pattern of discontinuities, toppling can manifest in two ways namely block toppling and flexural toppling. Block toppling is formed when steeply dipping joints daylight into the slope face causing top columns to topple, such as in bedded sandstone (Wyllie & Mah, 2004). Flexural toppling occurs when discontinuities are dipping steeply and forward such as bedded shale. The different types of toppling failure are illustrated in Figure 18.



a) block toppling b) flexural toppling c) block flexure toppling

Figure 18: Toppling failures (Wyllie & Mah, 2004)

Critical factors that are considered in the kinematic analysis of toppling failure is the dip of the blocks, that is, the discontinuities that form the slabs; the face angle and also the shape of the block. Furthermore, the centre of gravity should be outside the bottom base (Wyllie & Mah, 2004). Other factors for toppling failure include ability for shear displacement on the face contacts and parallelism of the planes striking across the slope. These conditions enable minimum constraints of the toppling blocks from the next and adjacent layers therefore resulting in instability of the slope. Wyllie & Mah (2004) further suggest that in order for toppling to occur the dip angle of the block should be greater than the slope angle such that the chance of sliding is eliminated and only toppling can occur. Factor of Safety for toppling failure is determined from limit equilibrium stability analysis.

2.4.4 Circular Failure

This is failure that occurs when the rock slope is not competent in nature, that is, when it is highly weathered, fractured and has rock fills such that failure of the slope occurs along a circular surface (Wyllie & Mah, 2004). Circular failure analysis takes into account the strength parameters (cohesion, friction angle and shear strength), groundwater conditions and specific weight of the rock or soil involved. A vital tool that is used in slope stability for circular failure are slope stability charts, which can be used in determining the Factor of Safety. Wyllie & Mah (2004) noted that circular failure happens on a circular surface of least resistance determined from limit equilibrium and the critical surface for which Factor of Safety is the lowest is used in stability analysis. Failure of this nature usually starts close to the toe of the tension crack in the crest of the slope. Circular failure is illustrated in Figure 19 and it shows the radius that is formed by the failure surface (intersection point of the failure arc) as well as the details of the forces that act on the individual slice making up the sliding material.

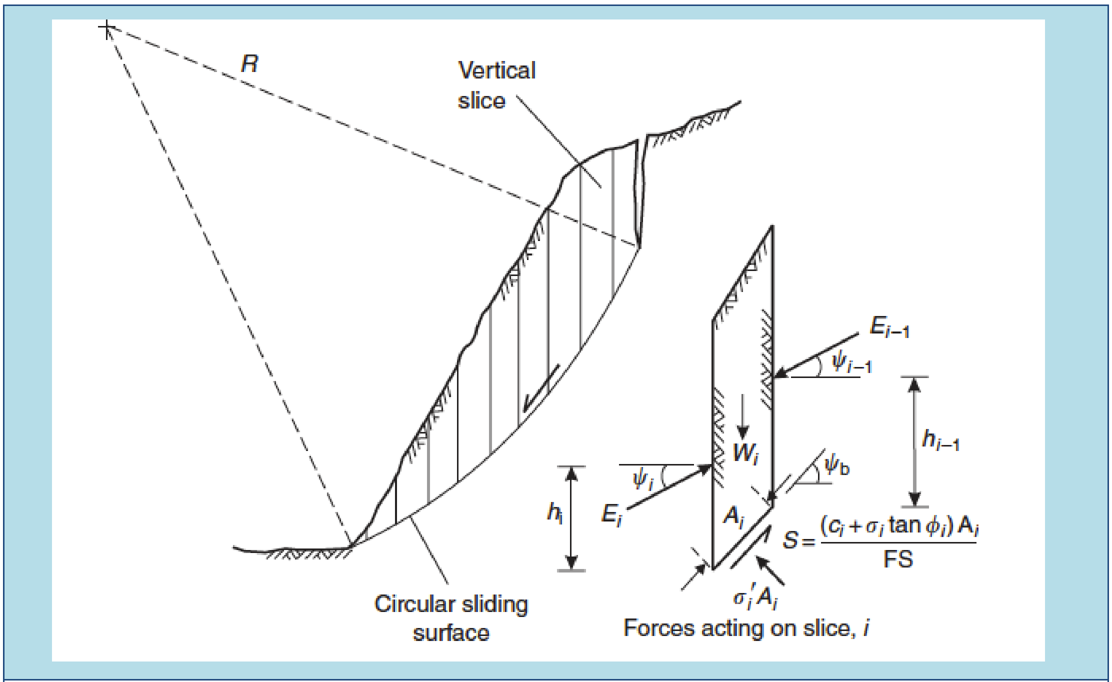


Figure 19: Circular sliding surface with detailed slice (Wyllie & Mah, 2004)

The dividing of the slope into several vertical slices allows determination of shear resistance (S) from cohesion and friction angle as well as calculating the lateral forces (E) exerted on the sides of the slice so as to individually determine equilibrium conditions for each vertical slice. From the limit equilibrium, Wyllie & Mah (2004) defined the Factor of Safety of the circular failure as:

Factor of Safety= $\frac{shear\ strength\ available\ to\ resist\ sliding}{shear\ stress\ required\ for\ equilibrium\ on\ slip\ surface}$.........Equation 4,

FoS=$\frac{(c + \sigma \tan \phi)}{\tau_e}$..Equation 5,

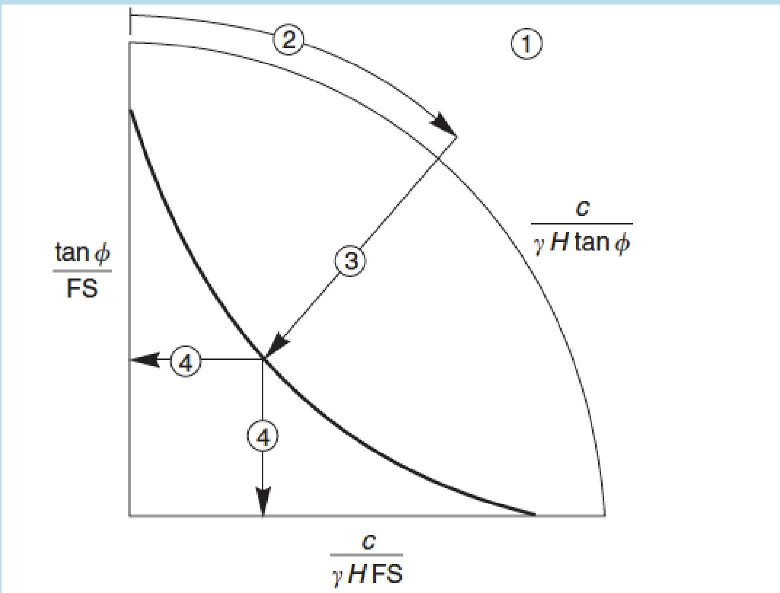
where c -cohesion, σ – normal stress, ϕ – friction angle, τ_e – shear stress

Besides the use of the formula noted above, the Factor of Safety of circular failure can be determined from the use of circular charts. An important point raised by Wyllie & Mah (2004) is that the slope conditions should satisfy the following assumptions:

- The slope material should be homogeneous with the same material properties

- The material should be of a Mohr-Coulomb nature, that is, shear strength should be determined in terms of cohesion, normal stress and friction angle
- A tension crack should be present in the crest of the slope
- For limit equilibrium analysis, the least Factor of Safety should be used for the geometry
- Effect of the different ground water conditions should be taken into account and correctly defined; and the correct chart which corresponds to the existing conditions should be used

When using the circular failure charts to get the Factor of Safety, primary selection of the chart is based on the groundwater conditions, then followed by consideration of rock parameters. A procedural representation of the steps taken to calculate Factor of Safety from circular failure charts is illustrated in Figure 20.



c- cohesion, H- slope height, ϕ - friction angle, ρ- density, FOS- Factor of Safety

Figure 20: Steps to find FOS from circular charts (Wyllie & Mah, 2004)

The following sequence applies:

Calculate value of:

$c/(\gamma\ H \tan \phi)$……………………………………………………..……….Equation 6

The value of $c/(\gamma H \tan \varphi)$ is traced on the outer scale of the chart and projected inwards from the radial until intersection with the curve that coincides to the slope angle (Stacey, 2001). The Factor of Safety is then calculated from the relationship:

$\tan \phi$ / FOS……………………………………………………….………Equation 7

2.5 Chapter Summary

In line with the objectives of the research, the researcher conducted extensive literature review to establish a framework on the research topic, that is 'Influence of pit wall stability on underground planning and design when transitioning from open pit to sublevel caving'. In the same manner as Olavarría, et al. (2006), the author initially identified the challenges encountered when transitioning from open pit to underground including:

- Influence of geotechnical aspects on mine planning and design; and
- Consideration of underground infrastructure

Of these broad challenges, the author sought to unpack the geotechnical aspects and their influence on mine stability. This was also emphasized by DME (1999), by highlighting the following pertinent points for ground control when transitioning from open pit to underground:

- Consideration for geological structures and their influence of wall stability
- Taking note of the shear strength of the rocks and the geological structure
- Analysis of groundwater, drainage patterns and dewatering strategies and their subsequent influence on the pit wall and underground stability over time
- Analysing the stability of the pit wall by considering the geometry of the pit and the sequence of mining
- Monitoring the pit wall and stability conditions as mining takes place

Through literature review, the author addressed the factors that influence open pit stability in detail as this area was pivotal towards understanding the structural integrity, stability and global safety of a mine. The literature review showed that during transition of open pit to underground an assessment of the stress conditions, evaluation of the geological discontinuities, groundwater, lithology and rock alteration were important because of their contribution to slope failure and therefore instability of the mine.

The author also reviewed the importance of rock mass classification towards the objectives of the project. The evaluation of stability and prediction of behaviour requires an understanding of the rock mass and its characterisation so as to enhance design and modelling. Rock mass classification methods such as the Q-System, Geological Strength Index as well as the Rock Mass Rating were discussed. Monitoring systems for identification of unstable areas were discussed, and significance of an active geotechnical monitoring system was shown. Slope failure mechanisms that were reviewed are plane, wedge, toppling, circular and/or a combination depending on the setup of jointing, composition, orientation and shear strength of joints in the rock mass. Knowledge of the likely mechanisms of failure allowed the author to concentrate monitoring efforts during open pit to underground transition.

The literature review enabled the author to bridge the gap between empirical and current practice in terms of transition from open pit to underground. In the context of the 'Case Study Mine', which will be denoted as 'Mine A', the researcher used numerical modelling as a tool to determine a safety factor shell as a guideline for mine design, hence the influence of underground infrastructure and workings on slope stability was simulated with the progression of mining from open pit to underground. The models were also used to site the underground infrastructure.

3 METHODOLOGY

In order to achieve the aims and objectives of the research, various techniques were used in the investigation. Transitioning from open pit to underground is a huge decision in terms of capital commitment and new risks are introduced. Hence a thorough investigation of the mine stability had to be done. The following methods were employed in the research:

- Use of practical knowledge derived from work experience. This entails open pit to underground transition projects previously worked on by the author
- Extensive literature review on pit wall stability and rock slopes on open pit to underground transition. This enabled the author to gain insight into the subject, particularly the factors that have an influence on stability.
- Numerical modelling to illustrate how the zone of influence changes with increasing mining depth. This was done in FLAC3D because of its ability geotechnically analyse rockmass with faults, dykes, groundwater as well as capture displacements and predict unstable zones and Factors of Safety.
- Generating Factor of Safety plots from which to choose where to position the underground infrastructure so that it lies beyond the extent of detrimental mining influence
- Geotechnical survey of the case study area taking note of the geology and geological structures, groundwater, slope angles and topography. An understanding of the project area was a necessary requirement of the project and fulfilment of site familiarisation.
- Data collection of geology, mine plans, mining and geotechnical information from the mine was done to deduce trends and extract relevant information towards the realisation of the project aims and objectives.
- Discussion about mine planning and rock engineering aspects with mine personnel allowed the author first-hand information from an operational level.

The methodology used for execution of this study was satisfactory as it addressed the aim and objectives of the research. Though challenges were met in some instances, the overall requirements of the research input were satisfied.

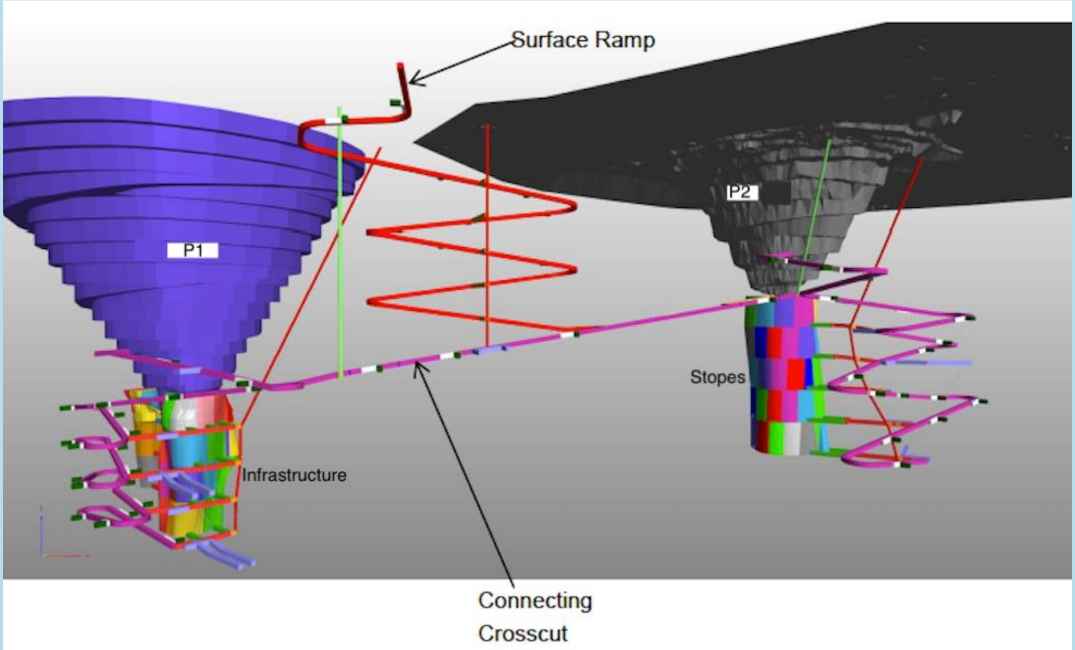
4 CASE STUDIES OF OPEN PIT TO UNDERGROUND

Several mines have transitioned from open pit to underground after reaching the economic pit limit. However, with a viable ore footprint that is feasible for continued extraction below the pit bottom, exploitation of the ore reserve by underground mining is considered. This research will concentrate mainly on Mine A, where the author has first-hand experience. Other studies were also conducted on Palabora and Chuquicamata mine. In particular, these studies concentrated on how pit wall instability influenced mine planning and design for underground mining.

4.1 Practical Case Study: Mine A

4.1.1 Introduction to Mine A

The researcher's interest in the topic was stimulated by a project done in Africa for Mine A. Mine A is a diamond mine, which consists of two kimberlite pipes namely P1 and P2 as shown in Figure 21. In addition to the P1 and P2 kimberlite pipes, there are also several other blow pipes in the vicinity. These pipes were initially mined by open pit until they reached their economic limit at 300m. The kimberlite pipes for Mine A intruded the granitic gneiss host rock. The deposition of the gneiss show strain and shear zones when mapped around the intrusion. In addition to the kimberlite intrusions, there are also north east trending dykes ranging from 0.3m to 1.5m between the two kimberlite pipes as shown on the plan in Figure 22. These dyke zones are sub parallel. The set up for the dual pit system and its simultaneous transition is illustrated in Figure 21. A key finding from the geotechnical investigations and mapping was the presence of a weak brecciated leached granite around the P2 kimberlite pipe. From information gathered at the mine, this zone had caused instability problems during the open pit operations.



Mine A dual pits

Figure 21: Set up of Mine A

4.1.2 Geotechnical Investigation of Mine A

In order to come up with a model with close resemblance to the actual mine conditions, a thorough geotechnical investigation was done. This investigation incorporated geological information and rock mass classification, which will be discussed in the following sections.

4.1.3 Geology

Mine A kimberlites are intruded into the Archean-aged Leonean granitic gneisses of the West African craton (Mine A, 2015). The gneissic fabric is not obvious everywhere, but appears to define areas of higher strain. From sight observations and geotechnical investigations, the kimberlite dyke zones are the most prominent structures in the Mine-A area. The dykes are not continuous, but pinch and swell, bifurcate and form eastward stepping echelon arrays. They vary from thin stringers (<30cm), separated by the country rock, to 1.5m wide. Four dyke zones were identified and named, from south to north, dyke zones A, B, C and D as shown in Figure 22. The dyke zones are sub-parallel, typically trending at 072° and separated by 100m to 200m.

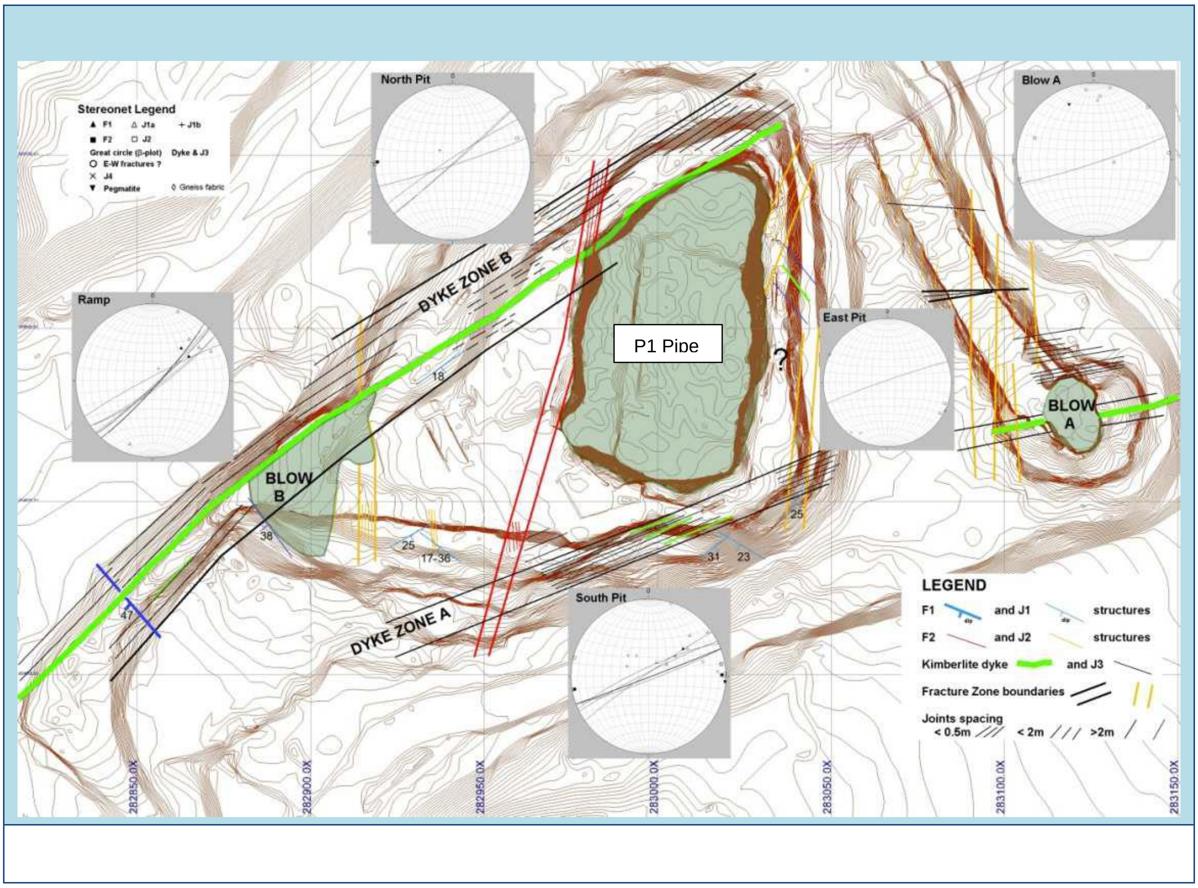
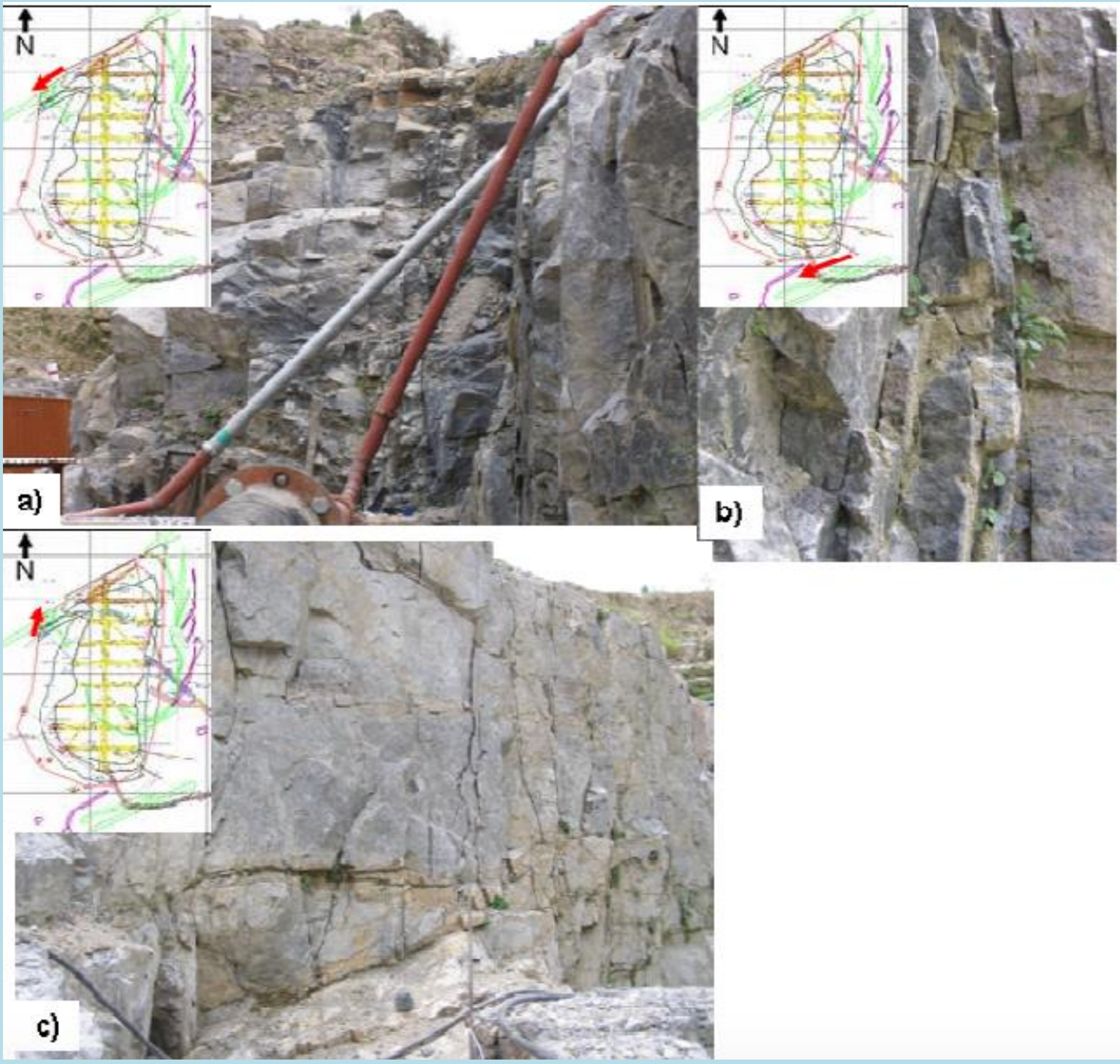


Figure 22: P1 pipe and the dyke zones (Mine A, 2015)

P1 pipe is located on dyke zone B near surface, but at depth appears to span out between dyke zone A and B. The P2 pipe is located on dyke zone A. The P1 pipe along with dyke zones A and B are shown in Figure 22. Description of P1 pipe suggest that it formed as a synchronous blow between dyke zone A and B, with kimberlite textures grading from the coherent magmatic textures, through transitional textures into the fragmental pipe textures (Geologist-A, 2015). The latest 3D interpretation of P1 suggests a complex pipe geometry, that begins to form a crescent shape in plan view as it expands southwards with increasing depth. The pipe footprint below the 300m elevation is approximately 80m x 40m, with the long axis aligned northeast – southwest. Kimberlite dykes are common in the surrounding country rock, particularly in the area east of the pipe (up to 30m long and 1.5m wide), as shown in Figure 23.



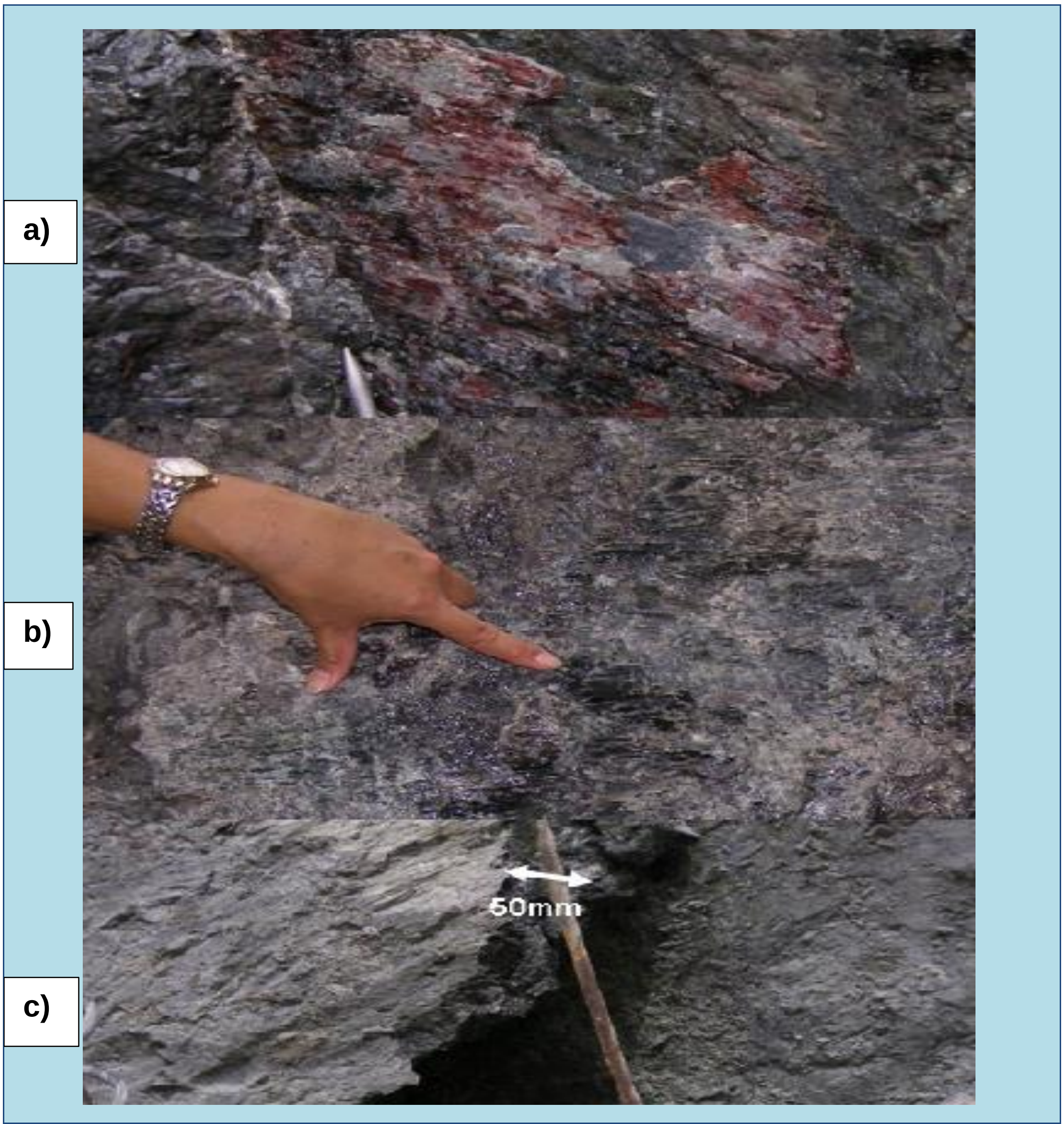
a) Dyke zone B stringer within granite;

b) Dyke zone A stringers within granite;

c) Immediately North of Dyke zone B

Figure 23: Sub vertical jointing parallel to dyke zones (Mine A, 2015)

The granite sidewalls of the pipe are fractured and sheared, with groundwater flowing along the shear planes. The P1 pipe and the dyke contacts are serpentinised joints, which have a low shear strength (Mine A, 2015). The dyke contacts create an area of low shear strength. A photograph of the dykes is shown in Figure 24.



a) Dyke zone B northern contact

b) Contact of a younger dyke intrude

c) Dyke zone B Southern contact intersecting with dykes in East wall with infill 50mm thick

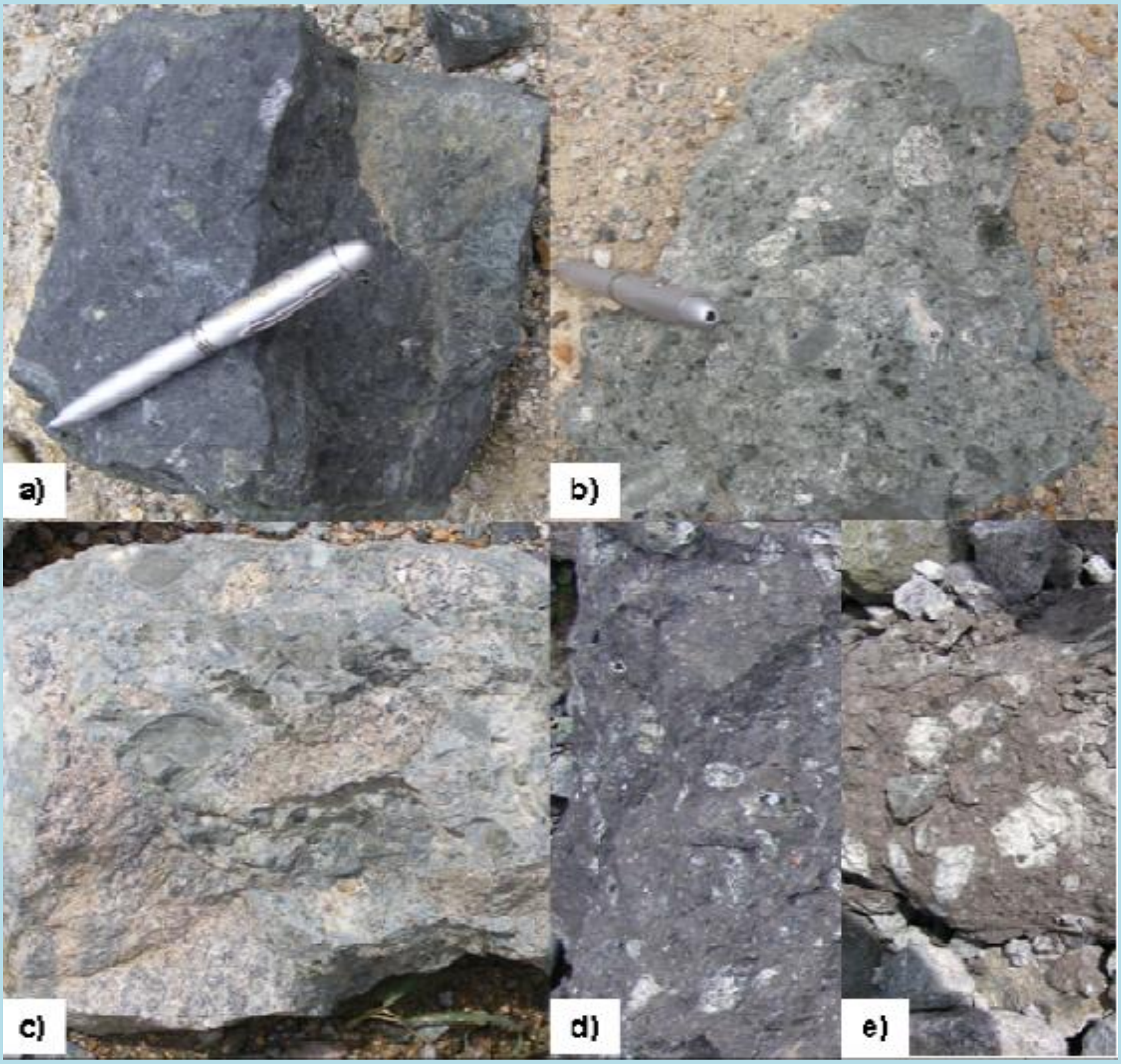
Figure 24: Dyke contacts of Mine A

Sub vertical joints are common in the country rock, mainly associated with dykes and pipe contacts. Low angle joints that have been observed in the pit walls are persistent and can often daylight into the excavation. These discontinuities are infrequent, but have been observed dipping to the south, east and west.

The P2 pipe is a more regular carrot shape (circular in plan view), and dominated by volcaniclastic kimberlite breccias (Geologist-A, 2015). Large, rounded granite boulders are found in one phase of volcaniclastic breccia and have been observed forming in fracture zones parallel to the dykes, including in fractures that do not necessarily contain kimberlite (Mine A, 2015). The sidewall granite is observed in places to be strongly altered, including the removal of quartz to form a remnant vuggy textured rock. It has very low strength (20 MPa) and is likely to fracture during mining and will require additional support (Mine A, 2015). The granitic rocks adjacent to the dyke zones are intensely jointed parallel to the dyke trend. The width of the jointed and sheared zone within granitic rocks is at least 5.0 mm, but sometimes is significantly wider.

4.1.4 Kimberlite Facies

Four kimberlite facies are identified in the project area namely Macrocrystic kimberlite (MK), Tuffisitic kimberlite (TK), Tuffisitic kimberlite Breccia (TKB) and Transitional kimberlite (T) (Geologist-A, 2015). The bulk of the K1 and K2 pipes is composed of volcaniclastic kimberlite (TK and TKB), which are relatively competent and more resistant to weathering than TKB exposed in the Kimberley region diamond mines (Geologist-A, 2015). The MK facies occur in Dykes Zones A and B, and in younger kimberlite dykes intruded into and around the pipes and these are of cross cutting nature (Geologist-A, 2015). In terms of facies, the translational facies weathers rapidly upon exposure and as such whenever they are encountered, additional support is required for the underground excavations. Furthermore, excavations with water are required to be sprayed with a sealant to prevent exposure to weathering agents. The kimberlite facies are illustrated in Figure 25.



a) Macrocrystic kimberlite (MK);

b) Tuffisitic Kimberlite (TK);

c) Tuffisitic Kimberlite Breccia (TKB);

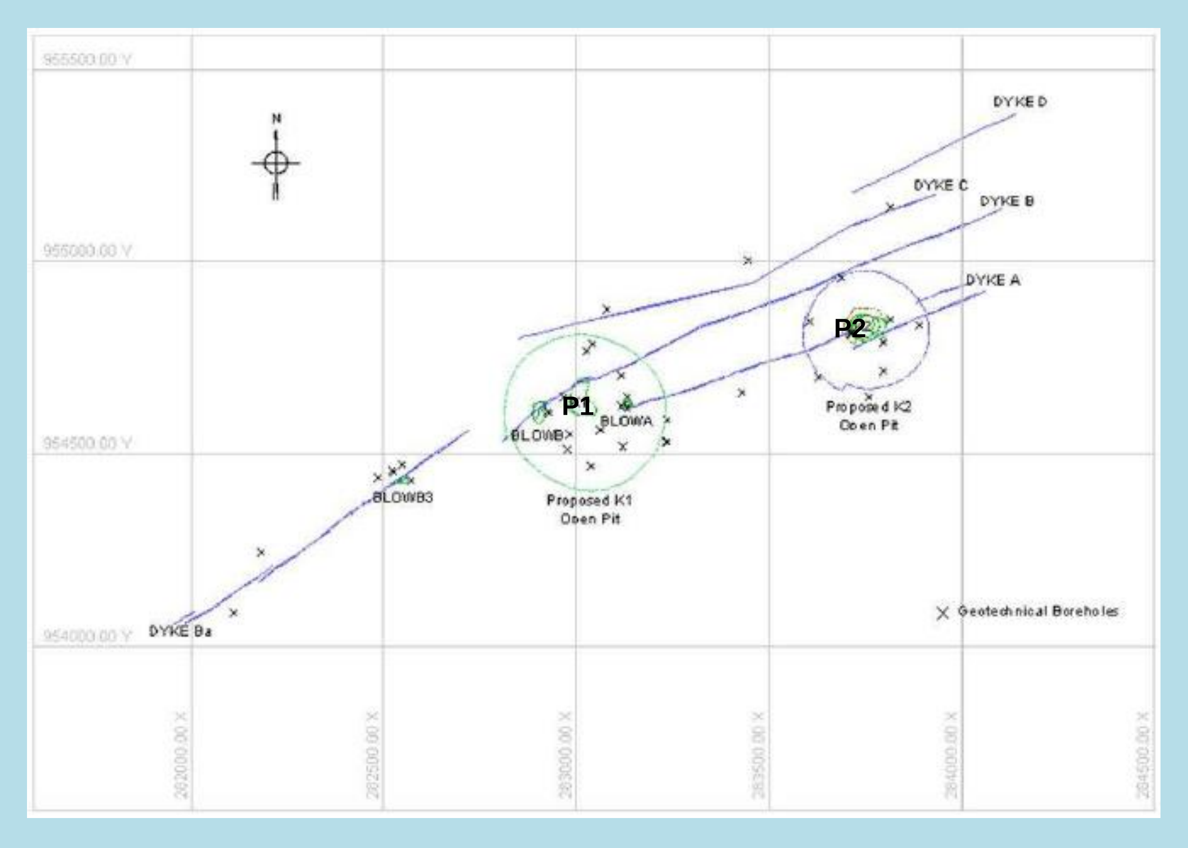
d) Fresh Transitional Kimberlite (T);

e) Weathered Transitional Kimberlite (WT)

Figure 25: Kimberlite Facies for Mine A

4.1.5 Geotechnical Data

Geotechnical data for the project was obtained from a comprehensive database which contained geotechnical logging of borehole cores collected during exploration, rock mass characterisation, laboratory testing and probing of the mine area. For the purposes of this research, that is planning and design of the underground mine in relation to the geotechnical design, the author was authorised to access specific pertinent geotechnical data. The boreholes from which geotechnical design data was extracted are shown in Figure 26.



Borehole plots

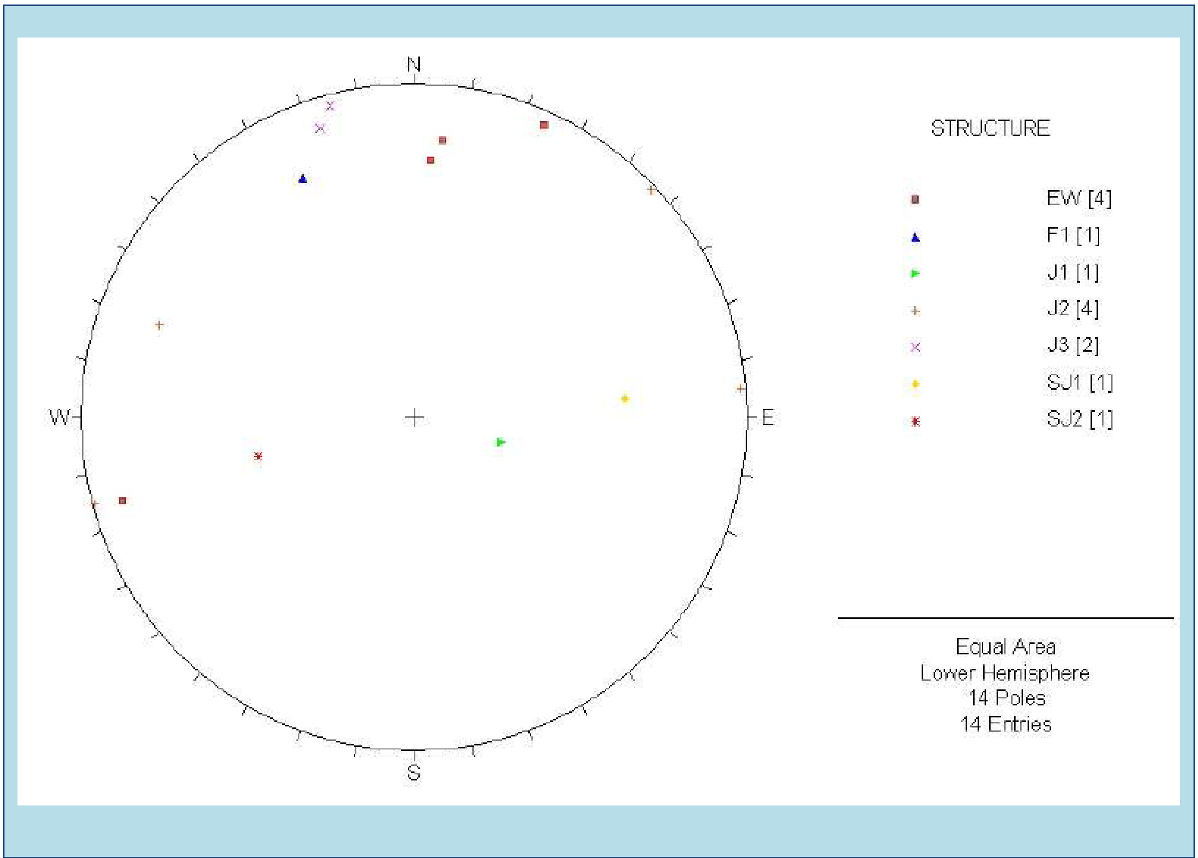
Figure 26: Position of geotechnical boreholes on Mine A

4.1.6 Rock Mass Characterisation

The geotechnical characterisation of the lithological units in the Mine A area was carried out utilising Laubscher's 1990 Rock Mass Rating (RMR) Classification System. This is based on the calculation of the RMR and MRMR indices as described by Laubscher (1990). The rock mass strength characterisation for the purpose of stability analysis and mine design in this research is based on the RMR values. The rock types identified in the area were grouped into lithological units that are granite, dyke kimbelites, pipe kimberlites, breccia and leached granite.

4.1.7 Joint Orientation

Buocz, et al. (2016), showed that the joint orientation is an important parameter as it defines the direction where shearing and sliding is more likely to happen. Consequently, an analysis of joint orientation was done for the Mine A. Six joint sets and a fault have been identified within the current Mine A pits. Of these, four constitute the predominant joint sets developed in the pit, the remaining two sets being more randomly developed. A stereographic projection of the joint sets orientation is shown in Figure 27.



Borehole plots

Figure 27: Joint orientations on plot for Mine A (Mine A, 2015)

The unweathered rock below 40m from surface is very competent and consequently, instability will be kinematically controlled. Potential modes of failure include rock mass failure, toppling, complex wedge and surface ravelling. The joint set characteristics are summarised in Table 7.

Table 7: Summary of joint set characteristics for Mine A

Set	Dip	Strike	Spacing	Length	Macro planarity	Micro Roughness
			P1			
J1	Shallow	a) SW b)E	2 m - 10 m	>5 m	Wavy	Rough undulating
F2/J2	Sub vertical	N-NNE	0.5m	>5 m	Straight	Rough undulating
J3	Sub vertical	ENE	<0.5 m - 2 m	>5 m	Straight stepped at intersections	Smooth undulating, sometimes slickensided
J4	Sub vertical	NW				
			P2			
J1	Shallow	a) SW b)SE	>10 m	>5 m	Straight	Rough undulating
J3	Sub vertical	ENE	<0.5 m - 2 m	>5 m	Straight stepped at intersections	Smooth undulating, sometimes slickensided
			Dykes			
SJ1	Moderate Sub vertical	60°-80° anti-clockwise from J3			straight, slightly curved	Smooth planar
SJ2	Shallow moderate	N			Wavy	Smooth planar
J3	Sub vertical	ENE	<0.5 m - 2 m	>5 m	Straight stepped at intersections	Smooth undulating, sometimes slickensided

4.1.8 Laboratory Testing

Geomechanical laboratory tests were done to provide input parameters for the numerical modelling and engineering. Specimens were selected from boreholes drilled in the project area. A four-phase laboratory testing programme was done and it included:

- Uniaxial Compressive Strength (UCS) tests (including Young's Modulus and Poisson Ratio determination).
- Triaxial Compression Strength (TCS) tests.
- Base Friction Angle (BFA) tests.
- Brazilian Tensile Strength (BTS) tests

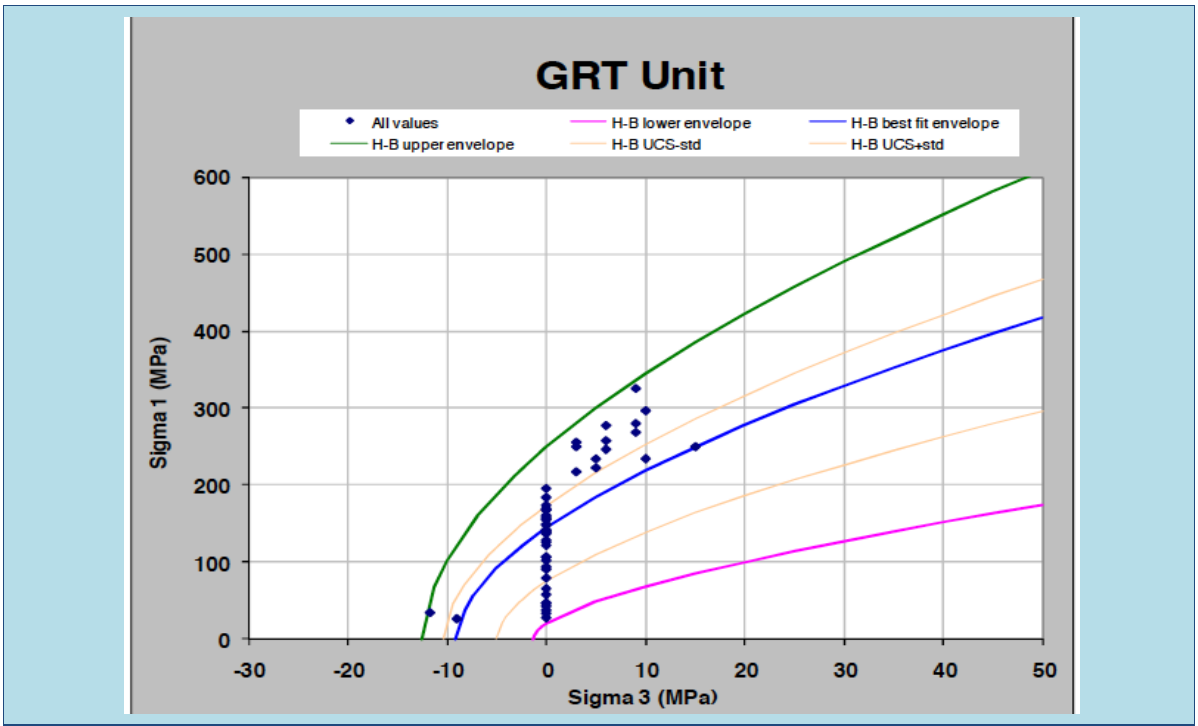
A summary of the laboratory results are shown in Table 8.

Table 8: Laboratory test results for rock specimens for Mine A

Rock Unit	Breccia	Granite	Kimberlite Dyke	Leached Granite	Translational Dykes
Density (kg/m^3)	2570	2680	2920	2260	2650
UCS (MPa)	64	124	120	24	55
Young Modulus (GPa)	55	65	82	15	32
Base Friction Angle (°)	28	36	30	35	20

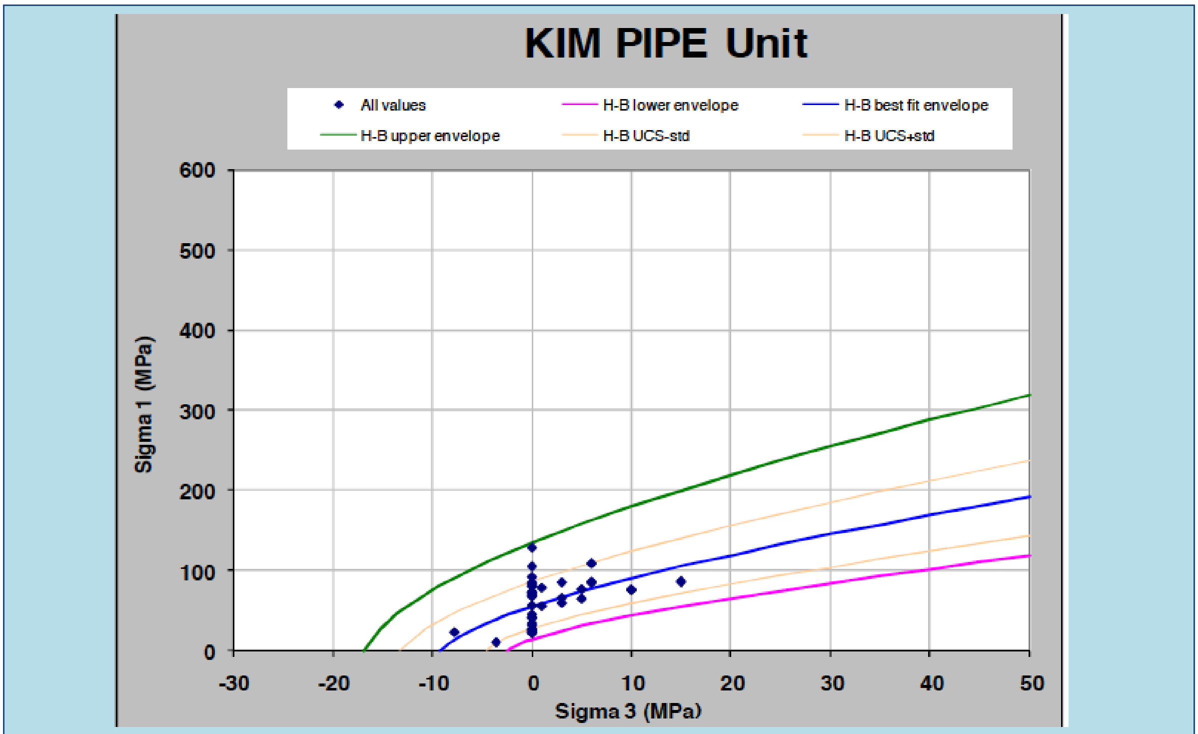
From the laboratory test conducted, the following points are noted:

- Density is determined from the samples from which UCS and triaxial tests are carried on, that is, intact rock.
- The average density values for the six rock types varies between 2.200kg/m^3 and 2.900 kg/m^3.
- A total of 95 UCS tests were carried out on samples distributed among the six rock types considered in this study. The average UCS values for the different intact rock types are between 24 MPa and 124 MPa. In general, the results indicate moderately strong (leached granite) to strong rocks (above 50 MPa).
- Young's modulus was carried out on the samples tested in uniaxial compression for the six units. The Average modulus values varied between 15 GPa and 82 GPa for the rock types.
- A total of 51 triaxial tests comprising of 14 granite (GRT), 14 kimberlite pipe (KIM PIPE), 12 leached granite (LGRT) and 11 translational dyke (TRANS) were carried out with confining pressures of 1, 3, 5, 6, 9, 10 and 15 MPa.
- The results of these tests are shown in Figure 28-31 as plots of minor ($\sigma 3$) versus major ($\sigma 1$) principal stress. These graphs also include the results of the UCS tests and were used for the estimation of the mi parameter for the Hoek-Brown strength criterion, required for the assessment of the strength characteristics of the rock mass and an input in numerical modelling.
- The Base Friction Angle (BFA) was measured from sawn rock samples (saw-tooth specimens) tested in the shear machine.



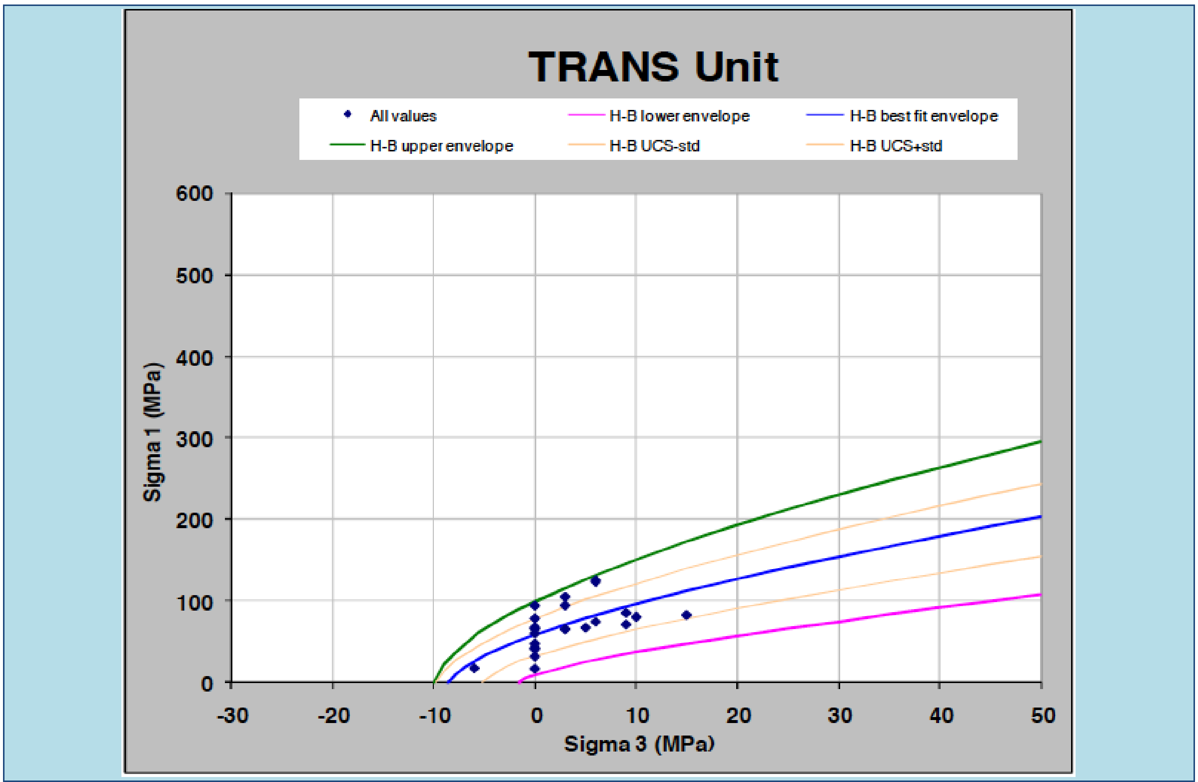
Granite laboratory test results

Figure 28: Granite Plots of σ3 versus σ1



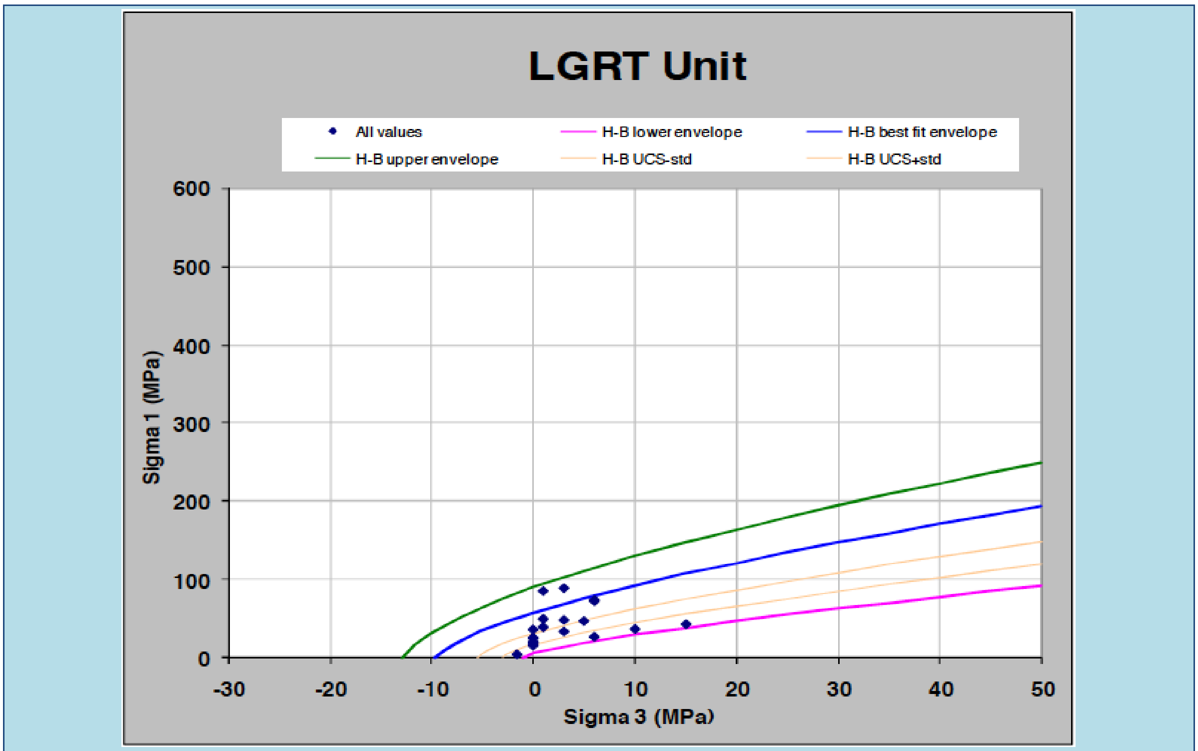
Kimberlite pipe laboratory test results

Figure 29: Kimberlite plot of σ3 versus σ1



Translational dyke laboratory test results

Figure 30: Translational Dyke plot of σ3 versus σ1



Leached Granite laboratory test results

Figure 31: Leached Granite plot of σ3 versus σ1

4.1.9 Pit Stability and Geotechnical Transition Challenges

For Mine A, the main geotechnical risks that faced the project were pit slope stability, ramp stability for the access of underground workings and also underground instability. Slope failure carried the risk of losing pit accesses. Hence numerical modelling was used to determine position of underground and surface infrastructure outside the zone of mining influence.

Observations from site investigations of the project area (Mine A) showed that the majority of the joints encountered in the pits were sub-vertical, resulting in a tendency towards localised toppling and ravelling failures. However, the J1 joint set had flatter dip angles in certain areas, resulting in the formation of complex wedges. The persistence of these flatter dipping joints is of crucial importance to pit slope stability and pit – underground interaction.

Both pits were at some point flooded with water from the pit bottom to about 20m. Water has a negative impact on the stability of pit slopes since it reduces the shear strength of joints and particles. However, the pits were dewatered and a pumping system was designed for the underground project to handle water flow as part of slope management.

Geomechanical design parameters for Mine A were derived from an adequate geotechnical material strength database and were considered equally valid for underground design purposes and modelling. A comprehensive summary of the geomechanical properties used in the geotechnical design parameters is tabulated below:

Table 9: Mine A geotechnical design parameters

Rock type	**UCS**	**RMR**	**GSI**	**mi**	**c (kPa)**	**Φ (°)**	**E (GPa)**
Breccia	72	45	40	6	262	34	1.9
Granite	133	63	57	16	1004	55	9.5
Kimberlite dyke	120	61	56	6	977	44	4.9
Kimberlite pipe	65	61	56	6	694	40	3
Leached granite	25	48	43	6	184	27	0.5

The main slope stability risk for the open pit operations for the case study of Mine A was small scale wedge and planar failure at a bench scale; with stack and overall slope stability influenced by individual continuous geological structures that could result in stack scale wedge formation.

The presence of an elevated water table had a significant negative impact on the overall slope stability and a well maintained dewatering programme was required for on-going stability of the slopes. Empirical slope angle analysis for Mine A showed that stack angles exceeding 60° were not suitable due to associated high rockfall risk. This rockfall risk was analysed and considered to be acceptable at the recommended slope angles in the ranges of 55° to 60° depending on the rock competency. A summary of the empirical rockfall analysis for Mine A is shown in Table 10.

Table 10: Relationship between slope angle and rockfall risk

Stack Angle (°)	**Ramp Width (m)**	**Probability of Rockfall Extending Beyond Ramp (%)**
60	20	1%
61	20	2%
62	20	5%
63	20	19%
65	20	18%

When underground caving has holed into the pit bottom, stress changes that are created pose a greater risk of rockfalls than the probability predicted for open pit study alone. This increased risk would thus require continuous monitoring until such a time the pit haul roads are no longer required for access into the mine.

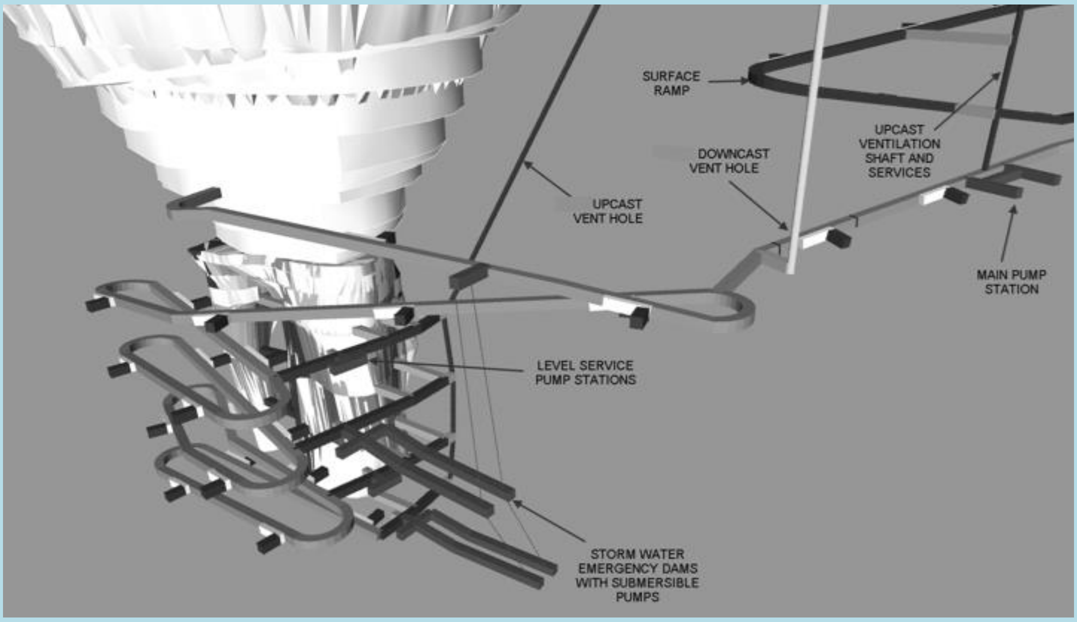
4.1.10 Infrastructure Considerations

For Mine A, siting of major underground infrastructure was done based on the geotechnical information which indicated green zones with minimum failure risks. Underground infrastructure that was considered for Mine A projects include siting and placement of:

- Ramp development
- Connecting drive for the two underground workings

- Ventilation shafts
- Underground workshop
- Drilling water reticulation
- Dewatering system
- Electrical system
- Secondary escape route

A diagram of the infrastructural requirements for the Mine A case study is illustrated in Figure 32.



Mine A underground infrastructure

Figure 32: Underground infrastructure for Mine A

Access Points

The initial breakaway point for establishment of underground access for both pits was to be made in competent ground, slightly above the pit bottom to minimise the risk of flooding. Thus, the portal entry has to be in a stable place with virtually no or minimal displacement.

Connecting Drive for the Two Underground Workings

In order to have a secondary exit during the initial development stages of either pit, it was decided based on accepted engineering principles to have a connecting drive between the two underground workings. This drive had to be shielded as well from the zone of instability.

Declines

The stopes would be accessed through declines with a gradient of 8°. These declines would constitute primary development of the underground workings. Correct siting of the declines outside the zone of instability was critical.

Ventilation System

Another underground infrastructural consideration for the mine was the ventilation shafts. Each pit would need raise bore shafts linking the successive levels to exhaust the foul air during the operation. Location of these ventilation shafts also had to be in stable rock conditions.

Underground Workshop

Servicing of machines such as drill rigs and Load Haul Dumps (LHDs) was considered to require an underground workshop so that time could be saved by not tramming machines out of the mine after shift. Consequently, an underground workshop was deemed to be a necessary infrastructure, in the form of a large excavation ideally between the two pits, that is, along the connecting drive.

Drilling Water Reticulation

Drilling water, ground water and normal rain water was planned to be pumped out of the mine using a service pumping system consisting of collection dams on each sublevel, equipped with mechanical agitators to keep solids in suspension, and suitable dirty water centrifugal pumps. The quantities that were predicted for the project are noted in Table 11.

Table 11: Projected water inflow for Mine A

Water Source	**Megalitres per month**
Drilling water per month based on 0.2 m^3/t broken	21.3
Maximum ground water inflow at P1 per month	60.8
Maximum ground water inflow at P2 per month	73.0

Dewatering System

A floodwater dam and pumping chamber are situated on every second level in the decline of each pipe (p1 and P2). In P2, the pump chambers consist of one 60 m long drive on every second level. In P1, the pumping chamber consists of two 60 m long drives on every second level. Pump chambers in each kimberlite pipe are connected via a 0.32m hole that acts as the pumping column. In the connecting crosscut the pumping columns from the two kimberlite pipes meet in the pumping chamber in the connecting drive before running up the upcast shaft to surface.

Electrical System

A power supply to the new underground workings would be required to run the machines (electric drill rigs) as well as the fans. This would require an underground substation, to feed the mining area, the fans as well as the underground pump stations.

Secondary Escape Route

During the initial development there will be no second egress until the connecting haulage between P1 and P2 is completed. Once production in the first stope commences, second egress via the pit adits will be achievable via the connecting haulage. Once development of the connecting drive is done, a centrally located ramp will be developed in stable ground between the two pits and this access will become the main access into the mine. In order to have a backup in case the pit adits eventually fail, one of the upcast ventilation shaft will be equipped with a ladder or a small headgear and single drum winder which can be used to evacuate people in an emergency.

Level Drives

On the footwall elevation of each stope the level drives break away from the decline. The level drives contain cubbies for mini substations and electrical switchgear. Pump chambers for service water will also be situated in the level drives. The level drives continue closer to the ore and eventually form the rim drives. From the rim drives, the drill drives are developed into the stope and create the drives from which longhole stoping (LHS) will be done.

4.2 Palabora

4.2.1 Introduction of the Mine

Palabora mine is located in the Phalaborwa Complex, in Limpopo province of South Africa. It lies roughly 400km North-West of Johannesburg (PMC, 2012). Rock formations that dominated the Phalaborwa complex are dunites, pyroxenites, apatite and the country rock is predominantly granite gneiss (Groves & Vielreicher, 2001). Within this rock formation are alteration zones pregnant with magmatic brines (Groves & Vielreicher, 2001). Minerals that are extracted from this operation are copper as the main base metal, and it also hosts phosphates, vemiculite, phlogopite, magnetite, nickel, gold, silver, platinum and magnetite (PMC, 2012). Copper ore is hosted in the carbonatite component of the complex (PMC, 2012). The coverage area of the Phalaborwa Complex is approximately 2000 hectares and mining is mainly on the central intrusion of the carbonates (Groves & Vielreicher, 2001). Distribution of the rock formations is illustrated in Figure 33.

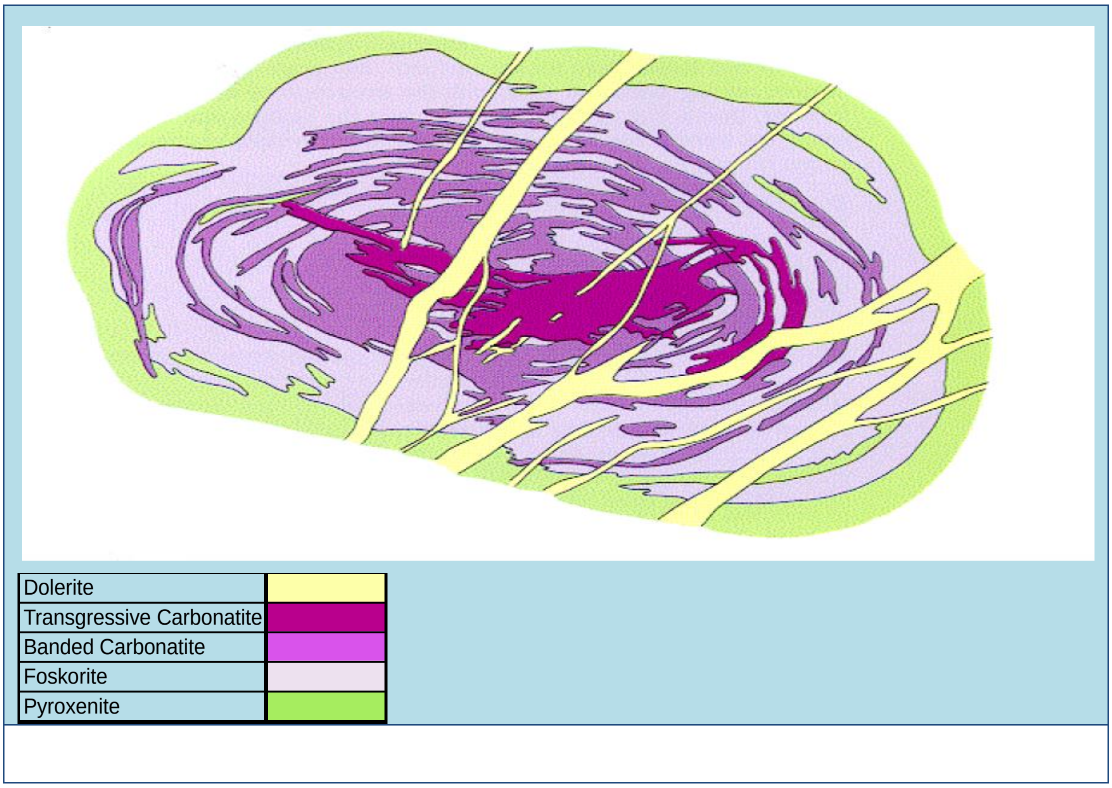


Figure 33: Geology of the Phalaborwa Complex (Cairncross & Roger, 1995)

The peculiar nature of this deposit is that copper ore is found in the carbonatite and with ultramafic rocks containing vermiculite (Groves & Vielreicher, 2001). Intrusions of various nature are also seen in Figure 33 and these include Foskorite, olivine magnetite and the transgressive carbonatite intrusions. In addition, the area is also intruded with mineral rich dykes which contain other by-product minerals such as zeolites, mesolite crystals and phrenite (Groves & Vielreicher, 2001).

As noted by Moss, et al. (2006), the Palabora mine orebody is elliptically shaped with a vast planar area of 1.4km and 0.8km. The depth of the volcanic pipe is 1.8km below the surface. As can be seen from Figure 34, the central part of the orebody is dominated by banded and transgressive carbonates.

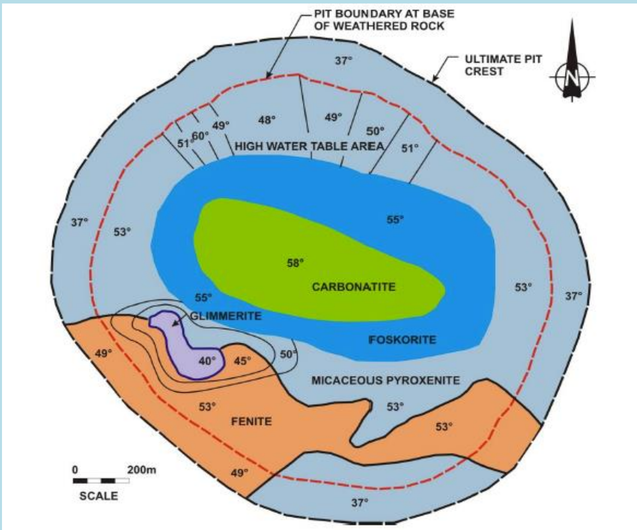


Figure 34: Palabora geology and pit geometry (Moss, et al., 2006)

4.2.2 Pit Stability Challenges

After reaching the approximate pit depth of 800m from surface to the open pit bottom, Palabora Mine took a decision to exploit the deposit using block caving. The Palabora open pit was described by Moss, et al. (2006) as having the following features:

- Depth of the final pit bottom was approximately 800m
- Inter ramp slope angles ranged from 37° to 58° depending on the competency of the rock mass

- The central core of the volcanic pipe was mainly composed of banded and transgressive carbonates
- On the deposit footprint, there were barren dolerite dykes that dip steeply in the North-East direction, as well as faults that trend North-West and North-East.
- The hydraulic conductivity of the Palabora Mine cave material was 0.01m/second
- Underground mining was to be done by block caving having a lift of 400m from the pit bottom to the production level

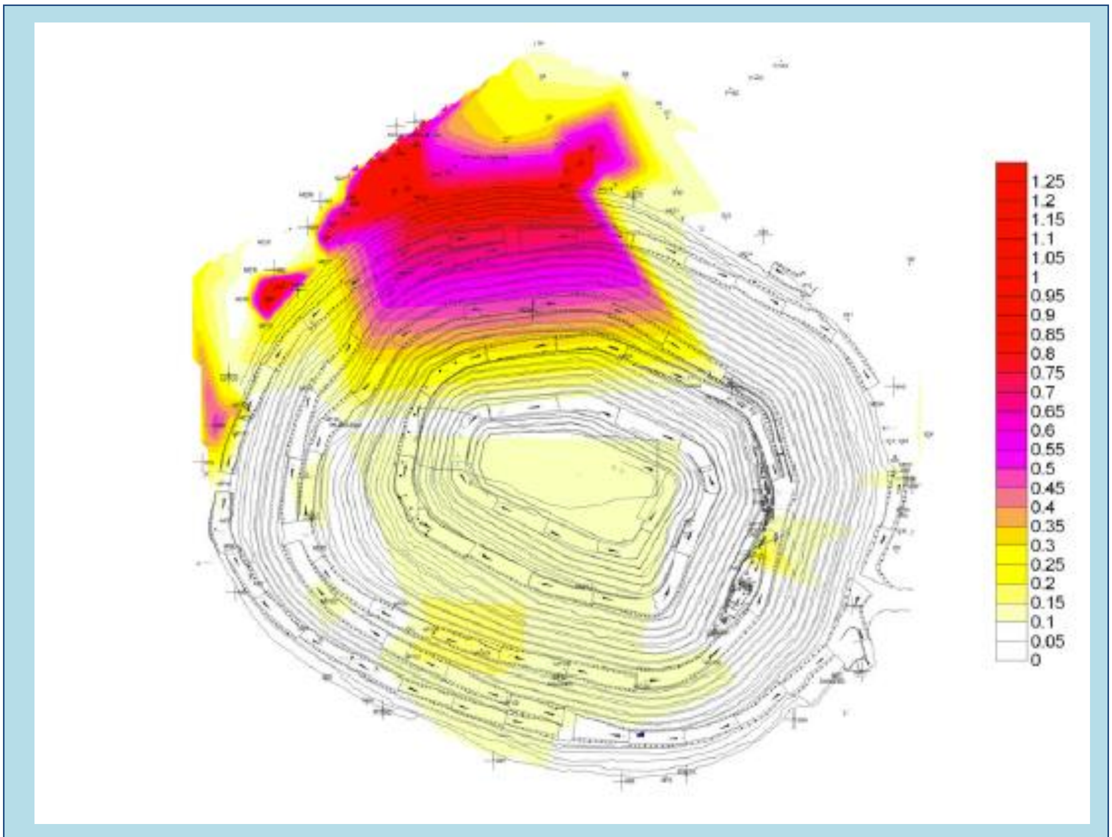
The challenges that were faced during the transition of Palabora from open pit to underground are discussed below.

4.2.3 Open Pit and Cave Interaction

Moss, et al. (2006) acknowledged that the open pit and the cave interacted in mainly two ways. The first mode of interaction was a positive one, in the sense that the pit concentrated stress on the block of ground between the pit bottom and the cave back. This stress concentration contributed positively to cave propagation through crack extension and creation of new fractures. This would result in good primary fragmentation of the ore, thereby reducing secondary crushing costs. The second interaction was negative, that is, the cave adversely affected the pit wall stability. This negative effect was a result of increased displacement of the Palabora pit walls. It occurred as a result of reduced confinement when the cave broke through to the pit bottom. From measurements through monitoring equipment, the biggest movement wall was recorded in the North Wall, with displacement recorded more than 1.5m (Moss, et al., 2006).

Signs of pit wall instability started to show with bench failure in proximity to the water sump in the pit as well as large cracks observed on the northern pit wall. Cumulatively, over a period of one and half years, a significant section of the North wall with approximate dimensions of 800m high, 300m along the wall and 50m thick had failed (Moss, et al., 2006). This zone of pit stability is illustrated in Figure 35. Extensive numerical modelling was then done and results showed that:

- The stress change induced by cave breakthrough in the pit bottom and pervasive jointing contributed to the pit wall failure
- Displaced zones were also associated with seismicity
- Though it was known caving would induce failure in the pit walls, there was limited knowledge of caving induced subsidence



Movement calibrated in mm/day

Figure 35: Palabora pit north wall zone of instability (Moss, et al., 2006)

4.2.4 Effects of Slope Failure

Failure of the northern wall had several implications on the Palabora project. The following effects stemmed from the slope failure:

- Groundwater flow: Due to the failure of the Northwest wall, the geometry, course and flow pattern of water into the underground working was altered as one of the major pit sumps failed.

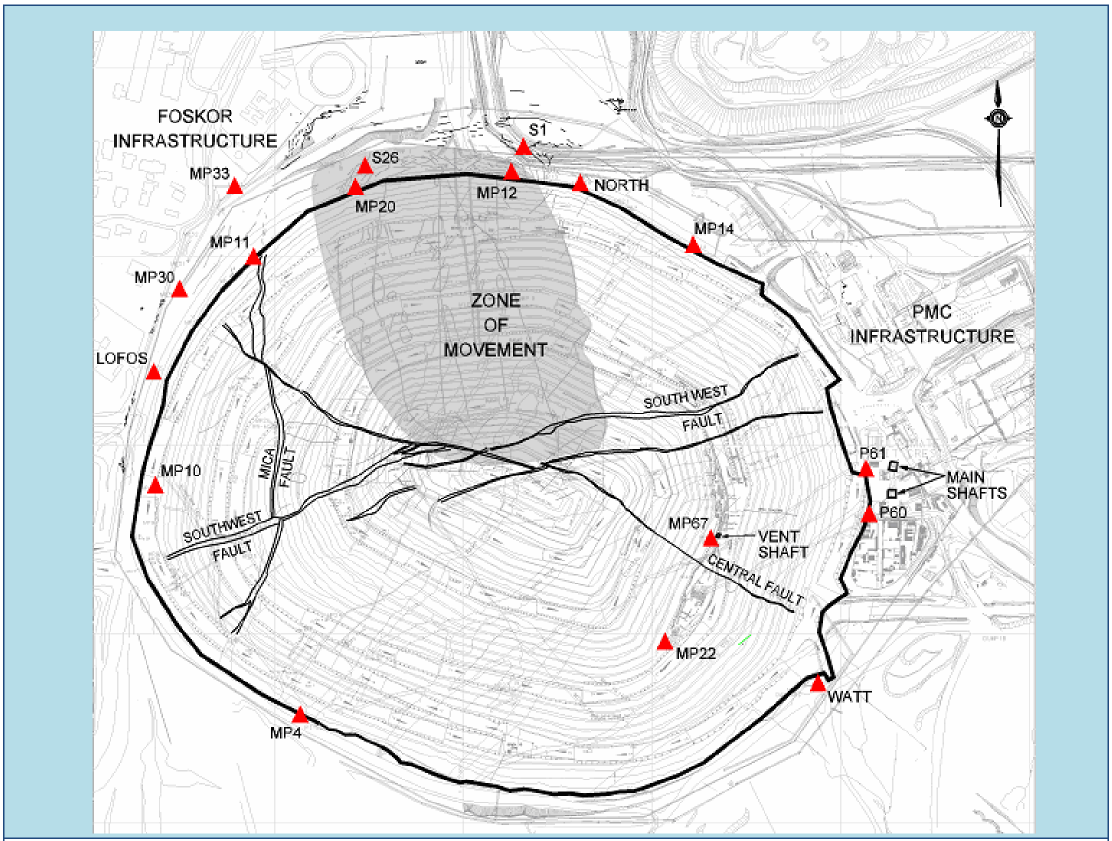
- Loss of access and haul roads: The displacement and movement that occurred in the North wall caused loss of access and haul roads in that pit section.
- Negative impact on underground production: The rock mass that failed was uneconomic in terms of cut-off grade thereby had the potential to dilute the caved ore causing premature dilution. The consequences of dilution ingress was increased cost per tonne.

4.2.5 Infrastructure Considerations

The Palabora north wall pit failure had a negative effect on the mine's infrastructure that was in the zone of influence:

- Access and haul roads in the pit
- Tailings
- Water and power lines
- Railway line

Due to the rate of failure which was relatively slow, the mine relocated various service facilities outside the projected zone of influence. For the other critical infrastructure that was in the vicinity such as the ventilation shaft in the East wall, monitoring equipment was installed to continuously detect any probable instability which could arise. A plan of Palabora pit and the relationship with infrastructure is shown in Figure 36.



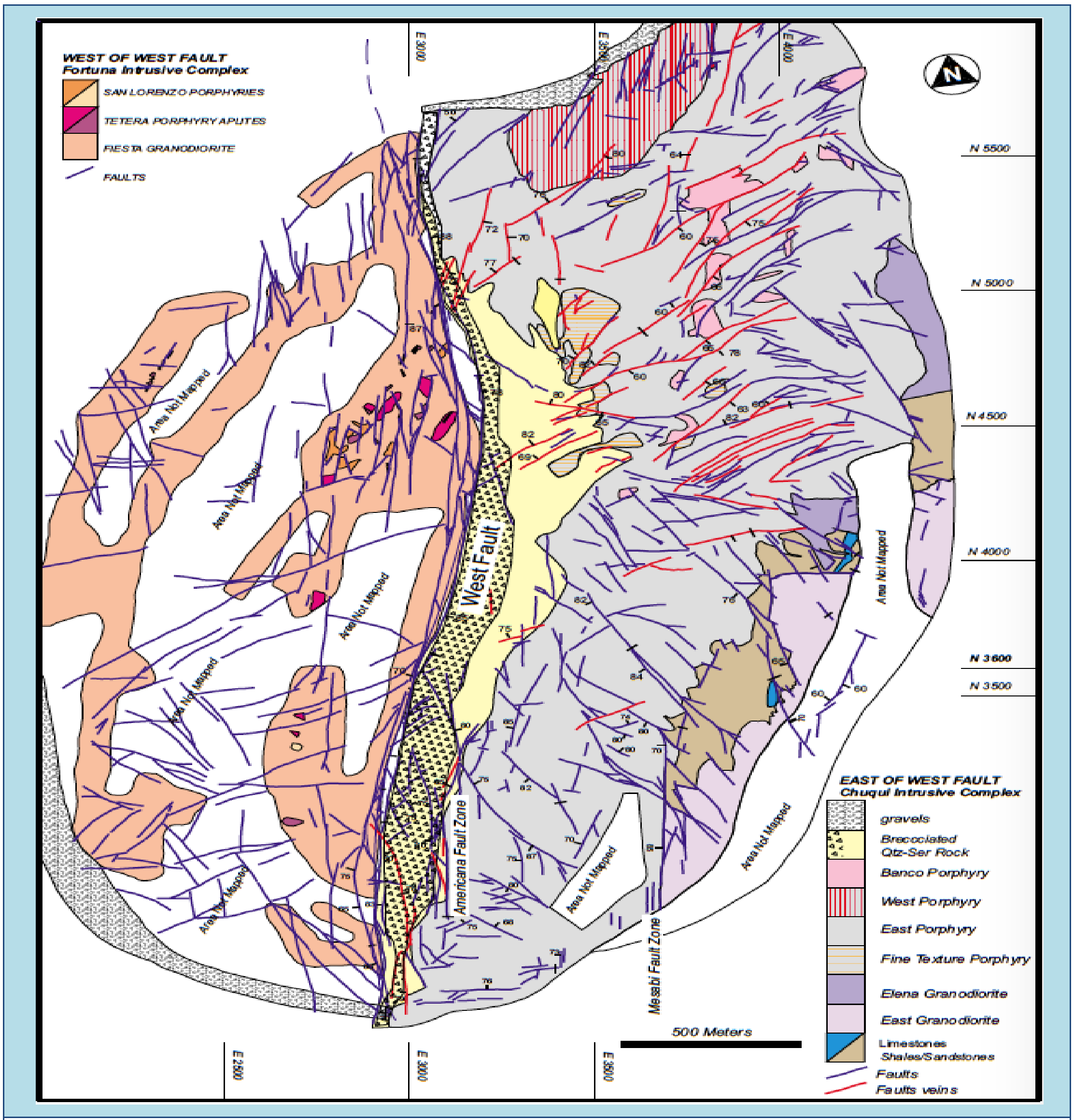
Palabora plan

Figure 36: Infrastructure and monitoring points (Brummer, et al., 2006)

4.3 Chuquicamata

4.3.1 Introduction of the Mine

Chuquicamata mine is located in the Atacama Desert in northern Chile, about 340km northeast of Antofagasta. It is the largest copper deposit in the world. The Chuquicamata mine orebody lies within the Chuqui Porphyry Complex which is made up three distinct porphyries namely the East, Fine texture and Banco porphyries. Some of the major rock types at Chuquicamata are gneiss, granite, metadiorite and porphyry intrusions. The rock types and geological structures found in the Chuquicamata pit plan are shown in Figure 37.



Chuquicamata geology

Figure 37: Geology of Chuquicamata pit (Guillermo, et al., 2001)

Open pit mining at Chuquicamata started in 1915 and in 2005, the pit depth was 850m. In 2014, the final pit depth reached 1100m. After reaching the economic limit, Chuquicamata explored options to transition from open pit to underground using a panel caving mining method. The underground ore footprint dimensions are 2500m by 300m, with three vertical mining cuts of 250m each.

As noted by Olavarría, et al. (2006), the Chuquicamata copper deposit is porphyry in nature, steeply dipping (vertical) and has a rectangular planar shape. The West Fault had a bearing on the mineralisation as evidenced by the ore on the eastern side of the fault and waste on the western side of the fault. The west fault also had contacts of some host rocks, together with shear zone on the western side as well as a regional fault trending North-South. The main geotechnical units of Chuquicamata mine are shown in Table 12.

Table 12: Main geotechnical units at Chuquicamata (modified after Olavarría, et al., 2006)

Geotechnical Unit	UCS (MPa)	FF (fract./m)	RMR	GSI
Quartz-sericitic rock	20	1 to 5	55 to 65	70 to 85
Highly sericitic rock	10	>10	35 to 45	25 to 40
East porphyry with sericitic alteration	31	1 to 5	60 to 70	55 to 70
East porphyry with chloritic alteration	84	1 to 10	55 to 65	55 to 65
East porphyry with potassic alteration	85	1 to 10	55 to 70	55 to 75
UCS- Uniaxial compressive strength of the intact rock				
FF - Fracture frequency (including weak veinlets)				
RMR- Laubscher's rock mass rating				
GSI -Geological strength index				

4.3.2 Chuquicamata Transition Geotechnical Challenges

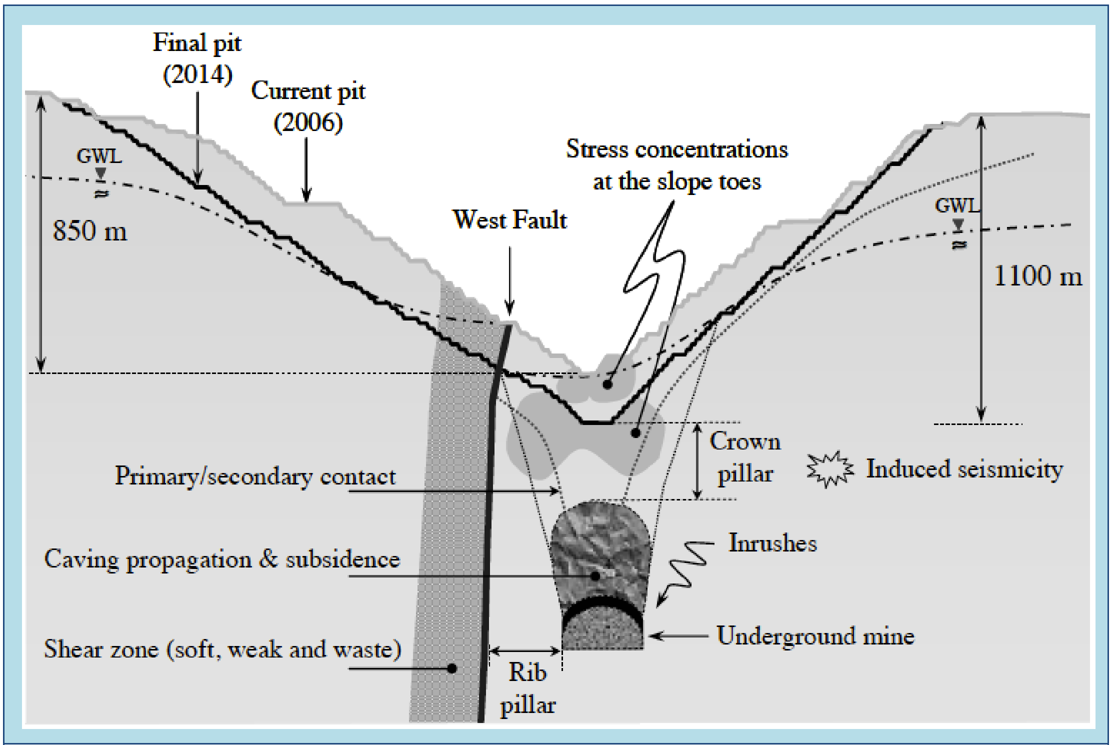
Owing to the size of the operation and geotechnical challenges, Olavarría, et al. (2006) said that it was fundamentally critical that the mine design of the transition of open pit to underground considered the pit and cave interaction. The behaviour of the pit to underground mining transition was also monitored. The geotechnical issues and challenges that were raised for Chuquicamata transition by Olavarría, et al. (2006) are illustrated in Figure 38 and are summarised as follows:

Open Pit Excavation

The presence of the open pit results in stress re-distributions around the excavation, that is, zones of high stress concentrations as well as regions of low confinement. The resultant stresses had a bearing on the stability of the mine, thus were incorporated in planning of the underground design.

Geological Structures

At Chuquicamata, large geological structures such as the west fault and shear zones posed geotechnical challenges in terms of both stability and potential dilution. The west fault was the contact line between ore and waste, with the waste having weaker properties such as being softer and heavily fractured compared to the ore. In order to counter the negative effect of the west fault, a rib pillar was factored in the design between the west fault and undercut area to preserve the stability of the mine. The west fault is shown in Figure 38.



Transition challenges

Figure 38: Chuquicamata transition challenges (Olavarría, et al., 2006)

Induced Seismicity

Due to the pit bottom depth of 1100m and the planned lift of 250m for the first level, significantly high stresses were generated, hence the likelihood of potential seismicity. Consequently, induced seismicity was expected and the mine proactively installed monitoring systems for seismicity in the underground operations.

Subsidence

Once the caving started, and systematically propagated until the cave back advanced and connected to the pit bottom, a considerable degree of confinement was lost and the pit subsided creating a zone of influence that extended beyond the pit perimeter. This was critical and relevant to mine design and planning, as the design must ensure the main accesses (declines and shafts) and underground infrastructure are outside the zone of influence.

Consideration of the existing surface infrastructure such as roads, rails, service lines and water supply lines were also be taken into account.

4.3.3 Infrastructure Considerations

To achieve a safe, functional and successful open pit to underground transition, infrastructural requirements of the mine had to be satisfied. This included correct siting of the main accesses, ventilation shafts, ore transportation and handling systems, water reticulation and pumping systems, underground services, crusher chambers, workshops and magazines. The location, size and sequencing of such infrastructure was critical during transition as they had to serve the mine for the life of mine. These excavations had to be away from zones of weakness or potentially unstable areas.

4.4 Case Studies Summary

The transition of open pit to underground for Mine A, as well as the desktop studies for Palabora and Chuquicamata shared common pertinent points to note. When an open pit has reached its economic limit and the economic evaluation allows it to go underground, a thorough geotechnical investigation with regards to the stability of the pit slopes has to be done. This involves consideration and understanding of geology and geological structures. The influence of regional and local structures such as faults, dykes, planes and joints in the rock mass has to be considered. Detailed site investigation in terms of geotechnical data collection is another key area, which will inform the mine design. During this stage, rock mass classification, strength parameters, groundwater and geomechanical parameters are established. When planning and designing, safety considerations and value preservation is taken into account by determining the pit-underground interaction, pit wall stability and siting of infrastructure in relation to the areas of geotechnical influence. If the transition is not properly managed and unexpected slope failure occurs, the implication could be loss of lives, equipment, water inrush into underground workings, loss of accesses, infrastructure damage, loss of ore and dilution.

5 RESULTS AND ANALYSIS

5.1 Numerical Modelling

In line with the aims and objectives of this research, the researcher investigated the effect of stress changes around the pit wall with progression of mining at Mine A using FLAC3D numerical analysis. From this exercise, mine planning and design was done with confidence in terms of siting of underground infrastructure such as declines, ventilation shafts, underground stations, pump chambers and primary development meant for life of mine. This chapter will include an introduction to numerical analysis, application of numerical modelling on Mine A case study and how numerical modelling was used in the project for Mine A to plan and design the underground mine.

5.2 Introduction to Numerical Analysis

Numerical analysis is a technique that involves the use of algorithms to apply, analyse and solve geotechnical problems. It is a useful tool in the field of rock engineering that helps predict stability or instability. As highlighted by Nikolic, et al. (2016), this computational simulation represents the behaviour of the rock mass by calculating the relevant insitu and induced stresses. FLAC3D is able to include the effects of fracture mechanisms, groundwater conditions, boundary conditions, structures with progression of mining and the influence of failed zones on the surrounding rock mass. The numerical model shows the degree of stability or failure. When constructing a model, it is important to divide the rock mass into geotechnical domains and zones, that is, ground control districts. Each zone is assigned its corresponding material properties. In terms of model behaviour, models can be elastic, elasto-plastic and plastic depending on the input parameters. In agreement with Lorig & Varona (2004), the reasons for using numerical analysis for this open pit to underground scenario in preference to limit equilibrium are:

- Parameters such as faults, joints, groundwater can be incorporated in the model
- Numerical analysis can be calibrated to field observations
- With numerical analysis, multiple options and possibilities can be evaluated in terms of design and planning (sensitivity analysis)

- Numerical analysis gives a full solution and not only deterministic as limit equilibrium
- With numerical analysis, a Factor of Safety corresponding to stability can be computed

The advantages of using a numerical solution rather than a limit equilibrium solution are shown in Table 13.

Table 13: Numerical solution and limit equilibrium comparison (modified after Lorig & Varona, 2004)

Analysis result	Numerical solution	Limit equilibrium
Equilibrium	Satisfied everywhere	Satisfied only for specific objects, such as slices
Stresses	Computed everywhere using field equations	Computed approximately on certain surfaces
Deformation	Part of the solution	Not considered
Failure	Yield condition satisfied everywhere; slide surfaces develop "automatically" as conditions dictate	Failure allowed only on certain pre-defined surfaces; no check on yield condition elsewhere
Kinematics	The "mechanisms" that develop satisfy kinematic constraints	A single kinematic condition is specified according to the particular geologic conditions

5.3 Construction of numerical models

Numerical models fall into two categories namely continuum and discontinuum models depending on the representation of discontinuities (Elmo, et al., 2013). The difference is that discontinuities in discontinuum are expressed in detail which include their location and orientation. Results for discontinuum models show movement. For continuum models, the discontinuities are expressed implicitly by creating interfaces between the continuum rock mass with output being subsequent deformation (Elmo, et al., 2013). Discontinuum models use the discrete element codes such as Universal Distinct Element Code (UDEC) whilst continuum models use finite difference codes such as found in Fast Lagrangian Analysis of Continua (FLAC). This research used FLAC3D because the model was continuum based. Faults and joints were not explicitly modelled thus making computation time less and solution scheme simpler that 3DEC. In model construction, it is important to select the right relationship in representing the rock mass behaviour. The most commonly used relationship is the linearelastic-perfectly-plastic model. Strength parameters (angle of friction and cohesion) are derived from the Mohr-Coulomb strength criterion.

When constructing a model, it is important to simplify and customise the model to represent the real situation. However, there are unknowns in the rock mass behaviour, hence some degree of conservatism is required. When setting up a model, a decision has to be made whether to model in 2-dimension or 3-dimension. As noted by Lorig & Varona (2004), three dimensional analysis is more suited and appropriate for the following conditions:

- When strike direction of major geological structures falls outside 25° range than that of the slope strike
- When there is no parallelism or perpendicularity of the principal stress direction to the slope
- When there is variation of geomechanical zones along the slope strike
- When the slope cannot be expressed in two dimension for plain strain conditions

5.4 Inputs of Numerical Modelling

The inputs pertinent to numerical modelling are discussed in the following sections:

Initial Conditions of the Model

The initial conditions include virgin stresses, topography and groundwater conditions. Lorig & Varona (2004) showed that if the initial k-ratio is less than one, vertical stresses have the potential to marginally decrease stability due to decreased normal stress on shearing surfaces or discontinuities in the rockmass.

Boundary Conditions

It is important to define the boundary of the model properly. The specified boundary should not interfere with modelling results hence should be truncated at far-field. A recommendation for an open pit boundary is shown in Figure 39.

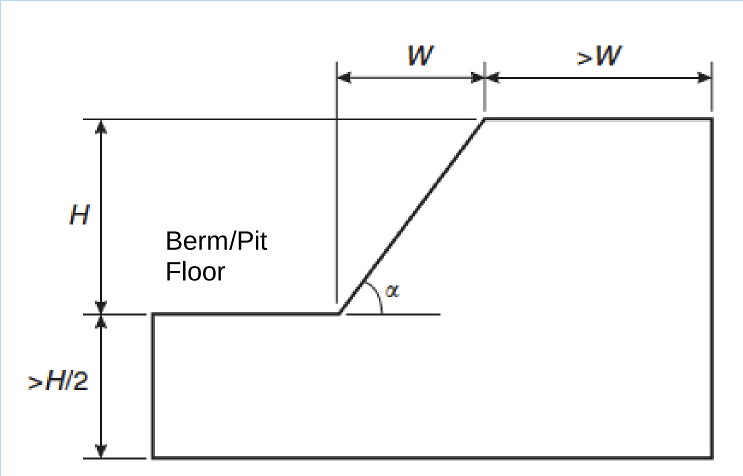


Figure 39: Boundary location recommendations (Lorig & Varona, 2004)

Lorig & Varona (2004) recommended the location of artificial boundaries away from the area of interest. For a slope with a width of 20m and a height of 100m, the horizontal extent (width) should be greater than 20m whereas the height extend should be greater than 50m as shown in Figure 39.

Groundwater

Groundwater is an important input when assigning model properties. The water pressure generated by groundwater results in reduction of normal stresses, hence reducing the shear strength of joints and discontinuities. Groundwater is analysed through flow analysis or by the water table approach with both methods using the phreatic surfaces (Styles, 2009).

Sequence of Mining

It is important to define the sequence of mining, that is, the excavation stages. This allows the model to be run using a two calculation approach for each mining step (Lorig & Varona, 2004). The first calculation is through elastic phase, which conditions the model for the removal of large material. This elastic calculation is then followed by allowing plastic behaviour to set in (Lorig & Varona, 2004).

5.5 Interpretation of Modelling Results

After running the model, there is need for Rock Engineering interpretation of results from the displacements, deformations and velocities that are calculated in the rock mass. The effects of the rock mass behaviour on the excavation are assessed (Hussain, et al., 2018). It is key to analyse the trends of the output parameters from the model to see whether they are increasing, decreasing or remaining constant. Lorig & Varona (2004) stated that instability is shown by increasing velocities and displacements whilst decreasing velocity and steady movement shows stability. Furthermore, stress redistribution analysis around excavations should be assessed.

5.6 Application of Numerical Modelling to Mine A

FLAC3D analysis was used to assess Mine A including the underground design. Initially numerical analysis was done on the open pit. Subsequently the transition to underground was done for both P1 and P2, taking note of pit slope behaviour, the interaction of pit and underground mining and siting of underground infrastructure was determined in the second step. Numerical analysis was also analysed for Phalaborwa.

5.6.1 Mine A

Mine A numerical analysis was done in FLAC3D using the Hoek-Brown failure criterion (Table 14). From site observations and geotechnical borehole logging, the majority of the joints encountered in the pits were sub-vertical, resulting in a tendency towards localised toppling and ravelling failures. From core logging, the parameters pertinent to numerical modelling were:

- Identification and vertical extent of geotechnical zones;
- Total and solid core recovery;
- Rock quality designation;
- Quantity of solid rock versus weak rock;
- Rock competence, i.e. the ratio of solid rock to matrix and the quality of the matrix material;
- defects (faults, shear zones, intense fracturing and zones of deformable material);

- Degree and nature of rock weathering;
- Intact rock strength / hardness;
- Relative orientation of discontinuities;
- Spacing between the sets of discontinuities;
- Total number / density / frequency of discontinuities;
- Condition of discontinuities, i.e. roughness profile, wall alteration and infilling; and
- Groundwater conditions

In addition, the J1 joint set had flatter dip angles in certain areas, resulting in the formation of complex wedges. The persistence of the low angled J1 joints plays a significant importance to the pit slope as well as the interaction of the open pit with underground. Input data for the model was derived from core logging and diamond drilling and is summarized in Table 14.

Table 14: Geomechanical parameters used in modelling

Rock type	UCS (MPa)	RMR	GSI	mi	c (kPa)	Φ (°)	E (GPa)
Breccia	72	45	40	6	262	34	1.9
Granite	133	63	57	16	1004	55	9.5
Kimberlite dyke	120	61	56	6	977	44	4.9
Kimberlite pipe	65	61	56	6	694	40	3
Leached granite	25	48	43	6	184	27	0.5

From site observations, the granite sidewalls of the pipe are fractured and sheared, with groundwater flowing along shear planes. The P1 pipe and dyke contacts are serpentinised joints, thus resulting in low shear strength. Also, the intrusion of smaller kimberlite dykes form contacts with low shear strength. The P2 pipe is a more regular carrot shape (circular in plan view), and dominated by volcaniclastic kimberlite breccias. Large, rounded granite boulders are found in one phase of volcaniclastic breccia and have been observed forming in fracture zones parallel to the dykes, including in fractures that do not necessarily contain kimberlite. The granitic rocks adjacent to the dyke zones are intensely jointed parallel to the dyke trend.

The plan for Mine A was to deplete the kimberlite pipes by top down sub level caving through 40m slices, 4 slices for Pipe A (Figure 40) and 5 slices for Pipe B. The initial plan was to develop the ramps and drives as close as possible to the orebody, so as to cut on the development costs and time. Stability analysis of the pit walls and the zone of geotechnical influence with progression of mining was done for both Pipe 1 and Pipe 2 using an elastic analysis based on the geomechanical parameters in Table 14. The FLAC3D analysis provided safety factor iso-shells for each pipe (Figure 40). The results of the FLAC3D analysis are provided in Figure 40 to 49.

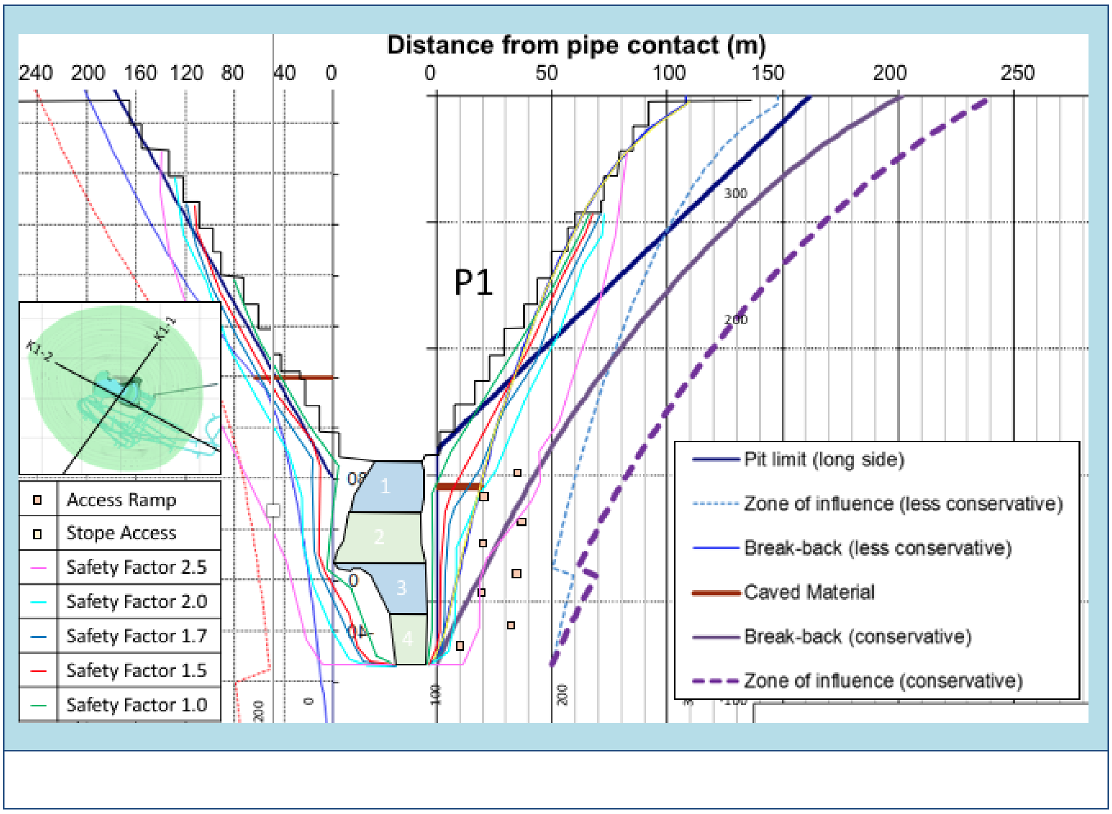


Figure 40: Section through FoS shells and back break limits for P1

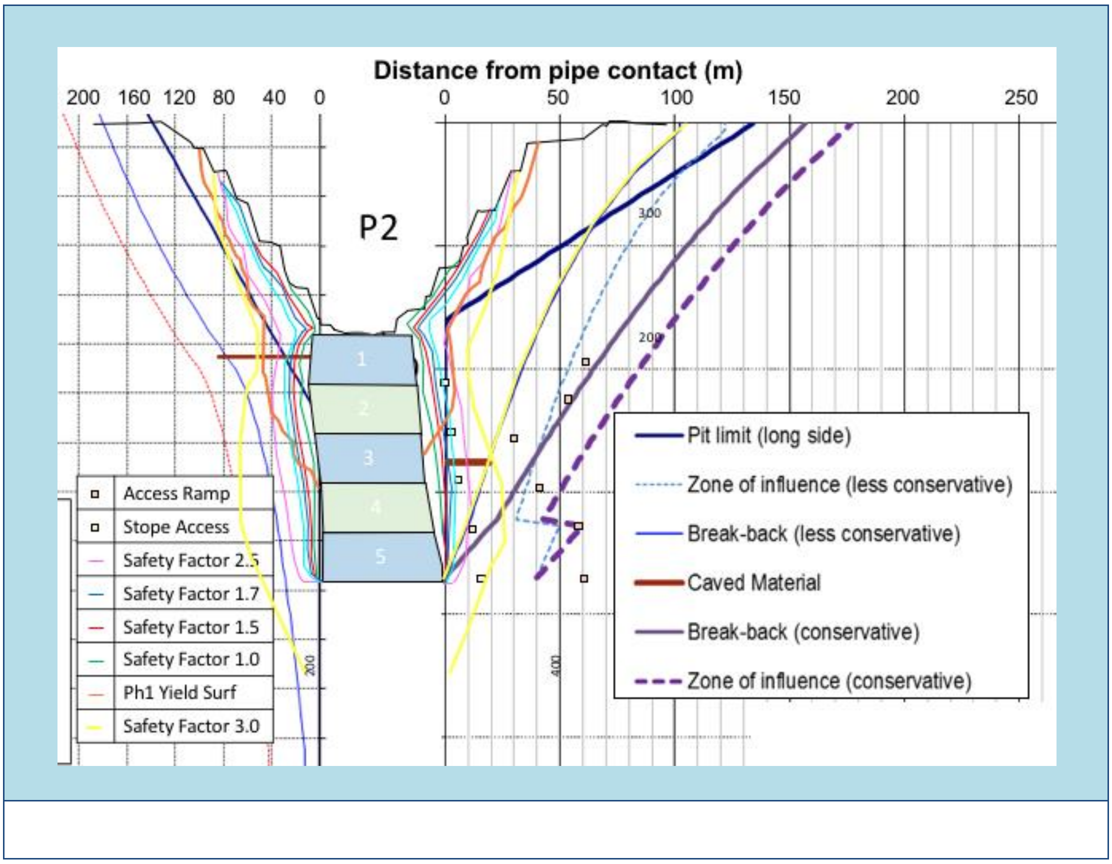


Figure 41: Section through FoS shells and back break limits for P2

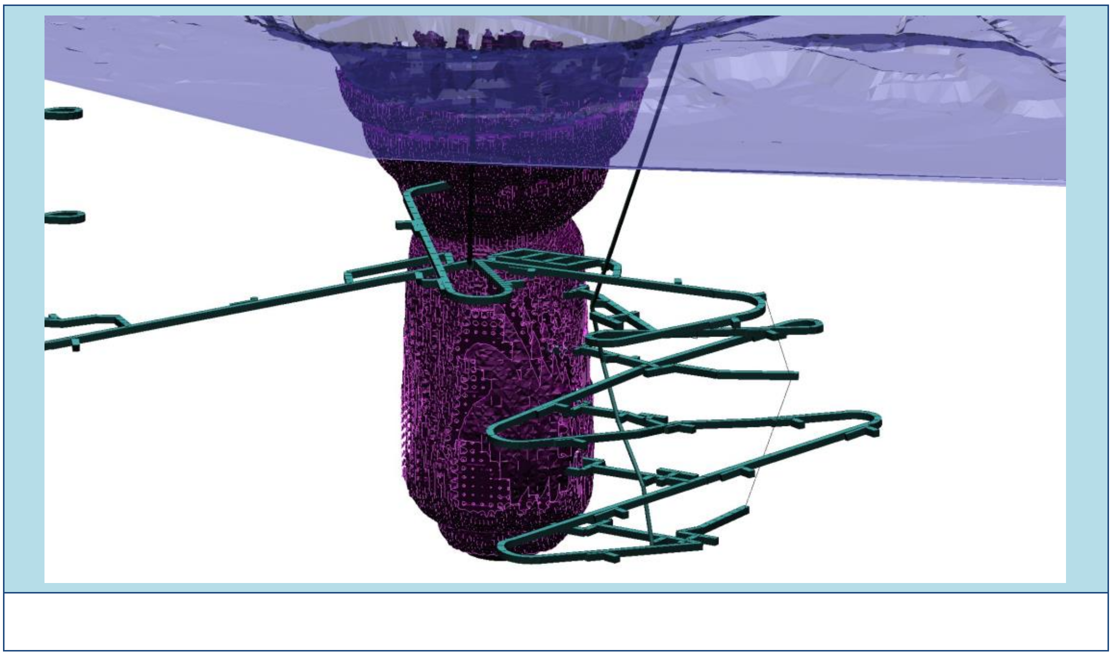


Figure 42: FoS 2 modelled for P2

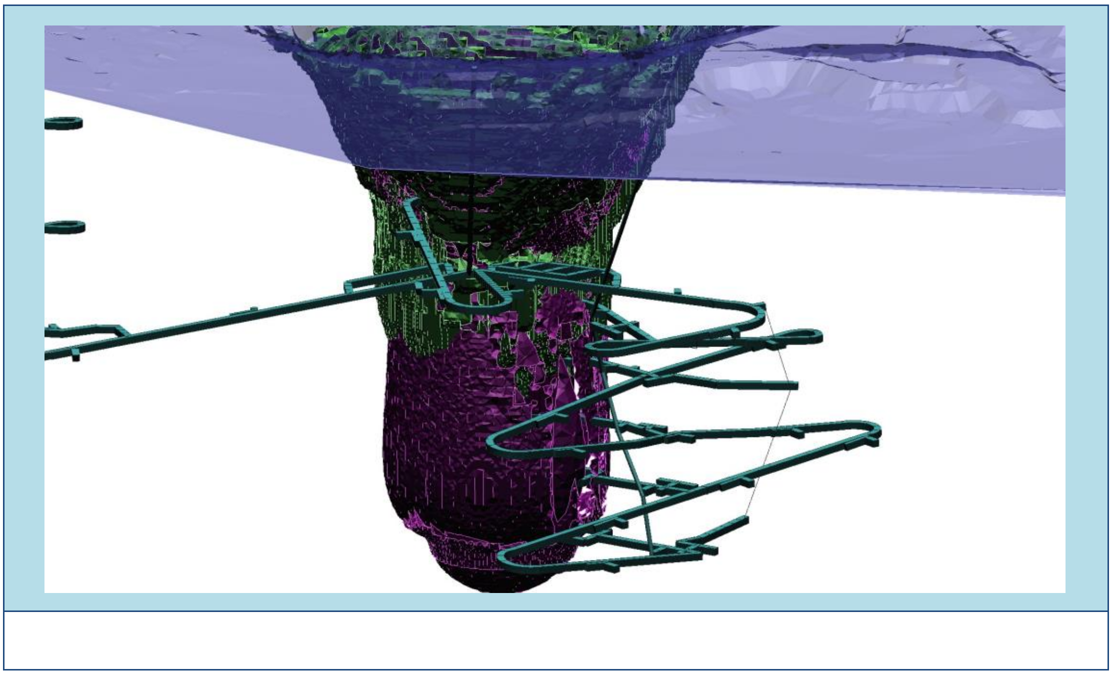


Figure 43: FoS 2.5 on P2

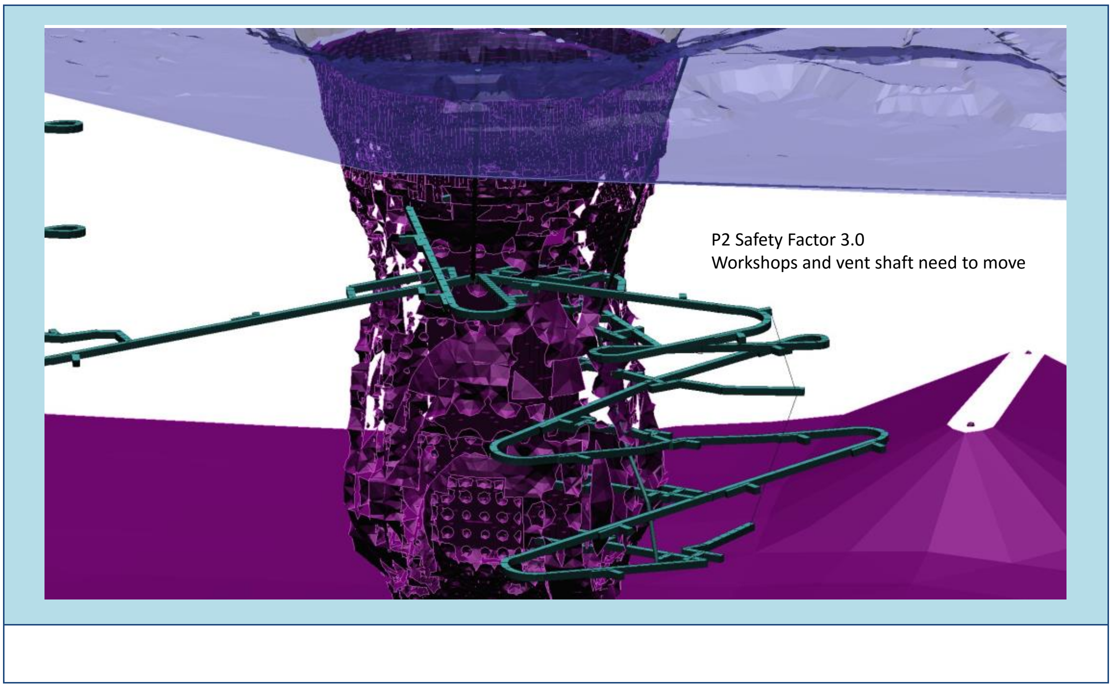


Figure 44: FoS 3 for P2

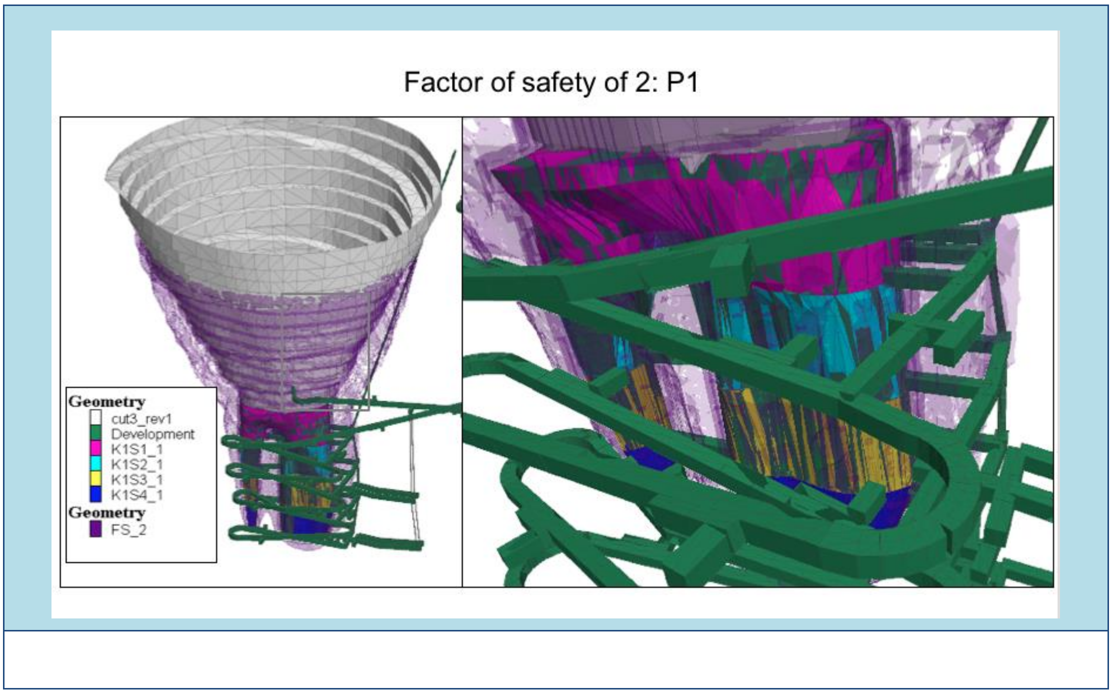


Figure 45: FoS 2 for P1 in relation to mine layout

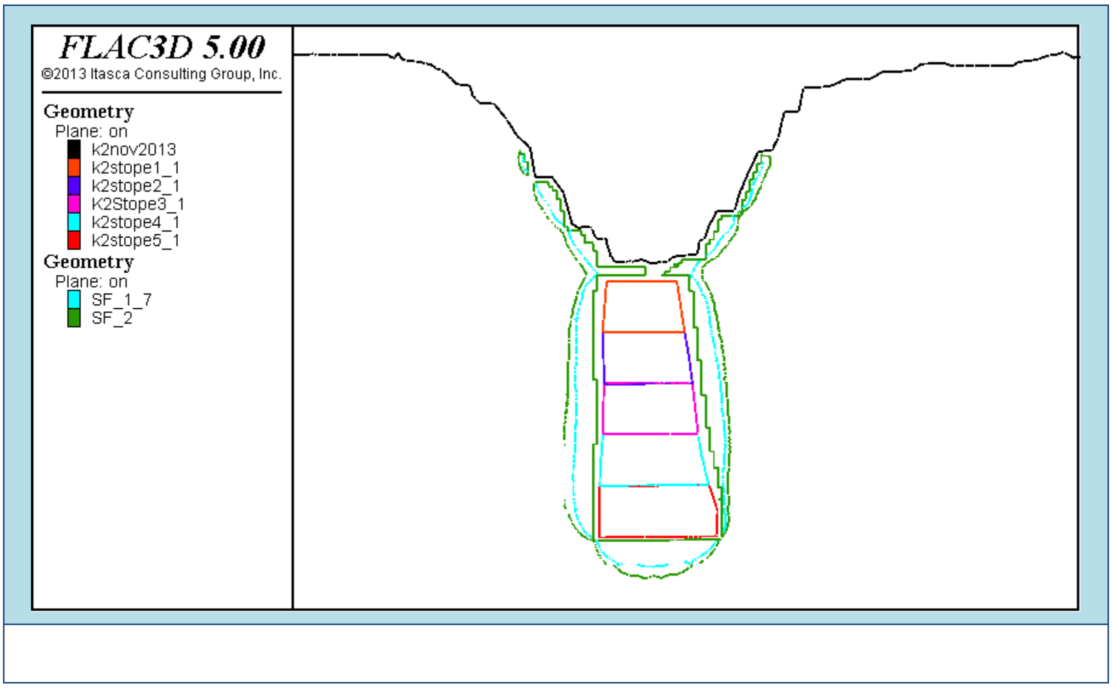


Figure 46: FoS Shells for 1.7 and 2 for P2

Sections were cut from the iso-shells showing the different outline of the Factor of Safety together with the underground drives. As for P1, the elliptical geometry of the orebody warranted for analysis along both the long and short span axis as illustrated in Figure 48.

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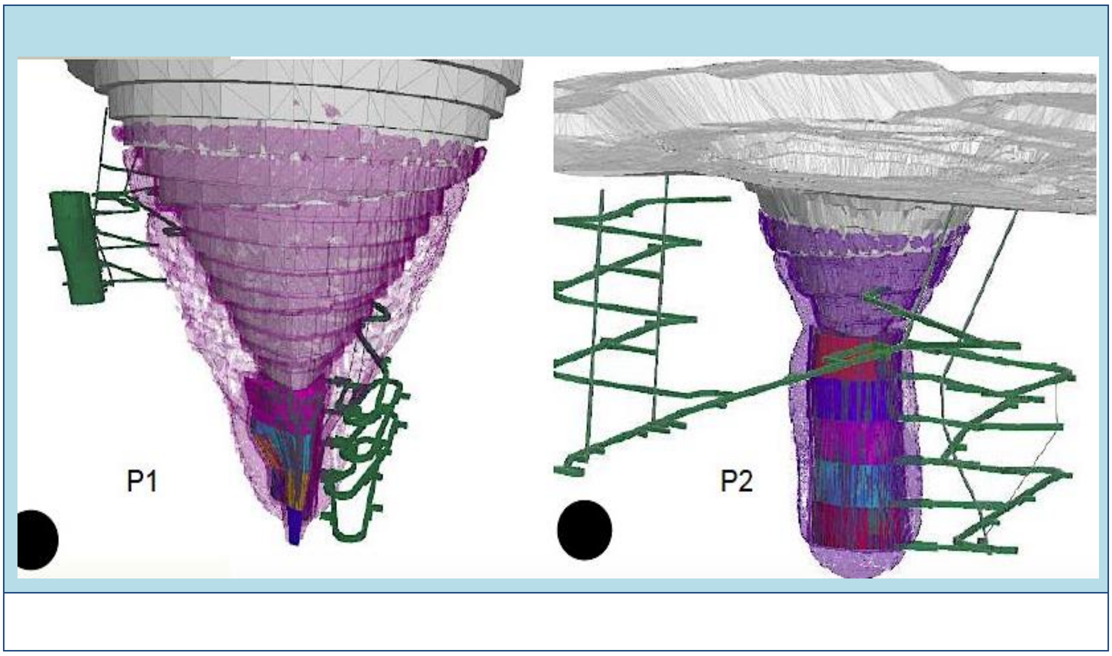


Figure 47: FLAC3D iso-shells for Factor of Safety

Sections were cut from the iso-shells showing the different outline of the Factor of Safety together with the underground drives. As for P1, the elliptical geometry of the orebody needed analysis along both the long and short span axis as shown in Figure 48. Owing to the cylindrical shape of P2, only one section was cut from the iso-shell model. This is also shown in Figure 49

As can be seen from Figure 48, the final slope failure footprint on surface is not expected to extend past the limit of the open pit. As for P2 in Figure 49, which has a shallower depth than P1, as well as brecciated leached granite; the final slope failure is slightly wider than P1 in terms of lateral extent. The presence of brecciated leached granite contributes to reduced stability on the P2 cave.

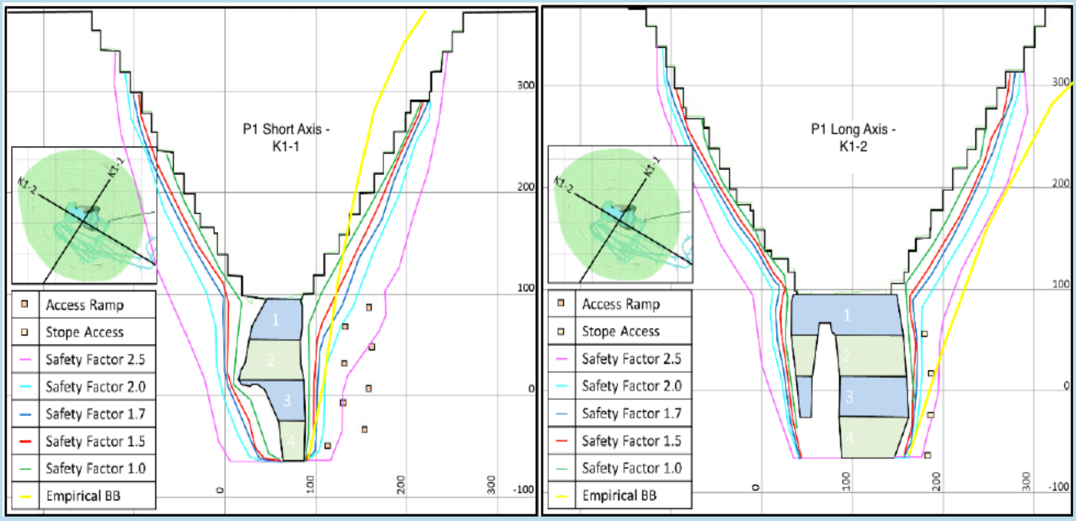


Figure 48: Modelled Factor of Safety shells for P1

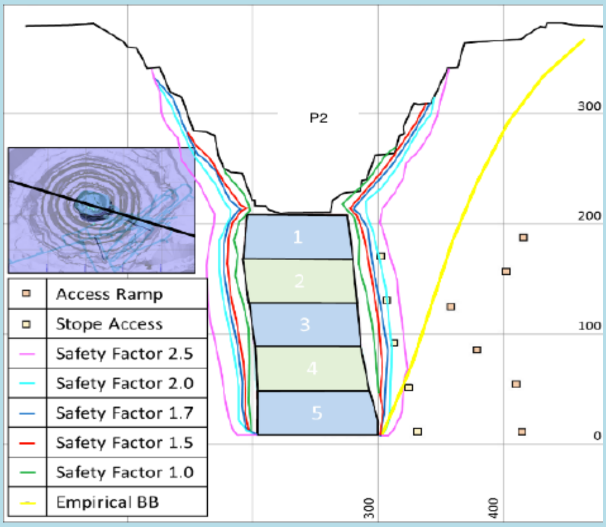
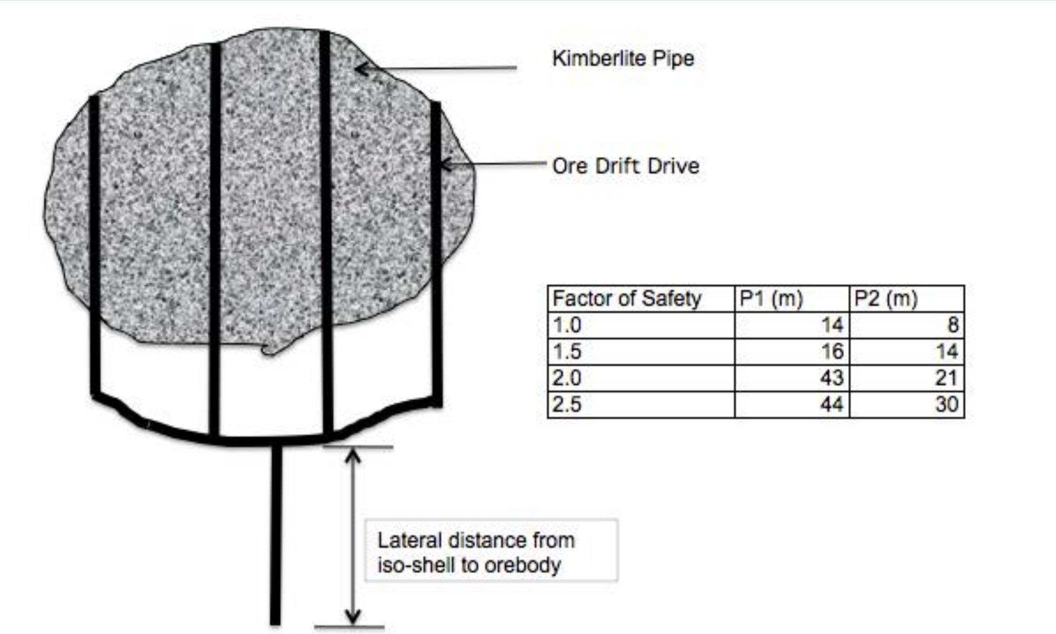


Figure 49: Modelled Factor of Safety shells for P2

As the underground mining progress from stope 1 to stope 5 in P2, confinement is lost on the walls of the mined-out area. This will progressively cause rockfalls into the excavation. The final slope failure profiles for the pits once underground mining has reached its limit is shown in Figures 48 and 49. A Factor of Safety for the two pits was chosen from Figures 48 and 49. This shell excluded underground infrastructure within its iso-shell. Therefore, the extent of the failure and zone of influence at depth has been used as a guide to position the underground infrastructure. Life of Mine infrastructure has been positioned outside of the failure zone and critical infrastructure such as workshops, ventilation shafts, crushers, spiral declines and underground pump chambers has been positioned outside the zone of influence. The effect of selecting a Factor of Safety of 2 on development metres is shown in Figure 50.



Factor of Safety	P1 (m)	P2 (m)
1.0	14	8
1.5	16	14
2.0	43	21
2.5	44	30

Figure 50: Difference in lateral development for different Factor of Safety

From Figure 50, the FoS has a development distance effect on both kimberlite pipes, P1 and P2, especially from FoS 1.5 to 2.0. As the FoS increase, the lateral distance of access ramp placement in plan also increases. This distance was deduced by measuring between the FoS iso-shell and orebody access drive.

5.6.2 Palabora

As noted by Brummer, et al. (2006), the open pit to underground transition of Palabora resulted in North wall slope failure, caused by the daylighting of a major joint set into the cave zone. Brummer, et al. (2006) subsequently modelled this failure in 3DEC. The view of Palabora together with the model representing the transition are shown in Figure 51.

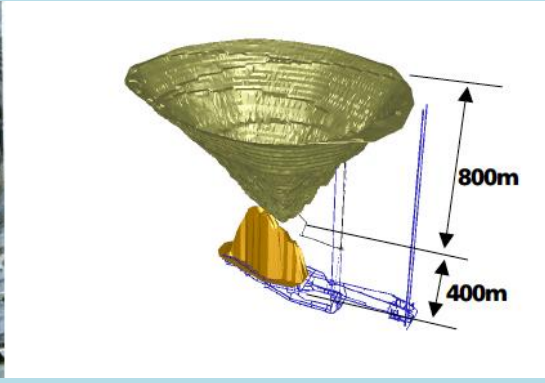


Figure 51: Palabora view and 3D model of pit and cave transition (Brummer, et al., 2006)

After mining to 800m by open pit, there was transition to underground. Block cave mining was used to deplete the 400m slice. Brummer, et al. (2006) noted that when the cave connected to the pit bottom, there was movement in the North wall, hence failure in that region. The presence of major infrastructure in that region such as Zirconium plant to the north-west, an exploration/ventilation shaft and production and service shafts to the east, as well as haul roads, rail lines, a power line, water line and water tanks were affected by the pit wall failure (Brummer, et al., 2006). The purpose of the Palabora modeling as highlighted by Brummer, et al. (2006) was to investigate:

- Pit wall failure and slope deformation
- Block cave and pit wall instability
- Long term stability of the pit walls and threats to infrastructure

A numerical model was developed which included input from geology, groundwater; modes of deformation, major structures (dominant joints and faults) and was assigned strength parameters shown in Table 15.

Table 15: Strength Parameters for Palabora modelling (modified after Brummer, et al., 2006)

Rock type	Moduli (Gpa)		Mohr-Coulomb Strength		
	Young's (Em)	Bulk (K)	Shear (G)	Cohesion (Mpa)	Friction Angle (deg.)
Carbonatite	14.1	9.4	5.7	2.9	36
Foskorite	9.9	6.6	4.0	2.3	31
Macaceous Pyroxenite	8.4	5.6	3.4	2.2	31
Weathered Macaceous Pyroxenite	3.1	2.1	1.3	1.6	23
Dolerite	13.3	8.9	5.3	4.2	44
Fenite	10.0	6.7	4.0	3.1	38
Glimmerite	6.1	4.1	2.4	1.7	24
Massive Pyroxenite	22.0	14.7	8.8	2.9	35
Feldspathic Prroxenite	8.4	5.6	3.4	2.2	31
Granite gneiss	31.6	21.1	12.6	6.2	51
Waste rock	0.5	0.42	0.19	0.0	35

The model that was constructed for Palabora is illustrated in Figure 52.

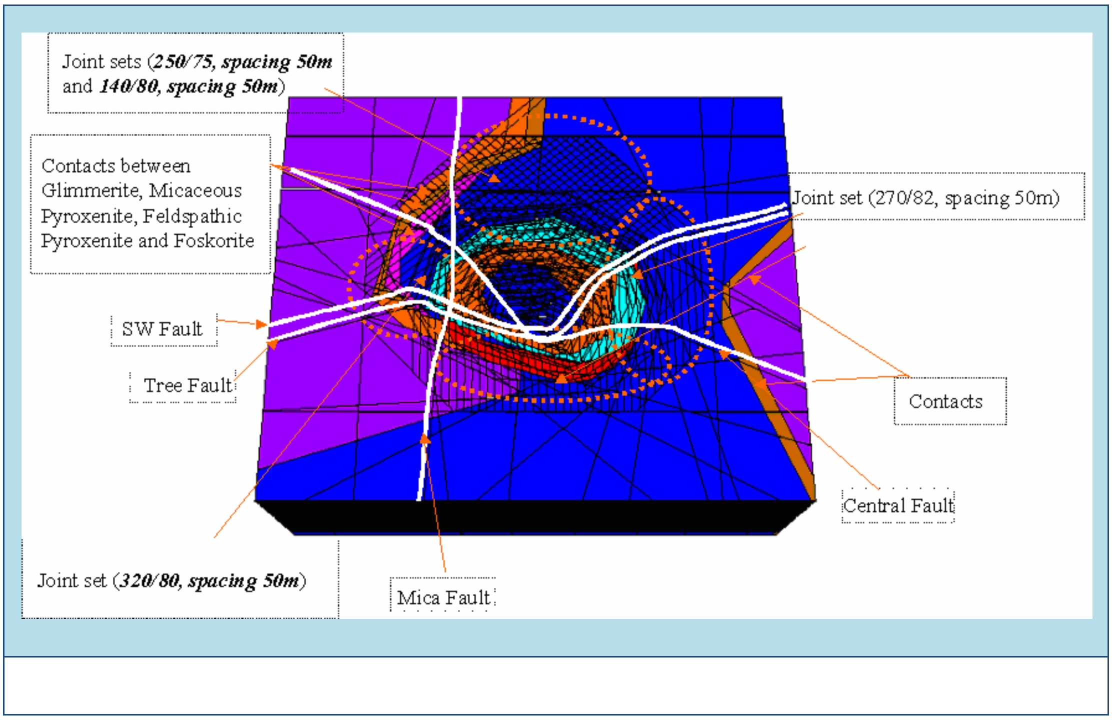


Figure 52: Palabora 3DEC model (Brummer, et al., 2006)

The results that were obtained from the Palabora model by Brummer, et al. (2006) are summarised as follows:

- Incremental displacements on the North wall, thus highlighting an unstable condition
- Failure occurred by sliding on joints and through intact material at the pit bottom
- All cardinal walls except north are stable
- Stability of the north wall is influenced by caving and progression of mining
- The production and service shafts are located outside the zone of influence of the cave

5.7 Discussion of Results and Analysis

The evaluation of pit wall stability when transitioning from open pit to underground is key to planning and safety aspects of an operation. At Mine A, the proposed underground infrastructure was long term, requiring to be stable during the life of mine. These excavations included access declines and ramps, ventilation shafts, underground workshops, pumping stations, refuge bays, ore access drifts and underground electrical distribution stations. This research sought to address positioning and siting of underground infrastructure outside the zone of geotechnical influence caused by stress changes and loss of confinement as mining progressed from pit bottom to the lower levels as well as stability of the pit slopes.

For Mine A, modelling was done in FLAC3D which incorporated the sequence of underground mining from the top stope to the last stope for Pit 1 and Pit 2. Various Factors of Safety ranging from 1 to 3 were used to determine the position of the underground infrastructure. A Factor of Safety of 1.6 is considered acceptable for the underground mine excavations, however the author chose a Factor of Safety of 2, to ensure all critical excavations would be outside the failure zone. A Factor of Safety of 2 also accounts for the unknowns of rock mass behaviour.

Signs of pit instability and slope movement were projected to start when mining the second stope, thus leading to loss of haul roads in the pit. By this time, provisions of a linking shaft between the two pits of Mine A would be completed.

In terms of the results, factors that were identified to have an effect on stability of the pit wall during the transition were geological structures such as faults and dykes, which had reduced the shear strength due to their weaker strength properties.

For Mine A, the pit wall stability was one of the biggest challenges during the transition project. An insight into the extent of slope failure and pit wall damage was required so that considerations could be made in terms of mine design. By comparing similar case studies, empirical data and numerical modelling, the author was able to address the influence of pit wall stability on underground planning and design during the transitioning period. This approach was implemented at Mine A.

The Palabora case study showed the importance of using numerical modelling as a predictive tool in evaluating pit wall stability, during the transition from open pit to underground particularly on cave-pit interaction. Major displacement of Palabora pit walls was experienced when the cave broke through into the open pit bottom. The unexpectedly high displacement was due to loss of confinement on the pit walls. The progressive failure into the North wall damaged Palabora mine's infrastructure that was located within the failure zone. It also threatened stability of key excavations such as ventilation shafts in the eastern wall. Extensive numerical modelling was subsequently done to assess the potential impacts of re-establishing the damaged infrastructure into a more stable zone.

6 CONCLUSIONS AND RECOMMENDATIONS

The transition of open pit to underground is often a difficult process with several challenges. If not dealt with properly, there is a high risk of having a failure that can result in loss of infrastructure, equipment, ore, human life and dilution. This research investigated the influence of underground planning and design on pit wall stability when transitioning from open pit to a caving operation. An extensive literature review during the transition, numerical modelling and research on the topic were done to address the objectives of the research and the research questions.

6.1 Conclusions

The major challenges that are faced during open pit to underground transition are geotechnical in nature. These aspects have to be considered during mine planning and design. Issues such as siting of underground and surface infrastructure are influenced by geotechnical conditions. Geotechnical factors that have an influence on stability are:

- Geological influences such as bedding planes, faults, dykes and weathering
- Groundwater
- Stress regime
- Geometry of the pit and underground
- Rock mass strength
- Alteration of the rockmass

During transition, it is important to monitor slope stability as an on-going process. This is pertinent to the often unexpected behaviour of the rock mass. It is important to identify stable and unstable areas around the pit in order to timeously counter-manage any rock mass instabilities that may arise. Remedial strategies include design changes to the underground workings and reinforcement strategies of slopes. In worst case situations, evacuation may be necessary to minimise losses of equipment, resources and human life. Monitoring includes regular observation, measurement and surveillance of displacements, stress changes and groundwater to keep informed about stability of the operation.

In line with the objectives of this research, the author used numerical modelling to investigate the effect of stress changes around the pit wall with progression of underground mining. The models included both the pit and underground workings incorporating faults, dykes, groundwater, joint sets and the rock material properties. From this exercise, the siting of underground infrastructure such as declines, ventilation shafts, underground stations, pump chambers and primary development could be designed with confidence. Finite Difference Method numerical modelling in FLAC3D assisted the author to plan the infrastructure outside the zone of influence of caving for the Mine A project, thus achieving the goal of the research. The Factor of Safety shells provided in the report can be used as a guide to position the underground infrastructure according to acceptable risk. A 3D surface was created and used in the design to ensure that important excavations are sited outside the expected zone of influence. However, as pointed out by Moss, et al, (2006), there is a grey area that is poorly understood in terms of interaction between induced stresses and fractures that are induced during cave propagation and breakthrough into the pit bottom.

6.2 Recommendations

The author makes the following recommendations for the study:

- There should be a strong cross-functional approach from both the Geotechnical department and the mine planning department in terms of implementation so that the activities of the project are well coordinated.
- New conditions discovered during the project should be recorded and added to the numerical model as ongoing continuous improvement
- The author recommends the presence of a spare water pump on standby with capacity to effectively prevent flooding in case there is failure on the installed water pumping system. This reduces the risk of mud-rush as well as reducing the effect of groundwater on rock mass, that is, reduction of normal stresses which in turn reduce the shear strength of joints and discontinuities.

- An effective monitoring system is required in place around the pit wall, to continuously assess and evaluate displacement and deformation as mining progresses. This also assists to refine the modelling for future work.

The author also makes recommendations for future work in terms of advancing knowledge in the area of caving induced subsidence as well as carrying out detailed risk assessment and mitigation measures for caving induced subsidence.

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APPENDIX A

Table 16: Mining Rock Mass Classification (Stacey, 2001)

Parameter		**Range of Values**										
1	*RQD*	100-97		96-84	83-71	70-56	55-44	43-31	30-17	16-.4	3-0	
	Rating (= *RQD* x 15/100)	15		14	12	10	8	6	4	2	0	
2	*UCS (MPa)*	185	184-165	184-165	164-145	144-125	124-105	104-85	84-65	44-25	24-May	4-0
	Rating	20	18	16	14	12	10	8	6	4	2	0
3	Joint Spacing	Refer to Appendix B										
	Rating	25					0					
4	Joint Condition											
	Including Groundwater	Refer to Appendix B										
	Rating	40					0					

APPENDIX B

Table 17: Adjustments for Joint Condition and Groundwater (Stacey, 2001)

Parameter	Description		Dry Condition	Wet Conditions		
				Moist	Moderate pressure	Severe
					25-125 1/mm	Pressure >125 1/min
A Joint Expression (large scale irregularities)	Wavy	Multi-Directional	100	100	100	95
		Uni-Directional	95	95	90	80
			90	90	85	75
	Curved		89	85	80	70
			80	75	70	60
	Straight		79	74	60	40
			70	65		
B Joint Expression (small scale irregularities or roughness)	Very rough		100	100	95	90
	Striated or rough		99	99	80	70
			85	85		
	Smooth		84	80	60	50
			60	55		
	Polished		59	50	30	20
			50	40		
C Joint Wall Alteration Zone	Stronger than wall rock		100	100	100	100
	No alteration		100	100	100	100
	Weaker than wall rock		75	70	65	60
D Joint Filling	No fill- surface staining only		100	100	100	100
	Non-softening and sheared material (clay or talc free)	Coarse sheared	95	90	70	50
		Medium sheared	90	85	65	45
		Fine sheared	85	80	60	40
	Soft sheared material (eg talc)	Coarse sheared	70	65	40	20
		Medium sheared	65	60	35	15
		Fine sheared	60	55	30	10
	Gouge thickness <amplitude of irregularity		40	30	10	0
	Gouge thickness <amplitude of irregularity		20	10	Flowing material	5