REFLECTIONS ON SUPPORT DESIGN IN GEOTECHNICALLY CHALLENGING GROUND CONDITIONS: A CASE OF ZIMBABWEAN GREAT DYKE PLATINUM MINING

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DECLARATION

I declare that this dissertation is my own, unaided work. It is being submitted for the Degree of Master of Science in Engineering at the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.

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(Signature of candidate)

........29th........ day of ....January........ (year) ....2018........

at ....Wits University........

ABSTRACT

Falls of ground pose costly hazards to personnel and equipment and thus measures should be taken to prevent them. The stability of excavations is ensured by good support design and sound mining practices. This research endeavours to analyse and improve the support systems used in geotechnically challenging ground conditions for Great Dyke platinum mines by analysing the current support systems and recommending effective support system thereof. Various techniques were used to determine the quality of ground conditions, predict the rock mass behaviour and to identify the appropriate support system. An analysis of the current ground control methods and their limitations was also undertaken. The reflections showed that the current support system and mining practices in geotechnically poor grounds need to be modified to improve safety and productivity. Stoping overbreak is influenced by poor ground conditions and the explosives currently used. The use of emulsion is recommended to replace ANFO. Redesigning of pillars through a reviewed design rock mass strength is also recommended taking into cognisance the current rock mass data. Pillar staggering was also seen as the best practice in geotechnically poor ground conditions in a bid to limit exposure. An evaluation of the current tendon system indicated an opportunity for improvement following comprehensive empirical and analytical design techniques. A new support system was recommended, taking into consideration cost-benefit analysis to clamp overlying layers as well as the catastrophic wedges. Barring down using pinch bars in poor ground was seen as a risky and time-consuming exercise, hence the use of mechanical scalers is recommended to achieve zero harm and to meet production targets. Smoothwall blasting is recommended in poor ground to minimize hangingwall damage. The results gathered and analysed showed that, technically, emulsion explosives are beneficial but the increase of operational cost down-weighs them. However, in solution to the problem which prompted this research, the author suggests the mines to take up emulsion as it promotes safety at higher productivity in terms of tonnage output. Other recommendations include the use of hydrological surveys to determine groundwater levels and implement corrective measures. Both empirical and numerical modelling approaches need to be utilized in determining the optimum support. Additional support is also recommended where there is pillar robbing and pillar scaling to increase the pillar strength. Poor support design and poor mining practices pose danger to employees, resulting in loss of profitable reserves and entrapment of expensive mining machinery thereby culminating in additional capital costs and reduced life of mine.

DEDICATION

This research is dedicated to:

my father Jeremiah Chikande and my mother Mynate Tau.

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In the accomplishment of this research many people generously dedicated their time and inputs which have led to its success. I feel greatly indebted for their unwavering and invaluable support they gave me even during odd times; their professionalism is par excellence.

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LIST OF ABBREVIATIONS

FOG	Fall of ground
FOS	Factor of safety
KPIs	Key perfomance indicators
MRMR	Mining rock mass rating
RQD	Rock quality designation
RMR	Rock mass rating
SRF	Stress reduction factor
UCS	Uniaxial compressive strength

LIST OF SYMBOLS

σ_c	Uniaxial compressive strength (MPa)
σ_v	Vertical virgin stress
ρ	Material Density (kg/m^3)

CHAPTER 1: INTRODUCTION

1.0 Introduction

Most platinum mines have failed to reach the target milestone of zero harm due to the presence of geological discontinuities (Roberts and Clark-Mostert, 2010). Geological discontinuities have a negative effect on the fortitude of feasible mining methods and impose implications on mine design and desired support systems. Geological structures along the Great dyke of Zimbabwe comprise of shear zones, sympathetic joints together with dykes. Mining is more arduous on predominant portions of the Great Dyke where there is an increase in joint frequency resulting in escalated incidents of support failure and falls of ground. Falls of ground inflict catastrophic effects on both personnel and equipment hence measures should be taken to mitigate them. Virtuous mining practices together with good support design will lead to improved productivity, less operating costs and improved safety.

This research analyses and endeavours to improve the current support design in geotechnically challenging ground conditions on the Zimbabwean Great Dyke. The study makes use of data from platinum mines on the Great Dyke as back analysis due to the increased frequency of fall of ground (FOG) incidents. In this chapter, the author discusses brief background information required to comprehend the study. Firstly, the author outlines the location and the geology of the Great Dyke where the research was carried out. This chapter also gives a brief overview of the mining operations. The case studies in this research cover extensive work on one mine on the Great Dyke but the research benefits can be extended to other mines which are in the same geological domain. In addition, the chapter addresses the problem statement, justification of the project, challenges, aims and objectives of the project. The author also gives the content of the research report in this chapter.

1.1 Background Information

Room and pillar mining method requires a reliable design system to avoid pillar run or excavation collapses. A comprehensive approach is essential in determining both regional and local support in geotechnically poor ground conditions. The approach on support design endeavours to bring up stable excavations, however the deficiencies in them has resulted in falls of grounds incidents. For Great Dyke platinum mines, tensile stresses accompanied by the prevailing geological discontinuities contribute to instability of designs. Inadvertent excavation collapses occur on underground shallow mines. Excavations instability endangers the safety of underground workers and also reduces the economic extraction of reserves.

Most excavations collapse as a result of poor span, imprecise pillar and support design according to a systematic design procedure and also as a result of the geological discontinuities. Bords are regularly dictated by the grade profile of platinum, equipment in use such as mobile equipment and also historical designs under similar conditions. Rock engineering design methodology is critical for the design of stable excavations for optimum safety and productivity in geotechnically challenging ground conditions. Vital parameters discussed in this research should then be used as inputs to the design of stable excavations in poor ground conditions. Support design practice is an iterative process, which includes components such as rock mass properties forecasting, identification of potential failure modes and consideration of appropriate stability analyses and other elements of the rock engineering design process. Design approaches included in the reflections of support design approach include empirical methods (comprising of rock mass classification methods), analytical methods, kinematic analyses, probabilistic analyses and numerical analyses.

1.2 Geological setting of the Great Dyke

The Great Dyke is the second largest reserve of Platinum group elements (PGEs) following the South African Bushveld complex (Oberthür, et al., 2012). It is a linear layered intrusion that extends for about 550 kilometres with a maximum width of 11 kilometres (Prendergast, 1989). It is located in the Zimbabwean craton and it is dated to be 2.50 billion years old. Geologically, the Great Dyke is not considered to be a dyke rather it is a lapolith (Prendergast, 1989). The generalised section of the Great Dyke is almost like a trumpet comprising of layers that are dipping towards the centre. The Great Dyke is longitudinally subdivided into a series of narrow contiguous stratified chambers and subchambers. There are four known geological complexes within the Great Dyke which contains platinum group of minerals (PGMs) and the base metal deposit, namely: Wedza complex, Musengezi complex, Selukwe complex and the Hartley Geological complex. In all these complexes the one that contains the largest PGM bearing is the Hartley Geological Complex, which contains about 80% of the known PGM resources in Zimbabwe (Oberthür, et al., 2012). Vertically, the Great Dyke is divided into ultramafic sequence and mafic sequence. Asymmetry in the layering pattern close to the walls is attributed to the physical shape of the chamber walls and the contrasting nature of the wall rocks, which are greenstones on the west, and granite on the east. Figure 1.1 shows the Great Dyke and various mines where the research was carried out.

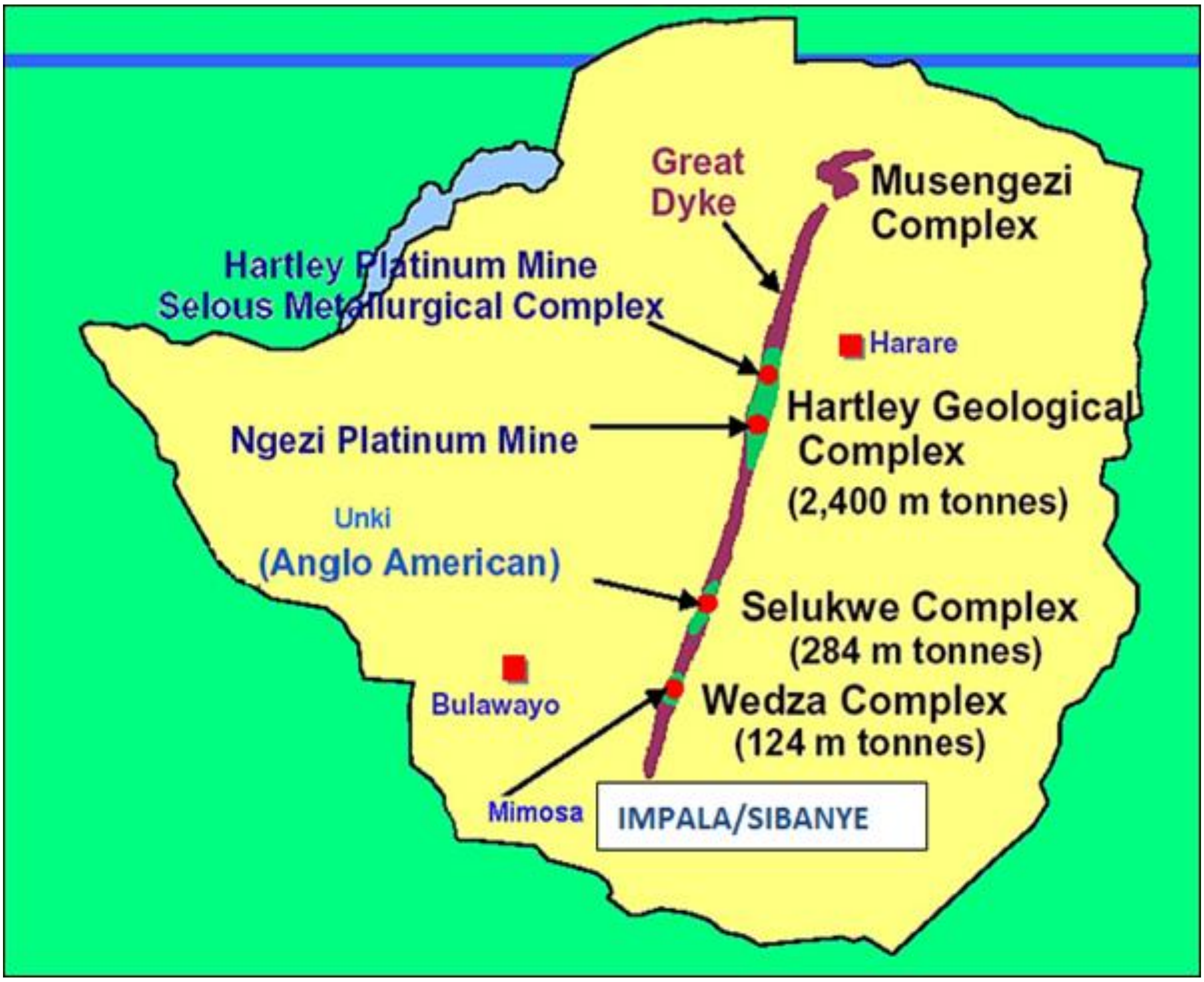

Figure 1.1: Great Dyke (Modified after Holding, 2010)

1.3 Local Geology

The ultramafic rocks (pyroxenite, harzburgite and dunite) crop out along the exposed margins of the central gabbronorite and peripheral to it with a narrow plagioclase websterite layer comprising the uppermost unit (Oberthür, et al., 2012). The underlying rocks include plagioclase pyroxenite, plagioclase olivine pyroxenite, plagioclase harzburgite, serpentinised dunite, and chromitite layers of the lower differentiated units. Figure 1.2 shows the transverse section presenting the synclinal structure of the layering and trumpet shape of the Dyke.

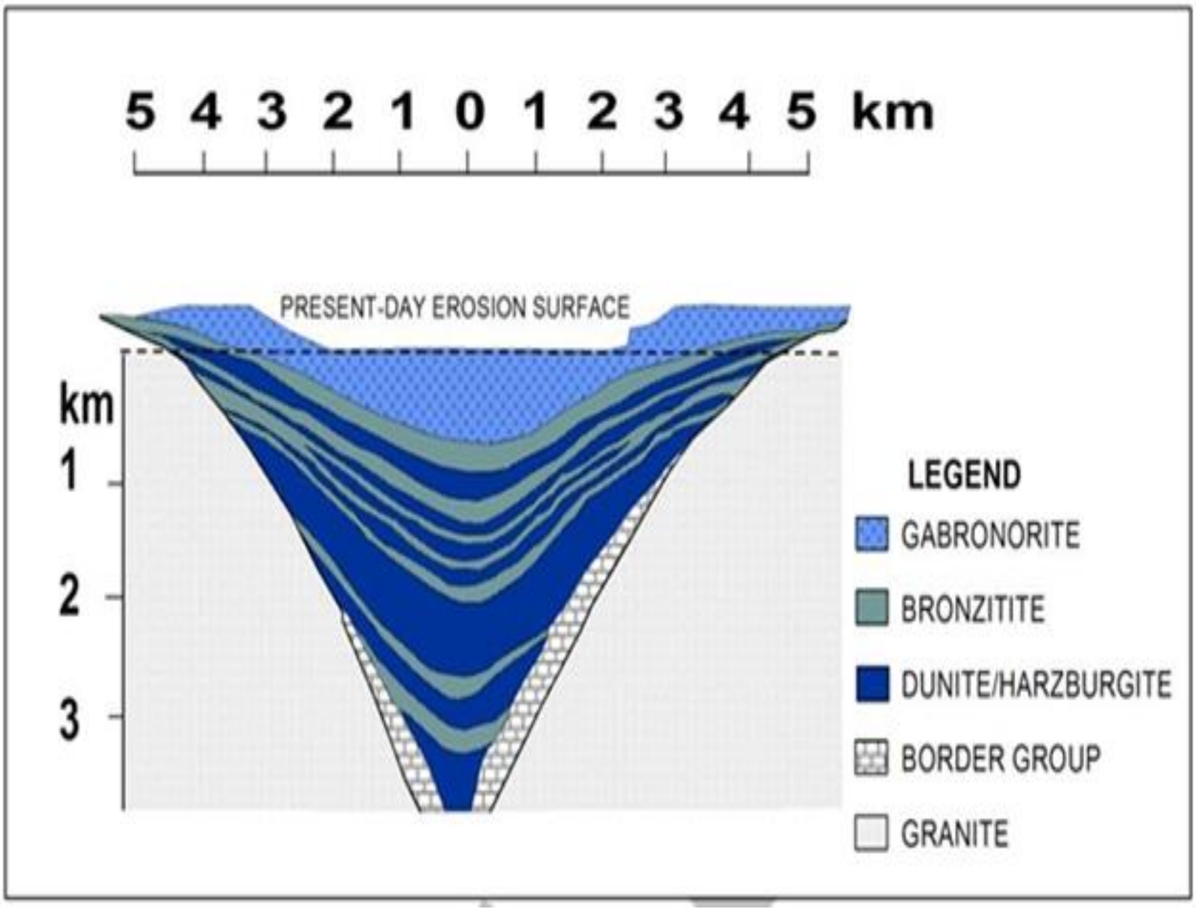

Figure 1.2: Transverse section showing the synclinal structure of the layering and trumpet shape of the Dyke (Modified after Prendergast and Wilson, 2002)

The target reef mined across the Great Dyke is called the main sulphide zone (MSZ). This reef is located in the pyroxenite layer, which is hosted in the ultramafic sequence. Figure 1.3 shows the location of the MSZ, which is between bronzite and websterite. The MSZ is a uniform layer which is about 2-3.5 metres thick dipping at around 10˚-14˚ from surface outcrop towards the axis of the basin. The visible scattered sulphide at MSZ shows a typical and consistent vertical distribution of PGMs and base metal value.

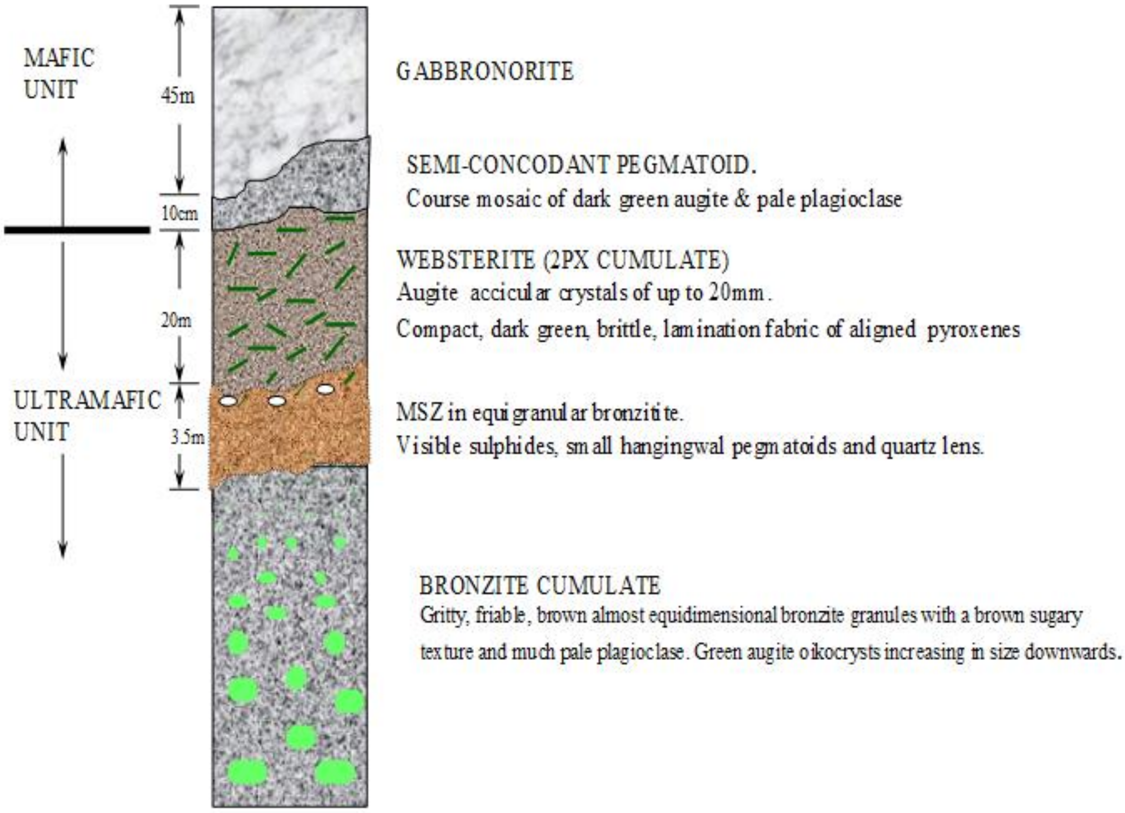

Figure 1.3: The Great Dyke stratigraphy – Schematic vertical section showing lithological features (Modified after Wilson and Prendergast, 2002)

The MSZ has a perfectly defined grade profile with a distinguishable reef horizon marker which aids grade control. Three main geological structures exist across the Great Dyke; East-West strike faults and aplite dykes, North-South striking shallow dipping joints and reef sub-parallel planes. Figure 1.4 shows the grade distribution of platinum, palladium, nickel and copper across the MSZ. From the grade distribution graph, the optimum recovery of platinum with minimum PGMs dilution can be achieved when mining within the prescribed boundaries.

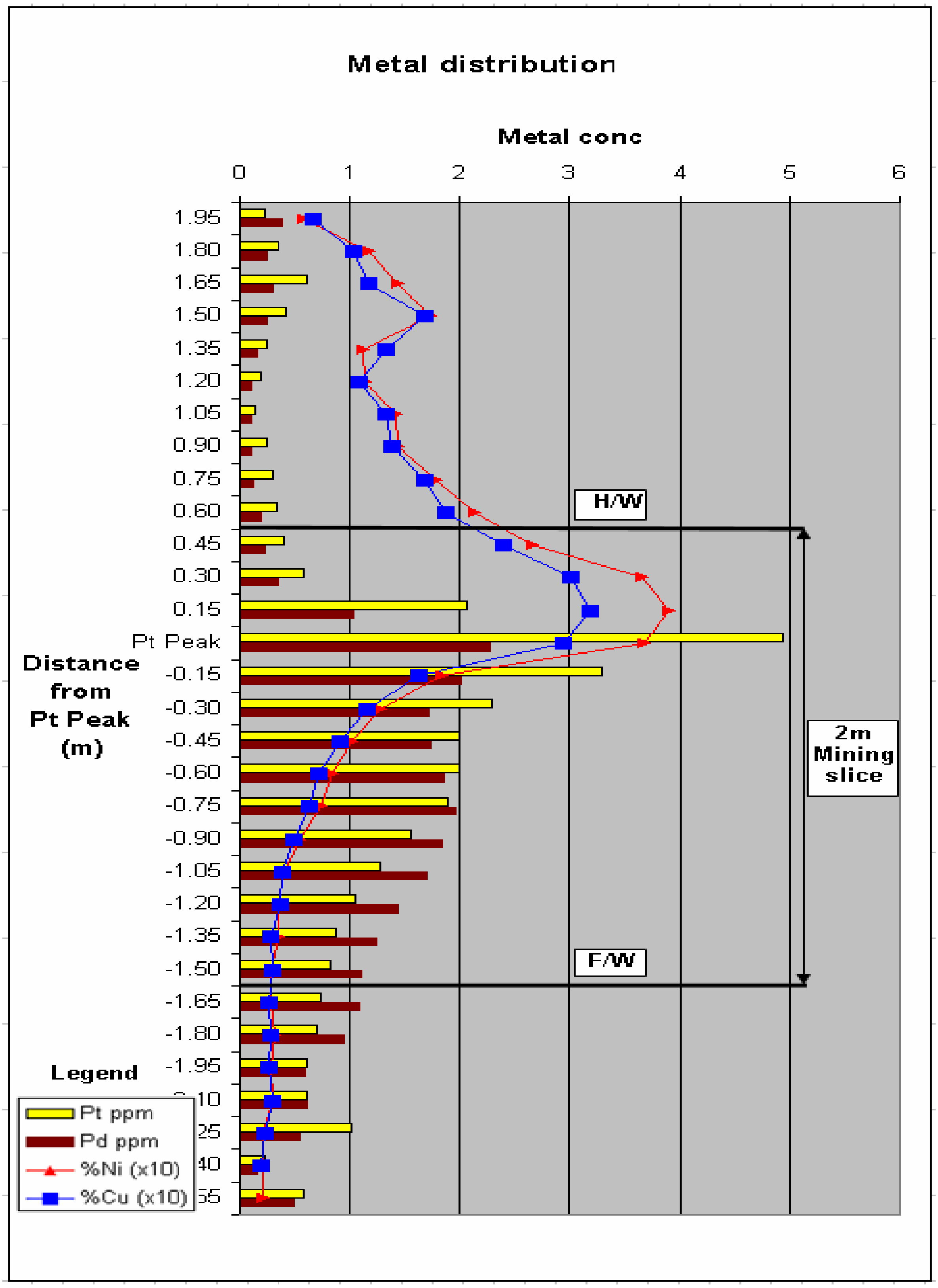

Figure 1.4: Grade profile across the MSZ (Du Toit and Duma, 2004)

1.4 Structural Geology

Faulting within the upper limits of the pyroxenite occur throughout all platinum mines of the Great Dyke. These faults have steeper dip and have relatively small throws hence are not necessarily detected during exploration. It can be anticipated that these faults and associated sympathetic joints have become prominent as mining progressed, resulting in poor ground zones around the fault zones. Advance horizontal borehole drilling should be used to predetermine the positions of these faults before mining approaches the area where they exist. Footwall faults exist in some areas that occur within the footwall of the MSZ. This feature consists of highly altered, mylonitised and brecciated plagioclase pyroxenite. The high strain central portion consists of slickensided, anastomosing mylonite, breccia, and soft gouge containing talc, sepiolite, serpentinite and magnesite. No potholes are expected to occur in the MSZ succession within the research area. Xenoliths are less common in the Eastern area of the Great Dyke and are known to exist on the western side of the dyke. The xenoliths intersected underground are very irregular and variable in size. Current mining has intersected granitic dykes which are associated with the step faults and poor ground zones emanating from fault sympathetic jointing. The granitic dykes generally strike from East to West.

1.4.1 Joints

Curvilinear joints are common in the vicinity of the faults and dykes giving rise to domes that pose ground stability challenges. Three prominent joint sets exist on the Great Dyke, namely; East-West trending, North-South trending and shallow dipping planes which are parallel to the orebody. The stability of the immediate hangingwall is critically dependant on the orientation, spacing, persistence and properties of the joints. Ground conditions in the Great Dyke are known to vary from good to very bad, largely due to the presence, density and degree of alteration of joint sets. The shallow dipping joints in the hangingwall are of concern to hangingwall stability, particularly where they are highly altered and form wedges when combined with the dominant joint sets.

1.4.2 Geotechnical properties

The MSZ Reef comprises of plagioclase pyroxenite with a UCS ranging from 160-172MPa for most platinum mines as determined from laboratory tests. The immediate hangingwall and footwall are of the same rock type and are assumed to have the same strength. The density of the rocks ranges between 3150 and 3260kg/m^3. The Young Modulus, E ranges from 99-143GPa and the poison's ratio, v ranges between 0.23 and 0.27.

1.5 Regional hydrology and Seismology

In Southern Africa, most of the rainfall occurs during the summer months from November to March, with the peak rainfall occurring from December to February. The occurrence of ground water in the plutonic igneous rocks of the Great Dyke is primarily a function of fracture-controlled permeability, degree of weathering, and rainfall recharge. The characteristics of a fractured aquifer depend on the number, length, depth, openness and distribution of the fractures (joints), and on connectivity to zones of recharge. The primary aquifers are located outside the gabbronorite, within the P1 pyroxenite either side, where the water table is typically located at about 12m below surface (Oberthür, et al., 2012). Rivers flow in both these localities. The gabbronorite hosts a seasonal aquifer that coincides with the rainy months, and this ground water has been noted to occur very close to surface. Aquifers are essentially restricted to the weathered zones. Most platinum mines assume that the groundwater conditions are dry however it is critical for the mines to use hydrological surveys in a bid to have an actual picture of the conditions. The presence of groundwater results in a loss of cohesion across the discontinuity surfaces, inevitably causing general deterioration in ground conditions.

From mining history and present conditions, the risk of a natural seismic event occurring on the Great Dyke mines is very low and hence will not be considered in this research. To date, no tremor has been felt or observed on surface or underground. There is also no evidence of seismic related failures underground. Due to the shallow depth of mining and the selected room and pillar mining method, no mining induced seismicity is anticipated on the Great Dyke mines in future.

1.6 Mining Operations Overview

All platinum mines on the Great Dyke are shallow underground mines with their operations carried out less than 400m below the surface. The mining method and the cycle of drilling, blasting, lashing and supporting are all described in this section. Shallow depths are associated with large tensile zones extending up to the surface which inflict a geotechnical challenge of hangingwall instability to platinum mines on the Great Dyke. Pillar design needs to be sound in a bid to suspend the hangingwall that is likely to be weakened by the tensile zones.

1.6.1 Mining Method

The mining method design philosophies applied to the design of pillar systems on the Great Dyke platinum mines is to provide a system of pillars that limit hangingwall deformations, surface subsidence and maintain bord stability. The pillar support system consists of non-yielding pillars with minimum Factor of Safety (FOS) of 1.6. The function of the pillars is to support the overburden from the mining horizon up to surface and maintain safe working spans in between the pillars. Permanent stable pillars are required to maintain the long-term stability of accesses to new mining areas, since all development is on-reef. The FOS of the pillars is determined using the industry accepted empirical methods that consider pillar strength and pillar stress. The room and pillar mining method is utilized by all platinum mines in layered orebodies to exploit resources. The mining operations are carried out from the main decline, advancing towards the strike direction. The main decline divides the mine into two regions, the northern part and the southern region. The size of the rooms mined varies with ground quality for each mine. At one mine, 15m rooms are mined out in good grounds and 6m wide stopes are mined out in weaker ground conditions. The stope height is maintained at platinum peak to avoid PGMs dilution. Regional pillars which measures 10m long by 3m wide are left out and are separated by 6m ventilation holings. In case of poor ground conditions, twin gullies which are 6m wide stopes are mined leaving insitu pillars of 3m by 3m. Drives were developed from the main decline to the working areas. Figure 1.5 shows the standard mining layout in good ground conditions for one of the mines. 15m rooms are mined out and regional pillars are left out for support. The 6m gulley leads the 9m panel by an advance of 3m.

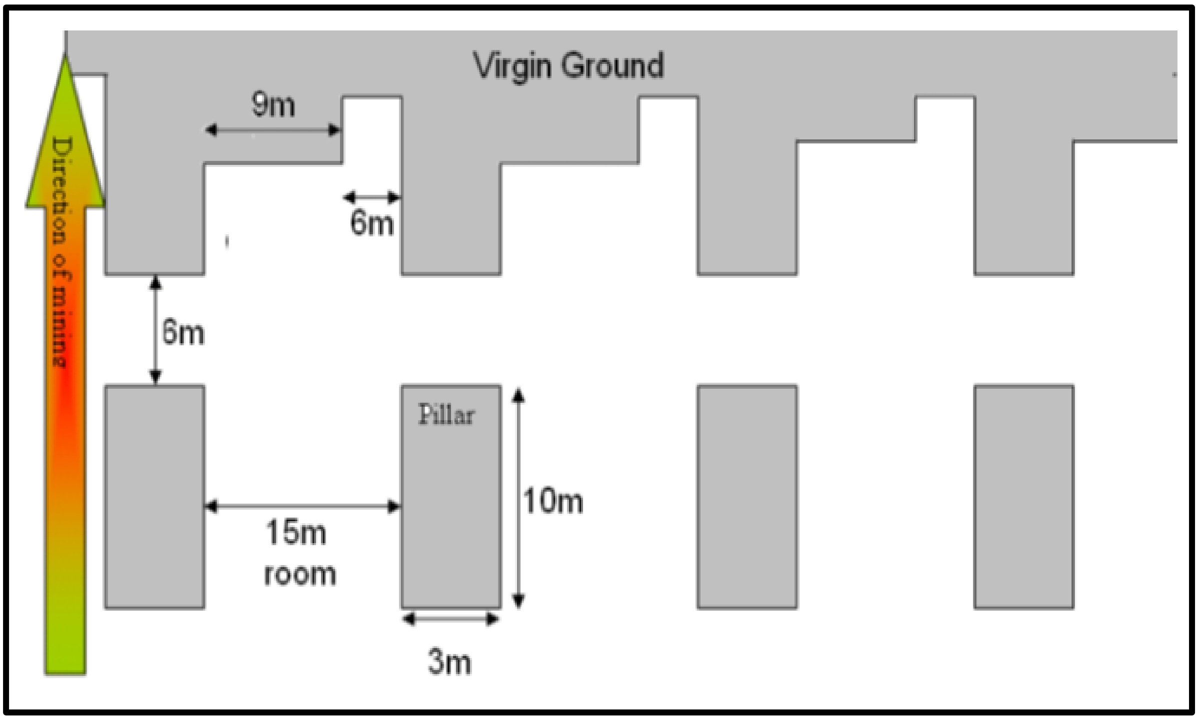

Figure 1.5: Mining layout in good ground conditions

Figure 1.6 shows the standard mining layout used in poor ground conditions. Twin rooms which are 6m wide are mined out and 3m by 3m insitu pillars are left out between the twin gullies. Regional pillars of 10m by 3m are also left out outside the boundaries of the twin gullies. The mining layout shown in Figure 1.6 will be predominantly used in this write up since the research was conducted in poor ground conditions.

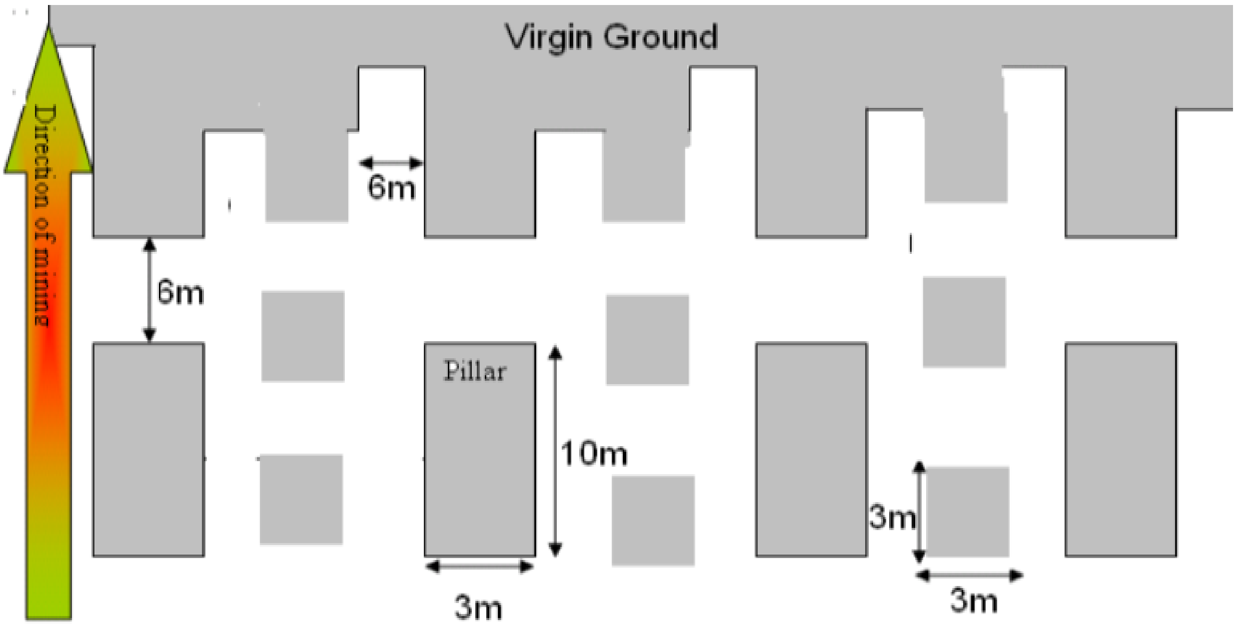

Figure 1.6: Mining layout in poor ground conditions

1.6.2 Drilling

All platinum mines where the research was carried out are mechanized. Drill rigs are used to drill holes with a diameter of 45mm. At one mine, 51 holes are drilled with three of these holes enlarged to 102mm to give a second free face (See appendix A). A 9m panel or a 6m gulley is first marked after being cleaned up by Load Haul Dump loaders (LHDs) and after all support installation has taken place. Drilling accuracy is critical for a good advance and also for maintaining the designed stoping height. The author observed the drilling accuracy since poor drilling accuracy results in stoping overbreak which affects both the effectiveness of pillars and also results in ore dilution. The blast design was also evaluated because of the implications associated with it on the support system used.

1.6.3 Charging and Blasting

Two way blow-pipes are used to clean holes after drilling. Shock tubes with detonators are used in conjunction with nitroglycerine cartridges to form primers. Ammonium Nitrate Fuel Oil (ANFO) is then used to charge the drilled holes. ANFO is a high energy explosive and its effects in geotechnically challenging grounds will be described in the next sections. The author looked at the explosives currently used because high gas explosives widen the joints, leading to the unpredictable unravelling of rocks which will result in decreased safety and productivity. It was thus critical to look at charging and blasting practices in geotechnically challenging ground conditions for Great Dyke platinum mines.

1.6.4 Ground Control

Regional support is provided by permanent insitu pillars thus barrier pillars and in-stope non yielding pillars. In addition to natural support systems, artificial support systems are also used to prevent the collapsing of the hangingwall and sidewall. Active support system is the one in which the element of support becomes part and parcel of the strengthening soon after installation (Stacey and Swart, 2001). Cable bolts, mechanical anchor bolts, resin bolts and pre-stressed timber props are a few examples of active support used in platinum mines where the research was carried out. In addition to active support, passive support is used to support a rock in response to the force imposed after sag (Stacey and Swart, 2001). Examples of passive support used include shepherd crooks bolts, mat packs, shotcrete and props. The author analysed each support unit used in bad ground conditions in a bid to improve safety and productivity. Q rating is used to classify the ground districts which require different support grid spacing for roofbolts. The mines design support grid patterns depending on the prevailing ground conditions. The author used Q rating together with other classification systems to classify the rock mass. This will be discussed in detail in section 4.2.

1.7 Problem statement

FOGs and pillar failure have affected room and pillar platinum mines as a result of the prevailing geological discontinuities on the Great Dyke. Underground collapse at one of the mines, as pointed out by Mhembere (2014) was as a result of the accelerated deteriorating ground conditions. The Great Dyke is truncated by brittle fault zones, copious shear zones and sympathetic joints which have a reflective effect on the determination of viable mining methods. Shear zones have wreaked havoc on the Great Dyke and have resulted in ground failure and elevated support requirements. Geological structures are the root cause of FOG and pillar failure resulting in concomitant production losses, injuries and fatalities. A poor to very poor rockmass based on rockmass classification is what constitutes geotechnical challenging ground conditions in this research. Understanding such *geotechnically challenging ground conditions* is of paramount importance for improved safety and productivity which is the core of this research. The associated problems and consequences include stoping overbreak, dilution, unpredictable unravelling of rocks and decreased factor of safety, support costs and a decrease in production. Mhembere (2014) pointed out that a large amount of capital is needed to address the issue of excavation stability on a regional basis. The research was carried out at one mine but the benefits can be extended to other mines on the Great Dyke which are mining the same type of deposit using the same mining techniques. Figure 1.7 represents FOG statistics from 2005 to 2016 at the research area.

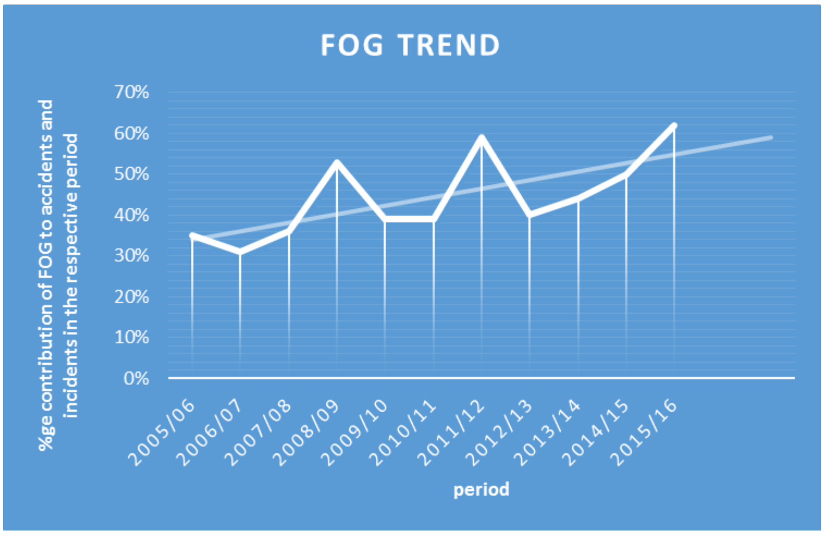

Figure 1.7: FOG trend (Unpublished internal report, 2016)

The increase of fall of ground accidents at the mine is mainly attributable to the deteriorating ground conditions. As mining progresses, the intensity of the joints is amplified making mining operations more onerous, hence the main focus of this research is to mitigate the aforementioned catastrophic threats. The stability of excavations is critical to exploit high grade ore in these regions without compromising safety. This research is thus a major stride in designing stable excavations on the Great dyke in geotechnically challenging ground conditions. Evaluation of support systems for platinum mining on the Great Dyke is vital to identify the "bottlenecks" of the current practice.

1.8 Justification of the research project

The research had to be conducted because of the following:

- To identify limitations of the current support systems in platinum mining and propose essential recommendations.
- The need to improve safety by ensuring that the support system used in bad grounds is adequate. FOG and pillar failure can claim human lives or even result in mine closure.
- The need to improve productivity by ensuring that the targets set are achieved as required without support failure.
- To minimise costs of re-supporting and litigation in the event of an accident. Currently the platinum price is at its dip hence the need to remain in the lowest cost quartile. Expensive mining machinery can be buried when pillars collapse, escalating the capital cost.
- The need to minimise PGMs dilution due to stoping overbreak and the unpredictable unravelling of rocks.

1.9 Research aim and objectives

The Research Project aims to evaluate and design stable excavations thereby optimising the three key performance indicators of platinum mining, which are; safety, productivity and costs. The main objectives were to analyse and improve the current support systems used in geotechnically challenging ground conditions. This was obtained through:

- Evaluation of the current support system and identifying its limitations.
- Designing an effective support system to be used in bad grounds based on structural mapping, empirical designs, numerical modelling and ground penetrating radar scans.

The research gives recommendations that can be used to improve the support systems.

1.10 Challenges

Inadequate support is associated with challenges of falls of ground which compromise safety and productivity.

1.11 Contents of the Project Report

Chapter 2 reviews the literature applicable to this research. Inputs for calculating the factor of safety of pillars which are pillar strength and pillar stress will be discussed. Rock mass classification systems used will also be looked at. Covered also are permanent support units and temporary support units used which are relevant to the research. In addition, a reflection on the current explosives used on stability of excavations will also be discussed in Chapter 2. Chapter 3 gives a comprehensive study approach used by the author to meet the research objectives. Research constraints are also covered in Chapter 3. A case study on a review of the tendon support system at one of the platinum mines is discussed in Chapter 4. Pillar design analysis will be covered in Chapter 5. Covered in Chapter 6 is a trial of bulk emulsion whereby a comparison of ANFO versus emulsion was done practically. The research benefits are given in Chapter 7. Conclusions drawn from this research are outlined in Chapter 8. Recommendations made based on this research study are given in Chapter 8. Appendix A shows the drilling pattern implemented by one of the platinum mines in the research area. Appendix B shows Classification of individual parameters used in the Tunnelling Quality Index, Q. Rock Mass Rating system Table is given in appendix C.

CHAPTER 2: LITERATURE REVIEW

2.0 Introduction

This chapter reviews fundamental points of contemporary knowledge on the research. The process includes categorizing relevant mining and underground support sources followed by the initial assessment of these sources. To effectively pin down the research problem and come up with effective solutions, critical review of relevant literature was undertaken. Rock masses experience primitive stress before mining. When rock is removed from within the excavation, the stresses in the immediate locality of the excavation are changed and new stresses are induced. Brady (1985) noted that virgin stresses exist in rocks prior to any excavations and such stresses will be used for pillar design. The research looks at the strength to stress ratios in a bid to determine the factor of safety using the appropriate equations. The research also looks at various authoritative rock engineering sources which are related to the topic to get an in depth understanding of rock engineering principles and techniques. Rock engineering and rock mechanics publications were used to get an indication of data acquisition and analysis. In this section, the author outlines rock mass classification systems, rock stress and stability analysis and rock characterization techniques such as geotechnical logging of underground excavations. Numerical modelling and explosives properties will also be summarized.

2.1 Rock Mass Stress

Virgin stresses exist in the rock mass prior to any excavations. Brady (1985) pointed out that the magnitude of the vertical component of virgin stress is given by the following equation:

$$\sigma_{virgin} = \rho g h \tag{1}$$

Where ρ = the density of the rock mass

g = acceleration due to gravity

h = depth below the surface in meters.

The area of interest where the research was carried out encompasses shallow mines having a maximum depth of 400m. Equation 1 was used to calculate the vertical component of virgin stress. The virgin rock stresses was used in the determination of pillar safety factors as noted by Martin and Maybee (2000).

2.2 Rock Strength

The UCS of rocks was determined from laboratory samples and can be read from the graph as the maximum stress that a rock specimen can resist without failing. The UCS of a rock is the highest point on a stress-strain curve (Stacey and Swart, 2001). The cylindrical rock specimen is a typical sample used to determine the rock strength. The tensile strength of a rock mass is approximately 10% of UCS (Wood, 1987). The tensile strength of the rocks was used in numerical analysis. The measured UCS of pyroxenite reef was used in this research for determining RMR and in calculating pillar strength. For pillar design, the UCS was downgraded to give the DRMS because the more extensive the rock mass is, the more it is influenced by geological discontinuities. The geotechnical data showing the UCS of the rocks, density and Poisson ratio is shown in Table 2.1. The outlined data is based on laboratory samples done by one of the consulting companies. No rock strength tests have been carried out at the mine for the past 20 years hence the data is outdated.

Table 2.1: Geotechnical data (Unpublished mine internal report, 2016)

Rock Type	**Density ($kg.m^{-3}$)**	**UCS (MPa)**	**Elastic (Young's) Modulus (GPa)**	**Poisson's Ratio, ν**
H/W Gabbro	3200	170 ± 5	79 ± 3	0.23-0.27
H/W Websterite		225 ± 45	126 ± 6	
MSZ Ore Zone		160 ± 10	99 ± 24	
F/W Bronzitite		191 ± 8	134 ± 2	

2.2.1 Causes of instability

Wood (1987) pointed out that ground instability can be caused from the following:

- A decrease in strength to stress ratios which results in failure of material around the excavation.
- Geological structures which results in collapse of rocks.
- A combination of the above two points.
- Seismic forces.

The area of interest is faulted and is also associated with sympathetic joints which prompted this research in a bid to improve safety and productivity. There is no history of seismic forces at the area of research hence nothing about seismic forces will be looked at.

2.2.2 Hazards associated with the geological setting

Geotechnically challenging ground conditions referred in this research are regions in the vicinity of faults, usually related to sympathetic faults and increased joint frequency. This imposes challenging mining conditions and increases the risk of rock falls. The associated consequences are outlined in section 1.7. The intensity of jointing affect excavation stability and this can be measured by rock mass classification at a particular site. Support systems and design in both the Bushveld Complex and Great dyke are greatly affected by the presence of planar joints as well as curved structures (Roberts and Clark-Mostert, 2010). The reef sub-parallel planes in the hangingwall can result in unstable hangingwall environments, resulting in block and wedge failures when the planes of weakness are intersected by the J1 and J2 joint sets. As the number of joint sets increase, the strength of the rock mass conditions deteriorates (Esterhuizen, 1997). Three prominent joint sets exist on the Great Dyke, namely:

- J1 – these joints trend East-West
- J2 – these joints trend North-South
- J3 – these joints comprise of a shallow dipping plane parallel to sub-parallel to the orebody.

JBlock software was used to determine potentially unstable blocks and the probability of support failure.

2.3 Mining method

Brady and Brown (2006) pointed out that the stability of excavations and the loading capability of the rock mass is improved by means of support. Both permanent and temporary support systems used in geotechnically challenging ground conditions in room and pillar platinum mines were reviewed to identify their limitations in a bid to improve safety. Shorter spans are mined out and in stope pillars are left out in poor ground conditions. Figure 1.6 shows a typical mining configuration at one of the platinum mines on the Great Dyke.

2.4 Support types

Two distinct areas of support should be considered thus regional type of support such as pillars and roof support such as roofbolts. Pillars of ore or waste rock are commonly used to provide regional stability in many mining methods. Examples of such pillars required to give overall mine stability include crown pillars, shaft pillars, barrier pillars and in stope pillars. Rock deterioration between the face and primary support is reduced by installation of support (Stacey and Swart, 2001). Local falls of ground are also prevented by temporary support.

Temporary support must be installed at the beginning of the shift to fulfil this function that is before any work commences until primary support takes over (Hoek, et al., 1995). Most mines on the Great Dyke are mechanised, however a review of temporary support is important for conventional mines. Temporary support must be installed and left in position for the rest of the shift. Some room and pillar platinum mines on the Dyke use hydraulic props and netting as part of temporary support. No bottlenecks were found in props and nets used in the research area hence less research was carried out to improve temporary support.

2.4.1 Primary support

Primary support should be installed immediately after an excavation and must conserve the rock strength by initiating the process of regulating displacement (Stacey and Swart, 2001). Primary support is the initial permanent support, for example rockbolts, which should maintain the integrity of excavations during further operations. Primary support is then used to either completely constitute the support system installed or to just form part of it (Brady, 1985). The current support system of roofbolts used in the research area was reviewed to check if it is adequate in bad ground conditions (discussed in Chapter 4). Various techniques were considered to determine the most effective rockbolt support systems.

2.4.2 Secondary support

Secondary support is used as additional support system to supplement primary support. It is used in order to control extremely bad ground conditions which may affect future excavations. Long anchor bolts and or shotcrete are examples of secondary support which must be used in challenging ground conditions and in areas intersected by geological discontinuities (Stacey and Swart, 2001). Long anchor bolts need to be considered where the fallout thickness exceeds the length of the current tendons in a bid to minimise support failure.

2.5 Installed support

The following materials are examples of installed support used in underground platinum mines on the Great Dyke:

- Roofbolts - roofbolts are used in underground operations to clamp rock layers together which tend to separate under gravity. Tendon design parameters such as length, bond length and spacing were reviewed for geotechnically poor ground conditions. Stacey and Swart (2001) noted that full column grouting of tendons is preferably in anisotropic jointed rockmass.

- End anchored roofbolts – are pre-tensioned and grouted to give active support and to provide complete column support along the length of the hole. These are more suitable for good ground conditions thus limited research was done since the research focused on geotechnically challenging ground conditions.
- Un-tensioned Grouted Rebar - resin setting cement grouted rebars used as support at the face provide passive support and can also be used as sidewall support in areas where rockbursts are expected. The Great Dyke does not have history of rockbursts, meaning un-tensioned grounded rebars are only used to provide passive support.
- Mechanical cable anchors - Cable anchors are used to support large brows and are also used to support huge wedges that may form in the drives or haulages. For large intersection where the Q rating is low, cable anchors are required to give secondary support of excavations. Cable bolt density is dependent on RQD, joint set number and hydraulic radius (Stacey and Swart, 2001). The density of such units was reviewed in line with the current geotechnical conditions.
- Straps - straps are normally used to provide temporary support in haulages and in the stopes as well (Stacey and Swart, 2001). They are installed to the rock face and made to follow the tunnel profile so that they may effectively clamp the rock mass together. This type of temporary support was not considered in this research due to their limited applicability in the research area.
- Shotcreting - Shotcreting is commonly used in heavily fragmented rock to give additional strength (Wood, 1987). A wire mesh can be used in conjunction with shotcrete and is also protected by shotcrete from corrosive areas such as return airways. Shotcrete is also used in pillar monitoring where there is pillar robbing so as to give additional strength and to prevent rock deterioration (Potvin and Hadjigeorgiou, 2008). Shotcreting and other confinements were considered in the research in areas where there is negative pillar infringement, thus pillar robbing, in a bid to increase pillar strength.
- Meshing - welded and diamond type is commonly used together with roof bolts to give protection by restraining heavily fragmented rock material (Brady and Brown, 2006). Rock mass classification systems were used to estimate support categories for the research area. The research area is faulted and jointed but not heavily jointed, hence the need for meshing is reduced.

2.6 Rock Mass Classification

The literature behind rock mass classification was also reviewed in relation to the research; thus Q system, Rock Mass Rating (RMR) and Mining Rock Mass Rating (MRMR). Rock mass classification systems are used to determine support requirements in tunnels (Brown and Hoek, 1980). Barton's (1974) empirical support design was considered and the limitations identified will be described later in this research. The literature behind numerical modelling was reflected on for optimum support design. The assessment of the support system used and the comparison of the qualities are provided by the quantitative classification of rock masses (Brown and Hoek, 1980). Rock mass classification methods are principally applicable in the planning and initial design stages of a rock engineering project.

2.6.1 Q System (Rating)

Barton et al (1974) noted that the *Q* system classification is based on the following three aspects:

- Block size (RQD/J_n)
- Inter block shear strength (J_r $/J_a$)
- Active stress (J_w/SRF)

Where: RQD is the rock quality designation

J_n is the joint set number

J_r is the joint roughness number

J_a is the joint alteration number

J_w is the joint water reduction factor

SRF is the stress reduction factor

Rock quality designation (RQD) is the percentage of core recovered by diamond drilling in intact pieces that have a length of 10cm or more in the total length of a borehole (Brown and Hoek, 1980). RQD therefore acts as a quantitative index of the quality of the rock mass.

$$RQD\ (\%) = \frac{\text{length of core} > 100\text{mm}}{\text{length of borehole}} \times 100 \qquad (2)$$

In case when there is no core, an approximation of RQD is obtained from a significant number of discontinuities per unit volume as noted by Brady (1985). Equation 3 was used to determine RQD since there are visible traces of joints and exploration samples will not be available for the research to calculate RQD based on length of core. Palmstrom (1982) noted that when there is no core but available traces of geological discontinuities, RQD may be estimated from the number of discontinuities per unit volume as given by equation 3.

$$RQD = 115 - 3.3\ J_v \qquad (3)$$

Jv is the sum of the number of joints per unit length for all discontinuity sets known as the volumetric joint count. RQD is a dependent factor and the use of the volumetric joint count is of paramount importance in reducing this directional dependence (Brady, 1985). The author used joint sets along the dip, along the strike and across the stoping width to estimate J_v. RQD is envisioned to give the quality of in situ rock mass. The calculated RQD is used to determine the Q rating and RMR.

Using Equation 3, the rock quality designation index is given by,

RQD = 115 — (3.3 x Jt)

Where: Jt is the total number of joints per unit length given by Jt = Jh + Jd + Js

J_h is the number of joint set per unit length in the hangingwall direction.

J_d is the number of joint set per unit length in the dip direction.

J_s is the number of joint set per unit length in the strike direction.

The author calculated all the necessary number of joint sets per unit length by counting the number of joints at a particular distance. The number of joint sets per unit length was determined by dividing the number of joints by the distance.

Values for all the above six parameters were substituted into the equation based on observed or estimated conditions to determine the value of the rock quality index.

$$Q = \frac{\mathrm{RQD}}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\mathrm{SRF}} \qquad (4)$$

The value of Q varies from 0.001 for exceptionally poor ground conditions to close to 1000 in high quality rock (Barton, et al., 1974). At intersections, a value of 3 x J_n was used as the joint set number. This is applicable to room and pillar intersections and directly lowers Q rating at intersections. The lower Q values and tensile zone implies that addition secondary support in form of full grouted cable anchors must be used. The author used the Q system to estimate the required support based on Barton's Q chart. Barton et al (1974) tabulated classification parameters used in the Q rating however the author adjusted some few parameters to match the ground conditions on the Great Dyke. The Equivalent dimension, De of an excavation was used to relate the value of the Q index to generate the recommended support. De is calculated by dividing the span (m) by the Excavation Support Ratio, ESR (Brady, 1985). The value of ESR relates to the intended use of the excavation and to the level of security required to maintain stability of excavation. Excavation category and corresponding ESR values are shown in Table 2.2.

Table 2.2: Excavation category and corresponding ESR values (Barton, et al., 1974)

	Excavation category	**ESR**
A	Temporary mine openings.	3-5
B	Permanent mine openings, water tunnels for hydropower, pilot tunnels, drifts and headings for large excavations.	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

The Great Dyke comprises of permanent mines opening its excavation support ratio *ESR* is equal to 1.6. The corresponding dimension, De, was then plotted against the value of Q in order to define the number of support categories. The recommended support from the Barton's Q chat was compared with the recommendations from the other systems such as JBlock software and ground penetrating radar and a cost benefit analysis was conducted.

Swart and Hendley (2005) pointed out that the *advantages* of the *Q System* are:

- Commonly used and well known
- Considers the effects of mining induced stresses on excavation stability
- Roughness and alteration of joints is considered separately
- Takes into consideration the effects of ground water

- Can be used to calculate rock deformability
- Offers detailed descriptions used for rating the various parameters

Swart and Hendley (2005) noted that the *disadvantages* of the *Q System* are:

- Does not take joint separation and continuity into consideration which are factors that affect joint strength
- It is generally perceived to be more applicable in tunnelling

Q rating is applicable to challenging ground conditions as it provides the tendon length and spacing. A Stress Reduction Factor of weakness zones intersecting excavations, which may cause loosening of rockmass when mining was used. Barton et al (1974) noted that the maximum unsupported span can be estimated from:

$$\text{Maximum unsupported} = 2 \text{ x ESR x } Q^{0.4} \quad (5)$$

2.6.2 Geomechanics classification / RMR

RMR system incorporates the sum of six parameters (Bieniawski, 1989). The parameters used as pointed out by Bieniawski (1989) include:

1. Uniaxial compressive strength (UCS) of rock material.
2. Rock Quality Designation (*RQD*).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities.

Brady (1985) pointed out that the relationship between RMR and Q is given by the following equation:

$$RMR = 9\ln Q + 44 \quad (6)$$

The author collected the data for RMR and made a comparison with the calculated RMR in order to determine the correct quality of the rock mass.

Stacey and Swart (2001) noted that the *advantages* of Bieniawski's (1989) *RMR* are:

- It is common and widely applied
- Adjusts for joint orientation
- Adjusts for groundwater influence

- Describes the various properties of joints i.e. alteration, continuity, separation, roughness and infill.
- Incorporates RQD and joint spacing which are easily measured to determine block size and joint frequency
- UCS which is easily measured is used in assessing intact rock strength.

The *disadvantages* of Bieniawski's *RMR* as pointed out by Stacey and Swart (2001) are:

- It needs substantial experience to be able to apply its categorisations for adjustments in joint orientation.
- Joint frequency is accounted for twice, when determining RQD and joint spacing and thus the RMR classification becomes sensitive to any changes in joint spacing
- Ignores the effect on excavation stability of mining induced stresses
- It was designed based on horseshoe excavation and its support recommendations as well
- Its design is fundamentally civil engineering and thus it is conservative in stope design
- Ignores the effects of weathering when fresh rock is exposed

RMR accounts for the joints twice, hence the applicability of such classification system is cautioned since the research area is comprised of anisotropic jointed rockmass. The results were compared with calculated RMR from Q values, however Q results were considered for empirical design as they give a better representation of the current ground conditions.

2.6.3 Mining rock mass rating (MRMR)

Laubscher (1990) came up with a method for rating rock masses in mining applications. The first phase of the system is assigning a rating to the in situ rock mass using the measured geological characteristics by weighting them according to importance. The maximum possible rating is 100. The insitu rock mass rating is called the RMR but should not be confused with Bieniwaski's RMR despite the fact that Laubscher's RMR describes the same individual characteristics though weighting them differently. The differences in weighting are highlighted in Table 2.3. Stacey and Swart (2001) pointed out that MRMR system takes into account the same parameters as the Geomechanics system, but combines groundwater and joint condition, resulting in just four parameters. MRMR system is better suited to real stability assessment.

Table 2.3: Various weighting input parameters in Bieniawski's (1989) and Laubscher's RMR (1990)

Laubscher's RMR Input parameters	Maximum Rating	Beniawski's RMR Input parameters	Maximum rating
Intact Rock Strength (UCS)	20	Intact Rock Strength (UCS)	15
RQD	15	RQD	20
Joint spacing	25	Joint spacing	20
Joint condition and Groundwater	40	Joint condition Groundwater	30 15

The adjusted mining rock mass rating (MRMR) is a result of adjusting Laubscher's (1990) MRMR to model the response of a rock mass in specific mining conditions due to the effects of:

1. Weathering of the rock mass
2. Mining induced stresses acting on the rock mass
3. Joint orientation and
4. The effects of blasting.

Laubscher (1990) also noted another parameter which is taken into consideration in the design of support systems for mining purposes, design rock mass strength (DRMS). DRMS is the rock mass strength adjusted for the four parameters (weathering, mining induced stresses, joint orientation and the effects of blasting). Rock mass strength (RMS) being defined from the intact rock strength (IRS) and the rock mass rating (RMR). The IRS is derived from the results of mechanical tests on small specimens and is then down rated by 80%.

$$RMS = \frac{(A - B)}{80} \times C \times \frac{80}{100}$$

Where: A = Total RMR rating,

B = IRS rating and

C = IRS in MPa.

The MRMR, RMR and DRMS are utilised in a versatile classification system for rock masses. The classification system provides a guideline of rock mass classes and recommends support techniques based on tables developed by Laubscher (Laubscher, 1990). The factors of adjustment are clearly detailed in tables developed by Laubscher. In the study area, the extent of weathering even after 4 years of mining is slight as agents of weathering are limited and thus an adjustment of 96% is taken as shown in Table 2.4.

Table 2.4: Adjustment to MRMR due to weathering (Stacey and Swart, 2001)

Rate of weathering and adjustments (%)					
Description of weathering extent	**6 months**	**1 year**	**2 years**	**3 years**	**4+ years**
Fresh	100	100	100	100	100
Slightly	88	90	92	94	96
Moderately	82	84	86	88	90
Highly	70	72	74	76	78
Completely	54	56	58	60	62
Residual soil	30	32	34	36	38

The adjustment in the case study area for joint orientation was taken as 80% as there were 3 joints defining the block and 2 faces facing away from the vertical as shown in Table 2.5.

Table 2.5: Adjustment to MRMR due to joint orientation (Stacey and Swart, 2001)

Number of joints defining the block	**Adjustment (%) Number of faces inclined away from the vertical**				
	70	75	80	85	90
3	3	-	2	-	-
4	4	3	-	2	-
5	5	4	3	2	1
6	6	5	4	3	2 or 1

The studied mines utilise conventional blasting without the use of special blasting techniques although standards exist with regards to drilling and blasting practices. Thus the RMR was adjusted by 94% for blasting effects on the rock mass (refer to Table 2.6).

Table 2.6: Adjustment to MRMR due to blasting effects (Stacey and Swart, 2001)

Excavation Technique	Adjustment (%)
Boring	100
Smoothwall blasting	97
Good conventional blasting	94
Poor blasting	80

Regardless of good conventional blasting techniques applied at the mine, there is a need to monitor blasting practices. Swart and Stacey (2001) also noted that good confinement promotes stability whilst poor confinement is associated with numerous closely spaced joint sets and deteriorates stability. The maximum positive adjustment for confining stresses is 120 and the minimum negative adjustment is 60.

The major *merits* of the *MRMR system* are:

- It is designed for mining applications and is free of the maximum span limitation.
- Unlike the Q and RMR rating systems, it considers the effects of blasting.
- It is cognizant of the effects of joint orientation.
- Considers also the effects of weathering exposed rock surfaces.
- It gives a guideline of suitable support systems.

The *demerits* of *MRMR system* are:

- It requires an elevated level of understanding of the system and rock mechanics to accurately take into consideration the effects of joint orientation and mining induced stresses.
- It implies that water affects only the joint condition which is not necessarily true especially in soft rocks.
- It was designed based on the caving mining system and all its case studies where in caving mining environments.

Most platinum mines on the Great Dyke use just one rockmass classification system which is the Q system. However, the author recommends the mines to consider multiple classification systems since each system has its own limitations as already discussed in this section. A conversion from one system to another was done in this research to determine the correlation between rock mass classification systems.

2.7 Pillar Design

A sound strategy for the overall mine stability is critical to avoid accidents or conditions that may give rise to incidents. The major hazards addressed by a sound mining method design and layout include uncontrolled collapses of the mine, surface subsidence, and major fall of ground incidents. Pillar support system is the chief basis of support in underground operations especially to mines that use room and pillar as their mining method. A pillar layout used in geotechnically poor ground conditions is shown in Figure 1.6. In order to design pillars for supporting mine openings, pillar strengths and pillar stresses need to be determined (Wilson, 1972). After determining pillar strengths and stresses, separate pillars and pillar layouts will be designed depending on the degree of stability needed.

Zvarivadza and Van der Merwe (2017) pointed out that there are four types of pillars in use for shallow mining practice. The pillars identified are non-yield, crush, yielding and barrier pillars. The author focused on non-yielding in stope pillars and barrier pillars in this research because crush and yielding pillars are not applicable to mines operating in the research area. The author reviewed the design criterion of current pillars using the width to height design graph shown in Figure 2.1.

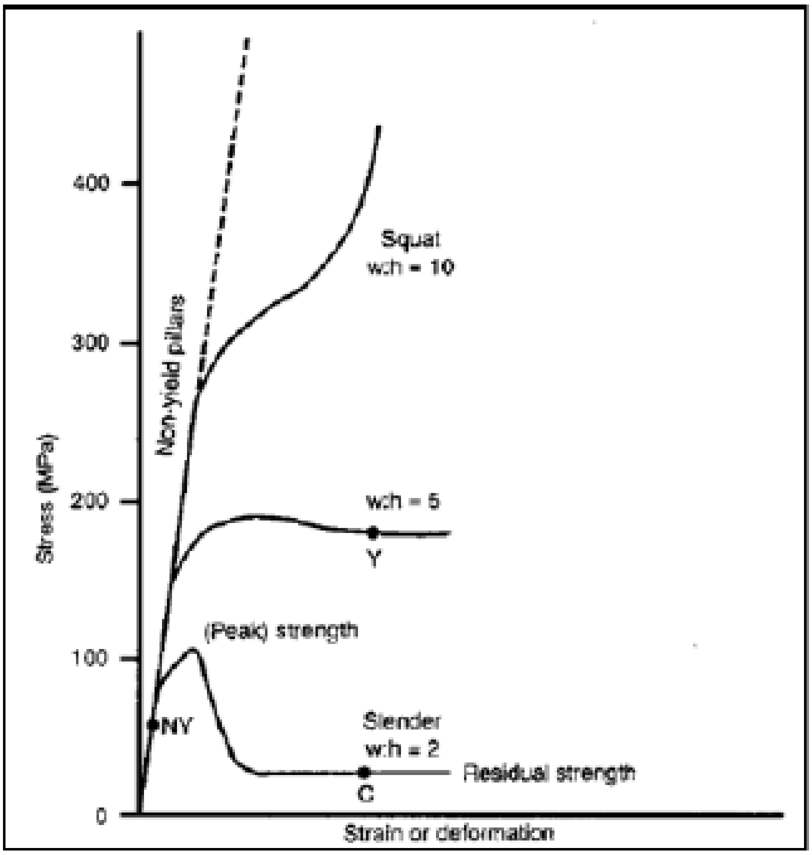

Figure 2.1: Typical stress-strain behaviour of hard rock pillars of different width-to-height ratios. Typical operating points are shown for NY (non-yield and barrier), C (crush), and Y (yield) pillars (Jager et al, 1995)

2.7.1 Non-yield pillars

These are rigid pillars that protect the mine workings and mitigate the effect of surface subsidence hence these pillars are designed in such a way that they do not fail. Jager et al (1995) defined non-yield pillars as those pillars which are envisioned to remain intact and elastic during the life of the mine. Stacey and Swart (2001) noted that at shallow depths, the tensile zone in the hangingwall can extend up to surface. The designed safety factor of in stope non yielding pillars should be greater than 1.6 in hard rock mines to avoid pillar run. Figure 1.6 shows that the current in stope non yielding pillars are 3m wide. The desired mining height at the research area is 2m, which gives a width to height ratio of 1.5. The industry accepted practice for bord and pillar design suggest that the w: h ratio must be greater than 2.5. The current layout shows that the current in stope non yielding pillars do not match the design criteria hence redesigning of pillars was considered in this research in line with the industry accepted practice.

Hedley and Grant (1972) formula was developed after modifying Salamon and Munro (1967)'s formula and is used in hard rock pillar design. Jager et al (1995) noted that the tensile zone in the hangingwall can extend to the surface at very shallow depth, and under such conditions, the design of hard rock pillars is similar to the room and pillar system in coal mining.

The application of the rockmass strength as noted by Hoek and Brown is given by the formula:

$\sigma_1 = \sigma_3 + (m\sigma_c\sigma_3 + \sigma_c^2 s)^{0.5}$

This predicts the unconfined rockmass strength comprising a pillar to be

$\sigma_1 = 0.32\sigma_c$

Where

σ_1 is the rockmass strength

σ_c is the laboratory UCS

σ_3 is the confining stress and

m and s are material constants described by Hoek and Brown

Jager et al (1995) noted that the standard value of 1.6 for the factor of safety (FS) is widely used in coal-mine rock engineering and is also used in hard-rock mining. The author therefore used the same FOS since the research was carried out in hard rock mining environment and the mines are utilising bord and pillar mining method.

2.7.2 Barrier pillars

Barrier pillars separate panels since they prevent the collapse in one panel to spread to other panels which can consequently result in pillar run. Barrier pillars in the research area are designed along panel boundaries giving a safety factor greater than 2.5. Jager et al (1995) pointed out that regional pillars must have w: h ratios greater than 5. The current design was also reviewed considering the effective pillar width of barrier pillars since the pillars are rectangular. The pillars are also designed in such a way that they have to be intact and elastic for the whole life of the mine. Barrier pillars are critical as collapse in one stope should not affect neighbouring stopes. They are also critical in reducing closure and surface subsidence which have a negative impact on pillar design. When barrier pillars intersect geological discontinuities, the size of the pillar must be increased in a bid to increase the area supported, thereby improving safety. These pillars should intersect geological discontinuities at ninety degrees as this improves excavation stability. The design of barrier pillars is similar to that of non-yielding pillars discussed earlier on.

2.7.3 Pillar Strength

Coates (1981) noted that the strength of the pillars depends on:

- The strength of the intact rock which makes the pillar material, suitably down rated to take into account the scale effect.
- The geometry of the pillar taking into account the shape and width to height ratios relationship.

The insitu rock material affects pillar strength thus the stronger the insitu material the stronger the pillar. Pillar size also affects the strength of pillars because undersized pillars will result in a concomitant pillar run as well as pillar scaling. The current pillar design in the research area was reviewed and its reflection in bad grounds will be discussed in chapter 5. Geological discontinuities have to be considered in assessing pillar strength (Zvarivadza, 2012). Different geological structures such as faults and joints exist in the MSZ in which pillars are established and these have a negative influence on the stability of pillars since they decrease pillar strength. Zvarivadza (2012) further pointed out that the existence of geological disturbance will result in pillar failure after mining despite having higher width to height ratio. Since geological discontinuities greatly affect the stability of pillars, the author looked at rock mass classification data to deduce an estimated value for the designed rock mass strength. Falls of ground occur usually due to the interaction of joints and lack of confinement. Pillar confinement is critical in undersized pillars as this increases pillar strength (Castro-Filgueira, et al 2017). This can be achieved through cable anchoring, wire mesh as well as shotcreting the pillar. Blasting activities have an undesirable effect on pillar strength hence the author reviewed the current blasting practices in bad ground conditions. The degree of blasting was observed and quantified so that it will be incorporated in pillar design to improve safety.

The strength of hard rock pillars is given by equation 7 using Hedley and Grant (1972) formula as noted by Martin and Maybee (2000):

$$Pillar\ Strength = K\frac{W_e^{0.5}}{h^{0.75}} \tag{7}$$

Where

K is the design rock mass strength (DRMS) in MPa

W_e is the effective pillar width

Wagner (1974) noted that the effective width for rectangular pillars is given by

$$W_e = 4\,\frac{Pillar\ area}{Pillar\ perimeter} \tag{8}$$

For pillars with a W_e : H ratio greater than 4.5 the strength of the pillar is then defined by the squat pillar formula as follows:

$$Pillar\ Strength = K\,\frac{2.5}{V^{0.07}}\left\{0.13\left[\left(\frac{R}{4.5}\right)^{4.5} - 1\right] + 1\right\} \tag{9}$$

Where: K is the strength factor of the rock,

H is the pillar height,

V = $\frac{W_e^2}{H}$,

R = $\frac{W_e}{H}$,

Zvarivadza and van der Merwe (2017) noted that the current pillar design systems for narrow reef platinum mining consider width-to-height-ratio and the strength of the pillar material. They pointed out that several important factors that have a bearing on pillar system stability were not considered. Some of the omitted factors include: contact of the pillar with the roof and floor, roof and floor conditions, effects of adversely oriented joints, spalling and side scaling effects, influence of pillar loading condition, blast damage effects, influence of weak layers and weathering, impact of k-ratio, time-dependent effects, geology, fractured zones, and effects of different types of discontinuities within the rock strata (Zvarivadza and van der Merwe, 2017). An in-depth study of these parameters with a view to establish effective narrow- reef platinum mining pillar design systems needs to be undertaken. Work by Martin and Maybee (2000), Hedley and Grant (1972) etc. was used in this research for pillar design despite their limitations.

According to Zvarivadza (2012), K value lies between UCS/3 and UCS depending on the rock mass quality. As the width to height ratio of a pillar decreases, the pillar becomes gradually weaker. The existence of joints in pillars will decrease pillar strength since geological disturbances represent weaknesses (Esterhuizen, 1997). Pillar monitoring need to be implemented to determine pillar performance and also for information gathering that will be used in planning. Measurements of mining heights together with pillar length and pillar width will be used to determine the factor of safety of pillars thereby monitoring pillars.

2.7.4 Pillar Stress

In situ stress conditions together with local and regional extents of mining will determine the stresses acting on a pillar (Wilson, 1972). The platinum mines use room and pillar mining method hence pillar stress is determined using the tributary area technique. The tributary area theory states that for a horizontal mining layout, pillar stress (P_{stress}) is given by

$$P_{stress} = \frac{\sigma_v}{1-e} \qquad (10)$$

Where: σ_v is the vertical field stress

e is the extraction ratio

Stacey and Swart (2001) noted that for inclined pillars, the average pillar stress is given by:

$$P_{stress} = (\sigma_v.\cos2\theta + \sigma_h.\sin2\theta)/ (1 - e) \qquad (11)$$

Where

σ_v is the vertical in situ stress

σ_h is the horizontal in situ stress

θ is the dip angle of the mining horizon

The Great Dyke has a shallow dipping orebody hence pillar stress equation for horizontal to sub-horizontal mining layout will be used to determine the pillar stress. Equation 1 discussed in section 2.1 was used to calculate the vertical stresses acting in the area of interest.

2.7.5 Pillar design procedure

Stacey and Swart (2001) pointed out that after determining the pillar strength and pillar stress, the factor of safety (FOS) of the pillar can be calculated as follows:

$$FoS = \frac{Pillar\ strength}{Pillar\ stress} \qquad (12)$$

The choice of the FOS value to be used for the design of the pillars and layout depends on the function of the pillars. Stope or panel pillars are required to provide stability to that section of stope between barrier pillars. The requirement for stability is therefore not as critical as that for barrier pillars, and occasional instability and failure of pillars is acceptable provided that it does not compromise safety (Wilson, 1972). An indication of some instability is when spalling begins to occur from the pillar sidewalls. The design principles are dependent on parameters such as rock strength and quality of the hangingwall rock mass.

Due to the effect of the explosives used and bad ground conditions, the author reviewed the current drilling and blasting practices by measuring the actual pillar dimensions in a bid to calculate the actual factor of safety. The effect of stoping overbreak is that it reduces the factor of safety of pillars because it reduces effective width of the pillars. This reduces the width to height ratio, thus the smaller pillar bears more load which increases the probability of failure. Apart from reducing the factor of safety, the effect of overbreak increases ore dilution which affects revenue, hence it was critical for the author to review the literature of pillar cutting practices. The function of the pillars is to support the overburden from the mining horizon up to surface and maintain safe working spans in between the pillar (Ryder and Jager, 2002). The permanent stability of pillars is also required to maintain the long-term stability of accesses to new areas, since all development is on-reef. The factor of safety of the pillars is determined using industry accepted empirical methods that consider pillar strength and pillar stress. It should be noted that the above considerations may be over-ridden by other requirements such as the necessity, in certain instances, to ensure that no subsidence occurs on surface. In such cases, the pillar system was designed so that regional stability is ensured. Redesigning of pillars was considered following a review of the current pillar system. The conclusion was drawn after analysis of results.

2.8 Tendon support requirements

Tendon design incorporates specification of appropriate bolt type, bolt capacity, roofbolt length, grid spacing for particular geotechnical conditions, stress level and application. The interfaces between the aforementioned parameters are very intricate. Ground conditions dictate the reinforcement to the roofbolts. Swart (2005) identified four control mechanisms contingent on the stress regime as well as the geology of the rockmass.

2.8.1 Skin Control mechanism

Sporadic loose rocks can be created at the skin of an excavation as a result of cracks, slickensides and joints in strong and self-supporting massive roof as shown in Figure 2.2. In this environment, the function of the bolts is to prevent local rock falls, not to prevent a major collapse (Swart, 2005). Skin control mechanism is also an imperative secondary function of tendon support in geotechnically challenging ground conditions. Skin control involves passive support that covers the surface of the excavation applicable in main accesses. Examples of such mechanism include shotcrete, liners, mesh and tendon faceplate.

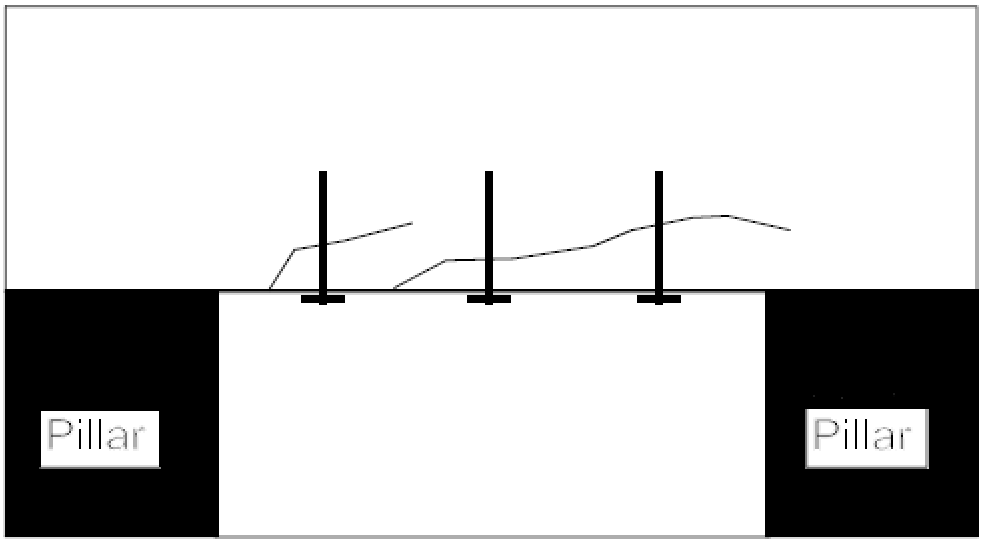

Figure 2.2: Skin Control mechanism

2.8.2 Suspension control mechanism

In most underground mines, a competent rock that is self-supporting overlays a weak hangingwall layer as shown in Figure 2.3. In these circumstances, roofbolts are important to suspend the weaker immediate layer on the competent overlaying rock.

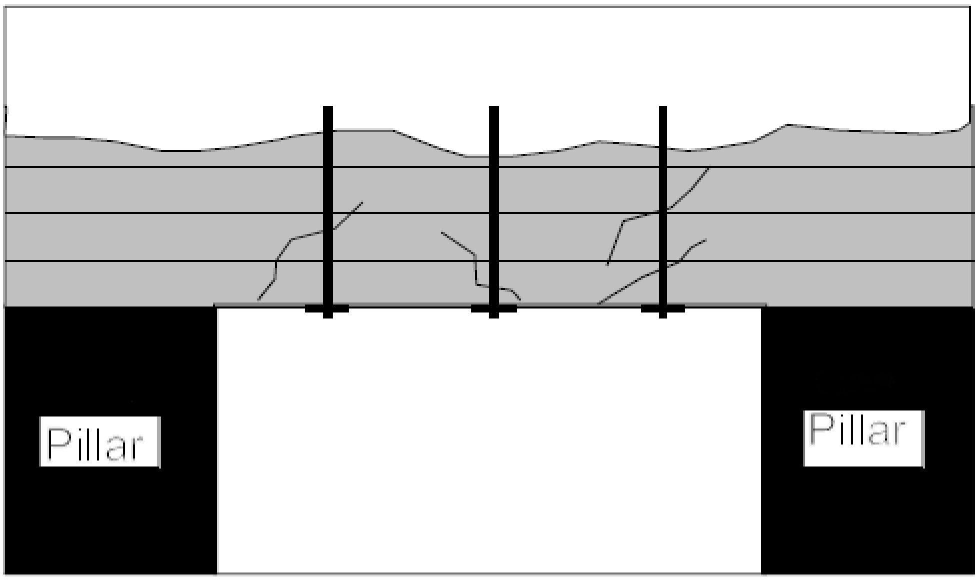

Figure 2.3: Suspension mechanism

2.8.3 Beam Building control mechanism

In circumstances where no self-supporting ground is within reach, tendons will clamp the overlaying layers to create a beam. The roofbolts thereby bond the blocks together and also controls dilation of failed rock layers. Beam building mechanism is shown in Figure 2.4.

There is need for greater support density in beam building as opposed to suspension mechanism.

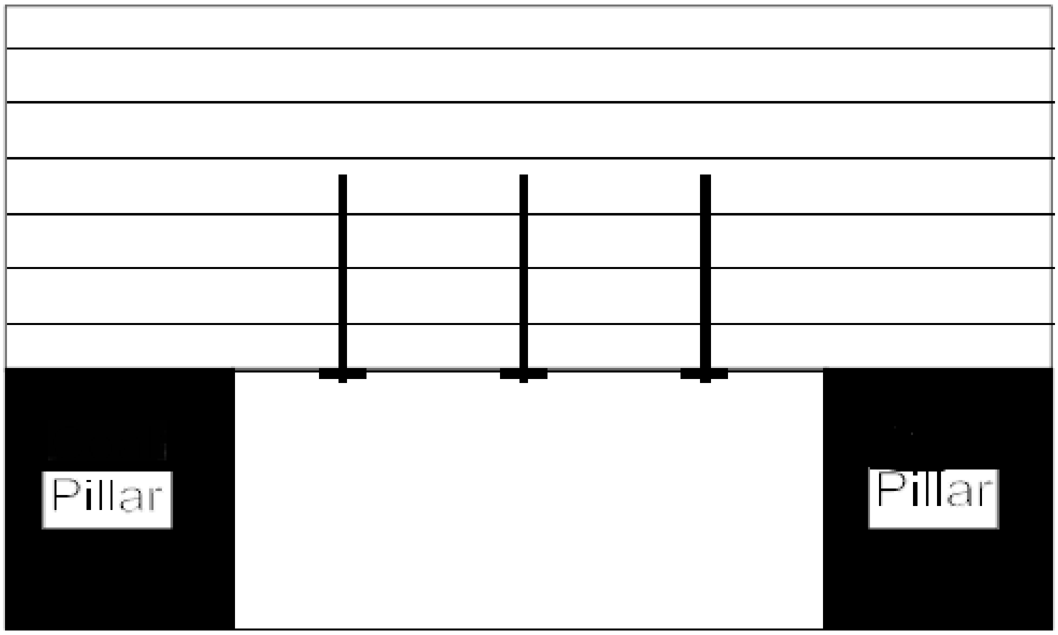

Figure 2.4: Beam building mechanism

2.8.4 Supplemental Support

Where the overlying rock is highly fractured and or in extremely high stress environment, short roofbolts may not be able to pin large wedges alone. In such environment, it is critical for the roofbolts to work in conjunction with cable bolts and cable trusses (Swart, 2005). In mines where there is highly fractured ground, such as the research area, mines are recommended to use longer cable bolts together with the designed tendon support systems following both numerical and empirical methods. The shorter roofbolts are important to prevent the unpredictable unravelling of the immediate hangingwall while cable bolts bear the dead weight of the block. Numerous joint sets increase frictional force along weak planes, separation along such discontinuities is reduced through supplementary support as shown in Figure 2.5.

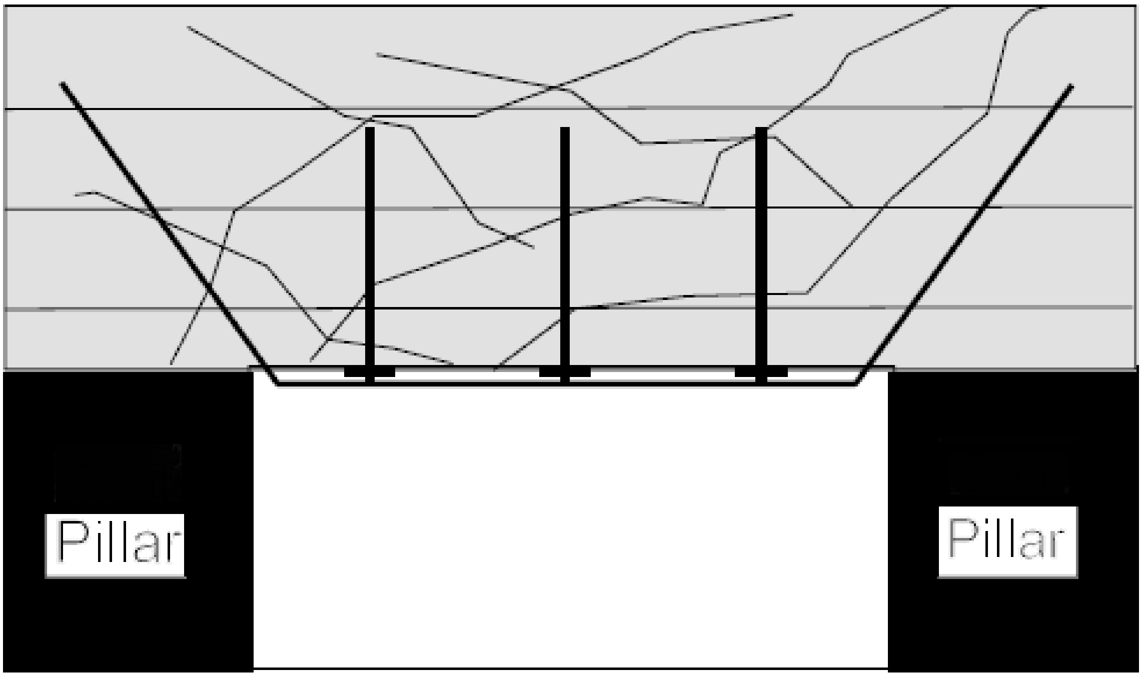

Figure 2.5: Supplemental Support

2.8.5 Fallout height

Excavations with different purposes have distinctive requirements for the support to be installed. The life of the excavation determines the support requirements. For effective support system, the tendon support should be greater than the fallout height which is considered to be the thickness of 95% of the cumulative frequency of occurrence of rockfalls, based on historical trends of past falls (Stacey and Swart, 2001). The two further pointed out that if there is insufficient data to form a cumulative frequency distribution, observations of brow thickness together with the height of wedge failures should be used to estimate the fallout height. Stacey and Gumede (2007) pointed out that in this design process, no account is taken of the actual sizes of rock blocks, slabs and wedges that might be present in the stope hangingwall (the empirical rock fall data required do take account of observed fall out thickness on a statistical basis, but not the lateral dimensions of the blocks). Therefore, the support design, based only on the expected height of rock fall, must be flawed. The author used fall out height approach for design purposes and used mapped data to conduct kinematics analysis in JBlock to determine potentially unstable bocks as well as probability of failure.

Stacey and Swart (2001) pointed out that the demand of systematic stope support systems should be intended to carry the deadweight of a potential wedge and is given by:

$$\text{Demand} = \rho t g \tag{13}$$

Where ρ is the density of the rock; thus 3 200kg/m^3 in the research area

t is the fallout height, and

g is the gravitational acceleration (assume 9.81m/s^2).

The capacity of a support unit is defined as the peak load that the support unit can carry (Stacey and Swart, 2001). In this research, the author looked at the capacities of the current roofbolts as well as the capacities of the proposed roofbolts. In case the underground performance data is not readily available, the uttermost capacity for support design should be 40% of the laboratory strengths (Stacey and Swart, 2001). The tensile strengths of the roofbolts used in this research were based on the pull tests done at one of the studied mines. Having deduced the fallout height, roofbolts need to be 20cm longer than the thickness in a bid to critically bond the parting plane or potential wedge. The bolt should also have an extra 10cm length for protrusion during tensioning. The bolts should be drilled vertically in a bid to maximise the use of the roofbolt length. After every blast, the first line of roofbolts needs to be retensioned and other roofbolts need to be retensioned on a regular basis. Shallow dipping joints, wedges, faults, dykes, recurrent joints as well as shear zones all require supplementary support (Stacey and Swart, 2001). Support should be installed near the edge of brows, faults and prominent joints and must be supported on the weaker side to clamp the geological discontinuities.

2.9 Numerical modelling

Numerical modelling is critical for optimum support design as a result of the limitations in empirical methods. Pillar failure and falls of ground on the Great Dyke need to be mitigated through the use of softwares since most designs are empirically based. Empirical methods have an inherent low accuracy as compared to numerical methods (Potvin, et al., 2012). Although empirical methods assess the stability of stope panels by use of statistical analysis, they are unable to verify the expected performance of designed excavations. Numerical modelling is the only tool to verify the expected performance of excavations designed (Swart, 2005). Software packages that simulate stress distributions and displacement discontinuities such as Phase 2, Examine 2D and MAP3D were used to deduce the behavior of rockmass after excavations had been mined. Esterhuizen and Streuders (1998) noted that unstable keyblocks of rock with the potential to fall are normally found in the hangingwall of excavations and can be delineated by geological structures. JBlock software was used to determine the existence of unstable keyblocks and deduce the probability of failure. Underground joint mapping was conducted to establish the joint orientations of the most prominent joint sets which were then used in the JBlock stability analysis.

2.10 Explosive properties

Various types of explosives were looked at and their benefits and detriments were weighed for support design purposes. Different explosives have different proportions of shock energy and gas energy (Bohanek, et al., 2013). Ammonium nitrate-fuel oil (ANFO) is a supreme explosive used in the mining industry. Its advantages include simple production, cheaper and its lack of sensitivity to mechanical impacts during mechanical loading into drill holes (Mather, 1997). ANFO has also some disadvantages which include lack of water resistance and low detonation parameters which reduce their range of use to dry blasting holes in truncated compactness rock masses (Maranda, 2011). For the ANFO being used to charge at some mines, the author found it relevant to review its literature since the effect of ANFO affects the support systems. In a bid to improve the mining practises, the author reviewed the other bulk explosives which include emulsion and watergel. The merits of bulk emulsion explosive over ANFO and packaged products include; easy transportation, handling, string charging, low gas emissions, water resistant, full coupling, increased velocity of detonation, detonator sensitivity and improved work environment (Maranda, 2011).

A study was carried out to find out the effect of ANFO on the supporting systems used in geotechnically poor grounds because of its property of generating a lot of gases, thereby widening the cracks. The preliminary review of ANFO shows that it will result in a frequent number of keyblocks which will lead to an unstable hangingwall (Bohanek, et al., 2013). In addition, the cut slice will increase due to overbreak and more bad hangings will be formed which require intense barring down. Barring down becomes a risky operation due to frequent keyblocks and also time consuming, hence failure to meet production targets.

2.10.1 Evaluation of explosive characteristics in jointed areas

ANFO is often regarded as a substantially copious gas explosive than bulk emulsions and is thought to cause extensively longer cracks thereby damaging the hangingwall further (Sellers, 2011). ANFO has a lower detonation pressure and a slower delivery of energy than other bulk explosives. The consequence of this is less expansion of the blast hole by the shock wave and leaving energy for driving crack growth and heaving of the fragments that have been created (Szendrei, et al., 2006). The extensive driving force that acts on the borehole widens the fractures. It is imperative to note that the densities of ANFO and bulk emulsion are different hence they deliver different amounts of energy at different stages during their reaction process. The stability of an excavation is not only determined by the blast induced fractures, but also by the anisotropic jointed rock mass. The formation of challenging hangingwall conditions in any given mining scenario can be changed to some degree by the correct choice of explosive type, drill holes diameter and round design. Minimum overbreak with good perimeter blasting can be achieved through smoothwall blasting (Lee, et al., 1993). Smoothwall blasting works more efficiently with bulk emulsion hence the need to compare the blasting results. The damage extent depends on the rock characterisation and the in situ geological settings. In smooth wall blasting, the final row of holes contains a lighter than normal charge and should be fired after the main charge is completed in order to limit the confinement of the holes and minimise damage back into the sidewalls. Hustrulid and Iverson (2010) pointed out that the accomplishment of smoothwall blasting pivots on sound design of blasting parameters.

High heave energy explosives have a negative impact on stability of excavations (Chikande and Zvarivadza, 2016). Chikande and Zvarivadza (2016) further pointed out that high brisance explosives are more preferred in anisotropic jointed rockmass. Sellers (2011) highlighted that the blocks of rock formed by the intersection of joints also determine the safety of the excavation in addition to the blast induced fractures. The research area is jointed, hence the author looked at the effect of explosive characteristics in geotechnically challenging ground conditions. The author also reviewed various explosives types and their impact on stability of excavations.

Sellers (2011) pin pointed the following implications of stoping overbreak:

- Increased spans with a higher probability of falls of ground between units
- Lowered support capacity of support units spaced wider than designed

- Unravelling of the rock between supports requiring regular rehabilitation
- Additional energy imparted to loose rocks during a seismic event
- Lowered support capacity due to poor installation under difficult conditions.

However, if the tunnels are blasted carefully, with minimum overbreak, there are a number of associated economic benefits.

2.10.2 Half cast factor

Half cast factor is defined as the ratio of the total visible drill barrel length in the sidewalls and hangingwall after blast and the total drilling length (Dey and Murthy, 2010). Half cast factor is vital in the determination of stoping overbreak. Stoping overbreak affect the demands of support hence the author analysed the half cast factors to determine the degree of stoping overbreak. Singh (1992) pointed out that blasting can be described as a destructive process and the effects of blast damage are deleterious to both safety and productivity. The author measured the length and number of barrels and determined the half cast factor using equation 14 as given by McKown (1984).

$$HCF = \frac{\sum_{i=1}^{n} Li}{\sum_{r=1}^{n} Lr} \tag{14}$$

Where; *HCF* = Half cast factor

Li = Post-blast drill mark length visible (m)

Lr = Pre-blast drilled length (m)

2.10.3 Powder factor

Powder factor is a relationship between how much rock is broken and how much explosive is used to break it. It can serve a variety of purposes, such as an indicator of how hard the rock is, or the cost of the explosives needed, or even as a guide to planning a shot. Powder factor can be expressed as a quantity of rock broken by a unit weight of explosives. The powder factor of the current design was evaluated in this research. The explosive ANFO is currently used at the research area due to its ease of use. However, due to the loading of the product with pneumatic Lategan loaders, there is a temptation to overfill the blastholes as it seems obvious to miners that more explosives will provide better breaking. A study undertaken to investigate the effect of changing from ANFO to Powergel 813 cartridge explosive by Sellers (2011) revealed that the prevailing poor conditions resulted from overcharging blastholes.

Jagged hangingwall conditions reflected the effect of overcharging with ANFO making it difficult for the mining personnel to position the support units correctly (Sellers, 2011). The ground conditions in the research area varies from very poor to poor, which implies that the chances of fall of ground incidents can be significantly increased due to the positioning of support on the ragged protrusions from hangingwall. Powder factor was analysed by the author in a bid to mitigate the aforementioned threats. Meeting the desired powder factor leads to enhanced safety and can significantly improve profitability and the long-term feasibility of a mine.

2.11 Conclusions

In a bid to achieve the objectives set, the author reviewed relevant literature to the research. Various rock mass classification methods were used to get an accurate picture of the rock quality. The stability of excavations in bord and pillar hard rock mining heavily depends on a comprehensive and competent pillar design and tendon support design methods. Current tendon system was reviewed to identify its limitations in geotechnically challenging ground conditions. The tendon support system must be strong enough to avoid falls of ground through wedge failures. Numerical and empirical methods were used to determine the optimum tendon support system. The research governing applicable support units within the research area was also analysed. The literature in pillar design, monitoring and cutting practices was also considered in a bid to get actual reflections on pillar design in geotechnically poor grounds. The literature governing the design of non-yielding and barrier pillars was also reviewed to determine the optimum design in the research area. Both empirical and numerical models were used in calculating pillar load for sophisticated mining layouts. Various explosives types applicable to the research area were reviewed based on their energy densities and partitioning. The brisance and shock energies of the explosives were reviewed with respect to their effect on excavation stability within the research area. The effect of overbreak was also analysed and its consequences were noted in a bid to improve the current mining practices and support systems.

CHAPTER 3: METHODOLOGY

3.0 Introduction

A comprehensive revision of the background literature was conducted as per the requirements of the research objectives. This chapter presents an indication and outline of the mode and methods inclusive of investigative, analytical and pragmatic techniques used to conduct the study. The background literature was then fused with contributions from the relevant experts in the field of study such as rock mechanics engineers on a practical level. A field study of the several areas of concern was supplemented by the contribution of field experts in order to get a system of results that would be used for analysis.

3.1 Research Criterion

The research started with an extensive literature review on geological information, geotechnical literature, underground support system, mining methods and types of explosive used. The study included the author's involvement in mining and support operations with the teams in sections working in challenging ground conditions. The study included involvement in all planning and management meetings for these sections. With the aid of qualified Geotechnicians, observations and analysis were done on the geological structure of the ore body to identify the nature and magnitude of the jointing and faulting system of the body. On analysis, the RMR, MRMR and Q values were used to classify the ground conditions encountered. An assessment was done on the magnitudes of the impacts posed by current mining practices and explosives used. In addition to the above methods, written literature pertaining to the ground control and mining practices was used to obtain data for the project. Consultations with rock engineers, geology managers, strata control officers, geotechnicians as well as overseer miners were conducted throughout the research in a bid to get further insight to address the problem. A schematic study approach for the entire research is outlined in Figure 3.1 following the proposed design of shallow hard rock mining by Swart and Hendley (2005). Reflections on support design started off by defining the aforementioned objectives outlined in Chapter 1.

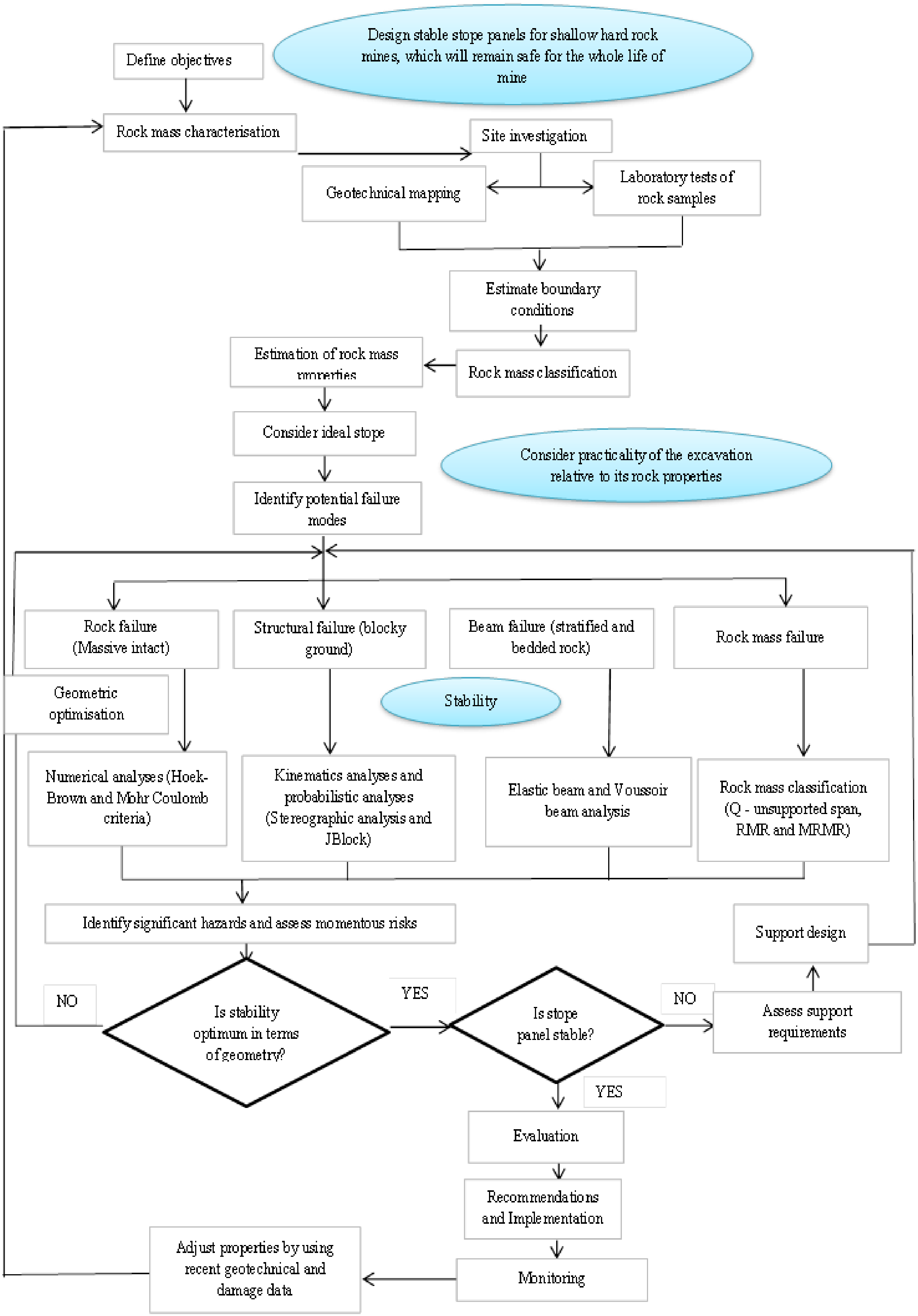

Figure 3.1: Support design methodology (Modified after Swart and Hendley, 2005)

Observations were made in each panel and the following parameters were measured and noted. In order to get a flawless understanding of the existing mining and support systems, the research was carried out in most sections with bad ground conditions. Underground observations, recordings and data collection was done on the geological structure of the ore body to identify the nature and magnitude of the jointing and faulting system of the body. An assessment was made on the magnitudes of the impacts posed by current mining practices and explosives used. Benchmarking studies with other platinum mines was also conducted. Observations were made in each section and the following parameters were measured and noted:

- Structural Data – number of joints, separation of joints, joint conditions, etc.
- Pillar dimensions from design to actual pillar cutting practice.
- Stope widths of each gulley since this have a reflective effect on ground conditions.
- Time taken to support one gulley and stand up times.
- Fallout heights.
- Type of support used thus both pillars and roofbolts and the reasons for failure.
- Ground Penetrating radar scans.
- Pillar monitoring strategies.
- Analysis of diamond drilling information.
- Use of stereonets to deduce joint sets and orientation.

The results of these observations were analysed using rock engineering principles; thus both empirical and numerical modelling were used for optimum analysis of results. A cost benefit analysis was considered in a bid to determine the viability of the proposed recommendations. Various experts in the field of rock engineering were consulted for validation of results. Lastly but not least, trials were carried out to bridge the gap between theoretical hypothesis and the practical aspects of the proposed recommendations.

3.1.1 Geotechnical data collection

The initial stage in conducting the project involved the determination of rock mass properties.

3.1.2 Geotechnical logging

This exercise comprised of the methodical collection of all fracture statistics of the rock face in the underground rock face. The data collected included joint roughness, joint sets, joint alteration, joint water, stress reduction factor and RQD. The aforementioned parameters were used in the calculation of the Q rating of the pillars.

3.1.3 Rock mass classification

The critical stage in carrying out the research was the determination of the rock mass state. Various techniques were applied to determine the ground classes in the research area. A comparative analysis on these techniques was then used to select the most effective method and substantiated recommendations of the suitable support to be installed would be based on the chosen method. The methods that were used include:

- The Q – system
- RMR
- MRMR

3.2 Analysis of performance of installed support elements

Installed support elements used at the mine include rock bolts, shotcrete and straps. These were then analyzed for their performance and effectiveness with regard to the fall out heights and the ground characteristics by means of log data and previous reports to ascertain whether failures could be attributed to them or the conditions or both. The analysis included looking at situations of failure and of likely failure.

3.3 Tendon support system

An analysis of the current support system was done through the use of structural data, GPR scans, fallout height determination as well as numerical modelling. Having reviewed the current tendon support system, the author proposed a new design which will improve both safety and productivity. A review of the tendon support system at one of the platinum mines is outlined in Chapter 4.

3.4 Investigation of the effectiveness of pillars

Evaluation of current pillar design and cutting practice within the research area was conducted. This was done on regional and non-yielding in stope pillars. The factor of safety approach was considered using both empirical and numerical modelling methods. Empirical method used include equations of pillar strength and pillar stress whilst for numerical methods, MAP3D andExamine2D were used. Since the research was conducted in geotechnically challenging ground conditions, the design rock mass strength was reviewed and updated. The current pillar design was then evaluated in line with the updated design rock mass strength. The effect of jointing on pillar strength was also considered. Underground observations of the conditions of the pillars were also made to confirm the results predicted by numerical and empirical analysis.

The pillar strength calculations were carried out through the practical measurement of pillar heights and widths. The stress acting on a selected pillar was determined by calculation. Pillars are usually shrunk from the designed size due to poor blasting and can undergo spalling attributable to bad ground conditions. The frequency of such occurrences was investigated with the view of highlighting the short term and long term problems associated with these practices.

3.5 Rock breaking

Since drilling and blasting practices affect excavation stability, an evaluation of the current practices was done. The powder factor, half cast factor and stoping overbreak for ANFO, which is currently in use, were analysed. A literature review of various explosives types amenable to poor ground conditions was conducted. This was followed up by a trial of bulk emulsion explosives presented in Chapter 6 which compares its performance with that of ANFO. A cost benefit analysis was also conducted to aid in explosives selection.

3.6 Project constraints

Constrains that the author encountered included:

- Measuring of pillars was rather dangerous as rock falls are mostly from the shoulders of pillars.
- Difficulties in logging of some ends due to water logging or delays in pumping out water delayed the progress of the project.

3.7 Conclusions

Falls of ground from unstable panels fluctuate in magnitude from rockfalls between support units to rockfalls bridging several panels and pillars. The author considered the proposed support design for shallow hard rock mining to get a reflection on the current support system that is being used in geotechnically poor ground conditions. Empirical stope design is principally used in isolation by most mines to design the required support units in the research area. The author used empirical methods in conjunction with observations and analytical techniques in a bid to formulate ideal support design attuned with the research objectives. Structural data was used to determine the stability of excavations in terms of stand up times. The prevailing faults, shear zones and sympathetic joints in the research area necessitated the execution of keyblock analysis to mitigate the release of unstable wedges. The fallout height was also determined for the purpose of designing tendon support. Pillar design, monitoring and pillar cutting practices were also incorporated in the study approach. This utilized empirical and numerical modelling techniques. Evaluation of various explosive types was also done to determine their impact on excavation stability.

CHAPTER 4: CASE STUDY 1: REVIEW OF THE TENDON SUPPORT SYSTEM

4.0 Introduction

This chapter gives an outline of a review of the tendon support system used at the case study area as per the methodology. The results obtained on rock quality and the effectiveness of support were analysed using various techniques that will be discussed later in this chapter. The results are mainly based on geotechnical data collected from the area of research. The chapter covers rock mass classification techniques, reef sub-parallel planes, factor of safety approach, fallout thickness evaluation and numerical modelling. The current support system data was included so that it could be analysed.

4.1 Tendon support units at case study area

Tendon support is used to support the roof of the panels between pillars. Different types of tendon units used at the area of research are shown in Table 4.1. The support system was derived from a suspension methodology which assumes that a beam of rock is suspended from the competent rock using bolts. Analysis of the current tendon support systems used in platinum mines on the Great Dyke will be made and weighed against the recommended support system.

Table 4.1: Different tendon support units in use at the research area

Support type	Length (m)	Diameter (mm)	Tensile strength (kN)	Pretension force (kN)
Shepherd crook	2	16	120	N/A
Resin grouted bolt	1.8	20	170	35
Cable anchors	4.5	15	250	100
Cable bolt anchors	6.5	15	250	100

4.2 Evaluation of rock mass classifications

The author conducted the research in regions with faults, sympathetic joints and collected the corresponding structural data. The rock mass was classified according to each of the three systems which are RMR, MRMR and the Q system. Comparison of the three methods was done to come up with the most suitable method of ground classification. An analysis of the structural data collected will be described later in this chapter.

4.2.1 Q system

Table 4.2 shows the data collected to calculate Q values in each bord. The calculated Q values from the study area are shown in Table 4.3.

J_x = number of joints ÷ distance (15)

Where J_x represents J_s, J_d and J_h in the table.

For 17 joints in strike direction at a distance of 11m, J_s =17/11 resulting in 1.5 joints per metre

$J_v = J_s + J_d + J_h$ (16)

When $J_s = 1.5$, $J_d = 2.3$ and $J_h = 3.5$

$J_v = 1.8+2.6+3.9 =7.3$

Using equation 3,

RQD = 115-3.3J_v

= 115 - 3.3 ×7.3

= 91%

For the calculated RQD of 91%,

J_n = 6, (from appendix B)

J_r = 1.5

J_a = 6

J_w = 1

SRF = 4

Q = 0.95 Using equation 4 (section 2.6.1)

The calculated Q value (1.01) corresponds to an RMR value of 44 based on:

RMR = 9lnQ + 44 (using equation 7)

= 44

From the gathered data, the minimum Q rating is 0.41 and the maximum Q rating is 2.78. The modal Q value is 1 and will be used for design purposes. However, variability in design will be incorporated for areas with minimum Q ratings. Such variability includes advance rates and support density. Ground control districts which require various support requirements can be drawn from the Q values obtained. The Q values obtained show that the ground conditions range from very poor to poor.

4.2.2 Rock Mass Rating

The parameters added in the determination of RMR are:

1. UCS of rock material.
2. Rock Quality Designation (RQD).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities.

The descriptions of these parameters together with their corresponding ratings are given in Appendix C. The RMR results obtained from these parameters are given in Table 4.4.

A minimum RMR value of 38 and a maximum value of 54 were recorded from the gathered data. Bieniawaski (1989) describes such rock mass class as poor to fair rock. There is a slight variation from Q rating rock mass class description. A conservative approach was adopted for design purposes by the author, therefore Q values were used for design rather than the more liberal geomechanics values. The relationship between RMR and Q is given by the following equation:

$RMR = 9\ln Q + 44$ (Brown and Hoek, 1980).

Most of the results from the calculated Q rating showed a correlation with the RMR values obtained using the Geomechanics system.

4.2.3 Mining Rock Mass Rating (MRMR)

Laubscher (1990) pointed out that the MRMR is given by the sum of the following four parameters:

- Rock material strength (UCS)
- RQD
- Joint spacing
- Joint condition and ground water

Rating values for each of these four parameters are outlined in Appendix D. Adjustments for joint conditions are given in Appendix E. Three major joint sets are encountered at the research area. The prominent joints are straight, smooth planar, unaltered and contain fine soft-sheared talc. All joints at the research area are assumed to be dry. However, the conduction of hydrological surveys is recommended. Other joints observed were straight and rough.

The calculated MRMR are shown in Table 4.5. For example, taking observations in 62nbg (north bottom gulley): Three major joint sets were observed with the following spacing: minimum = 0.1m; intermediate = 0.3m; maximum = 0.4m. The joints were straight, smooth and planar, containing fine softly sheared talc. The UCS of the host rock is 160MPa, which corresponds with a UCS rating of 16. From Q values obtained in Table 4.3, 62nbg has an RQD value of 87 with an RQD rating of 14. A dry straight joint with a smooth surface would have a minimum rating A = 75%, B = 60%, C = 100% and D = 60%; total adjustment = 0.75 x 0.6 x 1 x 0.6 = 0.27, and the joint condition and ground water rating = 0.27 x 40 = 11. The joint spacing adjustment as read from the chart in Figure 4.1 is given by 0.56 x 0.6 x 0.6 = 0.20. The joint spacing rating is therefore equal to 0.20 x 25 = 5. The MRMR in bord 62nbg is therefore equal to 16 + 14 + 11 + 5 = 46. The minimum MRMR value recorded is 43, the maximum MRMR value is 53 and the mean MRMR value is 49. A mean value of 49 will be used to calculate the desired rock mass strength.

Adjustments were applied to the MRMR value to take into account the weathering of the rockmass, joint orientation relative to the excavation, mining induced stresses and blasting effects. Blasting effect adjustment of 94% was used, Weathering adjustment of 96%, and joint orientation of 80% were used. Blasting effect adjustment of 94% was used because of good conventional blasting techniques utilized in the research area as described in Chapter 2. The rocks are slightly weathered hence a weathering adjustment of 96% was used. Three prominent joint sets defined the block in the research area with two faces inclined away from the vertical which correspond to a joint orientation of 80%. Table 4.6 shows the adjusted MRMR values for each bord mapped.

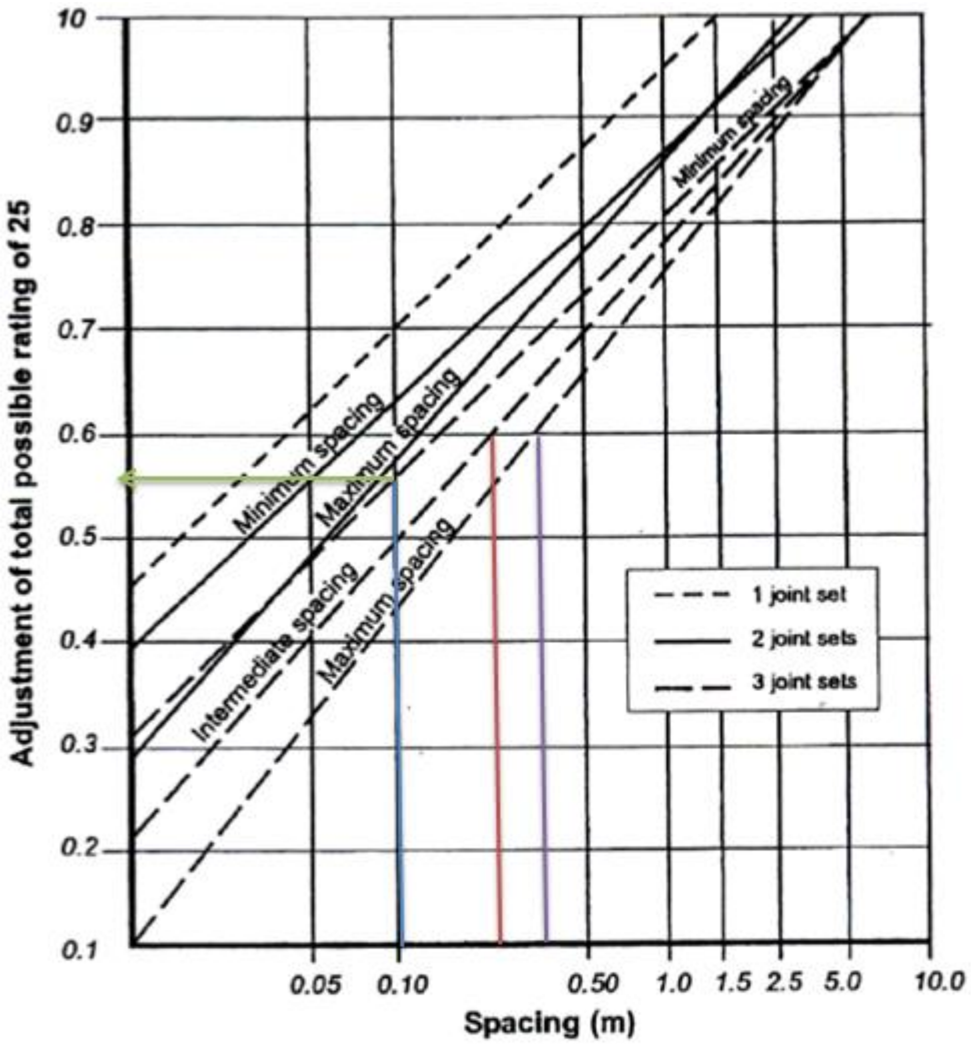

Figure 4.1: Determination of joint spacing rating (Modified after Laubscher, 1990)

The following tables show the structural data gathered by the author and the corresponding Q values, RMR and MRMR. Empirical and numerical analysis based on these results will be described later in this section. The data shown in this research represents a snapshot of the raw data collected during the research period.

Table 4.2: Rock Quality Designation

(nbg stands for north bottom gulley and ntb stands for north top gulley)

	No. of **joints in strike direction** per unit length			Max. Joint Spacing	No. of **joints in dip direction** per unit length			Max. Joint Spacing	No. of **joints in hanging wall** per unit length			Max. Joint Spacing	No. of joints per m³	Rock Quality Designation
LOCATION	**No.**	**Distance**	J_s		**No.**	**Distance**	J_d		**No.**	**Height**	J_h		J_v	**RQD**
51ntg	17	11	1.5	0.3	16.00	7.00	2.3	0.4	8.0	2.3	3.5	0.1	7.3	91
51nbg	19	8	2.4	0.4	11.00	5.00	2.2	0.3	8.0	2.4	3.3	0.2	7.9	89
52ntg	18	6	3.0	0.3	12.00	6.00	2.0	0.3	9.0	2.2	4.1	0.2	9.1	85
52nbg	21	11	1.9	0.4	13.00	6.50	2.0	0.2	8.0	2.2	3.6	0.1	7.5	90
53ntg	19	14	1.4	0.4	14.00	6.00	2.3	0.3	15.0	2.1	7.1	0.2	10.8	79
53nbg	18	10	1.8	0.30	14.00	6.00	2.3	0.20	11.0	2.3	4.8	0.2	8.9	86
54ntg	18	8	2.3	0.3	16.00	6.00	2.7	0.5	10.0	2.0	5.0	0.2	9.9	82
54nbg	24	11	2.2	0.4	13.00	5.00	2.6	0.4	12.0	2.2	5.5	0.1	10.2	81
55ntg	21	10	2.1	0.4	17.00	6.00	2.8	0.3	14.0	2.3	6.1	0.1	11.0	79
55nbg	19	9	2.1	0.35	13.00	6.00	2.2	0.30	9.0	2.4	3.8	0.2	8.0	89
56ntg	22	10	2.2	0.2	16.00	6.00	2.7	0.3	11.0	2.1	5.2	0.2	10.1	82
56nbg	23	11	2.1	0.2	12.00	6.00	2.0	0.2	9.0	2.3	3.9	0.1	8.0	89
57ntg	20	11	1.8	0.3	13.00	6.00	2.2	0.5	8.0	2.2	3.6	0.1	7.6	90
57nbg	21	13	1.6	0.4	16.00	5.00	3.2	0.4	11.0	2.1	5.2	0.2	10.1	82
58ntg	24	12	2.0	0.2	11.00	6.20	1.8	0.4	12.0	2.0	6.0	0.2	9.8	83
58nbg	24	12	2.0	0.3	11.00	6.00	1.8	0.3	12.0	2.0	6.0	0.2	9.8	83
59ntg	23	13	1.8	0.4	15.00	6.00	2.5	0.4	8.0	2.3	3.5	0.1	7.7	89
59nbg	25	15	1.7	0.3	16.00	6.00	2.7	0.4	9.0	2.2	4.1	0.2	8.4	87
60ntg	22	10	2.2	0.4	12.00	6.00	2.0	0.3	9.0	2.1	4.3	0.2	8.5	87
60nbg	23	10	2.3	0.2	14.00	6.50	2.2	0.5	13.0	2.3	5.7	0.2	10.1	82
61ntg	21	10	2.1	0.3	11.00	6.20	1.8	0.3	11.0	2.1	5.2	0.1	9.1	85

Table 4.2 : Rock Quality Designation Continuation

LOCATION	No.	Distance	J_s		No.	Distance	J_d		No.	Height	J_h		J_v	RQD
61nbg	17	7	2.4	0.3	12.00	6.00	2.0	0.3	11.0	2.2	5.0	0.1	9.4	84
62ntg	18	9	2.0	0.4	12.00	5.50	2.2	0.4	8.0	2.1	3.8	0.2	8.0	89
62nbg	21	11	1.9	0.3	13.00	6.00	2.2	0.4	9.0	2.1	4.3	0.1	8.4	87
63nbg	20	9	2.2	0.3	13.00	6.50	2.0	0.3	8.0	2.4	3.3	0.1	7.6	90
64ntg	16	7	2.3	0.3	12.00	6.20	1.9	0.3	15.0	2.2	6.8	0.1	11.0	79
64nbg	18	7	2.6	0.2	16.00	6.00	2.7	0.4	13.0	2.3	5.7	0.1	10.9	79
65ntg	18	11	1.6	0.3	16.00	7.00	2.3	0.4	11.0	2.3	4.8	0.1	8.7	86
65nbg	19	10	1.9	0.4	12.00	6.00	2.0	0.3	9.0	2.2	4.1	0.2	8.0	89
66ntg	18	10	1.8	0.3	16.00	6.00	2.7	0.3	8.0	2.3	3.5	0.2	7.9	89
66nbg	22	12	1.8	0.4	11.00	6.50	1.7	0.2	8.0	2.2	3.6	0.1	7.2	91
67ntg	19	10	1.9	0.30	11.00	6.00	1.8	0.20	9.0	2.1	4.3	0.2	8.0	89
67nbg	18	9	2.0	0.3	15.00	6.00	2.5	0.5	8.0	2.0	4.0	0.2	8.5	87
68ntg	24	13	1.8	0.4	11.00	5.00	2.2	0.4	8.0	2.0	4.0	0.1	8.0	88
68nbg	23	12	1.9	0.4	16.00	6.00	2.7	0.3	12.0	2.2	5.5	0.1	10.0	82
69ntg	20	10	2.0	0.35	12.00	6.00	2.0	0.30	8.0	2.1	3.8	0.2	7.8	89
69nbg	22	10	2.2	0.2	12.00	6.00	2.0	0.3	11.0	2.2	5.0	0.2	9.2	85
70ntg	21	12	1.8	0.2	14.00	6.00	2.3	0.2	13.0	2.2	5.9	0.1	10.0	82
70nbg	21	10	2.1	0.3	12.00	6.00	2.0	0.5	9.0	2.0	4.5	0.1	8.6	87
71ntg	23	12	1.9	0.4	16.00	5.00	3.2	0.4	12.0	2.3	5.2	0.2	10.3	81
71nbg	22	10	2.2	0.2	13.00	6.20	2.1	0.4	12.0	2.3	5.2	0.2	9.5	84
72ntg	24	12	2.0	0.3	16.00	6.00	2.7	0.3	11.0	2.2	5.0	0.2	9.7	83
72nbg	23	13	1.8	0.4	17.00	5.50	3.1	0.4	15.0	2.3	6.5	0.1	11.4	77
73ntg	25	15	1.7	0.3	12.00	6.00	2.0	0.4	10.0	2.1	4.8	0.2	8.4	87
73nbg	20	10	2.0	0.4	17.00	6.00	2.8	0.3	10.0	2.2	4.5	0.2	9.4	84

Table 4.3: Q Rating

LOCATION	Rock Quality Designation	Joint set number	Joint Roughness	Joint Alteration	Joint water	Stress Reduction Factor	Q-rating	ROCK MASS RATING
	RQD	J_n	J_r	J_a	J_w	SRF	Q	RMR
51ntg	91	6	1.5	6	1	4	0.95	44
51nbg	89	6	1	6	1	4	0.62	40
52ntg	85	6	1	6	1	4	0.59	39
52nbg	90	6	1.5	6	1	4	0.94	43
53ntg	79	6	1.5	6	1	6	0.55	39
53nbg	86	6	1.5	6	1	6	0.60	39
54ntg	82	6	1.5	2	1	4	2.56	52
54nbg	81	9	3	6	1	6	0.75	41
55ntg	79	6	1.5	6	1	7.5	0.44	37
55nbg	89	6	1.5	6	1	4	0.93	43
56ntg	82	4	3	6	1	4	2.56	52
56nbg	89	9	1.5	2	1	4	1.85	50
57ntg	90	6	1.5	2	1	6	1.88	50
57nbg	82	6	2	2	1	6	2.28	51
58ntg	83	3	3	6	1	8	1.73	49
58nbg	83	3	1.5	2	1	6	3.46	55
59ntg	89	4	3	6	1	4	2.78	53
59nbg	87	9	1.5	3	1	7.5	0.64	40
60ntg	87	9	1.5	6	1	6	0.40	36
60nbg	82	9	2	3	1	6	1.01	44
61ntg	85	9	1.5	3	1	6	0.79	42
61nbg	84	9	1.5	2	1	6	1.17	45
62ntg	89	4	3	6	1	4	2.78	53

Table 4.3: Q Rating Continuation

LOCATION	Rock Quality Designation	Joint set number	Joint Roughness	Joint Alteration	Joint water	Stress Reduction Factor	Q-rating	ROCK MASS RATING
62nbg	87	9	1.5	6	1	4	0.60	39
63nbg	90	9	2	6	1	6	0.56	39
64ntg	79	9	1.5	2	1	8	0.82	42
64nbg	79	9	1.5	2	1	6	1.10	45
65ntg	86	3	1.5	6	1	7.5	0.96	44
65nbg	89	3	1	6	1	7.5	0.66	40
66ntg	89	6	1	6	1	6	0.41	36
66nbg	91	6	1.5	6	1	4	0.95	44
67ntg	89	6	1.5	6	1	4	0.93	43
67nbg	87	6	1.5	3	1	6	1.21	46
68ntg	88	9	3	6	1	6	0.81	42
68nbg	82	3	1.5	6	1	8	0.85	43
69ntg	89	6	1.5	3	1	4	1.85	50
69nbg	85	4	3	6	1	4	2.66	53
70ntg	82	9	1.5	3	1	4	1.14	45
70nbg	87	3	1.5	6	1	4	1.81	49
71ntg	81	6	2	3	1	6	1.50	48
71nbg	84	3	3	6	1	6	2.33	52
72ntg	83	3	1.5	6	1	4	1.73	49
72nbg	77	4	3	6	1	4	2.41	52
73ntg	87	9	1.5	3	1	4	1.21	46
73nbg	84	9	9	1.5	3	1	0.78	42

Table 4.4: Rock mass rating

LOCATION	UCS	Rating	RQD	Rating	Spacing (mm)	Rating	Conditions of discontinuity	Rating	Groundwater	Rating	Adjustment for orientation	Rating	RMR	Calculated RMR from Q
51ntg	160	12	91	17	99	8	>5mm	0	completely dry	15.00	unfavourable	-10.00	42	44
51nbg	160	12	89	17	56	5	>5mm	0	completely dry	15.00	unfavourable	-10.00	39	40
52ntg	160	12	85	17	51	5	>5mm	0	completely dry	15.00	unfavourable	-10.00	39	39
52nbg	160	12	90	17	49	5	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	49	43
53ntg	160	12	79	17	48	5	>5mm	0	completely dry	15.00	unfavourable	-10.00	39	39
53nbg	160	12	86	17	59	5	>5mm	0	completely dry	15.00	unfavourable	-10.00	39	39
54ntg	160	12	82	17	79	8	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	52	52
54nbg	160	12	81	13	100	8	>5mm	0	completely dry	15.00	unfavourable	-10.00	38	41
55ntg	160	12	79	17	57	5	>5mm	0	completely dry	15.00	unfavourable	-10.00	39	37
55nbg	160	12	89	17	91	8	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	52	43
56ntg	160	12	82	17	201	10	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	54	52
56nbg	160	12	89	17	43	5	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	49	50
57ntg	160	12	90	17	203	10	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	54	50
57nbg	160	12	82	17	201	10	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	54	51
58ntg	160	12	83	17	99	8	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	52	49
58nbg	160	12	83	17	207	10	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	54	55
59ntg	160	12	89	17	202	10	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	54	53
59nbg	160	12	87	17	59	5	>5mm	0	completely dry	15.00	unfavourable	-10.00	39	40
60ntg	160	12	87	17	56	5	>5mm	0	completely dry	15.00	unfavourable	-10.00	39	36
60nbg	160	12	82	17	59	5	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	49	44
61ntg	160	12	85	17	55	5	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	49	42
61nbg	160	12	84	17	101	8	>5mm	0	completely dry	15.00	unfavourable	-10.00	42	45
62ntg	160	12	89	17	203	10	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	54	53
62nbg	160	12	87	17	102	8	>5mm	0	completely dry	15.00	unfavourable	-10.00	42	39

Table 4.4: Rock mass rating Continuation

LOCATION	UCS	Rating	RQD	Rating	Spacing (mm)	Rating	Conditions of discontinuity	Rating	Groundwater	Rating	Adjustment for orientation	Rating	RMR	Calculated RMR from Q
63nbg	160	12	90	17	100	8	>5mm	0	completely dry	15.00	unfavourable	-10.00	42	39
64ntg	160	12	79	13	56	5	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	45	42
64nbg	160	12	79	13	56	5	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	45	45
65ntg	160	12	86	17	102	8	>5mm	0	completely dry	15.00	unfavourable	-10.00	42	44
65nbg	160	12	89	17	58	5	>5mm	0	completely dry	15.00	unfavourable	-10.00	39	40
66ntg	160	12	89	17	51	5	>5mm	0	completely dry	15.00	unfavourable	-10.00	39	36
66nbg	160	12	91	17	53	5	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	49	44
67ntg	160	12	89	17	103	8	>5mm	0	completely dry	15.00	unfavourable	-10.00	42	43
67nbg	160	12	87	17	81	8	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	52	46
68ntg	160	12	88	13	100	8	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	48	42
68nbg	160	12	82	17	102	8	>5mm	0	completely dry	15.00	unfavourable	-10.00	42	43
69ntg	160	12	89	17	91	8	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	52	50
69nbg	160	12	85	17	241	10	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	54	53
70ntg	160	12	82	17	91	8	>5mm	0	completely dry	15.00	unfavourable	-10.00	42	45
70nbg	160	12	87	17	209	10	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	54	49
71ntg	160	12	81	17	211	10	>5mm	0	completely dry	15.00	unfavourable	-10.00	44	48
71nbg	160	12	84	17	121	8	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	52	52
72ntg	160	12	83	17	201	10	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	54	49
72nbg	160	12	77	17	203	10	1mm-5mm	10	completely dry	15.00	unfavourable	-10.00	54	52
73ntg	160	12	87	17	103	8	>5mm	0	completely dry	15.00	unfavourable	-10.00	42	46
73nbg	160	12	84	17	104	8	>5mm	0	completely dry	15.00	unfavourable	-10.00	42	42

Table 4.5: MRMR

LOCATION	UCS	UCS Rating	RQD	RQD Rating	Joint condition and groundwater rating	Joint spacing adjustment factor	Joint spacing rating	MRMR
51ntg	160	16	91	14	11	0.201	5	46
51nbg	160	16	89	14	11	0.258	6	47
52ntg	160	16	85	14	11	0.210	5	46
52nbg	160	16	90	14	11	0.180	4	45
53ntg	160	16	79	12	11	0.258	6	45
53nbg	160	16	86	14	11	0.180	4	45
54ntg	160	16	82	12	17	0.266	7	52
54nbg	160	16	81	12	11	0.229	6	45
55ntg	160	16	79	12	11	0.201	5	44
55nbg	160	16	89	14	17	0.258	6	53
56ntg	160	16	82	12	17	0.180	4	49
56nbg	160	16	89	14	11	0.176	4	45
57ntg	160	16	90	14	17	0.222	6	53
57nbg	160	16	82	12	17	0.270	7	52
58ntg	160	16	83	12	17	0.229	6	51
58nbg	160	16	83	12	17	0.258	6	51
59ntg	160	16	89	14	17	0.229	6	53
59nbg	160	16	87	14	11	0.258	6	47
60ntg	160	16	87	14	11	0.258	6	47
60nbg	160	16	82	12	11	0.218	5	44
61ntg	160	16	85	14	11	0.210	5	46
61nbg	160	16	84	14	11	0.210	5	46
62ntg	160	16	89	14	11	0.270	7	48
62nbg	160	16	87	14	11	0.202	5	46

Table 4.5: MRMR Continuation

Location	UCS	UCS Rating	RQD	RQD Rating	Joint condition and groundwater rating	Joint spacing adjustment factor	Joint spacing rating	MRMR
64ntg	160	16	79	12	11	0.210	5	44
64nbg	160	16	79	12	11	0.176	4	43
65ntg	160	16	86	14	11	0.201	5	46
65nbg	160	16	89	14	11	0.258	6	47
66ntg	160	16	89	14	11	0.258	6	47
66nbg	160	16	91	14	11	0.176	4	45
67ntg	160	16	89	14	11	0.180	4	45
67nbg	160	16	87	14	11	0.258	6	47
68ntg	160	16	88	14	11	0.229	6	47
68nbg	160	16	82	12	11	0.201	5	44
69ntg	160	16	89	14	11	0.258	6	47
69nbg	160	16	85	14	11	0.180	4	45
70ntg	160	16	82	12	11	0.166	4	43
70nbg	160	16	87	14	17	0.222	6	53
71ntg	160	16	81	14	11	0.270	7	48
71nbg	160	16	84	14	17	0.229	6	53
72ntg	160	16	83	12	17	0.258	6	51
72nbg	160	16	77	12	17	0.229	6	51
73ntg	160	16	87	14	11	0.258	6	47
73nbg	160	16	84	14	11	0.258	6	47
Average MRMR								**49**

Table 4.6: Adjusted MRMR

Location	MRMR	Weathering adjustment	Blasts effects adjustment	Joint orientation adjustment	Adjusted MRMR
51ntg	46	0.96	0.94	0.8	33
51nbg	47	0.96	0.94	0.8	34
52ntg	46	0.96	0.94	0.8	33
52nbg	45	0.96	0.94	0.8	33
53ntg	45	0.96	0.94	0.8	33
53nbg	45	0.96	0.94	0.8	33
54ntg	52	0.96	0.94	0.8	37
54nbg	45	0.96	0.94	0.8	32
55ntg	44	0.96	0.94	0.8	32
55nbg	53	0.96	0.94	0.8	39
56ntg	49	0.96	0.94	0.8	36
56nbg	45	0.96	0.94	0.8	33
57ntg	53	0.96	0.94	0.8	38
57nbg	52	0.96	0.94	0.8	37
58ntg	51	0.96	0.94	0.8	37
58nbg	51	0.96	0.94	0.8	37
59ntg	53	0.96	0.94	0.8	38
59nbg	47	0.96	0.94	0.8	34
60ntg	47	0.96	0.94	0.8	34
60nbg	44	0.96	0.94	0.8	32
61ntg	46	0.96	0.94	0.8	33
61nbg	46	0.96	0.94	0.8	33
62ntg	48	0.96	0.94	0.8	34
62nbg	46	0.96	0.94	0.8	33

Table 4.6: Adjusted MRMR Continuation

Location	**RMR**	**Weathering adjustment**	**Blasts effects adjustment**	**Joint orientation adjustment**	**Adjusted MRMR**
63nbg	46	0.96	0.94	0.8	33
64ntg	44	0.96	0.94	0.8	32
64nbg	43	0.96	0.94	0.8	31
65ntg	46	0.96	0.94	0.8	33
65nbg	47	0.96	0.94	0.8	34
66ntg	47	0.96	0.94	0.8	34
66nbg	45	0.96	0.94	0.8	33
67ntg	45	0.96	0.94	0.8	33
67nbg	47	0.96	0.94	0.8	34
68ntg	47	0.96	0.94	0.8	34
68nbg	44	0.96	0.94	0.8	32
69ntg	47	0.96	0.94	0.8	34
69nbg	45	0.96	0.94	0.8	33
70ntg	43	0.96	0.94	0.8	31
70nbg	53	0.96	0.94	0.8	38
71ntg	48	0.96	0.94	0.8	34
71nbg	53	0.96	0.94	0.8	38
72ntg	51	0.96	0.94	0.8	37
72nbg	51	0.96	0.94	0.8	37
73ntg	47	0.96	0.94	0.8	34
73nbg	47	0.96	0.94	0.8	34
Average adjusted MRMR					**35**

Stereographic plotting shown in Figure 4.2 illustrates a cluster of steep-angled joints and their strike directions at the research area. Few joints from the stereonets have an average shallow dip of 69^0 and a near N-S striking direction of 357^0. The most dominant joint set is the E-W joint set with an average strike direction of 086^0 and average dip angle of 73^0. This set is referred to as the J_1 joint set.

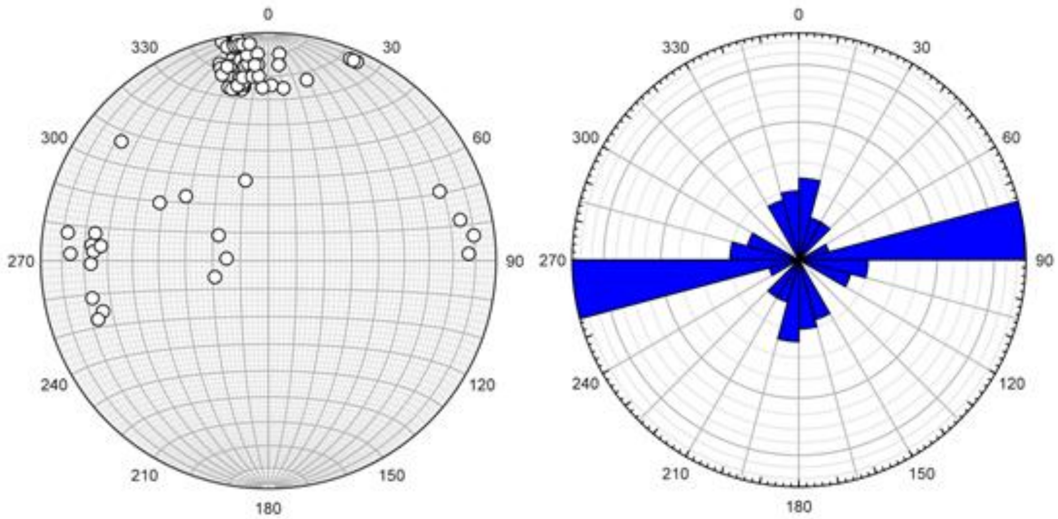

Figure 4.2: Stereographic plotting

From the mapping done (refer to Appendix F), most of the joints are dipping in the northern direction in the north sections and in the southern direction for the south sections. Of all the mapped joints, 15 had dip angle below 50^0 showing that the frequency of low angled joints is low. The J_2 joint set trends N-S with an average strike direction of 357^0 and average dip angle of 69^0 for the northern section. Most sections were characterised by a J_3 joint set (reef sub-parallel planes) with a varied thickness that ranged from 0.4m to 1.85m from the reef horizon.

4.3 Fracture surface profile

Three main types of profiles were encountered throughout the whole mapping exercise. These were planar rough, planar smooth and undulating smooth. The most dominant joint profile is the planar smooth with talc, chrysotile asbestos and serpentine as fill material. A completely planar fracture surface along the fracture plane is the one defined as planar smooth fracture profile. Shear quality description includes planar-smooth and flat shear surface. Planar smooth profile had some weak layers sandwiched in between the planes and these planes are vulnerable to failure as the filling material, talc, exhibited softness characteristics. Chrysotile asbestos material with thickness that ranged from 2-9mm also formed part of the filling material. The fibrous fill was mainly seen in northern heavily jointed area close to faulted areas.

4.4 Analysis of diamond drilling information

Diamond drilling is done underground to investigate geological complexities such as faults and deteriorating ground conditions ahead of mining. The objectives of the drilling program are to investigate the following:

- Structures, possible reef displacement and general ground conditions ahead of mining.
- Determination of the bracket pillar position around major fault areas and the competence of the ground.
- Determination of fault opening.

This will aid the mine planning process in order to optimize safe extraction of the mineral resources. Analysis of drilled holes close to an 11m fault was done in order to ascertain ground conditions for mining feasibility. Boreholes were analysed for geotechnical information. Borehole SH8N30 was drilled for 202m horizontally towards the north in 8Level north and the fracture frequencies are tabulated in Table 4.7.

The hole was collared in the footwall bronzitite and intersected the 11m fault at 36.8m. Approximately 101 joints were intersected to the 11m fault position with serpentine being the dominant fill material. The average joint spacing was 0.39m with an RQD of 75%. Beyond the 11m fault average joint spacing of 0.36m was recorded with an RQD of 71% with serpentine and talc being the dominant fill material. Five more faults were intersected within 110m spacing. The fracture frequency of the joints and intersected faults pose a challenge in mining the area beyond the 11m fault. Such geological discontinuities pose disastrous consequences when mining such regions. Because of the aforementioned geological structures in the research area, the current tendon system was reviewed to get a reflection on ground stability. Analysis was done based on the gathered structural data and current support system used at the mine. The implications of the collected results were analysed and recommendations were made in order to achieve the research objectives.

Table 4.7: Underground diamond drilling

Distance (m)	Number of joints	Fracture Frequency	Average joint spacing
0-10	37	3.7	0.27
10-20	30	3	0.33
20-30	21	2.1	0.48
30-40	20	2	0.50
40-50	24	2.4	0.42
50-60	24	2.4	0.42
60-70	24	2.4	0.42
70-80	38	3.8	0.26
80-90	23	2.3	0.43
90-100	21	2.1	0.48
100-110	26	2.6	0.38
110-120	19	1.9	0.53
120-130	40	4	0.25
130-140	30	3	0.33
140-150	33	3.3	0.30
150-160	44	4.4	0.23
160-170	23	2.3	0.43
170-180	29	2.9	0.34
180-190	36	3.6	0.28
190-202	37	3.7	0.27

4.5 Structural data analysis

Based on the Q results obtained, Figure 4.3 indicates that the rock conditions are very poor to poor. Most platinum mines on the Great Dyke use only one system for rock mass classification, the Q rating system. The author used RMR, MRMR and Q rating systems so as to have a clear picture of the rock quality. The values of the measured RMR were comparable to the calculated values from Q ratings. It can be deduced that the ground conditions in the area of research pose a risk due to high probability of potentially unstable blocks. The author recommends the mines to use more systems for rock mass classification since one system can not give a clear indication of the rock quality due to inherent limitations. Having noted that the rock is of low quality, pillar dimensioms should be adequate to support the area and the tendon systems used must clamp unstable blocks with minimum to no chances of support failure. The mine is currently using 1.8m bolts spaced at a grid spacing of 1m by 1m. Barton’s Q chart in Figure 4.3 was used to determine the appropriate support system for geotechnically challenging grounds. An average Q value of 1 was used to deduce the tendon length and grid spacing. This value was used because it is the most frequent Q rating from the gathered data.

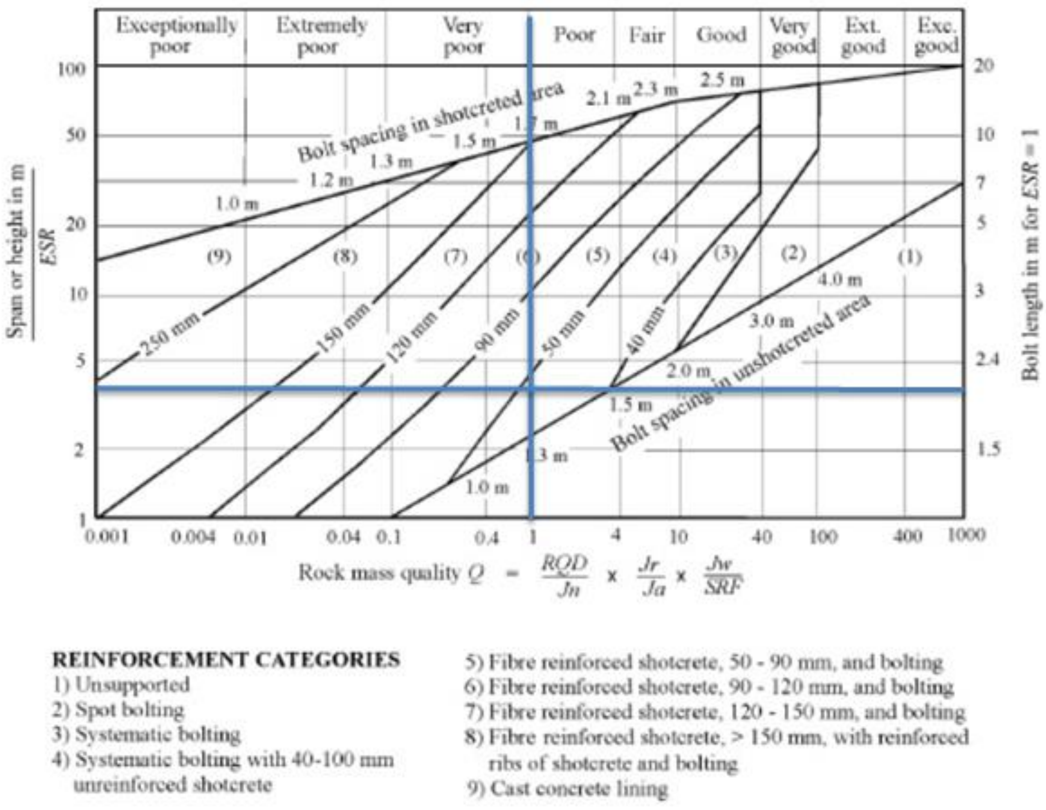

Figure 4.3: Barton's Q chart

Based on this chart, the current support system used at the mine seems to be adequate. However, other systems need to be considered too for designing the required support parameters. Using a span of 6m and an ESR value of 1.6, the Span/ESR ratio = 6/1.6 = 3.75. The fallout height was also determined in the research area so as to optimise support design. The issue of reef sub-parallel planes and numerical modelling will be discussed later in this chapter since they affect support design parameters. All joints at the case study mine are assumed to be dry; however, the author recommends the use of hydrological surveys at the research area because wrong assumptions will result in systematic errors. Ground water is known to reduce the normal force acting on discontinuities planes, hence weakening the rock. From the results of rock mass classification, it can be concluded that adequate support is required for safe mining practices.

4.6 Fallout thickness results

The fallout thickness is the most critical component in deducing the potential height of wedge instability. The thickness is used to define the support resistance as well as the energy absorption standard. Stacey and Swart (2001) noted that the criterion used to deduce support resistance depends on the tributary area theory. The weight of a rock mass given an area in the plane of the reef and the potential height of fall divided between defined support elements give support resistance. The support resistance is directly proportional to the fallout thickness as shown in Equation 13.

Support resistance = F/A

Where: F = load carried by support unit (N)

A = tributary area (m^2)

The fallout thickness was calculated based on insitu measurements and observations of wedges dislodged during rockfalls. Some of these rockfalls occurred in supported regions while others occurred in unsupported regions for example soon after blast, thus an area between permanent support and the face. Geological weaknesses such as faults and bedding planes have a significant impact on the fallout height. Stacey and Swart (2001) noted the fallout height (t) must be greater than or equal to the thickness of 95% of the cumulative percentages of occurrence of falls of ground. The fallout thickness should be determined from observations of past rockfalls. The information from the databases was outdated hence could not be used for the estimation of t since the ground conditions have been deteriorating. The author thus made some observations in areas with geotechnically poor ground conditions and measured the height of wedge failures and brow thickness as shown in Table 4.8.

Table 4.8: Fallout thickness raw data

Height(m)	0	0.2	0.4	0.6	0.8	1	1.2	1.4	1.6	1.8	2	2.2	2.4	2.6	2.8
Occurrence	0	1	2	6	8	17	11	8	7	4	2	0	0	1	0
Cumulative %	0	1	4	13	25	51	67	79	90	96	99	99	99	100	100

It is important to note that 96% of the cumulative occurrence was used to determine the support resistance in a bid to be conservative as it gives a slightly longer fallout thickness. The fallout height was found to be 1.8m (see Figure 4.4) and is in line with the average thicknesses determined from numerical modelling and GPR scans. It was also noted that fallout heights currently in use were obtained back in 2005 when mines were still operating in good ground conditions. It is thus of paramount importance for mines to have updated databases showing the current information, in particular geotechnically challenging areas so that tendon support can be continuously reviewed.

Destructive pull tests are carried out by the Rock Engineering Department on rock bolts through the rock bolt suppliers on a quarterly basis to ensure that the products supplied meet the requirements. However, non-destructive monitoring is preferred. Encapsulation pull tests were carried out to determine the critical bond length. Stacey and Swart (2001) noted that the bond force is a product of bond strength and bond area.

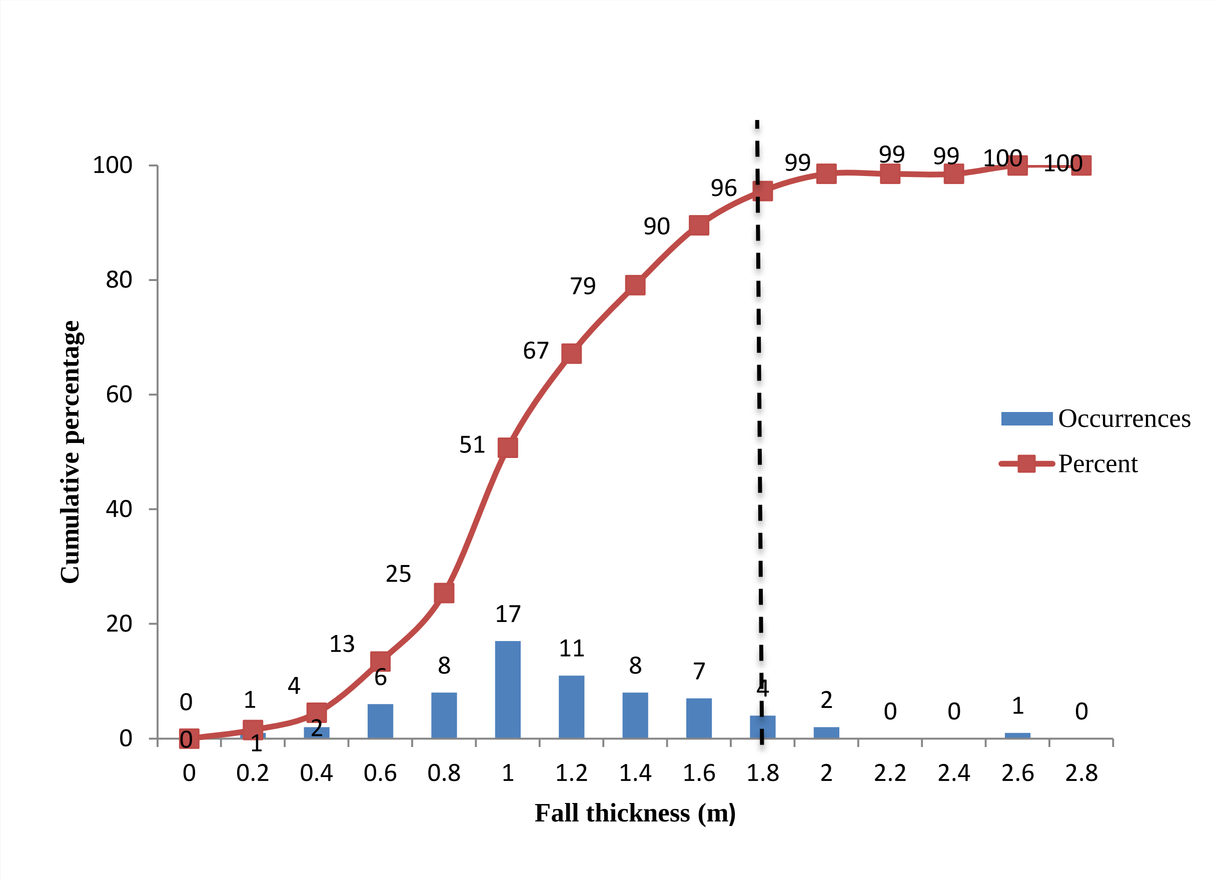

Figure 4.4: Fallout height determination

The rockbolts must be at least 200mm longer than the fallout thickness in order to anchor above the parting plane or potential weakness (Stacey and Swart, 2001). Rockbolt holes must be drilled vertically in a systematic support system to maximise use of the bolt length. In addition, a 100mm protruding length should be incorporated resulting in tendon length greater than the thickness by 300mm. From the determined fallout thickness, the minimum tendon length should be 2.1m (1.8m + 0.3m). The required length of rockbolts is usually a function of the dimensions of the opening. As a rule of thumb, for reasonable rock conditions, the length of bolts should be 0.33 times the span or the wall height (Stacey and Swart, 2001). For a span of 6m, at least 2m long roofbolts should be used plus 200mm critical bond resulting in a total roofbolt length of 2.2m. From these two methods, a good support system would be one with the 2.2m long roofbolts.

This is shown in the following calculations. Density of pyroxenite hangingwall material is 3200 kg/m^3. The current primary support consists of 15tonne strength tendons giving 147.15kN (given by 15tonne x 9.81ms^{-2}). The bolts are spaced 1m x 1m giving a total area of 1m^2 per bolt.

Considering the current 1.8m Videx Resin Bolt used at the mine,

Support thickness = 1.5m (excluding 0.20m critical bond length and 0.1m stick out)

Support Demand = density × gravity × fall-out height

= 1.5m ×3 200kg/m^3 ×9.81ms^{-2}

= 47.09kN/m^2

Support resistance = Tensile strength / Tributary area

= 147.15kN/1m^2

= 147.15kNm^{-2}

This suggests that the FOS of the bolts is given by:

FOS of the 1.8m Bolt = Strength/Demand

= 147.15/47.09

= **3.12**

When the support system at the mine was designed and the fallout height was 1.5m, the 1.8m rock bolts were adequate as reviewed by the calculation of the FOS. However since now the fallout height has been determined to be 1.8m, the current 1.8m rock bolt has no critical bonding length and is thus a waste as it provides no localised support as anticipated.

Considering 2.2m long Videx Resin Bolts (at 1.0m x 1.0m grid spacing) with a 20mm diameter with strength of 147kN, currently being rolled out following a successful trial, the resistance offered by the support system is:

Support resistance = 147.15kN/1m^2

= 147.15kNm^{-2}

Rock density = 3 200kg/m^3

Area per bolt = 1.0 m^2 (from the grid spacing 1.0m x 1.0m)

Support thickness = 1.8m (excluding 0.30m critical bond length and 0.1m stick out)

This therefore suggests that the support demand per bolt is given by

Support Demand = $1.8m \times 3\ 200kg/m^3 \times 9.81ms^{-2}$

= $56.51kN/m^2$

FOS of the 2.2m Bolt = Strength/Demand

= 147.15/56.51kN

= **2.60**

The 2.2m Videx resin bolt provides a safety factor of 2.60 which is acceptable. Stacey and Swart (2001) in their calculations on rock bolt support pointed out that FOS as low as 1.56 are acceptable. The optimum grid spacing of the proposed tendon support will be discussed later in this chapter after using numerical modelling.

Closer spacing of tendon support units was advocated in the research area as a result of the geological discontinuities. Currently used roofbolts of 1.8m length are considered inadequate to suspend the fallout height since there will be no bond length. Although increasing support density mitigates localised FOGs, in this case it is inadequate as the challenge is to clamp the overlaying layers. The use of shorter roofbolts has resulted in support failure hence new systems of tendons need to be designed.

4.7 Determination of maximum unsupported span

According to Barton et al (1974), the maximum unsupported span can be estimated from the following equation:

Max span (unsupported) = $2 \times ESR\ Q^{0.4}$

ESR is a value that is assigned to an excavation in terms of the degree of security that is demanded of the installed support system to maintain the stability of the excavation. The bords mined in the research area are permanent mine openings hence an ESR value of 1.6 must be theoretically assigned. The maximum unsupported span for Q rating = 1 is therefore equal to 3.2m. Based on engineering judgement, such spans cannot accommodate the current mining fleet hence an ESR value of 3 was used instead and the stability of such excavations was evaluated and proved to be satisfactory. Sellers (2011) also noted that other mines consider mine tunnels as temporary access ways. Hutchinson and Diederichs (1996) suggest that an ESR of not more than 3 be used for temporary mine openings. Using an ESR value of 3, the maximum unsupported span is given by:

Max span (unsupported) = 2 × ESR $Q^{0.4}$

$= 2 \times 3 \times 1^{0.4}$

$= 6m$

The maximum unsupported span of 6m is in agreement with the one currently used at the mine. The maximum unsupported span value of 3.2 will be limited to advances in order to reduce the blasted area. The minimum Q value of 0.4 will limit the maximum advance to 2.2m. When mining operations are carried out in very poor ground conditions, the maximum advance must be reduced to 2m with a corresponding increase in support density. Figure 4.5 illustrates the relationship between maximum unsupported span and the corresponding RMR and Q ratings.

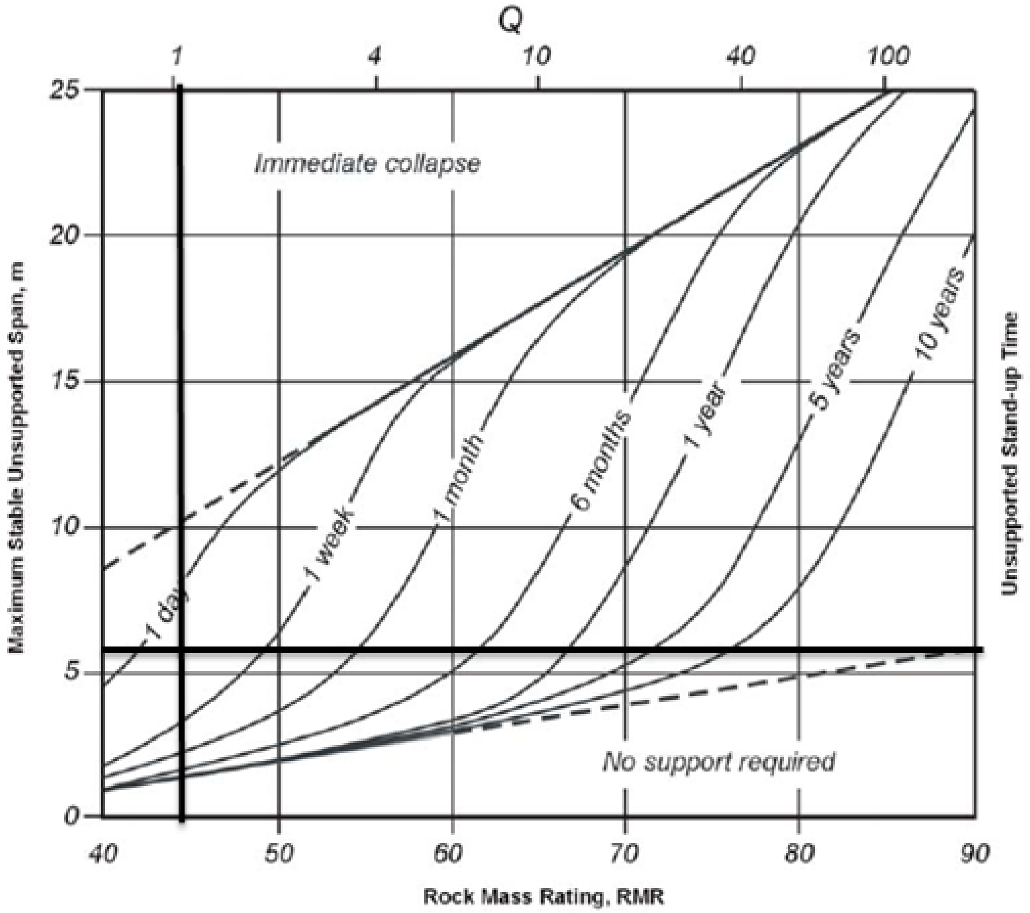

Figure 4.5: Stand up time of a 6m span

Comparing Figure 4.5 based on RMR system and Figure 4.6, a 6m span and an RMR of 44 (corresponding to an average Q of 1), results in 2 days stand up time, which corresponds to a factor of safety of about 0.9. The rock mass quality and excavation span plotted produced a factor of safety < 1 which implies that support of the excavation needs to be considered. The mine consists of 12 teams with each team required to blast three (6m) spans per shift. The studied mine is running on two shifts per day so a 2 day stand up time is more than enough for the team to support the blasted ends based on the mining cycle. Stacey and Swart (2001) pointed out that caving or collapse will only occur when the factor of safety drops below 0.8, which corresponds with an unsupported span of about 10m. The mined span at the research area will not result in caving since it is greater than 0.8.

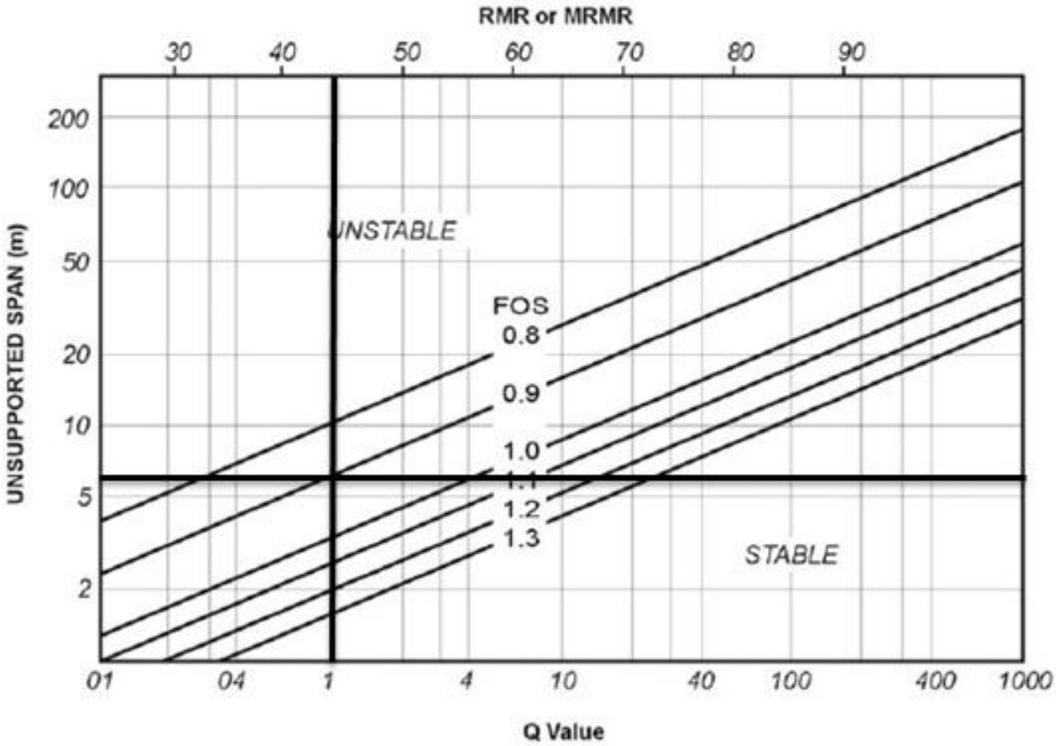

Figure 4.6: Unsupported span vs Q relationship

4.8 Wide spans

The possibility of rock falls occurring in excavations is increased when tunnels are wider than usual, such as in breakaways, or intersections. Lower Q values are obtained at intersections because the joint set number used is given by 3 x J_n. Intersections must be sited after a geotechnical assessment. Support requirements for wide spans will differ from those for normal spans. In addition to the primary support of rock bolts, full column grouted cable anchors must also be installed.

4.9 Numerical modelling analysis

JBlock was used to deduce the probability of failure for the keyblocks in each gulley. The author entered the structural data of the three joint sets shown in Table 4.2 and Figure 4.10 to simulate the potentially unstable keyblocks. The author started off by simulating unstable key blocks using the current support system of 1.8m roofbolts spaced at 1m by 1m. Figure 4.7 shows the 6m area simulated to identify unstable keyblocks in order to check the effectiveness of the current support.

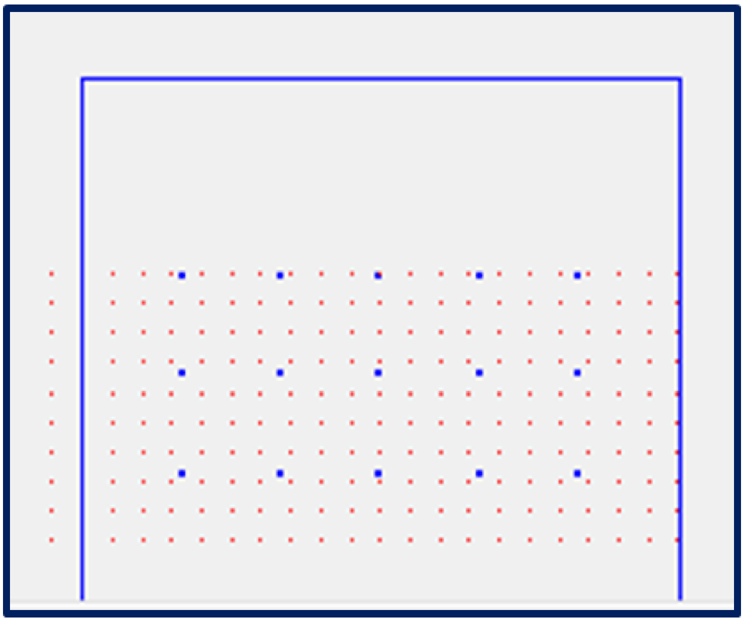

Figure 4.7: Area simulated to determine unstable blocks

Figure 4.8 shows the probability at which various blocks fail using the current support system for the case study area which constitutes of resin bolts (147kN) 1.8m length spaced at 1m by 1m in bad ground conditions. From the bar charts, it can be seen that the probability of block failure for 1m^3 blocks between support elements is 30% and the maximum probability of support failure is 16.3%. From these results, the probability of both block failure and support failure is too high. In agreement with the other systems, the current support system used in the case study area is inadequate. Integrating the results from all the techniques used, longer roofbolts are required to give an improved safety. From numerical modelling, the probability of block failure is shown together with block frequency, which can help planners and engineers to achieve zero harm.

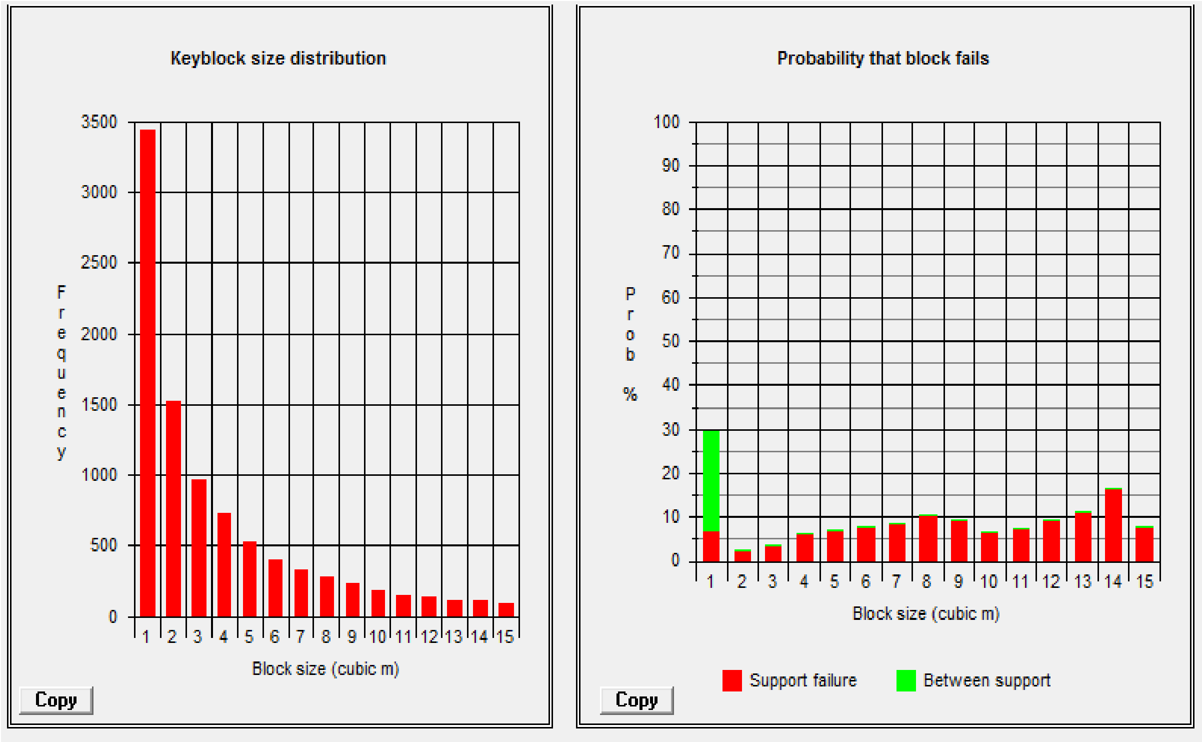

Figure 4.8: Probability that block fails using the current support system

The new support system designed and simulated by the author constitutes of grouted tendons with a capacity of 147kN, 2.2m length spaced at 1.2m by 1.2m. The proposed length follows an extensive structural data analysis, fallout height, GPR scans as well as numerical modelling. All these systems were integrated for optimum support design. The new support system suggested by the author gave results shown in Figure 4.9. Taking into account a cost benefit analysis, comparison between the current support system and the new support system was made to improve safety and productivity.

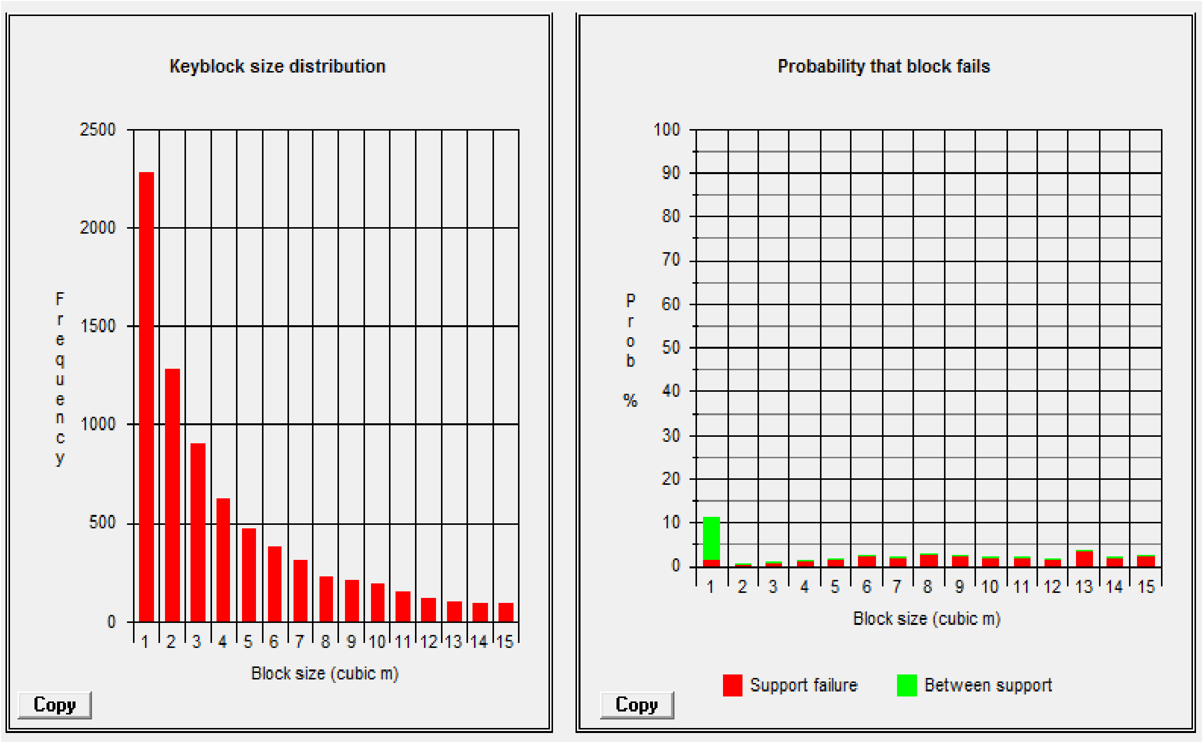

Figure 4.9: Probability that block fails using the new support system

From Figure 4.8, the probability of block failure for $1m^3$ blocks decreased to 10.9% and the maximum support failure is 4.1%. From the results, the probability of both block failure and support failure decreased, hence providing improved safety. Each designed tendon of 2.2m is more expensive than the current 1.8m, however taking into consideration the cost benefit analysis of the new system presented in Table 4.9, it can be concluded that the recommended system gives more benefits than the current support system. The designed support systems give an improved safety and a decrease in support density since the tendons will be spaced at 1.2m by 1.2m as opposed to the current roofbolts which are spaced at 1m by 1m. It is thus recommended to implement the new support system to avoid lost time injuries or even fatalities.

Using the support demand and resistance concept at 1.2m grid spacing, the 2.2m Videx Resin Bolt will give a support resistance of:

Support resistance = $147.15kN/1.44m^2$

$= 102.18kNm^{-2}$

Rock density $= 3\,200kg/m^3$

Area per bolt = $1.44m^2$ (tributary area theory at a grid spacing of 1.2m x 1.2m)

Support thickness = 1.8m (excluding 0.30m critical bond length and 0.1m stick out)

This therefore suggests that the support demand per bolt is given by:

Support Demand = $1.8m \times 3\ 200kg/m^3 \times 9.81ms^{-2}$

= $56.51kN/m^2$

FOS of the 2.2m Bolt = Strength/Demand

= 102.18/56.51kN

=**1.8**

The 2.2m Videx resin bolt provides a safety factor of 1.8 at 1.2m grid spacing, which, although lower than that for the same resin bolt at a spacing of 1.0m, is still acceptable. The recommended tendon support cannot be installed at 90^0 due to the limitations in the current mining height of 2m against a length of 2.2m for the bolt and also due to the feed arrangement of the bolter. Thus it can be installed at an angle greater than 70^0 and still effectively suspend the fallout thickness.

Geotechnically challenging ground conditions require full column grouted resin bolts. Currently, the mine is using one fast setting and 2 slow setting resin capsules of a length of 0.5m each. Each box contains 8 fast setting and 16 slow setting resin capsules costing $24.04. Each capsule costs approximately $1.00. The current tendon system requires 3 capsules per hole and 18 bolts per face. The recommended support has an increase in bolt length which entails an additional slow setting resin capsule. Therefore, each hole requires 4 capsules and each bord require 13holes. A cost benefit analysis is shown in Table 4.9.

Table 4.9: Cost benefit analysis

	Current	**Recommended**
Roofbolt length	1.8m	2.2m
Grid spacing	1m by 1m	1.2m by 1.2m
Advance/blast	3	3
Face length (m)	6	6
Bolts/ face	18	13
Number of capsules/face	54	52
Faces/ shift/team	3	3
Teams	12	12
Shifts/day	2	2
Days/ month	28	28
No. of months	12	12
Cost/bolt ($)	7.51	7.63
Cost/capsule	1	1
Roofbolts Cost/ year ($)	3 270 274.56	2 399 604.48
Cost of capsules per annum	1 306 368.00	1 257 984.00
Total support cost	4 576 642.56	3 657 588.48
Cost saved per annum ($)	**919 054.08**	

N.B: Roofbolts cost / year = bolts/ face × faces/shift/team × no. of teams × shifts/day × days/month × no. of months/year × cost/bolt

The proposed support system will give improved safety as shown by the decrease in probability of failure from the simulation carried out. There is also improved safety as indicated by the other methods used in support design. Apart from improved safety, the proposed support system will reduce costs by $919 054 per year. This implies that the key performance indicators of safety and costs will be optimised. The mines are therefore recommended to use various approaches for optimum support design.

4.10 Ground penetrating radar (GPR) analysis

The researcher took a slice showing reef sub-parallel planes shown in Figure 4.10. The process involved the use of a GPR machine which emits waves into the hangingwall. When the waves interfere a different layer, the velocity will change hence showing a distinction of layers. The information was taken to a computer where a slice such as the one shown in Figure 4.10 was produced. From this slice, it can be noted that shallow dipping planes occur at a depth greater than 1.8m into the hangingwall. Looking at the current support system of 1.8m length, it can be seen that it is not adequate enough to clamp these layers hence a new system of longer roofbolts need to be considered. The use of shorter roofbolts can results in support failure which will lead to injuries, fatalities, equipment damage, excavations closures (including high grade sections) etcetera. All these factors translate to hefty costs to the mines, hence it is of paramount importance to improve safety and productivity through the implementation of the new support system. The author therefore recommends the use of 2.2m long roofbolts having a grid spacing of 1.2m in addition to the cable bolts used. Failure to manage reef subparallel planes will result in massive falls of ground. It is crucial for the mines to have the best support design in areas with such structures. Data from these structures together with J_1 and J_2 joint sets was used by the author to recommend a new support system using JBlock.

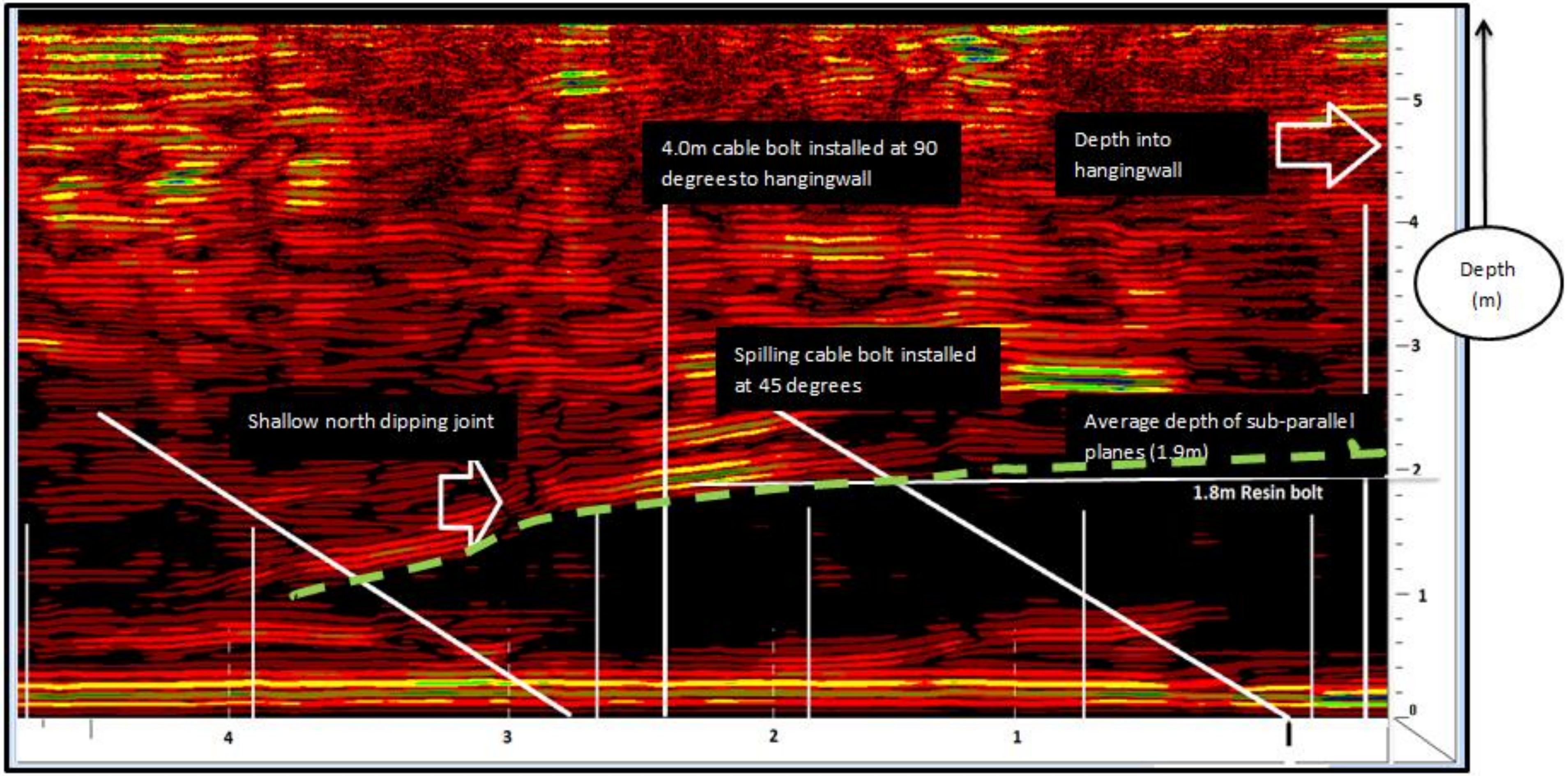

Figure 4.10: J3 structures

4.11 Risk evaluation model

In design and application of rock engineering designs, there is inherent uncertainty and risk. This is due to the variability in rock strengths within the same rock mass with the presence and distribution of multiple discontinuities of varying strengths and shear strengths. The characteristics of these discontinuities and shear strengths are also time dependent. This is also added to the fact that mining produces imperfect excavations from those designed as geotechnical conditions and compliance to standards vary. Since support standards are complied with to varying degrees, the designed support system may deviate from its intended performance. All these need to be taken into consideration in the design of mine support systems. Risk evaluation models are available to quantify expected injuries and economic loss stemming from falls of ground associated with support systems (pillars and tendon support) in various geotechnical environments (Joughin, et al., 2012). In line with this study, this enables comparison of the various support systems discussed based on their merits or demerits on expected injury frequency and overall cost of the support system including the expected economic losses that may arise in case of its failure. The models consider variability in joint orientation and strength within the geotechnical conditions and the various types of support systems installed or to be installed and their quality (Joughin, et al., 2012). Any benefits derived from risk mitigating measures taken such as barring and monitoring are incorporated as well.

4.12 Conclusions

The calculated RMR, Q and MRMR show that the ground conditions where the research was carried out is weak, hence adequate support system is required to prevent falls of ground through wedge failure. The fallout height thickness was found to be 1.8m. This showed a correlation with the results from GPR scans which indicated that overlying layers occur at a depth of 1.8m into the hangingwall. From the evaluation of the tendon systems used at the research area, the author concluded that the current tendon system used is not adequate to meet the desired support demand. A new designed system that can improve safety and productivity was proposed following a comprehensive empirical and numerical analysis. The directions and dip angles of joints were also measured and used to simulate the potentially unstable blocks. JBlock software was used to deduce potentially unstable blocks and the probability of wedge failure. The proposed tendon system is anticipated to reduce costs by $919 054 per year with further benefits of improved safety. It is recommended that the approach applied to the studied mine be adopted by the other platinum mines within the same geological location, mining the same type of deposits using the same mining method. Implementing such techniques will ensure stable excavations for all the platinum mines, hence long lasting benefits to the companies as a result of reduced falls of grounds as well as reduced costs.

CHAPTER 5: CASE STUDY 2: EVALUATION OF PILLAR DESIGN AND MINING PRACTICES

5.0 Introduction

Rock related risks due to the described geological weaknesses are reduced by sound pillar design and the implementation of adequate tendon support system. Having analysed the roofbolts system at one of the platinum mines in Chapter 4, the author evaluated the pillar support system in bad ground conditions. The author looked at both stability pillars of 10m by 3m and also in stope pillars of 3m by 3m as shown in the mining layout of poor grounds (Figure 1.6). The mine is a shallow underground mine and equations 7, 8, 10 and 12 were used in determining the factor of safety. The author also measured the main pillar dimensions and also calculated the pillar infringements which will later be analysed as they affect productivity. The author measured the actual pillar dimensions of in stope pillars and then calculated the factor of safety in the respective gullies. The desired mining height at the mine is 2m but due to overbreak, the author measured the actual pillar height in order to calculate the actual factor of safety.

The author also looked at barrier pillars that compartmentalise the mine workings as well as cutting practices. Lastly, pillars along the access decline which are scaling were reviewed and analysed so as to ascertain the integrity of the main access. The overall stability of the in-stope pillars is a function of pillar strength and the load acting on the pillars. The stresses acting on the pillars were estimated from elastic numerical modelling. The strength of a pillar depends primarily on the shape and size of the pillar, rock mass strength of the pillar material (ore body) and existence of geological structures such as the faults, major joints, weakness bands and shear zones.

5.1 Actual safety factors for insitu pillars

Table 5.1 shows the actual safety factors of 3m by 3m insitu pillars. The effective width was determined using equation 8. For bord 61,

$$effective\ width = \frac{(4 \times 3.1 \times 3)}{2 \times (3.1 + 3)}$$

$$= 3.05\text{m}$$

The mine is currently using a K value of 63 MPa to determine pillar strength for the whole mine in both good and poor ground conditions. The author reviewed this value since this value was deduced when the operations were carried out in good ground conditions during the early stages of mining. The author thus incorporated the gathered structural data for optimum designed rock mass strength through Laubscher's MRMR classification system. The pillar strength was calculated using equation 7 thus,

$$\text{Pillar Strength} = \frac{63 \times (3.05)^{0.5}}{(2.1)^{0.75}}$$

$$= 63.1\text{MPa}$$

Since the dip of the orebody is shallow, the effect of shear stress is negligible. Average pillar stress was calculated using equation 1 and equation 10:

$$Average\ pillar\ stress = \frac{\sigma_v}{(1-e)}$$

$\sigma_v = \rho\ g\ h$

Where,

Density ρ, of the overburden is 3200 kg/m^{3,}

g is acceleration due to gravity = 9.81ms^{-2}

h is the height below the surface.

For a depth of 130m in bord 61 (shown in Table 5.1):

The vertical stress, σ_v, is calculated to be 3200 kg/m^3 × 9.81 ms^{-2} ×130 m = 4 080 6960 Pa or 4.08 MPa.

Extraction ratio, e, = $\frac{(a+c)\times(b+c)-(ab)}{(a+c)\times(b+c)}$ (17)

The parametes a, b and c are shown in diagramatic form in Figure 5.1. Equation 12 was given out by Brady and Brown (1992) to explain the calculation of e (a = b for square pillars and a ≠ b for rectangular pillars).

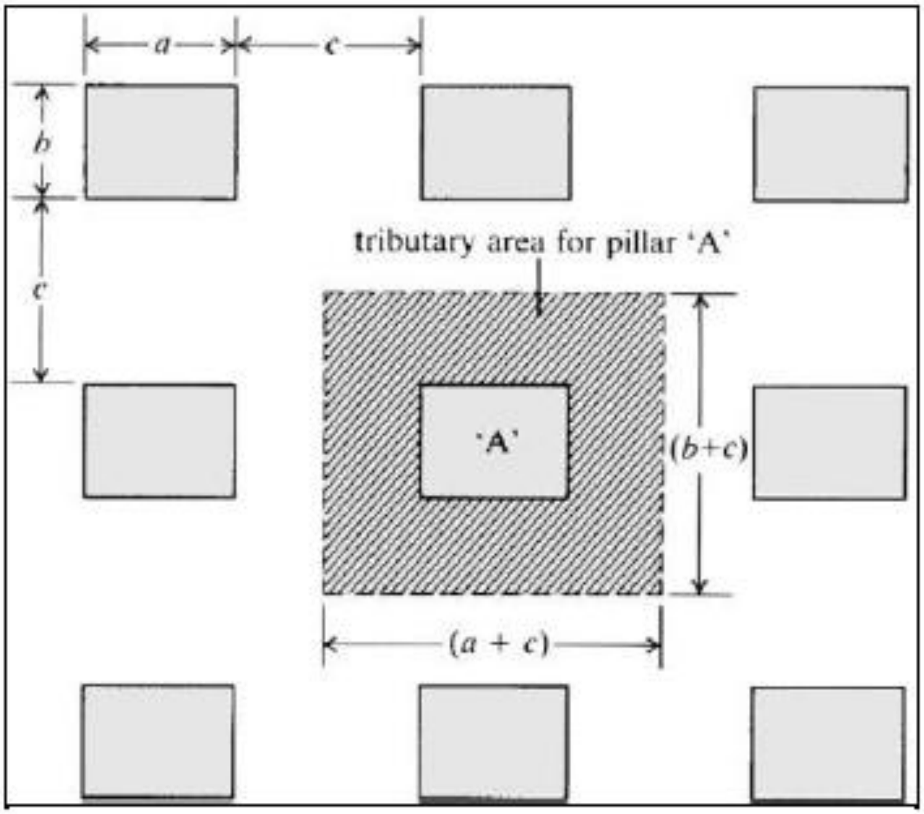

Figure 5.1: Plan view showing the tributary area analysis of pillars (Zvarivadza (2012) citing Brady and Brown, 1992)

Extraction ratio for example pillars in bord 61 is calculated to be

[(3.1+6) × (3+6) – (3.1×3)] ÷ [(3.1+6) × (6+3)] = 0.886.

This gives an average pillar stress of 4.08 / (1 – 0.886) = 36.1 MPa.

Using equation 12, the calculated Factor of Safety for this pillar layout is 63.1÷36.1 = 1.75. The design criterion at the research area requires a minimum Factor of Safety of 1.6. Table 5.1 shows a sample of the actual insitu pillar dimensions measured and their corresponding safety factors.

Table 5.1: Actual insitu pillar dimensions and safety of factor

Bord number	Average depth (m)	Average virgin Stress (MPa)	Pillar dimensions (m)			effective pillar width (m)	Percentage extraction	pillar strength (MPa)	pillar stress (MPa)	safety factor
			pillar length	pillar width	pillar height					
61	130	4.10	3.1	3.0	2.10	3.05	88.6	63.1	36.1	1.75
62	130	4.10	3.0	2.8	2.30	2.90	89.4	57.4	38.7	1.49
63	130	4.10	3.1	3.0	2.00	3.05	88.6	65.4	36.1	1.81
64	132	4.16	3.0	3.0	2.30	3.00	88.9	58.4	37.5	1.56
65	132	4.16	3.0	2.8	2.20	2.90	89.4	59.4	39.3	1.51
66	134	4.23	2.8	2.8	2.30	2.80	89.9	56.4	41.7	1.35
67	134	4.23	3.0	2.9	2.20	2.95	89.1	59.9	38.9	1.54
68	134	4.23	3.1	3.0	2.10	3.05	88.6	63.1	37.2	1.69
69	136	4.29	3.1	3.0	2.10	3.02	88.8	62.8	38.2	1.64
70	136	4.29	2.9	2.9	2.20	2.90	89.4	59.4	40.4	1.47

Pillar robbing and the effects of overbreak on safety factor will be discussed later in this section. Figure 5.2 is a representation of the data in Table 5.1, which shows a graph of factor of safety against insitu pillars in each bord from 61 to 70. The graph gives a reflection on the actual cutting practices against the desired practice.

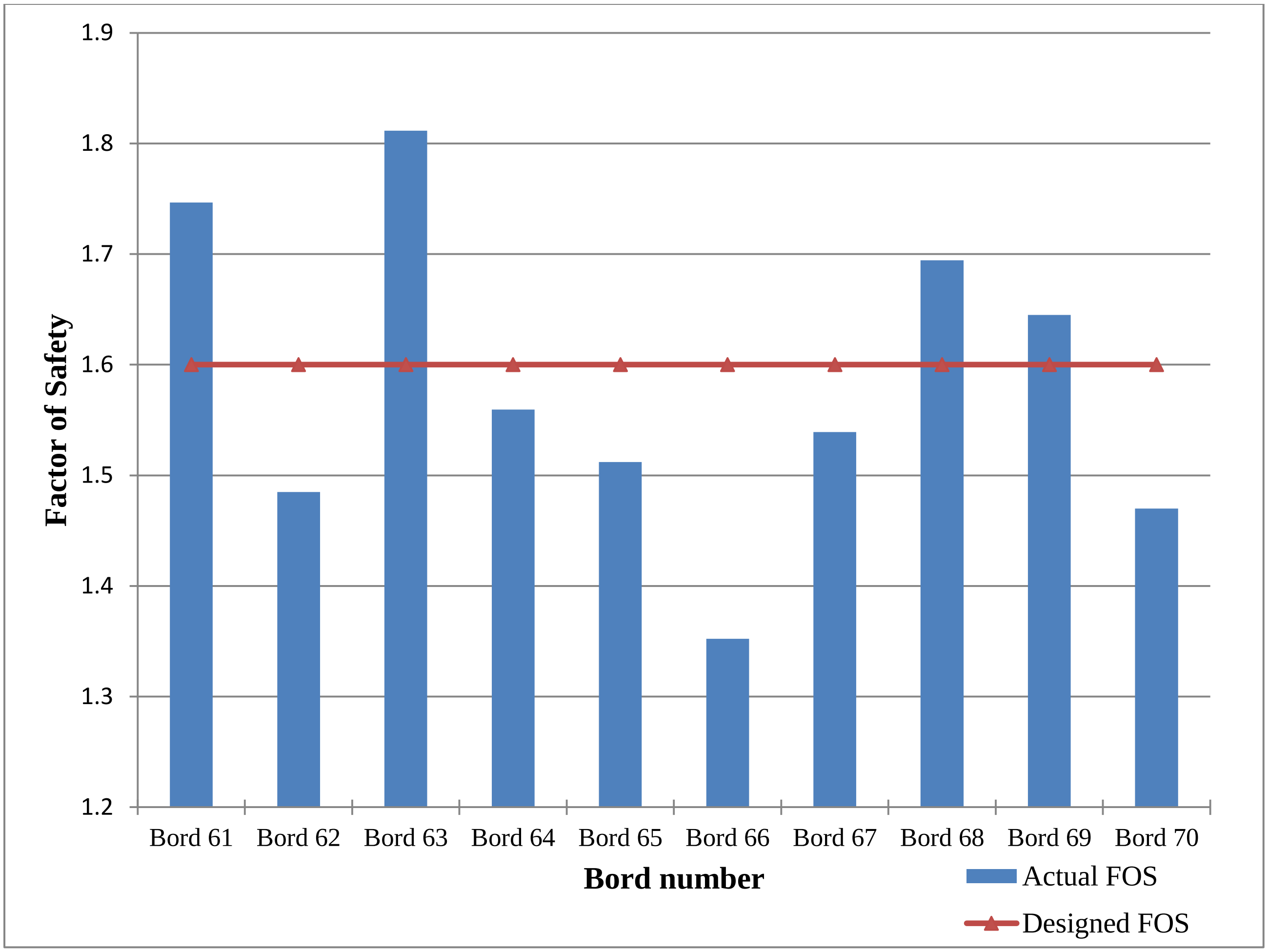

Figure 5.2: Factor of safety of insitu pillars

From the results, it can be noted that only 4 out of 10 measured insitu pillars in Bord 61, 63, 68 and 69 have a factor of safety greater than 1.6. The author looked at the reasons behind this and observed that stoping overbreak was one of the principal causes. The approved drilling pattern (refer to Appendix A) shows that the acceptable overbreak is 0.1m above the back holes and 0.3m below the lifters to give the desired mining height of 2m. Pillar design as a result of the overstated DRMS does not match the current ground conditions. As a result, the planned effective width will be smaller than the anticipated width in such areas. In addition, poor ground conditions and the type of explosives used (ANFO) were also seen as probable reasons behind decreased safety factors. From the results, most teams have failed to maintain a slice of 2m stoping height. Teams are mining a height greater than 2m due to overbreak as shown in Table 5.1. The author plotted a graph of factor of safety against overbreak to identify a trend of how the factor of safety changes with overbreak for the current design at a constant depth of 136m assuming an effective pillar width of 3m (refer to Figure 5.3).

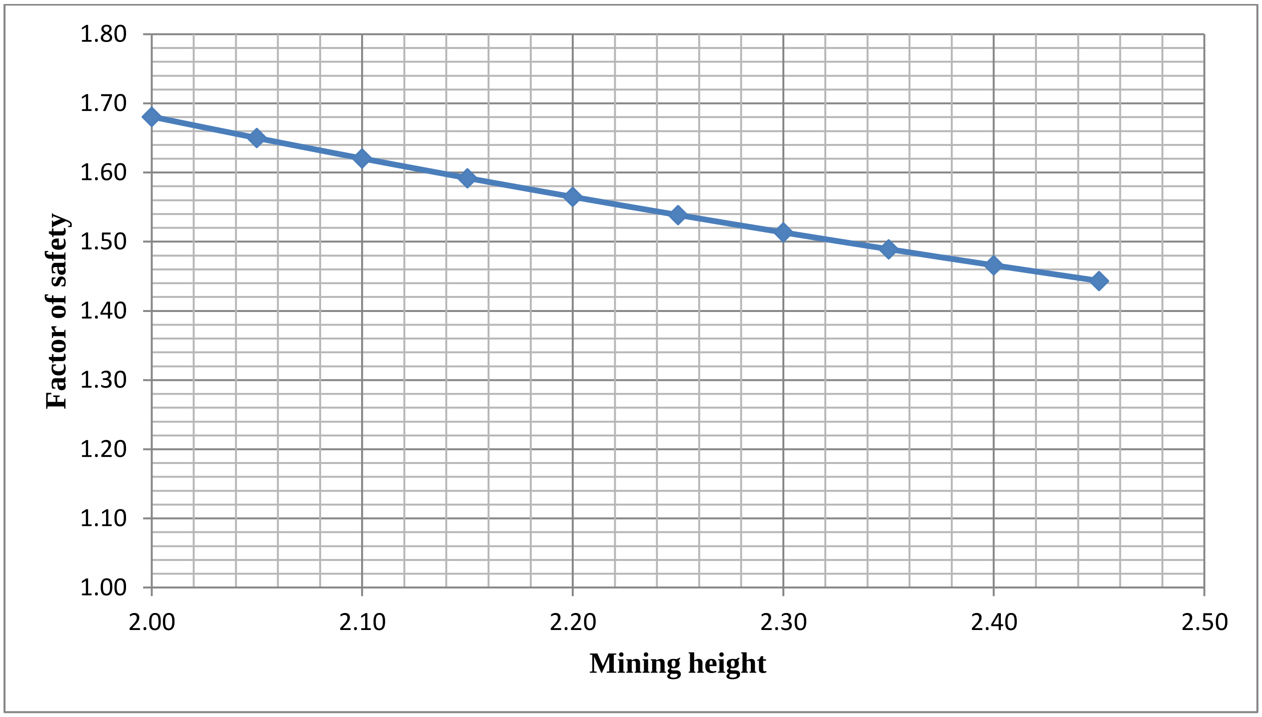

Figure 5.3: FOS against overbreak at 136m depth

Figure 5.3 shows how increase in mining height affects the factor of safety. The use of ANFO explosives results in stoping overbreak and generates excessive bad hangings which unravel unpredictably. A detailed analysis of ANFO will be discussed in section 5.5. Due to this effect, it is thus critical to redesign pillars that can be less influenced by a small change in overbreak. The author noted that the current pillar system used in bad ground conditions is inadequate based on the review done. Pillar robbing also affects the factor of safety as shown by a pillar located in Bord 66 (Table 5.1).

Zvarivadza (2012) illustrated a relationship of probability of failure and safety factors as shown in Figure 5.4. Using results from Table 5.1 and Figure 5.4, it can be seen that for a FOS of 1.6, the probability of pillar failure is 0.5% and at a lowest FOS of 1.33 obtained from Bord 66, the probability of failure increases to 5%. A decrease in FOS therefore increases the probability of failure hence it is critical to design and implement a pillar design that can result in stable excavations since the ground conditions are poor. Small isolated pillars are implausible to affect the overall stability of the mine however; a cluster of robbed pillars failing may affect the overall stability of the mine. This will have cost implications to the mine. An alternative for ANFO needs to be considered to reduce the effects of stoping overbreak and the unravelling of rocks. Ore dilution occurs as a result of overbreak as illustrated in Figure 1.4 which shows that the grade of PGMs decreases above and below the required slice.

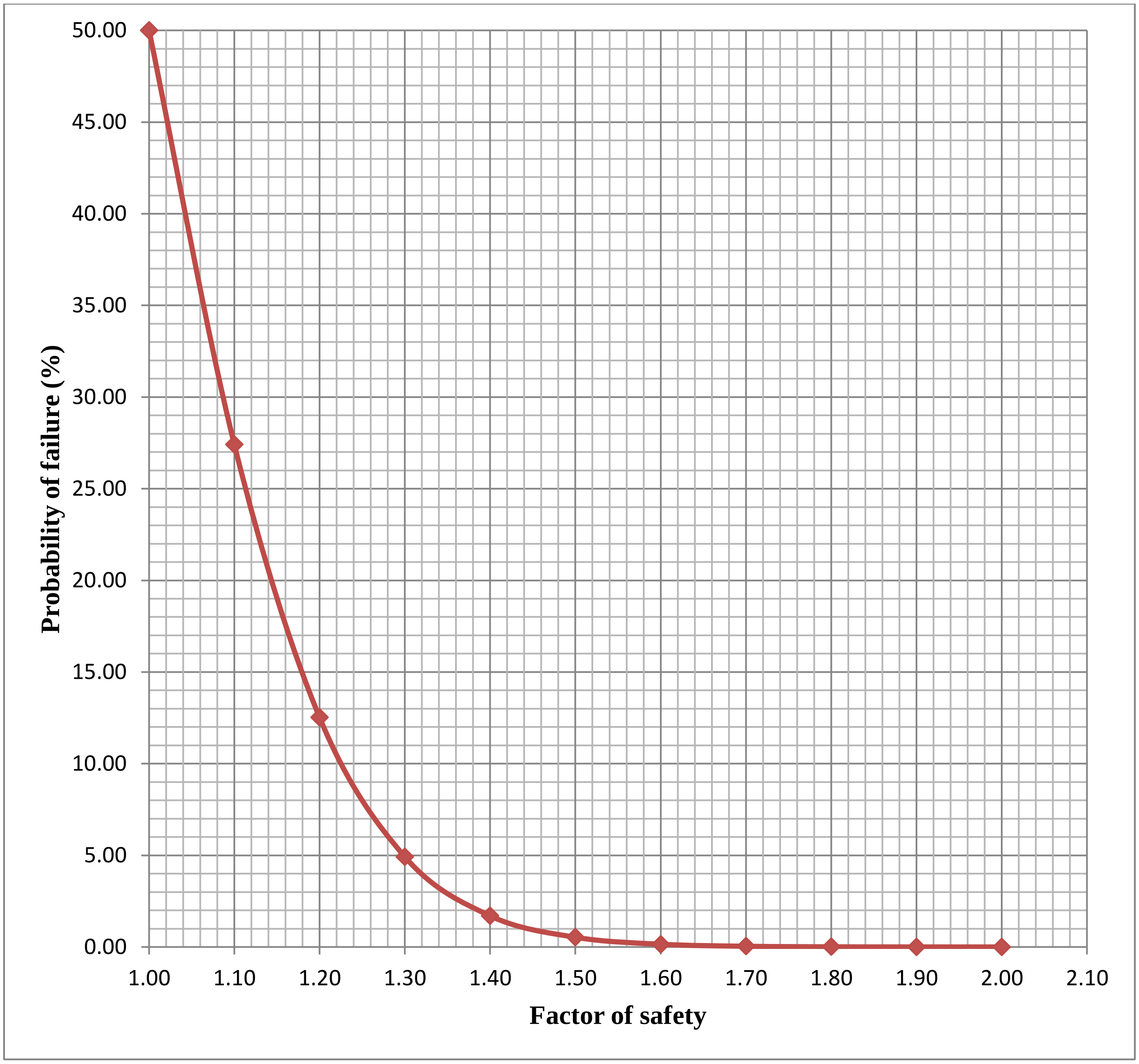

Figure 5.4: Relationship between probability of failure and factor of safety (Zvarivadza, 2012 citing Brady and Brown, 2006)

5.2 Pillar infringement

Good pillar cutting practice is ultimate in giving stable excavations. The results of actual regional pillar dimensions were compared against the designed pillar area in order to determine pillar infringement. Table 5.2 shows the results of pillar infringement. It was deduced that larger than designed pillar dimensions were left to support potential fall zones resulting from persistent shallow dipping structures in the vicinity of upthrow faults. This consequently reduced productivity but safety was at its best. The author thus requires a cost benefit analysis in this region which compares the effect of additional support against the locked ore in form of bigger pillars.

The pillar dimensions of regional pillars were measured and pillar infringement was determined. Pillar design in the study area requires pillar dimensions of 3m by 10m equivalent to an area of 30m^2. Mining teams in the aforementioned areas left pillars with the dimensions given in Table 5.2.

Pillar infringement is given by:

$$\frac{(Actual\ area - Design\ area) \times 100}{Design\ area}$$

For example, actual pillar dimension of 2.7m × 12.8m give an actual pillar area of 34.56m^2. Considering that the design area is 30m^2,

$$\text{Percentage infringement} = \frac{(34.56 - 30) \times 100}{30}$$

$$= 15.2\%.$$

Significant amount of ore was locked up in order to support potential zones of weakness that can impose catastrophic consequences to the mine.

Table 5.2: Pillar infringement in regional pillars

Actual pillar dimension		Pillar area		Pillar infringement	Comment on variance
Width (m)	Length (m)	Actual	Design	Percentage	
2.7	12.8	34.56	30	15.20%	Larger than design pillars were left to support potential fall zones resulting from persistent shallow north dipping structures in the vicinity of 11m upthrow as well as shear zones
3.1	10.4	32.24	30	7.47%	
2.8	13.5	37.8	30	26.00%	
2.8	9.9	27.72	30	-7.60%	
3.2	11	35.2	30	17.33%	
3	9.4	28.2	30	-6.00%	
3.1	11	34.1	30	13.67%	
3.6	10.5	37.8	30	26.00%	
3.3	11	36.3	30	21.00%	
3	12.3	36.9	30	23.00%	
3	10.6	33.92	30	13.07%	
3.1	12.1	37.51	30	25.03%	
2.9	9.3	26.97	30	-10.10%	
2.9	12.2	35.38	30	17.93%	
3.05	**11.1**	**33.99**	**30**	**13.29%**	**Averages**

In sections with negative pillar infringement, it shows that there was pillar robbing. Pillar monitoring is done at the research area and only shotcrete is applied to robbed pillars as shown in Figure 5.5 (Photograph taken by the author). Pillar robbing decreases the factor of safety hence it is critical to have good cutting practices. To improve the strength of robbed pillars, the author therefore recommends the use of timber props, rock anchors and confinement support other than just shotcrete. Buttressing a pillar with reinforced concrete will improve pillar strength while other confinement methods improve pillar integrity or prevent rock deterioration.

Figure 5.5: Shotcreted pillar at case study mine

A conservative design approach is vital to prevent pillars from failing. Castro-Filgueira, et al (2017), investigated the expediency of pillar strapping method in a bid to stabilize the robbed pillars. The author recommends the mines to consider installation of cables around robbed pillars apart from the use of shotcrete and rock bolting. Strapping increases pillar strength, ductility, slightly increases confinement as well as diminishes the ongoing degradation. Castro-Filgueira, et al (2017), pointed out that the use of cable bolts around pillars works very well where most of the surrounding non yielding pillars are stable and where spans are small. The mines on the Great Dyke are considered to be a perfect fit as they retain the aforementioned conditions for strapping.

Comprehensive revision of the design was done in jointed rockmass using software such as MAP3D and Examine2D to see the actual stress distributions. The designed effective pillar width at the case study mine for the in stope pillars is 3m, signifying that the effective pillar width does not meet the design requirements. The width-to-height ratio is 1.5 which is less than the industry accepted value of 2.5. It is thus of utmost importance for the mine to consider redesigning of pillars using the current available geotechnical information.

Due to increased scaling of in stope pillars and the need to mitigate the disastrous effects of pillar run, the author reviewed the pillar design at the case study mine using both empirical means and numerical modelling. Pillar strength is principally affected by the prevailing joints in the research area. The laboratory UCS of the orebody is 160MPa and the mine is currently using a DRMS of 63MPa for the whole mine. The author reviewed the DRMS that is currently used in geotechnically poor ground conditions. One third of the laboratory UCS need to be used as noted by Zvarivadza (2012) which is in agreement with Stacy and Swart (2001). A K value of 63MPa was deduced during the preliminary stages of mining when operations were still carried out in good ground conditions. As mining progressed, the ground conditions deteriorated as a result of faults and sympathetic joints, hence the need to use the least values of one third for the DRMS. The K value which must be used in new pillar design is therefore 53MPa (1/3 of 160MPa). A span of 6m will be maintained; however, the effective in stope pillar width needs to be determined as a result of scaling in some areas and also due to decreased actual safety factors. Most sections are failing to meet the desired FOS, which necessitated the need for a new pillar design in addition to enforcement. A depth of 250m was used in the new pillar design so that the designed pillars will withstand maximum stresses, the current operations are at 220m depth and soon mining operations will be carried out at a maximum depth of 250m. The stoping height will be maintained at 2m which is in line with the platinum peak as shown in Figure 1.4.

The DRMS of 53MPa is also in line with the one obtained from Laubscher (1990) equation:

$$DRMS = UCS \times (MRMR - R_{UCS}) / 100$$

R_{UCS} is the rating of the uniaxial compressive strength from MRMR. A UCS of 160Mpa corresponds to a rating of 16 hence:

$$DRMR = 160 \times (49-16)/100$$

$$= 53MPa$$

For pillars with a W_e: H ratio that is less than 4.5, the strength of the pillar is obtained as follows:

$$Pillar\ Strength = K\frac{W_e^{0.5}}{h^{0.75}}$$

The Factor of Safety (FOS) is defined as:

$$FoS = \frac{Pillar\ strength}{Pillar\ load}$$

The design Factor of Safety (FOS) for the in stope pillar must be greater than or equal to 1.6, as noted from hard rock bord and pillar literature.

Using the aforementioned equations, a K value of 53MPa, stoping height of 2m and the density of the rock, the effective width which can sustain the maximum stress is found to be 4.56m. Taking into consideration human errors in pillar cutting practices which account for 90% based on literature, the designed effective width will be equal to 5.06m. The mine is currently leaving rectangular pillars hence using the effective pillar width equation.

$$W_e = 4\frac{Pillar\ area}{Pillar\ perimeter}$$

The designed effective width of 5.06m corresponds to **7m** by **4m** regional pillars. The new pillar design will result in stable excavations as compared to the current 10m by 3m regional pillars. Improved safety was proven by both empirical means and numerical modelling.

The following safety factors were used to define the state of the pillar according to Van Der Merwe (1993):

- FOS < 1.0 Failure (Severe stress damage)

- 1.0 < FOS < 1.6 Critical stage (Pillar scaling)
- FoS > 1.6 Stable

Most pillars are currently experiencing preliminary scaling because their actual FOS is between 1 and 1.6. The configuration of the newly designed pillar dimensions should follow a staggered system which is explained in the next section.

5.3 Staggered pillar System

The staggered pillar system is a system that is meant to intercept jointing in the roof to prevent roof falls that tend to be continuous and dominant. The pillars are not in straight columns so that alternate rows capture any persistent faults and dykes. East-west joints are most prominent on the Great Dyke, hence they should be intercepted through staggering of pillars and also through stopping perpendicular to the joint sets in a bid to mitigate the disastrous effects of pillar run. Figure 5.6 shows an illustration of a plan view of a section with pillars intercepted by planes of weakness.

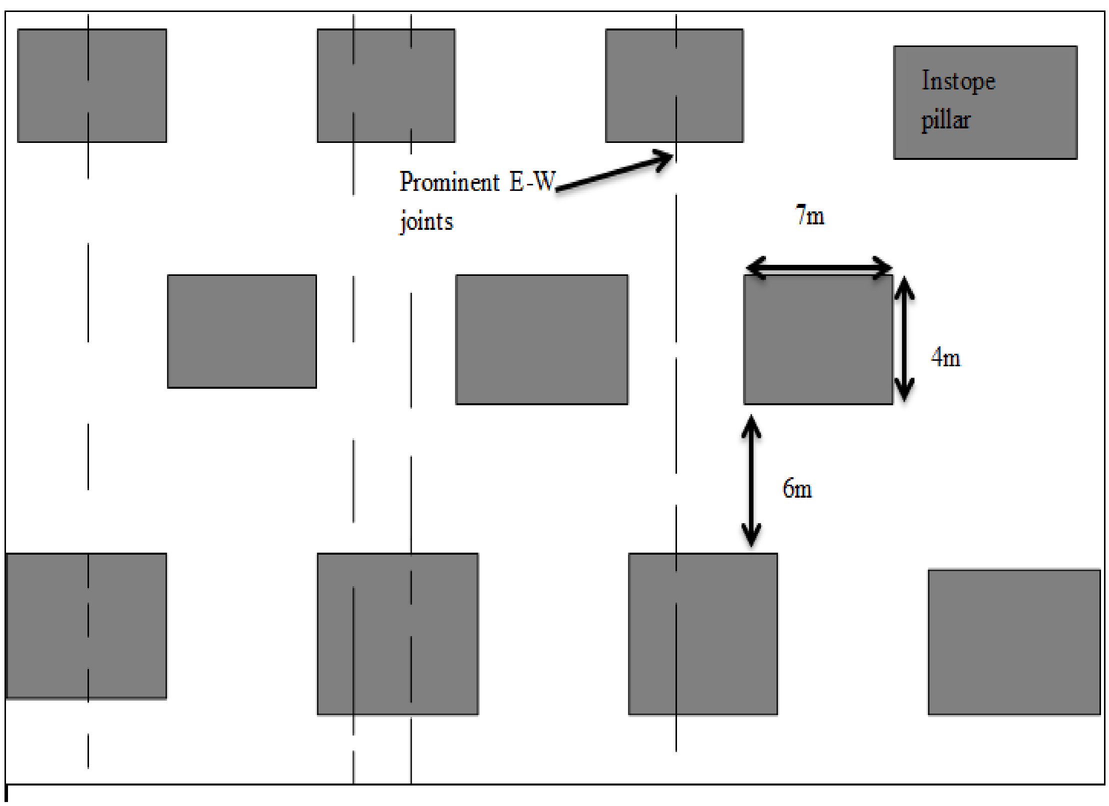

Figure 5.6: Proposed staggered pillar design

5.4 Effect of pillar width on Factor of safety

Poor cutting practices can result in reduced actual pillar width. This is attributable to poor drilling and also blast damage. Underground observations revealed that an average of 100mm of pillars sidewalls is being damaged due to blasting operations. The effect of such poor cutting practice is shown in Figure 5.7. The mining height was assumed to be constant at 2m at a mining depth of 180m. It can be seen that reducing the effective pillar width to less than 4.8m, will decrease safety factors to less than the desired FOS of 1.6. The effects of such poor cutting practices are pillar scaling and pillar run. The effect of blasting operations on support systems is described in detail in Chapter 6. It is therefore important for mines to cut the desired excavations using appropriate techniques that minimise overbreak in order to maintain the integrity of pillars.

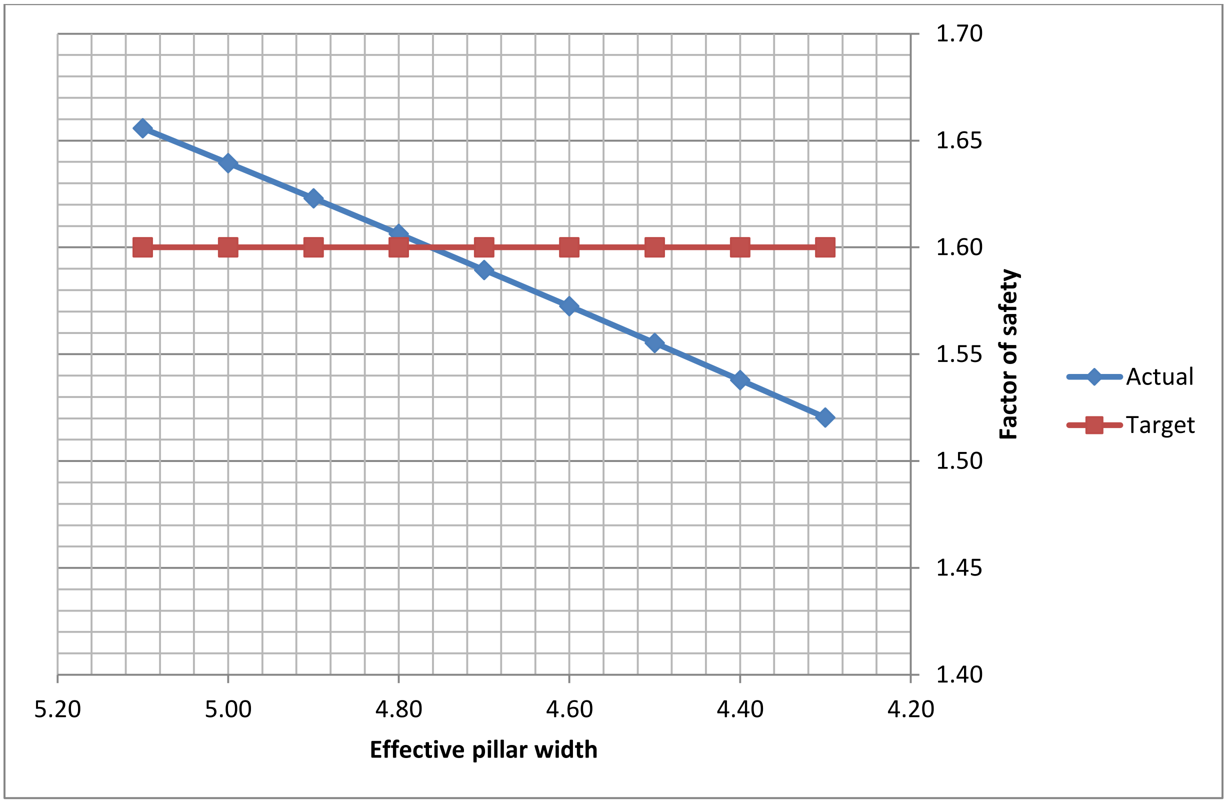

Figure 5.7: Effect of decreasing pillar width on FOS

5.5 Pillar Numerical Modelling

An analysis of strength factor tensions around a pillar line running through pillar centres was done using Examine 2D and MAP3D modelling software.

5.5.1 Examine 2D results

Examine 2D results showing strength factor tensions around non yielding pillars are given in Figure 5.8. Geotechnical data shown in Table 2.1 was used as input parameters. A constant mining height of 2m was used in all models. Pillar dimensions and bord dimensions simulated were taken from the current pillar layout shown in Figure 1.6. Strength factor tensions are used as a measure of safety (Chikande and Zvarivadza, 2016). Strength factors must be greater than one for excavations to be considered stable. Areas with strength factors less than one require additional support (Chikande and Zvarivadza, 2016).

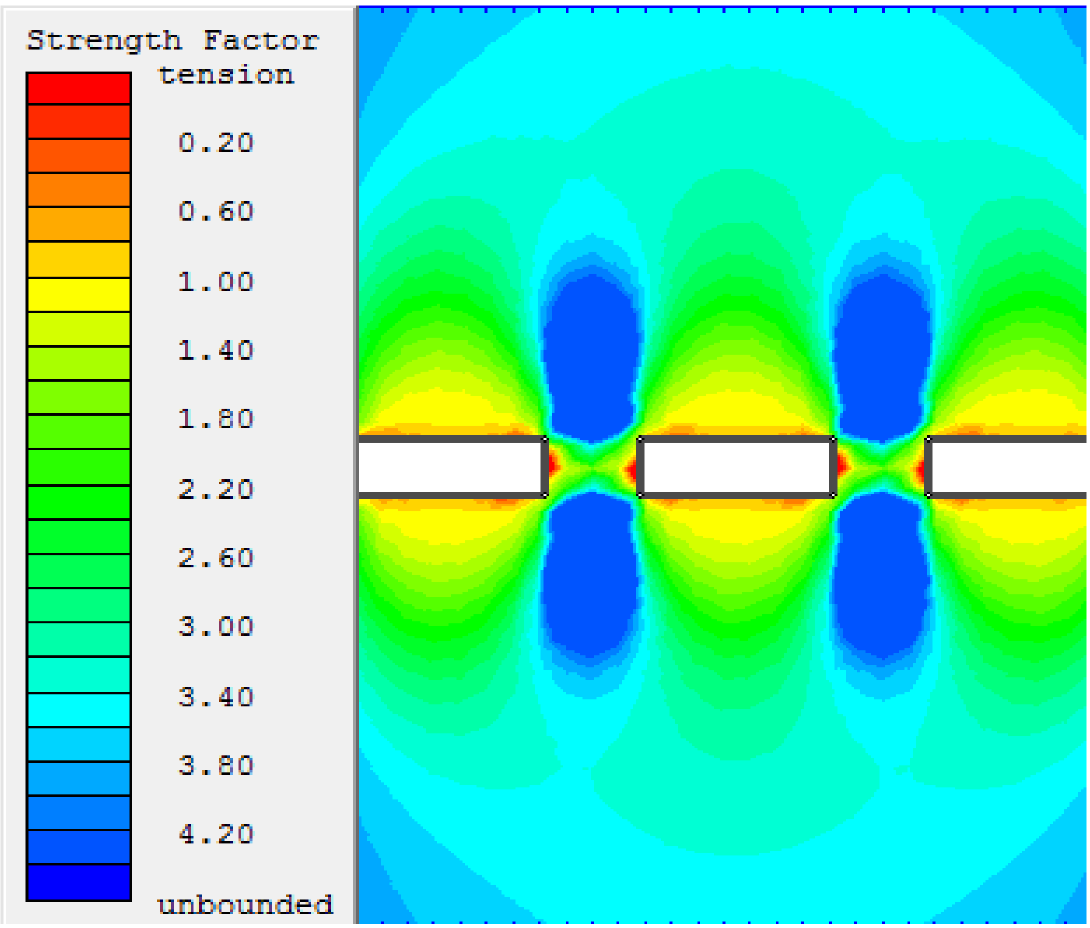

Figure 5.8: Stress distribution around non-yielding pillars

It was observed that pillar stress is high on the centres of the pillars and decreases drifting away from the centres to the edge of pillars. However, there is highest confinement at the centre of the pillar, therefore highest strength as well. This is reflected by high strength factor at the centres of the pillars and gradual decrease of the strength factor to 0.2 going towards the edge of the pillars.

Secondary support need to be considered in areas with critically low safety factors, areas with strength factors less than 1. The areas with low safety factors are the sidewalls of the pillar or excavation. These areas are then susceptible to pillar scaling.

5.5.2 MAP3D results

To ensure the integrity of the results, a sensitivity analysis was done on the trimmed models. MAP3D was used together with the mining layout to determine safety factors of pillars in anisotropic jointed rock masses. The footwall and the MSZ ore zone uniaxial compressive stress (UCS) and elastic modulus (refer to Table 2.1) were selected as input parameters into the analysis of the pillar design. The analysis of the mine layout was carried out using the output from elastic modelling of the mine layout and analytical methods to determine the stability of the existing and proposed accesses. Map3D applies a boundary element method of stress analysis and the program assumes that the rock is elastic, homogeneous and continuous. A portion of the selected mining layout was assessed using the application mXrap. A constant mining height of 2.0 m obtained from the mine was used in all models.

The distribution of the FOS for the pillars is presented in Figure 5.9 for poor ground. Pillars with a FOS less than 1.0 are considered to be failing or at high risk of failure. From the MAP3D results shown in Figure 5.9, the factor of safety decreases with depth and with poor cutting practices as evaluated on the actual mining layout used. Colour variations at the same depth are attributable to poor cutting practice. This poses an elevated risk of pillar run.

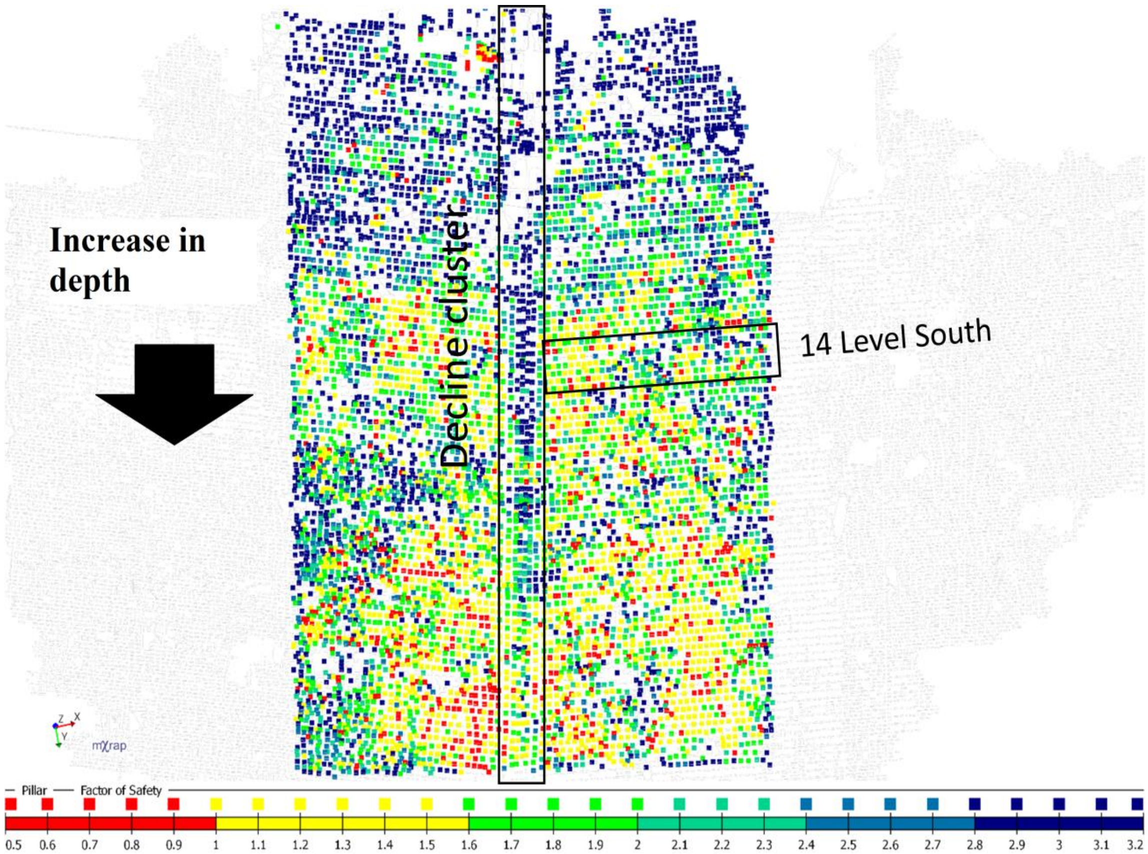

Figure 5.9: FOS of the pillars in poor ground conditions

5.6 Main access decline and barrier pillars analysis

The access decline in the research area is stable with a FOS > 2.5 signifying that the risk of decline failure is low. A few secluded small pillars do exist in the decline and are unlikely to influence the overall stability of the decline. Pillars that are experiencing stress damage next to the decline may be supported by bolts and wire mesh or buttressed with concrete to prevent scaling. A decrease in pillar sizes along the decline with depth reduces FOS to below the design criteria of 2.5, hence sound design is critical to ensure the integrity of the access way. A few pillars supporting the decline are in good ground conditions and these pillars may be experiencing stress damage. Closure and stress monitoring will need to be implemented in these areas to determine pillar performance. The decline protection pillars are (3.5m x 10m) and will experience stress increase at depth below 190m. Pre-mining state of stress has a significant influence on the stability of excavations, hence it is therefore imperative to make a reasonable estimate of the pre-mining state of stress for the design of underground excavations.

5.6.1 Regional pillar stability analysis

Regional pillars are pillars that are used to compartmentalise the mine in case of a pillar run. The fundamental use of these pillars is to prevent pillar run spreading to the neighbouring mining regions. Regional pillars should be designed to a width to height ratio greater or equal to 10:1 (Jager, et al., 1995). The research area is shallow, hence regional pillars are not anticipated to fail during the life of mine. Cognisance should be taken so as to avoid cutting through the footwall or the hangingwall. The design rule of thumb for failure basis as described by Ryder and Jager (2002) is defined as:

$$\mathrm{APS} \leq \mathrm{f_a}\sigma_\mathrm{c}$$

Where: APS is the average pillar stress,

f_a is the empirical factor taken to be 2.5,

σ_c is the Uniaxial compressive strength.

The design FOS for the stability pillars in the research area is 2.5. Observations and results showed that pillars along the decline carry low stress while small in-stope pillars carry relatively high stress. This means that the decline pillars together with relatively large pillars are unlikely to experience stress damage, however, this needs to be investigated. Stress levels tend to increase with increase in depth and could affect the stability of small pillars. It is recommended that areas with a cluster of small pillars be monitored.

The analysis (Figure 5.10) indicates that the barrier pillars are under designed when spaced at 156m dip span for all mining depths. The factor of safety dropped to 2.2, which is less than the required FOS of 2.5. The minimum barrier pillar width was determined to be 9m for normal ground conditions and 10m for poor ground conditions at a depth of 80m below surface.

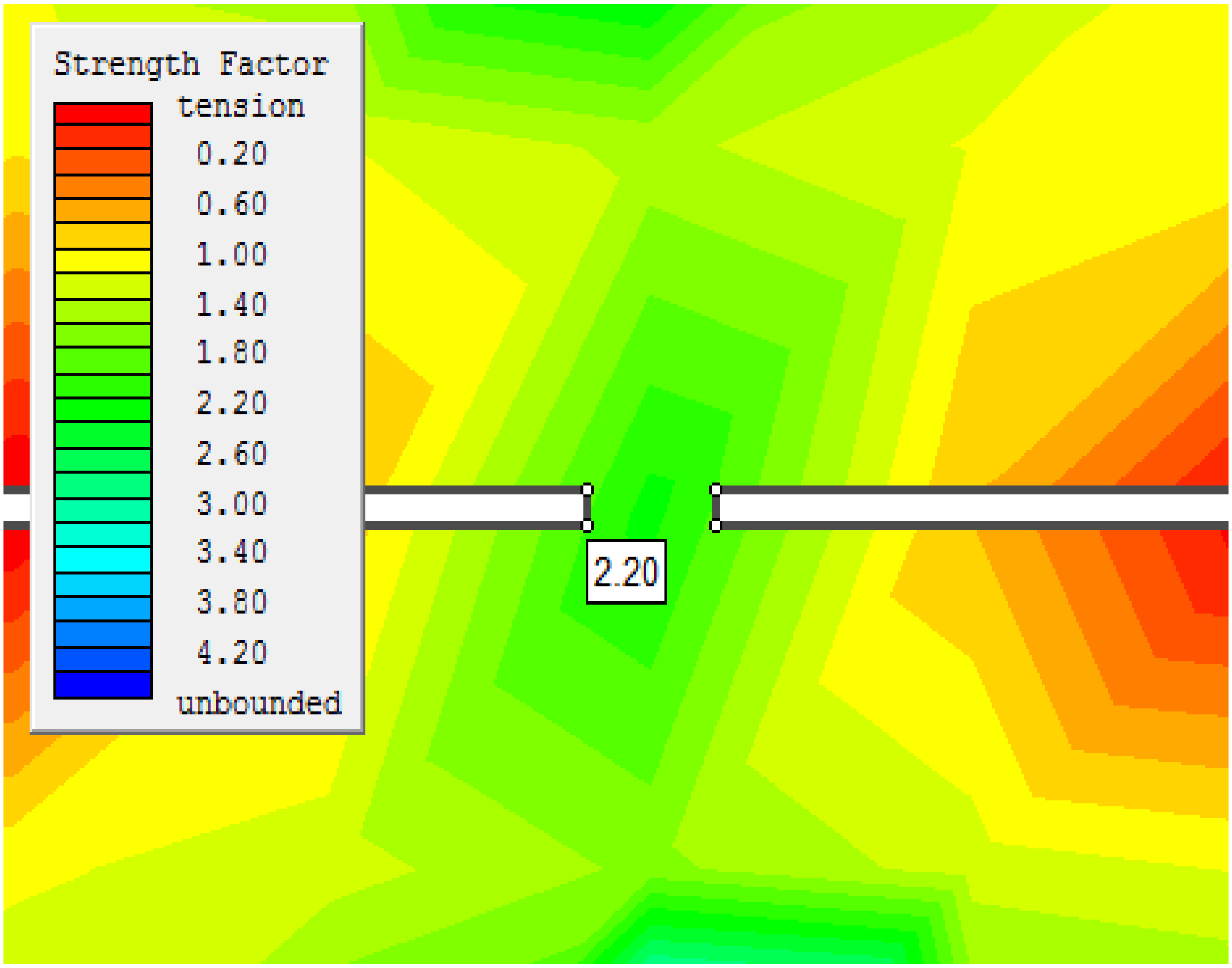

Figure 5.10: FOS for barrier pillars

Table 5.3 shows the new barrier pillar design in bad ground conditions following a review of the current pillar design.

Table 5.3: Barrier pillar dimensions for poor ground conditions

Depth (m)	Bord width (m)	Pillar holing width (m)	Stoping height (m)	Pillar length (strike) (m)	Pillar width (dip) (m)	K-value (MPa)	Pillar strength (MPa)	APS (MPa)	FOS
80	156	6.0	2.0	20.0	10.0	53	155	54	2.86
100	156	6.0	2.0	20.0	11.0	53	177	62	2.86
120	156	6.0	2.0	20.0	12.0	53	202	69	2.95
140	156	6.0	2.0	20.0	12.0	53	202	80	2.53
160	156	6.0	2.0	20.0	13.0	53	231	85	2.72
180	156	6.0	2.0	20.0	14.0	53	262	89	2.94
200	156	6.0	2.0	20.0	14.0	53	262	99	2.64
220	156	6.0	2.0	20.0	15.0	53	296	102	2.89
240	156	6.0	2.0	20.0	15.0	53	296	112	2.65

5.6.2 Main access pillar design analysis

Numerical modelling was done to assess the stability of the main access decline workings. The author used MAP3D and the mining layout to determine the stability of the existing and proposed main access declines. The anticipated pillar strength was calculated using a strength factor K = 63MPa for normal ground conditions. Pillar strength calculated using a strength factor of 63MPa for normal ground conditions will be high, hence a smaller effective width will be designed for. Pillar strength will be less for the same effective width when using a K value of 53 MPa for poor ground conditions. It can be seen that small pillars have a calculated strength which is less than the APS calculated for these pillars (shown in Figure 5.11), resulting in a safety factor of less than 2.5, indicating that these pillar have a high probability of failure. The main decline pillars have a FOS less than 2.5 beyond 190m depth, which is below the design requirements. It is therefore recommended that, to maintain stability over the main access decline and robbed pillars, there is need to increase confinement on the pillars by supporting them with mesh and lacing or buttressing the pillars with concrete. The FOS for the decline pillars decreases to 2.1 for geotechnically poor ground conditions, which is less than the design requirement for protection pillars. Figure 5.11 shows the FOS on the pillars for the access decline. As is expected, the FOS decreases with depth and for poor ground conditions, a few isolated pillars at depth have FOS < 1.6 for in stope pillars. The FOS for the decline pillars is less than 2.5 which is the design for large pillars in normal ground conditions. In order to maintain stability, the size of the decline pillars needs to be increased to 10m x 6m to protect access infrastructure. The effective width of 10m by 6m pillars has a FOS greater than 2.5 beyond 188m.

For poor ground conditions six panels will have $1.0 < \text{FOS} < 1.6$ which is less that the design criteria for in stope pillars ($\text{FOS} \geq 1.6$). The minimum FOS for the decline pillars for poor ground conditions is 2.3 which is less than the design criteria of 2.5. In poor ground conditions, the size of the pillar need to be increased with depth.

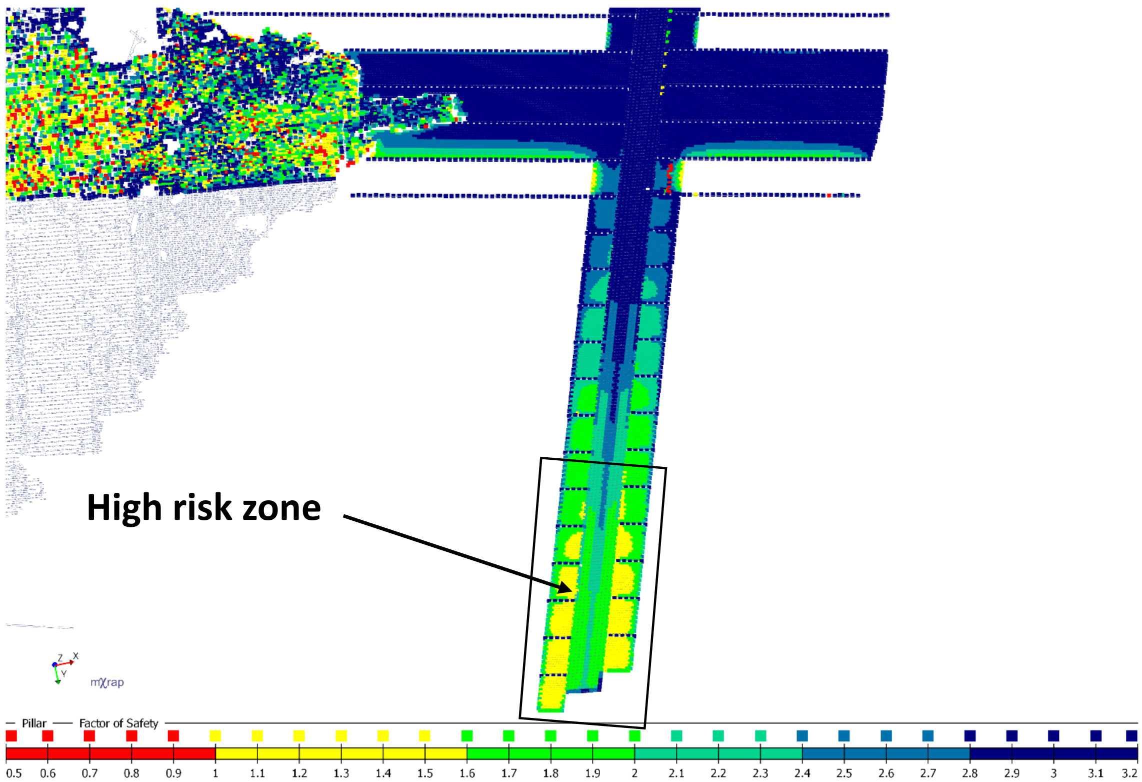

Figure 5.11: FOS main decline

5.7 Analysis of blast design and explosives used

Drilling and blasting affect excavation stability hence the effect of blast damage on falls of ground and bord cutting practice is described in Chapter 6. The mine is currently using ANFO for charging. ANFO generates copious gases which widen joints and fractures. This weakens the rock hence the author concluded that the use of ANFO is not appropriate in bad ground conditions. The area where the research was carried out was highly jointed. The use of ANFO explosives in this area will weaken cohesion between these joints hence further reducing the quality of the rock. Alternative explosives such as watergel and emulsion must be considered in bad grounds since ANFO is not suitable. The implications of using ANFO include the continuous unravelling of the rocks which will result in a decrease in safety and productivity. Mining is a cycle in which each stage depends on the preceding stage. From observations, barring down consumed more time before the ends were supported. This resulted in some teams failing to meet their set production targets. In addition, barring down using pinch bars in these areas was seen as a very dangerous exercise due to rock falls, hence the author recommended the use of mechanical scalers to improve both safety and productivity.

In addition, the blast design used has serious implications on the type of support system used. It is thus of paramount importance to come up with an effective blast design that minimises dilution, reduces rock overbreak and rock damage. Slender pillars due to overbreak will result in a decrease in factor of safety. It is thus critical to focus closely on the burden, spacing, correct initiation and drilling accuracy. The drilling pattern used at the research area is shown in Appendix A. Poor drilling accuracy causes stoping overbreak, hence it is required to practice good drilling accuracy for the given recommendations to be effective. The author recommends close monitoring when drilling in order to reduce rock overbreak. Furthermore, adequate training must be administered to drilling operators in order for them to achieve set targets without compromising safety.

5.8 Conclusions

The effects of overbreak together with pillar robbing have led to a decrease in the FOS of insitu pillars. Scaling of pillars and a decrease in actual safety factors gave an acute reflection that the current pillar design needs to be improved. The gathered results of regional pillars show that larger than design pillars were left to support potential fall zones resulting from persistent shallow north dipping structures. Pillar scaling was also influenced by the presence of geological structures. Undersized pillars and the existence of steeply dipping faults and sympathetic joints reduce the pillar strength. The failed pillars are no longer contributing to the overall mine stability but provide local support due to the residual strength. New pillar design (7m by 4m) which takes into consideration the current structural data must be implemented in a bid to improve safety. Staggering of pillars reduces the catastrophic consequences that may be spread over to the neighbouring ends, hence causing pillar run. It is also practiced in a bid to limit the long exposure of the structures within a panel. Routine pillar monitoring to be implemented on the main access routes to determine long term pillar performance. Numerical modelling softwares such as MAP3D and FLAC3D need to be used for optimum pillar design as they show stress distributions as well as strength tension factors. The findings of this case study, with the necessary adjustments, can be adopted by other platinum mines within the same geological location since they are mining the same type of deposit using the same mining method.

CHAPTER 6: BULK EMULSION TRIAL

6.0 Introduction

Observations made underground revealed that the outer 100mm of pillar sidewalls was damaged by blasting operations. This observation is in agreement with observations made by Swart (2005) which revealed that the outer 80mm of pillar sidewalls can be damaged as a result of blasting operations. Sockets in the hangingwall in the area of research indicated the effect of drilling and blasting on falls of ground (FOG) that had occurred. This accentuates the effect that poor drilling and blasting can have on stability of excavations. Since blast damage have a negative effect on hangingwall stability, the author reviewed the current blasting practices and analysed the effects of bulk emulsion explosives. The trial of bulk emulsion was based on one of the mines on the Great Dyke; however the findings of the trial can be extended to other mines on the Great Dyke thereby improving hangingwall stability.

Sound mining practices based on the appropriate choice of explosive energy lead to significantly safer mining operations. The selection of the exact charge mass, explosive type and blasting pattern is imperative in mining the desired slice thereby reducing stoping overbreak and subsequently minimizing Platinum Group Elements (PGE) dilution. ANFO explosive is used in numerous mines due to its simplicity of use and economics. The area of research is sited on the Great Dyke of Zimbabwe and is truncated by faults and sympathetic joints. Zero harm and innovation being some of the core values in the mining industry, there was need to reflect on the suitability of the current generation of explosive to the ground conditions that the mine is now experiencing. The accident statistics gathered as from 2012 show that the greatest contribution of accidents is coming from falls of ground incidents as shown in Figure 6.1 thereby making it an area of concern. The trial was carried out for validation of the research findings and recommendations.

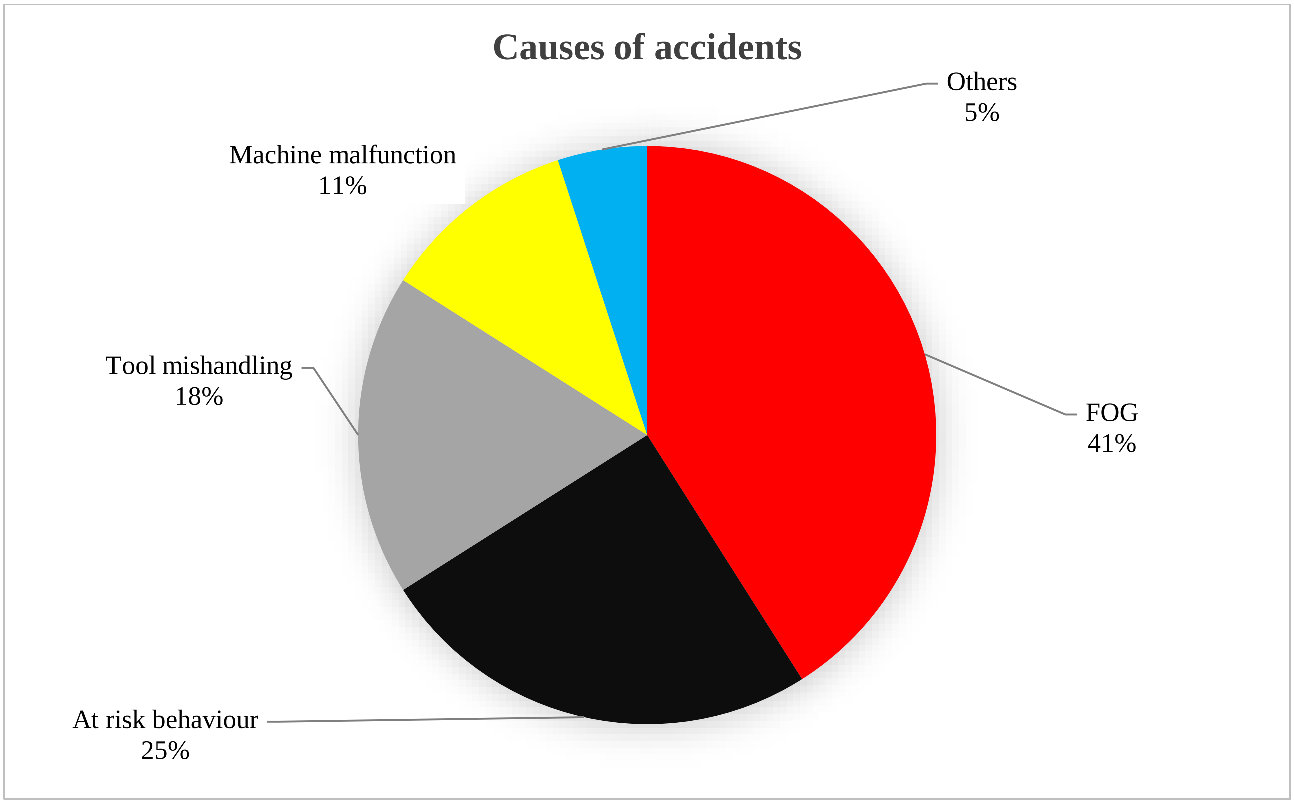

Figure 6.1: Summary of accidents by cause at the mine

6.1 Mining Operations Overview

The study was carried out at a shallow underground mine with its operations carried out less than 250m from the surface following the proposed recommendations. The mining method and the cycle of drilling, blasting, lashing and supporting are all described in this section.

6.1.1 Mining Method

The mining method utilized at the studied mine is room and pillar. Mining operations are carried out from the main decline advancing towards the strike direction. The main decline divides the mine into two regions, the northern part and the southern region. The size of the rooms mined varies with ground quality. Figure 1.6 shows the standard mining layout used in poor ground conditions. The stoping height is maintained at 2m to avoid PGMs dilution. In case of poor ground conditions, twin gullies which are 6m wide are mined leaving in stope pillars of 3m by 3m. Drives were developed from the main decline to the working areas.

6.1.2 Drilling

Drilling rigs are used to drill 51 holes with a diameter of 45mm. Three of these holes are enlarged to 102mm to give a second free face. A gulley is first marked after being cleaned up by LHDs and after all support installation has taken place. The length of holes drilled is 3.2m to give an advance of 2.8m to 3m. The average spacing is 0.5m while the burden is 0.67m. Appendix A shows the drilling pattern followed at the mine. Drilling accuracy is critical to have a good advance and also for maintaining the designed stoping height. Timing is then done by connecting the shock tubes according to the drilling pattern.

6.1.3 Charging and Blasting

After drilling the holes, water is pumped out of the holes and the holes are cleaned up. Shock tubes with detonators at both ends called Dual detonators are used, inserted in megamites cartridges to form a primer. ANFO is then used to charge the drilled holes. The author looked at the explosives currently used because high gas explosives widen the joints leading to the unravelling of rocks, which will result in a decrease in safety and productivity. In as much as an explosive is very essential in a mining set up to break the ground, it can also pose greater risk of worsening or damaging the rockmass thereby creating a hazardous working environment for employees. Due to the amplified generation of bad hangings and the unpredictable unravelling of loose rocks, there was need to find an explosive with energy to just break the rock into the required fragments and at the same time avoiding surrounding rockmass disturbances.

6.2 Aim

The study was undertaken with the main aim to improve the company Key Performance Indicators (KPIs) of safety and production. This was achieved through a comparative study of ANFO versus Emulsion use in geologically poor ground conditions.

6.3 Objectives

To achieve the main aim, the following objectives were set:

- To compare mining profiles after blasting (Emulsion vs ANFO)
- To analyse the effects of explosives on the rock mass
- To measure and compare advance per blast for ANFO, currently in use, with emulsion.
- To compare fragmentation for ANFO relative to Emulsion

- To compare re-entry periods
- To compare charging time per end.

6.4 Study approach

It is the aim of this research to optimize blasting at the studied mine to enhance safety. Both FOG accidents and pillar failure can be caused by poor drilling and blasting hence the need to review the current practices in a bid to improve both safety and productivity. Described herein is the research criterion and techniques used in collecting necessary and sufficient data in order to fulfil the study aim.

6.4.1 Research Criterion

The following study approach was adopted in carrying out the trial:

- The research started off with a literature review related to the study.
- Data collection, measurements and observations.
- Trials for the selected alternative were conducted and results were collected and a comprehensive comparison was carried out against the mine's actual results using ANFO.
- Trials for bulk emulsion were carried out in North 1(N1) and North 7 (N7) for the following reasons
    - These are adjacent sections hence easier for the two to share the 1.5t main charging unit (MCU).
    - All of them are operating in bad ground conditions.

6.4.2 Collection of current blast output

Data collection techniques for this project included observations and data capturing pertaining to the blast output which is in line with safety and production. Collected data gave the following:

i. Advance per blast
ii. Powder factor
iii. Fragmentation
iv. Half cast factor

6.5 Measurements and calculations

Measurements were divided into pre-blasting and post blasting parameters and calculations are based on these measurements.

6.5.1 Pre-blasting

The following parameters were measured and analysed prior to blasting:

- Hole diameter (mm) – was measured using a vienier calliper.
- Gulley length (m) – was measured using a distometer which measures to the nearest mm
- Mining width (m) - was measured using a distometer
- Holes depth (m) – was measured using a 5m tape.
- Burden (m) and spacing- were measured using a 5m tape.
- Advance (m) per blast- Subtract distance from peg to face before blast from distance from peg to face after blast.
- Blasts per day- were captured on daily blast measurement form.
- Average No of holes/panel - physical counting on the face.
- Average meters drilled/day (m) - was captured on daily blast measurement form.
- Tons per blast - calculated from data captured on the blast measurement form.
- Tons per day- calculated from data captured on the blast measurement form.
- Half cast factor - calculated from data captured on the blast measurement form.

6.5.2 Blasting

The following blasting parameters were observed, measured and analysed:

- Stemming length (m) - was measured using a one meter clino rule.
- Polypipes/round - physical counting of perimeter holes.
- Total primers mass (kg) per day- physical counting of primers used per end.
- Charged column Length (m) - subtract stemming length from drilled length.
- Column charge (kg/m^3) - to be calculated from data captured on the blast measurement form.
- Volume of Column Charge (m^3) per hole.
- Mass of explosives - column charge/hole (kg) recorded on the MCU.
- Total mass of explosive/section was calculated from the data captured on the blast measurement form.

- Total cost of explosives/gulley- was calculated from cost of explosives by the planning department.
- Explosives cost/ton - (USD) was calculated from cost of explosives and the blasted tonnage.

6.6 Results and analysis

The results and analysis of the bulk emulsion trial findings are given in this section. Table 6.1 shows a snapshot of the emulsion data collected from North 1 section during the month of May 2016.

Table 6.1: Sample of the collected data from one section

Date	Density	Distance before Blast	Distance after Blast	Advance	Height	Width	Tonnes Blasted	Mass Charged	Powder Factor	Number of Perimeter Holes	Expected sum of Barrel Length	Actual Barrel Length	Half Cast Factor (%)
5/23/2016	1.06	2.11	5.208	3.098	2.16	4.86	102.44	196.9	1.92	13	39	2.7	6.92
5/23/2016	1.06	24.015	27.033	3.018	2.02	6.03	115.80	231.2	2.00	13	39	4.896	12.55
5/23/2016	1.06	5.046	8.057	3.011	2.12	6.49	130.50	241.8	1.85	13	39	2.9	7.44
5/23/2016	1.06	10.695	13.762	3.067	2.12	5.019	102.80	200.2	1.95	14	42	3.1	14.52
5/24/2016	1.07	7.92	10.909	2.989	2.11	5.01	99.53	227.7	2.29	12	36	2.876	7.99
5/24/2016	1.07	2.043	5.321	3.278	2.4	5.1	126.39	237.2	1.88	12	36	2.65	7.36
5/24/2016	1.07	3.795	6.752	2.957	2.23	6.41	133.14	286.2	2.15	14	42	3.187	16.40
5/24/2016	1.07	5.267	8.089	2.822	2.11	5.98	112.16	210.5	1.88	11	33	2.548	7.72
5/25/2016	1.12	14.494	17.806	3.312	2.14	5.92	132.17	262	1.98	12	36	3.78	18.83
5/25/2016	1.12	2.962	6.346	3.384	2.033	4.947	107.21	246.1	2.30	12	36	2.467	6.85
5/25/2016	1.12	21.561	24.725	3.164	2.068	4.738	97.65	279.5	2.86	14	42	3.164	26.01

Table 6.1: Sample of the collected data from one section (continuation)

Date	Density	Distance before Blast	Distance after Blast	Advance	Height	Width	Tonnes Blasted	Mass Charged	Powder Factor	Number of Perimeter Holes	Expected sum of Barrel Length	Actual Barrel Length	Half Cast Factor (%)
5/26/2016	1.12	26.509	29.709	3.2	2.157	6.289	136.74	192.5	1.41	10	30	5.37	17.90
5/26/2016	1.12	18.737	22.098	3.361	2.181	4.86	112.22	255.6	2.28	15	45	7.89	17.53
5/26/2016	1.09	6.752	9.832	3.08	2.116	6.03	123.79	213.4	1.72	14	42	6.21	14.79
5/27/2016	1.09	5.724	8.843	3.119	2.197	5.024	108.44	189.9	1.75	11	33	6.892	20.88
5/27/2016	1.09	26.43	29.546	3.116	2.102	5.564	114.80	272.6	2.37	14	42	5.41	12.88
5/27/2016	1.09	3.833	7.075	3.242	1.871	4.192	80.10	229	2.86	12	36	6.27	17.42
5/27/2016	1.09	3.552	6.892	3.34	2.083	5.705	125.03	270.3	2.16	14	42	4.513	10.75
5/27/2016	1.09	16.328	19.508	3.18	2.235	5.187	116.13	209.5	1.80	13	39	4.915	12.60
5/27/2016	1.09	8.046	11.275	3.229	2.077	5.23	110.49	205.7	1.86	11	33	7.81	23.67

6.7 Bulk Emulsion Vs ANFO comparative study results

The results and analysis of the trial findings are given in this section. The trial was carried out over a three month period. *Actual* refers to the exact measurements on achieved in the field, *target* refers to the work plan for the mine and *budget* denotes the economic cut off parameter which is declared to the investors. ANFO performance was also analysed during the same emulsion trial period in North 2 and North 3 sections and the results are described in section 6.10. North 2 and North 3 are in similar ground conditions to North 1 and North 7.

6.7.1 Advance per blast

Figure 6.2 shows the advance per blast trend during the month of April 2016 using emulsion. The average advance per blast during the first month of trial (thus April 2016) averaged 2.85m which was lower than the targeted 3.0m mainly due to a calibration fault on the MCU causing holes to be undercharged, and also poor charging hose handling as employees were still familiarizing with the new charging system.

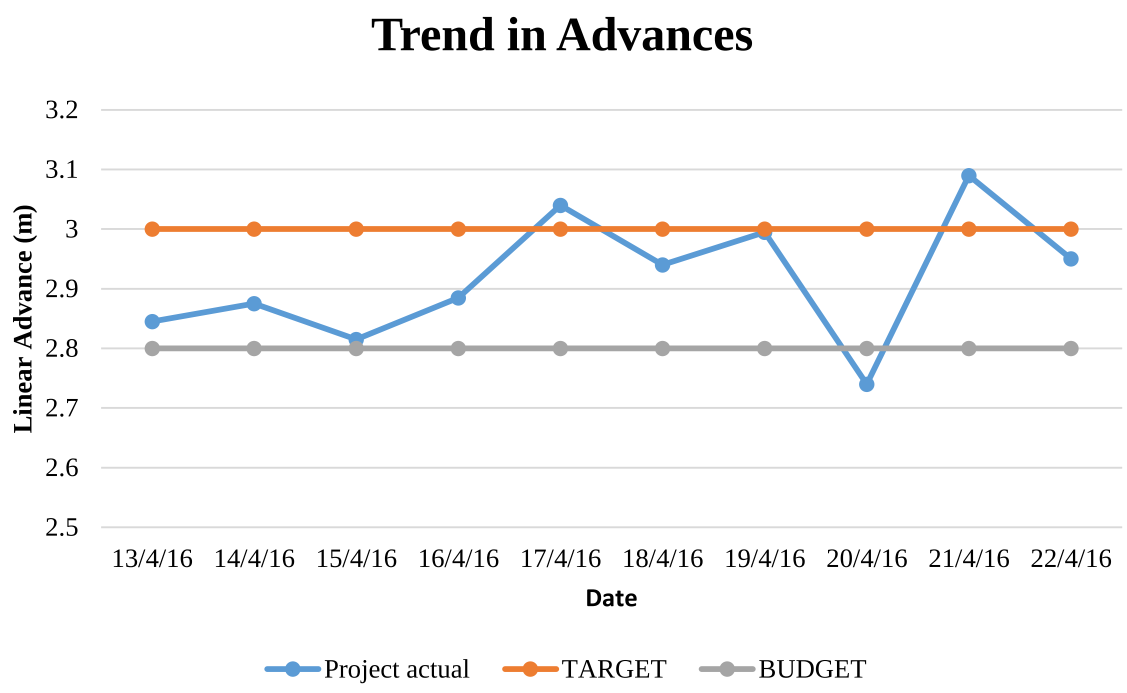

Figure 6.2: April trend in advances

6.7.2 April Half cast factor

Half cast factor is a measure of the blast induced over break. The half cast factors were determined using Equation 14. The literature behind half cast factor is outlined in section 2.10.2. Figure 6.3 shows the half cast factor trend for the month of April. From the graph, it can be noted that the average half cast factor using emulsion for the month was at 23.5%, which was below the target of 50%. Overbreak is predominantly affected by the properties of the host rock, blast design and explosive parameters (Dey and Murthy, 2010). Rock mass characterisation played a crucial part in lowering the half cast factors since operations were carried in geotechnically poor grounds.

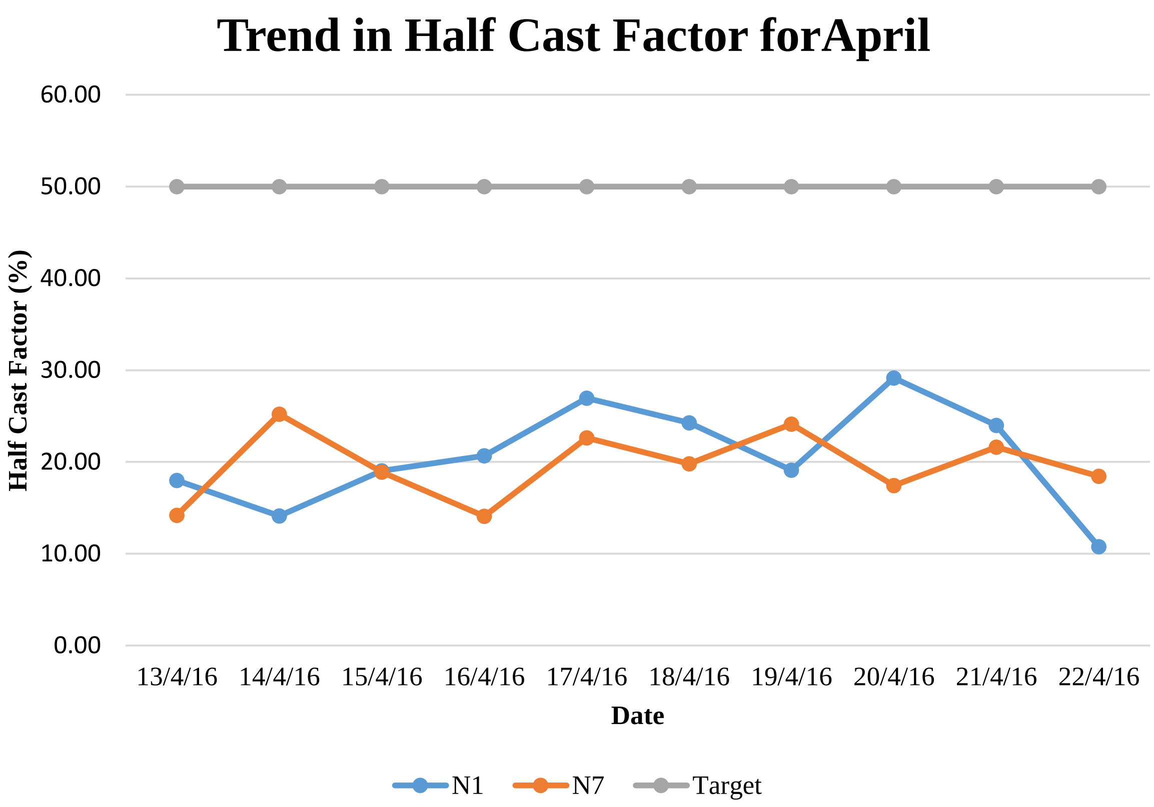

Figure 6.3: April trend in half cast factor

6.7.3 April Powder factor

Powder factor refers to the mass of explosives used to break one tonne of rock. Figure 6.4 shows the trends in powder factor for the 2 sections in April 2016. The actual powder factor for the month of April 2016 averaged 1.704kg/t. This was above the target of 1.66kg/t. The powder factor was above the desired as a result of the aforementioned lower than expected advances.

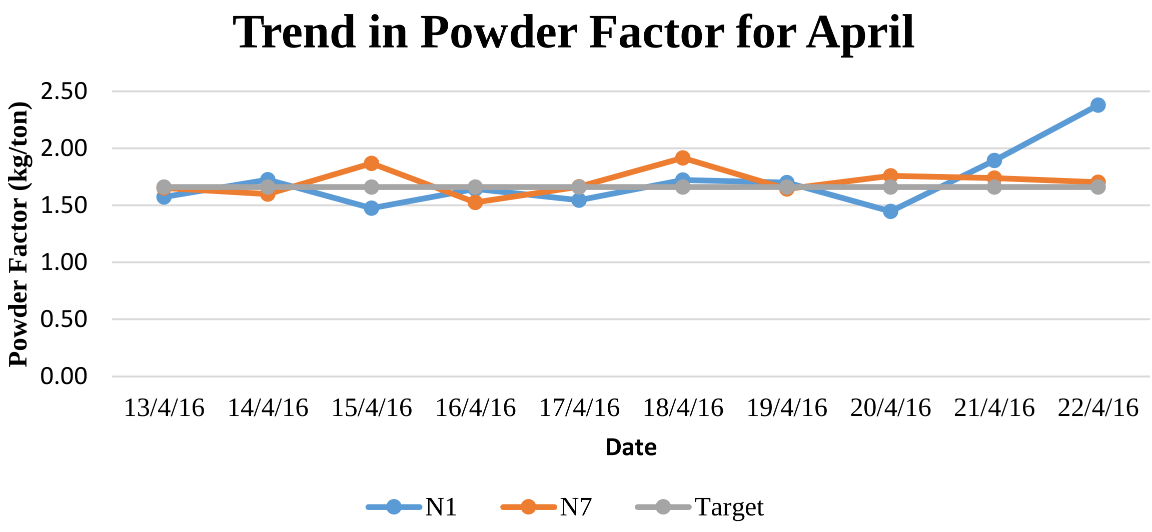

Figure 6.4: April trend in powder factor

6.8 May 2016 Results and analysis

The advance per blast, half cast factor and powder factor results for the month of May 2016 are discussed in this section.

6.8.1 Advance per blast

The linear advance increased to 2.95m during the second month of the trial, against a budget of 2.8m. Figure 6.5 shows the advance per blast trend during the month of May.

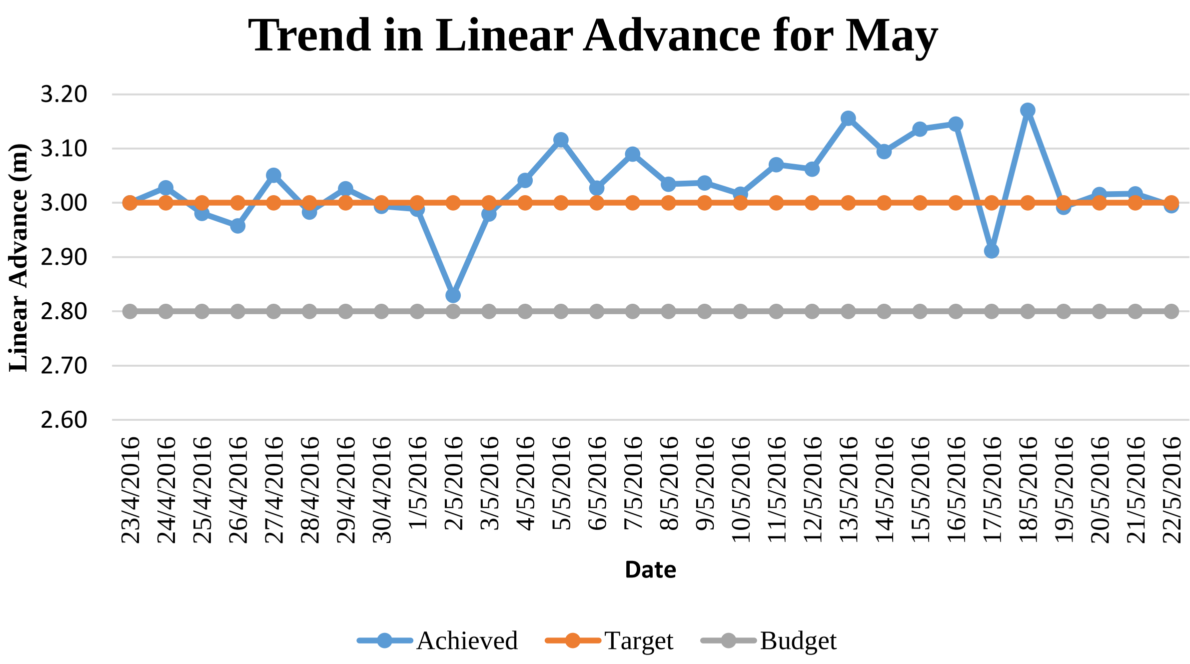

Figure 6.5: May trend in linear advances

6.8.2 May 2016 half cast factor

Figure 6.6 shows the trend in half cast factors during the month of May. The half cast factor figures were again below the targeted 50% as a result of the prevailing geological discontinuities.

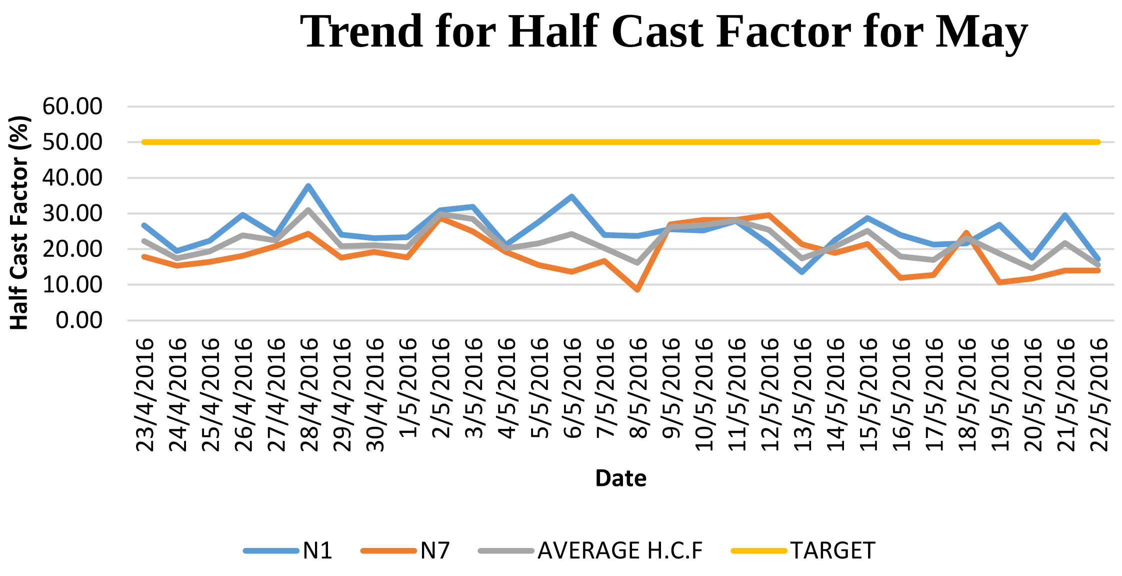

Figure 6.6: May trend in half cast factor

6.8.3 May 2016 powder factor

Figure 6.7 shows the trend in powder factor during the month of May against a target of 1.66kg/t. The average powder factor was still above budget.

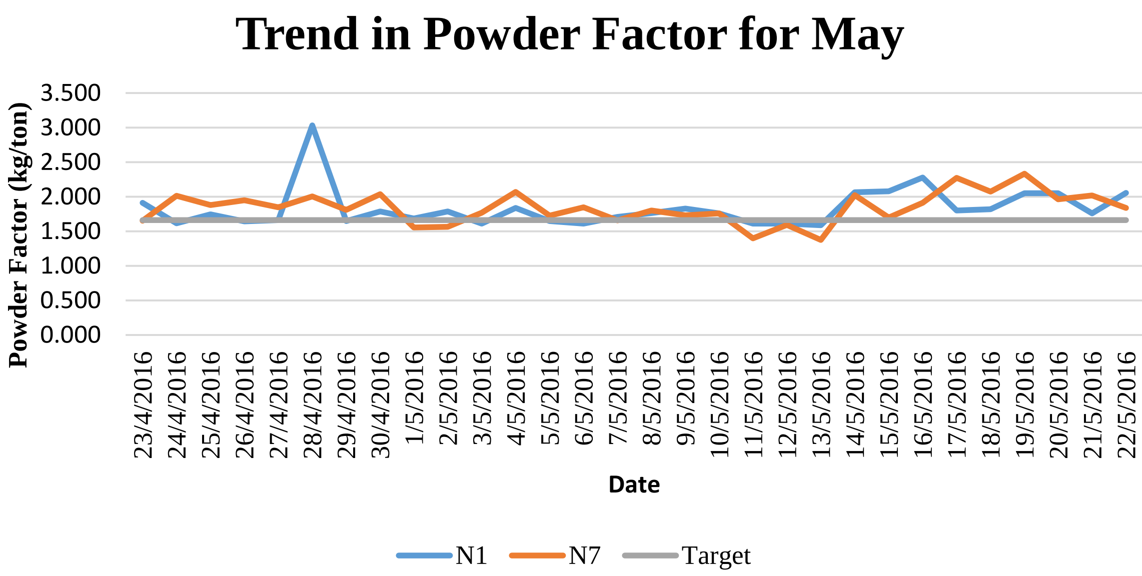

Figure 6.7: May Trend in powder factor

6.9 June 2016 results and analysis

Described in this section are the June results and the corresponding analysis of the advance per blast and half cast factor.

6.9.1 Advance per blast

The linear advance increased during the month June to 3.01m against a budget of 2.8m. Figure 6.8 shows the advance per blast trends during the month of June 2016.

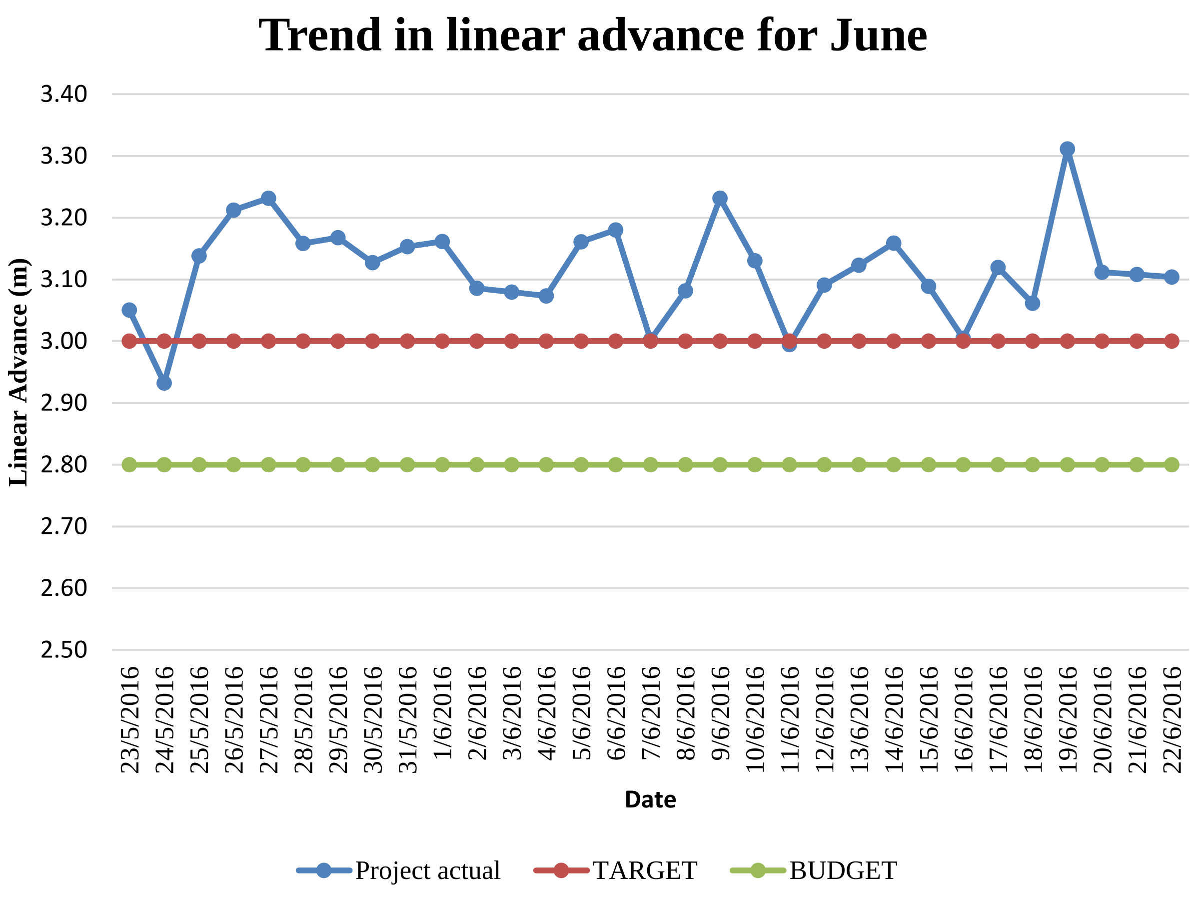

Figure 6.8: June trend in linear advances

6.9.2 June 2016 half cast factor

Figure 6.9 shows the trend in half cast factors during the month of June 2016. The half cast factor figures were again below the targeted 50% because smoothwall blasting was not yet implemented.

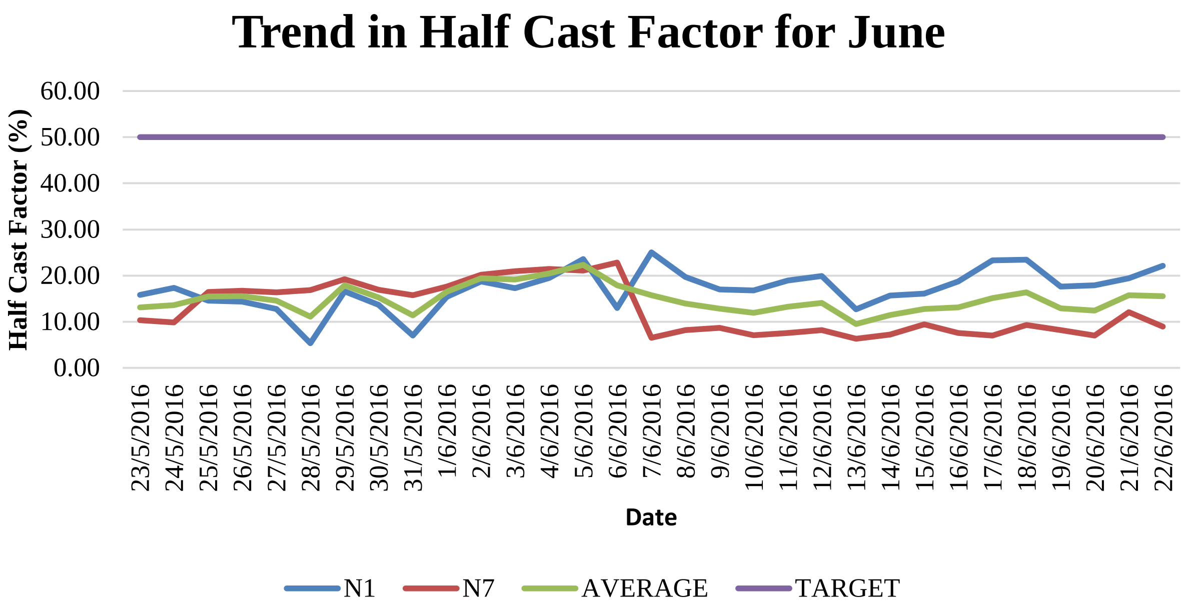

Figure 6.9: June trend in half cast factor

6.10 Summary of results

Linear advances for the bulk emulsion trial performed above the targeted advance of 3.0m to an average of 3.07m for the whole trial, this is 95.9% of the drilled length of 3.2m. The results are as shown in Figure 6.10.

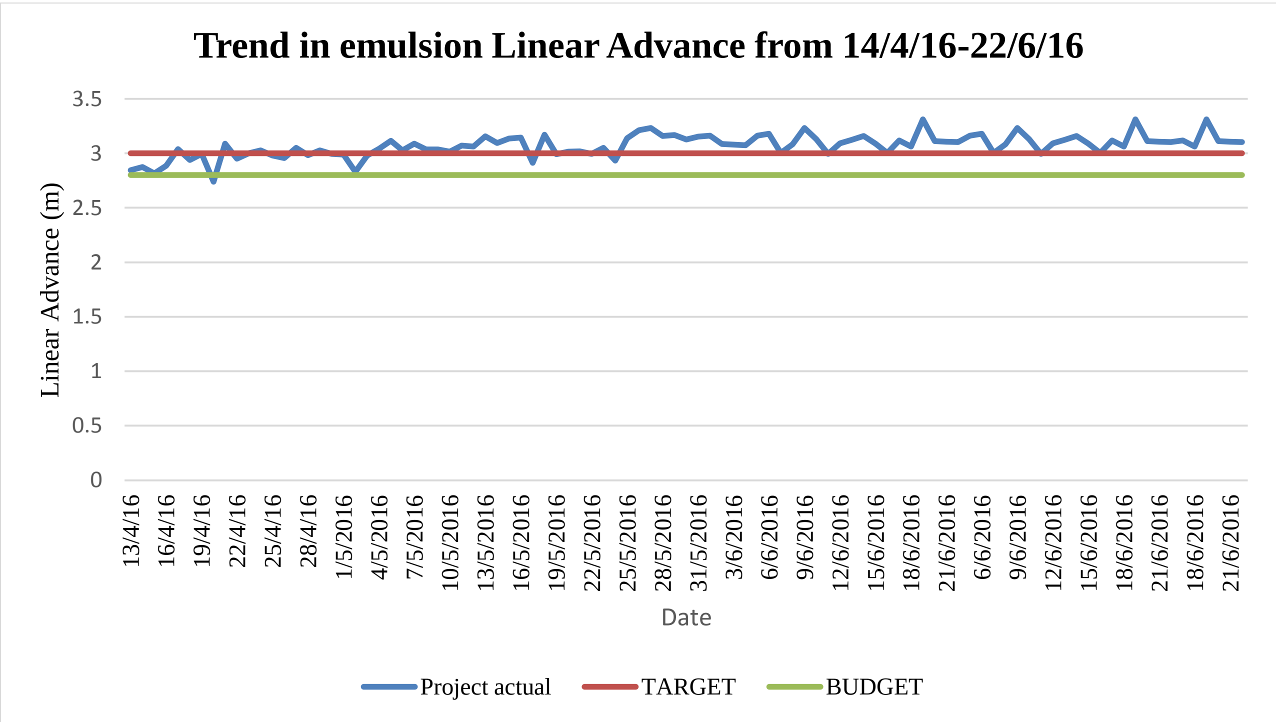

Figure 6.10: Trend in linear advances for the whole emulsion trial

During the same trial period, the average advance attained using ANFO was 2.783m. This figure is slightly lower than the budgeted advance but significantly lower than the targeted advance of 3m. From the two graphs (Figure 6.10 and Figure 6.11), emulsion explosives resulted in better advances hence improved tonnage.

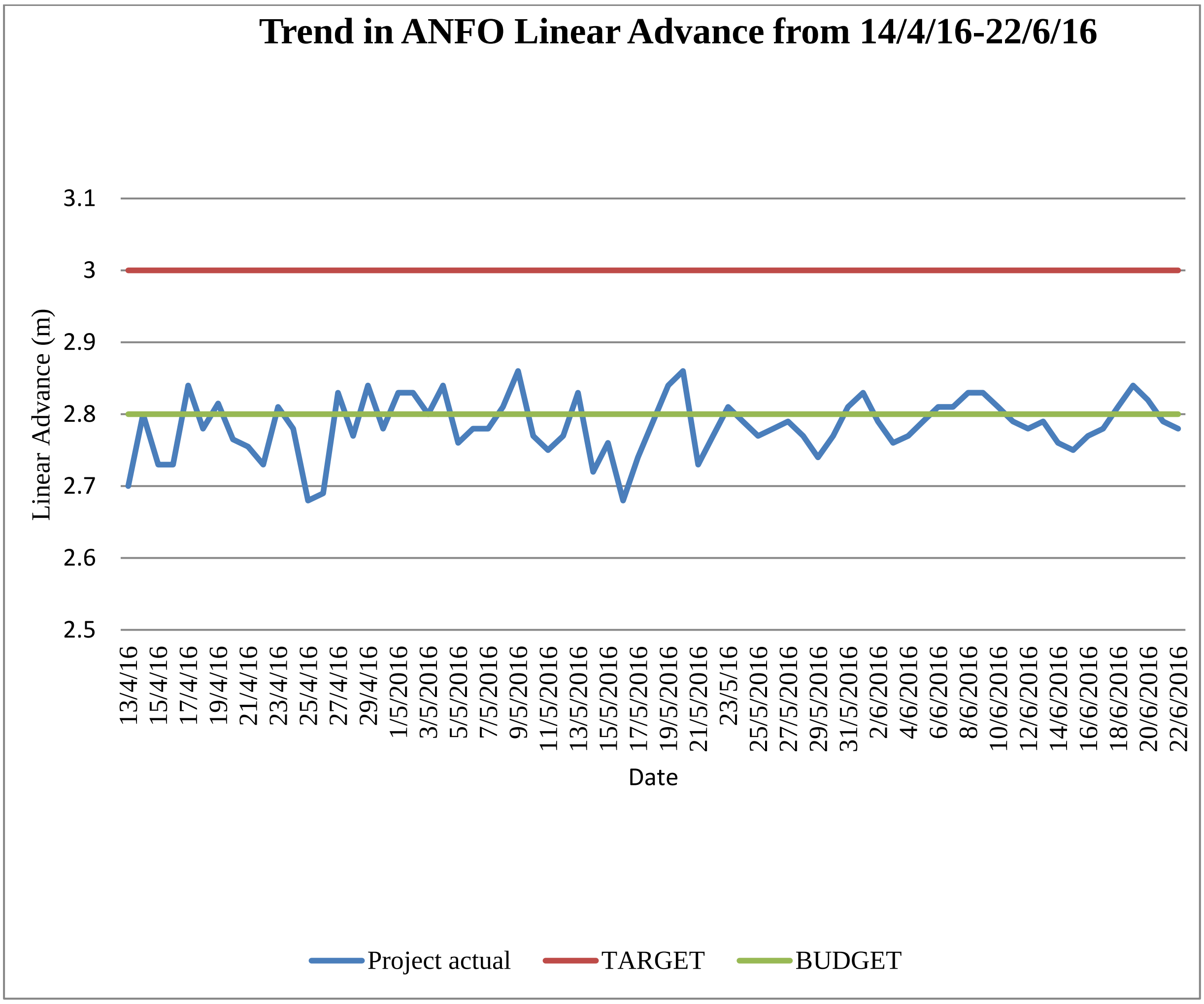

Figure 6.11: Trend in linear advances for ANFO

The average powder factor for the whole emulsion trial was 1.83kg/ton which is above the target of 1.66kg/ton as shown in Figure 6.12. The difference is attributable to the subsequent decrease in the span as a result of geotechnically poor ground conditions.

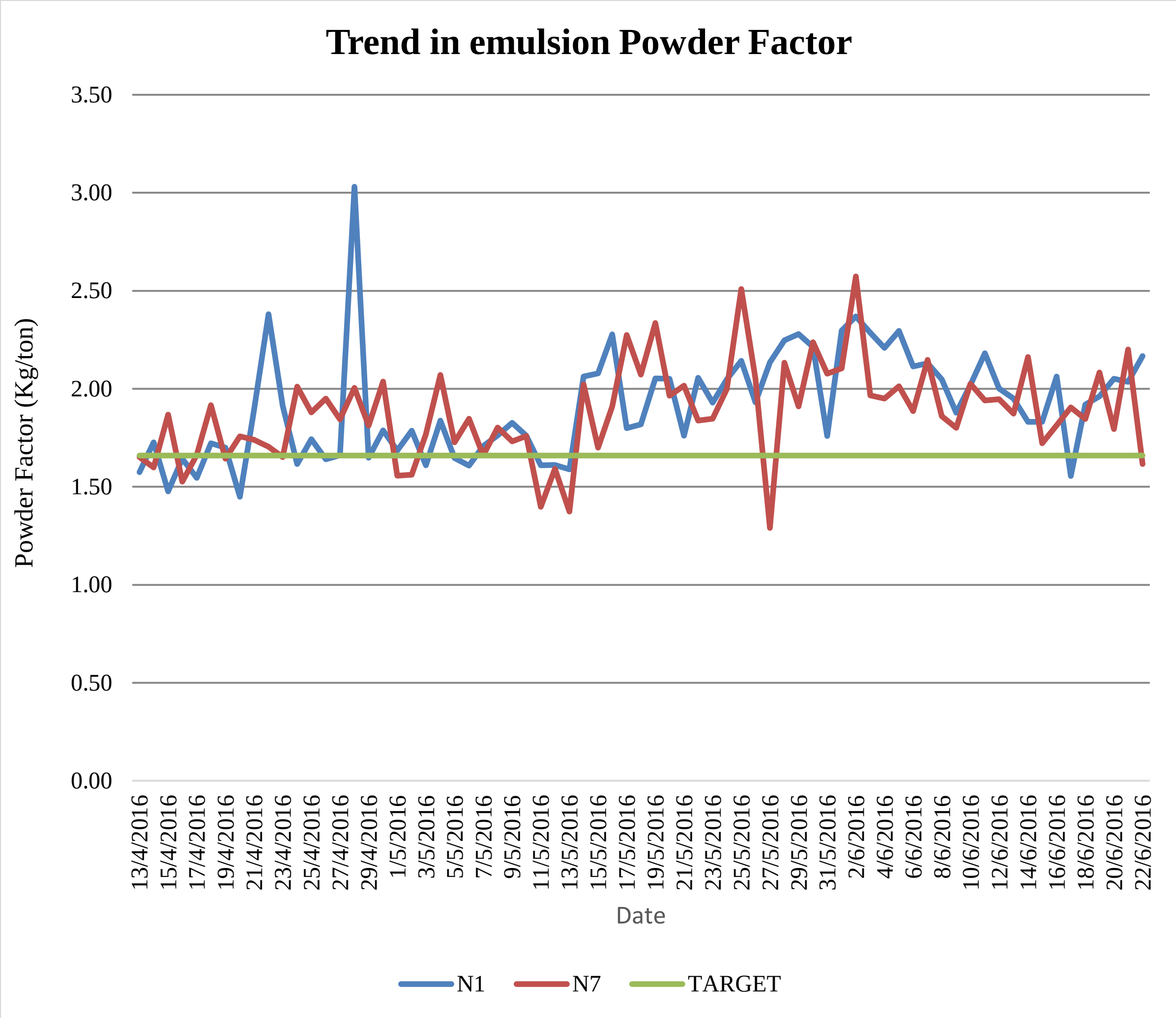

Figure 6.12: Trend in powder factor for the whole emulsion trial

The average powder factor for ANFO was 1.76kg/ton during the same period, which is above the target of 1.66kg/ton as shown in Figure 6.13. This powder factor is however less than the one attained using emulsion explosives. In terms of explosive quantity, less ANFO was used to fragment the same sized rock as compared to emulsion explosives; however a full cost benefit analysis is given in section 6.12.

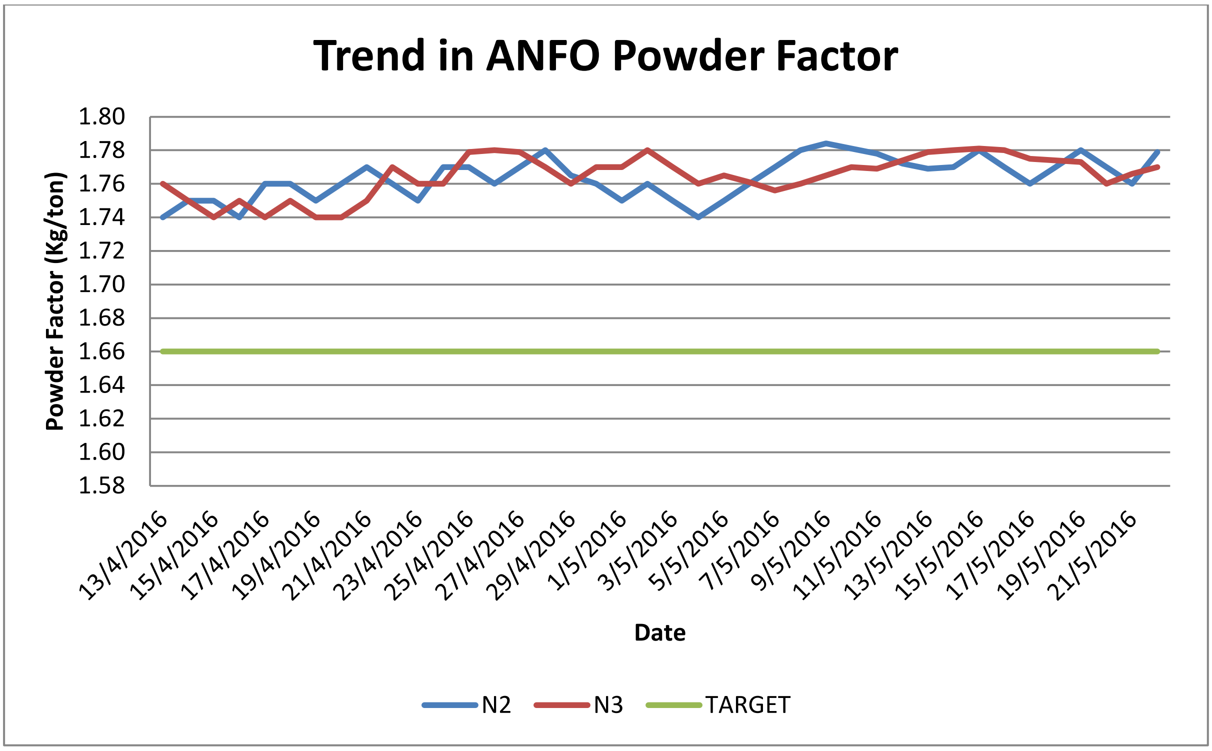

Figure 6.13: Trend in powder factor using ANFO

Figure 6.14 shows the targeted half cast factors for the whole emulsion trial. The targeted half cast factor was not being achieved due to bad ground conditions and the lag in purchasing polypipes for smoothwall blasting. The average half cast factor for the whole trial was 20%. Half cast factor is vital in the determination of stoping overbreak. Half cast factors greater than 50% shows that there is minimum overbreak hence less ore dilution and increased safety. Singh (1992) pointed out that blasting can be described as a destructive process and the effects of blast damage are deleterious to both safety and productivity. The average half cast factor for the trial is less than 50%; however, there is great improvement on excavation profiles when compared with the previous results from ANFO. More work is still being done to improve the half cast factors to a minimum of 50%.

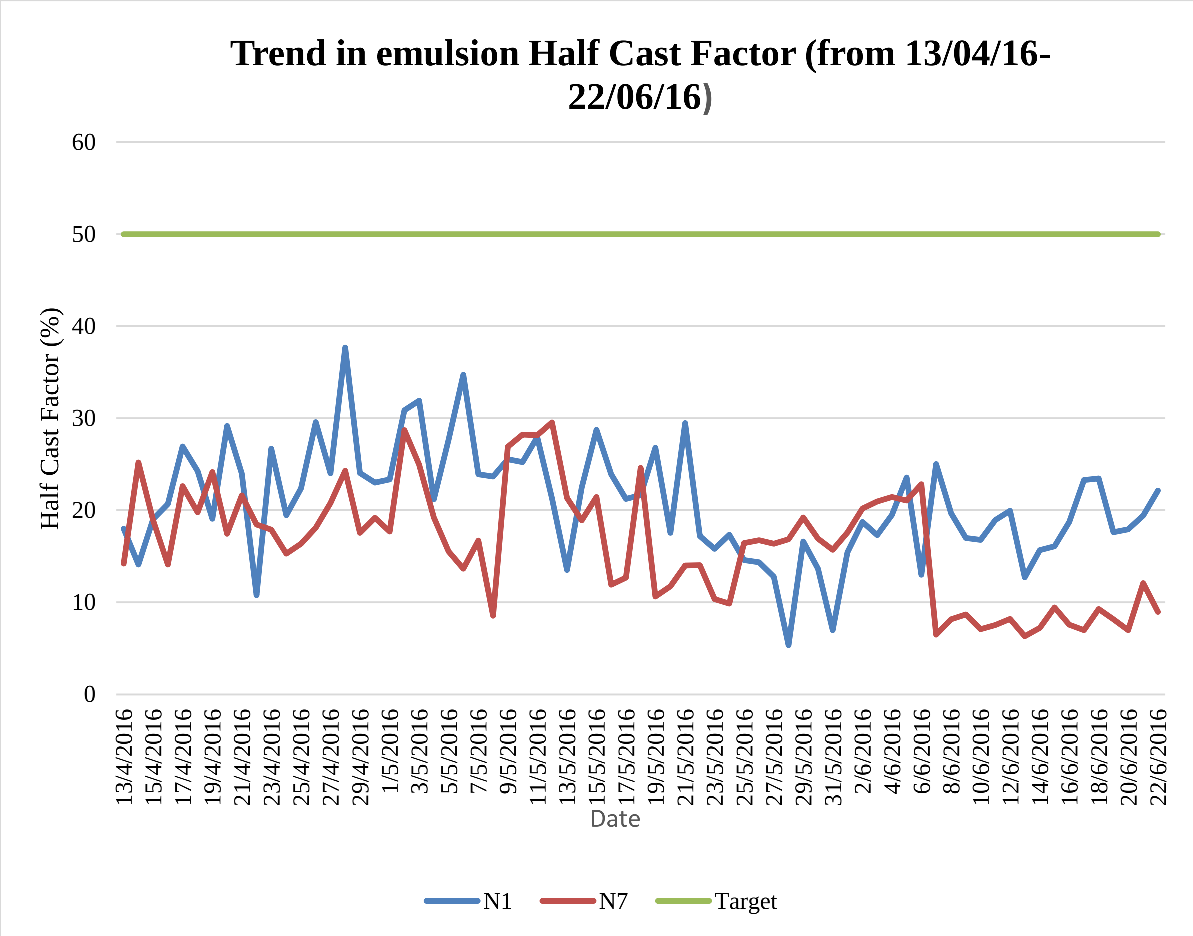

Figure 6.14: Trend in Half Cast Factor for the whole emulsion trial

During the same trial period, the average half cast factor using ANFO was 7.60% as shown in Figure 6.15. This number is significantly lower than the targeted half cast factor. Comparing the two half cast factors of emulsion and ANFO, it can be concluded that better and smooth profiles were produced in areas using emulsion as compared to areas using ANFO. This deviation arises from the excessive heaving property of ANFO which tends to damage the excavation perimeter resulting in fewer barrels noticeable. In terms of hangingwall stability as well as pillar overbreak, emulsion explosives produced better results hence improved safety.

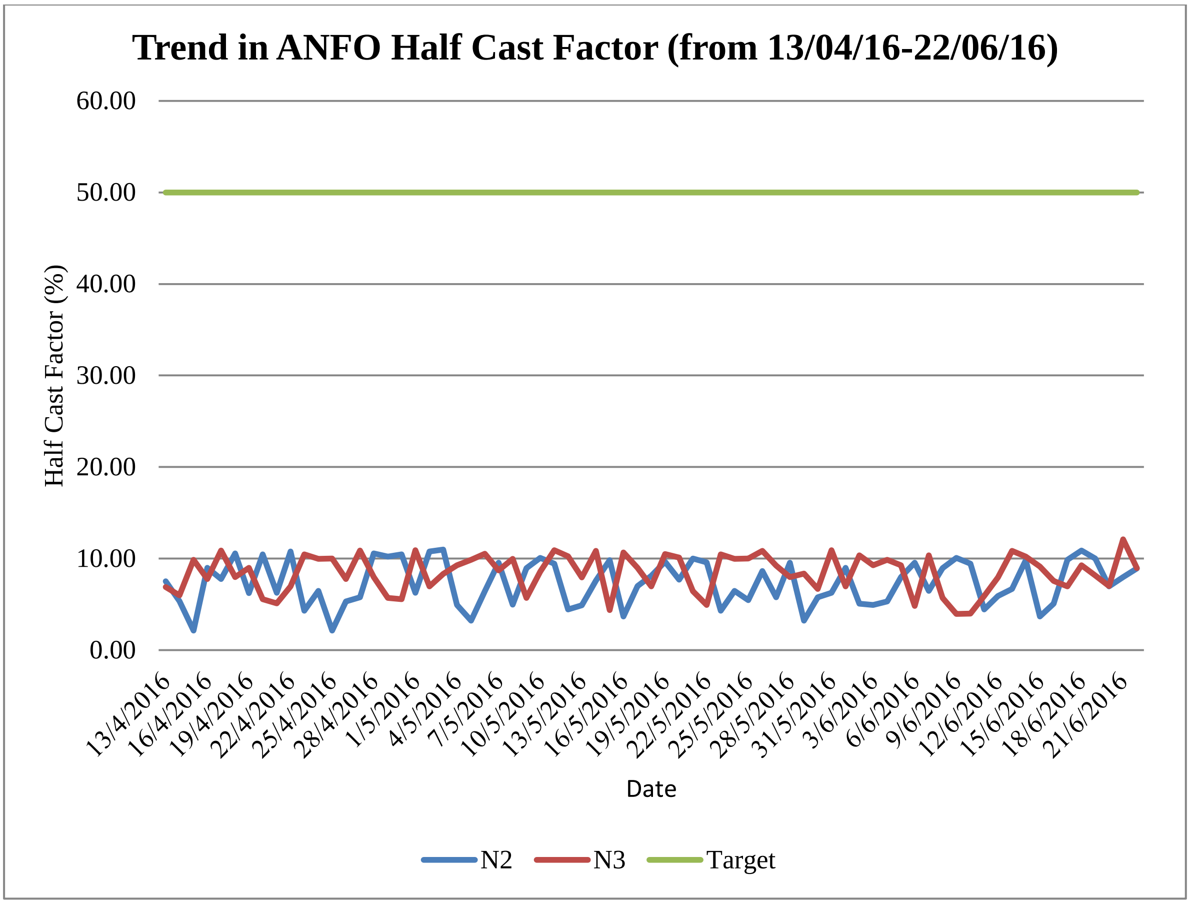

Figure 6.15: Trend in Half Cast Factor using ANFO

6.10.1 Blasting profile

Minimum overbreak into the hanging wall, footwall, and sidewall were observed during emulsion charging compared to ANFO loading. Visible barrels were an evidence of reduced blast over break (see Figure 6.16). Profiles that were not smooth were observed in very poor grounds, thus regions with a tunnelling index less than 0.4. There was reduced throw from emulsion explosives compared to ANFO, which actually lessened LHD lashing and scraping time.

Figure 6.16: Visible barrels after charging with emulsion

6.10.2 Fragmentation

Bulk emulsion proved that 97% of the blasted ore can pass through the 400mm×400mm grizzly apertures without difficulties compared to 94% from ANFO charging. This is against a budget of 95% passing.

6.11 Project findings

The project work done was to a greater extent conclusive to the following findings:

1. *Advance* improved from an average of 2.80m using ANFO to an average of 3.07m using emulsion explosive. This increased the blasted tonnage from 105t to 113t per 6m x 2m end.

2. The *average time taken to charge* an end also decreased from an average of 23 minutes using ANFO to an average of 13 minutes, saving an average of 10 minutes per end which translates to 1 hour within the mining cycle per team, since 6 (6m gulleys) ends are charged per day.

3. The increase in emulsion explosive quantity utilized per end increased the powder factor from 1.66kg/t to 1.83kg/t despite the increase in advances.

4. Emulsion produced an average *half cast factor* of 20%, which was below target. Observations made show that the bord profiles were fairly smooth as compared to those produced with ANFO.

6.12 Comparison of explosives

Table 6.2 compares ANFO explosives with emulsion. A comparison was drawn in a bid to choose an explosive type with the most merits.

Table 6.2: Emulsion vs ANFO

Parameter	**Emulsion**	**ANFO**
Cost per blasted tonnage	High	Low
Capital cost	High- MCU is expensive to buy	Low-ANFO loader is cheaper
Advance	Average 3.0m	Average 2.8m
Powder factor	High	Low
Productivity	Improved productivity	Decreased productivity
Safety	Improved safety as a result of less bad hangings generated	Generates a lot of bad hangings
Charging time	Less charging time 13mins per end	More time averages 23mins per end
Water resistance	Excellent water resistance	Dissolves easily
Overbreak	Less overbreak hence minimum PGM dilution	More overbreak resulting in PGM dilution
Throw	Reduced throw hence less LHD loading time	Greater throw hence more tramming time

The scaling time of bad hangings dropped remarkably with emulsion explosive. The time decreased from an average of 35 minutes to 11 minutes. This was accounted for due to a reduction in the generation of bad hangings brought about by the use of emulsion. The derivation of cost per blasted tonnage for a 6m gulley is shown in Table 6.3.

Table 6.3: Cost per ton blasted of a 6m gulley

Consumable	Units required	Unit cost ($)	Total cost of units used ($)
ANFO (25kg bag)	7	19.98	139.86
EZ stopper (4.2m)	30	1.43	42.9
Dual Dets (8.4m)	9	2.08	18.72
Megamite cartridges (200mm x38mm)	48	0.45	21.6
Safe start	1	4.19	4.19
Trunkline	1	1.7	1.7
Twisted cable	1	7.32	7.32
Emulsion (kg per end)	208	0.89	185.12
ANFO Total cost (6m round)			**236.29**
Emulsion Total cost			**281.55**

The expected tonnage from a 6m gulley after charging with ANFO is 105.84 tonnes; therefore the explosive cost per blasted tonnage is $2.23 per tonne. The expected tonnage from a 6m gulley after charging with emulsion is 113.4 tonnes, which corresponds to an explosive cost per blasted tonnage of $2.48 per tonne. The project was aimed at improving safety through reduction of FOG influenced accidents. The implementation of bulk emulsion offers additional improvements in some areas that the mine has been facing challenges such as minimum advance and poor fragmentation. Emulsion explosives pose high operating costs and capital cost as compared to ANFO, however, the performance results attained through this trial show that the use of emulsion as the main column charge is a worthy sacrifice which will yield benefits over a certain period of time. The high OPEX and CAPEX cannot be compared to the savings that the mine would have realized in achieving the goal of zero harm through avoiding or minimizing FOG related accidents.

6.13 Conclusions

The results gathered and analyzed showed that emulsion explosives are beneficial but high operational and capital costs down-weighs them. The author recommends the mine to take up emulsion in solution to the problem which prompted this research since this explosive promotes safety at higher productivity in terms of tonnage output. A cost benefit analysis clearly pinpoint to the implementation of emulsion as a result of optimized KPIs. By making the effort to implement cautious blasting practices in tunnel development the amount of overbreak is limited, which improves rock mass conditions and support integrity with the spin-off of reduced support and remediation costs thereby better project feasibility.

CHAPTER 7: RESEARCH BENEFITS

In order to have a sustainable development, there must be a balance in profit, safety and social responsibility (Wood, 1987). This research was carried out to improve mine safety and productivity. The research covers extensive work on one mine on the Great Dyke but the research benefits can be extended to other mines which are in the same geological location, mining the same type of deposit using the same mining method. Adequate support will benefit mines on the Great Dyke since the rate of rockfalls will be reduced. Rockfalls will lead to lost time injuries or fatalities which will affect mine production. Lost time injuries (LTI) at the mines result in absenteeism which lowers production. The employer is obliged to exercise a high level of care to all workers. Additional medical bills will add on to the company costs hence, affecting the profit margins. Moreover, falls of ground will result in equipment being buried by falling rocks hence more cost on machinery. The stability of excavations together with suitable mining method will improve safety and productivity of the mine. A high production will result in high revenue generation, hence economic growth.

From the analysis of results, the current support system is not adequate hence the probability of support failure is high. Support failure, together with overbreak, necessitates the use of additional support which is unplanned for. It is therefore viable to implement the designed optimum support system taking into account the cost benefit analysis to scrap off unplanned costs that may arise. In addition, overbreak affects the grade of PGMs, which lowers the metal content to be produced. Dilution of ore affects the mean head grade to be fed to the processing plant, hence compromising the financial component of the mine. The key performance indicators in the mining industry are safety, production and costs. The optimisation of these KPIs in relation to this research is described in this section.

7.1 Safety

A safety component for all underground hard rock mines is intrinsic and standing support systems. Listed herein are safety related benefits deduced from the major findings of the research.

- The recommended roofbolting system will result in enhanced safety due to a decrease in probability of tendon failure as depicted in Figure 7.1.
- With the proposed pillar design system, there will be a significant upsurge of safety factors for both barrier and in-stope pillars.

- The adoption of the new design rock mass strength in anisotropic jointed rock mass will lead to mobilization and conservation of the inherent strength of the rock mass since it becomes self-supporting.
- Barring down in poor ground conditions using hand held pinch bars was seen as a risky and time consuming exercise hence the use of mechanical scalers can give improved safety.
- Due to the unpredictable unravelling of rocks with the current practices, workers will not produce the best results due to current safety concerns which do not provide optimum working conditions.
- There will also be improved safety in areas with negative pillar infringement as a result of addition of confinement support as recommended. The current trend of scaling pillars can result in underground collapse, resulting in loss of lives and also damage to high capital equipment.
- The proposed solutions will also lead to less environmental impacts. Sound pillar design will result in minimum surface subsidence since the issue of uncontrolled pillar collapse will be reduced and consequently minimum rehabilitation costs.

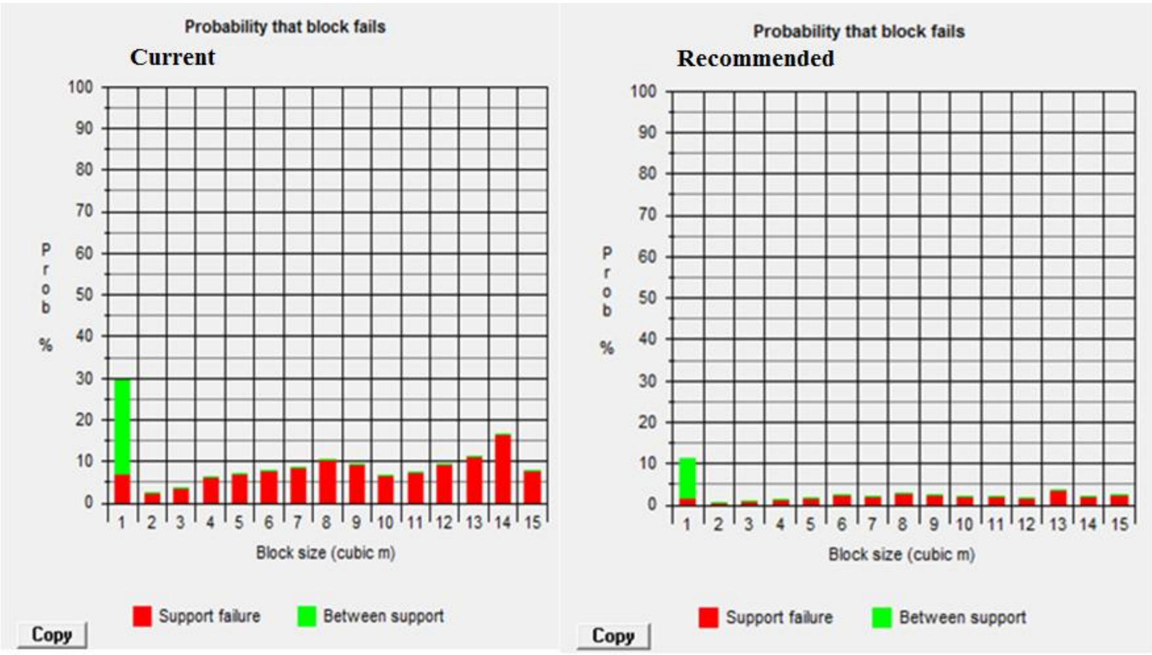

Figure 7.1: Current support versus recommended support numerical analysis

7.2 Production

Face advance improved from an average of 2.80m using ANFO to an average of 3.07m using emulsion explosive. A better advance indicates that the blasted tonnage per end will be approximately 113t per gulley as opposed to the existing budgeted tonnage of 105t per gulley. Total tonnage that can be gained by the use of emulsion is approximately 193 536t per annum. Table 7.1 illustrates the gained tonnage in monetary terms.

Table 7.1: Emulsion cost benefit analysis

Tonnage gained	=113-105 = 8t
Total number of blasted ends/ day	= 6ends/ section × 12 sections =72
Total tonnage gained/day	=72×8 =576tonnes
By using an average of 28 working days/ month and 12months per year	Blasted tonnage gained/year =28×576×12 =193 536 tonnes
Hoisted tonnage gain	=0.95×193 536 =183 859t
Platinum grams present (1.8 g/t feed grade)	= 183 859 ×1.8 =330 946g
Platinum grams recovered	=330 946 × 0.78 =258 138g
Total Pt. sales (using a Pt. price of \$1037.70/ oz.) (KITCO, 2017)	=258 138 × 1031.7/31.1 = \$ 8 563 385
Cost/ ton difference between emulsion and ANFO	=2.48-2.23 =\$0.25/t blasted
Additional explosives cost per year with the adoption of emulsion is \$0.25/tonne	= 0.25 × 193 536 = \$48 384
Platinum revenue gained per year (assuming milling costs will be covered by other commodities within)	=\$ 8 563 385 - \$48 384 **= \$8 515 001**

A maximum of 5% ore loss is anticipated during ore logistics in form of boulders, dust etc. hence 95% of the blasted ore was used in the calculation. The planned platinum recovery at the studied mine is 78%.

The research findings and recommendations will also improve production because of the following reasons:

- Emulsion requires less time for charging (time reduction from 23mins to 13mins) which equates to 10minutes gain per end. Since 6 ends are charged and blasted per day by one section, a total of 1 hour will be gained by charging with an emulsion unit. The time gained can be spread on other production activities hence meeting or exceeding the call.
- Barring down was seen as a stumbling block in the production cycle as it was time consuming. Less time will be required for barring down due to less bad hangings as a result of the implementation of smoothwall blasting. Barring down time will reduce to an average of 11 minutes from a budgeted 35 minutes, resulting in improved productivity. Failure to meet production targets results in lost production, hence less profit to the mine.
- In case of accidents, the section will be closed down for routine accident investigation which will take production operations to a halt. This research enhances mine safety, hence mitigating rock related accidents.
- Reserves in poor ground conditions will be exploited through a cost effective approach. Without the general findings and the implementation of the proposed design, valuable reserves will be locked up as pillars.

7.3 Costs

Mining companies must remain in the lowest cost quartile in a bid to provide business growth and also in order to produce superior returns to various stakeholders. The following points from this research pinpoint the cost cutting initiatives:

- Apart from improved safety, the proposed tendon support system will reduce support costs by $870 670 per year.
- There will be less litigation costs as a result of reduced number of accidents and incidents.
- Capital costs will reduce due to less frequency of equipment damage that may arise as a result of falls of ground.

- Less capital will be required in retrofitting the designs. Underground collapse as a result of failed pillars requires capital to reinstate the damaged infrastructure as well as to open access ways.
- Less opportunity costs due to minimum PGM dilution will be incurred.

The findings of this research together with the proposed recommendations will integrate safety measures, value addition and economic benefit to the mines on the Great Dyke. Mining is the backbone of the Zimbabwean economy hence safe work practices will lead to zero harm and economic development. The benefits of this research are in line with the global mine key performance indicators of production, safety and costs. The results of this study will therefore give long lasting benefits to the platinum mines on the Great Dyke as they can safely extract the remaining resources locked in geotechnically challenging ground conditions at a minimum cost. The study will therefore meet the vision of many companies of remaining in the lowest cost quartile of platinum producers and to provide superior returns to the stakeholders.

CHAPTER 8: CONCLUSIONS AND RECOMMENDATIONS

Safe mining practices and installation of adequate support give stable excavations. The research reviewed the current support systems used in geotechnically poor ground conditions in a bid to improve the stability of excavations of platinum mines on the Great Dyke. The study involved work on one mine; however the findings can be extended to other mines in the same domain since they use the same techniques to exploit the resources. From the findings of the research, it can be concluded that the current support systems used in bad ground conditions are inadequate. The presence of reef subparallel planes at depths greater than 1.8m implies that the current tendons used will not be able to support the prevailing keyblocks in the area of research. The presence of sympathetic joints associated with the faults has resulted in unstable keyblocks. The current type of explosives used has resulted in unravelling of rocks by generating a lot of gases which penetrate and widen the joints. The unravelling of rocks will consequently add waste rocks to the ore and also leads to stoping overbreak. Ore dilution and larger stoping widths will occur as a result of overbreak, which will decrease pillar factor of safety. A trial of the proposed explosive indicated that the use of emulsion will yield substantial benefits to the company. The contemporary monitoring systems of robbed pillars need to be improved as it only focusses on visual cracks on pillars instead of more technologically advanced remote monitoring systems. Where it has been determined that a pillar is robbed, additional confinement support can be utilised to conserve pillar strength.

By using different rock mass classification methods, it was deduced that the rock quality where the research was carried ranges from poor to very poor. The predetermined ground conditions thus require redesigning of pillars and tendon support system to improve productivity and safety. The proposed pillar design considered the revised DRMS that matches the existing ground conditions using Laubscher MRMR classification system. The use of one system in rock mass classification was seen as inadequate because every system has its own limitations. Most joints at the research area were assumed to be dry but groundwater can wet some joints, hence weaken the cohesion which will increase the frequency of unstable keyblocks. Barring down using pinch bars was seen as a risky and time consuming exercise in such areas since the barring team will be exposed to unstable keyblocks with a higher probability of failure. Because of frequent unstable blocks, barring down using pinch bars consumed more cycle time, which has led to most sections failing to meet their production targets since less faces will be prepared. Failure to meet the desired production targets implies that lower profit margins will be made, hence inferior returns to the stakeholders which will be against the industry goals.

Taking into considerations the safety required in bad ground conditions and the company profits, a cost benefit analysis was done and the author considered some recommendations to improve both safety and productivity but also remaining in the lowest cost quartile. From the evaluation of pillars and tendon systems, the support systems need improvements in order to achieve zero harm. From the analysis of the research, the following recommendations are proposed for optimisation of the companies' KPIs:

- A study of the new tendon support system designed by the author is recommended. Longer roofbolts of 2.2m length spaced at 1.2m by 1.2m are suggested compared to the current roofbolts of 1.8m length spaced at 1m by 1m. Shallow dipping planes occur at depths greater than 1.8m, hence the use of longer bolts will clamp overlying layers resulting in reduced support failure. The average fallout height determined by the author was 1.8m, hence longer roofbolts will give an improved bond length. In addition to improved safety, the implementation of the new roofbolt system will reduce costs by \$870 670 per annum. Considering the cost benefit analysis, a new roofbolt support system of 2.2m length spaced at 1.2m by 1.2m is recommended by the author in bad ground conditions because of its improved safety, reduced costs and lower support density.
- A substitute for ANFO explosives is recommended to minimise rockfalls. ANFO explosives generate a lot of gases which widen the joints. The use of ANFO leads to stoping overbreak which compromises both safety and production due to a decrease in safety factors of pillars and an increase in ore dilution. The author recommended the mines to use emulsion in jointed rockmasses. A trial of emulsion explosives showed enhanced results in terms of safety and productivity. Charging time decreased, meaning there will be an increase of in-shift available time hence the mine will be able to meet the call. Moreover, the use of emulsion will result in production gain as a result of improved advances. Production for the studied mine will increase by 193 536 tonnes of blasted ore which is equivalent to \$8.5 million revenue per annum. Re-entry time will be reduced as a result of minimum generation of noxious gases.
- Additional support is required where there is pillar robbing. The use of timbers, pillar bolting and confinement such as wire mesh and shotcreting is recommended where there is negative pillar infringement to improve strength. Only shotcrete is applied to monitor

cracks but this may result in collapse of pillars since no additional support is used. Additional support is a blow to a company as it escalates costs; however, pillar failure results in mine closure or collapse of hangingwall which implies more capital costs. Small isolated pillars are implausible to affect the overall stability of the mine, however, a cluster of robbed pillars failing may affect the overall stability of the mine. By taking into account all these factors, the author thus recommended additional support in areas where there is pillar robbing.

- The author recommended the mines to use more than one system for rock mass classification. Most mines use just Q rating which has its own limitations. The author thus recommended the mines to use RMR and MRMR in conjunction with the Q system to get a clear indication of the rock mass quality. More systems are required for comparison as this will give a clear indication of the rock quality. Numerical modelling is also recommended by the author since the use of MAP3D and JBlock will show the stress distribution and the probability of occurrence of unstable blocks respectively. Having identified the area which is highly unstable and knowing the probability of falling of keyblocks, safety measures can be put into place, hence minimising the potential falls of ground.
- The use of hydrological surveys is recommended to determine joint water conditions. All joint sets at all platinum mines on the Great Dyke are assumed to be dry based on exploration results. Wet joints weaken the cohesion which may result in falls of ground. Hydrological surveys will give a true reflection of the authentic groundwater conditions within joints thereby increasing safety by designing for the worst rock conditions.
- The use of remote controlled mechanical scalars is recommended in bad ground conditions since they improve both safety of workers and productivity. The use of pinch bars when barring down will expose the workers to unstable rocks. Mechanical scalars are used at some mines within the same geological environment in bad ground conditions. They have proved to be the safest equipment when dislodging bad hangings especially in highly fractured grounds. Apart from being a risky task, barring down using pinch bars was seen as time consuming which has led to fewer faces being prepared. Fewer faces being prepared result in a failure to meet production targets hence the use of mechanical scalars will improve productivity.
- The author recommends redesigning of pillars which will give improved safety and production. The structural data was converted to suitable Laubscher MRMR classification

system and the DRMS was determined to be 53 MPa, which is less than the 63 MPa currently in use. It was noted that production is low in bad ground conditions due to ore being locked up in pillars as illustrated in regional pillar positive infringement. Bad ground conditions together with the explosives used have led to slender pillars shown by most sections failing to maintain the designed height. Overbreak therefore affects the factor of safety which will result in pillar failure, hence the author recommended the redesigning of pillars for improved safety.

- Smoothwall blasting is recommended in bad ground conditions to minimise excavation damage hence maintaining the desired stoping height. Defined excavation profiles can be mined through the implementation of smoothwall blasting to protect the hangingwall and consequently minimum PGM dilution. Adequate training of workers and close monitoring is required when drilling in order to have effective operations.
- FOG lights (extensometers) are critical to monitor ground movement in poor ground areas, especially large excavations. Pillars at tips should be monitored using FOG lights for any ground movement and must be confined using straps. The use of FOG lights should also be extended in areas with negative pillar infringement to monitor ground movement which can result in underground collapse.
- Advance geological drilling is recommended in poor ground conditions whereby core is drilled in the direction of advance and 2-3 months cover is maintained. This gives geotechnical information ahead of the face. Advance drilling assist the mine in planning ahead so as to minimise stoping delays upon encountering geological discontinuities. The mine will be proactive in that case since the design and approach to economically exploit resources in such areas can be put in place well in advance.
- A borehole camera can be used for hangingwall inspection through drill hole. A borehole camera gives an actual picture of the shear zones. Monitoring of ground strata is imperative to mitigate rockfall related accidents.
- Staggering of pillars is critical in poor ground conditions. Staggering of pillars reduces the catastrophic consequences that may be spread over to the neighbouring ends, hence causing pillar run. It is also practiced to limit the long exposure of the structures within a panel.
- Routine pillar monitoring must be implemented on the main access routes to determine long term pillar performance. If damage occurs or alternatively is projected by means of numerical or empirical analysis, appropriate instrumentation will be installed to monitor

the anticipated movement or deformation. Where easily accessible, manual monitoring will be done to facilitate visual evaluation as well as crack monitoring. Where not, some form of remote monitoring must be implemented through the use of precise levelling, strain gauges, closure meters and borehole camera inspections.

- Risk evaluation models such as RiskEval must be considered by mines mining in geotechnically challenging ground conditions. Currently the mines on the Great Dyke are not using such tools. The use of these tools assist the mines in risk assessment and quantification of the expected injuries and economic consequences on the whole mining business arising from falls of ground associated with pillar failure and tendon support failure. This risk assessment includes the degree of, nature and location of the expected injuries within the mining area.

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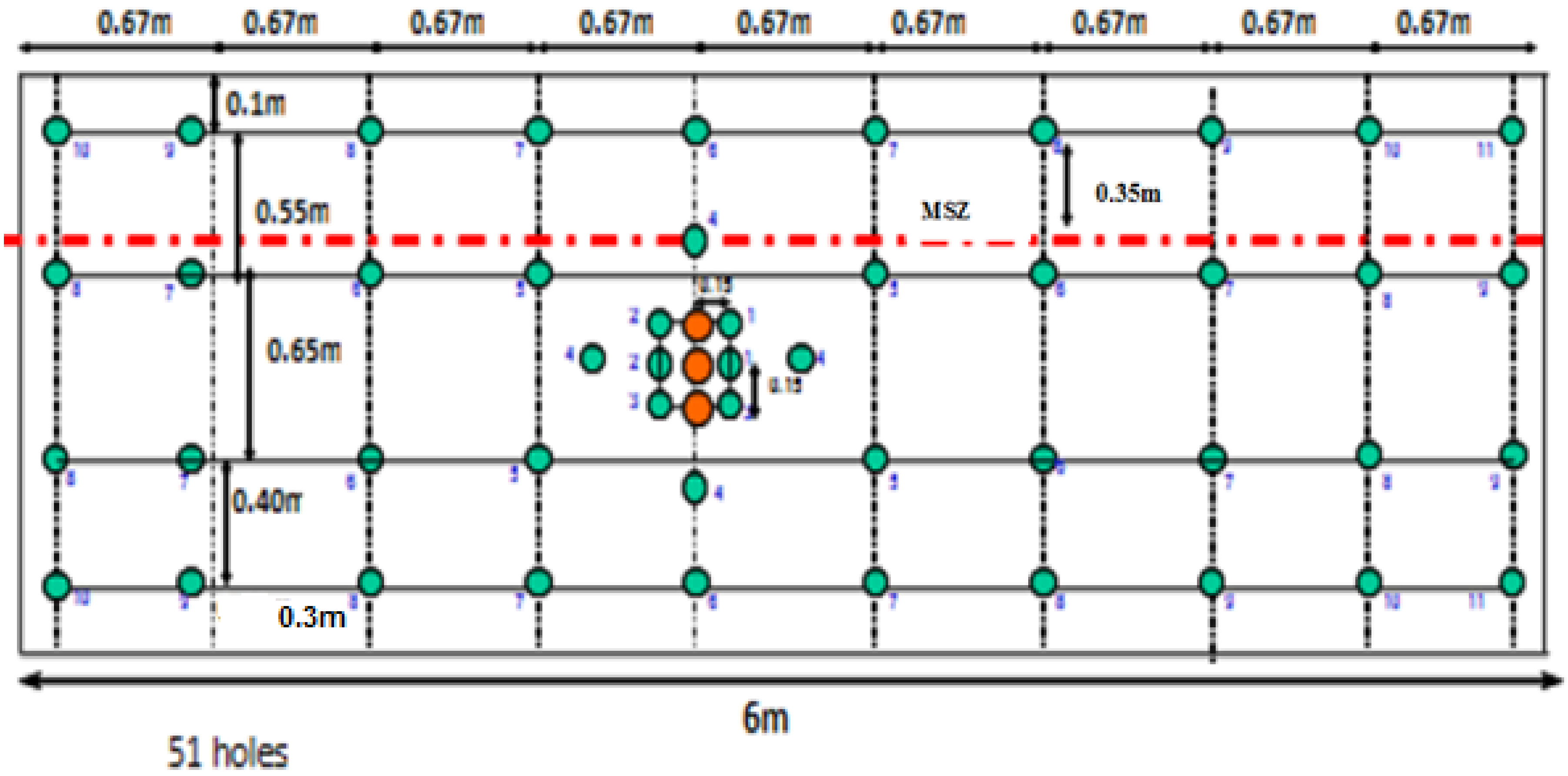
APPENDICES

Appendix A: 51holes drilling pattern



Appendix B: Classification of individual parameters used in the Tunnelling Quality Index Q

1. JOINT SET NUMBER (J_n)	
DESCRIPTION	***VALUE***
A. Massive, few random joints	**1.0**
B. One joint set	**2.0**
C. One joint set plus random joints	**3.0**
D. Two joint sets	**4.0**
E. Two joint sets plus random joints	**6.0**
F. Three joint sets	**9.0**
G. Three joint sets plus random joints	**12.0**
H. Four or more joint sets, random heavily jointed	**15.0**
I. Crushed rock	**20.0**

2. JOINT ROUGHNESS NUMBER (J_r)			
DESCRIPTION	***VALUE***		
	Discontinuous	Undulating	Planar
A. Rough or irregular	**4.0**	**3.0**	**1.5**
B. Smooth	**3.0**	**2.0**	**1.0**
C. Slickensided	**2.0**	**1.5**	**0.5**
D. +5mm thick gouges	**1.5**	**1.0**	**1.0**

3. ROCK QUALITY DESIGNATION (RQD)	
DESCRIPTION	***VALUE***
A. Very poor	**0 - 25**
B. Poor	**25 - 50**
C. Fair	**50 - 75**
D. Good	**75 - 90**
E. Very good	**90 - 100**
Note 1: Where RQD<10, use value of 10	
and if RQD>100, use value of 100	

4. JOINT ALTERATION NUMBER (J_a)			
DESCRIPTION	**Fill<1mm**	**Fill 1-5mm**	**Fill>5mm**
A. Tightly healed, hard rockwall joints,(Quartz)	**0.8**	**1.0**	**2.0**
B. Unaltered joint walls	**1.0**	**2.0**	**3.0**
C. Non-cohesive mineral (Calcite)	**2.0**	**4.0**	**6.0**
D. Serpentinite/Talc Infill	**3.0**	**6.0**	**10.0**
E. Clay	**4.0**	**8.0**	**12.0**
F. Shattered Zones or crushed rock	**5.0**	**10.0**	**12.0**

5. JOINT WATER (J_w)	
DESCRIPTION	**VALUE**
A. Dry	**1.0**
B. Wet/Moist	**0.8**
C. Dripping water (< 5litres/min)	**0.5**
D. Gushing >10l/min	**0.1**

6 STRESS REDUCTION FACTOR (SRF)	
DESCRIPTION	***VALUE***
A. No shear, faults,	**1.00**
B. One shear/fault or major E-W joints with opposing dips.	**2.50**
C. One shear/fault and blocky ground	**4.00**
D. Multiple faults or dykes	**6.00**
E. Curved Low Angled joints or domes	**7.50**
F. Joints sub // to advance direction (same/opposing dips)	**8.00**
H. Multiple faults/Wide shears mylonite zones	**10.00**

Appendix C: Rock Mass Rating System (Bieniawski, 1989)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence) Rating	< 1 m 6	1 - 3 m 4	3 - 10 m 2	10 - 20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

Appendix D: Mining Rock Mass Classification (Laubscher, 1990)

	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	164-145	144-125	124-105	104-85	84-65	64-45	44-25	24-5	4-0
	Rating	20	18	16	14	12	10	8	6	4	2	0
3	Joint Spacing	Refer Figure 10.3										
	Rating	25										0
4	Joint condition including groundwater	Refer Table 10.9										
	Rating	40										0

Appendix E: Adjustments for Joint Condition and Groundwater (Modified after Laubscher, 1990)

Parameter	Description		Dry Condition	Wet Conditions: Moist	Wet Conditions: Moderate pressure 25-125 l/mm	Wet Conditions: Severe Pressure >125 l/min
A Joint Expression (large scale irregularities)	Wavy	Multi-Directional	100	100	100	95
		Uni-Directional	95 90	95 90	90 85	80 75
	Curved		89 80	85 75	80 70	70 60
	Straight		79 70	74 65	60	40
B Joint Expression (small scale irregularities or roughness	Very rough		100	100	95	90
	Striated or rough		99 85	99 85	80	70
	Smooth		84 60	80 55	60	50
	Polished		59 50	50 40	30	20
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	
	Gouge thickness < amplitude of irregularity		20	10	Flowing material 5	

Appendix F: Stereographic plotting sample data

N-S joint set		E-W joint set		E-W joint set continuation	
Strike (degrees)	Dip angle (degrees)	Strike (degrees)	Dip angle (degrees)	Strike (degrees)	Dip angle (degrees)
350	71	83	76	83	79
352	75	84	79	85	68
350	78	89	77	83	75
008	68	88	68	86	70
343	55	88	67	85	74
351	70	86	71	85	68
007	79	84	75	85	75
348	80	84	73	89	79
353	61	83	68	83	79
359	80	89	72	84	74
351	81	86	70	88	67
349	50	83	71	87	77
008	70	86	73	86	67
359	75	85	75	84	75
352	75	87	69	83	77
350	73	85	77	87	78
349	75	84	75	85	70
358	60	86	72	88	79
353	65	84	75	85	77
349	70	84	79	84	77
349	75	83	79	88	72
351	75	83	76	88	74
358	71	89	70	85	74
019	65	84	79	88	71
355	72	83	69	83	79
355	60	86	70	87	74
009	50	85	74	83	70
357	78	89	77	88	76
356	69	83	79	83	68
358	50	88	77	89	67
358	70	86	68	88	76
357	63	85	79	85	75