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Pit Optimisation of Vondeling Quarry by Understanding Geotechnical Parameters Determined at Zoutkloof Quarry

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PIT OPTIMISATION OF VONDELING QUARRY BY UNDERSTANDING GEOTECHNICAL PARAMETERS DETERMINED AT ZOUTKLOOF QUARRY

Research Stream: Mine Planning

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X	

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

December 2023

Declaration

I, Tshinanne Matty Mukwevho declare that this assignment is my own, unaided work. I have read the University Policy on Plagiarism and hereby confirm that no plagiarism exists in this report. I also confirm that there is no copying nor is there any copyright infringement. I willingly submit to any investigation in this regard by the School of Mining Engineering and I undertake to abide by the decision of any such investigation.



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Abstract

The purpose of the study was to investigate the geotechnical parameters at Zoutkloof quarry and how they affect stability and the mine planning process. The geological features of Zoutkloof and Vondeling are similar, hence the lessons learned while mining Zoutkloof quarry can be used when mining Vondeling quarry. Zoutkloof quarry has already reached its limits and is no longer operational. It is important that mine planning considers the critical geotechnical parameters. The main reason for this consideration is to keep slope walls stable, employees and equipment safe, and to continue mining the ore in an economical manner. The methodology of the research incorporated highlighting the literature in the public domain on geotechnical considerations in open pit mining. Evaluating geotechnical parameters such as groundwater, rock mass strength, slope angle and monitoring; and additionally, showed scheduling of mining blocks from 2007 to 2008 formed part of methodology in the research. The results analysis indicated that the strategies implemented to control groundwater were successful to keep the production benches dry and walls stable. Good understanding of the discontinuities and the rock mass strength enabled the quarry to be divided into ground control districts. Kinematics analysis for possible failures was done and the results showed that there was no probability of failure on planar mode. However, there were minor possibility that failure can occur on wedge and toppling mode. Yearly mining scheduling was completed, focusing on tonnage and quality requirements. During this period, Zoutkloof had minimum waste mined and the quarry had narrowed significantly which required the operational team to work within mine design specifications to maintain safety and slope angles. Some resources had to be compromised as it was not practical to exploit them safely. The research concluded its findings as successful because Zoutkloof quarry was mined completely with approximately 10 slope failures that resulted with no injuries to employees or damage to equipment. The factors of safety (FOS) were evaluated to be well above one and slopes remained stable until mining ceased. The research also made recommendations that can be implemented while the Pretoria Portland Cement (PPC) continue to mine Vondeling quarry to aid same success as Zoutkloof while being cost effective.

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To my family and friends for all their support, which enabled me to continue doing what I do without much stress in this period.

To my Supervisor, Mr. Ohveshlan Pillay, I would like to say thank you for the guidance and a good working relationship.

Finally, I dedicate this study to my father, Mr E. Mukwevho whom I lost while busy with my research. May you continue resting in peace.

Table of Contents

Declaration i

Abstract ii

Acknowledgements iii

List of Figures vii

List of Tables viii

1. Introduction 1

1.1 Research Background 1

1.2 Limestone Quality Parameters 3

1.3 Research Motivation 4

1.4 Problem Statement 4

1.5 Assumptions 5

1.6 Research Questions 5

1.7 Research Methods 6

1.8 Criteria for Validation 6

2. Literature Review 8

2.1 Introduction 8

2.2 Slope Design 10

2.2.1 Slope design acceptance criteria 12

2.2.2 Factors affecting slope stability 14

2.3 Slope Failure Mechanism 17

2.3.1 Planar failure 18

2.3.2 Wedge failure 18

2.3.3 Circular failure 18

2.3.4 Toppling failure 19

2.4 Slope Stability Analysis 20

2.4.1 Limit equilibrium method 21

2.4.2 Numerical methods 22

2.5 Rock Mass Characterisation and Geotechnical Parameters 25
2.6 Geotechnical Model 27
2.6.1 Geological model 28
2.6.2 Structural model 28
2.6.3 Rock mass model 29
2.6.4 Hydrogeological model 33
2.6.5 Geotechnical model summary 34
2.7 Methods of Reinforcement and Monitoring 35
3. Data Collection 36
3.1 Introduction 36
3.2 Geological and geotechnical information 36
3.3 Field observation and other information 37
3.4 Regional geology 38
3.5 Summary 38
4. Geotechnical Analysis 39
4.1 Introduction 39
4.2 Hydrological Considerations 39
4.3 Blasting practices 40
4.4 Geological Structure and Rock Mass Strength 41
4.4.1 Ground control districts 41
4.4.2 Field observations 46
4.4.3 Joint analysis 47
4.4.4 Determining RMR 47
4.5 Slope Angles 51
4.5.1 Kinematic analysis 52
4.6 Pit Wall Design 54
4.7 Monitoring 57
4.8 Vondeling quarry geotech 59

4.9 Summary....61
5. Mine Scheduling....62
5.1 Introduction....62
5.2 Year 1 Scheduling (2007-2008)....62
5.3 Year 2 Scheduling (2009-2010)....64
5.4 Year 3 Scheduling (2011-2012)....65
5.5 Year 4 Scheduling (2013-2014)....68
5.6 Year 5 Scheduling (2015-2016)....73
5.7 Year 6 Scheduling (2017-2018)....75
5.8 Costs Analysis....77
5.9 Summary....78
6. Conclusion....79
7. Recommendations....81
8. References....82
9. Appendix A....90
10. Appendix B....103

List of Figures

Figure 1.1: PPC De Hoek location....1
Figure 1.2: Zoutkloof quarry 2
Figure 2.1: Potential impact of increasing overall slope angle....9
Figure 2.2: Open pit wall design parameters....10
Figure 2.3: Process of slope design 11
Figure 2.4: Impact of additional data POF and FOS....14
Figure 2.5: Failure models in rocks 17
Figure 2.6: Mohr-Coulomb yield before/after strength reduction 25
Figure 2.7: Haines and Terbrugge empirical design chart....31
Figure 2.8: Geotechnical model input and output....35
Figure 4.1: Evidence of pre-split blasting on Eastern side wall....40
Figure 4.2: Zoutkloof ground control districts....43
Figure 4.3: GCD on south wall 44
Figure 4.4: Undercut area 45
Figure 4.5: GCD on east wall 46
Figure 4.6: Empirical slope design chart 51
Figure 4.7: Stereonets at 60° slope angle 52
Figure 4.8: Stereonets at 50° slope angle for toppling failure....53
Figure 4.9: 50° slope angle wedge failure 54
Figure 4.10: Zoutkloof quarry cross section 56
Figure 4.11: Slope monitoring strategy (PPC De Hoek Mine, 2020) 58
Figure 4.12: Vondeling quarry structure cross section 60
Figure 5.1:2007-2008 mining blocks 63
Figure 5.2: Mining block for 2009-2010....65
Figure 5.3: 2011-2012 scheduling....68
Figure 5.4: Limestone mining blocks for 2013-2014....70
Figure 5.5: 2015-2016 Mining blocks 74

Figure 5.6: 2015-2016 waste blocks.....75

Figure 5.7: 2017-2018 mining block 77

List of Tables

Table 1.1: Major and minor oxides analysed 3

Table 1.2: Nak range.....4

Table 2.1: Criteria for stable slopes.....13

Table 2.2: Geotechnical test.....26

Table 2.3: Weathering adjustments.....32

Table 2.4: Joints orientation adjustments 32

Table 2.5: Blasting effects adjustments.....33

Table 3.1: Geological ang geotechnical information.....37

Table 4.1: Ground control districts.....42

Table 4.2: RMR evaluation.....48

Table 4.3: Summary of RMR values for all GCDs 49

Table 5.1: Quality requirements 2007-2008 63

Table 5.2: Quality 2009-2010 64

Table 5.3: Tonnage and quality requirements at Zoutkloof 2011-2012 66

Table 5.4: Limestone tonnage and quality according to mining blocks.....66

Table 5.5: Overburden block tonnages 67

Table 5.6: 2013-2014 tonnage budget 69

Table 5.7: 2013 Summary of tonnages 71

Table 5.8: Low quality limestone blocks.....72

Table 5.9: Internal waste tonnages 72

Table 5.10: 2015-2016 scheduling 73

Table 5.11: 2017-2018 limestone scheduling.....76

1. Introduction

1.1 Research Background

Optimisation of the ore body is vital for the life of mine for any operation. There are several optimisation techniques that can be used in surface mining. This includes variable cut-off grade, push backs and sequencing, stockpiling, pit shell optimisation, recovery optimisation and constraints. The research programme will focus on pit optimisation by better understanding the geotechnical parameters to increase Life of Mine (LOM) and to ensure overall safety of the mine. Pretoria Portland Cement (PPC) De Hoek (DH) quarry is in the Western Cape Province in South Africa, approximately 129 km from Cape Town (Figure 1.1).



Figure 1.1: PPC De Hoek location (AfriGIS(Pty)Ltd, 2022)

The mineral commodity of interest is limestone for cement manufacturing purposes. In this region, limestone is sourced from three quarries namely De Hoek, Zoutkloof and Vondeling. Presently Vondeling is the only quarry which is under operation while the other two quarries are mined out.

Zoutkloof quarry began operations in 1980 after the De Hoek pit was mined out. The initial LOM was set to be approximately 30 years. The orebody dips steeply to the east at between 50º to 80º. The dominant joint set, being the bedding, dominates the stability of the slope broadly defining the quarry into three domains, the west footwall, orebody and east hanging wall as illustrated in Figure 1.2.



Figure 1.2: Zoutkloof quarry

Technology was not as advanced as compared to current times, hence understanding the geotechnical nature of the material was a challenge. The quarry is highly jointed, foliated and a major slope failure on the eastern slope occurred in

1998. The failure was ascribed to weak boundary conditions, a steep slope, and the presence of water. The failure was a wake-up call to mine planners and site personnel to alter the thinking when it comes to slope stability. Based on this failure and other minor failures, a good understanding of geotechnical parameters in the pit was important.

1.2 Limestone Quality Parameters

The prime concern of quality control for the De Hoek plant is based on RCO_3 content in ROM limestone, almost all samples (about 98.7%) were analysed for RCO_3 and approximately 91.45% of total samples were analysed for NaK. Scheduling, grade, and tonnages were generated using Surpac software. The analysis for major and minor oxides were available only for about 87% of the total samples as shown in Table 1.1.

Table 1.1: Major and minor oxides analysed

SiO_2	Silicon dioxide	Mn_2O_3	Manganese oxide
Al_2O_3	Aluminium dioxide	TiO_2	Titanium dioxide
Fe_2O_3	Iron dioxide	SO_3	Sulphur trioxide
CaO	Calcium oxide	Cl	Chlorine
MgO	Magnesium oxide	Na_2O	Sodium oxide
K_2O	Potassium oxide	P_2O_5	Phosphorus pentoxide

Limestone found in De Hoek is primary limestone and quality is controlled by checking ROM limestone on LSF, RCO_3 and NaK values during scheduling and short term mine planning. The NaK quality ranges that will be used in the report is as detailed in Table 1.2.

Table 1.2: Nak range

NaK range	
0.0 to 0.49	High grade limestone
0.49 to 0.70	Requires screening
0.70 to 0.95	Requires screening
Greater than 0.95	Internal waste

1.3 Research Motivation

The motivation of this study is to understand the geotechnical parameters that played a major role when Zoutkloof quarry was still in operation and how these parameters influenced the mine planning process. This is done with the aim of applying the lessons in the current and future quarries with similar geological features. From new research, the information will be relevant to the company management, mine planners and operational teams. The new research information can also be used by other academia's and industry associates as a reference to other mines with similar challenges. The learnings and experience from the Zoutkloof quarry can be used as the Vondeling quarry continues to develop. This is because both quarries have similar geological features specifically the host rock mass and hydrogeology.

1.4 Problem Statement

The geological structure of the Zoutkloof quarry is complex which makes stability issues challenging. The slope stability challenges started as early as the 1980s soon after the opening of the quarry with a slope failure on the eastern side of the pit. The failure was due to steep slope on weathered phyllite material. The area was stabilised by the introduction of drainage holes and a cutback of the slope to unload its crest. Continuous monitoring of that and the surrounding area indicated

the area was stable and the slope well drained. Hence mine planning had to consider the geotechnical nature of the material and the associated parameters such as geological, hydrogeological, and blasting models.

To mitigate against slope instability, it becomes essential to constantly assess, monitor (PPC De Hoek Mine, 1995), and evaluate the controls put in place to ensure acceptable slope stability, good mining practices and mine planning that considers the different areas in the pit. The report will focus on analysing the geotechnical parameters at Zoutkloof quarry and recommendations will be made on how Vondeling quarry geotechnical challenges can be approached using the lessons learned.

1.5 Assumptions

The research will assume that the process of treating and analysing exploration data was done correctly as the process is explained below. It will further assume that the block model and orebody used for the study depict a true reflection of the quarry.

1.6 Research Questions

The questions that this research programme will attempt to answer for the understanding and consideration of the geotechnical parameters that influence mine planning and scheduling at De Hoek mine specifically the Zoutkloof quarry are the following:

- What are the critical geotechnical considerations adopted in mine planning?
- How did these affect the mine planning process?
- How did the geotechnical considerations affect the overall stability of the quarry?
- What was the impact on the costs?

1.7 Research Methods

The research will follow qualitative methods for data collection using both primary and secondary sources of data. Qualitative methods are mainly concerned with gaining insights and understanding of underlying reasons and motivations. Data was sourced internally from company documents and field observations. Additionally, Surpac software will be used to schedule the mining blocks for the period in consideration. Books, journal papers and academic publications will be used to support the theories and arguments made in the research. DIPS software will be used to do stereographic projections for kinematic analysis.

1.8 Criteria for Validation

Understanding the causes of data uncertainty is important when determining and reporting uncertainties in each component of the geotechnical model. In addition, it is important to understand its impact on how reliable the pit slopes are, how it is quantified and how it is reported to corporate mine management and the investment community. Data uncertainty originates from the recurrent inability of geologist, engineering geologist and geotechnical engineers being unable to correctly predict the properties and characteristics of natural materials in open pit mining. There are three group which the relevant types of uncertainty can be placed. These are geological uncertainty, parameter uncertainty and model uncertainty (Read and Stacey, 2009).

Geological uncertainty focusses on unpredictability associated with the identification, geometry of and relationships between the different lithologies and structures that constitute the orebody wall rocks being targeted. Unpredictability of the parameters used to account for the various attributes of the geotechnical model is represented by parameter uncertainty. Model uncertainty can be defined by the unpredictability that surrounds the selection process along with the types of analyses used to formulate the slope design and estimate the reliability of the pit walls (Read and Stacey, 2009).

FOS is commonly used for assessments of the performance of open pit mine slopes. FOS is the ratio of the nominal capacity (C) and demand of the system (D). Probability of failure (PoF) is another acceptance criteria that was introduced over the years. The FOS is the most basic design acceptance criterion in engineering. In the middle of the 20th century, it become importance in geomechanics when the geotechnical engineering was developed as an independent engineering discipline (Read and Stacey, 2009). For the validity of data used in the study, FOS has been used as the acceptance criteria of data.

2. Literature Review

2.1 Introduction

The establishment of a mine considers many important constraints such as safety, machinery, workers' productivity, and other factors that have a direct impact on the mine's output. Rebbah, *et al.* (2019) documented that the main challenges in the design and planning of open pits are:

- Ultimate Pit limit determination;
- Mining sequence and pushbacks; and
- Maximum economic value determination.

This implies that the mine designs must define the final pit configuration that maximises ore recovery safely. To achieve this, the slope design of the open pit must establish walls at individual benches and until final depth of mining is reached. Thus, mine planners must ensure that slopes of large open pit mines are stable so that mines can operate safely without posing risks to people, equipment, and reserve recovery. To maximise economic benefit to shareholders, waste stripping must be minimised, and ore recovery maximised resulting in a compromise in pit designs that establishes slopes that are as steep as possible and yet safe and practical to implement (Read and Stacey, 2009).

The stripping ratio is affected by slope angle of the pit. Read and Stacey (2009) demonstrated in Figure 2.1 that increasing the slope angle by a few degrees significantly reduces waste stripping and increases the ore recovery.

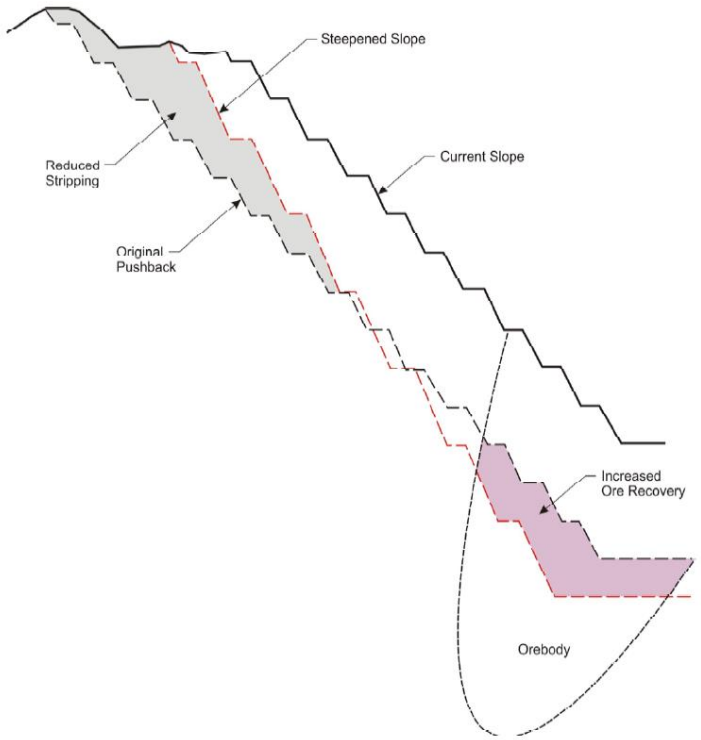


Figure 2.1: Potential impact of increasing overall slope angle (Read and Stacey, 2009)

Consequently, to make the best use of the mineral resource and to maximise the value of an open pit mine, the slopes must be as steep as possible (Read and Stacey, 2009). However, increasing the slope angle can reduce the stability of the slope and lead to failure. A large-scale slope failure or an uncontrolled slope instability can be catastrophic and may lead to many operational, economic, environmental, and social consequences. This includes operational delays, damage to equipment, blockage of access ramp, premature mine closure, loss of a mine's social license to operate and a possibility of loss of life (Read and Stacey, 2009; Fillion, 2018; Kolapo *et al.*, 2022). The general objective of any pit slope design process must be to define the geometry of the pit that minimises the risk of instability while minimising the volume of material to be excavated. Hence the final cost of the project and this can limit the maximum slope angle that can be achieved (Fleurisson, 2012b; Fillion, 2018). Specific environmental considerations especially as pertaining to procedures for mine closure and abandonment must also be considered in the design of long-term slopes (Fleurisson, 2012a).

2.2 Slope Design

According to Kolapo *et al.* (2022) practical experience has demonstrated the significance of having a robust design in rock slope projects. Figure 2.2 details the basic parameters to be considered in the design of pit slopes.

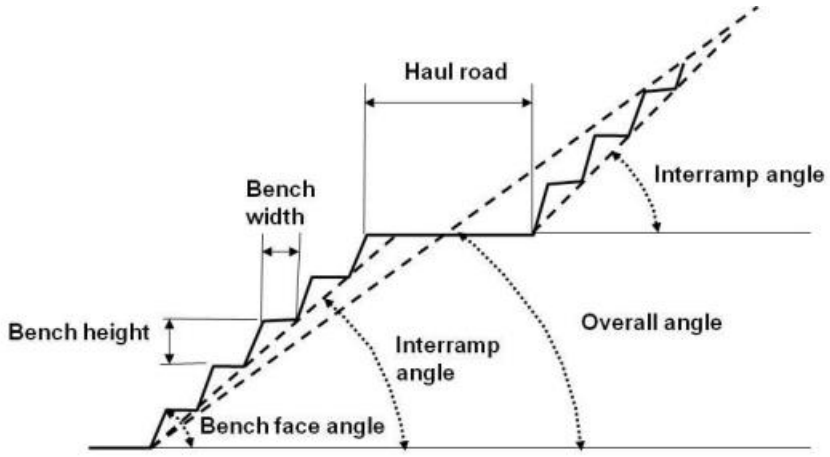


Figure 2.2: Open pit wall design parameters (Fleurisson and Cojean, 2014)

The process of designing rock slopes is challenging as it needs the knowledge of the geological structure, geotechnical properties of the rock mass, and rock lithology (Read and Stacey, 2009). Before developing the slope, defining the optimum design parameters is important (Read and Stacey, 2009). According to Wesseloo and Read (2009), the design of rock slopes and the assessment of its stability is an iterative process undertaken at bench scale, then at inter-ramp scale and finally, for the overall pit wall. Each one of them must be defined for every geotechnical domain contained in the geotechnical model and is characterised by a certain slope failure mode that is most likely to occur in each geotechnical domain and at a certain scale. The slope design process begins at the bench level where the bench parameters are defined, and its stability evaluated. Where the benches are assessed to be unstable, the parameters are redefined until they produce stable benches. The configuration that gives stable benches and the location of ramps are used to define the inter-ramp slope angles and its stability is evaluated. Subsequently, if this evaluation results in an unstable inter-ramp slope, the

parameters are redefined, and the stability is re-evaluated. If inter-ramp slope is considered stable, the overall slope angle will be defined by the inter-ramp slopes included within the overall slope. Finally, the overall slope's stability is assessed to determine whether the entire pit slope is stable (Hölck Teuber, 2016). In Figure 2.3, Read and Stacey (2009) diagrammatically illustrated the processes involved in designing a rock slope. These stages include models, domains, design, analysis, and implementation.

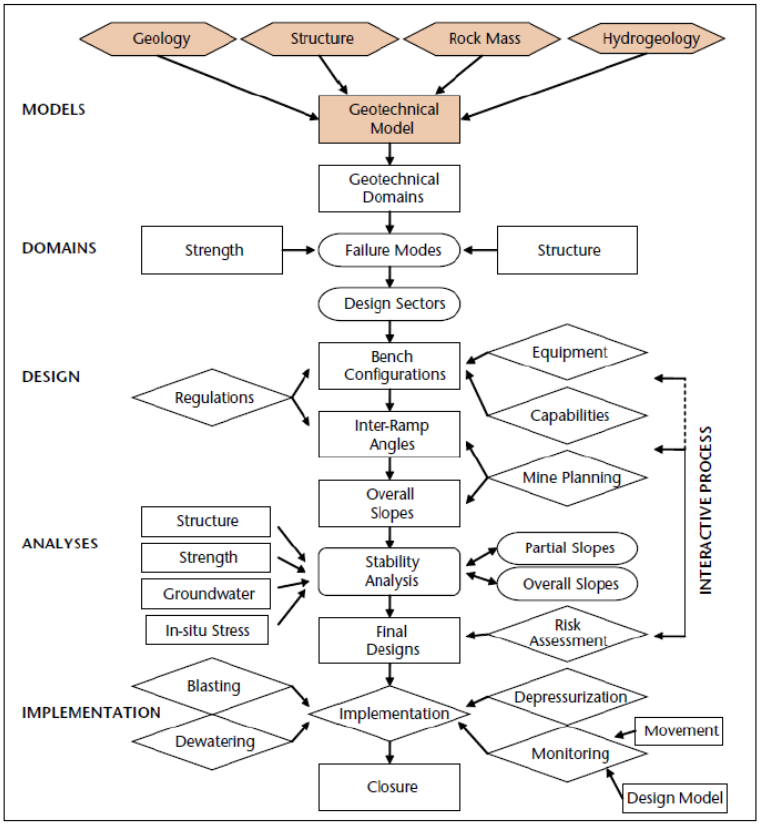


Figure 2.3: Process of slope design (Read and Stacey, 2009)

Before the design can be implemented to determine the economic and stability of a rock slope, it is important to conduct site investigation and collect data. If there are indications that slope failure might occur, structural characteristics of the rock mass need to be considered. During geological exploration, data collected from site can be used to provide information on the strength of the rock mass, deformability

properties, and geological structure; additionally, the data can indicate the presence of major planes of weakness during early stages of the design. These parameters can be used when predicting slope stability (Kolapo *et al.*, 2022).

2.2.1 Slope design acceptance criteria

Slope designs must meet both operational specifications and corporative tolerance of risk. Considerations must be given to management's risk acceptance level related to safety and economic outcomes of the open pit. Thus, the ultimate pit limits must factor in the distribution of ore grade, cost of production and the geotechnical parameters of the rock mass. Traditionally, the Factor of Safety (FOS) defined as the ratio of the shear strength of an intact rock to its shear stress defined the criteria for which a slope was accepted (Fillion, 2018). The factor of safety is mathematically expressed by Equation 2.1 below:

$$FOS = \frac{shear\ strength}{shear\ stress}$$

Equation 2.1

Fleurisson and Cojean (2014) documented the criteria for acceptance for slope design based on the FOS calculations. For FOS less than or equal to one, the slope is unstable since the shearing stress exceeds the shear strength of the rock material and for FOS greater than one, the strength of the rock mass exceeds the stress, and the rock is in limit equilibrium and a stable design. The stability conditions for are presented in Table 2.1.

Table 2.1: Criteria for stable slopes (Fleurisson and Cojean, 2014)

Calculated FOS	Stability Condition	Recommended Action
FOS > 2.0	Stable	None
1.0 < FOS < 2.0	Marginal	Analyse stability rigorously
FOS < 1.0	Unstable	Revise design or stabilise

In kinematic, limit equilibrium and numerical analyses, a tolerance on the calculated factor of safety greater than one is considered to accept the slope design as stable. This is to account for uncertainties in the input data (Hölck Teuber, 2016). The likelihood of rock slope failure can be impacted by the gaps in the analysis parameters and on the safety margin of the selected slope angle (Fillion, 2018). Thus, when the calculated FOS is greater than one, it may not account for the risk induced in the stability analysis by the variability that exists in the properties of the material. From experience, the acceptable value defined of FOS for one slope cannot be comparable between different areas of the same mine or from one mine site to another (Hölck Teuber, 2016). As a result, FOS calculations may not be conservative due to uncertainties in input data (mainly anisotropic characteristics of rock masses) and usage (Fillion and Hadjigeorgiou, 2016) and this introduces some level of risk in slope stability assessments based on the FOS calculations (Hölck Teuber, 2016).

The work of Fillion (2018) highlighted the importance of additional data collection in improving the slope design problem by increasing the confidence level of the input parameters. He demonstrated by Figure 2.4, that, for an acceptance criterion of POF ≤ 5% and FOS ≥ 1.2, collecting additional information yields positive economic benefits as the acceptance criteria will be respected even for increasing slope angles.

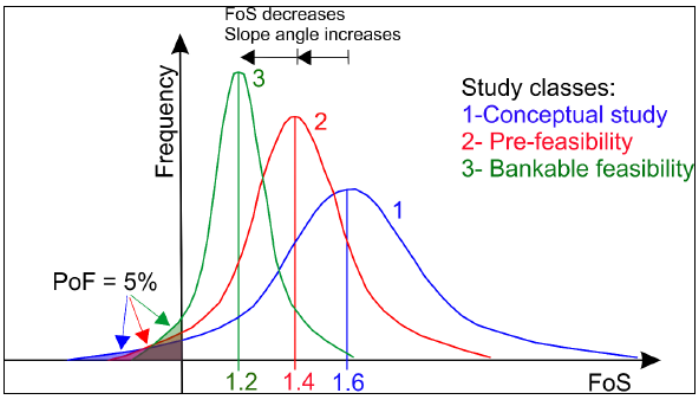


Figure 2.4: Impact of additional data POF and FOS (Fillion, 2018)

2.2.2 Factors affecting slope stability

The stability of the rock mass slopes can be influenced significantly by many factors. Hustrulid *et al*. (2001) argued that activities such as drilling and blasting, and the use of heavy equipment are the main causes of instability in rock mass slopes. Likewise, Read and Stacey (2009) identified that factors such as rock mass discontinuities, groundwater presence, complex rock geology, and mining operations influence the stability of rock slopes in mines. According Stacey and Swart (2001) the two major important factors that control the stability of slopes are the effects of blasting and groundwater. The significance of effects of blasting is ground vibrations which in turn may influence the stability of the slope. Gundewar (2014) demonstrated that the selected mining method and equipment used can affect the stability of slopes. Not all these factors are considered during slope stability analysis. Some are included when analysing their effects on a quantitative judgement basis during slope stability analysis (Kolapo *et al.*, 2022).

2.2.2.1 Slope geometry

Slope geometry plays an important role in the stability of open-pit mines (Kolapo *et al.*, 2022). The study by Chaulya and Prasad (2016) indicated that overall slope angle, bench height and surface area are what forms the basic geometric slope design. According to Kolapo *et al.* (2022), increase in pit wall height and slope

decreases the slope stability. Most natural and excavated slopes are convex or concave in shape, they are not infinite in the plane. The result of the study by Zhang *et al.* (2013) demonstrated how curving surface slopes impacts the FOS and the stability assessment. They proved that the relationship between the curvature and slope height of the wall, affects how stable slopes are. Wines (2016) further demonstrated that concave slopes are more stable than straight slopes and convex slopes are often less stable compared to straight slopes. In addition, the lack of confinement and effects of the side resistance in convex slopes makes their potential failures more structurally controlled.

2.2.2.2 Geological structure

. Slope stability is influenced by geological structures such as dip direction, shear zones within formations, discontinuities, faults and joints (Kolapo *et al.*, 2022). Slopes may fail because of structural discontinuities in intact zones (Higuchi *et al.*, 2015). Chiwaye (2010) argued that the density of material that forms part of the slope has impact on its stability. Thus, high-density rock mass limits water infiltration, while a very low-density rock mass promotes water seepage.

2.2.2.3 Groundwater

Groundwater conditions within the rock mass affects the slope stability. This occurs normally by either underground or surface water seepage. Kolapo *et al.* (2022) reported that the presence of groundwater mitigates rock properties by reducing the effective vertical stress and altering the rock strength parameters such as cohesion and friction. In addition, Johansson (2014) stated that groundwater pressure and transient flow of water within the rock and soil affect the pore pressure conditions, strength, and deformation behaviour of the rock. Additionally, the presence of water pressure in a rock expands the pores in the rock reducing its compressive strength.

2.2.2.4 Lithology

The mineral composition of rock mass determines its mechanical properties, strength and behaviour and thus influencing the stability of the slope (Suman, 2015). As highlighted by Kolapo *et al.* (2022), rock properties such as density, porosity, degree of cementation and mineralogy can influence the rock strength.

2.2.2.5 Blasting

Poor blasting in surface mining operations is detrimental to slope stability not only because of rock vibrations but because the rocks on the slope may crack and loosen (Simataa, 2019). The ground vibration induced by poor blasting leads to stress redistribution in the rock slope. This can result in the slope plane being unstable. (Suman, 2015). Kolapo et al. (2022) argues that poor blasting can lead to rock mass instability by;

- creating discontinuities such as fractured zones, in rock slopes,
- reducing the intact rock cohesion factor,
- increasing the potential for ingress of water which weakens the rock mass, and,
- also reducing the bench width.

2.2.2.6 Mining method and equipment usage

The overall stability of the slope should be considered when selecting the mining methods and equipment to be used. Kolapo et al. (2022) noted that natural slope is vulnerable to deformation due to loss of shear strength during excavation so the slope design process should consider in-situ stress areas. They further argued that the use of heavy mining and equipment has increased overburden which has increased the forces due to slope movement. Hoek et al. (2000) noted that lateral stresses during rock excavation have a significant effect on slope stability. Most of

the joints in high confinement conditions reach their residual aperture, resulting in rock stiffness no longer affected by the increment in the confining stress.

2.3 Slope Failure Mechanism

Slopes fail either by the action of external forces or when the shear stress exceeds the strength of the rock material. In most cases, the failure of the rock mass happens very suddenly and most of the time, it is the results of a combination of factors (Simataa, 2019; Kolapo et al., 2022). The analysis of geological formations and geotechnical parameters of the rock materials as well as the analysis of the mechanical stress generated by mining leads to the identification of the possible failures likely to occur. Simplification of these complex mechanisms is usually necessary to establish physical and then numerical models using homogenisation and normalisation methods that allow failure risk to be quantified (Fleurisson, 2012a).

To determine the possible failure modes in each operation geological parameters must be measured in different areas of the mine. Gathering information such as orientation spacing trace length and shear strength of geological structures is key to determining the probability of failure (Girard, 2001). There are four types of rock slope failure wedge, planar, circular, and toppling failure as shown in Figure 2.5.

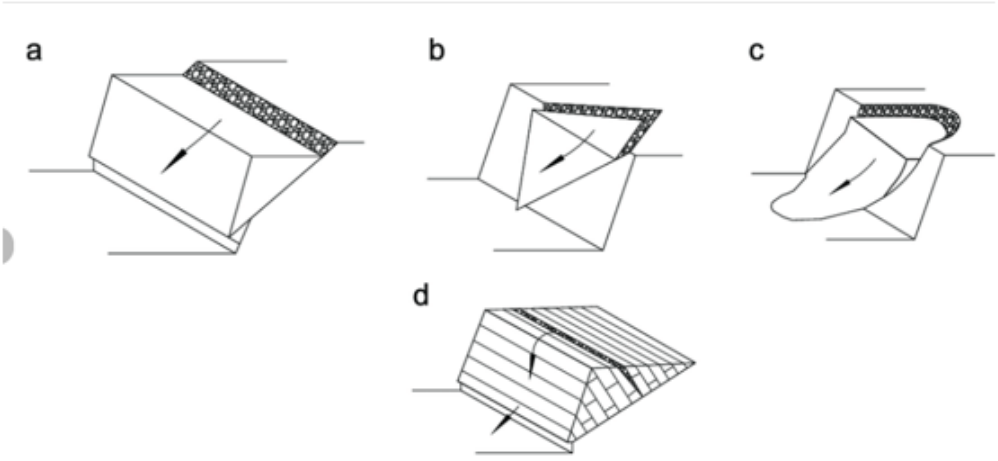


Figure 2.5: Failure models in rocks (Babiker, *et al*., 2014)

2.3.1 Planar failure

For planar failure to occur, a discontinuity that is approximately parallel to the slope face and dips at a lower angle to the slope face must intersect it, causing the slope to slide (Gundewar, 2014). A geologic discontinuity such as bedding plane that strike parallel to the slope face and dip into the excavation at an angle with a steeper slope than the angle of friction can result in a planar failure (Girard, 2001). The failure necessitate rock slipping or sloping downward and outward along a gently undulating surface. These conditions will result in planar failure due to lack of confinement.

2.3.2 Wedge failure

Wedge failure occurs due to the junction of two or more discontinuities leading to the formation of a tetrahedral block failure. This block may slide out when the angle of the line of intersection is greater than the internal angle of friction along the discontinuities (Simataa, 2019). This failure mode may occur when the sliding line is inclined less steeply than the slope face (daylights) (Nicholas and Sims, 2000). The rock sitting on two discontinuities that connect obliquely across a slope face will slide along the junction line (Kolapo *et al.*, 2022).

The most frequent failure mechanism in rock slopes is wedge failure (Goodman and Kieffer, 2000). One of the main elements in the design of a capable bench face angle-height arrangement is the stability analysis of a wedge failure. The stability evaluation includes large wedges, which may have an impact on the slope's overall stability, and joint planes, which may connect and have an impact on the stability of the bench or even the ramp (Kolapo *et al.*, 2022).

2.3.3 Circular failure

The most common circular failure occurs in soil and extensively weathered or tightly fractured rock. Circular failure occurs in continuum slopes with extensively jointed or weak rock mass. Circular failure can occur in hard rock as well. In weak strata

like soil or deeply weathered rock, the circular failure is defined by a single discontinuity surface but will typically follow a circular route. However, tension cracks at the upper ground surface are less likely to spread if the failure surface is curved (Simataa, 2019; Kolapo *et al.*, 2022). According to Gundewar (2014) circular failure can be divided into three categories which are : slope failure, toe failure, and base failure. Slope failure occurs when the rupture surface's arc intersects the slope's shape above the toe. Toe failure happens when the rupture's arc coincides with the slope at its toe, while base failure happens when the rupture's arc enters the slope's base below the toe. Base failure typically occurs when the material above the base is softer than the strata below the base (Kolapo *et al.*, 2022).

2.3.4 Toppling failure

When vertical or nearly vertical structures lean toward the pit, toppling may occur. The bench face height should be roughly equivalent to the bench width if this type of structure is present. This will lessen the probability of boulders hitting machinery operating on the pit floor below by helping to catch any toppling materials (Girard, 2001). The movement of the failed slope in toppling can be distinguished by the downslope overturning caused by the rotation and flexure of blocks with sharp discontinuities (Simataa, 2019). Additionally, rocks from the upper bench face could ricochet off the benches and over them, creating a risk at lower elevations. The bench should be wide enough to prevent rocks from rebounding over it in the area where toppling may be anticipated (Kolapo *et al.*, 2022).

In the case of substantial toppling instability, there will be lengthy warning period with noticeable occurrences such as the progressive widening of tension cracks close to the slope's crest. Generally, there will be enough time to put stabilising measures in place or leave the area before failures happen. When engaging in mining operations including drilling, charging, and mucking operations, there is a risk for a slight toppling failure that could be abrupt from the bench faces (Kolapo *et al.*, 2022).

2.4 Slope Stability Analysis

Sha (2016) indicated that there is a need to assess the geotechnical stability analysis and design in open pit mines which are represented by geological, structural, hydrogeological, and rock mass models. If surplus loading shear stress through a rock mass is rearranged, and the load exceed the strength of the rock, this eventually causes rock slope failure. The shear strength of a rock mass plays an important role in the stability of the rock mass. Hence Kolapo *et al.* (2022) directed a need of taking into account the elements that can alter the shear strength throughout the design process as they can impact on the stability of the slope. These rock failure events that occur are also supported by Eberhardt *et al.* (2004) as it was proposed in their study that slopes fail due to strength reduction and progressive failure in a rock mass. Long-term stability plays an important role in the design of slope engineering and rock mass extraction in open-pit mines. The slope stability in open-pit mines is an extremely complex issue that requires a special approach.

Cała *et al.* (2020) indicated that due to the variety of factors that can affect the safety of open-pit mining operations, proper maintenance with proper monitoring and prediction tools is necessary. Johansson (2014) mentioned that there are two methods that are mostly used after many were introduced, namely the Limit Equilibrium Method (LEM) and Finite Element Method (FEM) and Finite Differential Method (FDM). These two methods assess the presentation of rock slope's stability.

The choice of stability evaluation carried out, relies on the scale at which the evaluation is made. Kinematic and limit equilibrium analyses carried out at a bench scale as failure is highly influenced by structures near surface and action of gravity. According to Hölck Teuber (2016) for inter-ramp and overall scales, rock mass failures and non-daylighting wedges occur as complex failure modes. In stability analysis at a rock mass scale, slope failure depending on rock mass strength is

determined after inter-ramp stability evaluations have been carried out and resulted in stable inter-ramp slopes (Hölck Teuber, 2016).

2.4.1 Limit equilibrium method

Yang (2014) indicated that when analysing the stability of the slopes, the LEM is mostly used. The basic assumption at their main is that failure take place in the sliding of a mass down the slip surface. The suitable limit LEM is essentially because of their relative simplicity, prepared potential to assess the reactivity of firmness to different input parameters and the knowledge geotechnical engineers gained through the years in clarifying calculated FOS values. Although, the procedure neglects stress-strain behaviour of soils and rocks. However, The technique neglects stress-strain behaviour of soils and rocks. It also makes arbitrary assumptions to ensure static determinacy (Boniface, 2013).

LEM's are commonly used to analyse where the FOS is secured by dividing slices of the rock mass over the imaginary surface of failure into several vertical slices as introduced in Equation 2.1. They undertake the position shape of defeated surface and horizontal forces acting on the sides of the slides and their directions. Regardless of their built-in fragility, these methods were evolved and tried out based on actual case histories. Because of the simplicity of LEM's, the stress-strain behaviour of the rock mass is not observed when calculating the FOS Abdellah *et al.* (2022), noticed that the shape of critical slip ground controlled by trial and error.

According to Simataa (2019), part of safety calculations are determined by limit equilibrium. The mechanical problem is clarified, and the stability of the slope is interpreted using FOS which is defined by the proportion between the maximum resisting forces and the acting forces along a potential failure slope. Theoretically, the surface is stable if FOS is greater than one, however in reality, the theory degree of safety must be adjusted to the validity of the input data (Fleurisson, 2012b).

The LEM can be combined with other geotechnical analysis methods, to assess the initiation of failure in slopes to the worst credible scenario. In this way, it can be predicted that the shear strength of the rock through the likely failure zones is controlled by linear or non-linear relation between the shear strength and the normal stress on the failure ground as stated by FOS (Gundewar, 2014). Although LEM is highly recommended for studying the stability of slope, Ceryan *et al.* (2018) indicated some of the restrictions which are:

- The LEM access is unable to predict the effect of movement on the general stability.
- LEM is restricted to the evaluation of slope stability with simple problems, such as providing little insight into the slope failure mechanism.
- LEM can only recognise the beginning of slope failure. Complicated rock slope stability problems need a continuum-mechanics-based numerical modelling approach.

2.4.2 Numerical methods

Numerical methods have gained increasing popularity due to rapid development of computing efficiency in slope stability engineering (Boniface, 2013). Comprehensive studies of slope stability opportunity are provided by numerical methods as well as its failure mechanisms. They also provide landslide prediction by offering approximate solutions to boundary value problems for partial differential equations (Boniface, 2013; Cała *et al.*, 2020). The behaviour of the material can be modelled with various constitutive equations and numerical simulation techniques (Eberhardt *et al.* 2004).

Solutions for complex scenarios have been offered using numerical modelling method of analysing instability in rock slopes. Inherent geological conditions make it difficult to design open pit mines when using conventional LEM. Simulating potential rock slope failure mechanisms is one of the solutions that numerical

modelling approach in complex cases provides as well as carrying out a comprehensive rock slope investigation (Stead *et al.* 2006).

Technology improved over the years along with high computer software tools which enabled the introduction of computer codes and computational tools. This provided more comprehensive and reliable slope stability analysis for geotechnical engineering. According to Fredj *et al.* (2019), the finite element method (FEM) and finite difference method (FDM) are the commonly used numerical methods. The approached used in numerical modelling is to divide the slopes into elements. The slope stress-strain relationship and deformation properties are modelled to predict the behaviour of slopes. FOS is determined by numerical modelling software after defining the boundary conditions. In addition, the displacement of rock mass that will occur during failure is predicted (Ceryan *et al.* 2018).

Deformation analysis in slope stability can be predicted using numerical modelling approach which is the main advantage of this method. This can be used to interpret the slope behaviour (Hustrulid *et al.* 2001). There are two major advantages of using numerical modelling as stated by Hammah *et al.* (2005). The first one is more sophisticated and complex problems than LEM can be processed and the ability to compute deformation and displacement of a rock mass. “Secondly, numerical models can analyse complex geometries, simulation of stages in excavation and the influence of stress field conditions and groundwater seepage on the stability of the slope” (Hammah *et al.* 2005).

1.8.1.1 Shear strength reduction technique (SSR)

The shear strength reduction technique (SSR) is the most common method to estimate slope stability. Shear strength of rock can be reduced in stages until the slopes fails from which FOS can be calculated thereafter. Stress reduction factor (SRF) is determined by systematic use of finite element analysis which brings a slope to the verge of failure. The SRF reduces the shear strength of all the materials in FE model of the slope. Critical SRF value that induces stability is achieved after performing the conventional FE analysis of this model. Boniface (2013) stated that

in the SSR technique, a slope is considered unstable when its FE model does not converge to a solution. SSR technique is based on the use of the Mohr-Coulomb strength models for materials (Boniface, 2013).

The approach that SSRT takes is where the rock mass strength properties are reduced artificially in stages to determine the factor of safety. This then adopts elastoplastic FEM analysis until the rock slope failure occurs. Numerically, when convergence solution no longer exists, failure occurs (Abdellah *et al.*, 2022). Figure 2.6 depicts graphically Mohr–Coulomb yield surfaces and Equation (2.2) presents them mathematically.

$$SRF = \frac{\tan \varphi}{\tan \varphi_f} = \frac{C}{C_f}$$

Equation 2.2

where:

SRF is the strength reduction factor

C and tanφ ' are the soil/rock mass shear strength input values/parameters (e.g., cohesion and friction, respectively),

C_f and $\tan \varphi_f$ are the soil/rock mass shear strength reduced or mobilised values used in the analysis (e.g., cohesion and friction at failure, respectively) (Abdellah *et al.*, 2022).

At the beginning of calculations, the SRF is set to 1.0. Equation 2.2 defined SRF which corresponds to the factor of safety as explained in Equation 2.1, when failure occurs. When adopting FE method of analysis, there are no assumptions required about the shape or location of the failure surface. When shear stress is applied and the shear strength is unable to resist, failure occurs through the zones within the rock mass. This analysis was conducted in static drained conditions that assumes effective shear strength and deformation parameters. The effect of seismicity, groundwater level and distributed load was ignored. In the following section, the numerical modelling setup will be presented (Abdellah *et al.*, 2022).

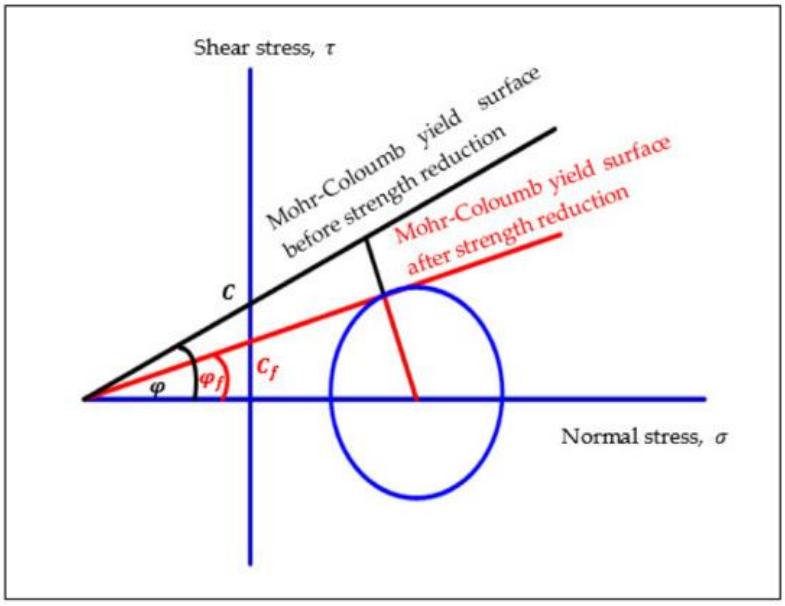


Figure 2.6: Mohr-Coulomb yield before/after strength reduction (Krahn, 2007)

In slope analysis, the approach commonly used to perform FEM is shear strength reduction (Kolapo *et al.*, 2022). A study was conducted by Hammah *et al.* (2005), based on the principal of systematically reduction in the shear strength of materials by FOS. Additionally, the FEM models of the slope computed until the deformations were unacceptably high. Shear strength reduction techniques relate the existing strength to the limit equilibrium strength, which was used to calculate the FOS in the numerical simulation methods (Ceryan *et al.* 2018).

2.5 Rock Mass Characterisation and Geotechnical Parameters

The information used to understand geological, hydrological, and geomechanically properties of the rock is obtained by observation and measurement. Earth sciences and mechanical sciences disciplines are used, especially the engineering geology, geotechnics, soil and rock mechanics, hydrogeology and groundwater hydraulics disciplines (Fleurisson, 2012b). According to SRK Consulting Engineers and Scientists (2009), rock mass characterisation is a largely empirical process of classification based on information obtained from field data. It is enhanced with

further data analysis and laboratory testing. Materials from ground surface to a depth of approximately 30% of the ultimate slope height below final pit bottom and for a distance approximately two time the ultimate pit height behind the slope crest is characterized and represented in the geotechnical model (SRK Consulting Engineers and Scientists, 2009).

To analyse the material behaviour, it is important to take the geological approach first. This information provides guidance and allows for geological and geotechnical field investigation optimisation. These investigations use subsurface geophysical methods, drilling operations or shallow excavations carried out with hydraulic excavators which can, cost effectively, provide valuable information. Pit slopes are usually located in waste materials hence is it important to characterise this material. To achieve this material characterisation , field investigations should be taken solely for geotechnical purposes (Fleurisson, 2012b). If this exercise is done accurately, major slope failures can be minimised. The acquisition of petrophysical and mechanical parameters required for subsequent calculations are then made from intact samples. Petrophysical parameters and characteristics of deformability and strength such as density, different deformability moduli, cohesion and internal friction angle of soil, Shear Strength (SS) parameters of discontinuities are determined by performing standard laboratory tests (Fleurisson, 2012b). It is useful in some cases to perform in-situ mechanical tests in boreholes or on surface (Gundewar, 2014). Table 2.2 details the various tests and the properties they measure.

Table 2.2: Geotechnical test (Jeffrey, 2002)

Field Index tests		Laboratory tests	
Test	**Property**	**Test**	**Property**
Point-load strength	Tensile and compressive strength	Brazilian, pull-apart, point-load	Tensile strength
		Uniaxial, triaxial	Compressive strength
Slake durability	Resistance of clay-bearing rocks to weathering (important for roof slaking in response to air circulation with varying humidity levels)	Direct shear, Mohr's test	Shear strength
		Porosity-permeability test	Porosity-permeability
		Atterberg limits (crushed rock or soil)	Water limit boundaries
		Vicker's hardness test	Coal hardness
		Mechanical strength index	Coal strength

Elastic modulus (stress: strain) is used to determine the engineering properties and deformation characteristics such as elastic, ductile, brittle and time dependency. The rock can be classified in various engineering classifications using strength and stress tests (Jeffrey, 2002). The weight of overburden (gravitational loading) is commonly approximated to be equal to vertical stresses. At depths from 100m to 1000m below surface, the horizontal stresses range from 0.3 to 5.5 times the vertical stress (Dunrud, 1998).

The primary goal of geotechnical investigations as suggested by Ellison and Thurman (1976) is to:

- Accurately outline the three-dimensional picture of the geology (lithology, structure, and hydrology); and
- Predict the interaction of geological components as related to mining; and
- Design the optimum configuration to suit the chosen mining method.

2.6 Geotechnical Model

It is important to have a representative geotechnical model for all slope design. Areas with similar geological, structural, and material properties should be captured in a geotechnical model (Johansson, 2014). The first aspect of a geotechnical model is to include a good understanding of the geological settings of the area where the open pit is going to be mined and a proper geological definition of materials that will be involved in the pit excavation. The deposit and surrounding waste rock should be included in this information. Secondly, the geotechnical model should include a full description of the rock mass structure in the area (Michaud, 2016). In addition, the structures in the rock mass such as folds at the mine scale and at the bench scale, joints or discontinuities should be included in the description. Accurate definition in which the slope is going to be excavated is another aspect of geotechnical model of the material properties.

Considerations such as rock mass strength and rock mass behaviour as well as any other possible change in time should be made. Additionally, the effect of mine activity and mine operation on the behaviour of rock mass should also be considered. Slope stability analysis should also take into consideration the effects of surface water and groundwater flow. Any mining activity that has possibility to change the hydraulic properties of slope materials along with intervention that could reduce their negative effect, should also be taken into consideration (Michaud, 2016).

The geological, structural, rock mass and hydrogeological models are components that build a geotechnical model for open pit slope design. This is basically the first step to be taken when designing the pit slope. These four components of the geotechnical model also include sub-components (Fillion, 2018).

2.6.1 Geological model

According to Read and Keeney (2009) the geological model has one sub-component which is the physical setting of the project site. This includes the geographic location, tectonic evolution, climate, geomorphology, topography, and drainage system. Basic geological units are defined by the orebody environment and the geotechnical requirements such as rock type, major structures, and mineralisation, which also forms part of geological model. Other sub-components of the geological model as defined by Fillion (2018), are the regional seismicity (distribution of earthquakes, seismic risk data) and the regional stresses.

2.6.2 Structural model

Structural model sub-components include the major structures, the fabrics, the geological environment, the structural modelling tools used and the structural domains. Folds and faults that are continuous along strike and down dip across the mine site, and metamorphic structures can be defined as major structures. Fabrics include joints and minor fold structures (fracture cleavage, tension gashes, boudinage structures and slickensides) (Read, 2009).

2.6.3 Rock mass model

Series of rock parameters such as the intact rock strength, the index properties, the mechanical properties, the strength of structural defects, the rock mass classification, the rock mass strength and some special conditions are included in rock mass model concepts (Karzulovic and Read, 2009).

2.6.3.1 Rock mass strength

Several factors that influence the overall strength of a rock mass are assessed using rock mass classification systems. These rock mass classification include the influence of the intact rock strength and the fractures, to essentially grade the quality of the rock mass to determine its overall strength and deformation characteristics (Golder Associates, 2016). It is important for rock mass characteristics to be considered since the implementation quality of drilling and blasting depends on intact rock characteristics and the structure of the discontinuity. Ultimately the results of drilling and blasting process significantly affects the effectiveness of the loading and hauling process (Taherkhani and Doostmohammadi, 2016).

2.6.3.2 Unconfined compressive strength testing

According to Hoek and Brown (1997) “The uniaxial compressive strength (UCS) test involves the application of a steadily increasing axial load upon a core sample with a length-to-diameter (L/D) ratio of, ideally, between 2.0 and 2.5”. In terms of stress, the UCS of the sample is the applied load that produces failure divided by the cross-sectional area of the core.

2.6.3.3 Triaxial compressive strength testing

Under triaxial compressive strength (TCS) conditions, a core sample is encased in an impervious membrane and subjected to a selected confining pressure (σ^3) while the sample is loaded axially (σ^1) until failure occurs. The applied load that results

in failure divided by the cross-sectional area of the core is the triaxial compressive strength given the confining pressure (Hoek and Brown, 1997).

2.6.3.4 Direct shear testing

Direct shear testing can be defined as “a load perpendicular to a discontinuity separating two blocks of rock is applied while continuously monitoring the shear stress necessary to displace the block relative to each other” (SRK Consulting Engineers and Scientists, 2009). This method is extensively used to determine the expected shear strength along natural rock discontinuities such as joints, fractures, and faults. In open pit, is it important to determine the discontinuity shear strength since displacement frequently occurs along pre-existing geologic discontinuities. Direct shear testing is the preferred method in open pit design instead of triaxial compression testing for estimating discontinuity shear strength. This is because direct shear testing permits a higher degree of control over the selection of the actual surface tested (SRK Consulting Engineers and Scientists, 2009).

2.6.3.5 Classification of rock mass in open pit mines

For geotechnical purposes several rock mass classifications have been developed such as Q-Index by (Barton et al. 1974), Rock Mass Rating (RMR) by Bieniawski (1973), etc. The Q-system of the rock mass classification was developed on Norway in 1974 by Barton, Lien and Lunde. It is a quantitative classification system and facilitates the design of the tunnel supports. The system is based on numerical assessment of the rock mass quality using six different parameters:

RQD is the rock quality designation;

Jn is the joint set number;

Jr is the joint roughness number;

Ja is the joint alteration number;

Jw is the joint water reduction factor; and

SRF is the stress reduction factor.

The Haines and Terbrugge empirical design chart were developed based on the Laubscher Modified Rock Mass Rating system (MRMR). It is one of the best known and most widely used chart. Using the design charts have limitations in their experimental and semi-quantitative nature. The Haines and Terbrugge chart (Figure 2.7) acknowledge these limitations in which the slope angle and slope height can be determined solely based on the MRMR, where the estimate is marginal and where additional analyses is required (Haines & Terbrugge, 1991).

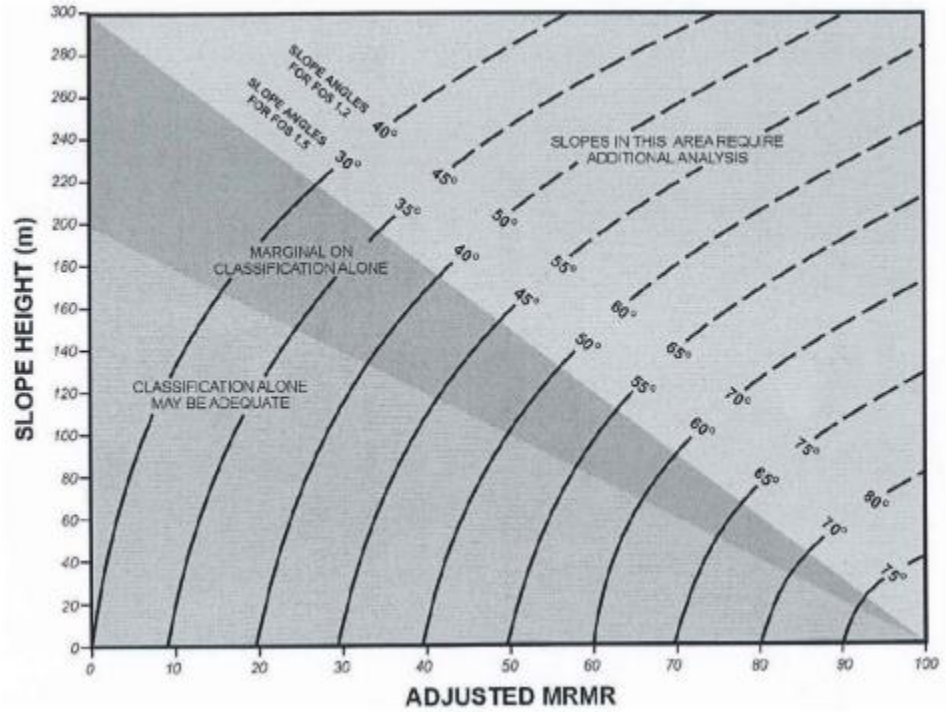


Figure 2.7: Haines and Terbrugge empirical design chart (Haines & Terbrugge, 1991)

The RMR system proposed by Bieniawski in 1973 provide the quantitative data for the selection of modern tunnel reinforcement measures. The RMR was initially developed for tunnels but has been applied to rock slopes. The purpose of RMR is determining the engineering properties of the rocks in shallow tunnels excavated in sedimentary rocks. Laubscher presented the modified classification of the rock

mass rating (MRMR), with changes in RMR system. (Karzulovic and Read, 2009; Taherkhani and Doostmohammadi, 2016)

To use the final ratings (MRMR) in the mine design, the RMR had to be adjusted according to the mining environment. Weathering, mining-induced stresses, joint orientation and blasting effects are the adjusting parameters. Some rock types weather easily, and this must be taken into consideration in decisions on the size of opening and support design. The timing of support installation and the rate of mining is influenced by weather as it is time dependent (Laubscher,1990). The adjustments that must be made as Table 2.3 depending on the degree of weathering on the rock mass.

Table 2.3: Weathering adjustments (Laubscher,1990)

Degree of weathering	Potential weathering and adjustments, %				
	½ y	1 y	2 y	3 y	4 + y
Fresh	100	100	100	100	100
Slight	88	90	92	94	96
Moderate	82	84	86	88	90
High	70	72	74	76	78
Complete	54	56	58	60	62
Residual soil	30	32	34	36	38

The behaviour of the rock mass is affected by the size, shape, and orientation of an excavation. The percentage adjustments for joint orientation that can be applied is as per Table 2.4. The stability of the excavation is significantly affected by the attitude of the joints, whether the bases of the blocks are exposed or not. The ratings must be adjusted accordingly (Laubscher,1990).

Table 2.4: Joints orientation adjustments (Laubscher,1990)

No. of joints defining the block	No. 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	2,1

The geometry and orientation of the excavation causes the redistribution of field stresses. This results in mining-induced stresses. Movement in the joints occurs as a result from blasting as it fractures and loosens the rock mass. Table 2.5 highlights the blasting effects adjustment that can be made for the excavation. It should be noted that poor blasting has its greatest effect on narrow pillars and closely spaced drifts owing to the limited amount of unaffected rock (Laubscher,1990).

Table 2.5: Blasting effects adjustments (Laubscher,1990)

Technique	*Adjustment, %*
Boring	100
Smooth-wall blasting	97
Good conventional blasting	94
Poor blasting	80.

2.6.4 Hydrogeological model

The following are the hydrogeological model sub-components (Beale, 2009):

- porosity and pore water pressure,
- storability (under confined conditions),
- mine dewatering and pore pressure controls,
- depressurisation effects,
- groundwater flow (heterogeneity and anisotropy),
- porous medium groundwater settings,
- fractured flow ground water settings,
- influences on fracturing and groundwater,
- mechanisms controlling pore pressure reduction, and
- conceptual and numerical hydrogeological model.

Examples of the contexts for groundwater in porous media include unconsolidated deposits, sedimentary rocks, clay alteration, and weathering. Fracture flow, lateral flow barriers and are all components of fractured flow of ground water systems. Groundwater flow, lithostatic unloading, hydrostatic unloading, and/or piping of slope materials are some of the mechanisms that oversee pore pressure reduction.

The conceptual hydrogeological model comprises of developing a mine scale hydrogeological model, a comprehensive hydrogeological model of the pit slopes, and integrating the pit slope model into the regional model (Fillion, 2018). Lastly, the numerical hydrogeological model for the mine scale dewatering application is established for the numerical modelling of the pit slope pore pressures (Fillion, 2018).

There are two common classes in which groundwater can be viewed, which are water extraction in advance of mining and water extraction during mining. The two methods are be used one at a time or collaborated to bring about required solution. The method of the relevant depressurisation procedure relies on the local and regional hydrological conditions (Department of Minerals and Energy, 1999).

2.6.5 Geotechnical model summary

The information needed by each component of the geotechnical model is shown in Figure 2.7, (Guest and Read, 2009). A substantial amount of time and money must be invested to develop a trustworthy geotechnical model. The mining industry recognises the significance of utilising reliable geotechnical data to construct the geotechnical model for the slope design. Conceptual or pre-feasibility level of inquiry has frequently been utilised to support inappropriate operations level investment decisions. This may lead to slope failures, which could cause operational delays and raise the cost of operating an open pit mine significantly (Fillion, 2018).

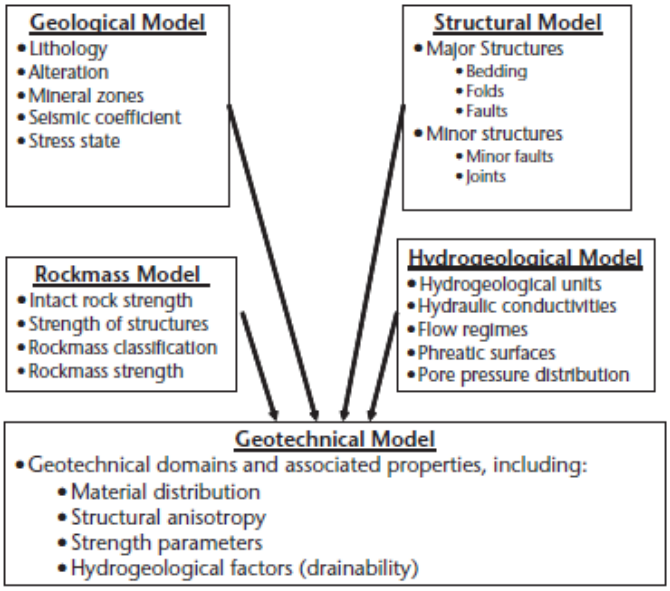


Figure 2.8: Geotechnical model input and output (Guest and Read, 2009)

2.7 Methods of Reinforcement and Monitoring

The slope stability expert will design the slope angles depending on the calculations above to achieve the desired level of stability. It is possible to study several scenarios that either include or do not include reinforcing methods (surface water drainage, rock mass dewatering, mechanical reinforcement employing rock bolts and grouted cables in rock mass, or soil nailing) (Fleurisson, 2012b; Gundewar, 2014). To make decisions that always benefit the mine operator, gains or losses connected to stability including the pertinent costs of these systems and their deployment will be quantified for each scenario. Ultimately, a slope monitoring that utilises a variety of auscultation equipment may be advised in various circumstances, which includes topographic monitoring, groundwater level management, measures of displacement and deformation in drillholes, etc. For all major mining or civil engineering works, monitoring has developed into an ally of modelling and calculations. If a considerable initial investment is made early in the LOM, then a continuous measurements and computations must be done. This always benefits the mine operator economically and in terms of security, (Fleurisson, 2012b; Gundewar, 2014).

3. Data Collection

3.1 Introduction

Geotechnical considerations should be applied during the mine design and planning phase, mine operation phase and lastly the closure phase. The information used to determine geotechnical input is selected early during feasibility studies. The accuracy and clarity of this information is evident as the mine develops. If geotechnical data collection is not done correctly, it can lead to slope failures. Large slope failures can have impact on costs, damage to equipment and in worst case, injuries or even fatalities can occur (Steiakakis *et al.*, 2013).

3.2 Geological and geotechnical information

Geological and geotechnical information has been interpreted to identify major subdivisions within the pit that dictate fundamental mining strategies. The information relative to these subdivisions is indicated in Table 3.1 with a summary of the geotechnical information identified in the various mining areas.

Table 3.1: Geological ang geotechnical information

Mining Areas	Geological and geotechnical	Regional hydrology	Porosity and secondary permeability
Area 1: Weathered Phyllite (Overburden Stripping)	Soft to very soft phyllite	The diversion of surface drainage by means of the eastern canal was effective and required continuous monitoring.	The overburden may become saturated under adverse conditions. Dewatering efforts using drain holes was inhibited by poor rock strength. Ingress of surface water into surface fill or major faults and joints was prevented.
Area 2: Hard Phyllite Foot wall mining	The phyllite schistosity weathers to extensive and smooth chlorite coated, planes dipping into the pit. Intercalated limestone layers and graphitic schist can occur at depth.	Drain holes were drilled into the weathered phyllite above the haul road.	The weathered phyllite has moderate primary porosity and was effectively drained by drilling of holes into the sidewall.
Area 3: Hard Phyllite and Limestone Hanging wall Mining	Highly laminated phyllite and limestone dipping into the sidewall.	Drain-holes for the dewatering of the semi-weathered phyllite was required.	The phyllite has low porosity but there was potential for secondary permeability within fault or bedding planes.
Area 4: Limestone production mining	The highly foliated limestone body generally dips to the east at a high angle. High angle, cross cutting joint planes occur.	Ground water pools at the base of the pit but was not evident on production benches.	The secondary porosity in the limestone results in relatively high volumes of ground water. This water has been depleted in the “bridge” due to drainage to the southern pit floor.

3.3 Field observation and other information

Digital photographs were taken of the pit along with old pictures that were relevant to the study was collected from the mine documents. Slope angles were taken from the plans. Rock mass classifications were made using the Q Index and

RMR/MRMR processes. Field observations allowed for the detailed visual analysis of different sections of the quarry to be done and will be discussed in the following chapter. Exploration core logs were used to obtain and analyse geotechnical features in the quarry. Due to limitation of not having the laboratory access, no laboratory tests were conducted to determine the strength of the rock. Surpac software was used to schedule production.

3.4 Regional geology

The Zoutkloof limestone deposits lie in the Porterville Formation of the Malmesbury Group. The rock formations are an assemblage of sedimentary and low-grade metamorphic rocks of Precambrian age. The limestone formations are intercalated with minor bands of phyllite and graphite. The body dips steeply to the north-east at 55 to 80° from the horizontal and has a strike of 325°. The strike length of the Zoutkloof limestone body is 1 600 m. The width of the limestone is in the order of 100 to 150 m. The phyllites of the Malmesbury Group vary from completely and highly weathered to fresh, hard phyllite rock at about a depth of about 40 m. Weathering also occurs along fault lines to greater depths. The original depth of topsoil was about 0,25 m. The Malmesbury Group is overlain to the west by the quartzite, shale and conglomerate of the Cape Supergroup and Piekenierskloof Formation which dip to the east at 12 to 25°

3.5 Summary

This chapter has detailed how the data to be used in the geotechnical analysis was collected. The geological setting of Zoutkloof quarry was highlighted to give an understanding of the deposit. The geotechnical analysis will mainly focus on the three sections of the quarry which is the footwall, Hangingwall and the limestone orebody.

4. Geotechnical Analysis

4.1 Introduction

The section will discuss the geotechnical parameters that were significant at Zoutkloof quarry and that had impact to mine planning and overall safety of the operation. This includes hydrological, blasting, pit wall design, ground support, geological structure and rock mass strength, and monitoring. The information will be based on observations made from the quarry and by measurement.

Kinematics analysis will also be discussed to determine the possible failure modes for different sections of the quarry. Other geotechnical characteristics that will be discussed is the joint sets and orientations. Additionally, stereographic analysis will also be included in the discussion.

4.2 Hydrological Considerations

Historically, before mining in Zoutkloof, there was a river that passed through the mining area. It was therefore necessary to start by diverting the river before any mining could commence. The stream was diverted to the eastern side of the quarry. For the case of Vondeling quarry, the stream should also be diverted to the western side of the quarry as it is in the natural low laying area and will have minimum disturbances to the mining operations.

To ensure slope stability, regular drainage holes were drilled up to 50 m into the surrounding rock mass. Most were dry and the few which were not, only exhibit seepage. This suggests that the slopes are not exposed to water pressure (uplift). It also suggests that the phreatic water table has been successfully drawn down and the risk of water causing uplift in the steep joints have been largely obviated. After heavy or persistent rain, a competent person examines the slopes to ensure they are still stable. The person pays particular attention to the drain holes and joints to ensure that the water condition has not changed.

In addition, a dam was constructed on the southern side of the quarry to catch all the surface water. Rainfall season in Western Cape is during winter months between May and August. During that period the mine is permitted to pump water into the river stream.

4.3 Blasting practices

One of the most significant effects on slope stability comes from blasting. By its very nature, blasting will lead to the formation of fractures and lower the rock mass strength. The amount is dependent on many variables, including the type of explosive, blast hole burden and spacing, direction and timing. At Zoutkloof, the slopes were pre-split leaving the characteristic barrelling behind (Figure 4.1).



Figure 4.1: Evidence of pre-split blasting on Eastern side wall

The pre-split is performed at the planned slope angle, and this leaves the overall face angle very close to that of the individual benches. Without the good drainage

and pre-split, the faces would not be left in such a stable shape. This was the lesson learned while mining at the Old De Hoek quarry and should also be carried into Vondeling quarry on the western side. However, since the slope stability strategy is dependent on such practices, mishaps could have negative consequences.

4.4 Geological Structure and Rock Mass Strength

Rock mass failure occurs when the driving energy being on a given body of material surpass the resisting forces within that body of material (Department of Minerals and Energy, 1999). The driving force rely fundamentally on the rock mass strength of the rock, and the shape and size of the wall or slope. As a result, the number, size, and shape of rock blocks that sometimes form inside the pit walls are mainly controlled by the design size, shape, and inclination of the open pit relative to the geological structure. The same applies to the design and selection of any ground support (Department of Minerals and Energy, 1999).

4.4.1 Ground control districts

Zoutkloof has been subdivided into areas (Table 4.1) of similar slope behaviour and failure modes that can be addressed similarly during mine design and mining. These areas are called Ground Control Districts (GCD). The GCDs are further illustrated in Figure 4.2.

Table 4.1: Ground control districts

Ground Control District	Main lithology	Location
GCD1a	Weathered phyllite	Top of West wall extending beyond the rock bridge into the secondary pit at a similar elevation
GCD1b	Fresh phyllite	Centre of west wall extending beyond the rock bridge into the secondary pit at a similar elevation
GDC1c	Limestone ore body	Base of west wall, extending on the pit floor
GCD2	Mixture of the units above, but with more structural disturbances and presence of graphite bands	Southern wall
GCD3	Limestone	North side of main pit
GCD4a	Weathered phyllite	Top of East wall extending beyond the rock bridge into the secondary pit at a similar elevation
GDC4b	Fresh phyllite with limestone near the base	East wall below GCD 4a, extending beyond the rock bridge into the secondary pit at a similar elevation

Ground Control Districts 1a, 1b and 1c – west wall of main pit.

The west wall is neatly encapsulated as a GCD with very similar rock structure leading to similar rock related risks. The wall has been spilt into three separate sub-units; 1a, for the weathered phyllite, 1b for the fresh phyllite and 1c for the limestone. This is because the three materials present have different strengths and therefore different slope angles, Figure 4.2.

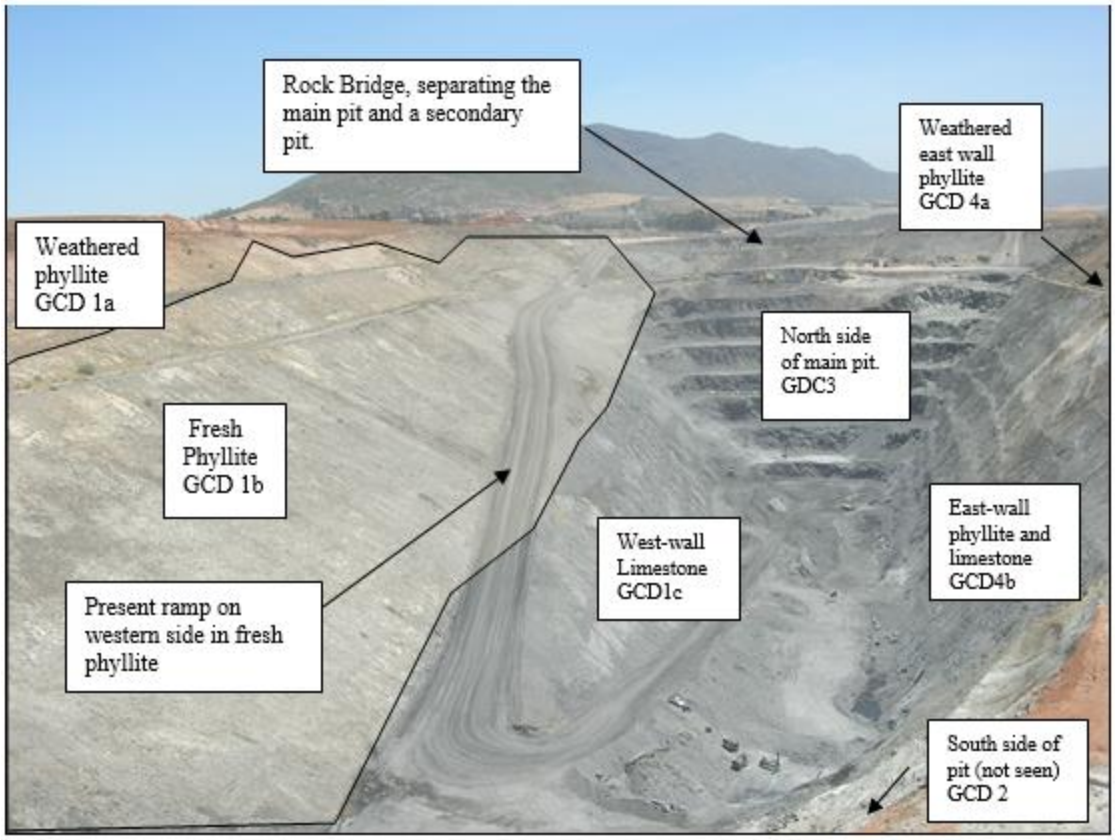


Figure 4.2: Zoutkloof ground control districts

Ground Control District GCD2- South wall of main pit.

This slope is in final position and due to the more complex geology, has resulted in a necessarily lower slope angle being employed. The potential for more deep-seated failure has been negated by the lower slope angle and the individual benches have been protected by backfilling over the most at-risk slope. Wide catch berms have been left to protect workers and machines in the lower levels from rolling rock and appear effective (Figure 4.3).

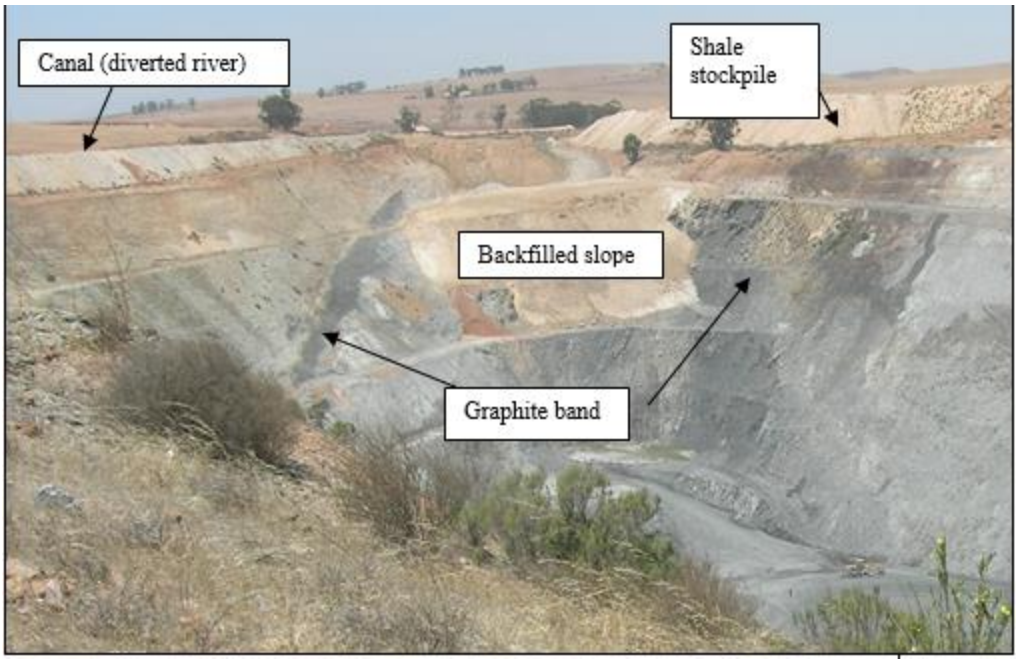


Figure 4.3: GCD on south wall

Ground Control District 3 – North face of main pit

This district is in limestone and the foliated nature of the material lends itself to unravelling and potential wedge failure of individual benches. Of interest in Figure 4.4 is the highlighted undercut of GCD4b. This undercut area vastly increases the risk of the slide failure by removing the toe and indicates a degree of carelessness in the pre-split blasting.



Figure 4.4: Undercut area

Ground Control District 4 – East wall of main pit

This district is in the weathered and fresh phyllite present in the east hanging wall (Figure 4.5). Four survey pegs are mounted around the weak zone which slumped, and these indicate that no movement has occurred for years, indicating that the push back and introduction of drainage holes after the initial movement has resulted in stability.

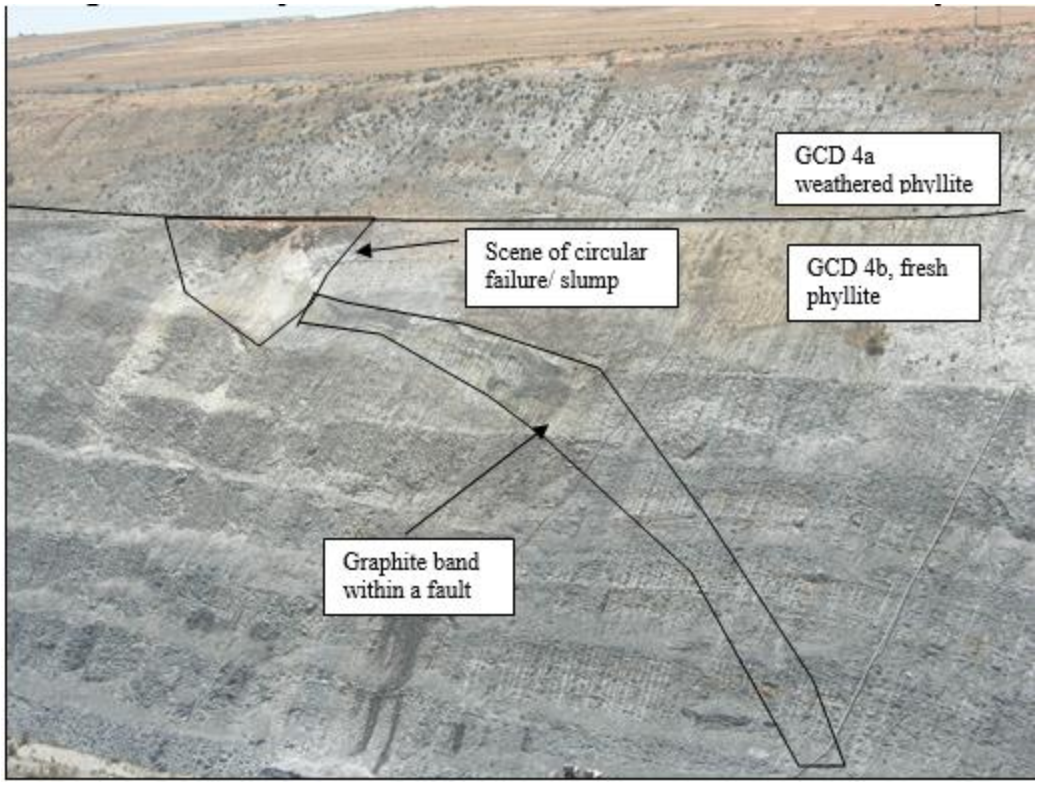


Figure 4.5: GCD on east wall

4.4.2 Field observations

No signs of immediate incipient major slope failure were noted. This was determined by inspecting the angle of vegetation growth, presence of bulging, crest slumping, tension cracks and general bench failure. All aspects suggested that the overall slopes were stable with no mass movement noted. Minor cases of joint dominated plane, wedge and toppling was noted.

No dykes were noted, but isolated shear and faults were picked-up. These will have only minor influence if the slopes remain well drained. The steeply dipping nature of the structures and only localised alteration, coupled with the near surface elevation suggests that their impact is minimal from a stress perspective.

On the western wall, the threat of slope instability comes from the potential for wedge failure. Most of the slides will be individual sheets of the surrounding phyllite

rock which provide little terminal threat to the operations as mining has already stopped in Zoutkloof.

The Eastern side is composed of phyllite on the upper levels with toppling as the main threat. The lower slopes are in the more competent, but structurally like limestone. Therefore, the main slope instability will also be in the form of toppling.

4.4.3 Joint analysis

The footwall slope is comprised of phyllite material with minor fault dipping from West to East on the North-West ramp. There are two sets of joints on the South which are slight rough and undulating. There was no presence of groundwater at the time of evaluation. The phyllite is foliated and graphitic layers in the limestone dip into the hanging wall. The slope is susceptible to blast damage that would create blocky conditions. The hanging wall is made of weathered, fresh, and harder phyllite from top to bottom respectively. The joints are rough undulation and with major fault in the South following the dip of the limestone from West to East. The joints spacing ranges between 1 m to 2 m. The joints on the limestone bedding are smooth, undulating with clay stains.

4.4.4 Determining RMR

To calculate the Rock Mass Rating (RMR) of the rock mass conditions, Table 4.2 which is the Bieniawski's method. An adjustment for strike and dip orientations of joints was applied to this value. The RMR was developed and provide quantitative data for the selection of modern tunnel reinforcement measures such as rock bolts and shotcrete, (Bieniawski, 1989).

Table 4.2: RMR evaluation (Bieniawski, 1989)

Parameter			Ranges 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 compressive Strength	>250 MPa	100-250 MPa	50-100 MPa	25-50MPa	5-25 MPa	1-5 MPa
	Rating		15	12	7	4	2	1
2	Drill core quality *RQD*		90%-100%	75%-90%	50%-75%	25%-50%	<25%	
	Rating		20	17	13	8	3	
3	Spacing of joints		>2m	0.6-2m	200-600mm	60-200mm	<60mm	
	Rating		20	15	10	8	5	
4	Condition of joints		Very rough surface Not continuous No separation Weathered wall Rock	Slightly rough Surfaces Separation <1mm Slightly weathered Walls	Slightly rough Surfaces Separation <1mm Highly weathered Walls	Slickensided surfaces, or Gouge<5mm thick, or Separation 1-5mm continuous	Soft gouge >5mm thick, or Separation >5mm continuous	
	Rating		30	25	20	10	0	
5	Groundwater	Inflow per 10m Face length (l/min)	None	10	10-25	25-125	>125	
		Joint water press-ure/major principal stress	0	0.0 – 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	
6	Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	
	Rating		0	-2	-5	-10	-12	

The RMR was calculated by the summation of the five parameters minus the joint parameter as shown in Table 4.3 for each ground control district (Appendix A). The conclusion of these parameters was done by assessing the joints and rock properties in the pit and exploration core logs. Interestingly, the results suggest that the south wall (GCD2) is the “best”. This is due the relatively large block size present. The reality is the graphite bands have a very low friction angle and cannot sustain a steep slope. For this reason, care must always be taken when using these empirical methods for determining slope angles.

Table 4.3: Summary of RMR values for all GCDs

Area	Strength	RQD	Joint spacing	Joint condition	Water	Joint orientation	RMR	Rating
GCD1a	2	8	8	25	15	-10	48	Fair
GCD1b	7	3	8	25	10	-10	43	Fair
GDC1c	7	8	10	25	15	-10	45	Fair
GCD2	4	10	15	20	15	-10	64	Good
GCD3	7	3	10	25	15	-10	50	Fair
GCD4a	2	8	8	25	15	-10	48	Fair
GDC4b	7	8	8	25	15	-10	53	Fair

The Mining Rock Mass Rating (MRMR) system is one if the methods used to characterise the rock mass competency, (Laubscher and Jakubec, 2000). The basic functions of the MRMR classification system (Laubscher and Jakubec, 2000) are to:

- Based on similar behaviour, it subdivides the rock mass into zones;
- Communication between various mining disciplines is provided; and
- Formulate design parameters for the actual mine design.

MRMR recognises that the in-situ rock mass rating (IRMR) must be used in the adjustments according to the mining environment so that the MRMR can be used in the mine designs. The four adjustments of Laubscher's MRMR are as detailed in Table 4.4. This are then used to determine MRMR.

Table 4.4: MRMR adjustments (Laubscher, 1990)

Parameter	Possible adjustments, %
Weathering	30 - 100
Joint orientation	63 - 100
Induced stress	60 - 120
Blasting	80 - 100

Apart from the southern wall, which has been commented upon, the figures correlate closely with each other across the methods used. Using the RMR values obtained in Table 4.3 and the adjustments in Table 4.4, the MRMR for Zoutkloof Pit was calculated to be between 37 and 46.

Haines and Terbrugge (1991) developed a relationship of MRMR against overall slope angle for a safety factor of at least 1.2 (active slopes) and this is given in Table 4.5. From the table, an MRMR of 37 (the lowest) will plot at a slope angle of between 45° and 50°. This gives a first approximation which can be used for slope design, and it suggests that the overall slope for Zoutkloof Pit can be up to around 50°.

Table 4.5: Relationship between overall slope angle and MRMR

Adjusted *MRMR* rating	100	90	80	70	60	50	40	30	20	10	0
Overall Slope Angle	>75^{o}	75^{o}	70^{o}	65^{o}	60^{o}	55^{o}	50^{o}	45^{o}	40^{o}	35^{o}	<35^{o}

4.5 Slope Angles

The overall height of the slope is also an important parameter. The maximum depth of mining reached around 165 m at the end of the LOM. Haines and Terbrugge (1991) offered the empirical method to quickly determine the maximum face height for a given MRMR (37 to 40) and overall slope angle with a factor of safety of 1.2, recommended for permanently static slopes. From Figure 4.6, at slope height of 165 m and MRMR between 37 to 40 gives the slope angle of 40° (Red lines) for design purposes. Additionally, chart indicates that the slope design would be marginal of classification alone. This might require additional analysis to ensure slope design safety if slope support is not considered.

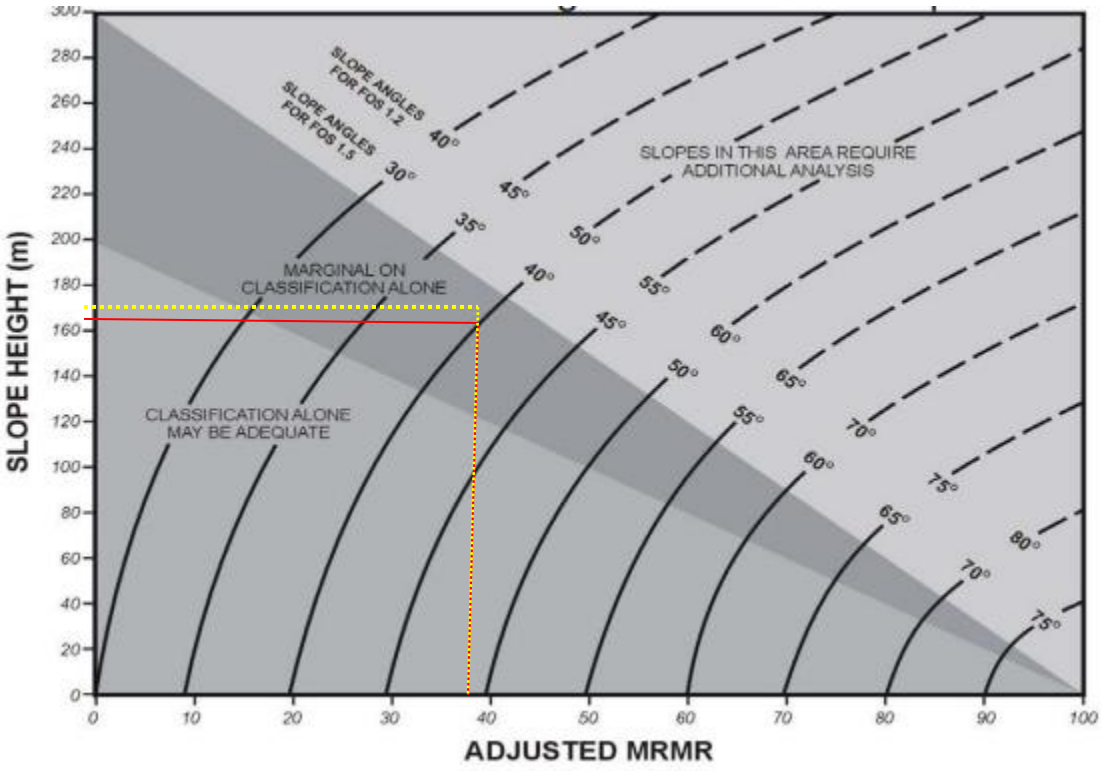


Figure 4.6: Empirical slope design chart (Haines & Terbrugge, 1991)

However, if another level was developed, the pit will become around 170 m deep (Yellow lines), and the slope angle would be around 38°. This means the overall

slope will still be in the marginal area, but the slope is a bit flatter compared to the height of 165 m.

4.5.1 Kinematic analysis

DIPS software was used to draw the stereographic projections to analyse possible failure modes that were likely to occur at different slope angles. The analysis investigation was for slope angles at 50°, 60° and 70°. The data used is as per Appendix B at Zoutkloof footwall +37 m, -81 m and hanging wall at -28 m.

Two major joint sets were identified for both footwall and hanging wall. The friction angle was assumed to be 30° and lateral limits to be 20°. The probability of planar failure for both footwall and hanging wall slope at 50°, 60° and 70° slope angle is 0%, as per figure 4.7.

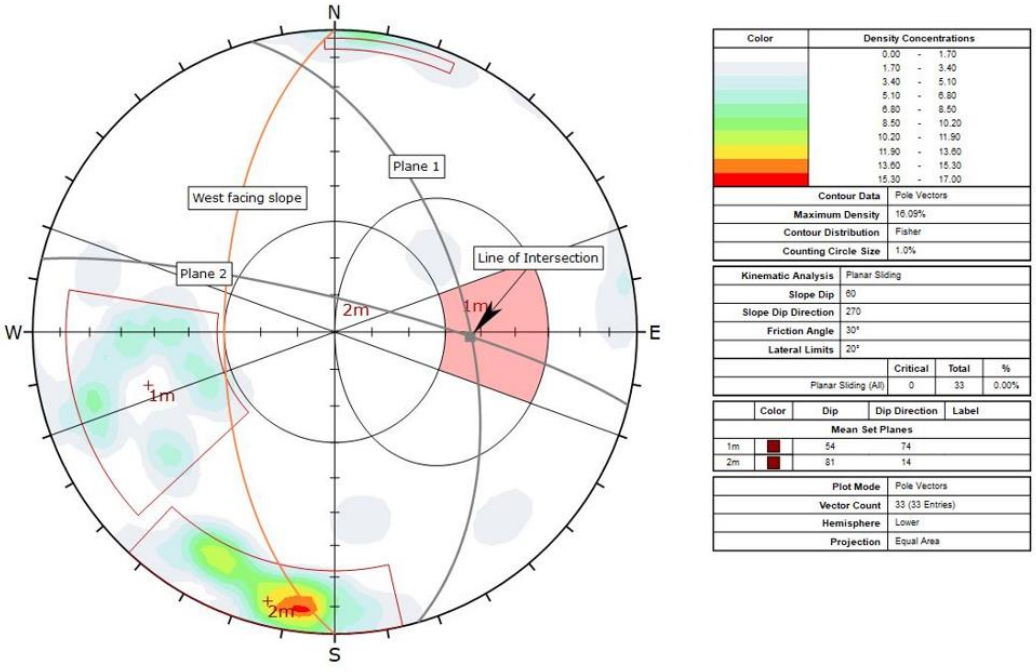


Figure 4.7: Stereonets at 60° slope angle

At the slope angle of 50°, it was determined that the probability of failure on toppling mode was possible at 27% for direct toppling, Figure 4.8. Similar results were

obtained for 60° and 70° slope angle with probability of failure at 31.5% and 33.7% respectively. This validates the results from using the MRMR to determine the overall slope angle which was up to 50°. Slope angle steeper than that would have been likely to fail, possible minor failure.

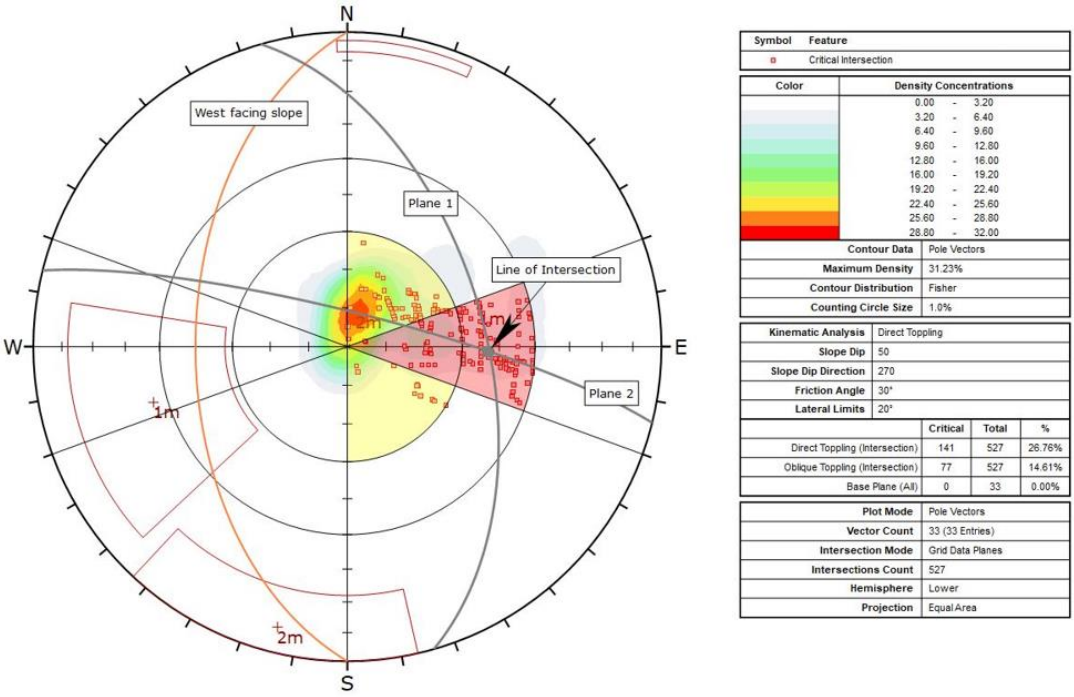


Figure 4.8: Stereonets at 50° slope angle for toppling failure

The analysis for wedge results showed that for all slope angle considered, there was minor probability of wedge failure occurring. The probability percentage ranged between 2.47% to 6.07%, Figure 4.9.

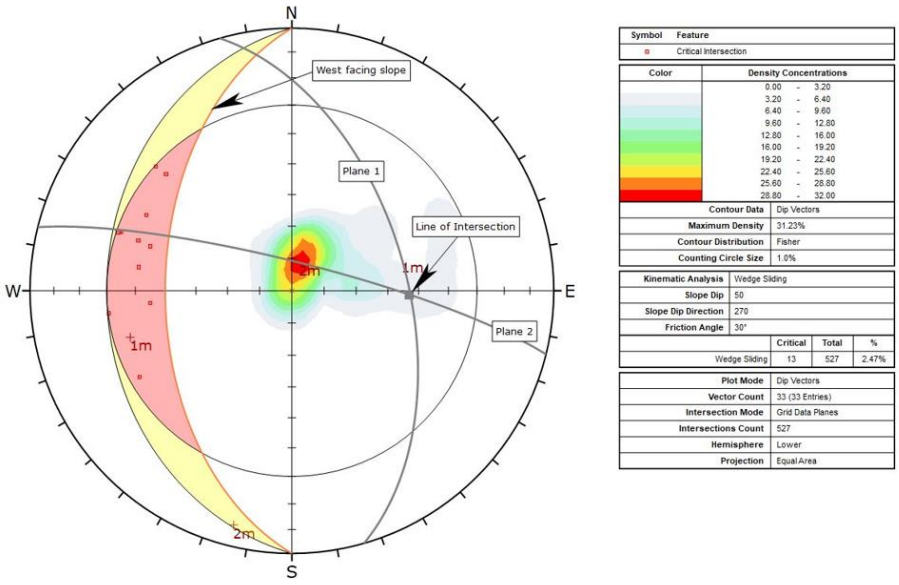


Figure 4.9: 50° slope angle wedge failure

4.6 Pit Wall Design

Prior to mining it is required to put in place relevant excavation design geometry that the mine plan can be based on. The design can change from time to time as more information becomes available as the mine develops. The final geotechnical design developed prior to mining should be adjusted sufficiently to anticipate local ground conditions before mining starts. Therefore, the possible hazardous rock mass failures to happen during mining is minimised (Department of Minerals and Energy, 1999).

The final open pit wall design should represent the balance between safety and the economic viability of the operations since it is not practical to plan the pit walls for permanent stability. It is mostly said that the best geotechnical design is one that fails the day mining ceases (Department of Minerals and Energy, 1999).

The slope design at Zoutkloof pit was based on the practical experience gained from the mining of the De Hoek pit. The De Hoek stability assessment was done by SRK.

The Zoutkloof pit sidewall design criteria strategy is explained below:

Footwall:

- Soft phyllite overburden is excavated to about 25°. This soft material extends to variable depths up to about 30 m. Two berms were established at the 80 m and 55 m levels. This coincides roughly with the base of the weathered zone.

- The harder phyllite is excavated or is pre-split and blasted to an angle shallower than the dip of the phyllite schistosity.

- Graphitic limestone on the footwall contact is pre-split and blasted to the angle of dip of the limestone.

Hanging wall:

- The weathered overburden is cut to 35° and a narrow bench is retained at its base.
- The harder phyllite is pre-split to an angle of 55° to 60°.
- The intersection of limestone at the base of the slope is pre-split to 60°.

North and south pit limits:

- The northern and the southern pit slopes are designed to 45°. The RMR and MRMR for the various GCDs were calculated using a variety of methods as explained above in previous sections.

The RMR for Zoutkloof pit is described as “fair” and ranges from 42 to 54. The adjustment in the RMR value caters for extraneous effects such as blasting damage. Blasting at Zoutkloof yields a very stable skin due to the use of pre-spilt holes. This good blasting coupled with the good drainage leads to the RMR being adjusted by the minimum of five points. Thus, the MRMR for Zoutkloof pit ranges from 37 to 46. Zoutkloof pit was then geotechnically mapped to confirm the joint pattern and rock mass parameters (Figure 4.10).

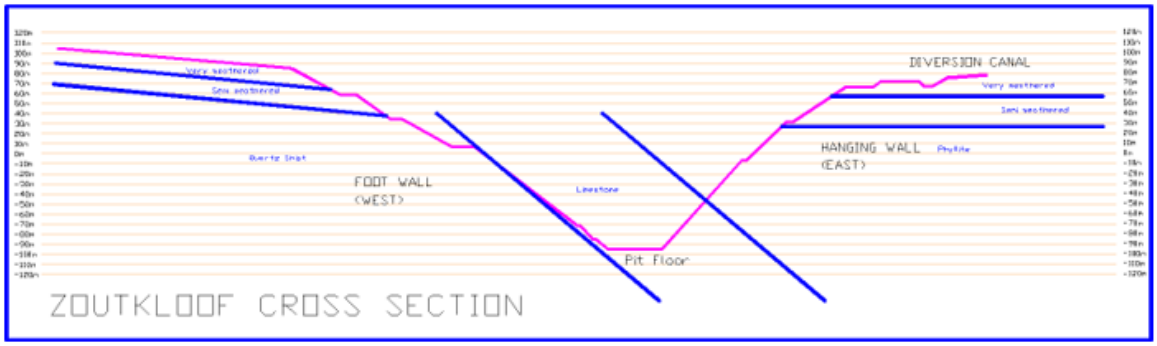


Figure 4.10: Zoutkloof quarry cross section (PPC De Hoek Mine, 1995)

This was duly performed with the following numerical modelling results as per Table 4.6. The results clearly show that with drain holes drilled at set intervals, keeping the phreatic water table away from the slopes, the factor of safety for both the hanging wall and footwall is acceptable in the pit.

However, if drainage is not used, then the factor of safety drops dramatically, due to an increase in pore water pressure. In the hanging wall, due to the general dip of the geology, the difference is more pronounced.

Table 4.6: Numerical modelling results of Zoutkloof slopes (PPC De Hoek Mine, 2020)

		Factor of safety (FOS)	**Maximum heave (mm)**
Hangingwall	With drainage	**1.83**	34
	Without drainage	**1.41**	42
Footwall	With drainage	**2.25**	3
	Without drainage	**2.14**	3

From the results, the model indicates that Zoutkloof is stable with the footwall being more stable than the hanging wall, due to its shallower slope. This agrees well with the experience that showed stability in all cases in De Hoek, except where:

- A graphitic fault was encountered in the footwall resulting in localised skin failure; and
- When the drain holes were missing and the hanging wall allowed to become saturated, resulting in widespread skin failure in the upper slopes and potential for the slope to unravel. This action was halted by the re-introduction of the drain holes (and catch benches).

4.7 Monitoring

Although incipient slope failure is not thought to be in progress, the relatively steep slopes and small catch berms mean that the operation has adopted a strategy of zero ground movement. The slopes are regularly monitored by means of beacons placed at frequent, strategic points. Visual observations and the results from the survey beacons suggest that the slopes are stable.

The objective of the slope monitoring and management programme is to ensure that the slope design objectives are not compromised by unforeseen geological or geotechnical features. Sidewall and production face instability that is related to blasting and loading procedures was identified by visual inspections daily and made safe concurrently with production.

Currently, visual monitoring during monthly inspections and annual Rock Engineering audits and COP reviews show that no mass movement is taking place. If that situation was to change, the monitoring would escalate as per Figure 4.11 to include direct survey monitoring and thence to real time survey monitoring by means of a Trigger Action Response Plan (TARP).

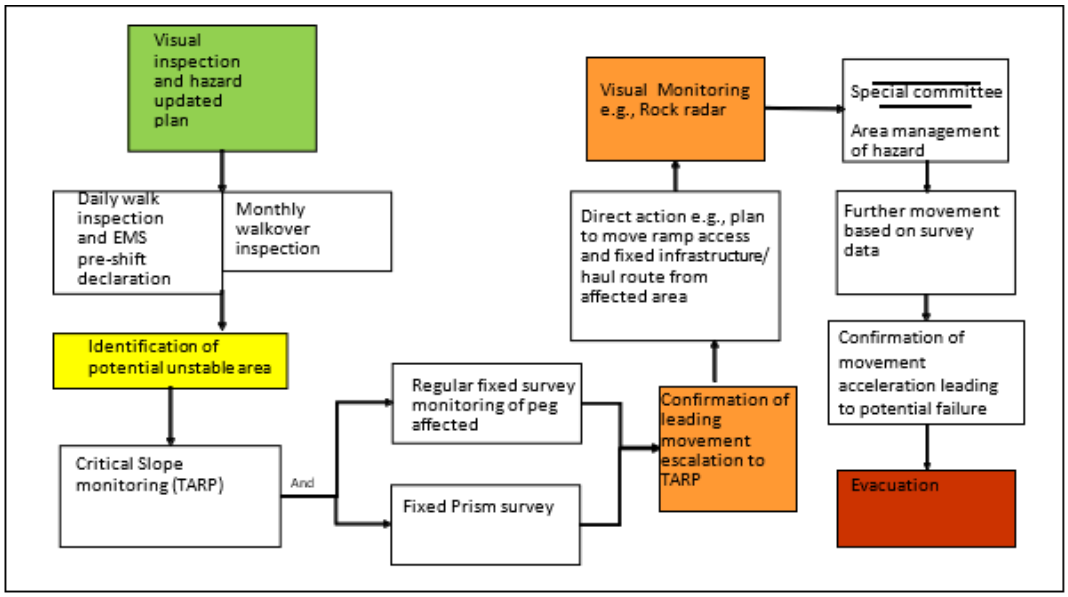


Figure 4.11: Slope monitoring strategy (PPC De Hoek Mine, 2020)

Simply put, TARP describes an escalation of actions in response to suspected or confirmed movement, whether by creep (soft rock and/or soils or ductile movement) or sudden dislocations (hard rock or brittle movement). As per Figure 4.11, if movement is suspected or considered possible, a simple survey network would be set-up to monitor slope movement. This would be response to tension cracks, bulging, increase in rolling rock or unravelling and any other signs of slope distress.

Movement can be broadly classified into two subsets:

- Sudden movement where only a few centimetres could herald a slope failure and;
- Slow, gradual creep over a longer period, where each day of a month, only a little incremental movement is recorded, but eventually it could result in a slope failure.

Prisms or pegs, placed at strategic locations (e.g., slope crests, mid-slope and base or prominent feature such as a bullnose) and surveyed at suitably regular intervals should prove the stability or otherwise of the slope.

Any slope movement generally starts slowly as the limit equilibrium (LE) of driving forces rise close to and exceeds that of the resisting forces. As time continues, especially if the LE condition continues to worsen, the rate of movement will accelerate, usually with “kick points” as a particular resisting point is overcome, and general movement accelerates.

It is most important to realise to ensure that if any person observes or suspects slope instability or if the slope monitoring indicates actual or potential slope instability then the frequency of monitoring would have to increase. This will only be performed in accordance with the recommendation of a competent geotechnical engineer utilising TARP.

A typical TARP is given below and would provide the base for De Hoek. It is probable that the “brittle” description would be used as highlighted in the Table 4.7.

Table 4.7:Typical example of TARP (PPC De Hoek Mine, 2020)

Rock Mass	“Soft”		
Height of slope	< 50 m	50 to 300 m	> 300m
Increase Monitoring Frequency TARP 1	5 to 10 mm per quarter – readings to be taken monthly	20 to 40 mm per quarter – readings to be taken monthly	5 to 10 mm/ day
Continuous Monitoring TARP 2	>40 mm/ per quarter	>40 mm/ per quarter	10 to 40 mm/ day
Evacuate TARP 3	To be formulated once TARP 2 has been activated	To be formulated once TARP 2 has been activated	To be formulated once TARP 2 has been activated

4.8 Vondeling quarry geotech

Based on the geotechnical analysis of Zoutkloof quarry, the experience and lessons learned can be used for Vondeling quarry. The regional geology of Vondeling is the same as Zoutkloof, at the same time the local geology is similar in nature. The strike length of the Vondeling orebody is approximately 1 700 m. The

true thickness of the limestone horizon is about 80 m. Towards north, the limestone deposit is bifurcated into two bands dissected by phyllite (Figure 4.12). The limestone band dips steeply to the north-east at 45 to 80° from the horizontal and has a strike of 310°. The major faulting in the area occurs to the north and to the west of the mining areas. Several minor east-west faults dissect the Zoutkloof (north of Vondeling pit-mined out) limestone body.

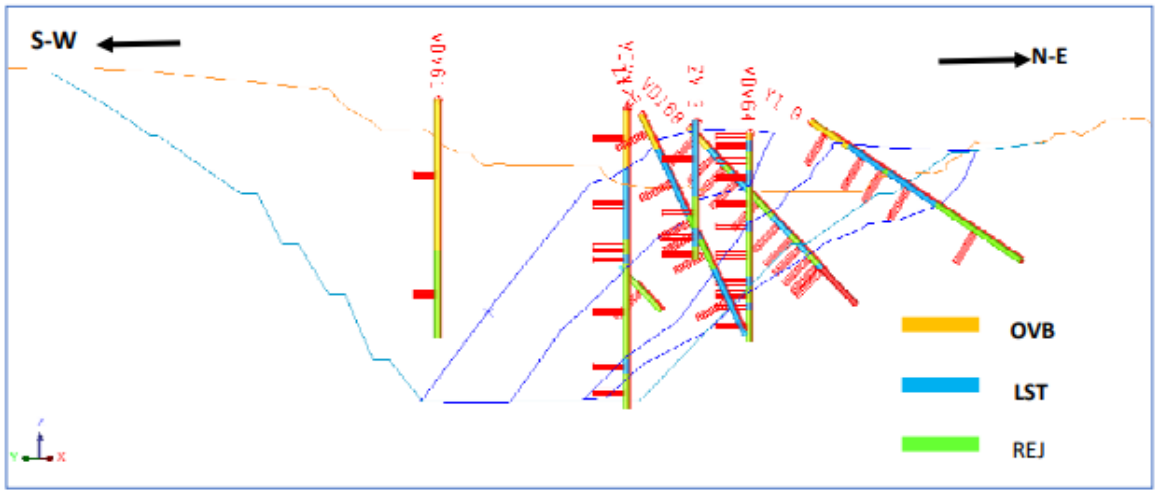


Figure 4.12: Vondeling quarry structure cross section

Based on the structure of the Vondeling quarry, it is evident that the limestone orebody on the west of both quarries was one as explained above. From 2016 there has been a series of minor slope failures in the footwall of Vondeling, with total of five slope related incidents. All are related to the presence of a steeply dipping graphitic schist vein that runs obliquely through the quarry. In 2019, a re-evaluation of the mine design took place and the main ramp had to be relocated from the foot wall to the hanging wall side due to these failures. In addition, 20 m from the footwall slope, there is a stream diversion canal which might affect the stability of the slope. Based on the information above, geotechnical re-evaluation is necessary considering that the quarry still has LOM of 15 years.

4.9 Summary

The geotechnical parameters taken into consideration were discussed in detail. The strategy to divide the quarry into two sections worked well for Zoutkloof open pit. This enabled an adequate dam to be constructed in the southern side which was the lowest level after mining it. The production benches remained dry while mining the northern section. Other water strategies put in place also aided to keep the slopes stable.

The field observations allowed for the quarry slopes to be analysed. Minor faults were noticed but did not pose any threat. Phyllite dominates the host rock mass for Zoutkloof quarry and varies from highly weathered to hard with depth. The joints follow the dip of the limestone from West to East.

Geotechnically, adequate approach into analysing and understanding geological structures and rock mass strength is important. The quarry was divided into different ground control districts and controls were put in place to manage these different areas. The anticipated failures were also outlined according to this GCDs, and this was to be considered during mine planning process.

By using the Bieniawski's method for determining RMR and Laubscher's MRMR, the overall slope angle of up to 50° could be implemented for design purposes. The Haines and Terbrugge empirical chart method indicated that the for the MRMR of 37 and slope height of 165 m, the slope angle can be designed to 40°. Monitoring beacons were installed along the long-term crests and stability confirmed that the slope was stable. The kinematic analysis showed that the chances of having planar failure was at 0% while there was minor possibility of wedge failure. Zoutkloof slope walls were more prone to have toppling failure at slope angles between 50° to 70°.

5. Mine Scheduling

5.1 Introduction

The objective of mine scheduling is to identify mining blocks to be mined for the period at the right quality requirements. The tonnages depend on the planned sales of cement for the financial year. On top of that the sustainable stripping ratio is determined. Overburden stripping ensures that there is enough limestone exposed for the financial year and adequate areas are exposed for the following year. Failure to strip overburden means tight mining faces and a risk of depleting exposed reserves. This situation ultimately has serious financial implications on the business.

This section will focus on mine scheduling between the period 2007 and 2017, which was the last ten years of Zoutkloof quarry before mining ceased in 2017. The aim is to highlight how the geotechnical parameters discussed in the previous chapter affected the mine planning process.

5.2 Year 1 Scheduling (2007-2008)

This was a transition year from mining in the southern section to moving production to the northern section of the quarry. Zoutkloof limestone was mined from southern side, which was the last year of mining. The northern side started to get developed and the main ramp construction was also taking place. The main benches were from level 55 m to level 30 m MASL (Meters above sea level). The northern section was the main limestone block with about 900 000t of limestone. The average screening of 25% was applied. The tonnages are as summarised in Table 5.1.

Table 5.1: Quality requirements 2007-2008

Source	Tonnage	NaK	NaK (After Screening)	RCO_3	RCO_3 (After Screening)	LSF	LSF (After Screening)
ZK south block	90 000	0.67	0.54	68.03	79.6	155	181.35
ZK north block	994 500	0.59	0.47	46.18	54.03	157	183.69

As the ramp got developed, drainage holes were drilled into the slope 50m. This was done to ensure stability on the north section of the quarry. The old entrance going to the overburden dump was closed (Figure 5.1). As mentioned in previous sections, all final wall slopes were pre-split.

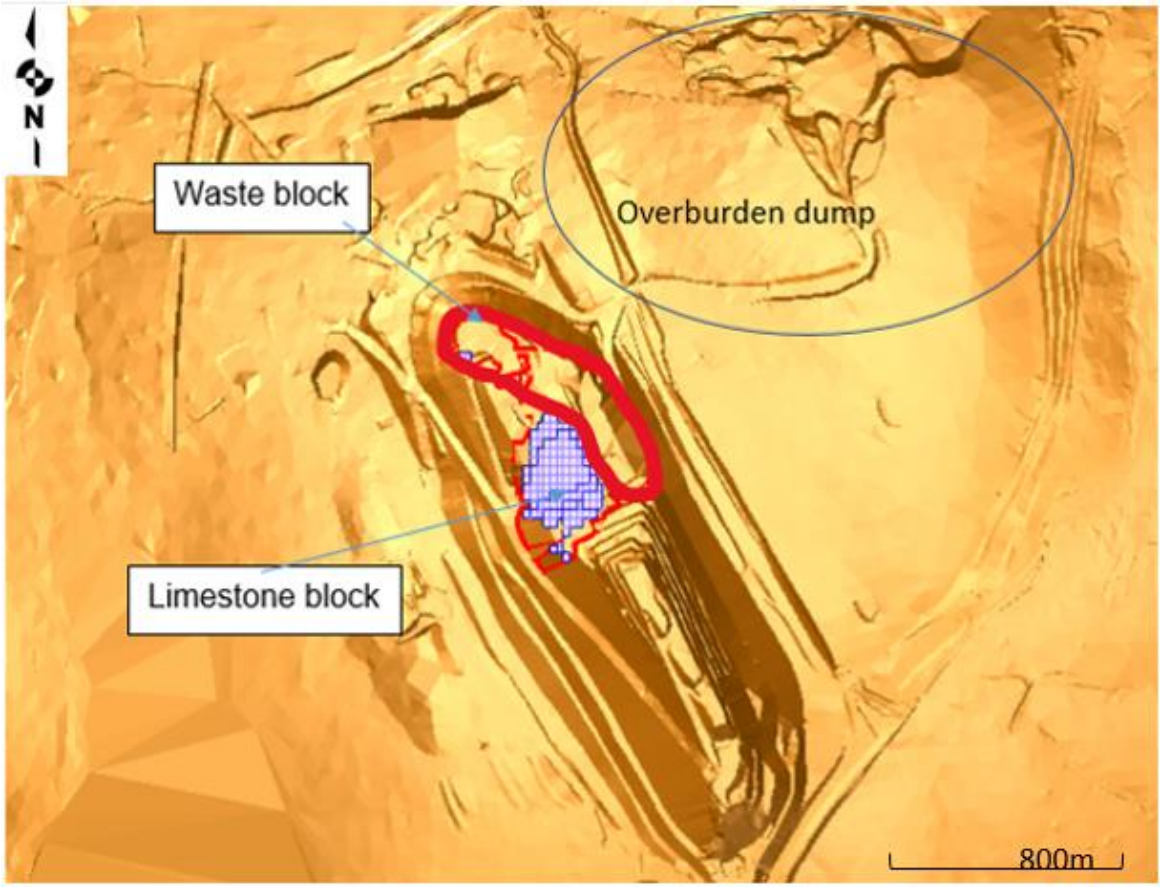


Figure 5.1:2007-2008 mining blocks

Overburden and Development

The access ramp construction had to advance ahead from production mining. The development of lower mining benches and temporary access ramps on the east had to be prioritised. Most of the waste came from ramp construction as the orebody narrows towards the north. As a way of optimisation, the slope design of the quarry followed the dip of limestone without the use of benching method. Even though the quarry is highly jointed in nature, the lessons learned from mining De Hoek quarry, were applied at Zoutkloof quarry. This is because both quarries have the same geological features.

5.3 Year 2 Scheduling (2009-2010)

The production tonnages required for the year 2009 to 2010 is shown in Table 5.2. It should be noted that this was the year when Vondeling quarry started producing limestone. At this point in Zoutkloof, the overburden tonnages had reduced significantly. The NaK, RCO_3 and LSF targets had to be achieved by means of screening. The average screenings for Zoutkloof were 20%.

Table 5.2: Quality 2009-2010

Source	Tonnage	NaK	NaK (After Screening)	RCO_3	RCO_3 (After Screening)	LSF	LSF (After Screening)
Level 30m to 0m	855 000	0.67	0.54	43.3	5066	154	180.18
Vondeling	50 000	0.74	0.59	62.9	73.69	120	140.40

Limestone

Production continued from Zoutkloof quarry on levels 30 m to 0 m in Figure 5.2. There was also an introduction of limestone from Vondeling quarry. The quality from Vondeling was relatively poor compared to Zoutkloof limestone. This required beneficiation in terms of screening at 35%. It is also important to note that limestone

from both quarries were blended during the crushing and screening process. This helped to improve quality blending.

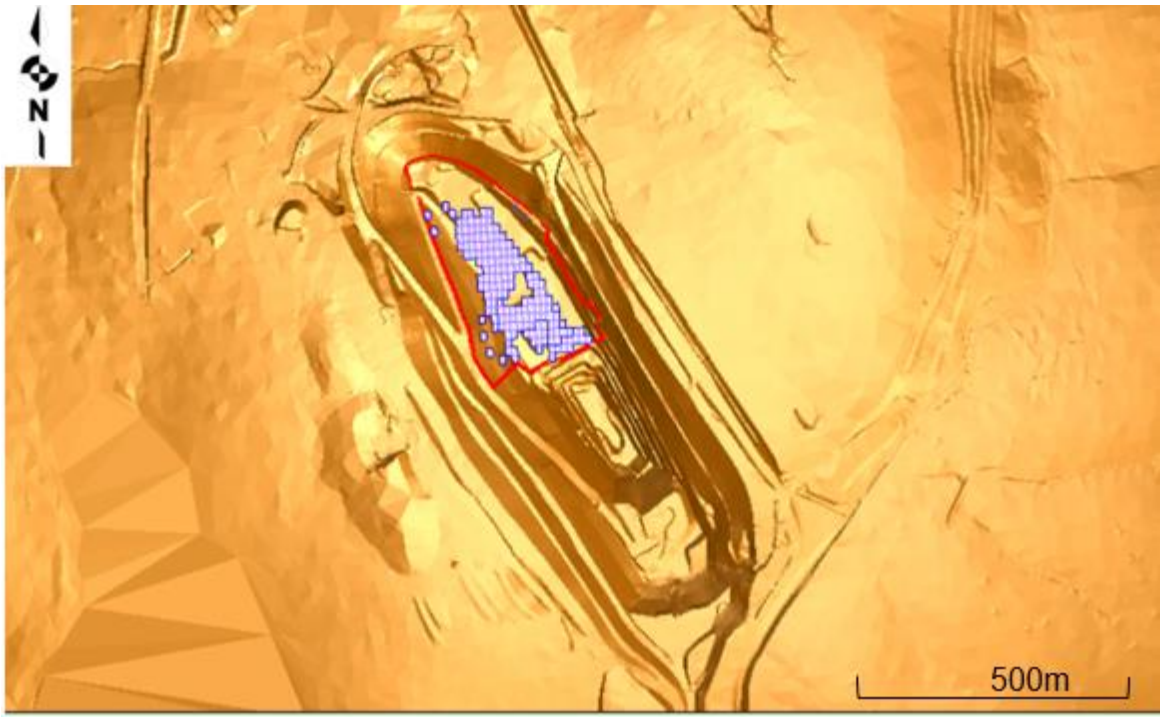


Figure 5.2: Mining block for 2009-2010

5.4 Year 3 Scheduling (2011-2012)

The proposed mining blocks for 2011 to 2012 financial year are shown in Figure 5.3. The budgeted tons for 2011 to 2012 financial year are reflected in Table 5.3. The overburden came from both quarries although majority comes from Vondeling mine.

Table 5.3: Tonnage and quality requirements at Zoutkloof 2011-2012

Material Type	Quarry	Tonnage	LSF	MgO	NaK
Limestone	Zoutkloof	821,304	>155	<0.8	<0.48
	Vondeling	185,413			
Overburden/waste		2,516,794	N/A	N/A	N/A

Limestone

Zoutkloof limestone was to continue to be mined from level 0 m, -15 m, and -30 m levels. It was essential that level -45 m limestone mining commences during 2011 to 2012 financial year. This was to ensure that this material should be available for blending with level -30 m in case internal waste be encountered in the upper benches. Table 5.4 summarises the limestone blocks and the qualities expected per block.

Table 5.4: Limestone tonnage and quality according to mining blocks

Zoutkloof limestone 2011/2012										
Block No	**Top**	**Bottom**	**Height**	**Tons**	**RCO_3 Grade**	**W.a RCO_3 grade**	**% Tons**	**NaK**	**W.a NaK grade**	**Block Model Volume**
1	-28	-45	17	35,100	77.62	3.15	4.1	0.58	0.02	13,500
2	-12	-27	15	198,900	77.05	17.70	23.0	0.63	0.14	76,500
3a	7	-9	16	468,000	81.12	43.85	54.1	0.62	0.34	180,000
3b	5	-9	14	23,400	81.25	2.20	2.7	0.65	0.02	9,000
4	7	-9	16	140,400	80.63	13.08	16.2	0.62	0.10	54,000
				865,800		**79.97**	**100.0**			**333,000**

It was to be noted that the tons and qualities provided in Table 5.4 reflects the in-situ qualities and do not include screenings. Certain portions within Block 3 contain schist material and this consequently dilutes the quality of the limestone mined. It was advised that grade control be done when mining this block to ensure that the

ROM did not contain excessive schist material. On the same token, it was recommended that the contaminated blocks were to be mined as Saldanha product as and when required.

Overburden and Development

A minimal amount of overburden was left at Zoutkloof. Zoutkloof Overburden was to be sourced from block 5 which comprises 44,881 tons (Table 5.5). It was to be noted that there were two more areas shown in green (Figure 5.3) that are on waste. However, those areas were used as temporary ramps/access to different benches. The tonnage from those areas were not included in the plan but was to be moved as faces advance.

Table 5.5: Overburden block tonnages

Zoutkloof Overburden						
Block No	Area	Top	Bottom	Height	R.D	Tons
5	1233	5	-9	14	2.6	44,881

The overburden been removed from Zoutkloof was to be used to construct the ramp to -45 m Level. Although the aforesaid overburden was not sufficient to build the access ramp to Level -45 m, there were some materials that was to be cut when a 1:10 ramp was constructed as shown in Figure 5.3.

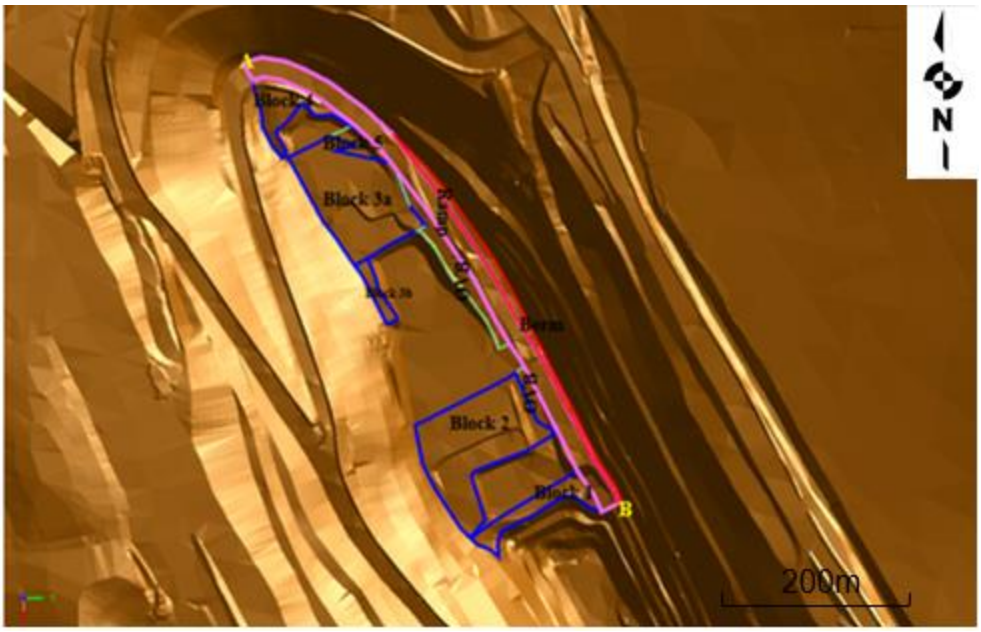


Figure 5.3: 2011-2012 scheduling

The distance calculated from point A of the ramp to Point B of the ramp was 698 m. The elevation at point A was 24 m whereas point B was at -45 m elevation. Excess material required to construct a ramp was also sourced from Vondeling quarry.

5.5 Year 4 Scheduling (2013-2014)

At this stage, Zoutkloof quarry LOM was coming to an end. The initial LOM indicated that mining would cease between 2014 and 2015. Hence another quarry was being opened next to Zoutkloof. Therefore, after year 4 scheduling, there was ramp up of production from Vondeling and ramp down from Zoutkloof quarry.

Table 5.6 below is a summary of planned tonnage and quality requirements for the 2013 financial year. The average screening percentage for both Zoutkloof and Vondeling quarry was at 27%. The stripping ratios for Zoutkloof and Vondeling respectively are 1:0.2 and 1:15.9. This means that almost all the overburden was stripped from Vondeling quarry as a strategy to expose limestone for the required consumption and going forward.

Table 5.6: 2013-2014 tonnage budget

Source	Limestone	Overburden	RCO_3	NaK
Zoutkloof	720 000	0	>75%	0,52
Vondeling	350 000	3 800 000		

Proposed mining zones

During the 2013 financial year Zoutkloof production was to be sourced from bench -15 m, -30 m, -45 m, -60 m (according to the original nomenclature). Level -60 m had not commenced yet but was to be developed during the financial year.

Face heights and Mining Hazards

It was noted that the benches at Zoutkloof were exceeding the design height of 15 m. The flyover (using Surpac software) depicts that level -30 m has an average height of 20 m, level -45 m bench having an average height of 19 m. It was important that level -60 m be developed to the correct design specifications of 15 m face height. It was then necessary to evaluate those benches for potential hazards associated with the prevailing mining parameters and endorsed by the geotechnical engineer. Change in design warrant risk assessment review and pit design review. The COP clearly states that “a pit design review and baseline risk assessment review will be conducted on an annual basis in consultation with a reputable rock engineering service” (PPC De Hoek Mine, 2020). Hence it was important for operational team to maintain mining parameters according to design.

Mining Controls - (Hanging wall and Foot wall)

It was imperative to ensure that the slope angle is maintained as it was instructed in the COP. The footwall slope was designed to be 54° to ensure stability. The mining minimum width target was 50 m, this was to ensure machines operate safely. The north section of the quarry was too narrow to continue mining. Hence, geotechnically, the reserves were left behind unmined. During this period, it was

crucial to comply with the planned mining block sizes such that mining width is not compromised further, or the slopes are not undercut.

Ramp

The ramp on the hanging wall side requires approximately 840 m to reach the bottom of the bench at -60 m elevation. The design width was at 25 m according to the geotechnical design. To establish the ramp, material was needed to fill the portion of the ramp to attain the 840 m distance, Figure 5.4.

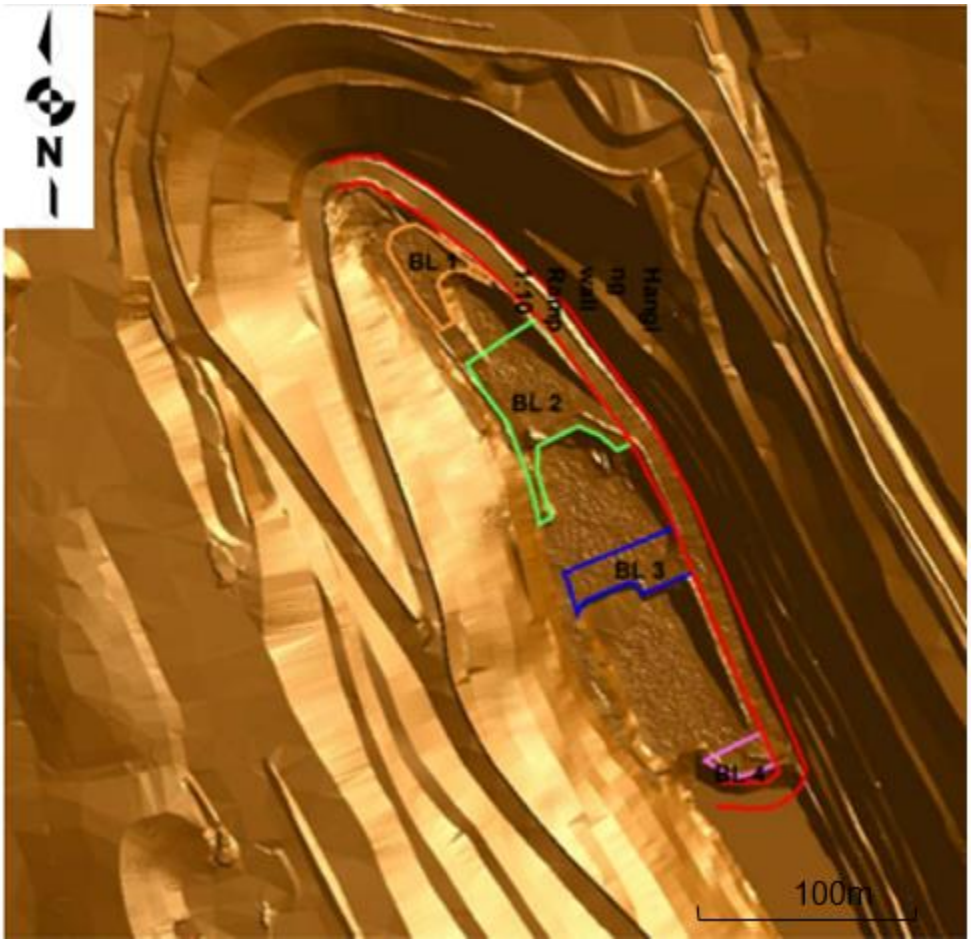


Figure 5.4: Limestone mining blocks for 2013-2014

Limestone Mining

Zoutkloof limestone will be sourced mainly from the three benches between -15 m, -30 m, and -45 m elevations. The development of new bench at -60 m elevation

will also be started during the current financial year. Figure 5.4 shows the proposed mining blocks at Zoutkloof quarry. It was important to note that the phase plan limits the limestone mining not to exceed 800 000t prior screening or 590 000t post screening.

Table 5.7 shows a summary of tonnage and qualities for the proposed Zoutkloof limestone blocks. It was noted that the tonnage was based on the 15 m bench heights rather than the actual heights as it was discussed above. It is important to note that the block model tonnage represented above was for all defined blocks and not the overall tonnage within the designated boundaries. 66% of the materials from the blocks at Zoutkloof were over the RCO_3 of 75, precisely RCO_3 of 80.73 and Alkali of 0.60.

Table 5.7: 2013 Summary of tonnages

Zoutkloof Mining (>75 RCO_3)								
Source	**From**	**To**	**Tonnage**	**RCO_3**	**NaK**	**W. a RCO_3**	**W. a NaK**	**BM tons >75 RCO_3**
Level-15m	7	-8	134 706	81.36	0.61	15.15	0.11	58 500
Level -30m	-8	-28	376 506	80.31	0.60	41.79	0.31	222 300
Level -45m	-28	-47	157 950	80.43	0.58	17.56	0.13	81 900
Level -60m	-47	-60	54 405	78.15	0.64	5.88	0.05	23 400
			723 567			80.73	0.60	386 100

Table 5.8 shows a summary of low-quality limestone. Level -45m consist of both low- and high-quality limestone. Total tonnages of this level are 157 950t from which about 2% is low quality. Although RCO_3 is low, the level shows good NaK values. This block was mined with other high-quality blocks to improve overall quality.

Table 5.8: Low quality limestone blocks

Zoutkloof Mining (>65 RCO_3 <75)								
Source	**From**	**To**	**Tonnage**	**RCO_3**	**NaK**	**W. a RCO_3**	**W. a NaK**	**BM tons >65 RCO_3 <75**
Level-15m	7	-8	134 706					
Level -30m	-8	-28	376 506					
Level -45m	-28	-47	157 950	74.62	0.49			11 700
Level -60m	-47	-60	54 405					
			723 567					11 700

Mining blocks that have RCO_3 values of less than 65 was regarded at internal waste (Table 5.9). This accounted for about 32% of the materials within all four levels. Having a high cut-off grade was mainly to accommodate low quality limestone from Vondeling mine.

Table 5.9: Internal waste tonnages

Zoutkloof Mining (<65 RCO_3) – Internal waste				
Source	**From**	**To**	**Tonnage**	**BM tons >65 RCO_3 <75**
Level-15m	7	-8	134 706	23 400
Level -30m	-8	-28	376 506	93 600
Level -45m	-28	-47	157 950	70 200
Level -60m	-47	-60	54 405	0
			723 567	18 200

5.6 Year 5 Scheduling (2015-2016)

Zoutkloof is to produce 34% of the 2015 limestone and 52% will be mined from Vondeling; with the additional 14% at LSF of 280-310 sourced from Riebeeck West quarry. The initial budget of 449 418 tons from Vondeling and 524 363 tons (including NDM) from Zoutkloof (Table 5.10) had been revised to increase the amount of limestone to be mined from Vondeling. NDM (non-deleterious material) is high quality limestone that is used as an extender in cement manufacturing. This plan had an additional 10% of limestone that had been used as a production buffer for unplanned demand.

Table 5.10: 2015-2016 scheduling

Source	Tonnage	NaK	NaK (After screening)	RCO_3	RCO_3 (After screening)
BK1	105 300	0.73	0.58	59.19	68.25
BK2	342 400	0.70	0.56	55.64	64.15

The 2015 financial year Zoutkloof production was mainly sourced from the level -60 m bench. A ramp remaining on level -45 m was blasted and loaded off for -60 m to reach its maximum production. Another 10 m bench was to be developed from level -60 m to level -70 m. This quarry should contribute about 388 000t to the total production in 2015 financial year. It was imperative to ensure that the slope distance and angle was maintained as instructed as per design since Zoutkloof quarry had narrowed.

Limestone

Zoutkloof limestone was mined from levels -60 m and when that face has been fully exploited then a new bench was to be developed to level -70 m. Figure 5.5 illustrates the planed sources of limestone from the Zoutkloof pit. It is to be noted that the tons and qualities provided in Table 5.10 reflects the in-situ qualities as per

exploration data, but this was to be improved after screening. The 2015-2016 screenings averages were to be about 24% and had to be brought down to 20.3% by end of 2015 financial year. The screening was to improve the quality of the Zoutkloof limestone, and it was expected that the NaK would have a 20% improvement while the RCO_3 would have a 17% improvement. NDM tonnage had been included in the limestone budget and was to be sourced from the limestone produced in Zoutkloof.

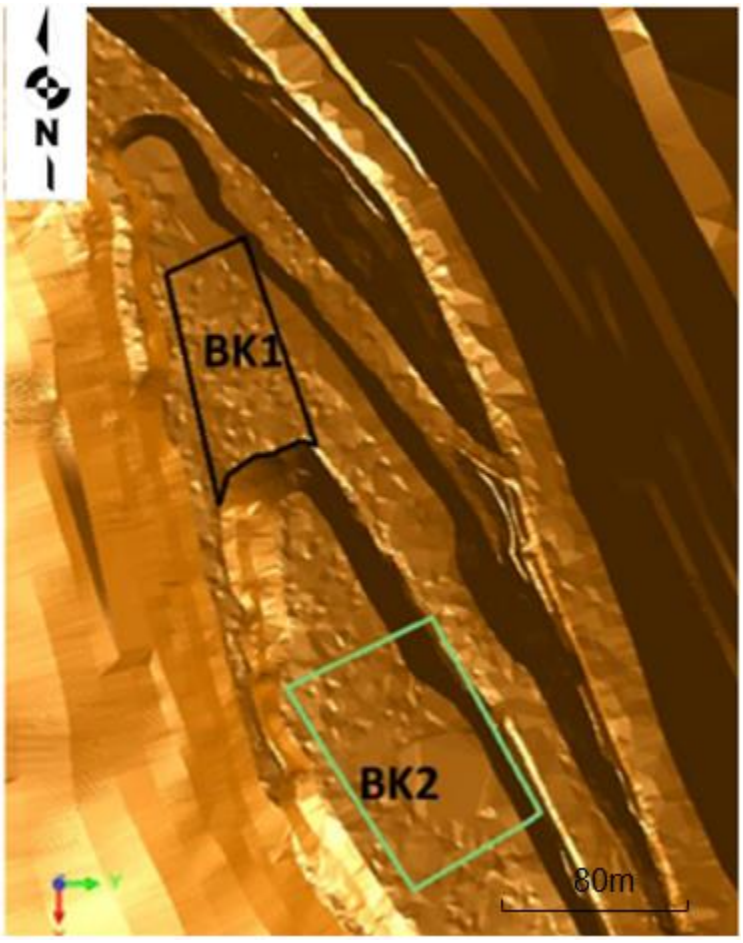


Figure 5.5: 2015-2016 Mining blocks

Overburden and Development

Zoutkloof mining was undertaken within the exposed limestone orebody for the 2016 financial year. Some phyllite on the ramps was to be mined as waste as indicated in Figure 5.6. Waste will be sourced from levels -30 m to -45 m and -45 m to -60 m. The total tonnage is about 269 100t.

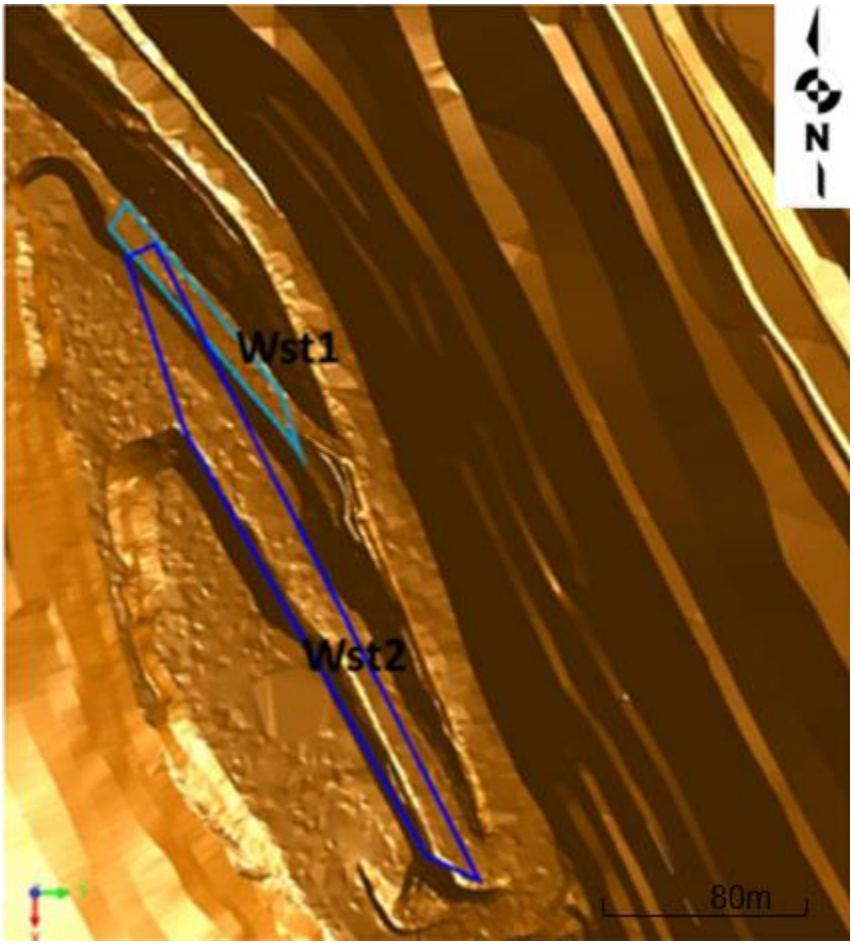


Figure 5.6: 2015-2016 waste blocks

5.7 Year 6 Scheduling (2017-2018)

Limestone was mainly to be sourced from Vondeling quarry during the 2017-2018 financial year. Zoutkloof limestone was to be mined from level -70 m. Figure 5.7 and Table 5.11 shows the planned block for limestone from the Zoutkloof pit.

It was noted that the 2017-2018, screenings averages were about 28%, where 35% was in Vondeling and 18% in Zoutkloof. The screening was to improve the quality of the limestone where NaK should have a 20% improvement while the RCO_3 had an expected 17% improvement. NDM tonnage had been included in the limestone budget and should be sourced from the limestone produced in Vondeling.

Table 5.11: 2017-2018 limestone scheduling

Source	**Tonnage**	**NaK**	**NaK (After Screening)**	**RCO_3**	**RCO_3 (After Screening)**
BK1	490 100	0.66	0.53	59.12	68.17

Zoutkloof was to be mined out and therefore there was no sourcing from it after October 2017.

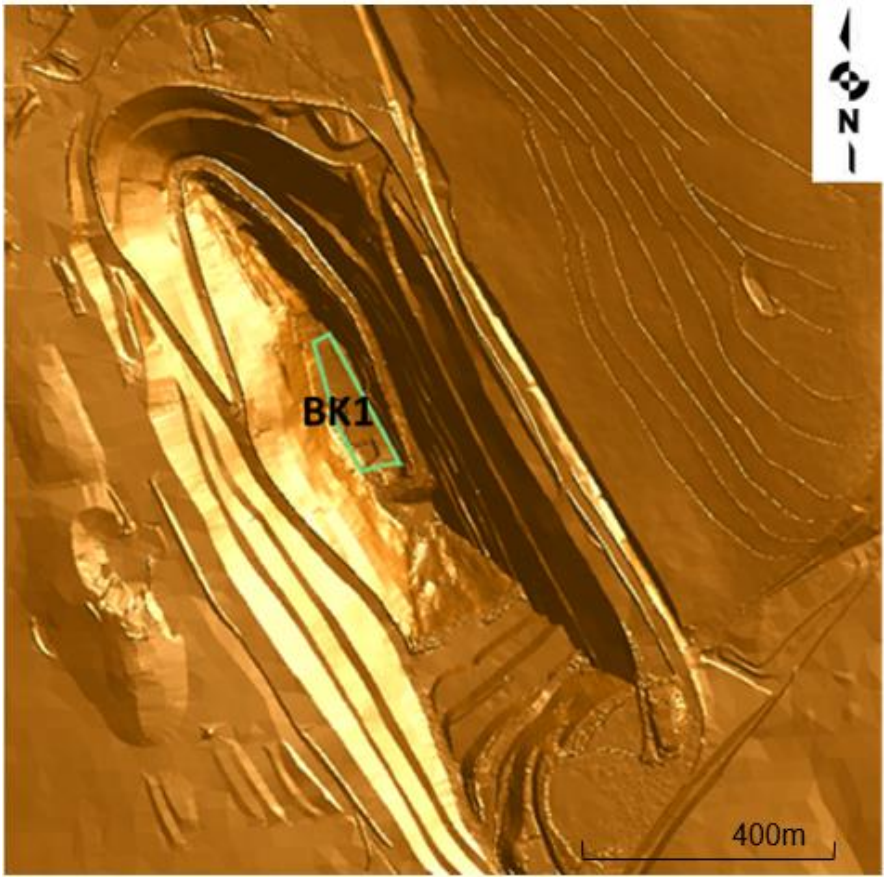


Figure 5.7: 2017-2018 mining block

5.8 Costs Analysis

Zoutkloof quarry LOM was 37 years in total. During this period, the operation produced an average of 800 000t to 900 000t of cement annually. The company remained the main source of cement in the Western Cape province. In financial results for the period ending on 31st March 2017, PPC coastal region experienced double digit volume growth (PPC, 2017). With all the geotechnical strategies and mine planning, the quarry had 10 minor slope failures with no damage to equipment or injuries to the workforce. Therefore, the impact on costs from having complex geology and slope failures were minimum.

5.9 Summary

This section showed the last 10 years of mining at Zoutkloof open pit. In this period there was minimum waste to be mined, as the quarry had narrowed down significantly. The need to maintain slope walls, especially the footwall, according to design was crucial to extract as much as possible limestone. According to design, the bench heights were supposed to be 15 m, but some were exceeding this and was about 20 m. emphasis was made during production years that mining outside the design parameters had to be stopped on the following lower benches.

The close working relationship between the geotechnical engineer, operational team and planning department was another factor that Zoutkloof quarry was mined out successfully without major slope failures. Some limestone reserves in the Northern side of the open pit had to be left behind due to narrow mining faces. Even so, the operation managed to continue making cement successfully until 2017 when mining ceased.

6. Conclusion

The focus of the study was to evaluate the geotechnical parameters of Zoutkloof quarry and how the lessons learned can be used when evaluating Vondeling geotechnical parameters. In open pit mining, keeping slopes stable is critical to successful mining of the ore at economical rate while keeping machines and mining personnel safe. The geological structure of Zoutkloof quarry was highlighted as complex in nature and highly jointed. The orebody dips east at between 50° to 80° and first slope failure was recorded around the 1980s.

Rock mass discontinuities, groundwater presence, complex rock geology and blasting (Read and Stacey,2009) were identified as some of the factors that affect slope stability. This necessitates the significance of understanding the mechanisms that cause slope failure. Failures such as planar, wedge, circular and toppling are amongst the common slope failures in open pit mines. Hence mitigation measures must be developed on how to reduce the chances of such from occurring. Concepts such as classification of rock mass under rock mass model are identified as important.

The key geotechnical parameters discussed were groundwater conditions, geological structure and rock mass strength, joints, slope angles, pit wall design and slope monitoring. For better water control, the quarry was divided into two sections. The southern section was mined out completely and the lowest areas were used as a dam to draw groundwater from northern section. That decision allowed the northern section benches to get dry during production years. The drainage holes drilled into the slope assisted to lower the remaining water level in the slopes to keep them stable.

It was also a must for all final slopes to be scaled after pre-split blasting. By using empirical methods, the slope angle to be used in the design was determined to be 40˚, although in chapter 5, it was highlighted that the overall slope angle was 54˚. Joints analysis and field observations showed that there are minor faults dipping toward the east. Two major joints sets were identified for both footwall and hanging

wall slope. The effect of the geotechnical parameters on the stability of the slopes in Zoutkloof was that the probability of having toppling failure was high compared to wedge and planar failure.

As part of slope monitoring, TARPs were developed along with visual monthly inspections. After mining ceased, annual inspections were completed with the appointed rock engineer. For this study, a period of two years was combined and reported as year 1 and the same was applied for other years. The mining benches in 2013-2014 FY were exceeding the designed height and this required a safety analysis to be done to evaluate safety in those benches. All the minor slope failures that occurred in Zoutkloof quarry did not have negative impact on the costs. The operation performed well financially in 2017. The mining method and all other decisions that were made regarding Zoutkloof quarry worked well as there was no slope failure related injuries or fatalities.

7. Recommendations

It is the recommendation of this study that the lessons from mining Zoutkloof quarry should be adopted as the company continues to operate and mine Vondeling quarry. Such lessons include blending, good blasting practices, understanding geological formations and key geotechnical parameters. Bench method should be adopted on the western side which is the footwall of Vondeling mine. This is due to the location and proximity of the diverted river which is 20 m from the final slope wall. The benches can serve as support and increase FOS. It is important for the water stream to remain dry or drain water completely, especially during the raining season such that slope walls are not weakened. Additionally, drainage holes should be drilled into the footwall slope to drain any water that might be in the rock mass to prevent potential failures.

The geotechnical report for Vondeling mine should be reviewed to map all the faults lines that exist in the host rock and within the ore, particularly in the western side. Hence it is the recommendation of this study to geotechnically map the Vondeling quarry. Limestone quality in Vondeling quarry is not consistent while compared to Zoutkloof quarry. One of the lessons from Zoutkloof mining is that blending is crucial to reduce the overall percentage of screenings. This will also ensure that the targeted LOM according to design is achieved. Hence, overburden stripping and doing push backs on time is important. Having multiple production benches will allow for blending to take place.

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9. Appendix A

Rock Classification

An important element in determining maximum slope angles is to define geotechnical areas and determine the rock classification for each area. Using the data available, the area was geotechnically classified using the Q system developed by Barton *et al.* (1974). The *Q* System classification is based on three aspects:

- rock block size (RQD/J_n)
- joint shear strength (J_r /J_a)
- confining 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.

These parameters and their corresponding values are described in the following sections, together with the values deemed suitable for Zoutkloof Pit.

RQD, developed by Deering (1989), is defined as the ratio of the cumulative length of sticks of core more than 100 mm in length in a drill run to the total length of the drill run:

$$RQD\% = \frac{Total\ \ length\ of\ core\ \geq 100mm\ x\ 100}{Length\ of\ core\ run}$$

Note, blasting fractures and minor joints (defined by any joint with low persistence and exhibiting no staining) were discounted in this exercise.

In the absence of core, sections of wall can be used with the following modification to the basic RQD formula. Where Jv equates to the number of joints per cubic meter.

$RQD = 115 — 3.3\ J_v$

It should be noted that the figures quoted are by the nature of this analysis, generalised to give a reasonable account of the entire quarry. Some areas have higher RQD values and others lower values. In the interests of conservatism, the lower figure for each section will be used in the subsequent slope design exercise.

Table A1: Estimated RQD values

Ground Control District	**Main lithology**	RQD (estimate)
GCD1a	Weathered phyllite	29
GCD1b	Fresh phyllite	16
GDC1c	Limestone ore body	29
GCD2	Mixture of the units above, but with more structural disturbances and presence of graphite bands	62
GCD3	Limestone	16
GCD4a	Weathered phyllite	45
GDC4b	Fresh phyllite with limestone near the base	49

J_n - Joint set number

Table A2: Joint set number

Number of Joint Sets	**Joint Set No. J_n**
Intact, no or few joints	0.5 — 1.0
One joint set	2
One joint set plus random joints	3
Two joint sets	4
Two joint sets plus random joints	6
Three joint sets	9
Three joint sets plus random joints	12
Four or more joint sets, random, heavily jointed, sugar cube, etc.	15
Crushed rock, earth like	20

Table A3: Summary of joint number for each GCD

Ground Control District	**Main lithology**	**Jn (estimate)**
GCD1a	Weathered phyllite	9
GCD1b	Fresh phyllite	9
GDC1c	Limestone ore body	9
GCD2	Mixture of the units above, but with more structural disturbances and presence of graphite bands	12
GCD3	Limestone	9
GCD4a	Weathered phyllite	9
GDC4b	Fresh phyllite with limestone near the base	9

J_r - Joint Roughness Number

Distinction is made between the large-scale nature of planes and the small-scale roughness as well as between continuous and discontinuous joints. Table 4 gives joint roughness number (J_r) values.

Table A4: Joint Roughness Number

Description of Joint Surface Roughness	**Discontinuous**	**Undulating**	**Planar**
Rough	4.0	**3.0**	1.5
Smooth	3.0	2.0	1.0
Slicken sided	2.0	1.5	0.5
Planes containing gouge thick enough to prevent rock wall contact	1.5	1.0	1.0

The joints were generally observed to be rough on a micro scale and undulating or even discontinuous on a macro scale. The phyllite zones showed smooth joints, almost to the point of being slicken sided. Staining was noted on some of the surfaces, but gouge was only present in a minor form on a few of the joints checked.

Table A5: Summary of joint roughness for each GCD

Ground Control District	**Main lithology**	Jr (estimate)
GCD1a	Weathered phyllite	1.5
GCD1b	Fresh phyllite	2
GDC1c	Limestone ore body	3
GCD2	Mixture of the units above, but with more structural disturbances and presence of graphite bands	3
GCD3	Limestone	3
GCD4a	Weathered phyllite	1.5
GDC4b	Fresh phyllite with limestone near the base	3

J_a - Joint Alteration Number

The joint alteration number considers the weathering and coating on joint surfaces as well as the thickness and nature of any gouge infill present in the joints. The parameter determines the shear strength of the rock mass as well as its deformability and potential to squeeze or swell.

Table A6: Joint Alteration Number

Description of Gouge	**Joint Alteration Number J_a for Joint Separation (mm)**		
	<1.0	1.0-5.0	>5.0
Tightly healed, hard, non-softening impermeable rock mineral filling	0.75	-	-
Unaltered joint walls, surface staining only	1.0	-	-
Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling	2.0	4.0	6.0
Non-softening, slightly clayey non-cohesive filling	3.0	6.0	10.0
Non-softening strongly over-consolidated clay mineral filling, with or without crushed rock	3.0	6.0	10.0
Softening or low friction clay mineral coatings and small quantities of swelling clays	4.0	8.0	13.0
Softening moderately over-consolidated clay mineral filling, with or without crushed rock	4.0	8.0	13.0
Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock	5.0	10.0	18.0

Table A7: Summary of joint alteration for each GCD

Ground Control District	**Main lithology**	Ja (estimate)
GCD1a	Weathered phyllite	3
GCD1b	Fresh phyllite	2
GDC1c	Limestone ore body	1
GCD2	Mixture of the units above, but with more structural disturbances and presence of graphite bands	4
GCD3	Limestone	1
GCD4a	Weathered phyllite	3
GDC4b	Fresh phyllite with limestone near the base	2

J_w - Joint Water Reduction Factor

The joint water reduction factor allows for the water pressure on the joint walls, as well as the potential for the outwash and softening of joint gouge.

Table A9: Joint Water Reduction Factor

Condition of Groundwater	**Head of water (m)**	**Joint Water Reduction Factor J_w**
Dry excavation or minor inflow 5 litre/minute locally	<10	1.0
Medium inflow, occasional outwash of joint/fissure fillings	10 – 25	0.66
Large inflow in competent ground with unfilled joints/fissures	25-100	0.5
Large inflow with considerable outwash of joint/fissure fillings	25-100	0.33
Exceptionally high inflow upon excavation, decaying with time	>100	0.2-0.1
Exceptionally high inflow continuing without noticeable decay	>100	0.1-0.05

As highlighted throughout this report, water is a weakening factor and if present in large quantities, even for a short while could result in slope instability. With the river diversion and drainage holes in place, the slope can be regarded as essentially dry. Therefore, a J_w **= 1.0** was adopted in this section of the analysis.

***SRF* - Stress Reduction Factor**

Table A10: Stress Reduction Factor for weakness zones

Description	***SRF* Value**
Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
Single weakness zones containing clay or chemically disintegrated rock (depth of excavation < 50m)	5
Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	2.5
Single shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5
Single shear zones in competent rock (clay-free) (depth of excavation < 50m)	5.0
Single shear zones in competent rock (clay-free) (depth of excavation > 50m)	2.5
Loose open joints, heavily jointed or “sugar-cube” etc (any depth)	5.0

For this investigation, a figure of ***SRF*** = 2.5 will be suitable except in the disturbed south section where an SRF of 5.0 was deemed more appropriate.

Table A11: Summary of parameters used for compiling RMR

Ground Control District	RQD	Jn	Jr	Ja	Water	SRF
GCD1a	29	9	1.5	3	1	2.5
GCD1b	16	9	2	2	1	2.5
GDC1c	29	9	3	1	1	2.5
GCD2	62	12	3	4	1	5.0
GCD3	16	9	3	1	1	2.5
GCD4a	45	9	1.5	3	1	2.5
GDC4b	49	9	3	2	1	2.5

Calculation of *Q*

All selected values for the above six parameters, based on observed or estimated conditions are substituted into the equation to obtain the value of the rock quality index *Q*.

$$Q = \frac{RQD}{J_n} x \frac{J_r}{J_a} x \frac{J_w}{J_a} x \frac{J_w}{SRF}$$

The *Q* system does not take the rock material strength into account explicitly, although it is implicitly included in arriving at the *SRF* assessment. The orientation

of joints is also not considered except were commented upon in the text since it is considered that the number of joint sets, and hence the potential freedom of movement for rock blocks is more important.

The range in values of *Q* is from 0.001 for extremely poor rock to 1000 for excellent rock. For Zoutkloof Quarry:

Table A12: Values for Q for all GCDs

Ground Control District	**Main lithology**	Q Index	Barton description
GCD1a	Weathered phyllite	0.6	poor
GCD1b	Fresh phyllite	0.7	poor
GDC1c	Limestone ore body	3.9	poor
GCD2	Mixture of the units above, but with more structural disturbances and presence of graphite bands	0.8	Poor
GCD3	Limestone	2.1	Poor
GCD4a	Weathered phyllite	1.0	Poor
GDC4b	Fresh phyllite with limestone near the base	3.3	Poor

According to Barton, the values achieved describe poor conditions. This is mainly due to the quantity of joints which result in a small block size. This highlights the reasoning why good drainage and pre-split, both which retain good shear strength properties are vital. Also, since Q is a logarithmic scale, small errors can result in

major moves up and down the stability curve. Therefore, it was decided to use the Q value in the more rigorous Rock Mass Rating system (RMR).

Rock Mass Rating

The Geomechanics Classification System derives an RMR, obtained by summing 5 parameter values and adjusting this total by considering the joint orientations. The parameters included in the system are:

- rock material strength (*UCS*)
- *RQD*
- joint spacing
- joint roughness and separation
- groundwater

The *RMR* value can range between zero and 100, and with its 5-part parameter scale, this system is conceptually easier to apply than the *Q* System. A relationship has been found between *RMR* and *Q* as follows (Bieniawski, 1989):

$$RMR = 9 \ln Q + 44$$

Table A13: RMR for all GCDs taken using the Q Index results

GCD	**Main lithology**	RMR	Bieniawski description
GCD1a	Weathered phyllite	40	Fair
GCD1b	Fresh phyllite	41	Fair
GDC1c	Limestone ore body	56	Fair
GCD2	Mixture of the units above, but with more structural disturbances and presence of graphite bands	42	Fair
GCD3	Limestone	51	Fair
GCD4a	Weathered phyllite	44	Fair
GDC4b	Fresh phyllite with limestone near the base	55	Fair

10. Appendix B

Stereographic projection data

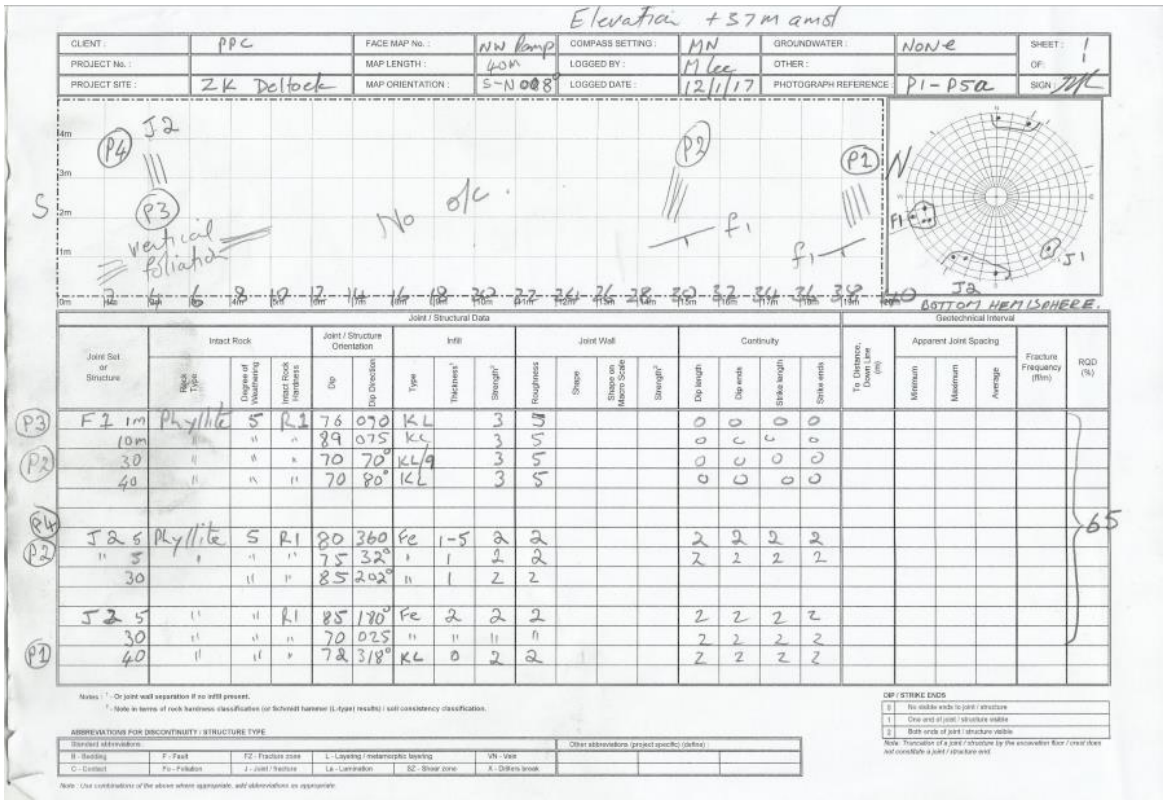


Figure B1: Zoutkloof footwall at +37 m

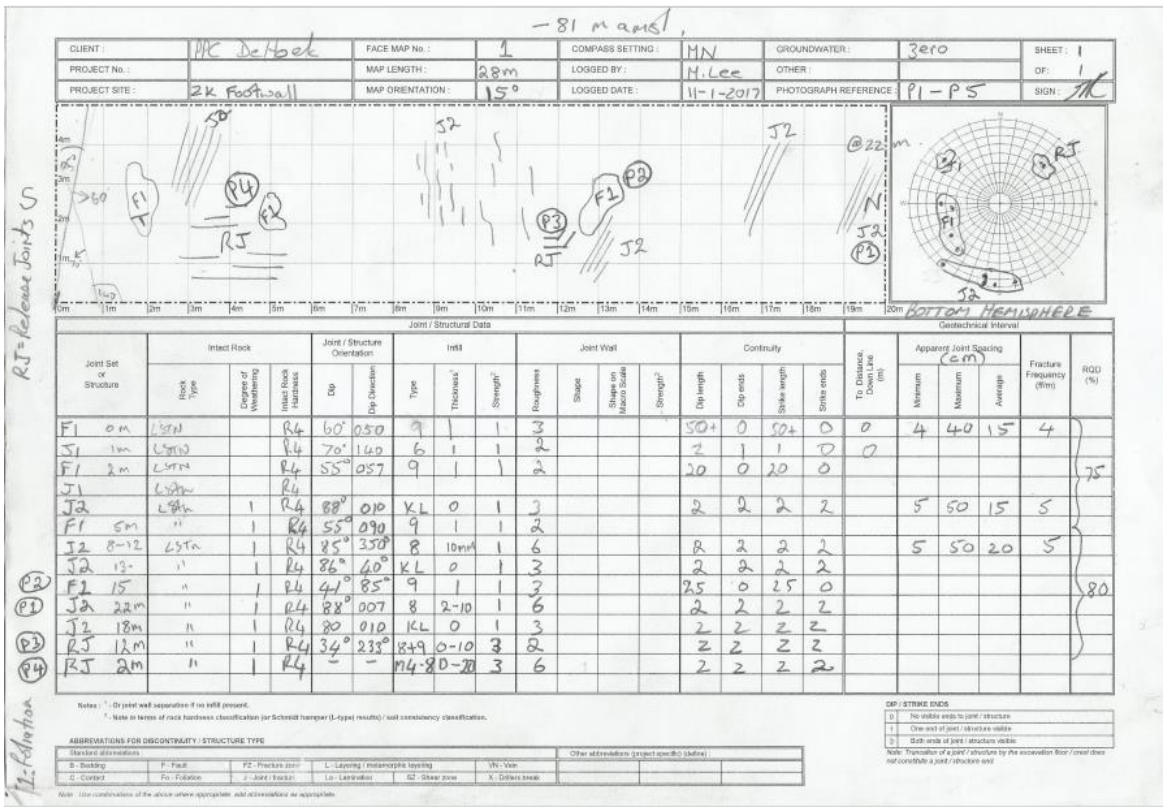


Figure B2: Zoutkloof footwall at -81 m

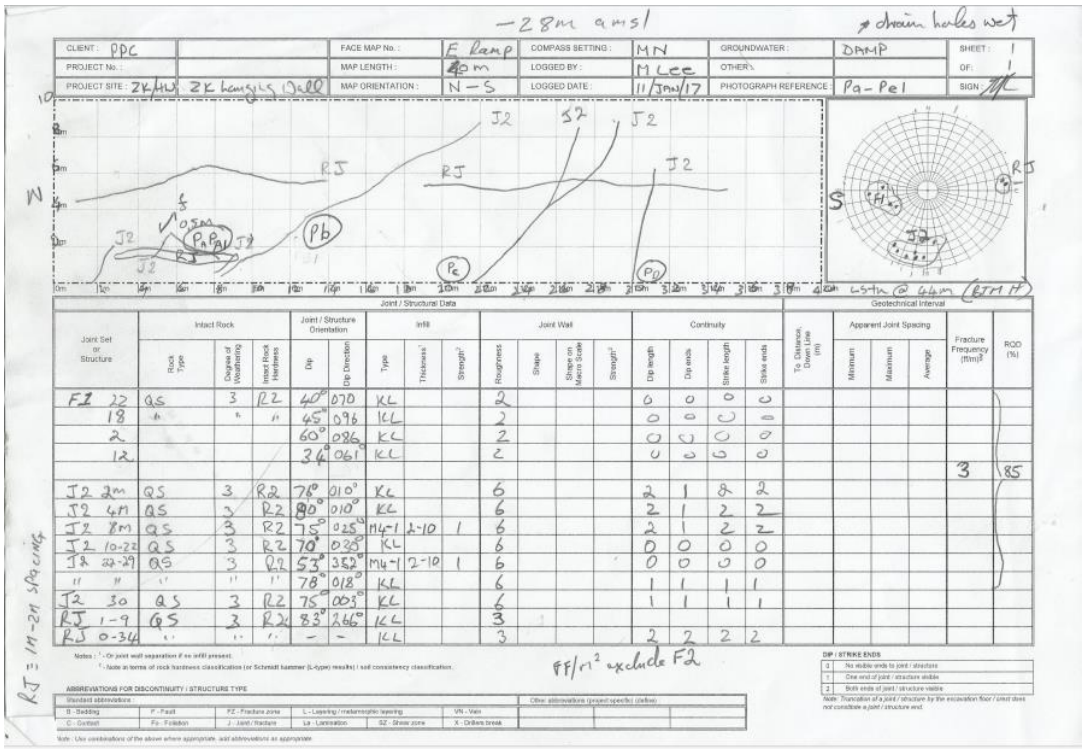
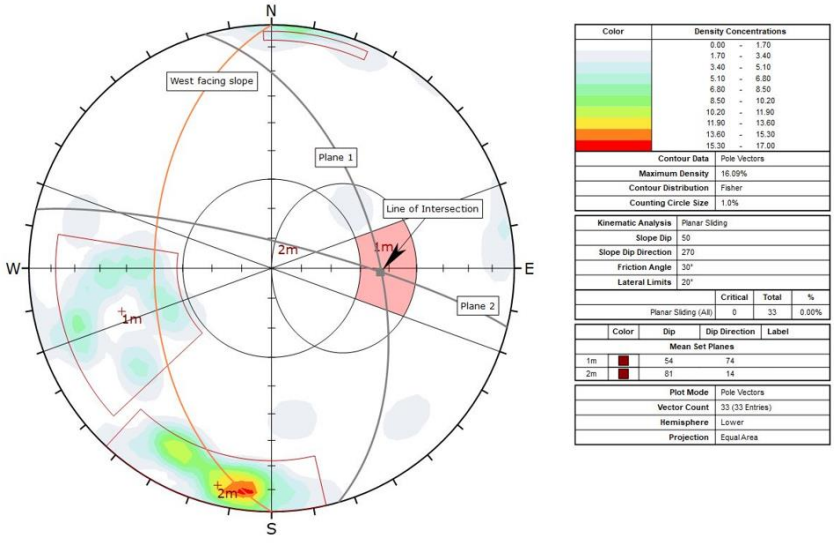


Figure B3: Zoutkloof hanging wall at -28 m

Planar failure



Color	Density Concentrations
	0.00 - 1.70
	1.70 - 3.40
	3.40 - 5.10
	5.10 - 6.80
	6.80 - 8.50
	8.50 - 10.20
	10.20 - 11.90
	11.90 - 13.60
	13.60 - 15.30
	15.30 - 17.00

Contour Data	Pole Vectors
Maximum Density	16.09%
Contour Distribution	Fisher
Counting Circle Size	1.0%

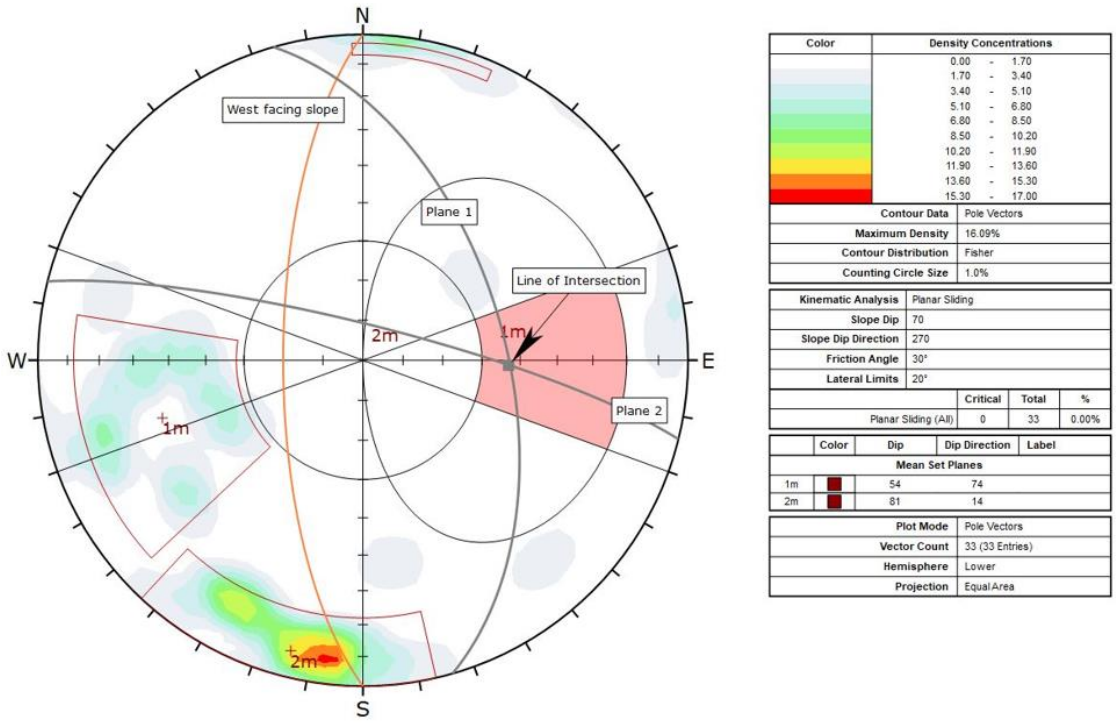
Kinematic Analysis	Planar Sliding
Slope Dip	50
Slope Dip Direction	270
Friction Angle	30°
Lateral Limits	20°

	Critical	Total	%
Planar Sliding (All)	0	33	0.00%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1m		54	74	
2m		81	14	

Plot Mode	Pole Vectors
Vector Count	33 (33 Entries)
Hemisphere	Lower
Projection	Equal Area

Figure B10.1: 50° slope angle planar failure



Color	Density Concentrations
	0.00 - 1.70
	1.70 - 3.40
	3.40 - 5.10
	5.10 - 6.80
	6.80 - 8.50
	8.50 - 10.20
	10.20 - 11.90
	11.90 - 13.60
	13.60 - 15.30
	15.30 - 17.00

Contour Data	Pole Vectors
Maximum Density	16.09%
Contour Distribution	Fisher
Counting Circle Size	1.0%

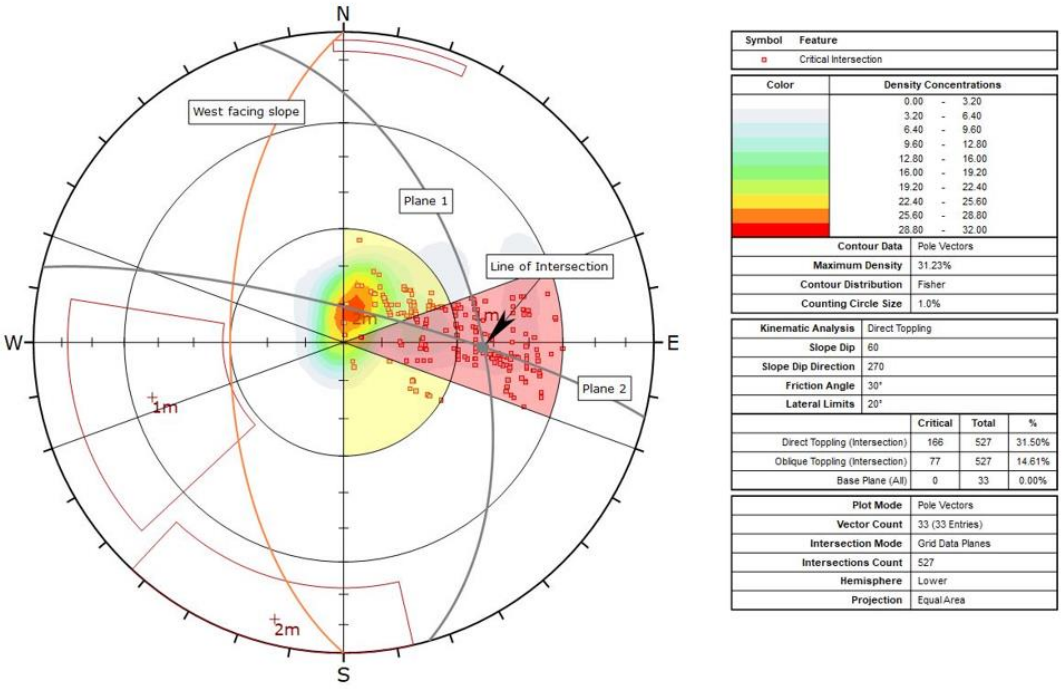
Kinematic Analysis	Planar Sliding		
Slope Dip	70		
Slope Dip Direction	270		
Friction Angle	30°		
Lateral Limits	20°		
	Critical	Total	%
Planar Sliding (All)	0	33	0.00%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1m		54	74	
2m		81	14	

Plot Mode	Pole Vectors
Vector Count	33 (33 Entries)
Hemisphere	Lower
Projection	Equal Area

Figure B10.2: 70° slope angle planar failure

Toppling failure



Symbol	Feature
▫	Critical Intersection

Color	Density Concentrations
	0.00 - 3.20
	3.20 - 6.40
	6.40 - 9.60
	9.60 - 12.80
	12.80 - 16.00
	16.00 - 19.20
	19.20 - 22.40
	22.40 - 25.60
	25.60 - 28.80
	28.80 - 32.00

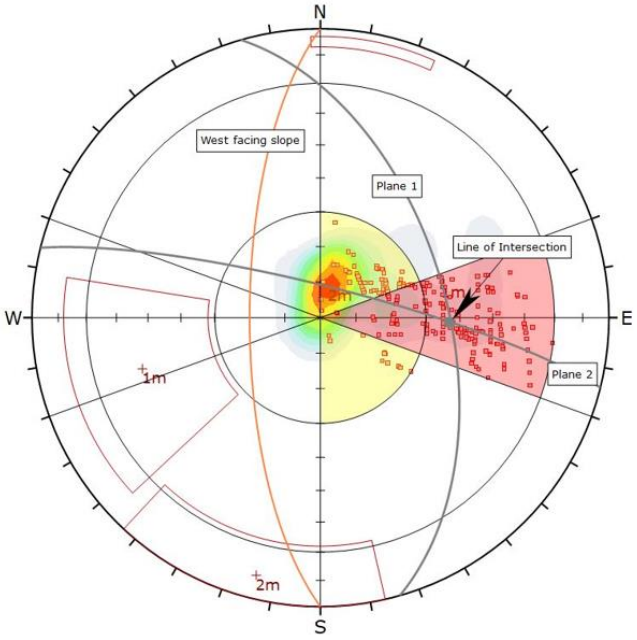
Contour Data	Pole Vectors
Maximum Density	31.23%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Direct Toppling
Slope Dip	60
Slope Dip Direction	270
Friction Angle	30°
Lateral Limits	20°

	Critical	Total	%
Direct Toppling (Intersection)	166	527	31.50%
Oblique Toppling (Intersection)	77	527	14.61%
Base Plane (All)	0	33	0.00%

Plot Mode	Pole Vectors
Vector Count	33 (33 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	527
Hemisphere	Lower
Projection	Equal Area

Figure B10.3: 60° slope angle toppling failure



Symbol	Feature
▫	Critical Intersection

Color	Density Concentrations
	0.00 - 3.20
	3.20 - 6.40
	6.40 - 9.60
	9.60 - 12.80
	12.80 - 16.00
	16.00 - 19.20
	19.20 - 22.40
	22.40 - 25.60
	25.60 - 28.80
	28.80 - 32.00

Contour Data	Pole Vectors
Maximum Density	31.23%
Contour Distribution	Fisher
Counting Circle Size	1.0%

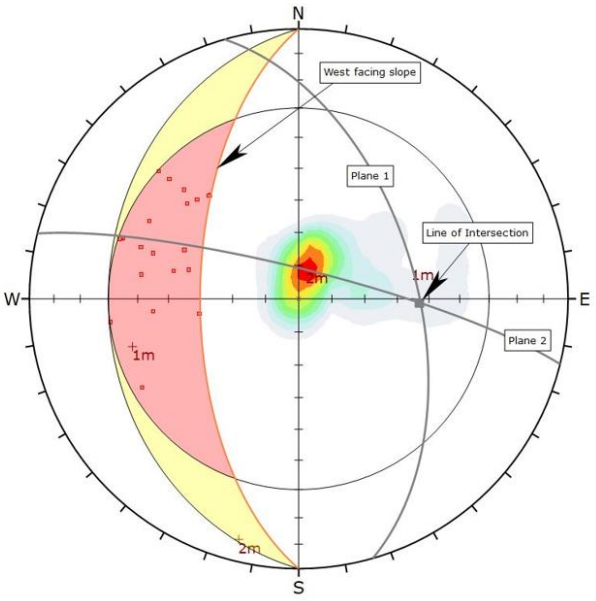
Kinematic Analysis	Direct Toppling
Slope Dip	70
Slope Dip Direction	270
Friction Angle	30°
Lateral Limits	20°

	Critical	Total	%
Direct Toppling (Intersection)	178	527	33.78%
Oblique Toppling (Intersection)	77	527	14.61%
Base Plane (All)	0	33	0.00%

Plot Mode	Pole Vectors
Vector Count	33 (33 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	527
Hemisphere	Lower
Projection	Equal Area

Figure B10.4: 70° slope angle toppling failure

Wedge failure



Symbol	Feature
▫	Critical Intersection

Color	Density Concentrations
	0.00 - 3.20
	3.20 - 6.40
	6.40 - 9.60
	9.60 - 12.80
	12.80 - 16.00
	16.00 - 19.20
	19.20 - 22.40
	22.40 - 25.60
	25.60 - 28.80
	28.80 - 32.00

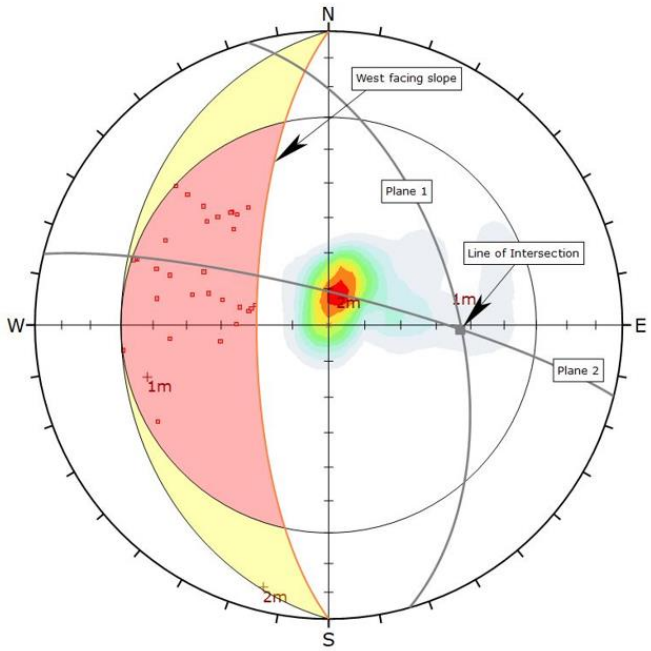
Contour Data	Dip Vectors
Maximum Density	31.23%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Wedge Sliding
Slope Dip	60
Slope Dip Direction	270
Friction Angle	30°

	Critical	Total	%
Wedge Sliding	23	527	4.36%

Plot Mode	Dip Vectors
Vector Count	33 (33 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	527
Hemisphere	Lower
Projection	Equal Area

Figure B10.5: 60° slope angle wedge failure



Symbol	Feature
▫	Critical Intersection

Color	Density Concentrations
	0.00 - 3.20
	3.20 - 6.40
	6.40 - 9.60
	9.60 - 12.80
	12.80 - 16.00
	16.00 - 19.20
	19.20 - 22.40
	22.40 - 25.60
	25.60 - 28.80
	28.80 - 32.00

Contour Data	Dip Vectors
Maximum Density	31.23%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Wedge Sliding
Slope Dip	70
Slope Dip Direction	270
Friction Angle	30°

	Critical	Total	%
Wedge Sliding	32	527	6.07%

Plot Mode	Dip Vectors
Vector Count	33 (33 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	527
Hemisphere	Lower
Projection	Equal Area

Figure B10.6: 70° slope angle wedge failure