IMPROVING SLOPE DESIGN and STABILITY ANALYSIS PROCESSES THROUGH DEVELOPING A RISK-BASED GEOTECHNICAL MODEL FOR OPEN CAST MINES AT MIDDELBURG MINE SERVICES.

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DECLARATION

I declare that this report is my own, unaided work. It is being submitted to partially fulfil the requirement for the Degree of Master of Science in the University of Witwatersrand, Johannesburg. 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.

06 July 2022

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Date

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DEDICATION

I dedicate this research work to my lovely three-year-old son, Nzumbululo Netshivhazwaulu, for being very courageous and confident in whatever situation he finds himself in. His courage is what sustained me through this research project.

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1 INTRODUCTION

Over the last five years at Middelburg Mine Services, it has been observed that although ongoing efforts are made to manage the risk of falls of ground and slope failures, slope failure incidents continue to occur. These incidents had adverse impacts on safety (injury and damage to property) and productivity. A concern about these incidents is that most of the failures were a surprise to the operation, and as a result, they were not controlled. This has also led to reserves being sterilised and productivity negatively impacted. Conservative design and stability analysis are often undertaken to minimise slope failure incidents that affect productivity due to the lack of a detailed geotechnical model.

Research has been undertaken in the past two decades with regard to slope stability in open-pit mines. Some of these research projects focused on geotechnical strategies and the optimisation of slope design through geotechnical modelling.

Butcher et al. (2001) described a geotechnical database relevant to highwall planning and geotechnical classification at South African collieries whilst developing a methodology for the safe cleaning of highwalls. The classification allowed for developing geotechnical domains used for design and highwall stability assessments. Bye (2003) demonstrated that the unforeseen risk can be reduced by improving geological and geotechnical knowledge, enhancing safety and productivity within an open-pit mining environment. This was confirmed by collecting detailed geotechnical information used to develop a three-dimensional (3D) geotechnical model for Sandsloot open pit at Potgietersrus Platinum Mine. According to Bye (2003) slope angle, Blastability Index (BI), smooth wall blasting and ore blast fragmentation were designed by making use of the 3D geotechnical model. Furthermore, it was demonstrated that application of the 3D geotechnical model resulted in considerable productivity and financial benefit. Little (2006) expanded from the research of Bye (2003) and described a geotechnical strategy for an open pit operation centred around a detailed geotechnical model. As described by Little (2006), this strategy improved safety and maximised the profitability of Potgietersrus Platinum Mine.

Through a rigorous review of slope performance and updating of the geotechnical model, Ekkerd (2011) demonstrated that steeper slope angles at Venetia Diamond Mine were viable. The optimised steeper slopes were then included in the mine's strategic business plan. Lato et al. (2013) detailed the importance of establishing a shared repository for storing rock mass characteristics acquired through LiDAR and photogrammetry. The shared storage is a standardised database that allows open access to view and adds information. This was said to have benefits in improving geotechnical modelling.

Vöge et al. (2013) noted that tools for geotechnical field investigations are being transformed by remote sensing technologies. It was further found that the evaluation, mapping, and

monitoring of slope hazards through terrestrial-based LiDAR and photogrammetry improves rock mass characterisation. Elmouttie & Karekal (2017) described an architecture that is centred on stochastic structural modelling, photogrammetric measurements and digital mapping for stability analysis for a highwall coal mining operation in India. In this architecture, through detailed probabilistic structural modelling, they demonstrated that the investigations of failures controlled by discontinuities are enhanced.

Russell (2018) has found that integrating face mapping data from laser scanners into the database improved geotechnical data confidence. It was further found that the geotechnical analysis features from the laser scanner and software became a vital tool for capturing geotechnical data provided a system exists for meaningful rock mass and structural information storage. Bester et al. (2019) used Sishen Mine to demonstrate that an ongoing risk-based approach, which includes a synthetic rock mass model, leads to a higher-confidence geotechnical model. Furthermore, this model allows for an integrated mine design process, which allows for optimised designs and early risk mitigation.

1.1 Problem Statement

Middelburg Mine Services have experienced many slope failures and related rock falls, which were not predicted. These failures have led to injuries, property damage and production losses. Investigation into the slope failures and fall of ground management strategy revealed no detailed geotechnical model during design and slope performance analysis.

1.2 Research objectives

Investigations into the fall of ground management revealed that no detailed geotechnical or risk analysis models are used through the design process, and that unpredicted slope failures continue to occur. The research described in section 1 above highlighted the importance of considering a detailed geotechnical model and risk-based approach throughout slope design and slope performance. The study aims to improve safety and productivity at Middelburg Mine Services by reducing uncontrolled rock falls and slope failures. The approach will be followed to develop a risk-based geotechnical design model that will minimise uncertainty in the design and slope stability analysis methods. The objective of the study is to improve the design and stability analysis process of an open cast mine through the following:

- Developing a risk-based geotechnical model;
- Incorporating a process for design and stability analysis that will minimise uncertainty;
- Incorporating remote sensing and photogrammetry evaluated rock mass data into the geotechnical model;
- Setting up a database for remote sensing technology-based data;

- Comparison of remote sensing data with that acquired following traditional field techniques;
- Setting up a geotechnical model which will be used for limit equilibrium analyses and numerical modelling of slope stability; and
- Following a methodology through design and analysis which incorporates economic risk evaluation.

1.3 Research Questions

It has been noted within the open cast coal industry, and specifically within the Witbank and Highveld Coalfields, that the slope design and slope stability analysis process often does not incorporate a detailed geotechnical model. As a result, the design and analysis processes are often highly dependent on local experience and rules of thumb. Knowing that generally, in open pit mines, geotechnical models form the basis for slope design and slope stability analysis, an investigation to determine whether a detailed geotechnical model at Middelburg Mines can be of benefit must be undertaken to answer the following research questions:

- Is the current design and stability analysis process very conservative?
- Will there be any value derived from developing a detailed geotechnical model?
- Will a risk-based geotechnical model improve the design and stability analysis process?
- Will implementing a risk-based geotechnical model improve slope failure and rock fall-related safety?
- Will the economic risk evaluation design approach be beneficial?

A process to attempt to respond to these questions constitutes the focus of this research.

1.4 Research Methodology

This research will lean more on quantitative methods and will use statistical tools, numerical methods, analytical methods and empirical methods as data analytical tools. The following techniques and tools will be used in conducting this research:

- Literature review to determine suitable analytical and verification procedures;
- Investigation of actual site geotechnical conditions and slope performance;
- Review of the existing slope management plan;
- Examination of the effectiveness of available geotechnical risk assessment techniques;
- Review of the current geotechnical database within Middelburg Mine Services;

- Acquiring a laser scanner and conducting mapping on exposed slopes to build the rock mass characteristics database;
- Developing geotechnical models;
- Conducting limit equilibrium and numerical modelling to analyse slope stability and verify the applicability of the geotechnical model; and
- To develop a complete economic risk evaluation process for Middelburg Mine Services.

1.5 Content of the Research Report

The research commences with a survey of literature related to slope design. This literature is recorded in Chapter 2 and is divided into general concepts of slope stability, namely, stability analysis, design, and risk evaluation. Chapter 3 describes the hypothetical slope stability model setting and analysis using both limit equilibrium methods from Rocscience software Slide2 and numerical methods from Rocscience software RS2. Results from these theoretical models are presented in Chapter 3 as a factor of safety, probability of failure and failure volume estimates. Next, the concept of a hypothetical model is used for a case study which is presented in Chapter 4. Whereby a risk-based evaluation method is applied to the same case study. This research's conclusions and recommendations are presented in Chapters 5 and 6.

2 LITERATURE REVIEW

A wide range of literature was reviewed to understand the challenge faced by Middelburg Mines. In Chapter 2 a review of literature related to slope stability is the subject at hand. The concept relevant to open pit slope stability, slope design principles and methodologies, slope stability analysis and slope risk assessment method will be briefly described.

2.1 Slope Stability In Open Pit

In 2014 South Africa's coal resource ranked 5th globally, with economic reserves projected to be 66700 million tons (Ramane, 2014). The Mineral Council of South Africa also stated that 47% of the coal is extracted using the underground bord and pillar method. The remaining 53% of coal is extracted using open pit mining methods (Mineral Council South Africa, 2019). This shift from underground coal production to open cast production increases the risk of rockfall and slope failure accidents. Therefore, the design of the pit slopes and analysis of their stability within the coalfields constitute the main priority for rock mechanics engineers within these environments. Pits within the coalfields are generally mined up to a depth of 80m, with multiple benches developed during the extraction of multiple coal layers, which are found within the South African industry. The strike length of pit walls within the coalfields extends for kilometres. However, where strip mining is practised, the life span of the pit slope is relatively short (1-10 months).

2.1.1 Slope Stability in Coal Open Pits

It has been stated by Wyllie & Mah (2004) that the acceptable stability requirements for any rock slope differ based on the acceptable consequence of failure for that rock slope. For example, it will be necessary for rock slopes along a highway carrying high traffic volumes to have a stable overall slope, such that few falling rocks, if any, reach the traffic (Wyllie & Mah, 2004). In contrast, Wyllie & Mah (2004) stated that open-pits are generally designed to give safety factors between 1.2 and 1.4. Therefore, it is acceptable that slope deformation and movements occur during the life of mine. They further concluded that a slope that fails immediately at the end of life of operation could be considered an optimum slope design. The stability of open pit slopes is influenced by a wide range of factors: regional tectonic setting, geological condition, and overall slope angle.

Common slopes created during coal open pit mining are highwalls, spoil piles, and waste dumps. The life span of these slopes, apart from the waste dumps, range between two and six months. Priest and Brown as quoted in Read & Stacey (2008), defined factors of safety (FoS) and probability of failure (PoF) as a function of acceptable slope failure consequence (see Table 1).

Table 1: FOS & POF guidelines by Priest & Brown (Read & Stacey, 2008)

Consequence of failure	Examples	Acceptable values		
		Mean FoS	Minimum P[FoS < 1.0]	Maximum P[FoS < 1.5]
Not serious	Individual benches; small (< 50 m), temporary slopes, not adjacent to haulage roads	1.3	10%	20%
Moderately serious	Any slope of a permanent or semi-permanent nature	1.6	1%	10%
Very serious	Medium-sized (50–100 m) and high slopes (<150 m) carrying major haulage roads or underlying permanent mine installations	2.0	0.30%	5%

Adams (2015) extended these acceptability criteria into an FoS-PoF selection matrix (see Table 2). This matrix provides a guide of the level of risk management required for each consequence level category. The level of risk will determine whether monitoring, comprehensive Ground Control Management Plan (GCMP) or Trigger Action Response Plan (TARP) is required. The benefit of this matrix is that it presents an opportunity to select more dependable acceptance criteria and thereby achieve an acceptable level of risk management. Wyllie & Mah, (2004) concluded that the design must balance stability and economics.

Table 2: FoS-PoF selection matrix (Adams, 2015)

Consequence Level[1]		Insignificant to Minor			Moderate			Major to Catastrophic		
Level of Design Confidence[2]		High	Med	Low	High	Med	Low	High	Med	Low
Permanent cut, fill or natural slope (Design Life > 10 years)	Min FOS	1.3	1.3	1.3	1.3	1.4	1.5	1.4	1.5	1.6
	Max POF	20%	20%	20%	20%	10%	5%	10%	5%	2%
	Level of Risk Management	No monitoring or access restrictions			No monitoring or access restrictions			Minimal monitoring for defined timeframe, and/or access restrictions		
Interim cut or fill slope (Design Life 0.5-10 years)	Min FOS	1.2	1.25	1.3	1.2	1.3	1.4	1.3	1.4	1.5
	Max POF	30%	25%	20%	30%	20%	10%	20%	10%	5%
	Level of Risk Management	Basic GCMP including periodic slope monitoring. Access dependant of safety risks			Comprehensive GCMP including slope monitoring and TARPS. Access dependant of safety risks			Comprehensive GCMP including slope monitoring and TARPS. Access dependant of safety risks		
Temporary cut or fill slope (Design Life < 6 months)	Min FOS	1.2	1.25	1.3	1.25	1.3	1.35	1.25	1.35	1.4
	Max POF	30%	25%	20%	25%	20%	15%	25%	15%	10%
	Level of Risk Management	Basic GCMP including periodic slope monitoring. Access dependant of safety risks			Comprehensive GCMP including slope monitoring and TARPs. Access dependant of safety risks			Comprehensive GCMP including slope monitoring and TARPs. No access to slope.		
Excavation for immediate backfill (Design Life < several days)	Min FOS	1.05	1.1	1.15	1.1	1.15	1.2	1.15	1.2	1.25
	Max POF	45%	40%	35%	40%	35%	30%	35%	30%	25%
	Level of Risk Management	Detailed risk assessment and robust operational controls, including continuous monitoring and TARPs. No access to slope			Detailed risk assessment and robust operational controls, including continuous monitoring and TARPs. No access to slope			Detailed risk assessment and robust operational controls, including continuous monitoring and TARPs. No access to slope		

2.1.2 Slope Failure Mechanisms

The technical services department focuses on minimising waste tonnage, maximising ore tonnage produced and improving profitability (Steffen, 1997). A planning engineer maximises the ore volume and minimises the volume of waste material produced. The ore volume is

maximised by steepening the slope of an open pit. This is because steeper slopes are more inexpensive to excavate compared with flatter slopes due to low waste volumes and reduced bench area (Wyllie & Mah, 2004). The higher the overall slope angle, the less the volume of rock removed. Common pit geometry is depicted in Figure 1.

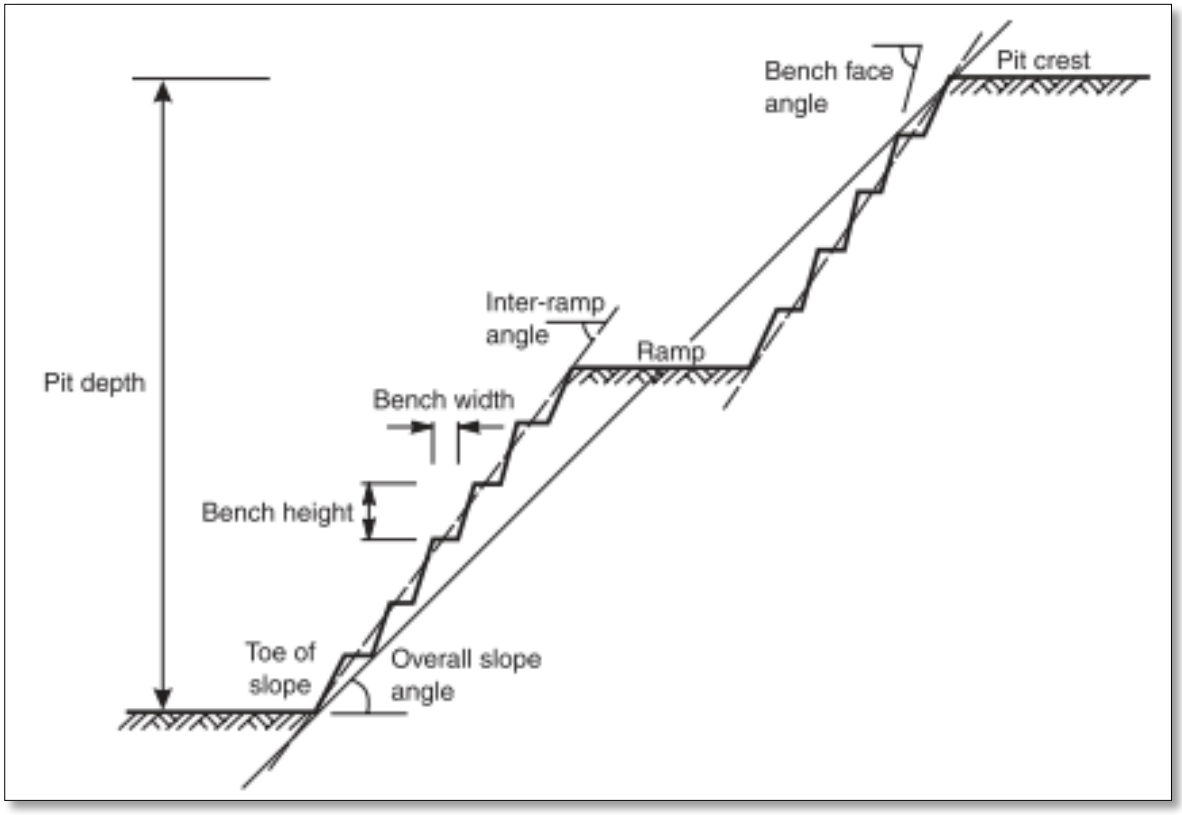

Figure 1: Typical open pit slope geometry (Wyllie & Mah, 2004).

During the design phase, it is relevant to consider the potential frequency and magnitude of likely bench or slope failures and whether they occur in a critical sector of the pit slope (Stacey, 2011). Wyllie & Mah (2004) identified three categories of slope failures, namely rotational, translational and toppling failure modes.

Rotational failure mechanisms

Rotational failure occurs in soil, overburden material, and deeply weathered rock material, whereby a geological structure does not affect the failure path (Stacey, 2011). Figure 2 depicts a typical failure outline for such a failure. In this case, the sliding path is at liberty to follow any plane with the lowest resistance throughout the slope (Wyllie & Mah, 2004). This rotational failure is regarded as a circular surface, and stability analysis for this failure is usually carried out by limit equilibrium procedures (Wyllie & Mah, 2004). The procedure compares shear stress acting onto the failure path with the resisting shear stress essential to maintaining slope

equilibrium. However, a reasonable stability estimate may also be determined quickly through stability charts (Read & Stacey, 2008).

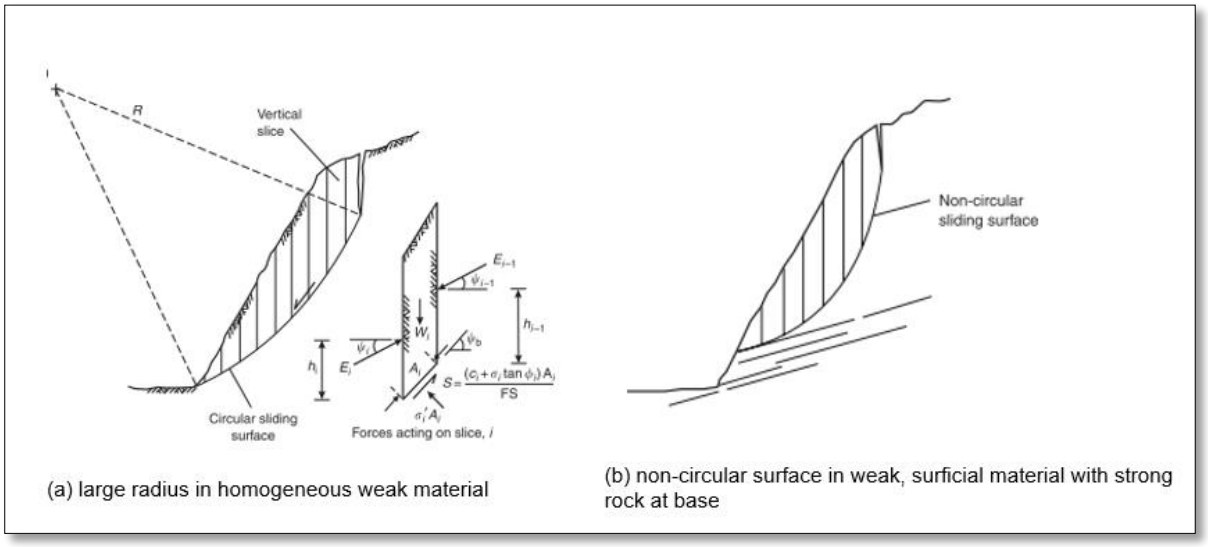

Figure 2: Typical shape of sliding surface for circular failure (Wyllie & Mah, 2004).

Translational failure mechanism

Translational failure occurs in situations where rock mass behaviour will be dominated by a geological structure (Stacey, 2011). The modes of translational failure include planar failure and wedge failure. It was stated by Stacey (2011) that kinematic stability analyses, which take into account the structural planes, are usually applied. However, it was argued by Wyllie & Mah (2004) that planar failure mode is reasonably rare since relevant slope geometry requirements are often not found in actual slopes. These requirements, as described by Wyllie & Mah (2004) are as follows (see Figure 3):

- The joint surface should be parallel/sub-parallel (±20°) to the highwall face.
- The joint surface should dip towards the pit, i.e. slope face angle > joint plane angle.
- The friction angle of the plane should be more than the joint dip angle.
- The plane's upper edge should either traverse through to the face crest or terminate on the existing tensile crack.
- Release surfaces that provide very little resistance must exist on both sides of the plane.

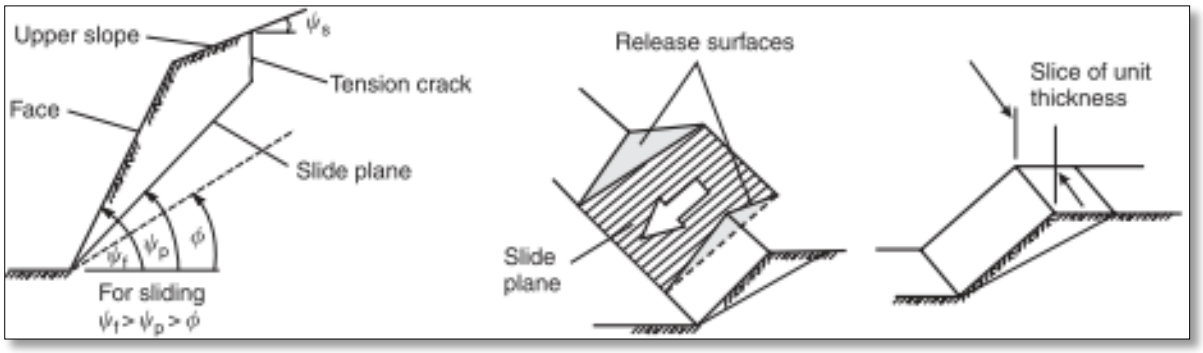

Figure 3: Geometry of slope exhibiting planar failure (Wyllie & Mah, 2004)

Rock mass characterisation defines the planes, the combination of which delineate potential wedge failure (Wyllie & Mah, 2004). Wedge failure is said by Wyllie & Mah (2004) to occur over a comprehensive range of rock mass and slope configurations compared to planar failure. As a result, the investigation of wedge failure forms an essential component of kinematic analysis during stability analysis. Wyllie & Mah (2004) also described the general geometric conditions which must be met for wedge failure as follows (also see Figure 4);

- Two joints should intersect, forming an intersection line;
- The plunge of the line of intersection must be flatter than the dip of the slope face but greater than the friction angles of the two planes; and
- The line of intersection must dip in the direction out of the face for sliding to be feasible.

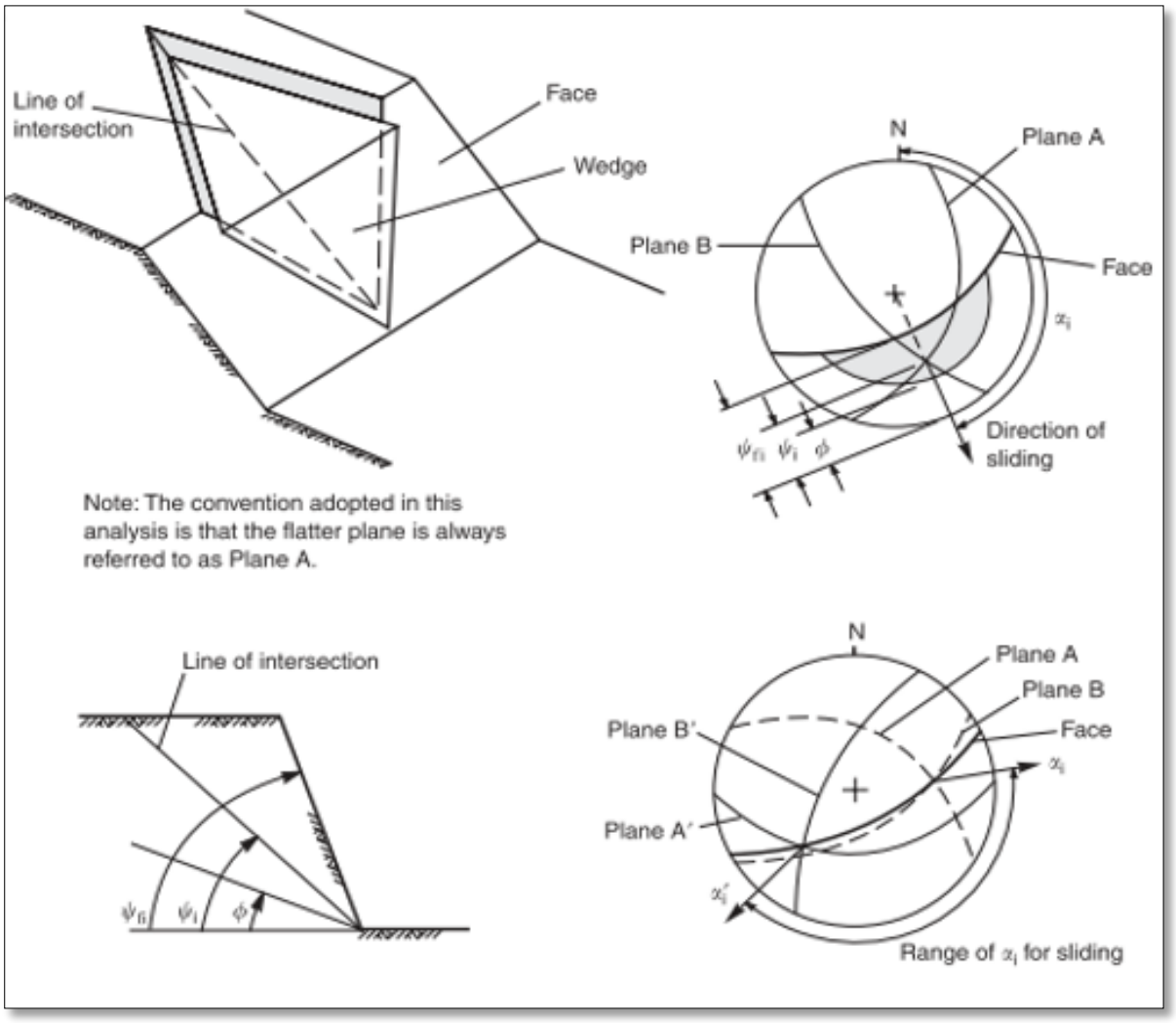

Figure 4: Geometric condition for wedge failure (Wyllie & Mah, 2004)

Design charts have also been used to evaluate two plane wedge stability (Wyllie & Mah, 2004). According to Wyllie & Mah (2004), these charts considers discontinuity strength alone, whereas cohesion and water are ignored.

Toppling failure mechanism

Toppling instability is known to be a special case of kinematic analysis (Wyllie & Mah, 2004). Wyllie & Mah (2004) further stated that, a large-scale toppling failure of rock slopes requires a combination of mechanisms, rotational, with associated shear on one or more structural planes. These often need time to develop to the ultimate failure state. Therefore, according to Wyllie & Mah (2004), it is essential to differentiate modes (block & flexural) of toppling failures because each follows a different stability analysis technique.

2.2 Slope Design

2.2.1 Design Principles

Mining slopes are generally designed to be as economical as possible and to provide safe operation. Steeper or economic slopes are often required to maximise profits without negatively impacting safety. Bieniawski (1991, 1992), as cited in Stacey (2009), defined design principles for rock engineering as summarised below;

- Design Principle No. 1: Clarity of design objective and functional requirements
    - This principle requires that, at the outset of the design, the objectives of the design and problem statements together with constraints are clearly stated.
- Design Principle No. 2: Minimum uncertainty of geological conditions
    - This principle requires that adequate geotechnical data and rock mass characteristic data be collected to aid design.
- Design Principle No. 3: Simplicity of design components
    - It is suggested with this principle that design is divided into a series of small components.
- Design Principle No. 4: State of the art practice
    - This requires that up-to-date concepts and analysis methods are used
- Design Principle No. 5: Optimisation
    - This principle is about minimising risk in the design by considering all other factors influencing risk.
- Design Principle No. 6: Constructability
    - A design should always be implementable.

Bieniawski (1984) further described a design methodology that was in line with these principles. The method is a ten-step linear process that can be used as a design checklist.

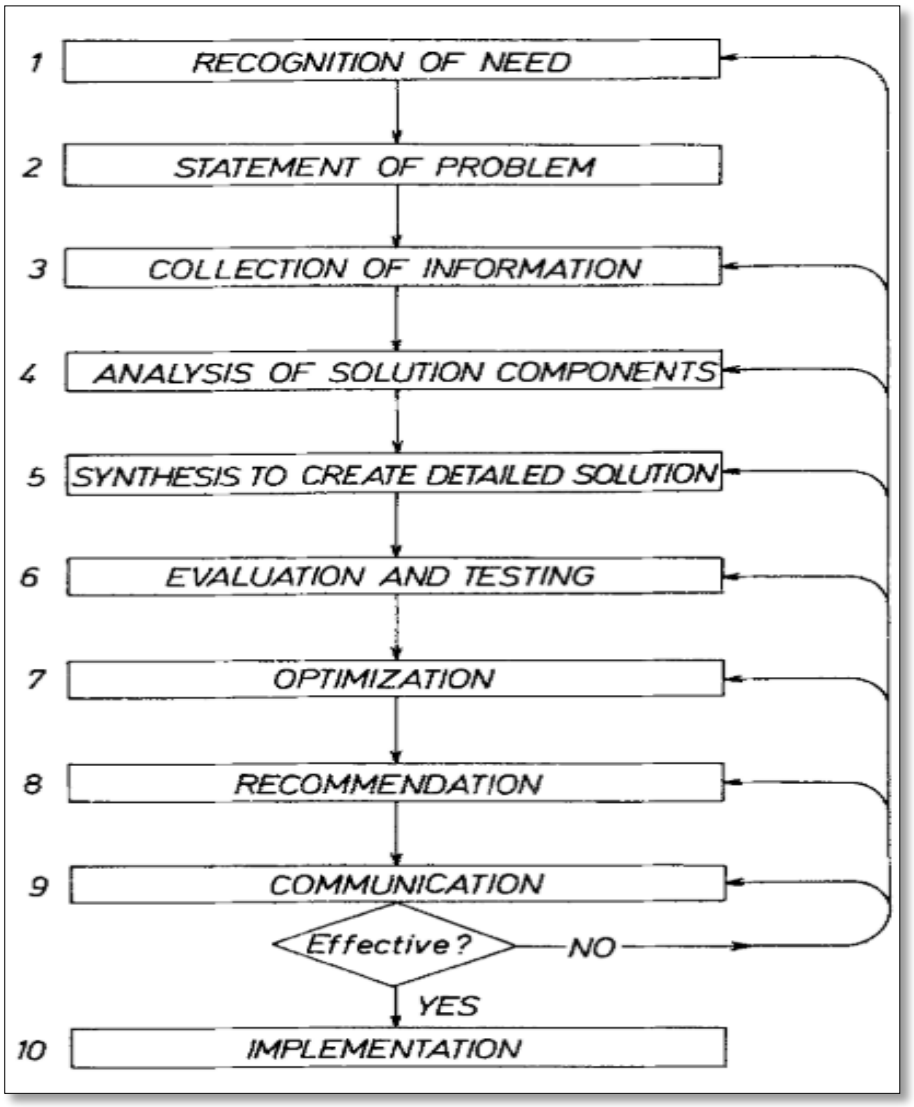

Figure 5: The engineering design process (Bieniawski, 1984)

2.2.2 Design Methodology

Bieniawski (1984) described a linear design methodology that comprised ten steps (See Figure 5). Stacey (2009) modified this design methodology after it was demonstrated that design could be split into two stages: definition of design and execution of design (wheel of design). Ilbury & Sunter (2005) described a strategic planning process termed strategic conversation. As illustrated in Figure 6, this process corresponded remarkably with Bieniawski's engineering design process (Stacey, 2009).

Figure 6: Strategic conversation model (Ilbury & Sunter, 2005)

The design wheel comprises the same ten parameters within Bieniawski's design methodology. The wheel of the design process (see Figure 7) only commences once the strategic process is completed (Stacey, 2009). It can be noted from the wheel of design that a review of the process is necessary after each step is completed. Stacey (2009) further stated that this review is critical in step No. 4 and step No. 9.

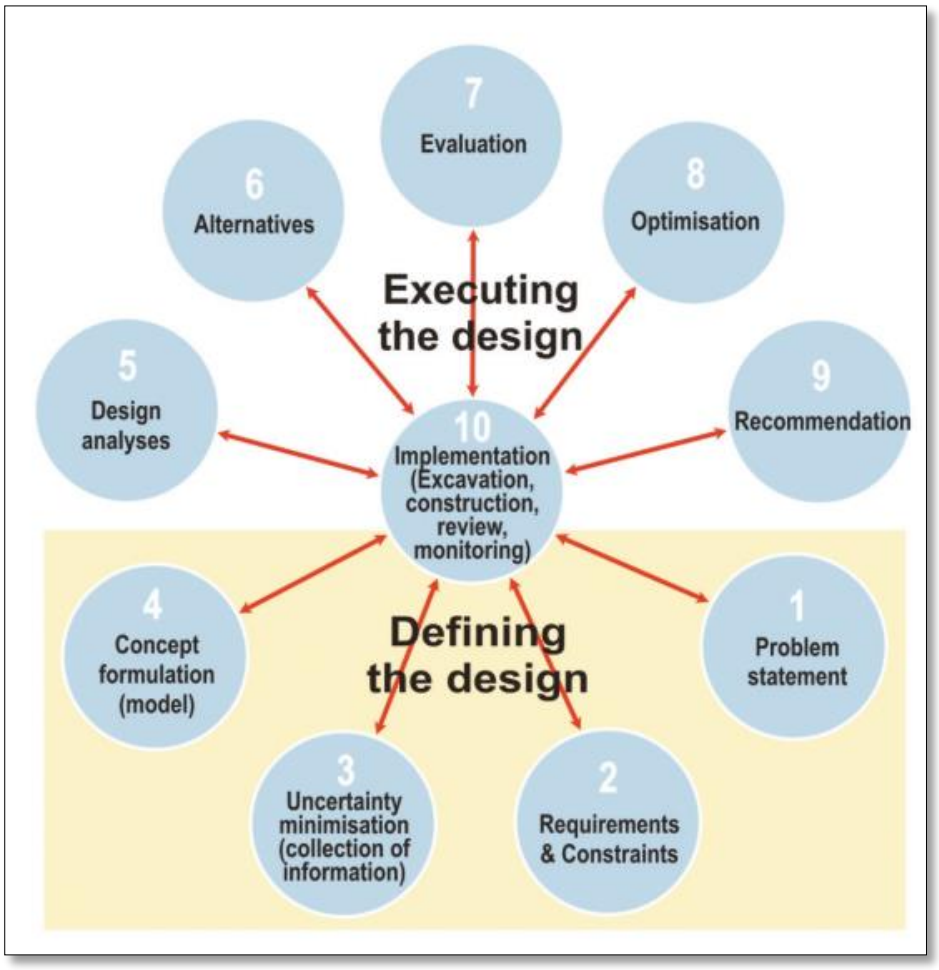

Figure 7: Wheel of design (Stacey, 2009)

The wheel of design methodology conforms with the six design principles described in the section above. Furthermore, it can be seen from this wheel that following this methodology will result in a detailed design, as stated: “Diligent use of a design process will ensure that a defensible design or evaluation has been carried out” (Stacey, 2009).

2.2.3 Uncertainty Effects on Design

Slope or rock engineering design takes place in natural geological materials that cannot be modified and are often not fully understood or known. Fillion & Hadjigeorgiou (2016) identified two sources of uncertainty regarding a design process: First is the aleatory uncertainty caused by the variability intrinsic to the rock material, whereby supplementary information does not remove this uncertainty nonetheless only allows for a better comprehension of it. Second is epistemic uncertainty, which comes from missing or limited information regarding incidents and methods, or even the absence of knowledge of these methods. Step No. 3 within the wheel of design is often concerned with epistemic uncertainty. This uncertainty cannot be eliminated from the design entirely, and as a result, the design can never be 100% reliable. Wong (2002)

wrote, “nothing can be 100% reliable and safe” and “human beings, one day, will invariably make a mistake.” Yet “mining companies often claim a ‘zero tolerance’ approach to accidents”.

The following methodology for constructing a reliable geotechnical model was suggested (Fillion & Hadjigeorgiou, 2016):

- Use appropriate guidelines for data collection;
- Collect data with a strategy for subsequent analysis;
- By collecting additional information, the level of confidence in the data increases and the precision range can be reduced; and
- With additional data collection, the ‘epistemic’ uncertainty due to lack of knowledge may be reduced to give a better understanding of the true variability (aleatory uncertainty) of the geotechnical data.

It was further suggested by Fillion & Hadjigeorgiou (2019) that, while creating guiding principles for choosing the best drilling spacings, geological and structural complexity should be well-defined.

In an attempt to understand the reliability of the geotechnical model, it was proposed by Steffen (1997) that the confidence of the slope design model should be classified according to a similar classification as those used for resource classification as per the following categories;

- A Proven Slope Angle (CAT1) - Detailed structural mapping of the rock fabric is implied and can be extrapolated with high confidence for the affected rock mass. In this case, the strength characteristics of the structural features and in-situ rock are determined by appropriate testing procedures to allow reliable statistical interpretations. Data reliability should be such that an analytical model can carry out the design to a confidence of 85%.
- A Probable Slope Angle (CAT2) - Testing (small sample) for the physical properties of the in-situ rock and joint surfaces will have been carried out. Similarly, groundwater data will be based on water intersections in exploration holes with few piezometer installations. This category allows for simplified design models to be developed where sensitivity analyses can be carried out.
- A Possible Slope Angle (CAT3) - corresponds to applying typical slope angles based on experience in similar rocks. Quantification will be based on rock mass classification and reasonable inference of the geological conditions within the affected rock mass.

Similar to Steffen (1997), Fillion & Hadjigeorgiou (2019) suggested that, for the geological model, confidence level criteria are used at various phases of a project. The confidence level for the geological model was described in Read & Stacey (2008) with the allowable variation in the data:

- Conceptual stage: ≤50% variation;
- Pre-feasibility stage: 30%–50% variation;
- Feasibility stage: 15%–35% variation;
- Design and construction stage: 10%–20% variation; and
- Operations stage: ≤10% variation.

The confidence levels described above are aimed at understanding the geotechnical model's reliability to minimise uncertainty and the associated risk. All risks in mine planning, including geotechnical risks, must be communicated quantifiably and transparently (Stacey et al., 2007).

2.2.4 Rock Mass Strength

The stability of any rock slope will rely on geological material strength and the discontinuities within the slope. Little (2006) stated that geotechnical engineering is complex compared to other types of engineering since slopes are made of natural material with inherent abnormalities, and as a result, failure may be difficult to predict. Furthermore, Little (2006) stated that exact material properties could be selected to match design requirements in other engineering fields, unlike geotechnical engineering.

Intact rock strength

The geomechanical properties of intact rock samples that occur between natural rock defects in a typical rock mass are determined in the laboratory from representative samples (Read & Stacey, 2008). It was said that the most common strength properties which are tested are tensile strength, uniaxial compressive strength (UCS) and triaxial compressive strength (Watson, 2021). Through this testing, it was also said by Watson (2021) that the elastic properties (elastic modulus and Poisson’s ratio) are also determined. Therefore, the in-situ rock strength is usually downgraded to estimate in-situ rock mass strength. One popular method of estimating rock mass strength from intact rock properties uses the Hoek and Brown Strength criterion (Hoek et al., 2002).

Strength of discontinuities

It was said by Hoek (2002) that rock masses encompass natural defects or structures which, include bedding planes, joint planes and fault planes. Therefore, it's essential to recognise all factors that affect discontinuity strength to be able to examine slope stability. Shear strength of a smooth discontinuity is usually expressed or assessed based on Mohr-Coulomb criterion, where peak strength is given by (Read & Stacey, 2008):

$$\tau_{max} = C_j + \sigma_n tan\emptyset_j \qquad \textit{Equation 1}$$

Where, τ_{max} is the maximum shear strength, C_j is joint cohesion, σ_n is normal stress and $\emptyset_j$ is the joint friction angle, the strength of a discontinuity can be affected by infilling material, joint opening, joint roughness and water properties. It was suggested by Barton & Bandis (1980) that the first estimate of peak friction angle can be:

$$\emptyset_j = tan^{-1}(\frac{J_r}{J_a}) \qquad \textit{Equation 2}$$

Where, J_r and J_a are joint roughness and joint alteration, respectively. J_r and J_a are based on the Q-System rock mass classification detailed in Read & Stacey (2008). Ueng et al. (2010) stated that, the concept of joint roughness was used to develop a non-linear empirical failure criterion where shear strength of discontinuities is:

$$\tau_{max} = \sigma_n tan\left(JRClog_{10}\left(\frac{JCS}{\sigma_n}\right) + \emptyset_b\right) \qquad \textit{Equation 3}$$

Where, $\emptyset_b$ is the basic friction angle, JRC is the joint roughness coefficient, and JCS is the uniaxial compressive strength of the rock wall.

Groundwater

The presence of water within any rock slope has negative impacts on the strength of the rock slope, resulting in detrimental effects on stability. Wyllie & Mah (2004) identified the reasons for this detrimental effect as follows:

- Water changes the moisture content of some rocks (e.g. shale), which thereby accelerate weathering and, in turn, reduce the shear strength of the rocks;
- Freezing of groundwater can cause wedging in water-filled fissures due to temperature-dependent volume sizes;
- Erosion of weathered rock by surface water and of low strength joint infillings by groundwater;
- Water pressure reduces the normal stress and thereby diminishes shear strength on potential failure surfaces; and

- Excavation costs may also be increased when working below the water table due to the need to use water-resistant explosives and pump water out of the pit.

In-situ stress

For in-situ rock masses, primitive stress is influenced by various events induced by gravitational factors and dynamic or tectonic processes (Read & Stacey, 2008). It was said by Read & Stacey (2008) that in the open-cast environment, stress is generally in dilationary, as opposed to confinement, and most slope failures are gravitationally driven. Due to this, they further stated that the effect of in-situ stress is considered to be low. However, the horizontal stresses usually determined at civil and mining projects worldwide have indicated the tendency towards high values of the K ratio close to the surface, which decreases with an increase in depth (Kanda, 2015). Therefore, locked in stress (LIS) is also an essential in-situ stress parameter that plays a role in rock mass behaviour. Furthermore, Tan & Kang (1981) affirmed that although the rock mass is subject to stresses related to rock formation, metamorphic and tectonic processes, residual stresses persist within the rock mass even long after these processes have stopped. These residual stresses are known as LIS.

It has been stated by Read & Stacey (2008) that the knowledge of pre-mining state of stress when conducting numerical analysis is beneficial to evaluate if they will possibly strengthen the stability of weak rocks, discontinuities and pore pressures.

Failure Criterion

A large number of failure criteria exist and have been used within rock engineering; however, the commonly used rock failure criteria are the Mohr-Coulomb (M-C) and Hoek-Brown (H-B) criteria (Watson, 2020). M-C and H-B are stress-based failure criteria, which according to Watson (2020) have advantages and disadvantages, which are depicted in Table 3.

Table 3: Stress-based failure criteria pros and cons (Watson, 2020)

Advantages	Disadvantages
Rock strength parameters derived from laboratory test, and rock mass classification	There is a need to account for unknowns using a safety factor
Variability in rock mass requires a safety factor, which may account for incorrect criteria	The assumptions of failure criteria dictate how the failure will be modelled
Served the rock engineering community well for design purposes, particularly at shallow depth	From a predictive rock mechanics point of view, stress-based criteria are not suitable

However, under deep level, where rock is brittle and there is elevated stresses, extensile crack growth is a likely mechanism causing failure to initiate around an excavation (Watson, 2020).

Mohr-Coulomb Failure Criterion

M-C criterion represents the residual strength of the rock when the shear failure progresses along the failure surface within an intact rock (Watson, 2020). It was stated by Wyllie & Mah (2004) that slope stability analysis involves an examination of the shear strength of the rock mass on a sliding surface, which is expressed by the M-C criterion. This M-C criterion is commonly expressed as follows:

$$\tau = C + \sigma_n tan\emptyset \qquad \textit{Equation 4}$$

Where τ is the shear strength, C is the cohesion, σ_n is the normal stress and $\emptyset$ is the friction angle. Therefore, the M-C criterion is likely to give incorrect results when the failure mechanism is not shear (Watson, 2020).

Hoek-Brown Failure Criterion

In order to determine input parameters needed for design tunnels and stopes in hard rock, Hoek and Brown developed an empirical criterion for rock and rock mass failure based on intact rock and rock mass models (Hoek et al., 2002). The generalised H-B failure criterion was given as:

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_{b/i}\frac{\sigma_3}{\sigma_{ci}} + S\right)^{\alpha} \qquad \textit{Equation 5}$$

Where, σ_{ci} is the UCS of intact rock, $\mathrm{m_{b/i}}$, α and S are material constants. For intact rock α and S are 0.5 and 1, respectively. The σ_{ci} and $\mathrm{m_i}$ are determined from rock laboratory testing.

Whereas for rock masses, rock mass classification values are required, from which other parameters are calculated as follows (Hoek et al., 2002);

$$m_b = m_i exp\left[\left(\frac{GSI-100}{28-14D}\right)\right] \quad \textit{Equation 6}$$

$$S = exp\left[\frac{GSI-100}{9-3D}\right] \quad \textit{Equation 7}$$

$$\alpha = 0.5 + \frac{exp\left(\frac{-GSI}{15}\right) - exp\left(\frac{-20}{3}\right)}{6} \quad \textit{Equation 8}$$

Where GSI is the geological strength index determined from rock mass classification, and D is a disturbance factor depending on blast damage and stress relaxation.

Hoek-Brown failure criterion is favoured because it has proven to be the practical failure standard (Kanda, 2015). However, according to Wyllie & Mah (2004), it is necessary to determine friction angles and cohesive strengths equivalent to the Hoek-Brown and Mohr-Coulomb criteria. This is because most geotechnical engineering software is Mohr-Coulomb failure criterion based (Kanda & Stacey, 2016).

2.3 Slope Stability Analysis

Stability analysis forms the basis of risk assessments which incorporate mitigating factors to achieve acceptable levels of risk in terms of safety and economics (Read & Stacey, 2008). According to Read & Stacey (2008), the main types of slope stability analysis are described as follows:

- Kinematic analysis, which is based on stereographic projections and is mainly applied to bench designs;
- Limit equilibrium analysis which is applied to;
    - Structurally controlled failures in bench and inter-ramp designs
    - Inter-ramp and overall slopes where stability is controlled by rock mass strength with or without structural anisotropy
- The numerical analysis uses finite and distinct element methods to assess and design for inter-ramp and overall slopes.

Only kinematic, limit equilibrium and numerical analyses will be described in this section. It is stated by Obregon & Mitri (2019) that rock slope stability assessments are mostly based on kinematics, numerical and analytical methods.

2.3.1 Kinematic Analysis

The selection of design slope angle is a critical decision that can have far-reaching effects on economics and operation (Obregon & Mitri, 2019). It is also said by Obregon & Mitri (2019) that the geotechnical slope design of a pit wall starts at bench scale configurations, where the slope stability is governed primarily by geological discontinuities within the rock mass. This necessitates structurally controlled failure (planar, toppling and wedge) to be the one determining whether the slope design is stable. Having a stable bench face provides a safe environment for personnel and equipment during activities adjacent to that bench.

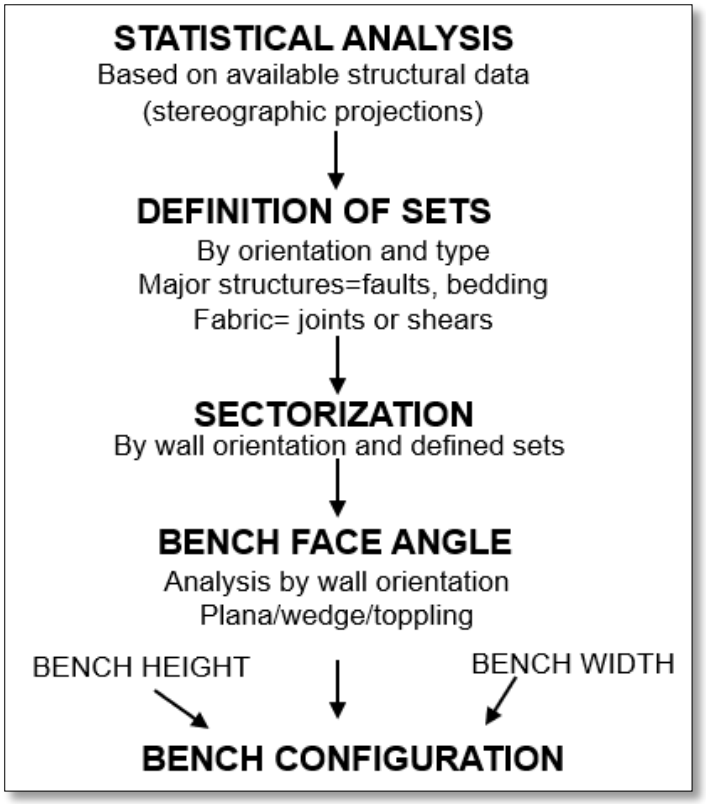

Figure 8: Bench face design process (Read & Stacey, 2008)

The design process depicted in Figure 8 is suggested when performing kinematic analysis on a bench scale. According to Wyllie & Mah (2004), a measure of slope stability is the probability of failure, and as a result, it is essential to conduct probabilistic analyses of structural failures. The probabilistic kinematic analysis is practical when joint orientation distribution for a slope lies within the stereographic projection's critical, unstable zone. For this reason, Obregon & Mitri (2019) defined the first step for probabilistic kinematic analysis as determining the probability density function for the joint set orientation. Obregon & Mitri (2019) suggested that the best methodology is a two-step approach; (1) determining the kinematic stability conditions through stereographic projections, and (2) limit equilibrium condition. Then both kinematic and limit equilibrium probabilities are combined to evaluate the total probability of failure for a given bench slope system.

2.3.2 Limit Equilibrium Analysis

Limit equilibrium analysis methods (LE) use representative geometry, material and joint strength, material unit weights, groundwater and external loading conditions to determine slope safety factors based on a set of simplified mechanical assumptions (Read & Stacey, 2008). According to Wyllie & Mah, (2004), limit equilibrium analysis is suitable for incorporating external forces acting on the slope to simulate a wide variety of actual conditions that may exist in the field. Limit equilibrium analysis (LE) is used efficiently during slope design to calculate the FoS & PoF related to the geotechnical model. LE has been a preferred risk analysis method because it is efficient compared with numerical modelling (NM) (Chiwaye & Stacey, 2010). For this reason, limit equilibrium methods have been well adopted when conducting slope stability analysis (Ekkerd, 2011).

Hoek, (2009) stated that LE programs generally process very fast, and as a result, a well-designed limit equilibrium program is the best tool for the “what if” type of analysis at a conceptual slope design stage. In addition, the LE program uses slices methods of analysis. It has been reported by Utili & Crosta (2015) that although different solution techniques for the method of slices exists, the first method was developed by Fellenius in 1936. Various slice methods have been used in LE programs. The Fellenius method was known as the Swedish circle method, where the factor of safety is achieved by the overall moment of equilibrium around the centre of a circular slip surface (Utili & Crosta, 2015). Following the development of the slices method, different solution techniques were used for stability analysis. These methods, including Janbu-1954, Bishop-1955, Nonveiller-1965, Spencer-1967, Morgenstern and Prince-1965 and Sarma-1976, all include dividing the sliding mass into a series of vertical slices (Chiwaye, 2010). Figure 9 depicts an illustration of a typical method of slices.

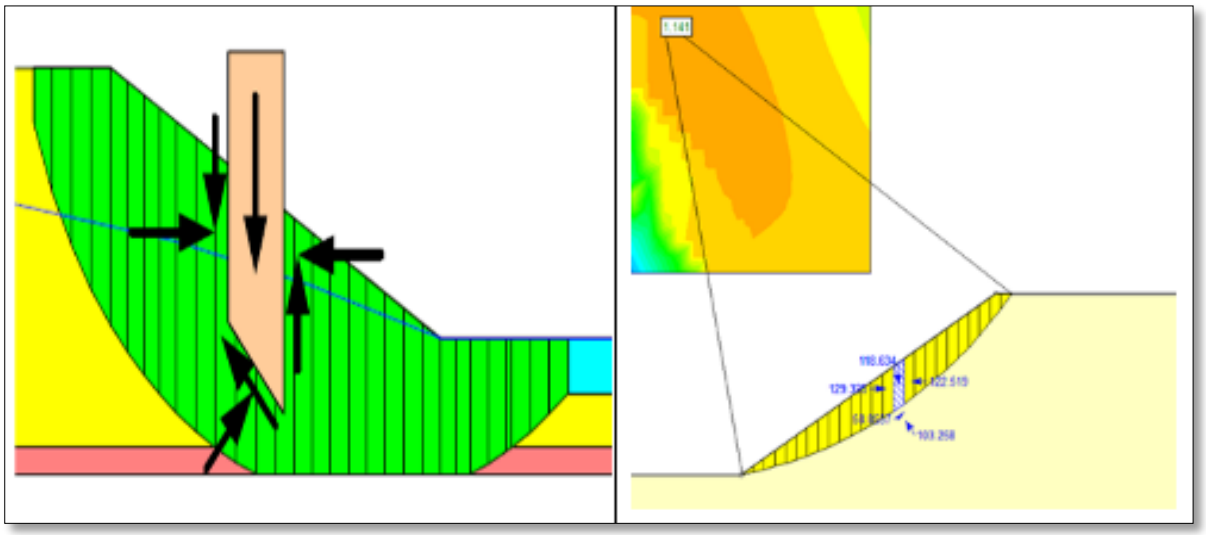

Figure 9: Typical method of slices illustration (Kanda, 2015)

The method of slices gained popularity and became economically viable for developing commercial software based on limit equilibrium methods (Utili & Crosta, 2015). Methods of slices are classified by Utili & Crosta (2015) according to the following criteria:

1. Those suitable to circular failure surfaces;
2. Rigorous or simplified: rigorous satisfies all the equilibrium conditions, whereas simplified satisfies only some equations. For this reason, some authors developed two versions (simplified and rigorous);
3. Depending on assumptions made to render the problem statically determinate; and
4. Based on the parameter used to determine the critical surface, i.e. FoS.

The common solution techniques for slices are very similar but have different assumptions (Kumar et al., 2021), as briefly described below:

- Janbu 1954 - this method allows a sliding surface of any shape and assumes that the side forces on the slices are horizontal and equal on all slices (Janbu, 1954).
- Bishop 1955 - this method was described and tested on various case studies by Bishop (1955), who defined both the rigorous and simplified versions. However, it was said by Bishop (1955) that the simplified slices method is not sufficiently accurate where field measurements of pore pressure are used;
- Morgenstern and Price 1965 - this method was developed to determine the safety factor of a sliding body on any surface containing material of varying shear strength and pore pressure (Morgenstern & Price, 1965);
- Spencer 1967 - this method was proposed as a two-dimensional slope stability analysis method that employs a circular failure surface and parallel inter-slice forces. Spencer (1968) argued that this method gives better results in certain classes of practical problems.

The different solution techniques for the method of slices described above have different assumptions and applicability slices (Ayati et al., 2016). These differences are summarised in Table 4.

Table 4: Summary of methods of slices (Ayati et al., 2016)

Method	Assumptions	Equations used	Slip surface
Ordinary method of slices (1936)	• Resultant of side forces (Ei) is parallel to the base of the slice	• Overall moment	Circular
Bishop (1955)	• Resultant of side forces is horizontal	• Overall moment • Vertical forces	Circular
Janbu(1956)	• Location of side force resultants on the sides of the slice (location can be varied) • Uses a correction factor f_o To account for the effect of the inter-slice shear forces.	• Overall moment • Vertical forces • Horizontal forces • Slice moment	Any
Morgenstern and price(1965)	• Inter-slice forces (Xi) related by • V = λ f (x) E form of f (x)	• Overall moment • Vertical forces • Horizontal forces • Slice moment	Any
Spencer(1967)	• Inter-slice forces are parallel	• Overall moment • Vertical forces • Horizontal forces • Slice moment	Any

It has been observed by Ekkerd (2011) that from all various slice models, the only three which satisfy force and moment equilibrium are Bishop, Spencer and Morgenstern-Price. However, the preferred solution techniques for hard rock slope stability are Spencer, Morgenstern-Price and Janbu because they can model irregular failure surfaces (Lorig et al., 2008). Furthermore, Bishop's method also computes better solutions than circular failure surfaces, whereas Janbu's method is of benefit for composite failure surfaces (Chiwaye, 2010).

2.3.3 Numerical Model Analysis

Numerical modelling is defined as a mathematical representation of physical behaviour of materials that is based on relevant hypotheses and simplifying assumptions (Brischke & Humar, 2017). It was argued by Rathod & Rao (2012) that, the three main aspects which influence slope stability analysis are as follows: 1) material properties of slope model; 2) definition of slope failure; and 3) the method of calculating factor of safety.

Although limit equilibrium analysis methods are simple and have been well adapted to slope stability problems in jointed rock masses, still, they cannot represent the deformation and displacement of a failing rock mass (Read & Stacey, 2008). However, numerical methods, on the other hand, are stress-based. As a result, they are primarily focused on analysing rock mass deformation instead of LE methods that deal with slope stability conditions (Kanda, 2015). Therefore, where movement is expected or detected in a slope, LE methods are not suitable.

For this reason, in their research, Chiwaye & Stacey (2010) noted that NM had found increased usage among slope designers. This is because NM has a more remarkable ability to handle models with complex failure mechanisms, and complex geometry, geology and in-situ stress conditions (Chiwaye & Stacey, 2010).

The most common numerical analysis methods are; Finite Element Methods (FEM), Discrete Element Methods (DEM), Finite Different Methods (FDM) and Boundary Element Methods (BE) (Watson, 2020). However, only FEM and FDM are the most common numerical methods for slope stability analysis in 2010 (Chiwaye, 2010). Numerical modelling methods divide the slope into a finite number of zones or elements. According to Chaulya & Prasad (2016) the relevant equation of equilibrium is approximated over each element using applicable constitutive laws for the material making up the slope.

Satyanarayana et al. (2021) observed that evolution in computer efficiency paved the way for employing several numerical methods in the slope stability engineering domain. This statement is supported by observing research by Bester et al., 2019, who used the Distinct Element Method, 3DEC software to conduct stability analysis. Furthermore, Bester et al. (2019) used a ubiquitous joint rock mass (UJRM) modelling technique in 3DEC to improve the definition of geotechnical design sectors and optimise pit slope angles.

2.3.4 Comparison of LE and NM Analysis

In their research, Chiwaye and Stacey (2010) came to the following essential conclusions regarding limit equilibrium and numerical analysis:

- In NM, in contrast with the LE approach, the failure surface is not chosen but is determined by the development of failure corresponding with the stress distribution in the slope and the specified failure criterion;
- Those parameters, such as dilation angle, not accounted for in LE analysis tend to lower the probability of failure of slopes; and
- Thus, numerical models are better tools for determining the failure surfaces/volumes for slope failures.

Chiwaye (2010) concluded that:

- LE methods estimate FOS with no information on deformation of the slope, something which NM methods provide; and
- LE solutions only identify the onset of failure, whereas NM solutions include effects of stress redistribution and progressive failure after the initiation of failure.

Kanda and Stacey (2016) have also concluded the following:

- Using the Mohr-Coulomb criterion in evaluating failure volume leads to lower sliding volumes than when the Hoek-Brown criterion is employed;
- NM methods consistently predict greater risk than the LE method; and
- The LE technique does not consider the k-ratio and the locked-in-stresses (LIS), and therefore underestimates the predicted risk of slope failure.

Rathod & Rao (2012) stated the following advantage of the finite element method:

- No assumptions must be made in advance about the shape and location of failure;
- No assumptions are made about slice slide forces and their directions;
- It is applicable in complex slope configurations in both two and three dimensions;
- Equilibrium stresses, strains and shear strength are calculated accurately;
- The critical failure mechanism is general and does not need to be circular or spiral arcs; and
- This method is able to monitor progressive failure.

The research described shows that using LE and NM techniques during slope analysis and risk evaluation can yield valuable results.

2.4 Risk Evaluated Design

The mining business is regarded to be taking place in a risky environment. This risk extends from orebody or reserve to economic, environmental and safety risks. In open pit mining, the safety risks often include the risk of slope failure, which also impacts the economics of the business. Bester et al. (2020) described the objectives of a risk-based design to be as follows;

- Defining acceptable risk in terms of safety and economics
- Assessing relative risk levels for different slope configurations
- Benchmarking risks against industry norms and the corporate mission statements

2.4.1 Risk Management Principles

The inherent risk profile of a mining business suggests that those undertaking a mining operation should have accepted risk to a certain extent since it cannot all be eliminated. This is

also true for design and was brought to light by Stacey et al., (2007) when they stated that “An inability to accept a non-zero tolerance for design indicates a lack of appreciation of human capabilities”. As a result, risk management has gained popularity in the mining industry to reduce the occurrence of catastrophic events known to have occurred in the past. However, formal risk assessment and management have a long history in the aviation, military, nuclear, petrochemical, and space industries (Read & Stacey, 2008).

On the subject of risk management, Taleb et al. (2009), in their research, noted the following shortcomings:

- Low-probability events are challenging to predict but dominate the environment;
- Humans think they can manage risk by predicting extreme events. In addition, low probability events often don’t have precedents;
- Today’s world does not resemble the past because interdependencies and non-linearities have increased;
- Humans mistakenly use hindsight as foresight;
- Humans do not listen to advice about what they should do;
- It is assumed that risk can be measured by standard deviation;
- It is also assumed that what is mathematically equivalent is psychologically so; and
- The most significant risk lies within risk managers who overestimate human abilities and underestimate what can go wrong.

International Standards Organisation (ISO) developed risk management practices and guidelines to address the listed shortcomings. The Risk Management-Practices and Guideline, known as ISO 31000 of 2018, is a standard that can be used across all industries throughout the world (Institute of Risk Management, 2018). According to Olechowski et al. (2016), the ISO standard focuses on eleven principles of risk management that will lead to improved risk management: risk management creates value; is an integral part of optimisation process; is part of decision making; explicitly addresses uncertainty; it is systematic, structured and timely; based on available information; transparent and inclusive; dynamic, iterative and responsive to change; takes human and cultural factors into account and it facilitates continual improvement.

The risk management framework, depicted in Figure 10, was suggested by the ISO 31000 documentation. As was detailed in the University of Pretoria (2021), the scope, context, criteria in the framework encompass the following:

- The organisation's governance policy documents should record the risk appetite statement and principal risk evaluation; and

- Documentation of the risk capacity statement and total revenue expenditure related to risk control research, risk control framework and risk control advice

Figure 10: Risk management framework (Institute of Risk Management, 2018)

Both regulatory and corporate standards require appropriate geotechnical design and slope management in the mining industry. According to Dixon et al. (2011), this is because engineers have a professional and legal 'duty of care' to design products, processes, and systems as safe as reasonably practicable. This makes risk assessment in slope design a critical part of risk management in open pit mines. Therefore, Read & Stacey (2008) defined risk assessment in slope engineering as overall risk identification, analysis, and evaluation.

2.4.2 Risk Design Process

The success of a mine design is dependent on three factors. These factors were described by Stacey et al. (2007):

- Resource and Geotechnical model, performing as predicted;
- The assumptions made for productivity and costs being achievable; and
- Overall skills supporting the plan.

It can be noted from these factors that a geotechnical model impacts the success of the mine plan and must perform as predicted. Therefore, in the risk analysis process, as opposed to the traditional non-risk design approach, the risk criteria for each consequence are determined at the outset and design is performed to meet these criteria (Stacey et al., 2007).

According to Contreras (2015), the standard risk design approach includes the factor of safety approach, probability of failure approach and risk analysis approach. These are described as follows:

Factor of Safety (FoS) Approach

This is a deterministic design approach where mean parameters are used as input into design to determine the factor of resistive forces to demand. The FoS approach is the oldest for slope design and is defined as the ratio between resisting forces and driving forces along a potential failure surface (Tapia et al., 2007). Contreras (2015) noted two disadvantages of slope design in this approach: Firstly, the acceptability criteria for this approach are based on limited case studies. Secondly, FoS does not provide a linear scale for the likelihood of failure. This means that a higher FoS does not automatically represent a safer slope than a lower FoS, as the influence of uncertainties is not apprehended within FoS. Traditionally FoS of 1.3 has been used as an acceptable value for pit slope design. According to Tapia et al. (2007), the FoS of 1.3 criteria was based on back analysis results of slopes at various mine sites, as depicted in Figure 11.

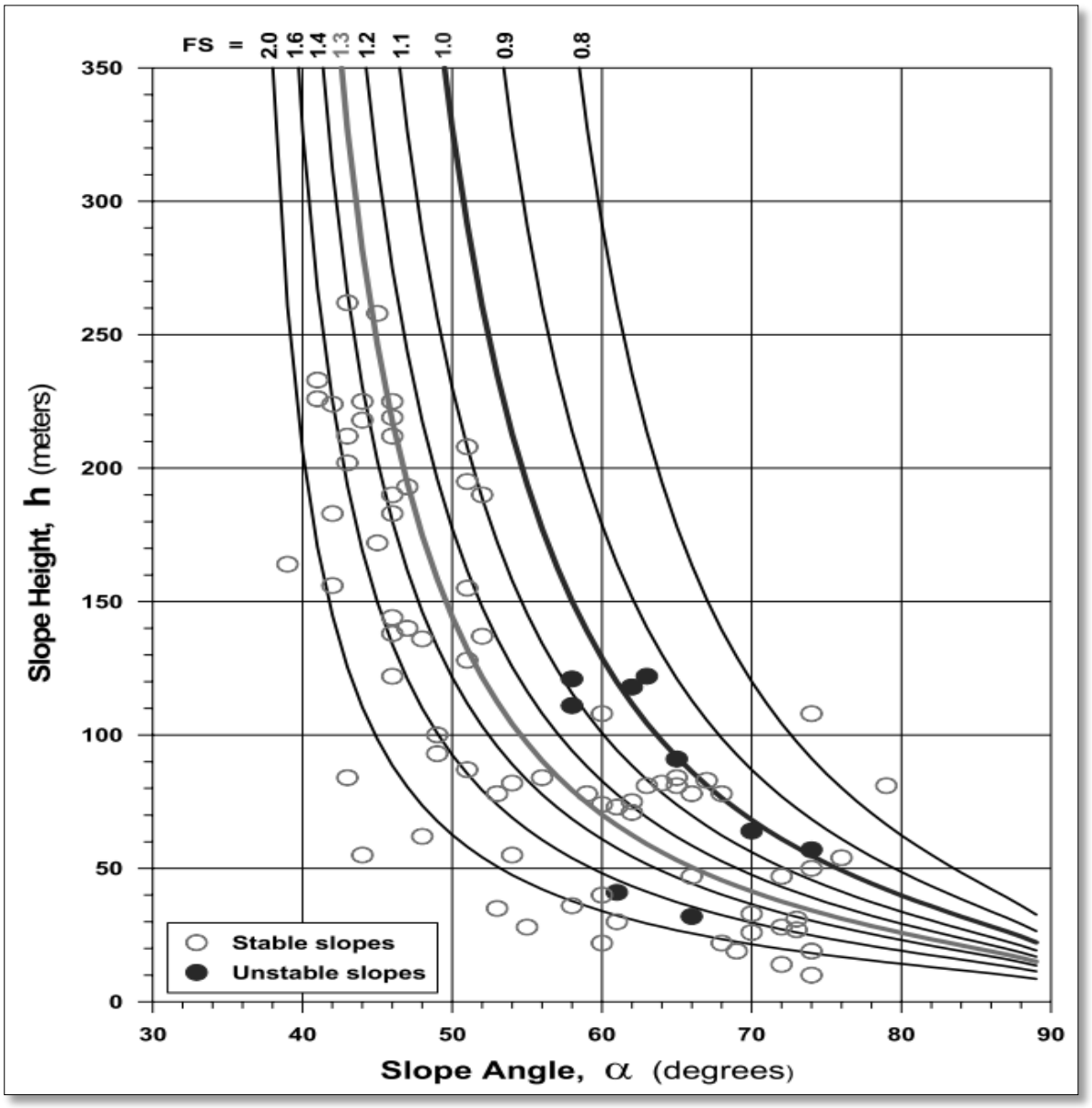

Figure 11: Cases of stable and unstable rock slopes, cited in Hoek and Bray, 1974 (Tapia et al., 2007).

Probability of Failure (PF) Approach

This is a probabilistic method where a statistical distribution of input parameters is used. The probabilistic methods have gained popularity and have been frequently used by geotechnical engineers for slope design. However, the PoF approach requires a deterministic model because input parameters are described as probability distributions (see Figure 12), instead of point estimates to calculate the probability of stability (Tapia et al., 2007).

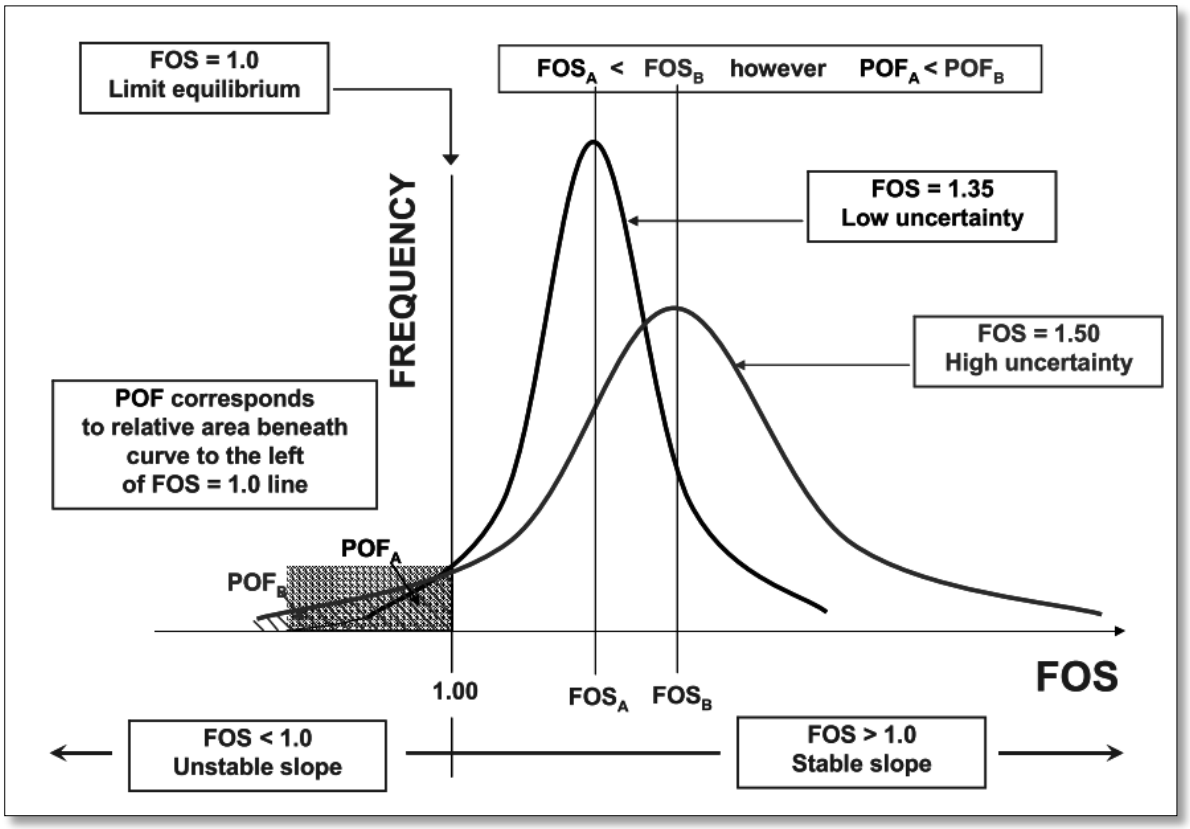

Figure 12: PoF and FoS relationships (Tapia et al., 2007)

Unlike FoS, the PoF value has a linear relationship with the likelihood of failure, but the acceptability criteria disadvantage persists (Contreras, 2015). For example, the linear relationship with likelihood means that a 7% PoF is double the stability of a 14% PoF, but the FoS of 2.0 is not double the stability of a FoS of 1.0. Many researchers have provided tables for PoF acceptability criteria, such as in Table 5. However, Tapia et al. (2007) argued that although PoF acceptability criteria tables exist, they should be considered only a guideline. The actual criterion to be used must be specific to the operation where it is used based on the analysis of the consequence of failure. Therefore, these PoF values are incomplete representations of the consequence of failure because they only consider geotechnical parameters.

Table 5: Acceptability criteria, PoF (Read & Stacey, 2008)

Category	Description	Acceptable PoF
1	Critical slopes where failure may affect continuous operation and pit safety	<5%
2	Slopes where failure have a significant impact on costs and safety	<15%
3	Slopes where failure has a notable impact on costs and where minimal safety hazards exist	<30%

One of the great benefits noted in probabilistic modelling is that once the probability of occurrence of a slope failure has been determined (PoF calculated), it can be directly imported into the risk assessment and optimisation framework of an engineering design process (Zevgolis et al., 2018).

2.4.3 Risk Analysis Approach

Adopting a risk approach to slope design in open pit mines allowed mine owners to define the risk criteria taking into account specific consequences of failures (Terbrugge et al., 2006). The design process following risk analysis process was summarised by Terbrugge et al. (2006) as follows:

- Determination of risk criteria for each consequence from the outset;
- Establish best practice management tools for slope performance;
- Calculate the required POF for slope design;
- Perform the slope design to the required reliability at the required level of design; and
- Collect geotechnical data to the required confidence level.

Risk is defined as a product of the likelihood of an event occurring, and the consequence should such an event occur (University of Pretoria, 2021). The risk was also described by the ISO73 in terms of risk sources, potential events, their consequence and their likelihood, where it is generally expressed by the following equation (University of Pretoria, 2021);

$$R(likelihood\ of\ event\ occuring\ \&\ Consequence\ should\ event\ occur)$$

which is loosely expressed as;

$$Risk = likelihood * consequences$$

When it comes to slope design, likelihood is represented as a PoF of the slope, and the consequence can be two-fold, personnel and economic impact (Tapia et al., 2007). However, it is essential to note that a thorough evaluation of PoF incorporates other sources of uncertainty. According to Tapia et al. (2007), this is because PoF determined as part of the design process is usually based on a slope stability model and accounts only for a part of the uncertainty of the slope.

The risk analysis method tries to solve the disadvantages of the FoS & PoF methodology regarding the appropriate acceptability criteria. Zevgolis et al. (2018) argued that in geotechnical engineering, a safety factor could be misleading regarding the corresponding probability of failure due to the ambiguity and non-linearity between them (FoS & PoF) and the corresponding risk level. This is because the FoS or PoF approach does not base the acceptability criteria on the consequence. An ALARP (As Low As Reasonably Practical) principle is generally adopted when it comes to risk with personnel impact or safety impact. The Mine Health and Safety Act in section 2 state, “The employer of every mine that is being worked must- (a) ensure, *as far as reasonably practicable*, that the mine is designed, constructed and equipped-(i) to provide conditions for safe operation and a healthy working environment” (Department of Mineral Resource, 2018). An example of risk acceptability criteria based on safety where the ALARP region is indicated is shown in Figure 13.

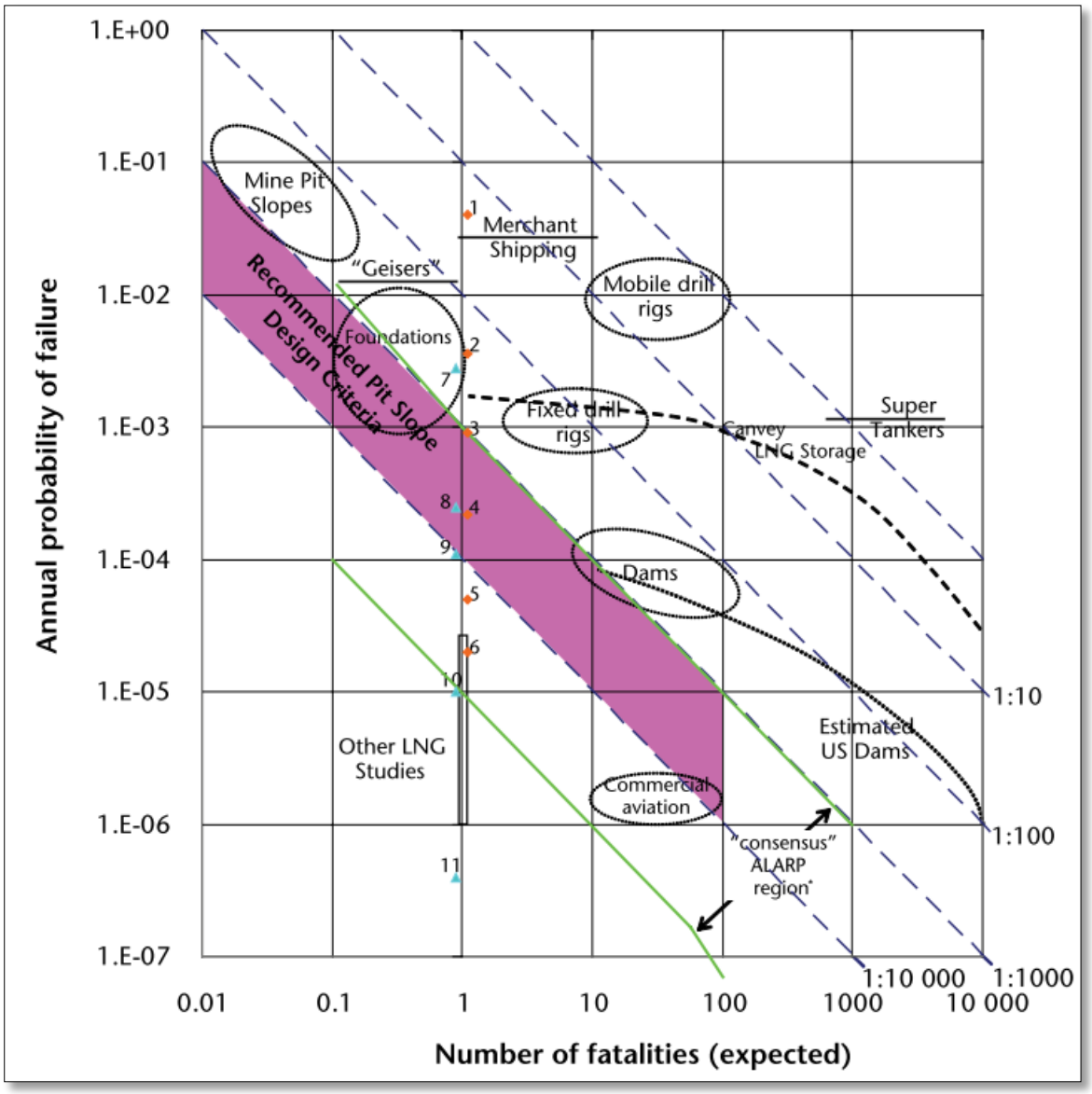

Figure 13: Risk acceptability criteria (Terbrugge et al., 2006)

Various techniques for analysing risk exists, and some of the standard techniques in slope design are as follows (Read & Stacey, 2008);

- Event Tree Analysis- is a classical qualitative tool that provides systematic mapping (see Figure 14) of a realistic event scenario with the potential to result in a significant incident.

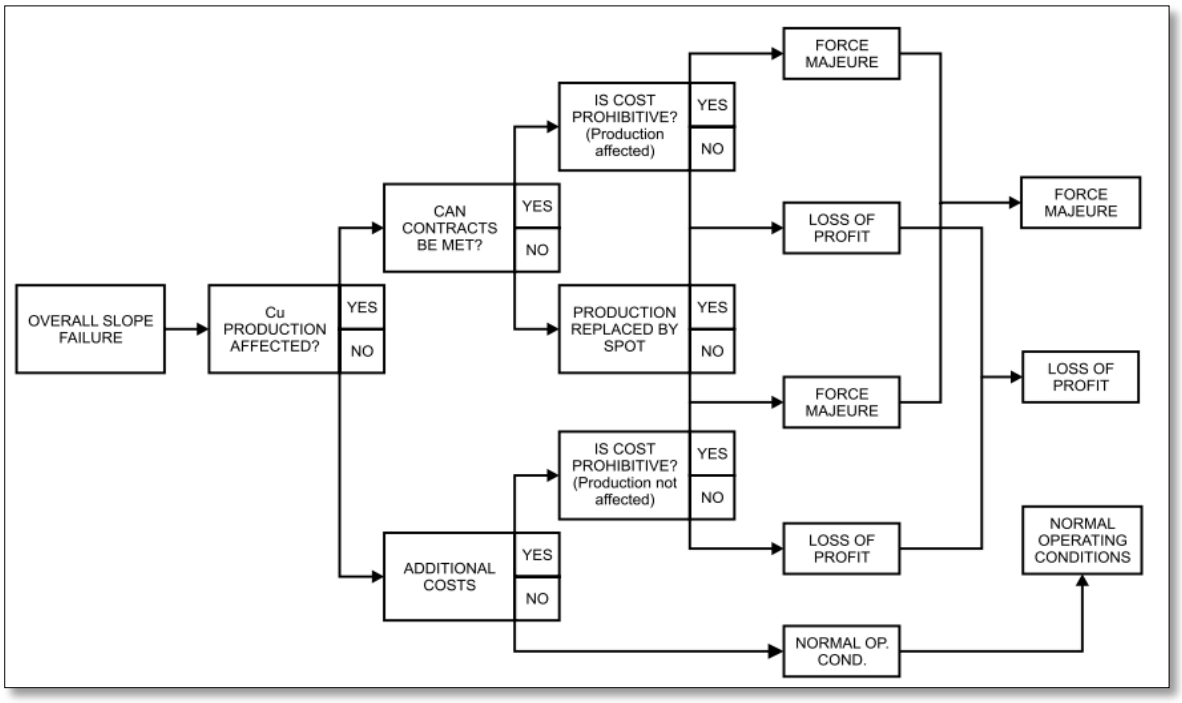

Figure 14: Event tree diagram for slopes (Tapia et al., 2007)

- Risk Matrices- are a product of likelihood and impact in a table form where a risk outcome of likelihood and consequence of an event is evaluated based on the acceptance criterion or severity criterion. Matrices are valuable means of communicating the results of a risk assessment (see Figure 15).

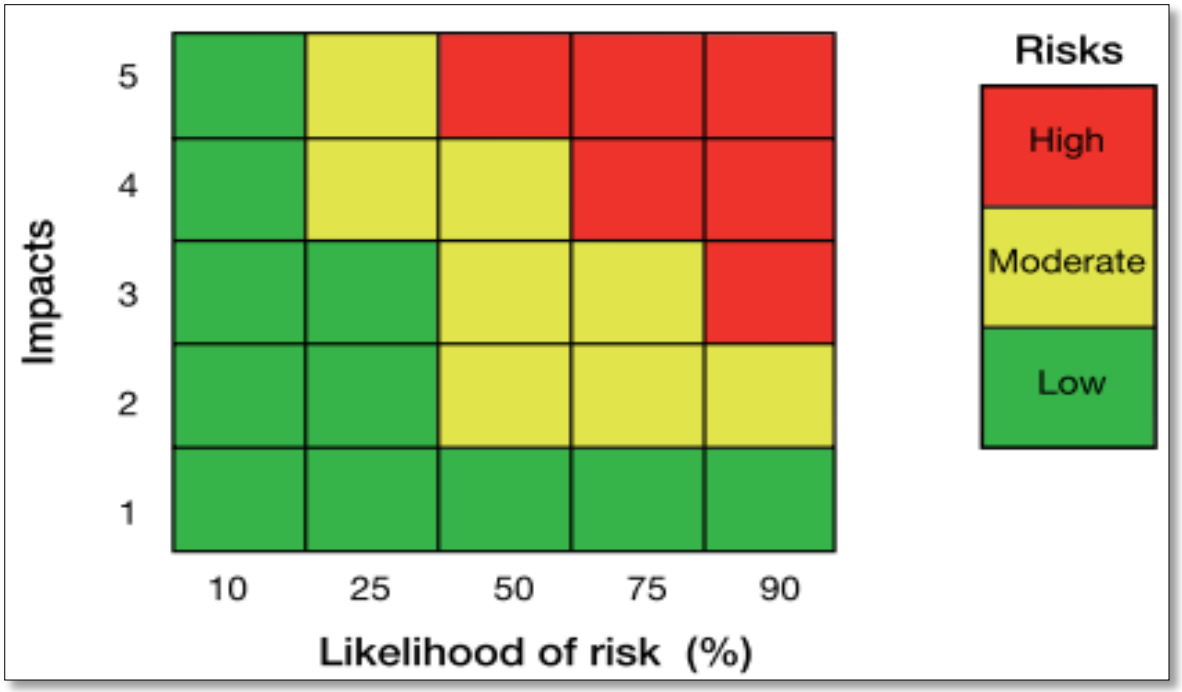

Figure 15: Typical risk matrix (Read & Stacey, 2008)

- Fault Tree Analysis- this tool identifies, quantifies and represents the faults or failures and the combination of faults or failures which can lead to a significant event (See example in Figure 16)

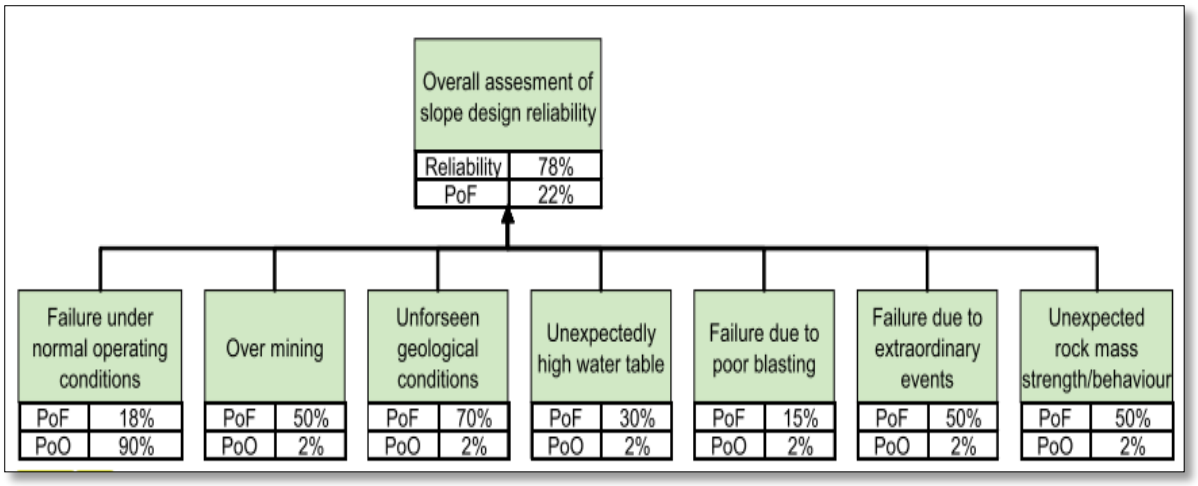

Figure 16: Fault tree analysis example (Read & Stacey, 2008)

Although the risk analysis approach addresses the drawbacks of the FoS and PoF approach, its disadvantage is that it does not provide a complete evaluation of the economic risk (Contreras, 2015).

An economic risk evaluated design methodology was described by Contreras (2015) as an attempt to avoid the drawbacks of the risk analysis approach. This methodology is based on the quantitative risk evaluation of the slopes and the construction of the risk map that relates the probability of impact to its magnitude. The summary of this methodology described by Contreras (2015) is given below and also shown in Figure 17:

- *Slope section analysis* - Slope sections are selected from the mine plan in identified necessary periods. Selection is based on higher risk areas of the mine, including locations where the likelihood of slope failure is high. Therefore, PoF values from slope models (PoFmodel) are incomplete and need adjustment.

- *Model and total PoF* - The method quantifies the contributions to the PF caused by departures from the normal conditions assumed for the design of the slopes from which a total PoF is determined.

- *Estimation of the economic impact of slope failures*- In general, the economic impact of a slope failure results from the disruption of the planned ore feed during the time required to restore the site and the additional costs caused by these activities.

- *Risk map for economic impact* - The results of the probability of failure and economic impact calculations for individual failure events are used to construct the economic risk map per year and for the mine life. The risk map defines the relationship between the probability of a particular economic impact and the magnitude. The risk map is based on the concept of event tree analysis.

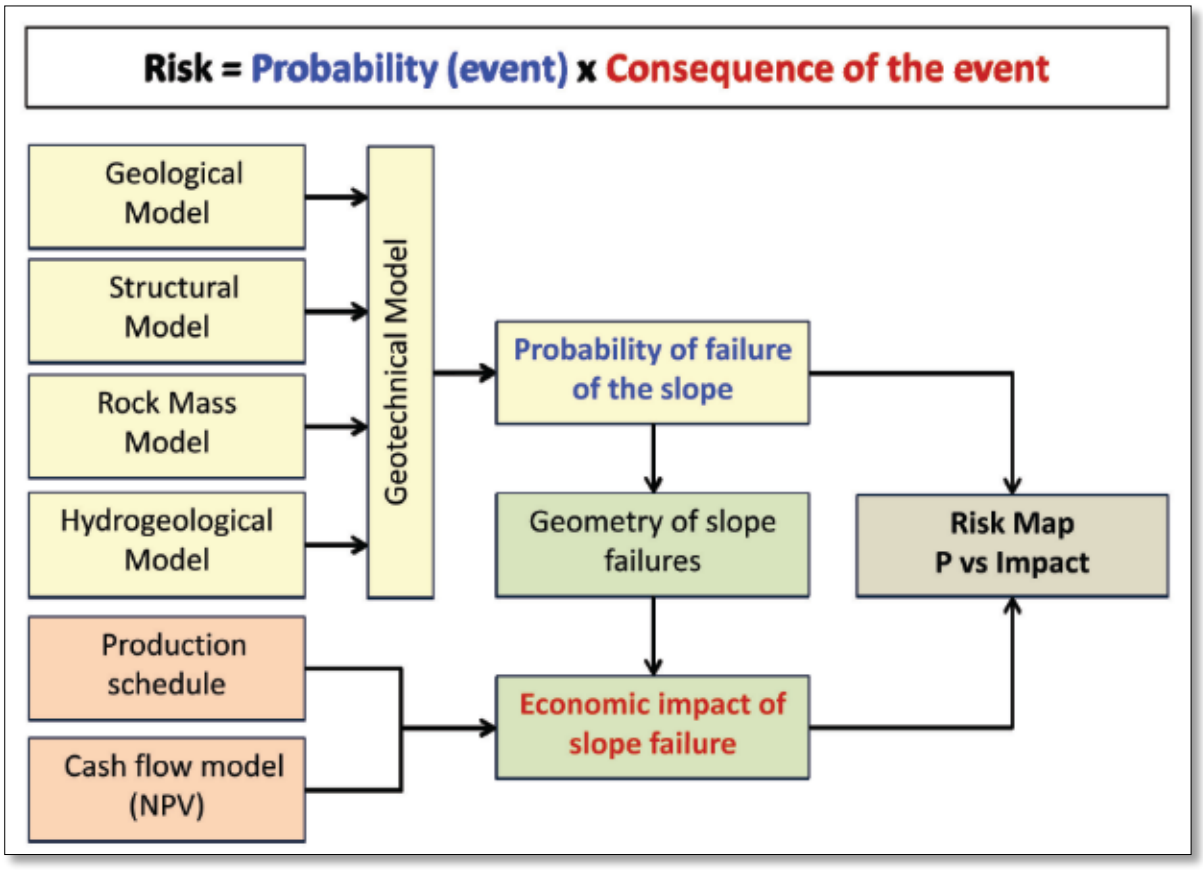

Figure 17: Economic risk-rased slope design approach (Contreras, 2015).

3 SLOPE STABILITY MODEL SETTING

It was demonstrated in section 2.3 that numerical modelling and limit equilibrium techniques are used for the evaluation of FoS and PoF, which are then used for risk evaluation of open cast slope failure. This chapter discusses limit equilibrium models and numerical models conducted using the RocScience Software Packages. In addition, material properties, model geometries, loading conditions, and all related parameters are outlined.

The same model geometry will be used for comparative results between limit equilibrium and numerical model analysis. According to Wyllie & Mah (2004), boundary conditions are essential and must be considered when creating the slope model geometry in numerical analysis. Furthermore, according to Wyllie & Mah (2004), it is required that the infinite extent of the actual problem domain is artificially truncated to include only the immediate area of interest. The typical recommendation for setting up this geometry is given in Figure 18.

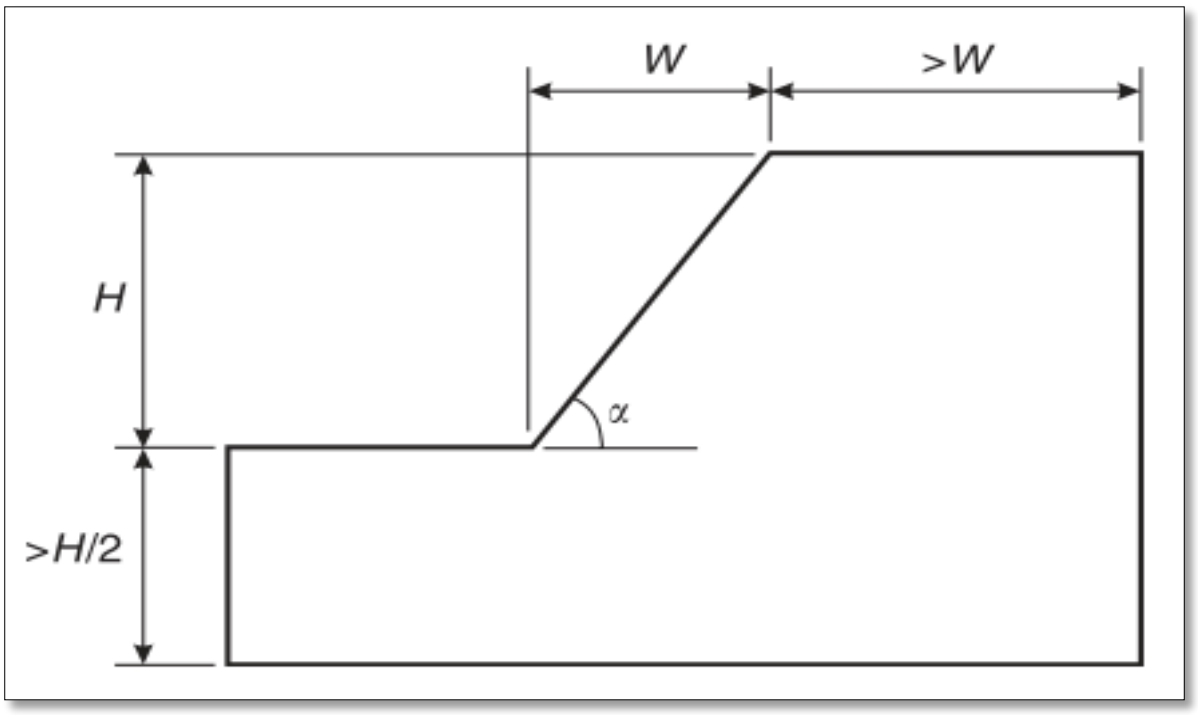

Figure 18: Recommended slope model geometry configuration (Wyllie & Mah, 2004)

In the South African coal industry, the open cast slope is typically made up of two burden benches with a height not exceeding 43m (Butcher et al., 2001). The typical bench height suggests the typical overall slope height of an open cast mine within South Africa rarely exceeds 100m. The slope geometry, which is used for both limit equilibrium and numerical model analysis, is depicted in Figure 19. This geometry shown in Figure 19 conforms to the boundary recommendations in Figure 18.

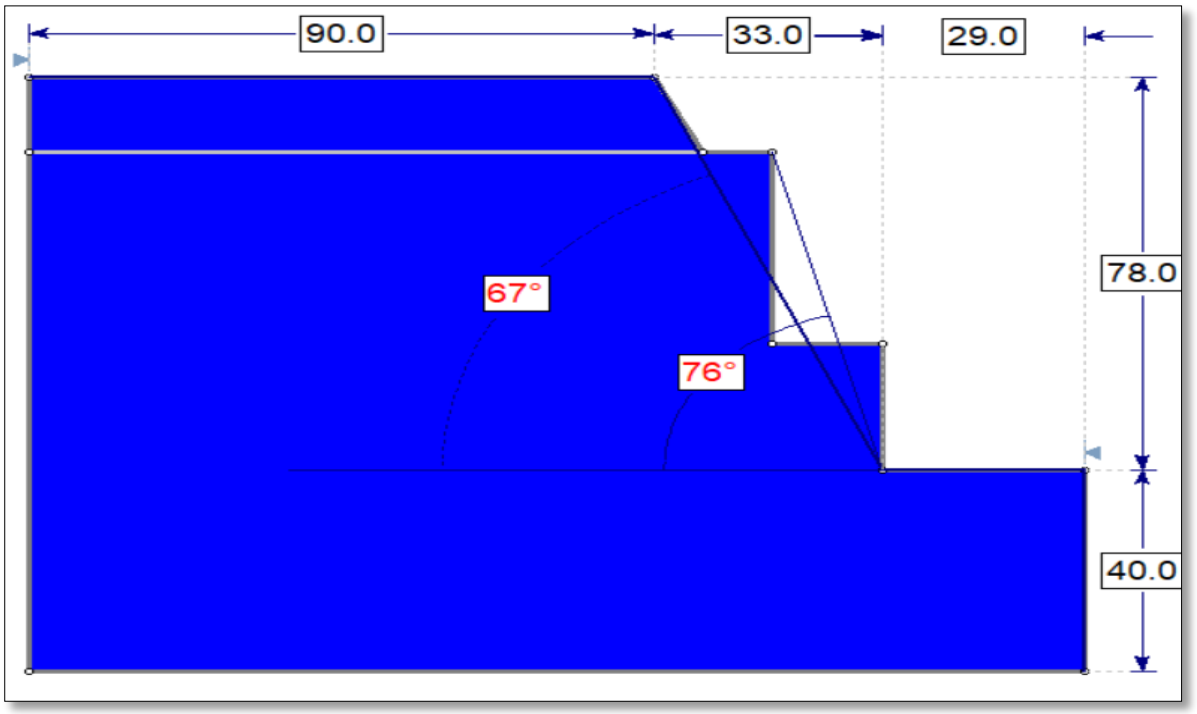

Figure 19: Hypothetical homogeneous slope model geometry.

The geometry above in Figure 19 is for a homogeneous rock mass model, which comprises only shale rock mass. Similarly, the heterogeneous rock mass model used for analysis is made from the same slope model geometry as the homogeneous one and is depicted in Figure 20.

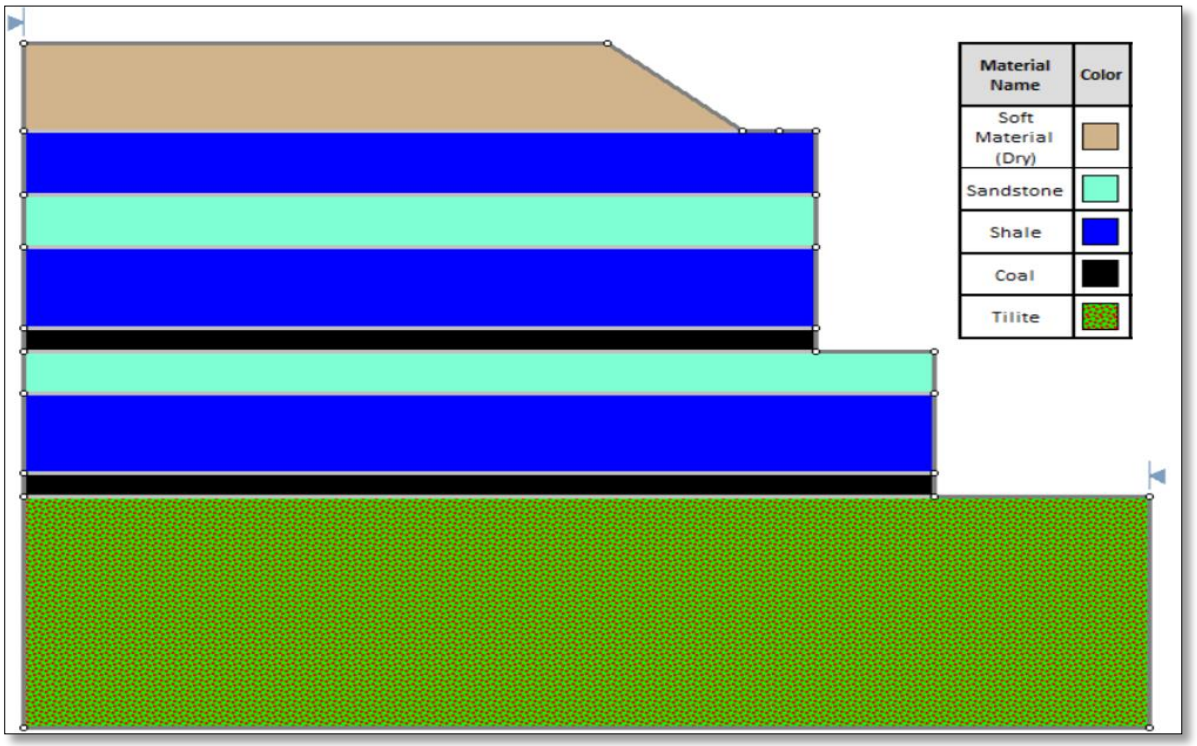

Figure 20: Hypothetical heterogeneous slope model.

3.1 Limit Equilibrium Analysis

The RocScience Slide2 software package was used to carry out limit equilibrium analyses. Slide2 is a frequently used 2D package for slope stability analysis in mining and civil projects. According to Rocscience (2021), Slide2 is used to evaluate the safety factors and/or probability of failure of both circular and non-circular slip surfaces, using vertical and non-vertical slice limit equilibrium methods.

3.1.1 General Material Properties

The analysis was initially conducted for a homogeneous rock mass used for a hypothetical model. Shale, Sandstone, and Coal will be used for the heterogeneous model as they are typically the most common sedimentary layers within the Witbank coal fields. On the other hand, the homogenous model will be made up of only Shale.

The Mohr-Coulomb and Hoek-Brown Failure criteria were used for analyses. Primary failure criterion used is H-B and the corresponding M-C inputs were determined using Rocscience Rocdata relationships. Hoek and Brown is a favoured criterion because it has proven to be the practical failure standard (Kanda, 2015). All analyses will be conducted for dry conditions and the worst disturbance factor as far as blasting is concerned. The material properties which were used are depicted in Table 6.

Table 6: Base model properties

Material Name	Hoek-Brown			Mohr-Coulomb		
	Parameter	Mean	Standard Distribution	Parameter	Mean	Standard Distribution
Shale	W(kN/m^3)	24.8	2.8	W(kN/m^3)	24.8	2.8
	UCSi (MPa)	75	13	C(kPa)	198	5.6
	GSI	50	10	Phi(°)	48.6	8.1
	mi	6	1			
	D	1				
	E (GPa)	15	2			
	v	0.2	0.1			
Sandstone	W(kN/m^3)	24.8	2.5	W(kN/m^3)	24.8	2.5
	UCSi (MPa)	75	17	C(kPa)	1132	30
	GSI	75	8	Phi(°)	63	6.3
	mi	17	4			
	D	1				
	E (GPa)	13	1.2			
	v	0.33	0.03			
Coal	W(kN/m^3)	15.1	1	W(kN/m^3)	15.1	1
	UCSi (MPa)	25	2	C(kPa)	10	2
	GSI	20	5	Phi(°)	17	1.6
	mi	3	0.6			
	D	1				
	E (GPa)	5	0.2			
	v	0.23	0.02			
Tilites	W(kN/m^3)	27	0.6	W(kN/m^3)	27	0.6
	UCSi (MPa)	175	15	C(kPa)	9180	80
	GSI	89	3	Phi(°)	62.1	3.1
	mi	17	4			
	D	1				
	E (GPa)	18	3			
	v	0.4	0.02			

Typical rock strength parameters used for Sandstone, Shale, and Coal were taken to be a good representation of strength in the Witbank Coalfields (Van der Merwe & Madden, 2003). Geertsema, (2000) recorded the rock strength properties used for Tillites.

3.1.2 Analysis Methods and Surface Options

A method of slices is used by Slide2 whereby a default number of 50 slices is employed for analysis. The default number of the slices was used because Chiwaye (2010) reported that 25 slices are sufficient to obtain an accurate solution. However, 1000 slices are preferred when analysing failure volume (Kanda, 2015). Therefore, both Janbu's simplified and Bishop's

Simplified methods will be used. This allowed for the comparison of results between methods that analyse different failure surfaces. It was mentioned in Section 2.3.2 that Janbu's method could handle non-circular failures well, whereas Bishop's method is most suited for circular failure.

Slide2 presents two options for analysis: circular and non-circular surface options. The non-circular option was selected where the path search method was used for 5000 total number of surfaces analysed. According to Chiwaye & Stacey (2010), the non-circular method is preferred.

3.1.3 Probabilistic Analysis

The probabilistic analysis was conducted on the global minimum, determined from deterministic analysis. Monte-Carlo sampling methods were used for a total of 5000 samples.

3.2 Numerical Analysis

The Rocscience RS2 software package was used as a numerical analysis tool. According to Rocscience, 2021 RS2 is a general-purpose finite element analysis program for tunnel and support design, underground excavations, surface excavations, slope stability, embankments, dynamic analysis, foundations, consolidation and groundwater seepage. As illustrated in section 2, RS2 has gained popularity in slope stability analysis.

3.2.1 General Model Setting

The material properties used were discussed in section 3.1.1 and depicted in Table 6. The following conditions which apply to RS2 were chosen:

- The number of iterations for stress analysis was left at the default setting of 500, and absolute energy was chosen as a convergence type
- The lateral boundary was fixed on the x-axis to allow deformation in the y-axis
- The bottom boundary was restricted along with both x and y directions
- The upper edge forming the slope was free from any restrictions
- A plane strain condition will also be assumed
- The field stress type was chosen to be gravity
- A six noded uniform mesh was used with an estimated 3000 elements

3.2.2 Deterministic Analysis

The RS2 program, unlike Slide2, does not compute the FoS explicitly but uses the method known as *shear strength reduction* (SSR) to determine FoS. It was stated by Satyanarayana et al. (2021) that the shear strength reduction technique is the most common method to understand the stability of slopes. SSR systematically reduces the shear strength of all material

in the model until deformations are unacceptably large or solutions do not converge (Hammah et al., 2005). SSR is taken to be equivalent to FoS. This means that the method reduces the material's shear strength up to a point where the model is unstable. The initial estimate of SRF step size was taken to be automatic with a tolerance of 0.01.

3.2.3 Probabilistic Analysis

Three methods of computing the PoF exist within RS2 package: Latin Hypercube, Monte-Carlo, and Point Estimate Method (PEM). However, RS2 running time is significantly prolonged when calculating the probability of failure using these methods within RS2. Therefore, PEM is chosen over the other two methods because it takes a relatively shorter time to calculate PoF. In addition, it was noted by Hoek (2006) that the PEM technique does not provide a complete distribution of the output variables compared with Monte-Carlo and Latin Hypercube. However, it was also stated that PEM is very simple to use, but it can be very accurate (Hammah et al., 2009).

PEM uses a series of point estimates, a point-by-point evaluation of the response function at selected values of the random input variables, to compute the moments of the response variable (Hammah et al., 2009).

3.2.4 Failure Volume

The failure surface and volume is explicitly given within Slide2 results, from which failure volume for the unit slope length normal to the model can be calculated. However, RS2 does not calculate failure volume. Therefore, to determine the failure volume with RS2, the user must estimate it by drawing a polygon along the failure surface and then exporting it to calculate volume using CAD programs. The maximum shear strain contours must be selected to get a reasonable estimate of failure volume (Chiwaye, 2010).

3.3 Computed Slide2 Models

An analysis is conducted on both homogeneous and heterogeneous models for H-B and M-C criteria and subsequent results are discussed below.

3.3.1 Sensitivity Analysis

To determine which material property influences the FoS the most, sensitivity analysis was conducted on H-B parameters and results are depicted in Figure 21. It is revealed that for a non-circular failure path, using Janbu’s simplified method, GSI is the most sensitive parameter to FoS. This suggests that it is essential to ensure that this parameter is accurate and representative of the actual field condition.

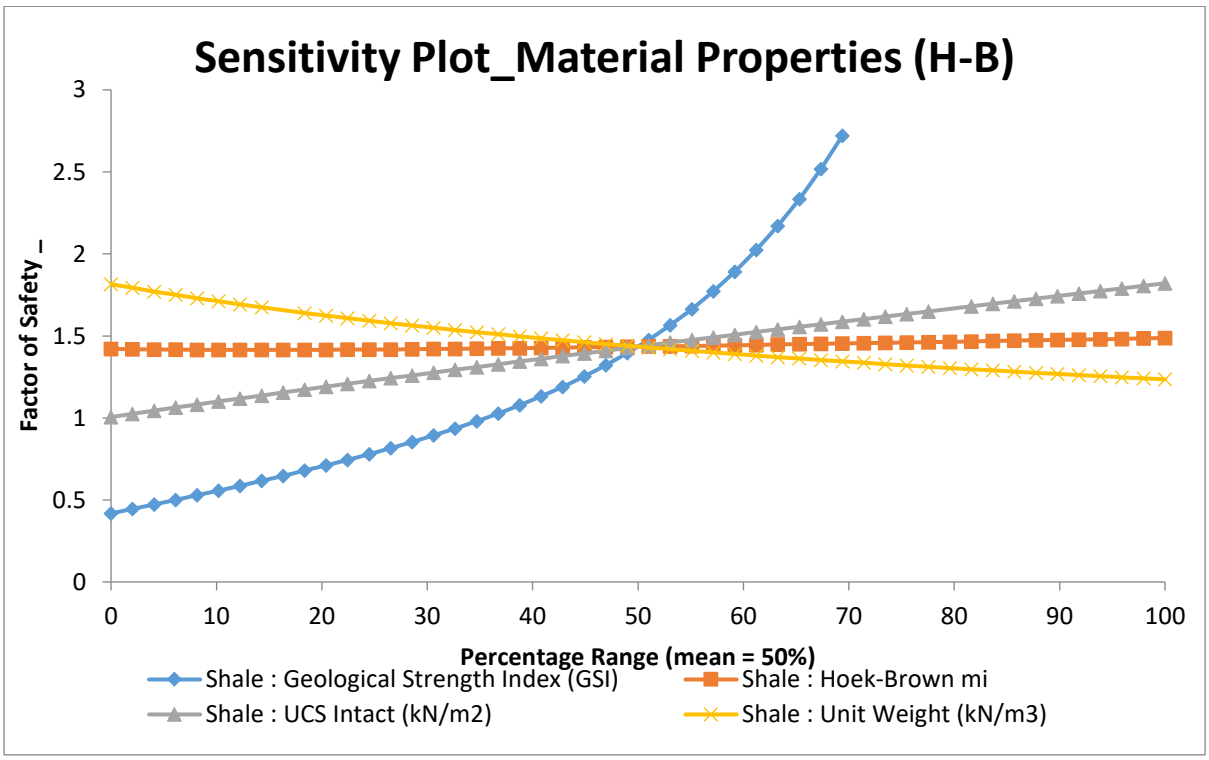

Figure 21: H-B material properties sensitivity

Similarly, Figure 22 shows the sensitivity of the M-C material properties, where it is noted that the material's friction angle and unit weight are very sensitive. Again, this means that it is essential to ensure that uncertainty is reduced when it comes to friction angle and unit weight of each material when using M-C failure criteria.

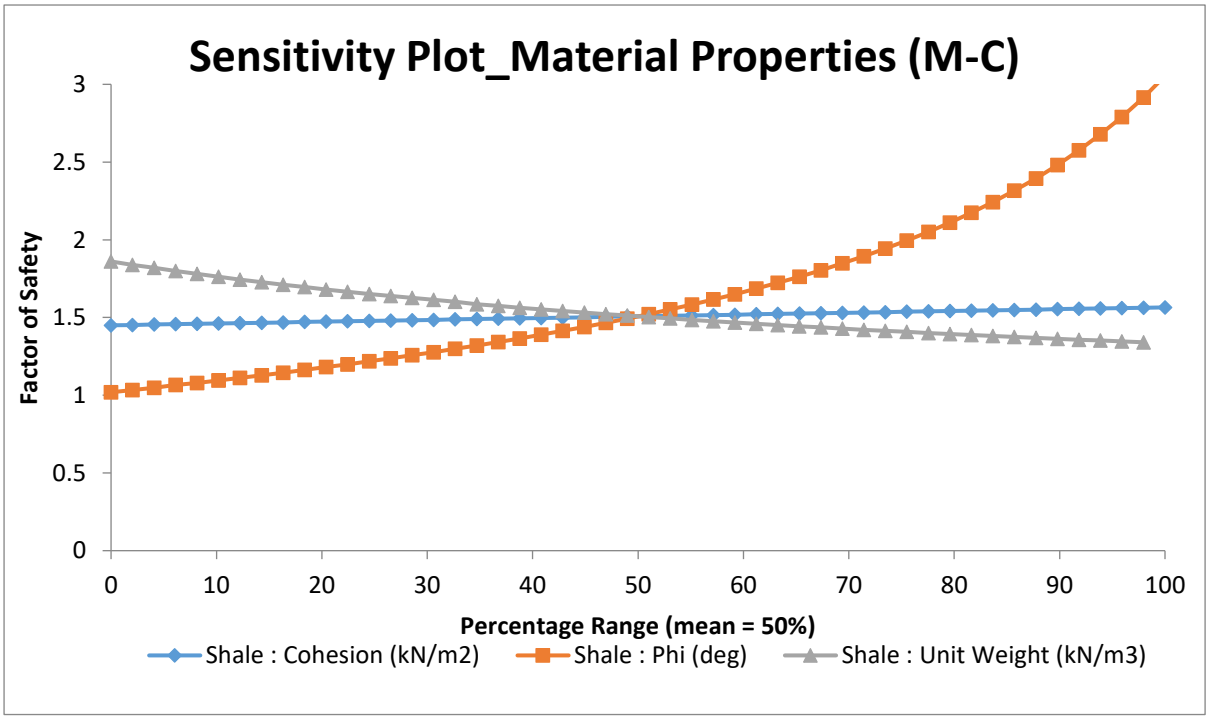

Figure 22: M-C material properties sensitivity

3.3.2 Model Conformance

The conformance report for the analyses done using Slide2 is shown in Figures 23 and 24. The figures represent homogeneous (Homo) and heterogeneous (Hete) rock mass models. Figure 23 depicts the mean FoS for various models and analysis methods conducted for comparison purposes plotted against several samples or iterations. It can be noted from Figure 23 that there is no benefit to the quality of the FoS results by increasing the number of samples to be analysed beyond 1000 samples.

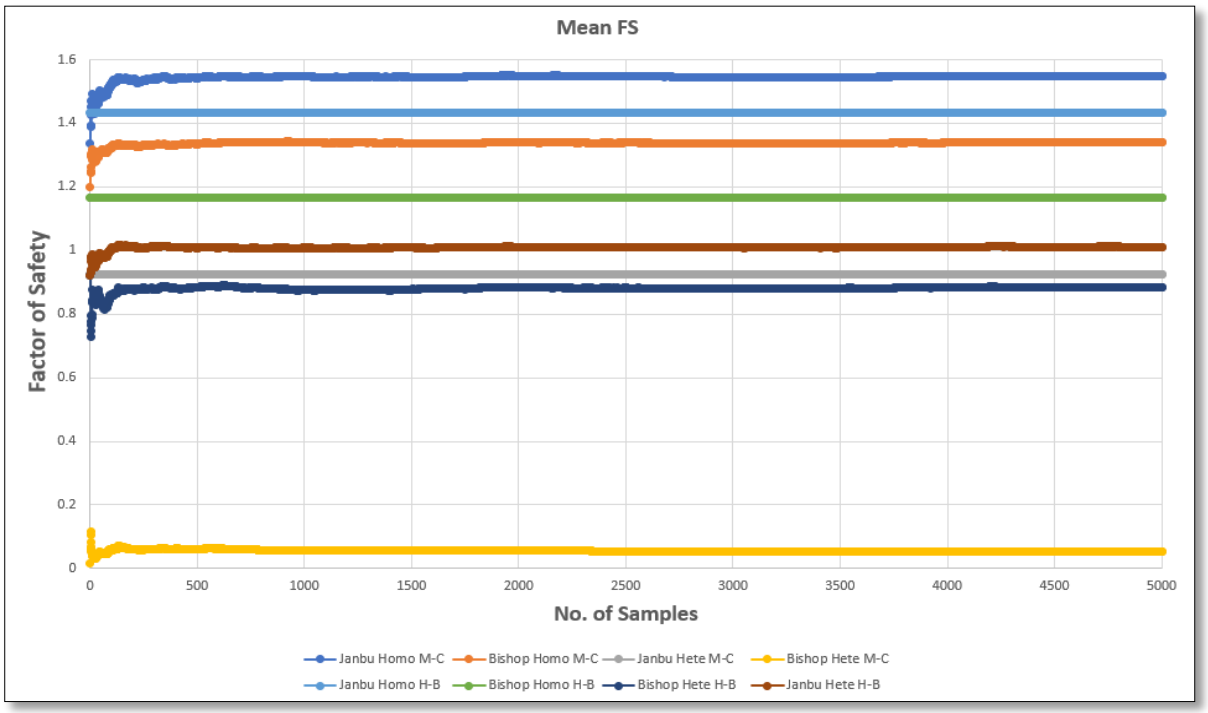

Figure 23: Mean Factor of Safety

Similarly, Figure 24 shows the mean PoF for various models and analysis methods conducted for comparison purposes plotted against a number of samples or iterations. This graph indicates that no value is added by increasing the number of samples analysed beyond 1000 samples. Increasing this number also increases the analysis's running time.

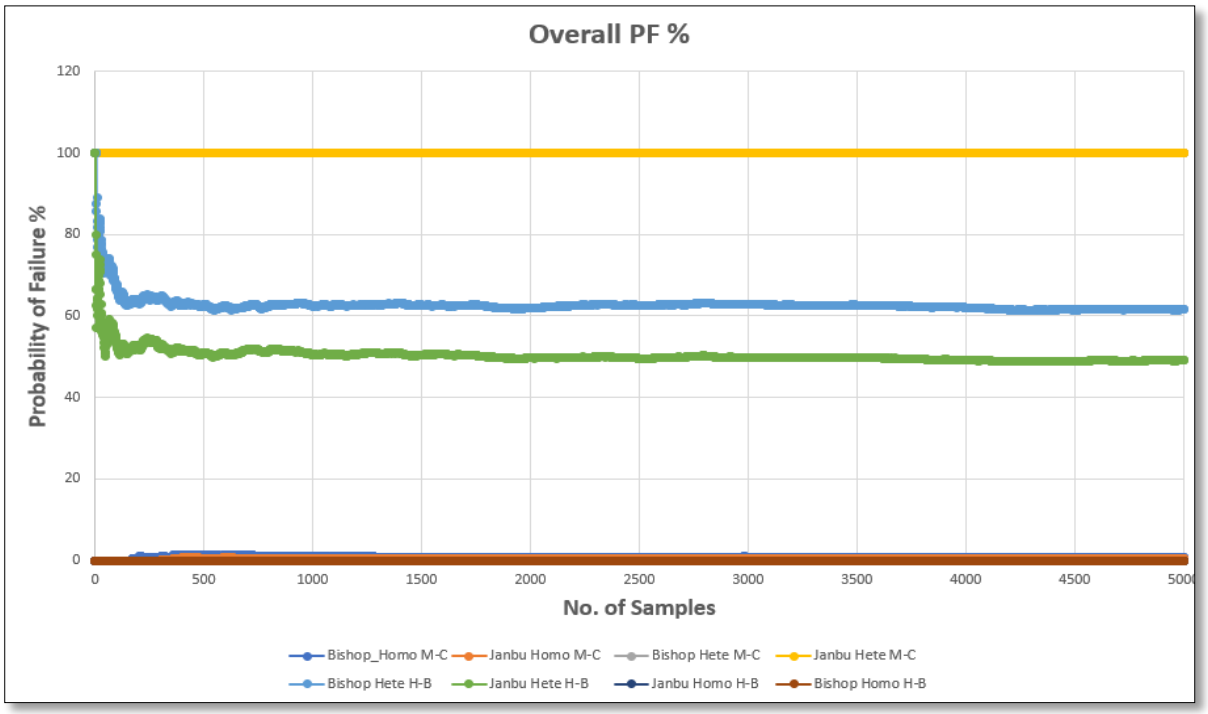

Figure 24: Overall PoF

3.3.3 LE Slide2 Results

The results for the two analysed rock mass models are depicted in Table 7. Both the H-B and M-C failure criteria were analysed by Bishop's Simplified and Janbu's Simplified method on each rock mass model. It is noted in this analysis that Bishop's method poorly estimates failure surface and volume in a heterogenous rock mass. The results reveal the following:

- The M-C criterion provides a higher overall mean FoS than the H-B criterion for heterogeneous and homogeneous rock mass models
- M-C provides a higher volume of failure material than the H-B criterion
- Bishop's Simplified method underestimates the failure volume compared to Janbu's method. This is more pronounced in the heterogeneous model since the failure is generally not circular
- Both Bishop's simplified methods underestimated the heterogeneous model's stability for M-C and H-B criteria since the most critical failure path was on the weak layer or soft material for the heterogeneous model
- Generally, it is observed that Bishop's Simplified method underestimates the failure volume

Table 7: Slide2 results summary

Category		Method	Overall Mean FoS	Overall PoF (%)	Critical FoS	Critical PoF (%)	Critical Failure Volume
H-B	Homo	Janbu	1.4	0.0	1.4	0.0	0.0
		Bishop	1.2	0.0	1.2	0.0	0.0
	Hete	Janbu	1.0	49.0	1.0	46.4	91.0
		Bishop	0.9	61.6	1.0	46.9	91.0
M-C	Homo	Janbu	1.6	0.4	1.6	0.3	1583.0
		Bishop	1.3	0.8	1.6	0.6	1404.0
	Hete	Janbu	1.2	0.0	2.3	0.0	0.0
		Bishop	0.0	100.0	0.1	100.0	52.0

3.4 Computed RS2 Models

An analysis is conducted on both homogeneous and heterogeneous models using only Hoek-Brown failure criteria. The results will be discussed in section 3.4.1 and 3.4.2.

3.4.1 Numerical Analysis RS2 Results

Through running RS2 analysis, it was observed that computational time for the statistical analysis and calculating probability of failure is significantly prolonged. This is because calculating the probability of failure for the heterogeneous rock mass model using 25 000 elements took more than 30 hours. Due to the increased running time, only Hoek and Brown criteria were used for the analysis. Table 8 gives a summary of results where the following is observed:

- The mean critical SRF decreases with an increased number of mesh elements, whereas the volume of failure material is increased with an increasing number of elements; and
- Although the probability of failure for both heterogeneous and homogeneous rock mass models is 0%, as expected, the mean critical SRF for the heterogeneous model is higher than for the homogeneous model. This is due to the strong sandstone layer in the heterogeneous rock mass model.

Table 8: RS2 result summary

Category	No. of Elements	Mean Critical SRF	PF %	Volume per unit length
H-B (Homo)	1500	3.1	0	0
	3000	3.0	0	0
	5000	3.0	0	0
	25000	3.0	0	0
H-B (Hete)	1500	5.4	0	0
	3000	5.2	0	0
	5000	5.1	0	0
	25000	4.6	0	0

3.4.2 Sensitivity Analysis

An analysis was conducted to determine how sensitive the numerical model is toward the k-ratio and locked in stress, which are both not accounted for in the limit equilibrium model. In addition, an investigation was conducted to determine the mean critical SRF to k-ratio and locked in stress. Figure 25 shows the k-ratio sensitivity plot, run for 1500 mesh elements. It can be noted from this sensitivity plot that as k-ratio increases and decreases above and below a value of 1, the SRF decreases.

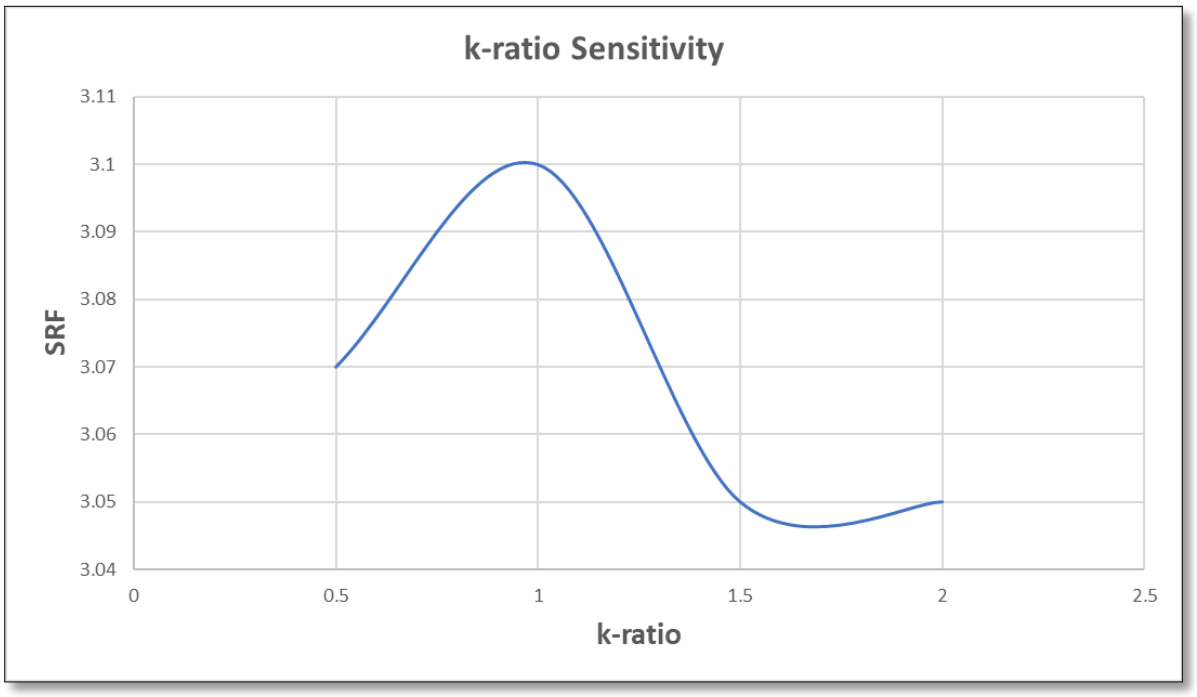

Figure 25: k-ratio sensitivity plot

Figure 26 shows the sensitivity of the locked-in-stress (in-plane) for 1500 elements for a k-ratio of 1. It can be seen in both Figures 25 and 26 that SRF is sensitive to a certain degree to both k-ratio and locked in stress.

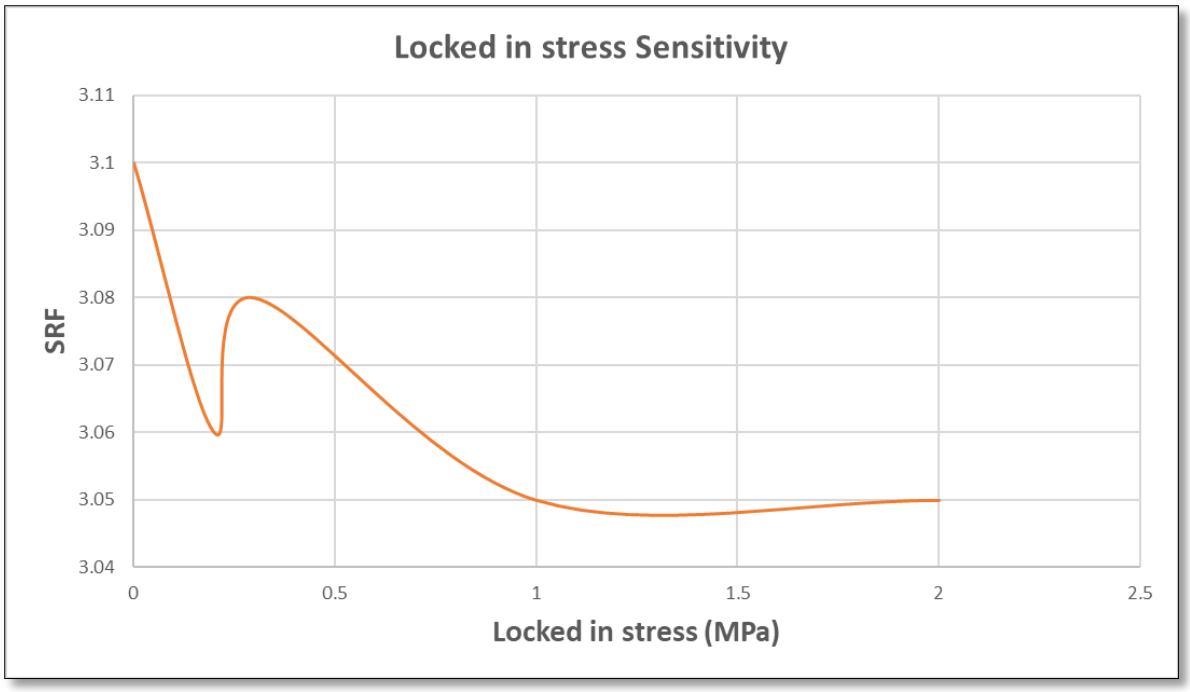

Figure 26: Locked in stress sensitivity plot

4 CASE STUDY: KLIPFONTEIN EAST PITS, MIDDELBURG MINE SERVICES

4.1 Introduction

The findings of the hypothetical models validated in section 3 will be applied to a case study for a mining project. Klipfontein East (KE) Pit is a mining project aimed at extending the life of Middelburg Mine Services (MMS). Deterministic and probabilistic analyses will be performed using the LE program Slide2 and NM program RS2. Furthermore, risk analysis is performed from the resulting PoF from both LE and NM analysis. A complete economic evaluation will then be conducted on the design.

4.2 Locality plan

MMS is in the Witbank Coalfield, historically one of the crucial coalfields for South African power generation. It was stated that more than 70% of coal reserves in South Africa are found in the Witbank, Highveld and Waterberg Coalfields (Jeffrey, 2005). The colliery is 100% owned by Seriti Power (Pty) Ltd (formerly South32 SA Coal Holdings Proprietary Limited). MMS is in the Mpumalanga province of South Africa between Middelburg and Witbank (See Figure 27). The coal production from MMS is of national interest because it is supplied to the Eskom Duvha Power Station, adjacent to the mine lease area.

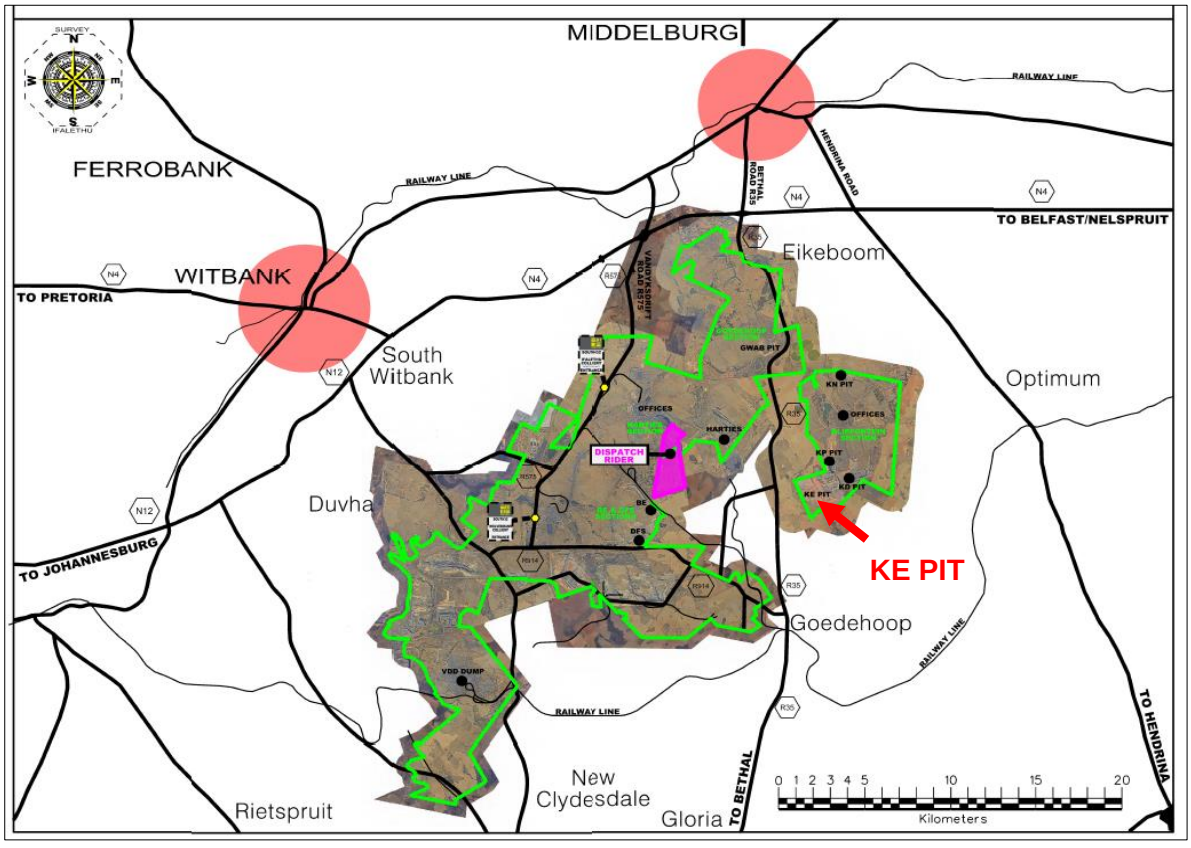

Figure 27: MMS locality plan, Microstation plan

The sedimentary coal-bearing strata in the Witbank Coalfield are contained within the Vryheid Formation of the Ecca Group of the Karoo Basin (Middelburg Mine Services, 2019). The colliery employs open cast strip mining methods to exploit the coal reserves. This colliery makes use of both dragline and truck and shovel operations. KE Pit, which is the life-extension project of MMS, is situated within the eastern part of the mine boundary. KE Pit project area is depicted in Figure 28.

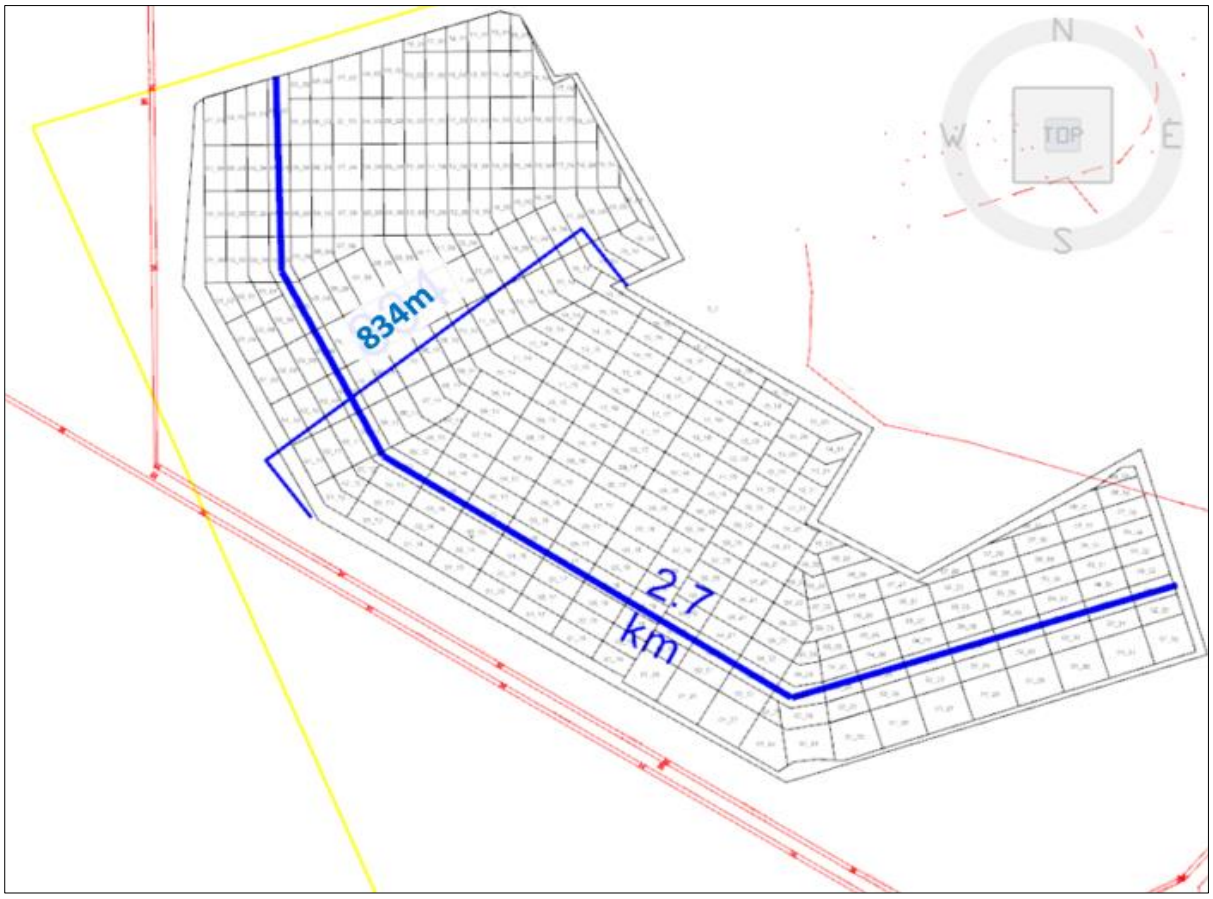

Figure 28: KE pit layout_Deswik plan

4.3 KE Project Background

The KE mining project comprises a strip-mining layout 2.7km in length and 834m in width. Two economic coal seams are planned to be mined, #4L Seam and #2 Seam, with an average width of 4m. The planned mining method for this project is the open cast strip method, where the blasted overburden is removed by dragline (DL), and dozers remove some material. Draglines remove the burden, exposing coal on one strip and then dump the material on the previous strip, creating spoil pile heaps. The exposed coal is then drilled and blasted before it is loaded by front end loaders and hauled by a fleet of dump trucks to the stockpiles or crusher. Table 9 depicts the summary of the fleet of the main equipment to be used at KE Pit, and Figure 29 shows the dragline to be used at KE Pit.

Table 9: Main mining equipment used at Klipfontein

Activity	Equipment	Make
Burden Movers	Dragline (DL)	Bucyrus Erie 1570
	Dozers	CAT D11, CAT D10
Loaders	Front End Loader (FEL)	CAT 994, CAT 995
Haulers	Dump Trucks	CAT 785, CAT 777
Drillers	Blast hole Drills	Atlas Copco, Pit Viper 271

Figure 29: Dragline used at Klipfontein

In the planning stage of Klipfontein East Pit, the slope and the pit configuration were designed based on design engineers' experience and operational rules of design as recorded in the Code of Practice to Combat Rock Falls and Slope Instabilities COP_010. According to an internal report, SERITI-MMS, 2021a, the main design requirements are as follows:

- Soft material must be pre-stripped before drilling of burden and leave not more than 3m of softs over competent burden. This material will then be trucked out and dumped in a designated Waste Dump;
- A smooth wall blasting technique (pre-splitting) must be practised on all burden made of competent rock mass;
- The catchment width between #2 Seam and #4 Seam must be at least 16m; and
- Spoil piles must be placed at least 5m away from the edge of the active strip.

The slope configurations within KE Pit were designed to meet the above requirements and are shown in Figures 30 and 31.

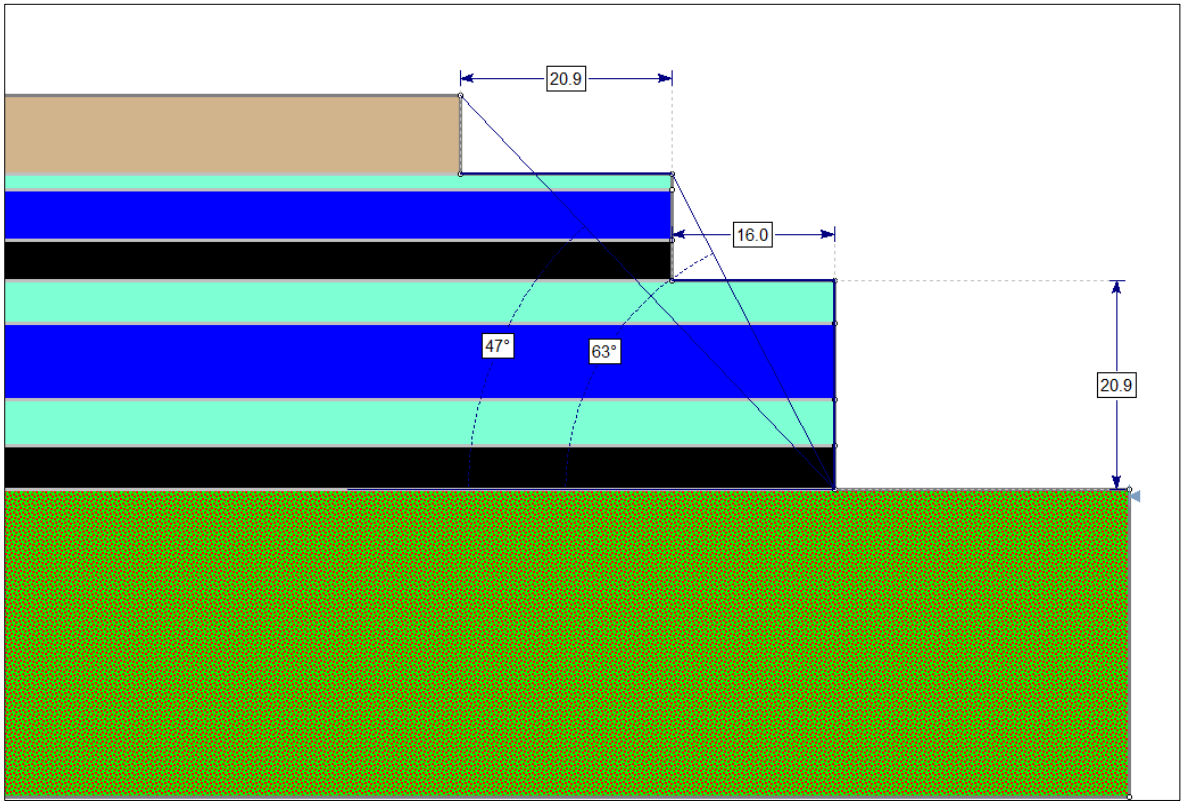

Figure 30: KE HW slope configuration

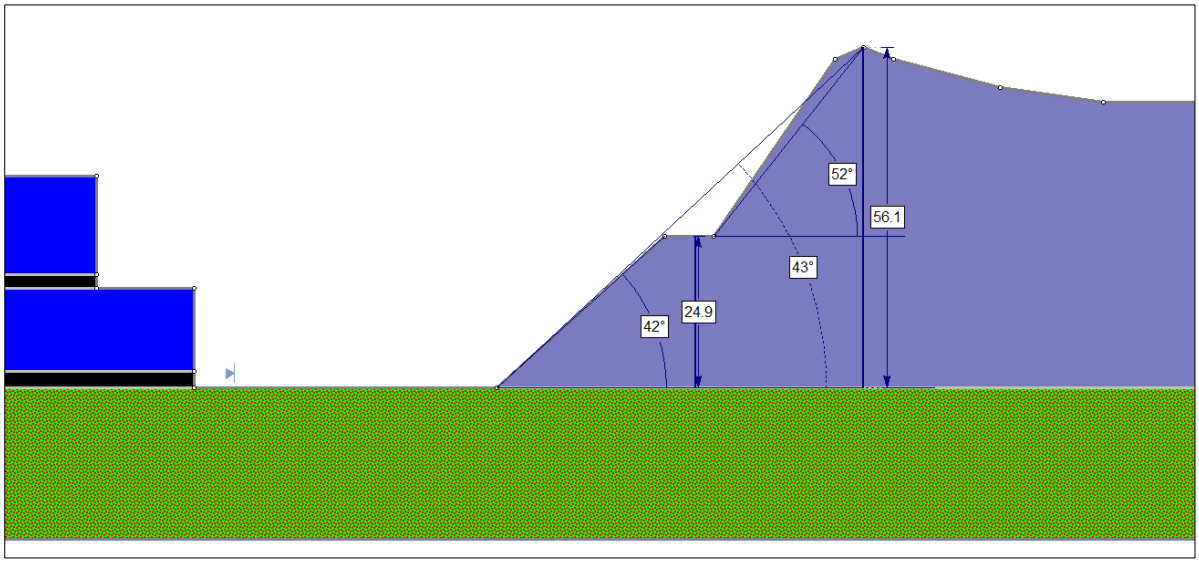

Figure 31: KE spoil slope configuration

4.4 Geotechnical Database

As part of the KE project, the geotechnical database within MMS was reviewed to determine whether an opportunity exists to improve this project's design. The findings from this database are briefly reviewed in this section.

Soft Material Strength

The strength of soft material or residual rock was historically never measured within MMS. Instead, the depth of soft material was historically estimated from the exploration core borehole. The length of the core, which cannot be recovered from the collar of the borehole, is then regarded as the depth of soft material. Figure 32 depicts the thickness contour plan for soft material.

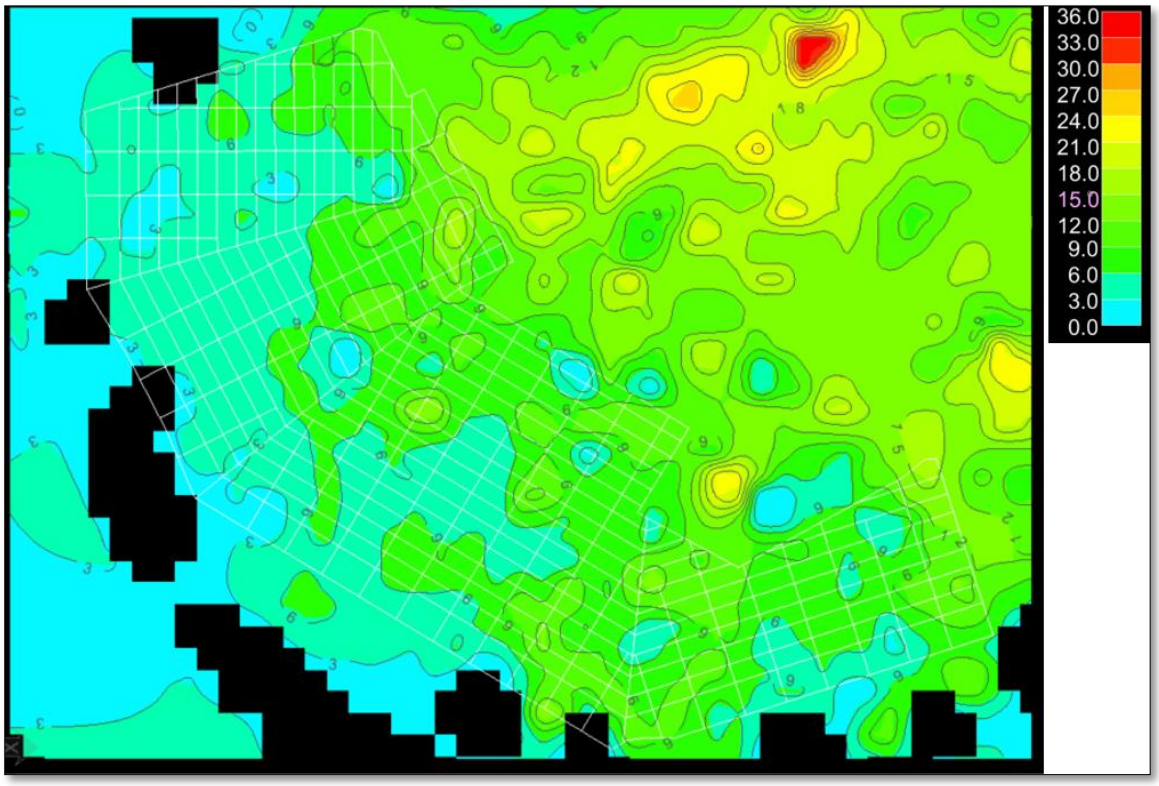

Figure 32: Soft thickness estimation_ KE

Rock Strength

Laboratory testing of rock core samples is conducted to determine in-situ rock strength. Geotechnical database contained laboratory test results for core samples drilled within MMS over the years. A summary of laboratory results for the 187 rock samples for core drilled within MMS is shown in Table 10. It is worth noting that none of the core was taken from within the KE project area.

Table 10: MMS rock strength summary

Rock Type	Density(t/m^3)		UCS(MPa)		Elastic Modulus (GPa)		Poisson's Ratio		Brazilian Tensile (MPa)	
	Mean	Std. Dev	Mean	Std. Dev	Mean	Std. Dev	Mean	Std. Dev	Mean	Std. Dev
Shale	2.44	0.12	79.54	18.36	11.72	5.19	0.25	0.04	5.61	2.46
Shaley Sandstone	2.53	0.11	89.71	32.03	14.96	9.04	0.29	0.08	7.75	2.97
Sandstone	2.41	0.1	75.42	30.23	20.03	10.23	0.36	0.1	5.75	2.63
laminated	2.5	0.08	89	22.22	13.6	7.61	0.27	0.07	7.51	2.33

Major Joint Sets

Joints were historically mapped on exposed highwalls, and joint orientation data were recorded within the database. This database contains only information from the northern part of MMS, which the KE Project is part of. Figure 33 below depicts the major joint set orientations and friction angle distribution.

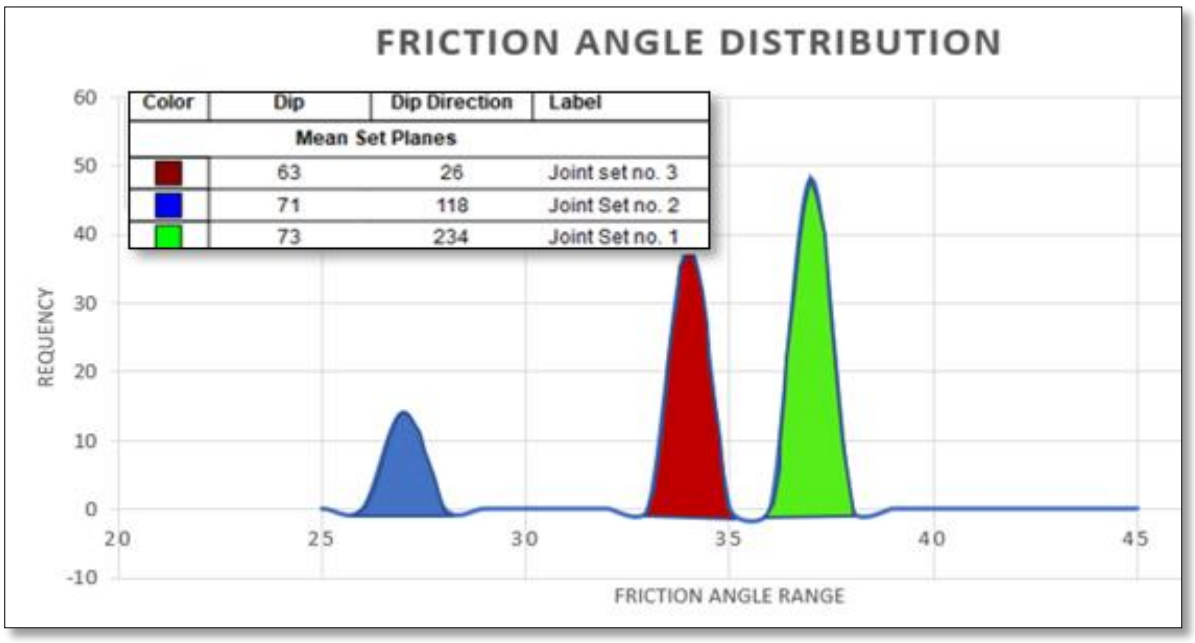

Figure 33: Major joint set data

Rock Mass Strength

Rock mass classification has gained application in rock mechanics as input to defining the strength of rock mass. Rock mass classification, such as the Geological Strength Index (GSI), has gained wide acceptance as a tool for estimating the strength of rock masses which are jointed (Hoek & Brown, 2019). GSI is known to be a very useful input parameter for numerical modelling. However, the geotechnical database within the MMS does not have any rock mass classification data recorded.

4.5 Data Collection

Like any other mining slope, KE slope design takes place in a natural geological material that can never be altered. Therefore, the design engineer must fit the design within the geological environment because this environment can never be modified to fit in the design. It was demonstrated in Section 4.4 of this report that the strength of soft material and rock mass classification has never been measured within MMS. Justification for not measuring soft material strength has been that all soft material is removed from both the highwall and spoil

slopes. However, no explanation can be made for not collecting rock mass classification data, mainly because it was revealed in Section 3.3.1 that GSI is the most sensitive parameter when in H-B criteria. Therefore, a data collection project was instituted to minimise the epistemic uncertainty in the design described in Section 2.2.3.

4.5.1 Soft Material Strength

The investigation to collect soft material or residual rock strength properties included conducting a field Standard Penetration Test (SPT) using a Dynamic Probe Super Heavy Test known as DPSH tests. DPSH involved forcing a cone probe into the ground using a drop hammer and then calculating the number of blows per 100mm. The DPSH test is continuous and is performed from the surface until refusal. It was stated by Byrne & Berry (2008) that, penetration tests are relatively inexpensive and rapid as no core or sample is recovered. Cone penetration testing is a procedure where a conical-shaped probe is pushed into the ground and penetration resistance recorded (Grider et al., 2003).

A geotechnical contractor was appointed to conduct these field tests using a TG63-100 automatic Pagani Rig with an 80kg drop hammer; see Figure 34. The hammer was dropped on each hole tested until there was refusal.

Figure 34: DPSH drill rig

A total of 33 DPSH tests were conducted on the undisturbed ground within the KE Pit project area (See Figure 35). The number of blows (hammer drops) was correlated with the material's load-bearing capacity during the tests: the higher the number of blows, the higher the bearing capacity. Load Bearing Capacity is a concept used to understand foundation strength in construction and the function of the effective friction angle of the soil material (Grider et al., 2003).

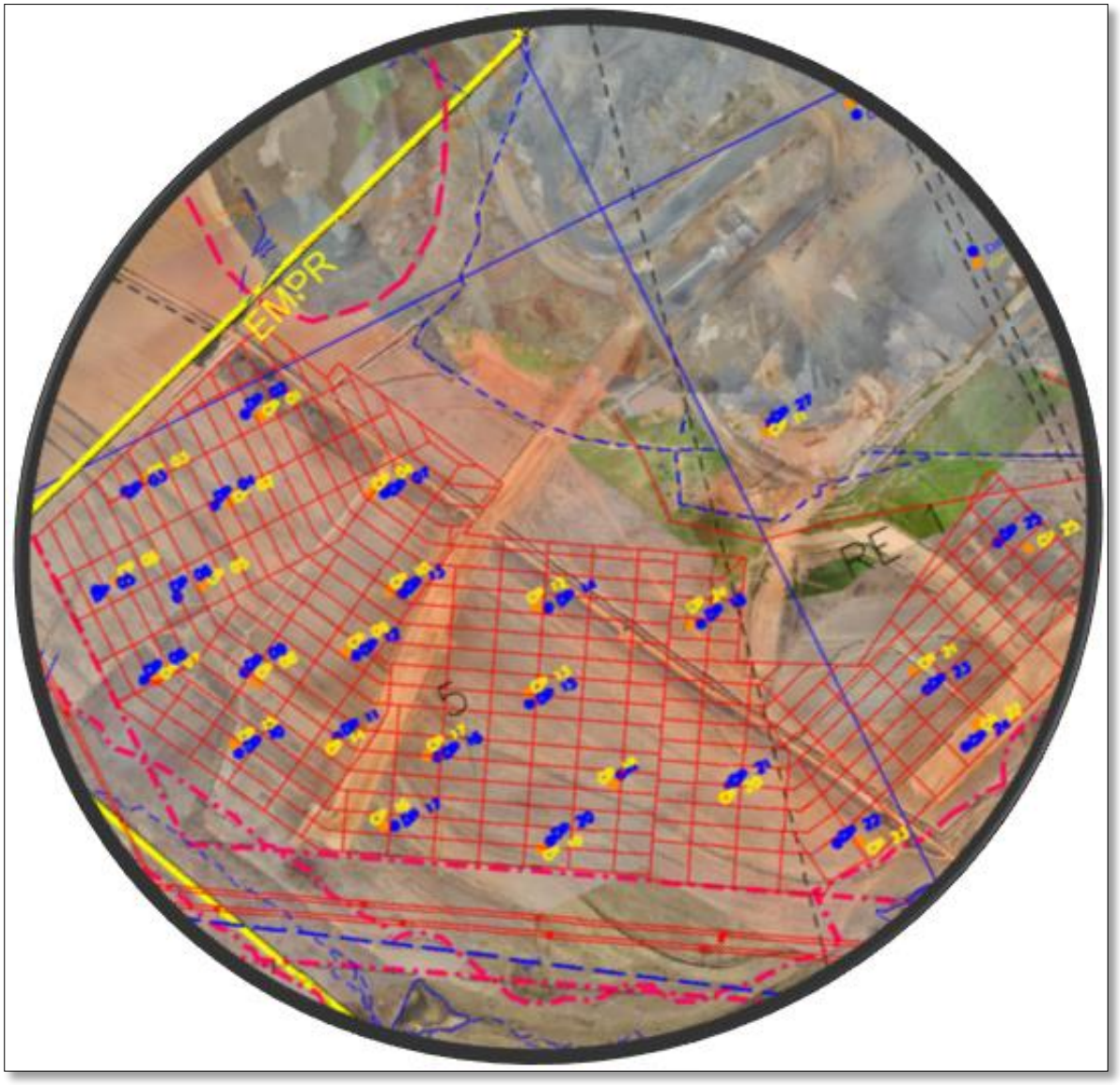

Figure 35: Field test location_KE Pit

Investigation results

The results from the investigation conducted were analysed, and it was noted that the refusal depth throughout the project area is 6.8m at a 90% probability of occurrence. The refusal depth does not indicate the depth of soft material, but the depth of material with strength is low enough

to be penetrated by the DPSH cone. This means that at a depth below 6.8m, the residual rock strength is much higher.

Figure 36 shows the depth horizon distribution within soft material with a load-bearing capacity of 200kPa. It can be noted from this figure that at this strength, there is a 90% probability that the depth horizon will be less than 4.3m. To make a good comparison and understand what this load-bearing capacity means, it was compared against the dragline pad demand. The weight of the dragline used at Klipfontein pits is distributed through a circular tub with an 18m diameter to give stress equal to 140kPa. This means that an FoS for the dragline pad made of this in-situ material will be **1.4**.

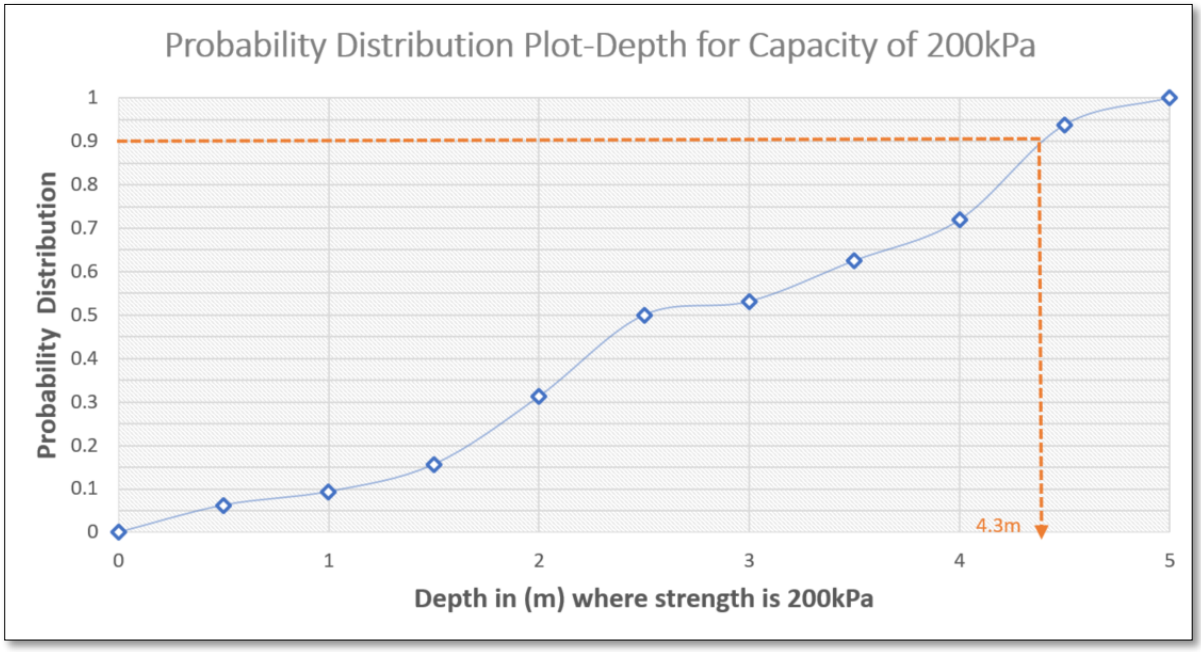

Figure 36: Distribution of depth where bearing capacity_ 200kPa

Figure 37 shows the distribution of a depth horizon within soft material with a load-bearing capacity of 300kPa. It can be noted from this figure that at this strength, there is a 90% probability that the depth horizon will be less than 5.8m. This strength translates to a dragline pad FoS of **2.1**.

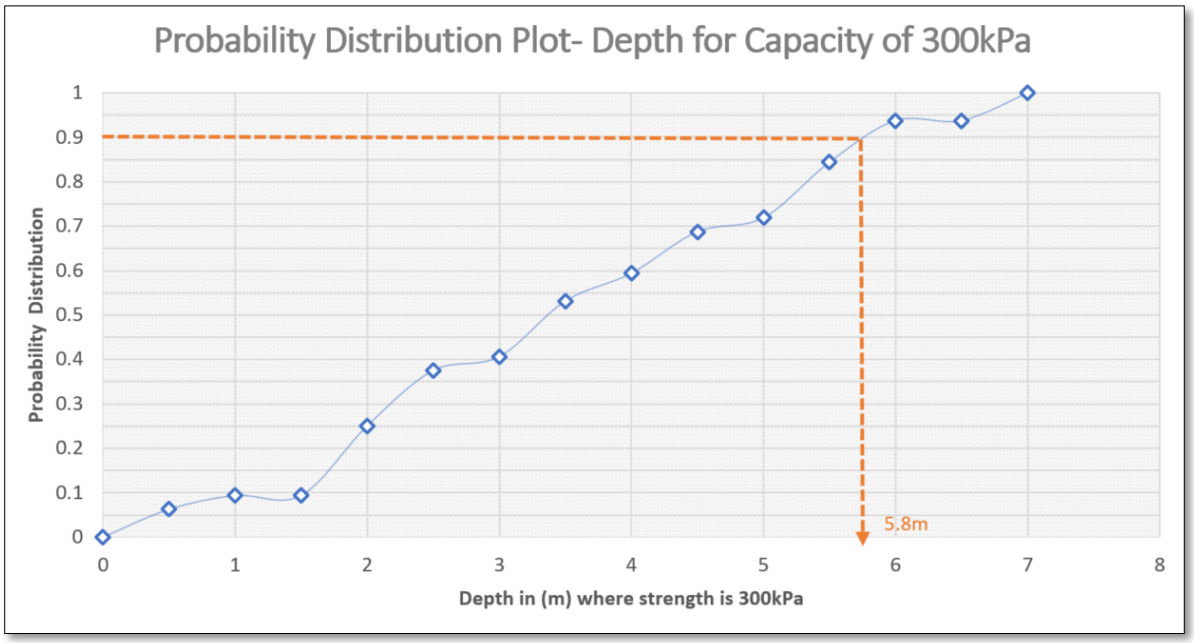

Figure 37: Distribution of depth where load-bearing capacity is 300kPa

A contour plot of the distribution of depth of soft material on the horizon where the load-bearing capacity is 300kPa is indicated in Figure 38. This depth also includes topsoil material thickness between1.5m and 2m. Topsoil material is ordinarily removed before mining and preserved for rehabilitation purposes.

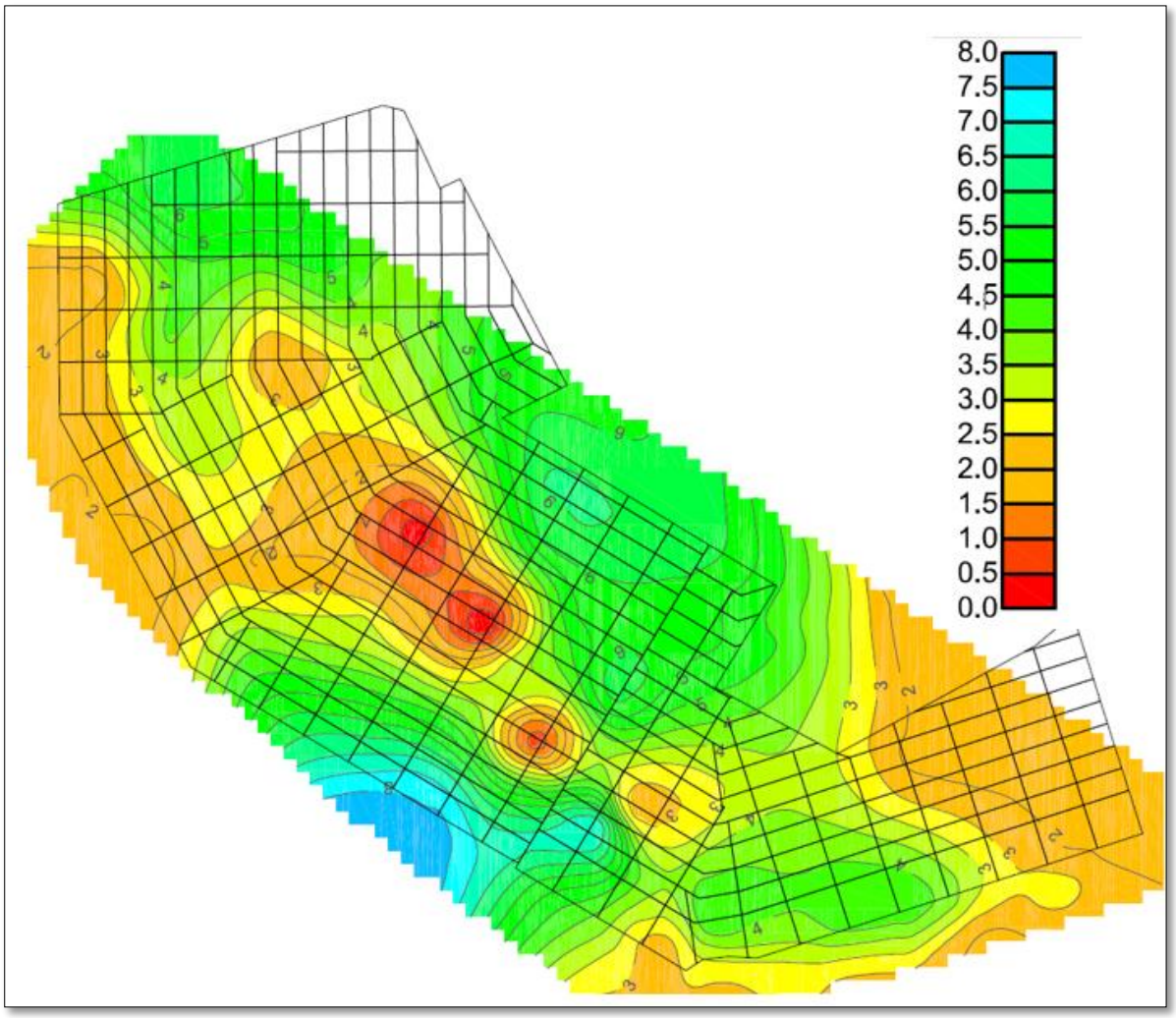

Figure 38: Contour plot depth for 300kPa Horizon

Application of Collected Data

The results described in this section indicate that the soft material (or residual rock) strength at KE Pit is relatively high at shallow depth. It is envisaged that based on this, the soft part of the slope will be stable for the short life of the pit and as a result, only topsoil must be pre-stripped. Figure 39 show a hazard plan generated for each mining block where the strength of 300kPa was targeted. All areas where the depth of targeted strength is less than 3m and greater than 5m were hatched green and red, respectively.

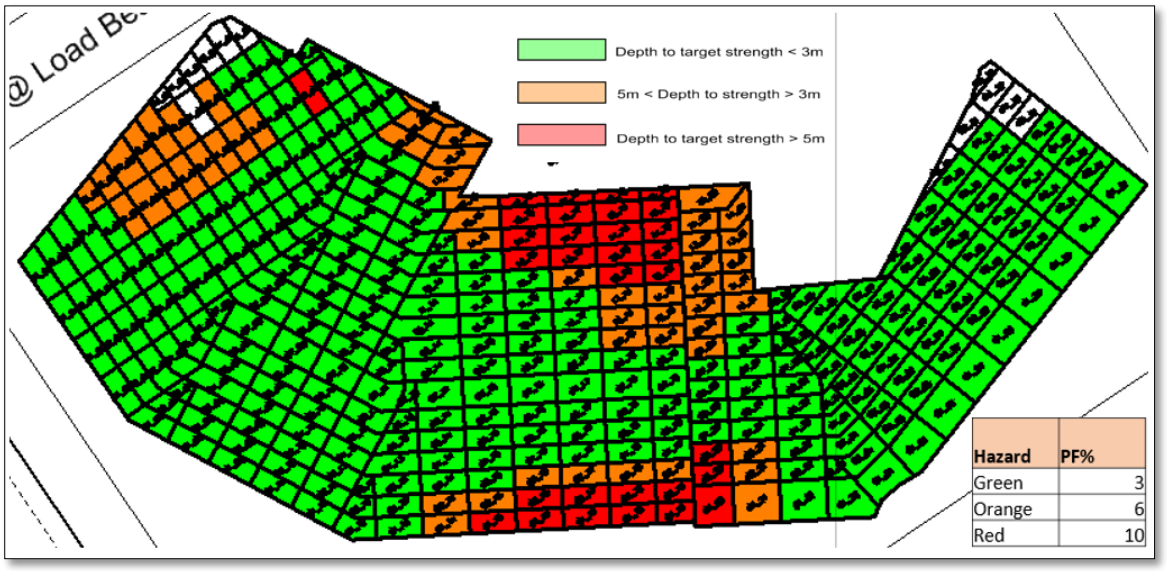

Figure 39: Soft material hazard plan

The probabilities of not achieving the 300kPa load-bearing capacity within the KE pit are summarised and depicted in Figure 39.

4.5.2 Photogrammetric Mapping

MMS acquired a Maptek Laser Scanner commissioned for use in August 2021. The laser scanner will implement a photogrammetric mapping program and a passive movement monitoring program. Mapping will be conducted on every exposed highwall during the life of the KE Pit, where mapping data will be used for rock mass classification and joint kinematic analysis. However, no mapping has yet been conducted at the KE pit since there is not yet an exposed HW. Furthermore, KE Pit is currently mining within the box cut where the HW is buffered and is not available for meaningful photogrammetric mapping. Therefore, the photogrammetric mapping will be initiated when the highwall is exposed to improve the geotechnical model.

4.6 Design Model Analysis

The approach described by Read & Stacey, 2008 was adopted (see Table 11). Therefore, based on the information available during this research project, the KE geotechnical model is classified to be in the boundary between Design and Construction and the Feasibility model. This is based on the following:

- Field testing was conducted within site, and laboratory test data of samples collected within Klipfontein was used.
- Detailed mapping is not done at the site, but the general joint data orientation used is from the same operation, although in different locations.

- Specific rock mass parameters are required to validate the rock mass strength not obtained from the site.

Table 11: Model confidence level

Model Category	Variation	Basis
Conceptual	≥ 50%	Data inferred reports and regional data from other mines in similar environments
Pre-feasibility	30-50%	Experience in similar mines, but using data inferred from other mines. Limited borehole data, mapping on proposed sites.
Feasibility	15-35%	Based on increased density of sampling during pre-feasibility
Design and Construction	10-20%	Based on specific data obtained to validate data used in the pre-feasibility stage. Detailed mapping, field testing and laboratory testing
Operational	<10%	Additional mapping on exposed walls, performance monitoring data

4.6.1 Limit Equilibrium and Numerical Design Analysis

The design analysis conducted is based on the *Feasibility-Design Model* variability. Both limit equilibrium and numerical tools were used for deterministic analysis, but only limit equilibrium was used for probabilistic analysis.

LE Analysis

LE program Slide2 was the favoured tool for analysing slope stability due to favourable run times. The Slide2 model comprised 1000 samples, each with 1000 slices, and a non-circular path was analysed for 5000 iterations using a Janbu Simplified method. The analysis was conducted on two slope configurations, 1) The original KE configuration based on operation rules and 2) the configuration where the catchment on the original KE design was changed from 16m to 10m. The results of the analysis are shown in Table 12.

Numerical analysis

A deterministic numerical analysis was carried out only for a 10m catchment width rock mass model. This was because of the prolonged model run time required for numerical analysis. The

model contained 100 000 mesh elements, and the maximum number of iterations was 500. Results for this model are also shown in Table 12.

Table 12: LE and NM analysis

Configuration	Overall Mean FoS	Critical SRF	Overall PoF (%)	Slide2 Volume (m^3)	RS2 Volume(m^3)
16m Catch	2.7	NA	0.0	NA	0.0
10m Catch	2.0	2.1	0.3	542.0	480.0
Spoil 16m Catch	1.1	NA	9.7	NA	414.0
Spoil 10m Catch	1.0	NA	18.2	NA	414.0

4.6.2 Kinematic Analysis

To conduct kinematic analyses, the KE project pit layout was divided into four zones according to the orientation of the slope (see Figure 40). The kinematic allows the identification of potential failure mechanisms for a proposed or an existing slope orientation. Both deterministic and probabilistic kinematic analysis was conducted, and the result is shown in Table 13.

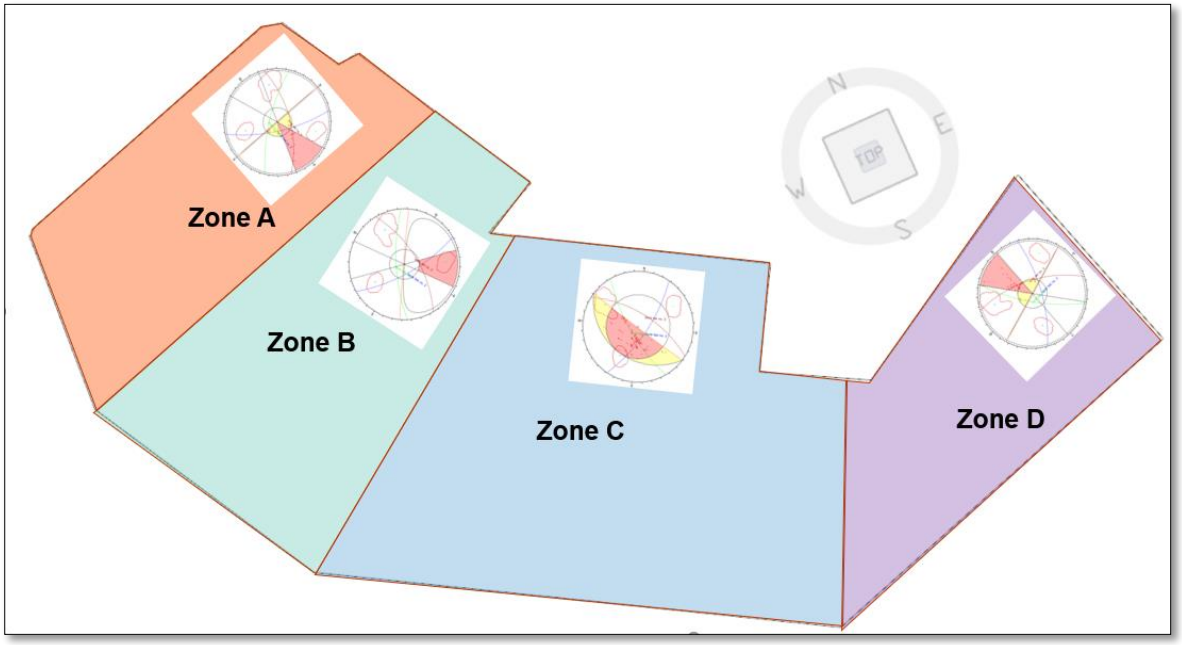

Figure 40: Kinematic analysis per slope zone

The kinematic probability approach is advantageous because the analysis is conducted for the entire joint orientation distribution. When geological structures control failure of the slope, the failure mechanism may either be 1) planar failure or 2) toppling failure or 3) wedge failure. According to Obregon & Mitri (2019) when a case of events is connected by an ‘or’ connector,

it corresponds to a parallel system, and the PoF of the system can be calculated by multiplying the kinematic and kinetic probability of failures

Table 13: Kinematic analysis results

Slope Zone	POF %		
	Toppling	Planar	Wedge
A	4.3	18.0	42.0
B	14.0	18.5	44.0
C	9.4	14.0	31.1
D	13.1	0.0	27.4

4.6.3 Model Probability of Failure

Kinematic and limit equilibrium models were used to compute the PoF. This PoF is based on all inputs and on the limitations of each model and does not fully describe the total PoF considering all factors. It was said that PoF calculated from a geotechnical model typically only accounts for the uncertainty of the material properties and can be referred to as model probability of failure (PoF_{Model})(Contreras, 2015). According to Obregon & Mitri (2019), when a case of events is linked by an 'or' connector, it corresponds to a parallel system, and the PoF of the system can be calculated by multiplying the kinematic and limit equilibrium probabilities of failure. To determine the total PoF, only wedge PoF was used because it is the highest probability and results from the combination of all joint set orientations. The final stability analysis is shown in Table 14.

Table 14: Model PoF for all zones analysis_HW slope

Domain	Slide2 PoF%	Kinematic PoF%	Total Model PoF%	Volume per metre along failure slope (m^3)
Zone A	0.3	42.0	0.12	480.0
Zone B	0.3	44.0	0.13	480.0
Zone C	0.3	31.1	0.09	480.0
Zone D	0.3	27.4	0.08	480.0
Spoil 16m catch	9.7	NA	9.7	414.0
Spoil 10m catch	18.2	NA	18.2	414.0

4.6.4 Total PoF

To minimise the drawbacks of the PoF_{model}, a total probability of failure approach has been suggested. A total probability of failure (PoF_{total}) must incorporate the other sources of uncertainty that may exist within the slope but are not accounted for in the geotechnical model (Contreras, 2015). The origins of this uncertainty may include groundwater, abnormal changes in geology, mining inefficiencies, seismic events and changes in geometry. Contreras (2015) described a method for evaluating the probability of failure based on other sources of uncertainty, where the exposure to atypical conditions. The atypical condition was said to be analysed on an annual basis as described by the following equations;

$$q_{atypical} = 1 - (1 - P_{atypical})^{\frac{1}{n}} \qquad \textit{Equation 9}$$

In this methodology, Contreras (2015) stated that $P_{atypical}$ is the probability of occurrence of a particular uncertain atypical situation leading to slope failure associated with a defined mine duration in years(n). Whereas the $q_{atypical}$, is the annual probability of occurrence of such an atypical situation. It was also further stated that the probability of failure of a slope given the eventuality of atypical condition ($PoF_{model|atypical}$) could be evaluated together with the slope stability model. Results from this are expressed as a factor ($f_{atypical}$) of the model probability of failure, which was assessed under normal conditions, and the relationship is defined as follows:

$$PoF_{model|atypical} = PoF_{model}\, x\, f_{atypical} \qquad \textit{Equation 10}$$

The final probability of failure due to an atypical condition ($PoF_{atypical}$) can be calculated for a particular year (i) of the mine plan as follows;

$$(PoF_{atypical})i = PoF_{model|atypical}\, x(1 - \left(1 - p_{atypical}\right)^{i}) \qquad \textit{Equation 11}$$

The total probability of failure PF_{total} for given uncertainties such as groundwater, geology, and mining is provided by the following equation;

$$PoF_{total} = 1 - (1 - PoF_{model})\, x\, \left(1 - PoF_{groundwater}\right) - \left(1 - PoF_{geology}\right) - (1 - PoF_{mining})$$

Equation 12

The approach described above has indicated that even though the calculated PoF_{model} is zero, the likelihood of failure must never be taken as zero due to the epistemic uncertainty. This approach will be applied to the KE pit, where four atypical conditions were identified: blasting, geometry, geology, and groundwater. It was concluded by Contreras (2015) that to manage subjectivity, expert judgment must be considered. The estimation of these atypical conditions is shown below

- Blasting- This includes surface blasting leading to poor smooth wall condition or high peak particle velocity causing high ground vibrations. There is no certainty that this will not happen, but history during the operation of Klipfontein and blasting of the box cut reveals that it is likely. Therefore, it is estimated that for the life of the KE project, the probability of failure due to this condition is estimated at 15%. The estimation was based on the fact that the slope failure and rockfall database for the last five years within MMS identifies that frozen highwall material due to blasting inefficiencies contributes 11% to total failure (SERITI-MMS, 2021a).
- Geometry-This includes the deviation from planned geometry, which can be observed for both highwall and spoil slopes. During the establishment of the KE box, cut deviations from the scheduled geometry were already observed and recorded in the first year (Mokoena, 2021). The recorded variations allowed the estimation of atypical probability at 10%.
- Geology-This includes unplanned geological structures such as dyke and the loss of rock mass strength due to abnormal geological conditions. The probability of failure due to this is estimated to be 7%.
- Groundwater-this include the probability of failure because of groundwater seepage. The rockfall database within MMS has revealed that groundwater contributes to failure in both highwall and spoil slopes. This probability is estimated at 10%.

The life of operation for the KE pit is planned to be between 2021 and 2027, a total of 7 years (SERITI-MMS, 2021b). The calculated PoF_{total} per year in Tables 15 and 16 is based on Zone B of a rock mass model with 10m catchments. Zone B was selected because it has the highest PoF_{model}.

Table 15: PoF_{total} for 10m catchment _HW Zone B

Year	2022	2023	2024	2025	2026	2027
n	2	3	4	5	6	7
PFmodel	0.1	0.1	0.1	0.1	0.1	0.1
PFmodelblasting	0.2	0.2	0.2	0.2	0.2	0.2
PFmodelgeometry	0.2	0.2	0.2	0.2	0.2	0.2
PFmodelgeology	0.2	0.2	0.2	0.2	0.2	0.2
PFmodelground	0.2	0.2	0.2	0.2	0.2	0.2
PFBlasting	0.9%	1.3%	1.8%	2.2%	2.6%	3.0%
PFGeometry	0.5%	0.8%	1.0%	1.2%	1.5%	1.7%
PFGeology	0.4%	0.6%	0.8%	1.0%	1.2%	1.4%
PFground	0.5%	0.7%	0.9%	1.1%	1.4%	1.6%
PFTotal	**15.2%**	**16.1%**	**17.0%**	**17.9%**	**18.8%**	**19.7%**

Table 16: Input factors_PoF_{total} for 10m Catchment_HW Zone B

LOM	7.0		
	q	P	f
Blasting	0.023	15%	1.5
Geometry	0.015	10%	1.3
Geology	0.010	7%	1.5
Ground	0.015	10%	1.2

Table 17 and 18 show the PoF_{total} for the spoil slope, corresponding to the highwall slope with 10m catchments. Again, $P_{atypical}$ and $f_{atypical}$ were adjusted for the spoil pile slope because the impact of the described uncertainties is low on spoil pile slopes.

Table 17: PoF_{total} for 10m catchment _Spoil

Year	2022	2023	2024	2025	2026	2027
n	2	3	4	5	6	7
PFmodel	18.2	18.2	18.2	18.2	18.2	18.2
PFmodelblasting	0.91	0.91	0.91	0.91	0.91	0.91
PFmodelgeometry	1.82	1.82	1.82	1.82	1.82	1.82
PFmodelgeology	1.092	1.092	1.092	1.092	1.092	1.092
PFmodelground	0.91	0.91	0.91	0.91	0.91	0.91
PFBlasting	3%	4%	5%	7%	8%	9%
PFGeometry	3%	4%	5%	7%	8%	9%
PFGeology	2%	3%	4%	6%	7%	8%
PFground	2%	2%	3%	4%	5%	5%
PFTotal	**9%**	**13%**	**17%**	**21%**	**24%**	**28%**

Table 18: Input factors_PoF_{total} for 10m catchment Spoil

LOM (years)	7.0		
	q	P	f
Blasting	0.0149	0.1	0.05
Geometry	0.0073	0.05	0.1
Geology	0.0103	0.07	0.06
Ground	0.0088	0.06	0.05

4.7 Risk Analysis

Risk has been defined in Section 2.4.3 as a product of the event's likelihood and consequence. In slope engineering, the risk of failure has various possible consequences: safety, economic, reputational, environmental, and legal consequences. This section will describe the safety and economic impact of Zone B of the rock mass model with 10m catchment and the corresponding spoil slope.

4.7.1 Safety Risk Analysis

For a safety impact of a slope to be realised in the form of injury or fatal injury, the failure of the slope must coincide with the spatial and temporal presence (Spatio-temporal coincidence) of personnel underneath the slope. This suggests that slope failure can occur without any impact on personnel safety. Therefore, in order to assess personnel safety risk, personnel exposure

levels to slopes must be known. Contreras et al., (2006) indicated that personnel exposure is established from that of equipment because equipment operators made the most significant proportion of people exposed to slope failures. This exposure can be expressed in terms of spatial factors and temporal factors. Furthermore, equipment exposure to HW can also be estimated from equipment utilisation data which is generally available and reported on.

KE Pit plans to mine, on average five million tons of run of mine (ROM) coal as its annual production. All this coal would have been exposed by a dragline. This coal will be mined through three ramps, each established on the north edge of Zone A, C and D, as shown in Figure 40. It is expected that the entire strip, which is 2.7km, will be mined in a year.

The Spatio-Temporal Coincidence or exposure factors are estimated following the approach adopted by Contreras et al. (2006). This approach uses idealised pit geometry and actual statistical performance data for all types of equipment at risk. The exposure factors will be estimated from planned equipment operational time, cycle time and operational distances. Table 19 indicates the planned operational time per annum for the main equipment within KE Pit.

Table 19: KE equipment annual face time

Equipment	Planned equipment Utilisation (%)	Annual planned face time (days)	Annual planned operational time %
Dragline	**78**	**285**	**78%**
Dozers	**80**	**352**	**96%**
FEL	**75**	**321**	**88%**
Drills	**71**	**207**	**57%**
Trucks	**75**	**321**	**88%**
Blasting crews	**N/A**	**103**	**28%**

The temporal (cycle) exposure factors tabled in Table 20 are based on the times each piece of equipment spent travelling in or working while being fixed within a risk area. It is noted that the only items of equipment that have travel times as part of their operational cycle are the coal dump trucks, whereas the rest of the equipment spent all their operational time exposed to the highwall and spoil failure risk. These trucks will spend a maximum of 30 minutes completing one operational cycle; loading coal from the loading face in the pit, travelling with coal to the stockpile or crusher; dumping coal, and then travelling back to the loading face in the pit. The total cycle time in various zones is partitioned: 13 minutes for travel within the pit (where 6min is along both HW and Spoil), 10 minutes for travel outside the pit and 5 minutes at the loading face and 2 minutes at the crusher.

Table 20: Cycle exposure factors

Equipment	Fixed Part of Cycle			Travel Part of Cycle		
	Fixed (In-pit loading face)	**at HW**	**at Spoil**	**Travel**	**at HW**	**at Spoil**
Coal truck	**20%**	**0.4**	**0.6**	**80%**	**0.16**	**0.125**
Dozer	**40%**	**0.6**	**0.4**	**0**	**0**	**0**
Drills	**100%**	**0.67**	**0.33**	**0**	**0**	**0**
FEL	**100%**	**0.5**	**0.5**	**0**	**0**	**0**
Dragline	**100%**	**0**	**0.1**	**0**	**0**	**0**

KE Pit was planned with three ramps, where each ramp will be responsible for accessing 800-900m of linear strip advance independently. The length of each ramp ranges from 350 to 400m. The spatial exposure coincidence factors correspond to the slope length worked from a single ramp compared with the total length of the pit or cycle worked in a year. This factor was calculated based on the loading-hauling circuit, which is 12km long and from the strip length of 2.7km. The results of the spatial exposure factors are shown in Table 21.

Table 21: Spatial exposure factors

	Cycle	at HW	at Spoil
Spatial	Fixed	**29.60%**	**29.60%**
	Travel	**11.60%**	**6.60%**

Where,
Fixed = length of slope accessed from one ramp/length of the strip
Fixed at HW/Spoil=800m/2700m and
Travel=travel length along HW or Spoil / total travel length
Travel at Spoil= 800m /2700m and Travel at HW=1400m/12000m

The temporal coincidence factors were calculated as a ratio between mining blocks that are at risk of slope failure and the total number of blocks mined per year. This ratio will correspond to the total fraction of time the equipment is exposed to the risky zone. The total number of mining blocks which are at risk is equivalent to the number of blocks that are accessed by a single ramp, which is eight blocks. It is expected that equipment will be fixed in each block along the strip during each year's production advance. One year's advance is equivalent to the advance

of the entire strip length. This means that it will take twelve months to mine out a single strip that is 2.7km long. Results for temporal exposure are depicted in Table 22 below

Table 22: Temporal exposure factors

	Cycle	at HW	at Spoil
Temporal	Fixed	**29.60%**	**29.60%**
	Travel	**43.70%**	**25.00%**

Where,

Fixed=number of mining blocks at-risk/total number of mining blocks mined in one year
Fixed= 8 blocks/ 27 blocks and
Travel= Time equipment is exposed/total cycle time
Travel at HW=13 minutes/30 minutes and Travel at Spoil= 6 minutes/30 minutes

The exposure for specific equipment was calculated based on multiplying all relevant factors in Tables 22, 21, 20 and 19, according to that equipment's fixed and travelling condition as exposed to each slope. The exposure for personnel to safety impact is equivalent to that of the equipment. The results shown in Table 23 are for the fixed condition. Only fixed condition results are depicted because they have higher personal exposure factors. A sample calculation shown below is for the drills for a highwall exposure condition. For coal trucks with both the travel and fixed portion of their cycle time, the same calculation is repeated for both parts, and their products are added.

Calculation
Exposure = temporal factor X spatial factor X cycle factors X annual planned operational time
Exposure=0.296 x 0.296 x 0.67 X 0.57

Table 23: Personnel exposure analysis_fixed

Equipment	No. of people	Exposure HW (%)	Exposure Spoil (%)
Coal truck	5	1.2	1.3
Dozer	2	0.6	0.4
Drills	4	**3.3**	**1.6**
FEL	1	3.8	3.8
Dragline	2	0.0	0.3
Blasting	5	0.6	0.3

The highlighted factors are selected as the critical exposure factors because they are the highest with the highest number of people exposed.

A simple event tree similar to the one described by Terbrugge et al. (2006) was used to calculate the probability of multiple fatalities based on the critical exposure factors indicated in Table 23. Figures 41 and 42 represent the event tree analysis for the rock mass slope and spoil slope, respectively. In addition, the highest probability of failure was used as a basis for designing the KE project during this study. In this fault tree analysis, the probability of multiple fatalities was determined to be 1.56×10^{-4} % and 1.08×10^{-4} %, respectively.

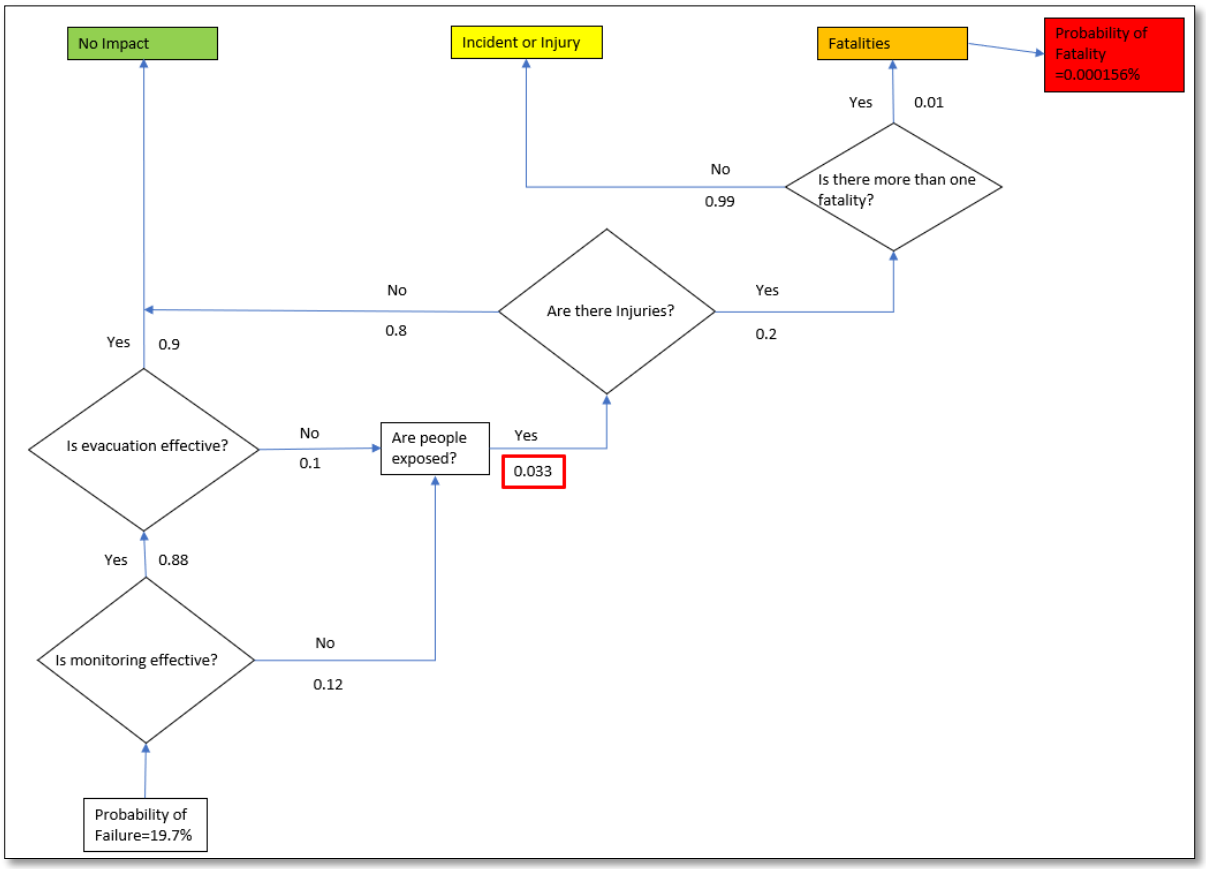

Figure 41: Event tree analysis probability of fatality along HW

Where,

Probability of Fatality = PoFtotal x probability of ineffective monitoring x probability of people exposed x probability of injuries x probability of more than one fatality

Probability of fatality = 19.7% x 0.12 x 0.033 x 0.2 x 0.01= 1.56 x 10^{-4}%

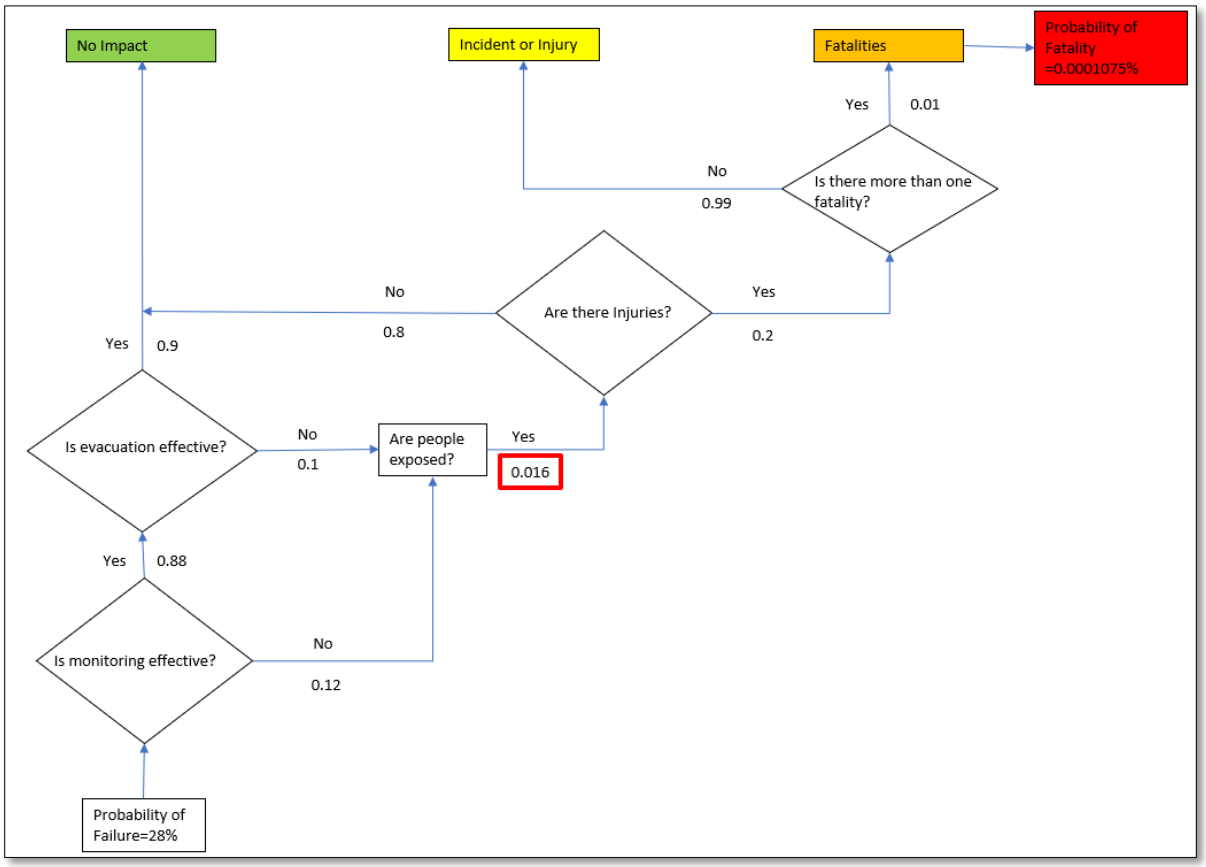

Figure 42: Event tree analysis probability of fatality along spoil slope

Where,

Probability of Fatality = PoFtotal x probability of ineffective monitoring x probability of people exposed x probability of injuries x probability of more than one fatality

Probability of fatality = 28% x 0.12 x 0.016 x 0.2 x 0.01= 1.08 x 10^{-4}%

Acceptability Criteria

The calculated probability of fatality was plotted against the safety acceptability criteria described in section 2.4.3 of this report. The results depicted in Figure 43 indicate that the probability of multiple fatalities from rock mass and spoil design is within the ALARP consensus region and recommended slope design zone.

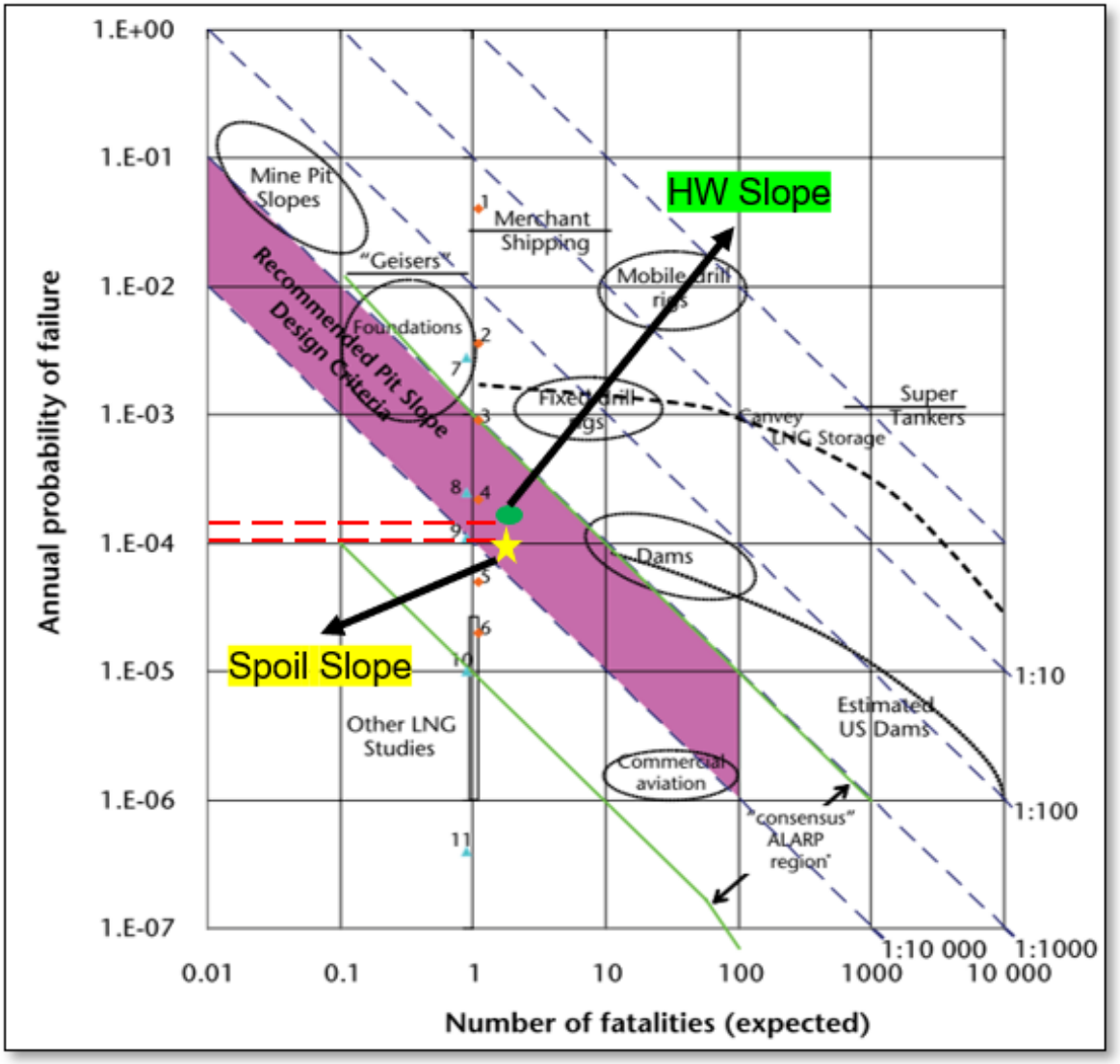

Figure 43: Application of risk acceptability criteria (Terbrugge et al., 2006)

4.7.2 Economic Risk Analysis

The economic evaluation for the KE pit is presented in terms of possible costs incurred during slope failure due to equipment damage, loss of productivity and clean-up of failed material. In three years, MMS incurred damage to the excavator and shovel due to rock slope failure with the estimated cost of ZAR4 239 414 (South32-MMS, 2020). This translates to an annual value of ZAR1 413 138. When slope failure occurs in strip mining within MMS, it generally leads to a two-day loss/delay of coal production, estimated at ZAR 320 000. In addition, the cost of removing the resulting waste material with the truck fleet R41 per cubic metre of volume removed.

Two-dimensional numerical and limit equilibrium models were used for the analysis, which only shows a cross-sectional view of the slope. This means that the actual failure volume can only be calculated by multiplying the failure area with the estimated failure length. MMS fall of the

ground database was reviewed to determine the maximum failure length from historic slope failures. Figures 44 and 45 show the distribution of failure length from the failure database of highwalls and spoil, respectively. The maximum failure length on the highwall and spoils is 91.6m and 71.2m, respectively. It is, therefore, plausible for a slope failure event to have a linear distance of 91m and 71m on highwall and spoil, respectively. The plausible failure length that was being used for the analysis is 91m and 71m. Table 24 summarises the cost of failure contributors.

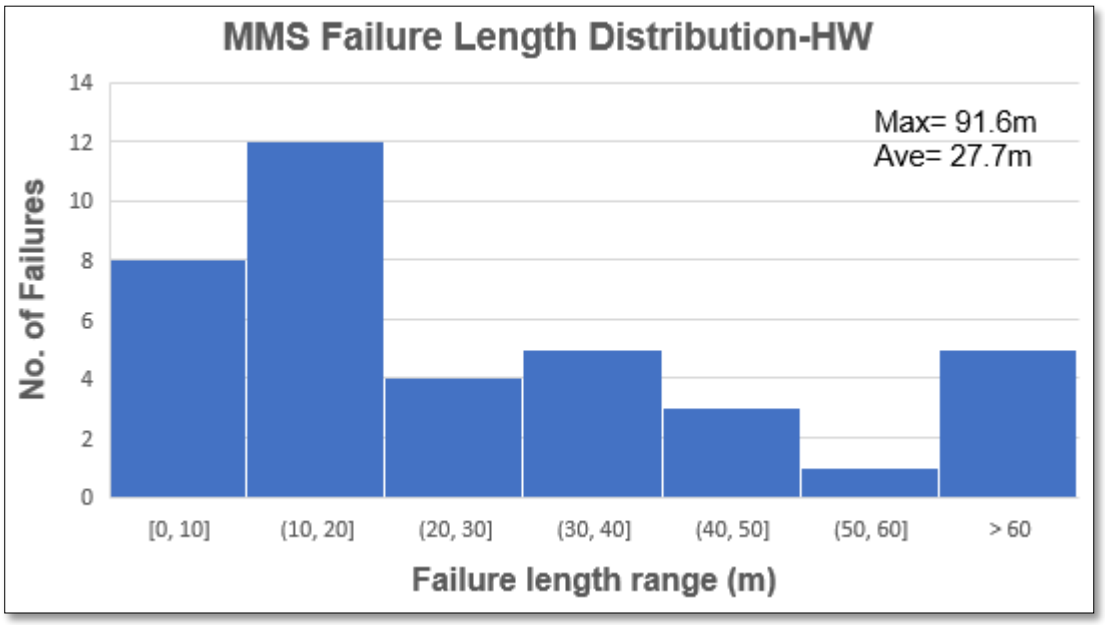

Figure 44: MMS failure length distribution-HW

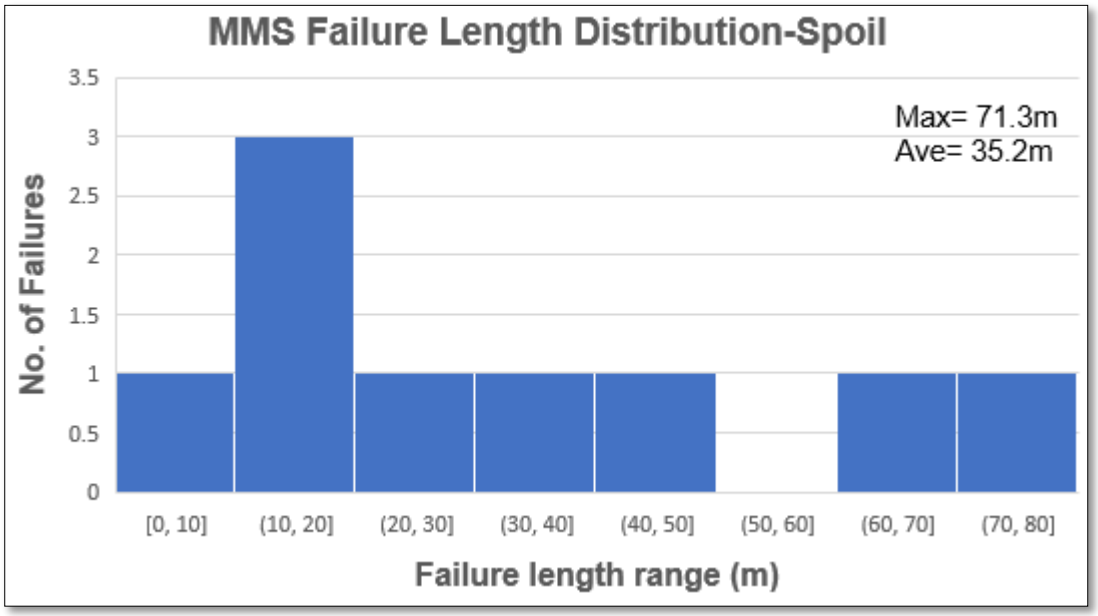

Figure 45: MMS failure length distribution-spoil

Table 24: Cost rates associated with the slope failure event

Category	Annual cost (ZAR)
Equipment Damage	1 413 414
Productivity loss	320 000
Clean-up Cost per m^3 of volume	41

The total cost for an event that is plausible within a year is shown in Table 25 below. The cost presented in this table represent the impact or consequence a slope failure event will have.

Table 25: Total cost per slope failure event

Slope	Volume per linear metre (m^3 /m)	Plausible Failure volume for 91m and 71m length (m^3)	Cost (ZAR)
Highwall	480	43 680	3 524 294
Spoil	414	34 080	3 130 694

Risk Map

A risk map approach defined by Contreras (2015) and based on the event tree methodology first described by Tapia et al. (2007), was adopted for KE pit economic risk analysis. This approach invokes the logic that says intermediate results are impossible, deducing that an event either occurs or does not occur. Consequently, the impact is either caused or not. The event tree analysis conducted for the KE pit was based on impact input in Table 24 and 25 and the probability of occurrence in Table 15 and 17. An example of such an event tree for the first year is shown in Figure 46.

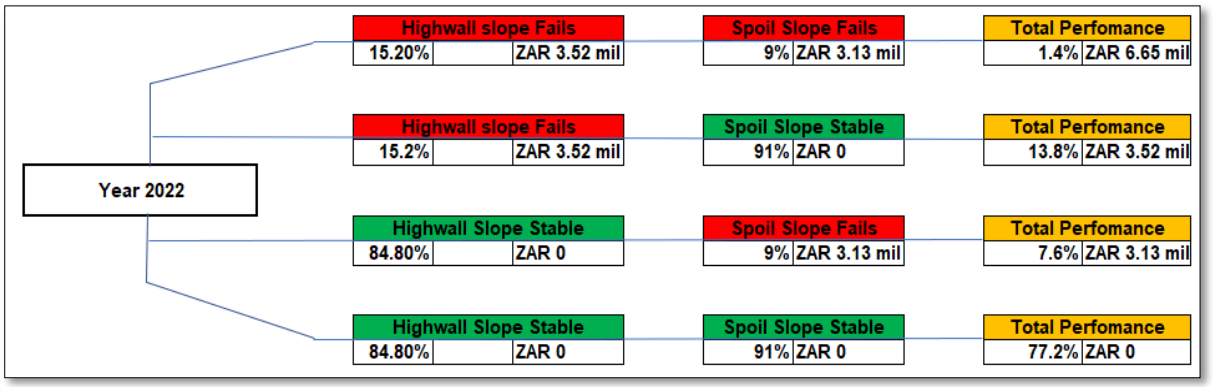

Figure 46: Event tree, economic impact year 2022 (ZARmil)

A risk map in the form of a graph was constructed for 2022, using the total performance in Figure 46.

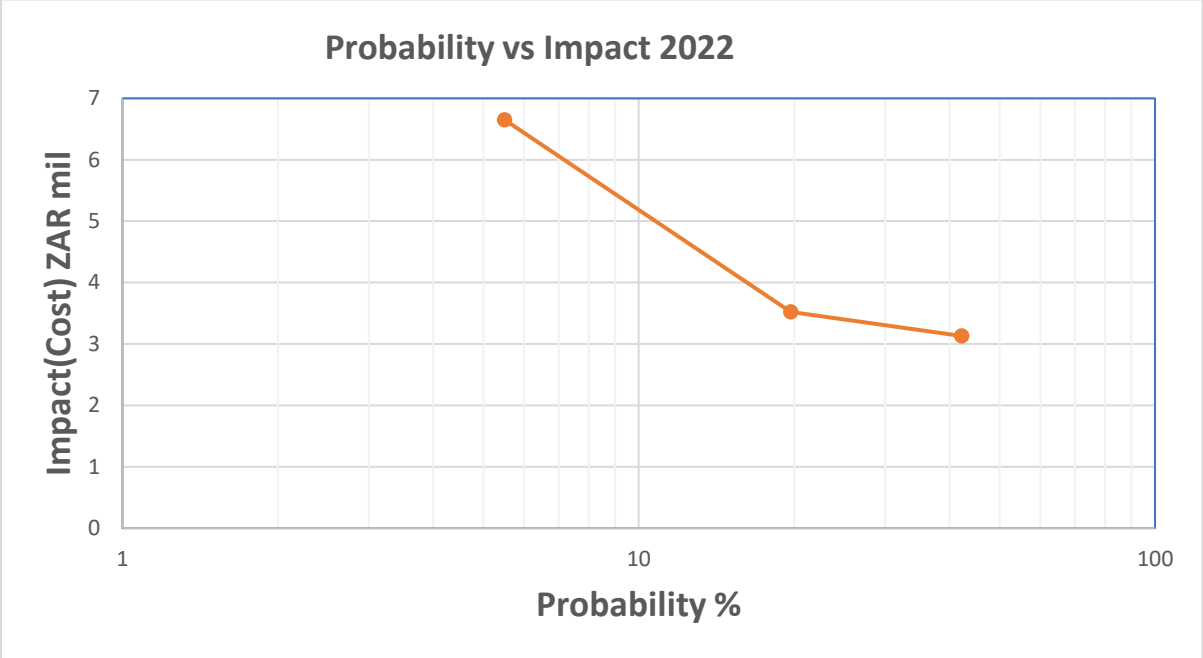

Figure 47: Total slope probability impact performance year 2022

The plotting of the risk map described above was repeated for each year. The risk map for the KE project life or Life of Mine (LOM) is shown in the graph in Figure 48. It is noted in this graph that there is a 10% probability for the operation to lose ZAR5.5mil due to slope failure.

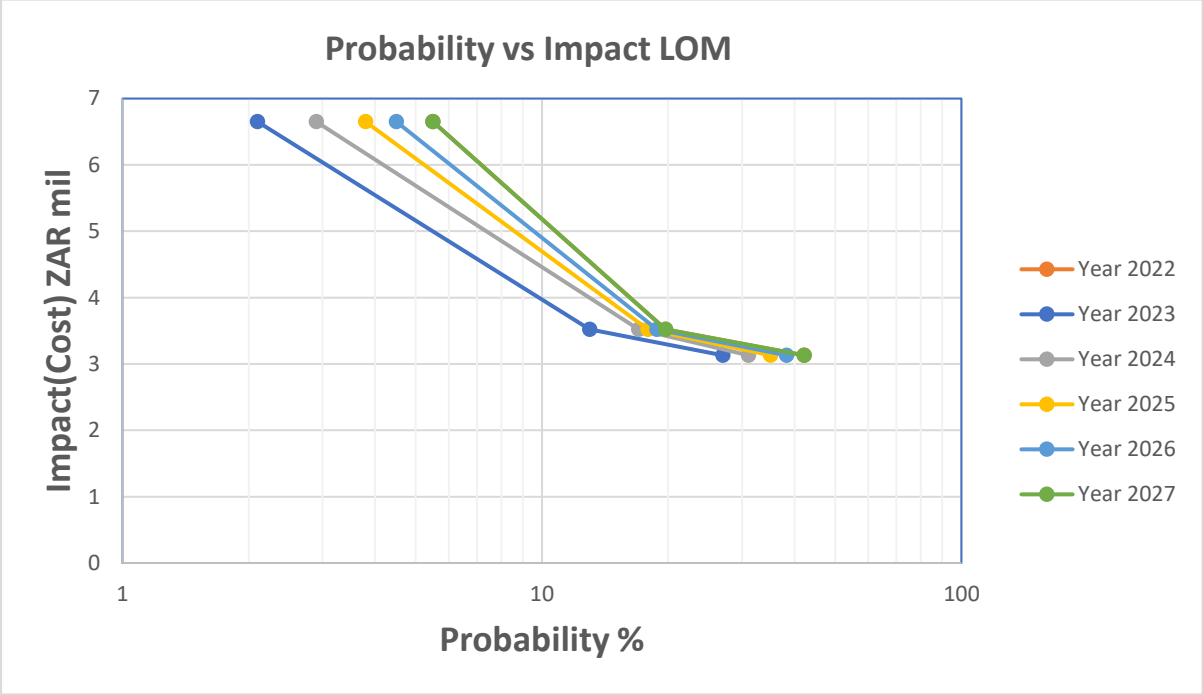

Figure 48: Risk Map KE cumulative probability performance through the life of mine

Acceptability Criteria

The risk map constructed and depicted in Figure 46 is based on cost impact or losses due to cost incurred. According to Contreras (2015), the risk map can have two primary uses; 1) for evaluation of the specific slope design in terms of economic risk and 2) for comparative analysis of open pit design options in terms of value and risk, which can assist in identifying the optimum pit layout.

The risk map constructed is used to compare results with the acceptability criteria. Middelburg Mine Services have a well-defined risk management framework that defines tolerable risk levels, which form part of the policy for risk management. The framework adopts the use of likelihood and severity tables shown in Appendix 1 and states the following:

- Risks with a financial impact MFL (maximum foreseeable loss) ≥ Level 4 and higher must be reviewed bi-annually.
- Risk is deemed intolerable when the MFL is Level 6 and the residual risk rating (RRR) >90.
- Risk is defined as material when financial impact MFL≥ 4 and when MFL≥5 for all other impact types.

Based on the criteria described above, the 5.5% probability of a cost impact of ZAR6.65mil from the risk map in 2027 is not classified as a material risk. It can then be concluded that the risk evaluation of the KE design with a 10m catchment results in a tolerable risk within the operation.

4.8 Creating Value Through Design

The design of open-pit slopes is concerned mainly with defining an optimal slope angle. A slope angle is often regarded as optimal when the angle is steep enough but without compromising safety and economic impact as far as practically possible. It has been stated in section 2 that, generally, the steeper the slope angle, the more economical the open-pit mine will be due to the reduced volume of waste to be removed. In the case of open cast strip mining, steeper slopes result in a reduced volume of broken waste, which shovels and draglines must handle due to the increased volume of waste material that gets ‘cast’ into the void of the previous strip.

In order to demonstrate the cost-benefit associated with the design of the KE Project, a design with a 10m catchment with soft material not pre-stripped is compared with the original design that has a 16m catchment and soft material removed. Table 26 summarises the differences between the two designs.

Table 26: Differences between 10m catch and 16m catch design

Parameter	Original Design (16m Catch)	Proposed Design (10m Catch)
Volume of soft material removed (at least 4m removed)	4 371 088m^3	0m^3
Catchment	16m	10m
Blast Gain	17%	23%
Geotech study required	No	Yes
Rock mass model FoS	2.7	2
Rock mass model PoF	0.0%	0.10%
Spoil model FoS	1.1	1
Spoil model PoF	9.70%	18.20%

The indicative cost-benefit is driven mainly by the two parameters, namely stripping of soft material and blast volume gain, which are briefly described below.

Soft material stripping

Soft material stripping entails removing a layer of soft material, before the drilling of the burden, and placing the material in the dedicated waste dump. This material would then remain in the dump until it is time for rehabilitation, when it would be loaded and transported back to the pit. The same volume pre-stripped would therefore be handled twice at an indicative rate of ZAR40/m^3.

Blast Gain

Blast gain in the strip or open cast mining is a vital productivity parameter, and it is based on the muck pile profile after blasting. It is defined as the volume of the material which will not be handled by either the dragline or dozer because it would have been cast in its final position. The gain difference for the two designs is 6%, translating to a volume of 4567m^3 per mining block. This volume will never be handled by both dragline and dozer, which provides saving of ZAR23/m^3 and ZAR32/m^3 for the dragline and dozer. Therefore, the cost-benefit can be calculated for 359 mining blocks that are part of the KE pit layout.

Table 27 indicates the difference in cost for the two design models. By applying the 10m catchment design following the risk analysis, the project will have a cost-saving of just under half a billion ZAR through its LOM.

Table 27: Cost benefit comparison

Parameter	16m Catch Design Costs (ZAR)	10m catch Design Costs (ZAR)
Soft Stripping	349 687 040.00	0
Blast Gain Comparison	90 175 415.00	0
Geotechnical study	0	1 346 763.00
Monitoring Instruments Capital	0	4 531 230.00
Total	439 862 455.00	5 877 993.00
Cost-Benefit	**R433 984 462.00**	

5 CONCLUSIONS

Coal mining in South Africa is shifting from underground to open pit mining. This shift increases the risk of rock fall and slope failure accidents in open pit mining. Therefore, it necessitates pit slope design and slope stability analysis processes as the main priority of rock mechanics engineers. Traditionally these slopes are designed, and stability is analysed based on the factor of safety approach, whereby factors between 1.2 and 1.4 are deemed acceptable as design criteria. Slope designers adopted the factor of safety design approach through Middelburg Mine Services. However, the design analysis processes were highly dependent on local experience and rules of thumb and, therefore, not based on a detailed geotechnical model.

The literature survey identified design principles in rock engineering, which Stacey (2009) described and incorporated into the design methodology known as the wheel of design. Before the design wheel commences, a strategic planning session must be completed and will determine the design requirements. Next, the literature review identified three main risk design approaches: the factor of safety, probability of failure and risk analysis. The risk analysis method attempts to solve the disadvantages that the factor of safety and probability of failure approach have because it allows the definition of risk criteria which take the consequence of failure into account. Furthermore, Contreras (2015) defined an economic risk evaluated design approach, which provides a complete evaluation of the economic risk associated with slope design.

Middelburg Mine Services was used as a case study to develop a risk-based geotechnical model during the design of its life extension project, KE Pit. The geotechnical model was based on data collected through various techniques, including field geotechnical testing, laboratory testing of rock samples, and structural mapping. In addition, the confidence level model described by Read & Stacey (2008) was adapted to minimise uncertainty in the geological condition.

The work conducted in this research included using limit equilibrium models and comparing them with numerical models for stability analysis of both highwall and spoil slopes in an open cast mine. It was found that the numerical analysis takes significantly longer to compute the probability of failure, and as a result, its use was limited. However, it was demonstrated through this research that the probability of failure from conventional limit equilibrium analysis such as Slide2 models could be combined with the kinematic-based probability of failure to generate the model factor of safety. It was demonstrated through the research that value was created by conducting geotechnical investigations, which allowed for a risk-based design for both highwall and low wall slopes. This value which was created, translated to a cost-saving benefit of ZAR0.5 billion.

A case study was conducted for the KE Pit project to develop a risk-based design for the project, with the following important conclusions:

- More data must be collected to define design parameters such as GSI to reduce uncertainty further and thereby increase the confidence level of the geotechnical model to the operational class.
- LE model analysis underestimates the risk compared to NM. This is because the volume of failure estimated from LE models is lower than that estimated by numerical models.
- NM are not a good tool for computing the probability of failure. This is because the computational time for NM is significant. Therefore, an alternative method for calculating the probability of failure is required when using NM.
- It was demonstrated that KE or MMS design rules were very conservative. The design rules required a model FoS to be 2.7, but it was demonstrated through this research that a model FoS of 2.0 also results in an acceptable risk level.
- Funding data collection and geotechnical investigation programs allow for detailed designs which can create value for the organisation.
- A risk-based design that was followed demonstrated that the KE pit optimised design model was still within the acceptable safety and economic evaluation criteria.
- Adopting a risk-based design at the KE pit resulted in an overall productivity improvement, which translates to a cost saving of ZAR0.5 billion throughout the life of mine (LOM).

6 RECOMMENDATIONS

The main recommendations which are made from this research report are shown below:

- When conducting numerical modelling, alternative statistical methods must be sought to compute the probability of failure.
- Additional geotechnical data must be collected within KE Pit to improve the confidence level to less than 10% variability. This data must be used to optimise the KE design.
- A risk-based design method must be adapted to all other pits within Middelburg Mine Services.

REFERENCE LIST

Adams, B.M. 2015. Slope stability acceptance criteria for opencast mine design. In *12th Australia New Zealand Conference on Geomechanics*. Wellington, New Zeland. 916–923. Available: www.anz2015.com.

Ayati, A.., Samani, H.M.. & Haghighi, A. 2016. Slope stability analysis using genetic algorithm. *Journal of Hydraulic Structures*. 2(1):48–57. DOI: 10.22055/jhs.2016.12650.

Barton, N. & Bandis, S. 1980. Some effects of scale on shear strength of rock joints. *Int. J. Rock Mech. Min Sci & Geomech.* 17:69–73. Available: http://www.nickbarton.com/downloads_01.asp.

Bester, M., Stacey, T.R., Basson, I., Koegelenberg, C., Creus, P., Lorig, L. & Cabrera, A. 2019. A risk-based methodology to improve the definition of geotechnical sectors in slope design. *Journal of the Southern African Institute of Mining and Metallurgy*. 119(12):1027–1038. DOI: 10.17159/2411-9717/685/2019.

Bieniawski, Z.T. 1984. The design process in rock engineering. *Rock Mechanics and Rock Engineering*. 17(3):183–190. DOI: 10.1007/BF01042549.

Bieniawski, Z.T. 1991. In search of a design methodology for rock mechanics, Rock mechanics as a multidisciplinary science. In *32nd U S Symp. on Rock Mech*. Roegiers, Ed. Balkema. 1027–1036.

Bieniawski, Z.T. 1992. Invited Paper: Principles of engineering design for rock mechanics. In *33rd U S Symp. on Rock Mech*. Tillerson & Wawersik, Eds. Balkema. 1031–1040.

Bishop, A.W. 1955. The use of the slip circle in the stability analysis of slopes. *Geotechnique*. 5(1):7–17. DOI: 10.1680/geot.1955.5.1.7.

Brischke, C. & Humar, M. 2017. Performance of the bio-based materials. In *Performance of Bio-based Building Materials*. Woodhead Publishing. 249–333. DOI: 10.1016/B978-0-08-100982-6.00005-7.

Butcher, R.J., Walker, D.J., Joughin, W.C., Birtles, A.N. & Terbrugge, P.J. 2001. *Methodology for safe cleaning and making safe of various height (10-35m) highwalls*. (GEN 703). Johannesburg: Safety in Mines Research Advisory Committee.

Bye, A. 2003. The development and application of a 3D geotechnical model for mining optimisation, Sandsloot open pit platinum mine, South Africa. University of Natal.

Byrne, G. & Berry, A. 2008. *A Guide to Practical Geotechnical Engineering in Southen Africa*. Fourth ed. Johannesburg: Franki Africa.

Chaulya, S.K. & Prasad, G.M. 2016. Slope Failure Mechanism and Monitoring Techniques. In *Sensing and Monitoring Technologies for Mines and Hazardous Areas*. Elsevier. 1–86. DOI: 10.1016/B978-0-12-803194-0.00001-5.

Chiwaye, T.H. 2010. A comparison of the limit equilibrium and numerical modelling approaches to risk analysis for open pit mine slopes. University of Witwatersrand. Available: http://hdl.handle.net/10539/8376.

Chiwaye, H.T. & Stacey, T.R. 2010. A comparison of limit equilibrium and numerical modelling approaches to risk analysis for open pit mining. *Journal of the Southern African Institute of Mining and Metallurgy*. 110(10):571–580. Available: https://0-www-onemine-org.innopac.wits.ac.za/document/document.cfm?docid=170010&docorgid=9.

Contreras, L.F. 2015. An economic risk evaluation approach for pit slope optimization. *Journal of the Southern African Institute of Mining and Metallurgy*. 115(7):607–622. DOI: 10.17159/2411-9717/2015/v115n7a7.

Contreras, L.-F., LeSueur, R. & Maran, J. 2006. A Case Study Of Risk Evaluation at Cerrejon Mine. In *International Symposium on Stability of Rock Slopes in Open Pit Mining and Civil Engineering*. The South African Institute of Mining and Metallurgy. Available: https://0-www-onemine-org.innopac.wits.ac.za/document/abstract.cfm?docid=47910&title=A-Case-Study-Of-Risk-Evaluation-At-Cerrejon-Mine.

Department of Mineral Resource. 2018. *Mine Health and Safety Act 29 of 1996 and Regulations*. Johannesburg: Mine Health and Safety Council.

Dixon, R.A., Johnson, R.A., de Graaf, P.J.H., Wessels, S.D.N. & Venter, J. 2011. Risk Based Geotechnical Slope Reconciliation at Rio Tinto Iron Ore, Pilbara Operations. In *International Symposium on slope stability in Open Pit Mining and Civil Engineering*. Vancouver, Canada. Available: https://www.researchgate.net/publication/325082853.

Ekkerd, J.N. 2011. Geotechnical Optimisation of the Venetia Open Pit. University of Witwatersrand. Available: http://hdl.handle.net/10539/12331.

Elmouttie, M. & Karekal, S. 2017. A Framework for Geotechnical Hazard Analysis in Highwall Mining Entries. *Procedia Engineering*. 191:1203–1210. DOI: 10.1016/j.proeng.2017.05.296.

Fillion, M.H. & Hadjigeorgiou, J. 2016. Implications of collecting additional data for slope design in an open pit operation. *Journal of the Southern African Institute of Mining and Metallurgy*.

116(5):357–366. DOI: 10.17159/2411-9717/2016/v116na1.

Fillion, M.H. & Hadjigeorgiou, J. 2019. Quantifying influence of drilling additional boreholes on quality of geological model. *Canadian Geotechnical Journal*. 56(3):347–363. DOI: 10.1139/cgj-2017-0653.

Geertsema, A.J. 2000. *The Engineering Characteristics of Important Southern African Rock Types With Emphasis on Shear Strength of Concrete Dam Foundations .* Pretoria. Available: http://www.wrc.org.za/wp-content/uploads/mdocs/433-1-00.pdf.

Grider, A., Ramirez, J.A. & Yun, Y.M. 2003. *The Civil Engineering Handbook*. Second ed. W.F. Chen & J.Y. Richard Liew, Eds. London: CRC Press LLC.

Hammah, R., Yacoub, T.E. & Curran, J.H. 2009. Numerical modelling of slope uncertainty due to rock mass jointing. *International Conference on Rock Joints and Jointed Rock Masses*. (July):1–8. Available: http://www.rockscience.com/assets/files/uploads/7778.pdf.

Hammah, R.E., Yacoub, T.E., Corkum, B.C. & Curran, J.H. 2005. *The shear strength reduction method for the generalized Hoek-Brown criterion*. Toronto. Available: https://www.rocscience.com/assets/resources/learning/papers/The-Shear-Strength-Reduction-Method-for-the-Generalized-Hoek-Brown-Criterion.pdf.

Hoek, E. 2006. *Practical rock engineering*. Available: https://www.rocscience.com/assets/resources/learning/hoek/Practical-Rock-Engineering-Full-Text.pdf.

Hoek, E. 2009. Fundamentals of slope design. In *Slope Stability 2009*. Santiago, Chile.

Hoek, E. & Brown, E.T. 2019. The Hoek–Brown failure criterion and GSI – 2018 edition. *Journal of Rock Mechanics and Geotechnical Engineering*. 11(3):445–463. DOI: 10.1016/j.jrmge.2018.08.001.

Hoek, E., Carranza-torres, C. & Corkum, B. 2002. Hoek-Brown failure criterion – 2002 Edition Hoek-Brown failure criterion-2002 edition. Toronto: Proc. NARMS_TAC Conference. 267–273. Available: https://www.rocscience.com/assets/resources/learning/hoek/Hoek-Brown-Failure-Criterion-2002.pdf.

Ilbury, C. & Sunter, C. 2005. *Games foxes play-planning for extraordinary times*. Human & Rosseau Tafelberg.

Institute of Risk Management. 2018. *A Risk Practitioners Guide to ISO 31000: 2018*. London. Available: www.theirm.org.

Janbu, N. 1954. Application of composite slide circles for stability analysis. In *European conference on stability of earth slopes*. Stockholm.

Jeffrey, L.S. 2005. Characterization of the coal resources of South Africa. *Journal of The South African Institute of Mining and Metallurgy*. 105(2):95–102. Available: https://www.saimm.co.za/Journal/v105n02p095.pdf.

Kanda, M.J. 2015. Limit equilibrium and numerical modelling approaches in slope stability analyses with regard to risk assessment for open pit mining. University of the Witwatersrand. Available: http://hdl.handle.net/10539/20094.

Kanda, M.J. & Stacey, T.R. 2016. The influence of various factors on the results of stability analysis of rock slopes and on the evaluation of risk. *Journal of the Southern African Institute of Mining and Metallurgy*. 116(11):1075–1081. DOI: 10.17159/2411-9717/2016/v116n11a10.

Kumar, S., Kayet, N. & Pathak, K. 2021. A geospatial approach to analyze the stability of mine overburden dump over reclaimed land. In *Modern Cartography Series*. V. 10. Academic Press. 205–220. DOI: 10.1016/B978-0-12-823895-0.00033-6.

Lato, M., Kemeny, J., Harrap, R.M. & Bevan, G. 2013. Rock bench: Establishing a common repository and standards for assessing rockmass characteristics using LiDAR and photogrammetry. *Computers and Geosciences*. 50:106–114. DOI: 10.1016/j.cageo.2012.06.014.

Little, M.J. 2006. Geotechnical Strategy and Tactics At Anglo Platinum ' S Pprust Open Pit Operation , Limpopo Province , South Africa. University of Witwatersrand. Available: https://wiredspace.wits.ac.za/handle/10539/3899.

Lorig, L., Stacey, P. & Read, J. 2008. Slope design methods. In *Guidelines for open pit slope design*. Collingwood, Australia: The Commonwealth Scientific and Industrial Research Organisation (CSIRO). 235–260.

Middelburg Mine Services. 2019. *MANDATORY COP COMBAT ROCK FALL AND SLOPE INSTABILITY RELATED ACCIDENTS IN SURFACE MINES*. Witbank.

Mineral Council South Africa. 2019. *Coal can be extracted from*. Available: https://www.miningforschools.co.za/lets-explore/coal/the-coal-mining-life-cycle [2021, December 13].

Mokoena, M. 2021. *Design Compliance Audit _ KE Pit*. Witbank: SERETI, MMS Internal report.

Morgenstern, N.. & Prince, V.. 1965. The analysis of the stability of general slip surfaces.

Geotechnique. 15(1):79–93. DOI: 10.1680/geot.1968.18.1.92.

Obregon, C. & Mitri, H. 2019. Probabilistic approach for open pit bench slope stability analysis – A mine case study. *International Journal of Mining Science and Technology*. 29(4):629–640. DOI: 10.1016/j.ijmst.2019.06.017.

Olechowski, A., Oehmen, J., Seering, W. & Ben-Daya, M. 2016. The professionalization of risk management: What role can the ISO 31000 risk management principles play? *International Journal of Project Management*. 34(8):1568–1578. DOI: 10.1016/j.ijproman.2016.08.002.

Ramane, L. 2014. *South Africa's coal industry overview, 2014*. Pretoria. Available: http://www.energy.gov.za/files/media/explained/South-African-Coal-Sector-Report.pdf.

Rathod, G.W. & Rao, K.S. 2012. Finite Element and Reliability Analyses for Slope Stability of Subansiri Lower Hydroelectric Project: A Case Study. *Geotechnical and Geological Engineering*. 30(1):233–252. DOI: 10.1007/s10706-011-9465-2.

Read, J. & Stacey, P. 2008. *Guidelines for Open Pit Slope Design*. Collingwood, Australia: The Commonwealth Scientific and Industrial Research Organisation (CSIRO).

Rocscience. 2021. *Slide2 Documentation and Theory of Application*. V. 26. DOI: 10.7591/9781501728761-005.

Russell, T.M. 2018. Increasing Geotechnical Data Confidence through the Integration of Laser Scanner Face Mapping Data into the Sishen Iron Ore Mine Geotechnical Database. University of Witwatersrand. Available: https://hdl.handle.net/10539/25215.

Satyanarayana, I., Budi, G. & Murmu, S. 2021. Stability analysis of a deep highwall slope using numerical modelling and statistical approach—a case study. *Arabian Journal of Geosciences*. 14(3). DOI: 10.1007/s12517-021-06476-x.

SERITI-MMS. 2021a. *Mandatory code of practice to combat rock fall and slope instability related incidents in surface mines*. Witbank.

SERITI-MMS. 2021b. *MMS Life of Mine Plan*. Witbank.

South32-MMS. 2020. *SAEC Slope Failure Management Strategy*. Witbank.

Stacey, T.R. 2009. Design - A strategic issue. *Journal of the Southern African Institute of Mining and Metallurgy*. 109(3):157–162. Available: http://www.scielo.org.za/scielo.php?script=sci_abstract&pid=S2225-62532009000300002.

Stacey, T.R. 2011. *Rock Engineering Lecture Notes-MINN4010*. Johannesburg: School of

Mining Engineering, University of Witwatersrand.

Stacey, T.R., Terbrugge, P.. & Wesseloo, J. 2007. Risk as a Rock Engineering Design Criterion. In *Challenges in Deep and High Stress Mining — Y. Potvin, T.R. Stacey, J. Hadjigeorgiou*. Perth: Australian Center for Geomechanics. 19–26.

Steffen, O.K.H. 1997. Planning of open pit mines on a risk basis. *Journal of The South African Institute of Mining and Metallurgy*. 97(2):47–56. Available: https://www.saimm.co.za/Journal/v097n02p047.pdf.

Taleb, N.N., Goldstein, D.G. & Spitznagel, M.W. 2009. *The Six Mistakes Executives Make in Risk Management*. Available: www.hbr.org.

Tan, T.K. & Kang, W.F. 1981. Locked in stresses, creep and dilatancy of rocks, and constitutive equations : Rock Mech, V13, N1, Aug 1980, P5–22. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*. 18(1):4. DOI: 10.1016/0148-9062(81)90320-X.

Tapia, R.A., Contreras, L.F., Jefferies, M. & Steffen, O. 2007. Risk Evaluation of Slope Failure at the Chuquicamata Mine. 477–495. DOI: 10.36487/acg_repo/708_32.

Terbrugge, P.J., Wesseloo, J., Venter, J. & Steffen, O.K.H. 2006. A risk consequence approach to open pit slope design. *The Journal of the South African Institute of Mining and Metallurgy*. 106:503–514.

Ueng, T., Jou, Y. & Peng, I. 2010. Scale effect on shear strength of computer-aided-manufactured joints. 5(2):29–37. Available: http://yo-1.ct.ntust.edu.tw/jge/files/articlefiles/v5i2201012071996878529.pdf.

University of Pretoria. 2021. Lecture Note_Certificate for Managers in Risk Management. In *UP Enterprise Lecture*. Pretoria.

Utili, S. & Crosta, G.B. 2015. Analysis Tools for Mass Movement Assessment. In *Landslide Hazards, Risks, and Disasters*. Elsevier Inc. 441–465. DOI: 10.1016/B978-0-12-396452-6.00013-6.

Van der Merwe, J.N. & Madden, B.J. 2003. *Rock Engineering for Underground Coal Mining*. Special Pu ed. The South African Institute of Mining and Metallurgy: Special Publication Series 7.

Vöge, M., Lato, M.J. & Diederichs, M.S. 2013. Automated rockmass discontinuity mapping from 3-dimensional surface data. *Engineering Geology*. 164:155–162. DOI:

10.1016/j.enggeo.2013.07.008.

Watson, B. 2020. Numerical Modelling techniques in rock engineering_Course Notes_MINN7008. Johannesburg: School of Mining Engineering, University of Witwatersrand. 1–87.

Watson, B. 2021. Mechanical Properties of Rocks and Rockmasses_Lecture Notes_ MINN7005_. Johannesburg: School of Mining Engineering, University of Witwatersrand.

Wong, W. 2002. *How dit that happen? Engineering safety and reliability*. London: Professional Engineering Pub.

Wyllie, D.C. & Mah, C.W. 2004. *Rock slope engineering: Civil and mining, 4th edition*. 4th ed. New York: Spon Press. DOI: 10.1201/9781315274980.

Zevgolis, I.E., Deliveris, A. V. & Koukouzas, N.C. 2018. Probabilistic design optimization and simplified geotechnical risk analysis for large open pit excavations. *Computers and Geotechnics*. 103:153–164. DOI: 10.1016/j.compgeo.2018.07.024.

APPENDIX

Appendix 1: Likelihood and Severity Tables_MMS

Likelihood Table

Uncertainty	Business	Projects	Likelihood Factor
	Based on SAEC and industry experience and expected future conditions, the risk event:	Based on SAEC and industry experience and expected future conditions, with similar studies or projects, the risk event:	
Almost certain	Could be incurred more than once in a year.	Could be expected to occur more than once during the study or project delivery.	**10**
Likely	Could be incurred over a 1 - 2 year budget period.	Could easily be incurred and has generally occurred in similar studies or projects.	**3**
Possible	Could be incurred within a 5 year strategic planning period.	Incurred in a minority of similar studies or projects.	**1**
Unlikely	Could be incurred within a 5 - 20 year time frame.	Known to happen, but only rarely.	**0.3**
Rare	Could be incurred in a 20 - 50 year timeframe.	Has not occurred in similar studies or projects, but could.	**0.1**
Very rare	**For a system failure:** This consequence has not happened in the industry in the last 50 years. **For a natural hazard:** The predicted return period for a risk of this strength/ magnitude is one in 100 years or longer.	Conceivable, but only in extreme circumstances.	**0.03**

Severity Level	Impact Types						Severity Factor
	Health[1] and safety	Environment	Community	Reputation	Legal	Financial[2]	
7	>50 fatalities. Permanent impairment >30% of body to more than 500 persons.	Permanent severe impact/s to land, biodiversity, ecosystem services, water resources or air.	Severe, widespread community health, safety or security impacts (>1000 households) or human rights violations; complete destruction of >1000 houses or community infrastructure; complete irreversible desecration of multiple structures/objects/places of global significance.	Crisis event or publication of highly confidential material information resulting in international media, government, regulator, NGO campaigning and employee condemnation of the company (>6 months). Long term damage to company reputation.	Bankruptcy, closure / nationalisation of operations on multiple sites.	≥ US$1 billion	1000
6	>20 fatalities. Permanent impairment >30% of body to more than100 persons.	Severe impact/s (>20years) to land, biodiversity, ecosystem services, water resources or air.	Extensive community health, safety or security impacts (>200 households) or human rights violations; extended serious disruption to people's lives (>1000 households); extensive damage to >1000 houses or community infrastructure or structures/ objects/places of global cultural significance.	Crisis event or publication of confidential material information resulting in international media, government, regulator, NGO campaigning and employee condemnation of the company (< 6 months). Ongoing condemnation results in damage of the reputation of the company.	Lack of valid operating title, forced closure of an operation, competition , anti-corruption , international trade law or tax breach; Major personal injury class actions. Nationalisation of Operation by host government.	≥ US$250 million to <US$1 billion	300
5	2-20 fatalities Permanent impairment >30% of body more than 10 persons.	Serious or extensive impact/s (<20 years) to land, biodiversity, ecosystem services, water resources or air.	Serious community health, safety or security impacts (>50 households) or human rights violations; extended disruption to people's lives (>200 households), extensive damage to >200 houses or structures/ objects/places of national cultural significance.	Serious national and international negative media attention. General public and NGO adverse reaction with interest from regulators (< 3 months). Structured campaigning from employees, NGOs or communities having a major impact on the company / Operations reputation.	Prosecutions for criminal breaches resulting in jail terms for employees or agents or defendant to major civil litigation.	≥ US$100 million to < US$250 million	100
4	Single Fatality. Permanent impairment >30% of body to	Major impact/s (<5 years) to land, biodiversity, ecosystem	Serious community health, safety or security impacts (<50 households). Multiple allegations	Adverse national media attention. General public and NGO adverse reaction with interest	Significant civil litigation.	≥ US$25 million to <US$100 million	30

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	Health[1] and safety	Environment	Community	Reputation	Legal	Financial[2]	
	one or more persons.	services, water resources or air.	of human rights violations; extended disruption to people's lives (>50 households); extensive damage to >50 houses; moderate irreversible damage to structures/ objects/places of national cultural significance.	from regulators with no material outcome. Structured campaigning from employees, NGOs or communities having a major impact on the company / Operations reputation.			
3	Permanent impairment <30% of body to one or more persons. Restricted or lost days due to injury or illness.	Moderate impact/s (<1 year) to land, biodiversity, ecosystem services, water resources or air.	Moderate community health, safety or security impacts (<50 households). Single allegation of human rights violations; moderate disruption to people's lives (<50 households); extensive damage to <50 houses; moderate reversible damage to structures/objects/ places of national cultural significance.	Attention from media and/or heightened concern by local community. Criticism by community, NGOs or activists. Operations reputation adversely affected.	Breach of regulation. Lack of valid exploration title.	≥ US$5 million to < US$25 million	10
2	Objective but reversible impairment. Medical treatment injury or illness.	Minor impact/s (<3 months) to land, biodiversity, ecosystem services, water resources or air.	Minor community health, safety or security impacts (<10 households) or human rights infringements; inconvenience to livelihoods <6 months; moderate damage to <50 houses or community infrastructure; minor, reversible damage to structures/ objects/places of regional cultural significance.	Adverse local public or media attention and complaints. Heightened scrutiny from regulator. Operations reputation is adversely affected with a small number of people.	Minor legal issues and non-compliances with commitments.	≥US$500,000 to <US$5 million	3
1	Low-level short-term subjective symptoms or inconvenience. No medical treatment.	Low-level impact/s to land, biodiversity, ecosystem services, water resources or air.	Single low-level community health, safety or security impact; low-level inconvenience <2 weeks; minor, reversible, low-level disturbance or minor damage to a single house or structure/object/place of regional cultural significance.	Public concern restricted to local complaints. Low-level interest from local media and/or regulator.	Low-level legal issue.	<US$500,000	1