# RELIABILITY ANALYSIS OF OPEN PIT SLOPE: CASE STUDY OF BOZSHAKOL MINE

by

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# ORIGINAL STATEMENT

I, Anyasodor Kemakolam Hayes hereby declares that the submission of this thesis was done in my own self-interest and without violating the academic standards of this learning institution. It does not include any writings or materials from other people or organizations unless they are properly acknowledged in the thesis.

**Signed on:** **Supervisor signature:**

# ABSTRACT

The increase in the probability of failures in pit slope design are influenced by the presence of the inherent variability of rock mass properties. This could result in uncertainties during geotechnical design. Although, utilizing conservative approach such as overdesigning to account for the worst-case scenario, can effectively address these issues in practice. Despite this, one of the approach's shortcomings is that it is quite expensive. In order to tackle these difficulties, it is crucial to devise a probabilistic technique that allows for thorough consideration of these uncertainties associated with geotechnical parameters when designing mine pit. With this in mind, the objective of this study is to execute the reliability analysis of the Bozshakol mine pit slope design. This method would involve utilising advanced numerical models, such as Slide 2 and RS 2 to design selected pit sectors by determining their factor of safety and probability of failure based on the input variables obtained from the case study area. The first-order reliability method was used to analyse the results obtained from the model. Furthermore, the expected outcomes provide a means of addressing these uncertainties and establishing a specific target for the probability of failure in pit slope design. Subsequently, an evaluation was conducted to estimate the reliability indices of geotechnical domains pertaining to the pit sectors. This was accomplished in order to determine the reliability of the pit design and to identify probable risks of unforeseen pit failures that could be addressed in a timely manner.

*Keywords: reliability analysis, pit slope design, numerical modelling*

# DEDICATION

This thesis is dedicated to God Almighty, who made the success of this thesis possible.

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# CHAPTER ONE

## 1. INTRODUCTION

Surface mining, also known as open-pit mining, is a method that involves extracting resources from the near surface of the earth, by employing heavy machinery and equipment. This method entails removing overburden from the top soil to gain direct access to the desired valuable minerals through the use of various mining techniques, such as drilling, blasting and hauling. It is widely used for a variety of resources such as coal, ore deposits, marble, and others because it provides high output, low production costs, and mining flexibility.

Today, most nations' economies rely heavily on this method, and Kazakhstan is no exception. However, despite its benefits, it also poses significant challenges, particularly in pit slope design as the geometry of the pit changes over time. These difficulties stem from uncertainty datasets caused by the inherent variability of rock mass properties used in geotechnical model design, which could endanger mine personnel and equipment (Baecher & Christian, 2005; Kjekstad & Highland, 2009).

In practice, these uncertainties are typically addressed by employing a conservative design approach, such as assuming the worst-case scenario, using a deterministic method that relies on a single safety of factor value to represent the overall slope stability. However, previous research has shown that this conservative approach does not explicitly account for the variability of input design parameters and other uncertainties, implying that they are not reliable and can still result to pit slope failures (Duzgun, Yucemen, & Karpuz, 2003; Jimenez-Rodriguez, Sitar, & Chacón, 2006; Phoon, 2008).

In order to address these limitations, a reliability technique that accounts for design uncertainties is required. Although, this method has been accepted generally by experts in rock engineering (Musah Abdulai & Mostafa Sharifzadeh, 2019; Chen et al., 2022; Duzgun et al., 2003; Li et al., 2019; Miller, Whyatt, & McHugh, 2004). Generally, the primary goal of this method is to evaluate the probability of accomplishing the desired performance target while mitigating failure risks. This will then allow for informed decision-making by determining the probabilities of slope failure.

## 1.1 Problem Statement

The Bozshakol mine, situated in the Kazakhstan Pavlodar region, is a crucial global source of copper through its open pit mining operations. However, the mine is currently grappling with challenges related to slope instability as mining activities progress. This has raised serious concerns among the management regarding the safety of mine personnel and equipment, potentially resulting in production delays and increased costs. For example, in May 2022, a major crack due to tension failure was observed in a wall of the pit (the south wall of sector 6) as shown in Fig 1.1. However, the mine was designed with low geotechnical risks according to the internal report from Kazminerals (2022). Therefore, it is important to investigate the reliability of the design implemented at the Bozshakol mine to ensure the safety and efficiency of ongoing operations.

Figure 1.1: An example of tension crack during field inspection.

## 1.2 Aims and Objectives

Motivated by the aforementioned challenges, this study aims to conduct a reliability analysis on the pit slope design of selected sectors at Bozshakol mine. This will be achieved by evaluating the factor of safety and probability of failure through numerical modelling, and by determining the reliability indices associated with each geotechnical domain using the first order reliability method (FORM). The specific objectives are as follows:

1 To collect and analyse the datasets based on this study from Kaz Minerals;
2 Conduct a thorough literature review;
3 To perform a numerical modelling and probability analysis of selected sectors of the pit;
4 To compare and analyse the results from both approach;

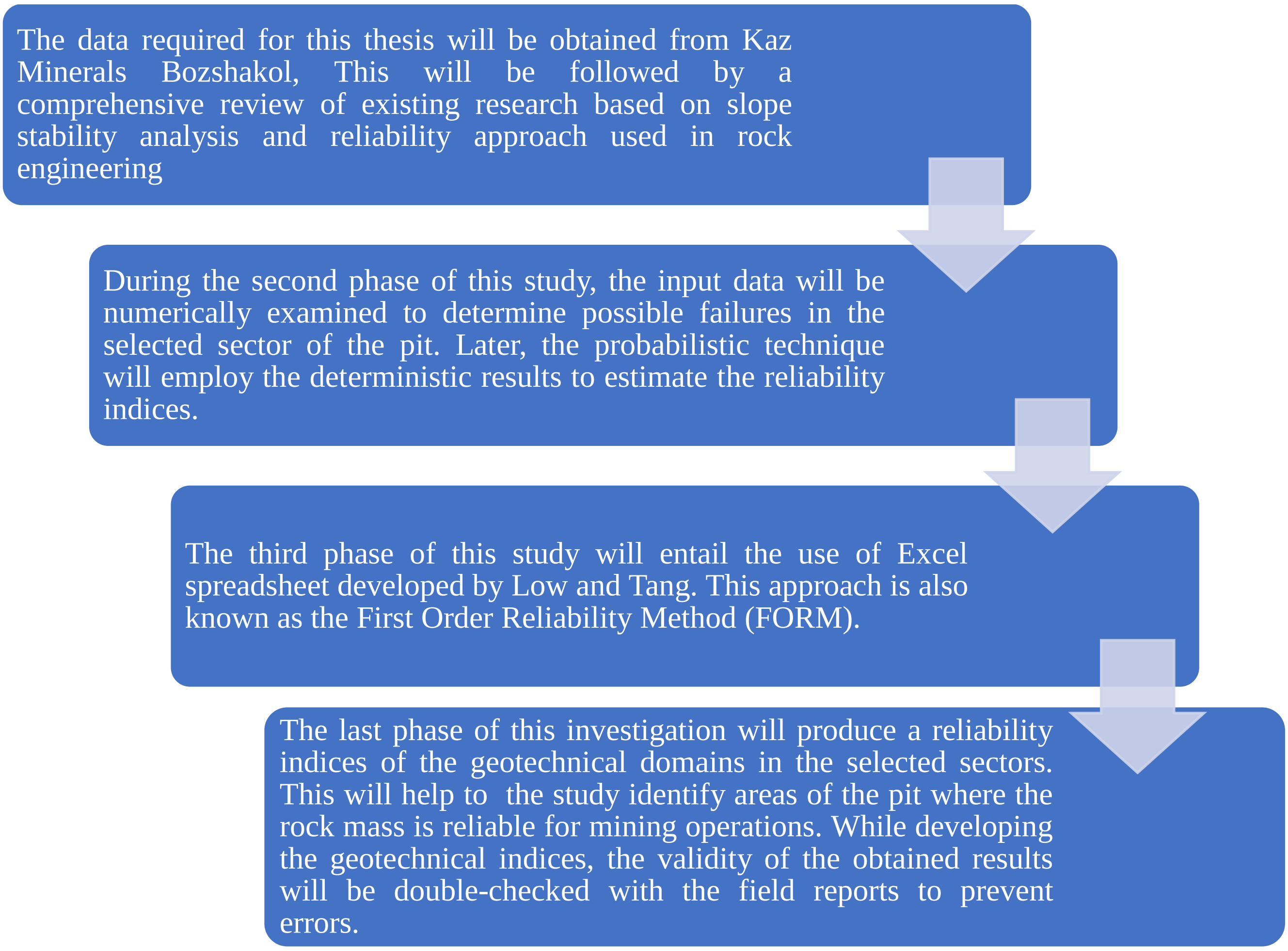
5 To provide recommendations to Bozshakol mine based on the analysed results to enhance the reliability of the pit slope for safety during operations.

## 1.3 Method

The overall goal of this study is intended to thoroughly examine the geotechnical sectors of the Bozshakol pit, utilising reliability analysis to assess potential slope instabilities by considering various input variables. Firstly, a critical review of existing literature related to this thesis in rock engineering are conducted, aimed at identifying gaps in knowledge, through Nazarbayev University library and online resources. The required data for the study, including the rock mass properties, structures, geology and hydrogeological information, are readily available and sourced from Kaz Minerals Bozshakol.

Subsequently, numerical models such as Slide 2 and RS 2 are employed to analyse the variable data, developing a slope stability model to determine the factor of safety and parametric studies of the pit slope. The results were then evaluated using a probabilistic approach. To determine the reliability indices of the slope design in each domains, the First-Order Reliability Method integrated spreadsheet developed by (Low & Tang, 1997) is utilised.

Additionally, the analysed results are then used to identify areas where there is high risk of failures within the geotechnical sectors. The methodology phases of this study are illustrated in Fig. 1.3, showcasing the research task conducted concurrently whenever possible. Overall, this study employs a critical approach to thoroughly examine geotechnical domains through reliability analysis, utilizing various techniques to assess slope stability and potential rock instabilities.



The data required for this thesis will be obtained from Kaz Minerals Bozshakol, This will be followed by a comprehensive review of existing research based on slope stability analysis and reliability approach used in rock engineering

During the second phase of this study, the input data will be numerically examined to determine possible failures in the selected sector of the pit. Later, the probabilistic technique will employ the deterministic results to estimate the reliability indices.

The third phase of this study will entail the use of Excel spreadsheet developed by Low and Tang. This approach is also known as the First Order Reliability Method (FORM).

The last phase of this investigation will produce a reliability indices of the geotechnical domains in the selected sectors. This will help to the study identify areas of the pit where the rock mass is reliable for mining operations. While developing the geotechnical indices, the validity of the obtained results will be double-checked with the field reports to prevent errors.

Figure 1.2: Schematic flow chart of the research

## 1.4 Originality of the Thesis

This study intends to explore the application of reliability analysis in evaluating slope stability in the context of open pit design at Bozshakol mine. In contrast to previous studies conducted in other fields, the study will use a field-based data from the specified case study area, as well as addressing the limitations of data unavailability restrictions from mining industries. The incorporation of the numerical and reliability approach will be employed for optimizing safety of mine personnel, properties, and profitability. Overall, the research will contribute to a better understanding of slope stability analysis in open pit slope design by incorporating real-world data.

## 1.5 Significant of the Thesis

In the mining field, this study will provide both practical and scientific benefits. Judging from a scientific perspective using the reliability approach, the study will integrate multidiscipline from various field of studies, such as geology, geotechnics and rock engineering. Therefore, this interdisciplinary approach will contribute significantly to the advancement of these fields and foster better collaboration among them. Hence, from a practical perception, the application of the probability technique will help to capture the unique properties in the design of pit slope. This will improve the assessment, reliability and accuracy of pit slope design as a means of overcoming slope instabilities challenges. Furthermore, geotechnical experts and scholars will gain valuable insight into understanding and considering complex boundaries of rock properties and uncertainties. Finally, the outcome of the reliability approach will serve as a basis for conducting further studies on pit optimization and risk analysis. This will enhance the understanding of potential risks associated with slope stability and aid in developing effective strategies in mitigating them.

## 1.6 Scope of the Thesis

The primary focus of the present study is to assess the reliability indices of selected geotechnical sectors in various domains. While several probabilistic techniques, including point estimation methods, performance approach, second-order reliability method, and others have been employed in previous research in reliability analysis, this study will utilise the first-order reliability method based on the case study area. Hence. other reliability methods and specific type of uncertainty, such as numerical characteristics, will not be taken into considerations.

# CHAPTER TWO

## 2. LITERATURE REVIEW

### 2.1 General Concept and Principle of Pit Design

The structurally controlled failures that result from the discontinuities associated with the rock mass properties are the main factors that affect how geotechnical parameters are designed. As past studies have neglected to consider the inherent variations encountered during pit expansion, these challenges will continue to persist as the most significant concerns in the field of rock engineering (Phoon & Kulhawy, 1999a, 1999b). Despite this, several authors have taken advantage of this chance to broaden their understanding of the significance of slope design concerning rock mass structural features (Cylwik, Ryan, & Cicchini, 2011; Grenon & Hadjigeorgiou, 2010; Stead & Wolter, 2015; Wyllie, 2014). Fig. 2.1 presents an illustration of the primary key input factors that are utilized in the rock engineering design process. These parameters include field data collecting and laboratory analysis.

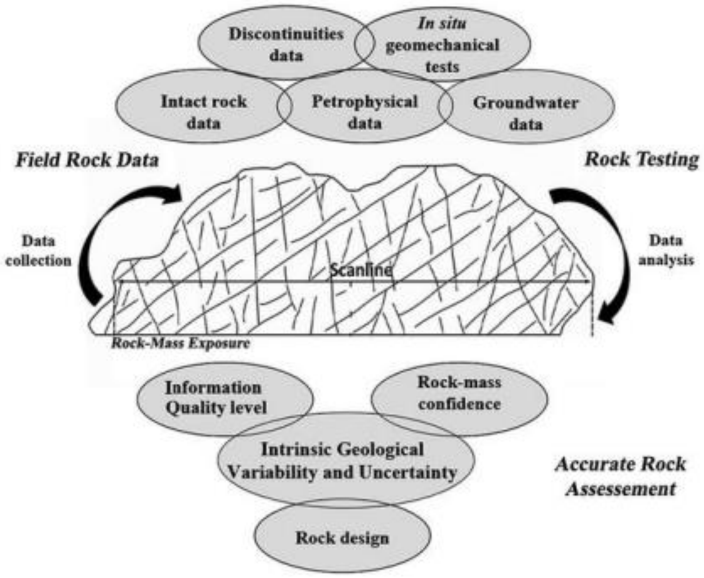


Figure 2.1: Rock mass parameters in rock engineering (Chaminé, Afonso, Ramos, & Pinheiro, 2015)

### 2.2 Slope Stability Analysis

When there is an abrupt occurrence of slope failure, the terms collapse and failure are frequently utilised conversely in rock engineering. As described by Call, Cicchini, Ryan, and Barkley (2000) they are characterised as disastrous situations which could lead to the deviation of rock mass displacement accelerating from its original pit configuration. As a result, mining activities will continue to have an impact on the orientation of the pit slopes. The presence of uncertainties in the design process, such as inadequate slope angles or inconsistency in geotechnical data, and the unpredictability of natural phenomena such as seismic activity, rainfall, and others, are the main factors contributing to pit failures.

Furthermore, if possible, it is exceedingly difficult to maintain stable slopes with significant depth and height in rock that is extremely hard and strong. Hence, slope stability is fundamentally determined by several factors, such as, presence of groundwater, frictional angle, cohesiveness and many others (Armstrong, 1990; Prakash, 2009). Nevertheless, it is crucial to carefully determine the angle of the slope to avert the risk of rock formation collapsing.

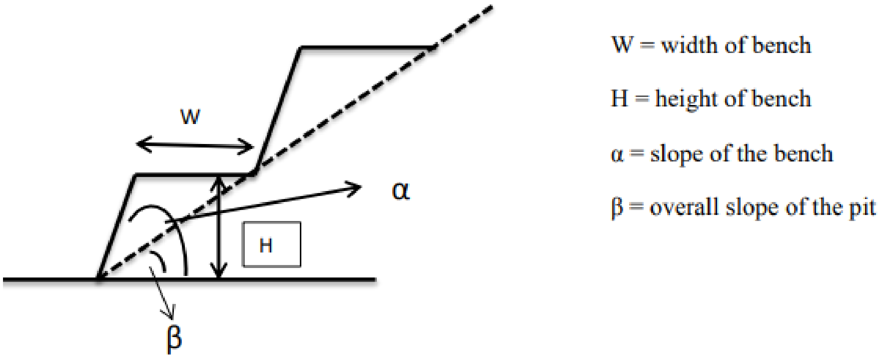


Figure 2.2: Open pit design parameters (Singh, 2005)

## 2.3 Failure Mechanism

Pit slopes have four distinct modes of failure, which are planar, circular, wedge and toppling failure.

### *2.3.1 Wedge Failure*

They occur as a result of intersecting points of two discontinuities on the surface of the slope, where their points become visible as daylight. The factors that govern these phenomena include the frictional angle and line of conjunction of the slope face.

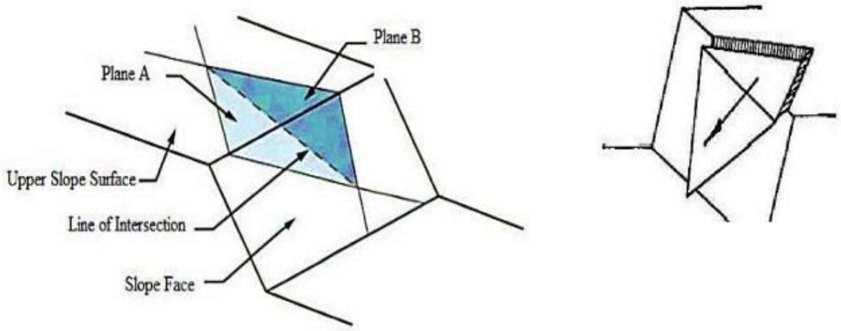


Figure 2.3: A typical wedge failure, after Evert Hoek and Bray (1981)

### 2.3.2 *Planar Failure*

Planar failure is the commonly and the relatively straightforward type of failure often observed on benches, which are horizontally formed during the excavation of mining activities. They occur when a discontinuity, such as joint or fault acts as a sliding plane along which materials above it moves downslope. The failure surface tends to form a straight path due to the nature of the discontinuity, which can include joint sets, flat bedding surfaces, or other weaknesses in the rock.

Hoek and Bray (1981) identify several specific conditions that can trigger planar failures. One condition is the slope crest distance, which is the highest point of the slope, and the toe which is the lowest point. If this distance is significant, it can create a potential for failure due to the increased stress and load on the slope. Another condition is the orientation of the weakness plane, which should strike towards the crest of the slope. This means that the direction of the discontinuity’s strike should be toward the highest point of the slope.

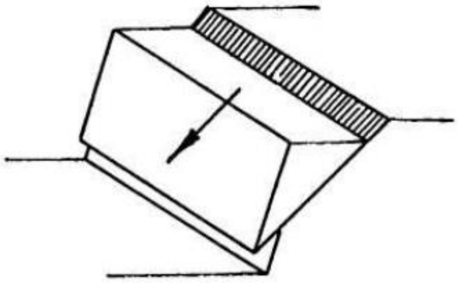


Figure 2.4: A typical plane failure, Coates and Yu (1977)

### *2.3.3 Toppling Failure*

They occur when rock blocks experience instabilities along nearly vertical joints. The severity of these failures can vary ranging from minor incident to major events, depending on how stable the blocks are. Furthermore, they involve due to the rotation of rock blocks around a rigid base, which is often observed on hilly terrains. They are illustrated in Fig. 2.5, where the blocks are shown pivoting due to gravitational pull, resulting in potential instability and failure.

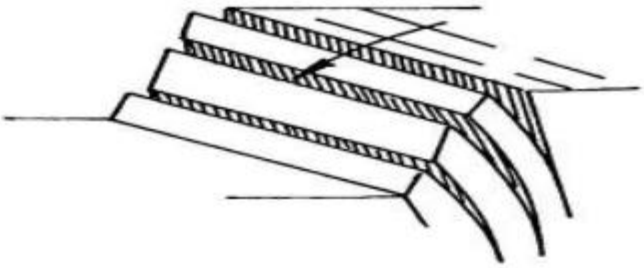


Figure 2.5: Failure mode under the influence of toppling, (Amini & Ardestani, 2019)

### *2.3.4 Circular Failure*

In many cases, soft rock formations contain specific features that make them prone to failure, such as joint sets or structural characteristics that can impact their stability. However, in some instances, when soils are excavated from slopes, these mechanical properties may not be clearly defined or dominant. This can happen when the soil is taken from a slope that lacks distinct structural features or when the joint sets within the soil slope are poorly characterized.

In many cases, soft rock formations contain specific features that make them prone to failure, such as joint sets or structural characteristics that can impact their stability. However, in some instances, when soils are excavated from slopes, these mechanical properties may not be clearly defined or dominant. This can happen when the soil is taken from a slope that lacks distinct structural features or when the joint sets within the soil slope are poorly characterized. The lack of clear structural features or poorly characterized joint sets can make it difficult to accurately predict the failure mode and potential risks associated with soil slopes, and may require additional investigation and analysis to ensure proper slope stability and safety.

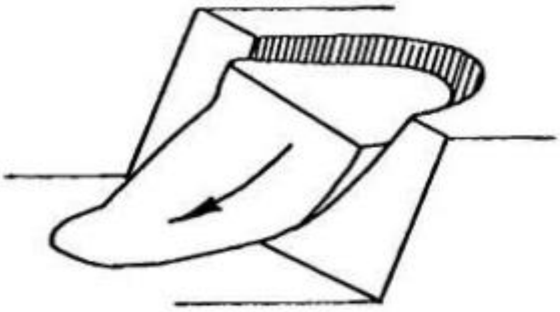


Figure 2.6: Slope failure under circular failure mode, Evert Hoek and Bray (1981)

## 2.4 Geotechnical Domain Modelling

The geological model serves as a comprehensive representation of the mineral deposit, providing a visual depiction of its characteristics in three dimensions. It offers valuable insights into the presence and distribution of various mineral types along the pit wall, which can help in understanding the behaviour and reactions of these minerals during excavation. To enhance the understanding of how these minerals will interact with the surrounding rock during excavation, they are categorized into different domains based on their lithology (the physical characteristics of rocks), as well as the degree and type of alterations they have undergone. This classification allows for a more detailed analysis of the mineral deposit, taking into account the specific properties and characteristics of the minerals in different areas. The geological domains depicted in Figure 2.7, as outlined in the research by Read and Stacey (2009), include structural geology, geology, hydrogeology, and rock mass properties. Structural geology involves the study of the deformation and arrangement of rocks, including features such as faults, folds, and fractures. Geology encompasses the overall composition and characteristics of rocks, including their mineralogy and texture. Hydrogeology focuses on the presence and movement of water within rocks, which can affect the stability of the mineral deposit. Rock mass properties encompass the physical and mechanical properties of the rock mass, such as its strength, stability, and deformation behaviour. By categorizing the minerals into these domains based on their lithology and alterations, the geological model provides a comprehensive understanding of the mineral deposit and its properties. This information is crucial for mining and excavation operations, as it allows for better planning and management of the extraction process, taking into consideration the behaviour and characteristics of the minerals and the surrounding rock mass.

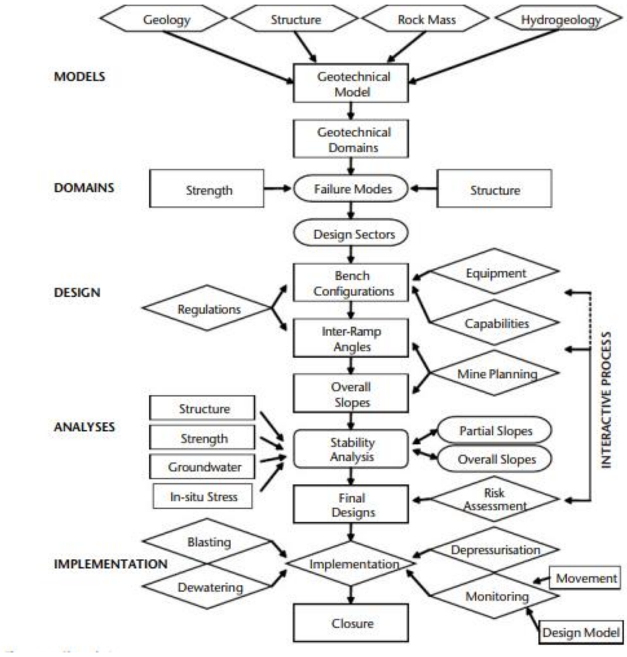


Figure 2.7: An illustrative analysis of geotechnical domain, Read and Stacey (2009)

## 2.5 Geotechnical Pit Slopes Design Methods

Today, various approaches are utilised in the designing and analysing pit slopes in rock engineering. They include observational, empirical, numerical, and analytical methods. According to Jing and Hudson (2002) findings confirmed that these categorized methods are also divided into four (4) approaches as depicted in Fig. 2.8. The mechanisms connected to each level however, determine how these categorized methods are carried out and maintained.

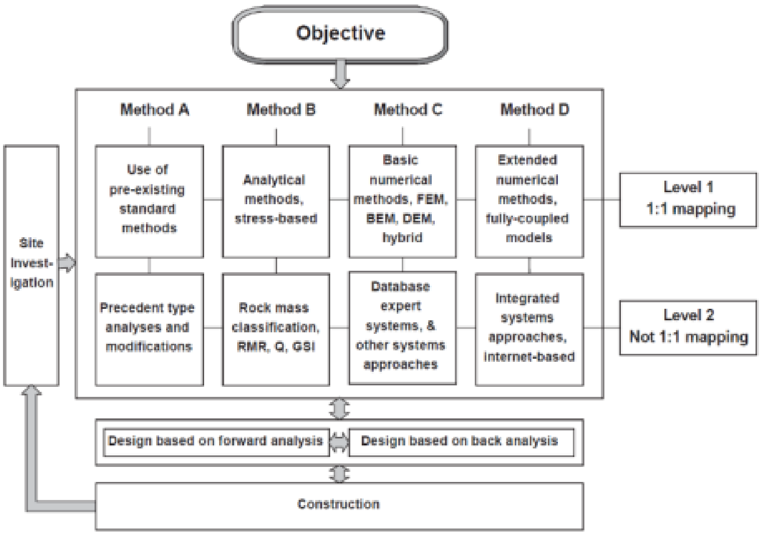


Figure 2.8: Various methods used in pit slope design and analysis

### *2.5.1 Empirical Methods*

This method is used in rock engineering for assessing slope design at the initial assessment stage are known for their simplicity and widespread use. These methods rely on information obtained from previous experiences and field studies, and are based on critical judgment (e Sousa, Vargas Jr, Fernandes, & Azevedo, 2012). They are considered to be first-hand approaches for evaluating the mechanical characteristics of rock mass, such as the deformation modulus and rock strength. These methods also provide detailed quantitative descriptions of rock mass characteristics, which are crucial in evaluating slope stability analysis. However, it's important to note that these methods are not based on traditional models, but rather on comparing and contrasting data from previous studies or projects. As a result, geotechnical engineers must exercise extreme caution when selecting and applying these methods to ensure that the same field conditions used in the assessment are applicable to the specific site under consideration. This requires careful consideration of site-specific factors and conditions to avoid potential inaccuracies or misinterpretations.

There are several types of rock mass classification methods currently used in rock mechanics, including NGI Q-System (Barton, Lien, & Lunde, 1974), Rock Mass Rating (Bieniawski, 1989), and GIS (E Hoek, Kaiser, & Bawden, 1995). Among these, the slope mass rating (SMR) method, which is based on the Q-system and incorporates the method derived from rock mass rating and the Q-slope (Bar & Barton, 2017), is considered the most appropriate for estimating rock slope stability. SMR takes into account various factors such as rock mass quality, structural conditions, and slope geometry, making it a comprehensive approach for evaluating slope stability. However, it's worth noting that empirical methods, including SMR, are typically used only at the preliminary stage of slope design to estimate slope stability performance and identify potential failure modes associated with the rock mass. These methods serve as useful tools for providing initial assessments but may require further refinement and validation through additional investigations and analyses to ensure accurate results. Geotechnical engineers should use these methods as part of a comprehensive slope design approach, considering multiple factors and using professional judgment to ensure slope stability and safety (Salmi & Hosseinzadeh, 2015).

### *2.5.2 Analytical Methods*

Analytical methods provide a variety of applied mathematical models that can all be solved in closed form (Nikolić, Roje-Bonacci, & Ibrahimbegović, 2016). Despite their applicability, they have certain limitations due to their reliance on field conditions and assumptions. These include the extent of the failures within the rock mass, locations where it occurs, the presence of localized uniform shear stress, and the presence of rigid block displacement. If these fundamental hypotheses regarding rock slope analysis are true, it can be justified that the simplified analytical method yields accurate estimates in the complex numerical model (Read & Stacey, 2009).

In addition, these conventional slope stability analysis techniques are generally accepted by well-respected experts due to their ease of use and reliability (Yong, Li, Ye, Huang, & Du, 2016). However, in evaluating geotechnical problems, the limit equilibrium method (LEM) are widely accepted by experts for investigating the moment of forces under certain failure conditions (Stead, Eberhardt, Coggan, & Benko, 2001). However, they are utilized to learn more about the strength of materials as opposed to stress-strain behaviour research. Consequently, unlike numerical models, they are also used to estimate the factor of safety (FoS) without any idea of rock mass deformation on the pit slope. Even today, numerical methods like finite element analysis are used within the limit equilibrium framework to solve geotechnical problems like slope stability (Krahn, 2003).

### *2.5.3 Numerical Methods*

In recent years, experts in rock engineering have turned to highly advanced computing abilities in conjunction with numerical modelling to find solutions to slope stability issues (Stead et al., 2001). However, predicting their behaviour has become extremely difficult to quantify because of the variability in rock mass properties, such as discontinuity, homogeneous nature, anisotropy, and many others (Nikolić et al., 2016). For this reason, numerical techniques are now being employed to resolve complex rock mechanics problems where closed-form solutions do not exist. These methods also use complicated math equations, like partial differential equations (PDEs), to solve problems that cannot be solved analytically with nonlinear methods. One benefit of numerical methods is that they work better than analytical methods. This is because numerical methods consider how rocks behave and help estimate the stress caused by pit slope expansion. Rock engineering relies on various numerical methods to analyse and predict the behavior of rocks in different scenarios. Four commonly used methods are the finite difference method (FDM), boundary element method (BEM), discrete element method (DEM), and finite element method (FEM). Among these methods, the finite element method (FEM) is often preferred due to its accuracy and ease of use when assessing slope instability. The FEM divides the rock mass into smaller, interconnected elements and uses mathematical equations to calculate the behavior of these elements. It can handle complex geometries and boundary conditions, making it suitable for analysing rock slopes with irregular shapes and varying material properties.

### *2.5.4 Observation Methods*

The frequency of rock failures in open-pit environments has been on the rise, necessitating the implementation of robust monitoring and warning systems to predict and mitigate rock instability. It is crucial to understand the time-dependent behaviour of slopes in order to effectively monitor them and take necessary precautions. To this end, several monitoring tools have been developed by various authors to further elucidate the complexities of rock mass failures. With the advent of displacement monitoring techniques and models, it is now possible to determine slope failures in open-pit mines and anticipate any movements that could pose risks to personnel and equipment. These monitoring techniques have evolved from extensive research and advancements in technology, providing valuable insights into the behaviour of slopes and aiding in the prevention of potential disasters. Researchers have put forth the hypothesis that slope failures do not occur suddenly. Early studies by Terzaghi (1950) indicated that landslides would be more effectively predicted if warning signs were noticeable before their occurrence. Evert Hoek and Bray (1981) later proposed about the unawareness of slope failure. This highlights the importance of establishing a reliable monitoring system that can measure and detect slope deformation, enabling timely action to prevent catastrophic events.

While the primary goal of slope monitoring is to ensure the safety of personnel and equipment, understanding the behaviour of rocks can also have implications for pit optimization. E Hoek, Rippere, and Stacey (2000) emphasized that a comprehensive understanding of slope behaviour can aid in optimizing pit design and operational strategies. By gathering data on the behaviour of rocks during monitoring, it is possible to refine pit designs, reduce risks, and improve mining operations.

Based on the findings of previous studies, the objectives of pit slope monitoring are multi-fold. First, it aims to protect both mining equipment and workers from potential slope failures. Early detection of slope instability can trigger timely responses, such as evacuation or re-routing, to mitigate risks. Second, it aims to provide advance notice of potential problems so that plans can be adjusted to lessen the impacts of slope failures. This can include changing mining sequences, adjusting blasting patterns, or implementing additional support measures. Third, it aims to collect geotechnical data that can help understand the causes of slope failures, develop effective solutions, and improve future designs. This data can aid in identifying geotechnical parameters, analysing failure mechanisms, and developing better strategies for slope stability (Stacey, 2007). Monitoring slope failure in mining operations should not solely rely on surface movement. While surface movement can provide valuable information, it may not be sufficient to fully understand the behaviour of slopes. Therefore, monitoring systems should incorporate multiple methods, such as inclinometers, piezometers, and geodetic surveys, to track various parameters that can indicate slope instability.

### *2.5.5 Survey Monitoring*

In the past, mining operations relied on manual observation techniques using total stations and traditional survey systems to determine target points. These methods required human operators to physically measure and record data, which could be time-consuming and labour-intensive. However, with the rapid advancement of technology, there has been a significant shift in the mining industry towards automation and robotics. They use prisms that are subjected to regular surveys to obtain feedback information in the form of data. These data are analysed, and their previous reviews are examined in greater depth.

The disadvantages of using a conventional monitoring system include that it requires much human effort, is time-consuming, and is prone to making mistakes. One of the advantages of using an automated system is that it allows for continuous surveying, which increases the amount of data collected, and enables more frequent inspection of particularly hazardous areas. However, the obtained data are assessed easily than the results of a conventional survey. When there is significant movement in the pit, users will receive feedback alerts on their monitoring and mobile devices in the form of emails and text messages. The notifications will be activated when the automated systems integrate alarm systems with customizable tolerance levels.

## 2.6 Uncertainties in Geotechnical Engineering

The process of pit design in rock engineering today involves evaluating various parameters, such as rock mass properties, loads, and geometry, which are critical for ensuring the stability and safety of rock structures. However, conducting rock mass investigations can be challenging due to the inherent uncertainties associated with geological formations, rock properties, and data limitations. These uncertainties can significantly impact the design decisions and outcomes of a project. One of the primary sources of uncertainty is the model uncertainty, which refers to the reliability of the selected model in representing the complex behaviour of rock masses. Different models may have varying levels of accuracy and applicability depending on the specific geological conditions and rock types (Baecher & Christian, 2005).

Therefore, the choice of model can introduce uncertainty in the design process, and careful consideration is required to select an appropriate model that best represents the rock mass behaviour. Furthermore, the quality and representativeness of the data collected during field investigations, such as rock core samples and geophysical measurements, can impact the accuracy of the characterization results. Factors such as measurement errors, sampling bias, and spatial variability can introduce uncertainties in the estimation of rock mass properties.

To mitigate these uncertainties, various methods can be employed during the project design process. These may include sensitivity analysis to identify the most critical parameters and assess their impact on the design outcomes, statistical analysis to quantify the uncertainty using techniques such as coefficient of variation, $(\sigma/\mu)$, and incorporating safety factors or margins to account for uncertainties in design calculations (Müller, Larsson, & Spross, 2014). Additionally, obtaining more reliable data through rigorous field investigations, laboratory testing, and site-specific monitoring can help reduce uncertainties associated with rock mass properties and improve the accuracy of design decisions.

$$COV_{tot}^2 = COV_{sp}^2 + COV_{err}^2 + COV_{\mu}^2 + COV_{tr}^2 \qquad (1)$$

Given that $COV_{sp}^2$ represent the characteristics of the inherent variability of rock mass, $COV_{err}^2$ is the measurement errors that occurs randomly, $COV_{\mu}^2$ pertains to determining the mean value of the rock mass properties the rock mass properties mean value determination, and $COV_{tr}^2$ is the estimated desired property for the bias conversion.

The incorporation and assessment of the presence of uncertainties determines the type of methods that will be adopted in estimating slope stability analysis, that is, deterministic or probabilistic (Ceryan, Kesimal, & Ceryan, 2018). For estimating the factor of safety from an average datasets gleaned from laboratory and field reports, the deterministic approach employs the limit equilibrium method. However, the method is easy to implement but tends to ignore the possibility of errors and variations in the rock mass properties. This method is also considered conservative because it allows for the determination of various safety factor values under varying conditions in slope design analysis. In addition, overdesigning in slope stability analysis is regarded as the most effective methods for mitigating the adverse effects of uncertainty and increasing relative reliance (El-Ramly, Morgenstern, & Cruden, 2002).

## 2.7 Concepts of Reliability Analysis Approach

The primary drivers for the advancement of mining projects throughout the mine cycle are the strategic plans of mining industries, which serves as a guide during the design stage. Rock engineers are responsible for developing a practical and workable design that satisfies project goals and limitations. However, the reliability of pit slope design can be affected by various factors, including the geometry of the orebody, rock mass conditions, and financial constraints during mineral extraction. When pit slope design objectives are not met during the development phase, a probabilistic approach, known as reliability-based design, is implemented to optimize pit performance and manage risks. These approaches are widely used to identify uncertainties and assess variable impacts on rock mass structural factors, while prioritizing safety and achieving mineral resource benchmarks. Despite their benefits, these methods have not been widely adopted in geotechnical engineering, as they require familiarity with statistical concepts and additional time compared to conventional design methods.

By comparison, the reliability analysis outperforms the deterministic approach based on their effectiveness. They serve to incorporate input parameters with uncertainty by utilizing a logical approach to assess risk design. This allows for the determination of failure probability $(p_f)$ for specific failure modes, where "failure" denotes the complete breakdown of rock mass disintegration. In recent times, this method has replaced the traditional factor of safety (FoS) approach as the assessment of probability of failure (PoF) (Nilsen, 2000). One of the notable advantages of this approach is its ability to account for the inherent variability and uncertainties in rock mass properties, which cannot be determined by other means.

Renowned authors in rock engineering have presented a diverse range of probabilistic methods for estimating slope failures (Basahel & Mitri, 2019; Gravanis, Pantelidis, & Griffiths, 2014; Grenon & Hadjigeorgiou, 2010; Irigaray, El Hamdouni, Jiménez-Perálvarez, Fernández, & Chacón, 2012; Obregon & Mitri, 2019; Tatone & Grasselli, 2010). These studies have garnered significant attention in the quest to quantify the probability of failure along slip surfaces associated with discontinuities in rock masses. One notable aspect of these methods is that they provide a consistent and comprehensive measure of risk by incorporating additional uncertainty information, as the likelihood of failure remains the same regardless of the specific circumstances with regards to safety (Sayed, Dodagoudar, & Rajagopal, 2010). Various practical tools are commonly utilized in reliability analysis: they include, the second-order reliability method, first-order second method, first-order reliability method, point estimation method and many more.

### *2.7.1 Second Order Reliability Method (SORM)*

Breitung and other esteemed researchers developed this method to tackle asymptotic analysis (Karl Breitung, 1984, 1989; K Breitung & Hohenbichler, 1989). The calculation steps for this method are illustrated below using a hypothetical scenario with three arbitrary dimensions:

1. Assuming that common normal space, $P(z_1, z_2, z_3)$, is denoted as the performance function. And the reliability index is calculated from Eqn. (4), therefore, the reliability index $\beta$ will now be expressed as $(z^{*\prime} z^*)^{1/2}$, where $z^* = (z_1^*, z_2^*, z_3^*)'$ is defined as the design point.
2. Then the estimated value of the vector gradient at $z^*$ will be expressed as:

   $$\nabla P(z^*) = \begin{cases} \partial \mathrm{P}(z^*)/\, \partial z_1 \\ \partial \mathrm{P}(z^*)/\, \partial z_2 \\ \partial \mathrm{P}(z^*)/\, \partial z_3 \end{cases} \tag{2}$$

   Thus, gradient vector magnitude will then be:

   $$||\nabla P(z^*)|| = [\nabla P(z^*)\ \ \nabla P(z^*)]^{1/2} \tag{3}$$

3. This will therefore express the evaluated probability of failure as:

   $$p_f = \Phi(-\beta)|J|^{-1/2} \tag{4}$$

   where $J$ is expressed in a 2 × 2 matrix:

   $$J = \begin{bmatrix} 1 & 0 \\ 0 & 1 \end{bmatrix} + \frac{\beta}{||\nabla P(z^*)|} \begin{bmatrix} \partial^2 \mathrm{P}(z^*)/\, \partial z_1^2 & \partial^2 \mathrm{P}(z^*)/\, \partial z_1\, \partial z_2 \\ \partial^2 \mathrm{P}(z^*)/\, \partial z_2\, \partial z_1 & \partial^2 \mathrm{P}(z^*)/\, \partial z_2^2 \end{bmatrix} \tag{5}$$

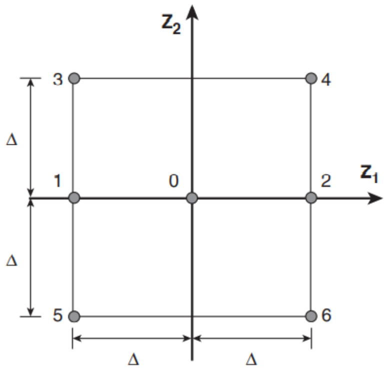


Figure 2.9: A graphical representation of the typical central finite difference scheme

Using the central finite difference scheme depicted in Figure 2.11 above, equations (7) and (10) can be quantified. Given that, $P_i$ to represent the result of the performance function assessed at node $i$. The first derivative is then calculated as follows:

$$\frac{\partial \mathrm{P}}{\partial z_1} \approx \frac{P_1 - P_2}{2\Delta} \tag{6}$$

The estimated second derivative will be express as:

$$\frac{\partial^2 P}{\partial z_1^2} \approx \frac{P_2 - 2P_0 + P_1}{\Delta^2} \tag{7}$$

Therefore, the mixed derivation will then be estimated as:

$$\frac{\partial^2 P}{\partial z_1 \, \partial z_2} \approx \frac{P_4 - P_3 - P_6 + P_5}{4\Delta^2} \tag{8}$$

### *2.7.2 Monte Carlos Simulation Method*

In the Monte Carlo simulation method, the mean and standard deviation of a function of random variables are calculated repeatedly using randomly chosen points assessed for the component variables. However, running a large number of trials with this simulation will necessitate not only a fast computer but also a ready-made program that can automatically perform repetitions and accumulation for a given function. For some complicated functions, it might be very challenging to create (A. H.-S. Ang & Tang, 1984; A. S. Ang, 1975). The advantage of using this methodology is that the dependent random variables' complete probability distributions can be obtained. Its disadvantages include the potential for the input to be more effective than the output (assumed) and the typical requirement for a lot of computer time. Additionally, each simulation case needs to be handled differently for the best possible outcome.

## 2.8 Existing Reliability Approach for Pit Slope Analysis

The study of reliability analysis in rock engineering has received much attention in open pit design. Several studies have been conducted over the last few decades using this technique to determine the factors that affect the geotechnical design parameters in slope stability. Further studies have raised several concerns about the inherent variability and characteristics of naturally occurring rock mass formation being the primary source of these variables, which could significantly impact slope instabilities in pit slope design (Aladejare & Akeju, 2020). In most cases, they occur when the pit geometry changes continuously during pit expansion, i.e., when they encounter new geological domains. It makes them challenging to manage and could result in pit error design. In addition, these difficulties are influenced by the presence of groundwater conditions, the presence of significant fault zones, the locations of lithological boundary conditions, and a variety of complex rock mass properties and structures that are implicitly related to the inherent variabilities in the geological and geotechnical domains (Musah Abdulai & Mostafa Sharifzadeh, 2019). On the other hand, the presence of these uncertainties also results to frequent unpredictability of rock mass displacement during mining activities, potentially putting lives and property at risks (Giacomini et al., 2020; Thoeni, Giacomini, Sloan, Lambert, & Casagrande, 2011). Despite this, eliminating these uncertainties will be extremely difficult, but a systematic approach could reduce them to an acceptable level. Despite this, eliminating these uncertainties will be extremely difficult, but a systematic approach could reduce them to an acceptable level (Baecher & Christian, 2005).

To address these issues, pit design decisions are primarily incorporated into the geological domains to quantify these uncertainties. Regrettably, existing design techniques have yet to adopt reasonable justifications for dealing with these uncertainties and assessing their effects on potential failures in slope stability. Therefore, traditional methods are used to assess and analyse the unquestionable challenges associated with pit slope designs by determining the factors of safety (FOS) using conservative values. To ensure a stable and reliable pit slope, the factor of safety is estimated and assigned for each pit bench to assess the potential structural failures such as the wedge, planar, toppling, and circular controlled by the presence of geological discontinuities in the rock mass (Lana, 2014). However, one of the drawbacks of this approach is that the outcomes of the rock slope design will not yield the best outcomes. The reason for this is that the application of the factor of safety in slope stability is said to be deterministic, which is more stable when it is greater than one and unstable when it is less than one.

Despite their flexibility in slope stability design, many researchers have expressed concerns about their shortcomings (Duzgun et al., 2003; Jimenez-Rodriguez et al., 2006; Phoon, 2008; Phoon & Kulhawy, 1999b). They argued that when using variable input parameters, a deterministic approach does not take into account estimating the degree of uncertainty in the rock mass properties. However, their suggestion later indicated that the results could be confusing because the same factor of safety values only applies to conditions comprising extensive variable nature of inherent variability.

Motivated by the drawbacks above of the deterministic approach, the reliability techniques have recently been employed in most mining environments and rock engineering to address the geotechnical problems dominated by various rock mass uncertainties affecting pit slope safety design (Fenton & Griffiths, 2008; Kirsten, 1983). These techniques have made significant contributions to the geotechnical design by thoroughly investigating the reliability of the stability in the pit slope at the design and analysis stages when estimating these uncertainties. Moreover, they are accomplished by using a variety of random events that occur during slope failures as input parameters for the design of rock slopes. This implies that these random variables, on the other hand, rely on the estimated correlations, level of dispersion, and mean values. As a result, the related geotechnical issues are considered when making risk analysis decisions to determine the probability of failures and reliability indices in various domains of pit slope designs, as well as emphasizing failure outcomes (Hicks & Jommi, 2014).

Many probabilistic methods have been developed for analysing the reliability index and failure probability and addressing rock stability issues (Chowdhury & Xu, 1995; Hasofer, 1974; Vanmarcke, 1977). These approaches are frequently implemented using numerical techniques such as Monte Carlo Simulation or point estimation (Hicks & Jommi, 2014; Low, 2003; Miller et al., 2004). Previous studies highlighted that in evaluating slope stability design, the FOSM approach could be applied based on the probabilistic approach (Christian, Ladd, & Baecher, 1994). Nevertheless, they have been compared with several other techniques, such as PEM and MCS, by considering various sources of uncertainties to establish a fixed factor of safety in underground tunnel supports (Russo, Kalamaras, Xu, & Grasso, 1999).

Later, with the same comparison mentioned above, Johari and Javadi (2012) assessed the reliability of infinite slope stability through a jointly distributed random variables method. Their findings revealed that they contain independent random variables that do not require input variables like mean and variance. However, they have some limitations because they cannot be used elsewhere. Prior to Johari and Javadi (2012) studies on the probabilistic approaches, Mbarka, Baroth, Ltifi, Hassis, and Darve (2010) contradicted the deterministic methods by combining a modified FOSM with other probabilistic approaches such as MCS, SORM, FE, and RSM to predict slope stability failures. It was highlighted that the results of the methods are inconsistent and cannot be used in reliability analysis to identify any critical slip surface in predicting potential failures at a lower factor of safety.

Additionally, Duzgun et al. (2003) employed an advanced FOSM to pinpoint various sources of errors and uncertainties relating to the peak friction angle of rock discontinuities before implementing the studies mentioned above. It implies that the research results were used to incorporate these uncertainties into slope stability designs as random correction factors. However, the reliability of the slope design will not produce accurate outcomes because of the limitations of available data. This limitation is comparable to that of the study of Mbarka et al. (2010), which relies on soil cohesion and friction angles as input parameters.

More recent reliability in slope stability design analysis has been extensively used for models such as PEM, MCS, and RSM to evaluate the probability of failures in various rock engineering aspects and to indicate the interrelationship of non-linear multiple variables. The studies proposed by Onisiphorou (2010) when combining PEM and MCS, show that increasing MCS will not significantly improve the reliability analysis results due to fluctuations in safety factors caused by a small degree of random variables to produce fewer realizations results. In contrast to the MSC models, the PEM offered a quicker and simpler computation method. Later, González Shand and Sepúlveda Zamorano (2015) combined FORM and the Taylor series to calculate the likelihood of slope failure following critical analysis, with or without making any assumptions. When compared to the findings of Onisiphorou (2010), they concluded that the degree of confidence concerning the input parameters with higher values contains several restrictions. In addition, some studies were conducted to evaluate the reliability analysis using probabilistic pressures in tunnels for a targeted reliability index by considering the coefficient of variance (CoV) and random variables (Yang, Zhou, & Li, 2018). Later, the need for more research was suggested as the best way to determine these uncertainties in the rock mass properties so that design engineers can estimate design parameters methodically rather than subjectively.

Yakubov and Adoko (2020) utilised a deterministic approach to estimate various uncertainties supporting design parameters likely to influence failure probability. According to their study, the outcome can only be accomplished by indicating a desired safety factor. Later, Samui, Kumar, Yadav, Kumari, and Bui (2019) were able to incorporate genetic programming (GP) and Gaussian Progress Regression alongside the FORM to assess slope stability. The comparison of the two models produced higher performance coefficient results. Hence, it is crucial to fully understand the functional relationship between the input parameters to achieve this. Siacara, Beck, and Futai (2020) was able to estimate the random parameters that have the greatest influence on the probability of failures in order to ensure the long-term safety of an earth dam. Chen et al. (2022) also used the BPNN-based FORM and SRLEM to assess reliability in bipolar sliding. According to their findings, the deterministic approach cannot determine the degree of uncertainty associated with slope stability or variation in design parameters. However, the study suggests that it would be preferable for researchers and geotechnical engineers to use numerical simulation software when assessing slope stability reliability.

Since the reliability and probabilistic approaches tend to provide greater accuracy in estimating the uncertainties in rock slope stability, they have become popular in rock engineering. To determine the various uncertainties that could cause a slope failure, various slope stability software such as Swedge, RS2, RS3, and Slide 2, Slide 3, among others, are used in conjunction with the probabilistic approach to determine any potential failures (M Abdulai & M Sharifzadeh, 2019; Deng, Zhao, & Li, 2015; Stead & Wolter, 2015). Therefore, implementing this slope stability software will aid in decision making during slope stability design.

# CHAPTER THREE

## 3. DESCRIPTION OF BOZSHAKOL MINE

### 3.1 Introduction

This chapter will introduce a brief overview of the case study area, stating the mining methods carried out on the deposit. Later, the various geotechnical sectors will be introduced, stating various pit slope instabilities in the domains, which will be the focal point of this study.

### 3.2 Geology of Case Study Area and Reserve

The Bozshakol deposit is an open pit mine located in the northern portion of Kazakhstan at 51° 50' 48" North and 74° 17' 28" East as illustrated in Fig. 3.1. This greenfield was discovered in 1930 by R.A. Borukaev and contains a copper-porphyry deposit. The geological structure of the deposit is composed of a mixture of Cambrian and Ordovician volcanogenic sedimentary rocks, which are overlain from above by siliceous sandy-pebble formations with lenses of Paleogene kaolinite clays and clays, loams, sands, sandy loams, and pebbles.

Figure 3.1: Geological map indicating the case study area (Aben, Orazaliyev, & Suorineni, 2020)

Multiple geological features, such as fault zones, contribute to the formation of the folded-block structure, magmatic complexes, and ore-metasomatic systems in the Bozshakol mines deposit. The ore-bearing areas where the intrusive bodies are confined are found along these fault zones, which run east to northeast along the length of the deposit (mainly granitoid dikes). In contrast, it is hypothesized that the presence of these fault zones significantly impacts the slope stability analysis and design of the pit. Several fault features along the north-eastern strike that causes substantial displacement have been identified, and they are as follows: cracks that are almost latitudinal and correspond to the main crushing zones, their angles of incidence 60-80°, azimuth of incidence 160-190°.

i. northwest direction cracks angles of incidence range from 20° to 70°.

ii. north-eastern cracks, their fall has an azimuth of 310-340° and angles of 50-30°.

However, the region with the greatest increase in mineralization corresponds to the region with the development of pre-ore tectonic processes. Despite this, they possess many regions of fissures available for filling and hydrothermal changes due to subsequent ore deposition. Therefore, the occurrence of ore deposits within this region is typically quite steep, reaching a vertical angle in some cases, and has a slight declination toward the north-northwest.

Table 3.1: The mineral reserve at the Bozshakol deposit (as of 01.01.2020)

| | Units | Balanced | | | |
|---|---|---|---|---|---|
| | | Categories (depending on the stage of exploration) | | | Uncounted |
| | | B | C1 | C2 | |
| General | | | | | |
| Ore | TMT | 144, 153.6 | 751,857.90 | 102,873 | 511,256 |
| Cu | TMT | 476.9 | 2,565.10 | 300.8 | 1,079 |
| Mo | t | 7,689.40 | 18,486.60 | 37,241.70 | 3,745.10 |
| Au | kg | 0 | 0 | 116,145.90 | 41,907.80 |
| Ag | t | 0 | 0 | 2,386.90 | 721.6 |
| Central deposit | | | | | |
| Ore | TMT | 144, 153.6 | 378,828.90 | 102,873 | 112,814 |
| Cu | TMT | 476.9 | 1,334 | 300.8 | 216.7 |

## 3.3 Mining Operations at Bozshakol Mines

The mining operation at Bozshakol mine is currently in operation as the pit dimension is at a top-level depth of 150 meters and measures 4112.7 meters in length. The deposit development is carried out by 10 m ledges, and several intermediates define the boundaries of the pit. In planning the mineral extraction, the Central Pit's borders are separated into four stages: Stage 1 from 2015 to 2021, to the horizon of +40 m; Stage 2 from 2018 to 2024, to the horizon of 0 m; Stage 3 from 2021 to 2033, to the horizon –40 m; and -Stage 4 from 2024 to 2032, to the horizon –80 m. Plate 3.1 shows a general overview of the open pit at the Bozshakol deposit.

Table 3.2: Pit design parameter (as of 01.0.2032)

| Parameter | Unit | Central of the pit |
|---|---|---|
| Length | | |
| on the top | m | 4,294.30 |
| on the bottom | m | 549.4 |
| Width | | |
| on the top | m | 1,112.40 |
| on the bottom | m | 201.7 |
| Bottom measurement | m | -40 |
| Depth (from the maximum surface) | m | 270 |
| Surface area | m^2 | 4,068.20 |
| Bottom area | m^2 | 50.7 |

Figure 3.2: Bozshakol mine open pit overview

## 3.4 Drilling and Blasting Operation at Bozshakol Mines

The Sandvik D55SP is a down-the-hole (DDH) rotary percussive drilling machine used in the pit for drilling operations. On the other hand, the Sandvik DI 550 machine is utilized for flexibility in rough terrains. The "Methodological suggestions for the technological design of mining businesses using an open way of development" are used to estimate the drilling and blasting pattern for efficient mineral recovery. For example, given that:

$$\text{ledger height}, \mathrm{H} = 10\mathrm{m},$$

$$\text{ledge slope angle} = 75^{o}$$

$$\text{limit} = \text{up to } 68^{o}$$

$$w\text{idth of the prism to detect potential failure:}$$

$$\Pi_6 = \geq 2.13m$$

Table 3.3: Symbols for calculation

| | |
|---|---|
| The width of the prism of possible failure | $\Pi_6$ |
| The angle of the slope of the ledge in the working position | α |
| The angle of the slope of the ledge in the non-working (stable) position | φ |

For effective blasting operation, the boreholes are charged and stemmed adequately to produce the desired blasted materials for further size reduction. Both electric and non-electricity devices initiate the detonation of the charged holes. They consist of 500 grams of PT-P or TNT-hexogen checkers. However, before detonation, the blasting crew shifts to a 500-meter safe zone to prevent fatalities. After blasting, the fragmented materials are conveyed by dump truck through the run-off mine to the crushing facility, where they are furtherly reduced to 20-40 mm particle sizes.

## 3.5 Geotechnical Sectors at Bozshakol Mines

The deposit design began with creating a geotechnical database using data gathered from geological wells and laboratory studies. This database was subsequently used to divide the intended pit area into seven geotechnical domains (regions) with comparable or separate mountain range features. These sectors, as indicated in Fig 3.1, contain several geological domains that are distinct from one another, including weathered, fractured, and typical. Nevertheless, they are primarily identified based on their lithology and the fracturing intensity within the geological column. Because of the characteristics of the deposits, it is necessary to consider various slope orientations for the proposed pit slope face within the structural domains.

From a geotechnical standpoint, the deposit's slope parameters were utilized to evaluate the likelihood of failure affecting the slope's stability and the overall pit slope angle. The geotechnical characteristics of the rocks and the type of failure have been divided into distinct domains using this analysis in accordance with the criteria for stability loss. For this reason, it is crucial to evaluate the safety factor and incorporate a deterministic and probabilistic analysis into developing pit slope design parameters to guarantee operational control.

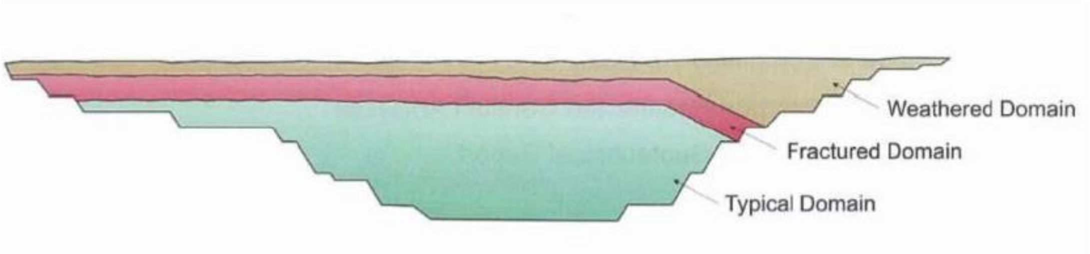


Figure 3.3: A typical domain at Bozshakol deposit

In 2010, more research was undertaken to evaluate the geotechnical design parameters of open-pit mining for a conventional feasibility study. The majority of the geotechnical information used for the feasibility assessment was gathered from 21 wells sunk particularly for geotechnical research. The data that are included are borehole well drilling (pertaining to the interval and structure), geological interpretations, laboratory experiments, and field tests.

## 3.6 Field Data Collection

For the purpose of this thesis, the primary source of data was obtained from the Bozshakol mine, which served as the foundation for the research. The datasets used in this study were meticulously collected and thoroughly analysed to provide crucial insights for the geotechnical department. The aim was to develop a comprehensive geotechnical model that could effectively assess the stability of the pit slope.

The process involved gathering diverse datasets from the Bozshakol mine, which included geological, geophysical, and geotechnical data. These datasets were carefully examined and processed to identify patterns, trends, and anomalies that could impact the stability of the pit slope. Advanced data analysis techniques, such as statistical analysis and numerical modelling, were employed to extract meaningful information and generate valuable results.

Furthermore, the core samples obtained from various exploratory drilling operations conducted within the pit area were subjected to rigorous analysis. The International Society for Rock Mechanics (ISRM) approved procedures were followed to ensure the accuracy and reliability of the findings. These procedures involved laboratory testing of the core samples to determine their physical and mechanical properties, as well as their response to different geotechnical conditions. The results of these analyses provided critical insights into the rock mass characteristics, which were essential for the development of the geotechnical model.

Therefore, the thesis relied on extensive data collection and analysis from the Bozshakol mine, including diverse datasets and core sample testing. These datasets will be utilized to develop a robust geotechnical model that could provide valuable insights for assessing the stability of the pit slope and guiding safe mining practices.

### *3.6.1 Data from Slope Monitoring System*

The displacement of rock mass at Bozshakol mine has been a matter of concern, and efforts have been made to monitor and mitigate potential slope failures. The IBS ArchSAR Georadars A-27 and A-28 have been employed for this purpose, focusing on the first bench located in opposite directions of the pit at the north and south walls. These advanced radar systems have been utilized to detect any signs of instability and provide feedback on the slope conditions. However, the monthly feedback report for the year 2022, generated by these slope monitoring radars, has consistently shown a very low level of risk in sector six. This finding seems to contradict the field reports observed by geotechnical engineers during inspections. Fig. 3.4, illustrates the field report, clearly demonstrates that continuous mining activities within the geotechnical sectors may actually increase the confining stresses in the surrounding environment. The resulting excessive strain energy can potentially lead to rock mass displacement and other stability issues. To better understand this contradiction, the rock mass properties in sector six have been carefully studied. The rock mass rating, which serves as an indicator of the competency of the rock mass, has been found to vary from good to very good. This suggests that the rock mass is generally stable and capable of withstanding the applied stresses. However, despite these positive ratings, there have been concurrent occurrences of rock instabilities in April and August of that year, pointing to a moderate risk in sector six. This discrepancy between the field observations and radar reports raises questions about the actual stability of the rock mass in this sector.

Further investigation reveals that a borehole, labelled as VWP007, has been installed for piezometer monitoring in sector six. Piezometers are devices used to measure water pressure within the ground, and the presence of this borehole suggests the possibility of high levels of water saturation during the winter and rainy seasons. This could potentially affect the behaviour of the rock mass, as water may infiltrate into ubiquitous joints and structural impacts, causing changes in the mechanical properties and triggering slope instabilities.

In light of these findings, it is imperative to conduct a thorough slope stability analysis study in the selected geotechnical domains to gain a better understanding of the situation and establish a correlation with field observations. This study would involve a comprehensive assessment of various factors, such as rock mass properties, stress distribution, water pressure, and geological conditions, to determine the true stability of the slope and identify potential triggers for rock mass displacement. The results of such an analysis would provide valuable insights for developing effective mitigation measures and ensuring the safe and sustainable operation of the Bozshakol mine.

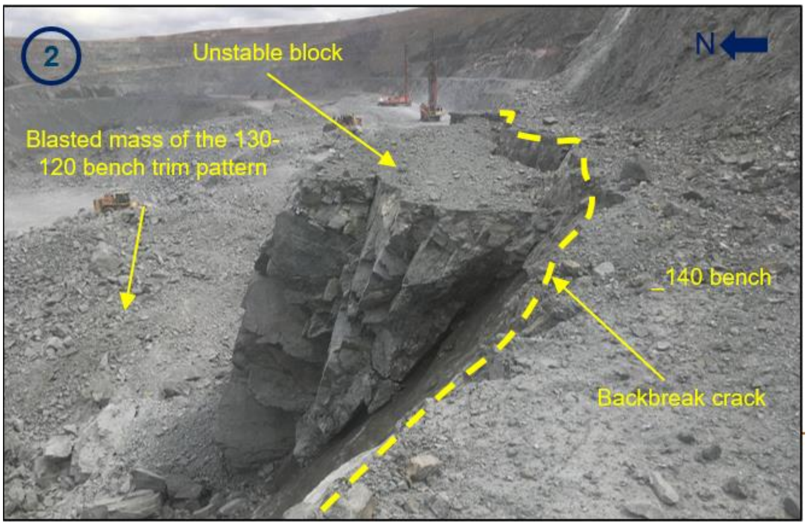


Figure 3.4: Geotechnical risk at Sector 6

## 3.7 Data Description

As shown in Table 3.4, the data selected for this study include rock mass properties (RQD, RMR, Weathering, and so on), intact rock mass properties, various lithologies, and slope design parameters. These data are handled and scrutinized with care by selecting specific geotechnical sectors that will meet the research objectives. The selected data for this thesis will be obtained in Sectors 2 and 6 of the deposit. Figure 3.2 and Fig. 3.3 depict the various geotechnical sectors and RQD of the Bozshakol deposit.

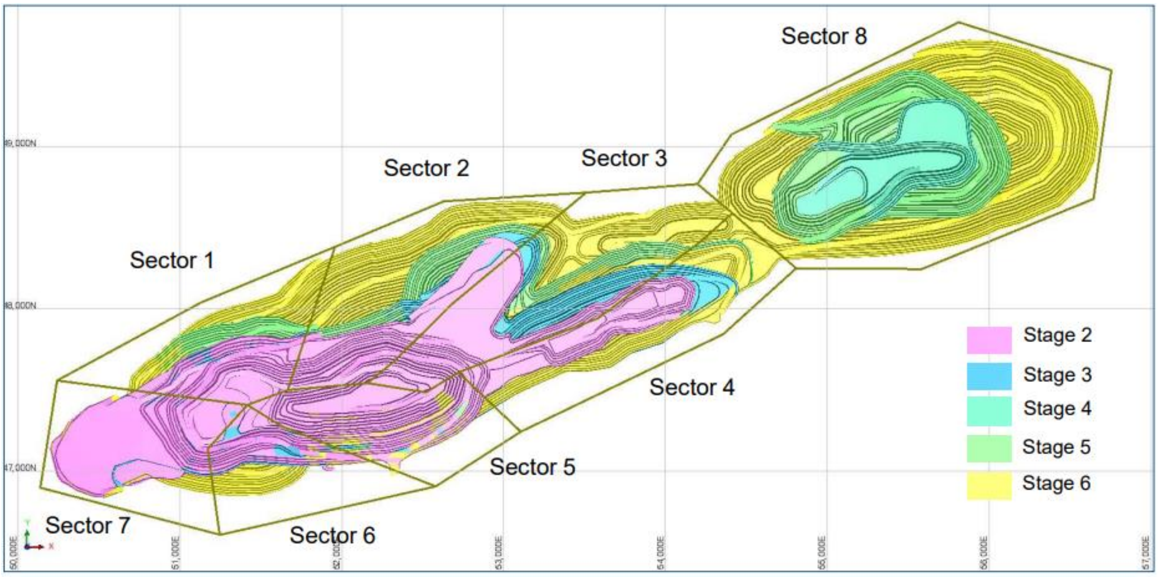


Figure 3.5: Various geotechnical sectors of Bozshakol pit

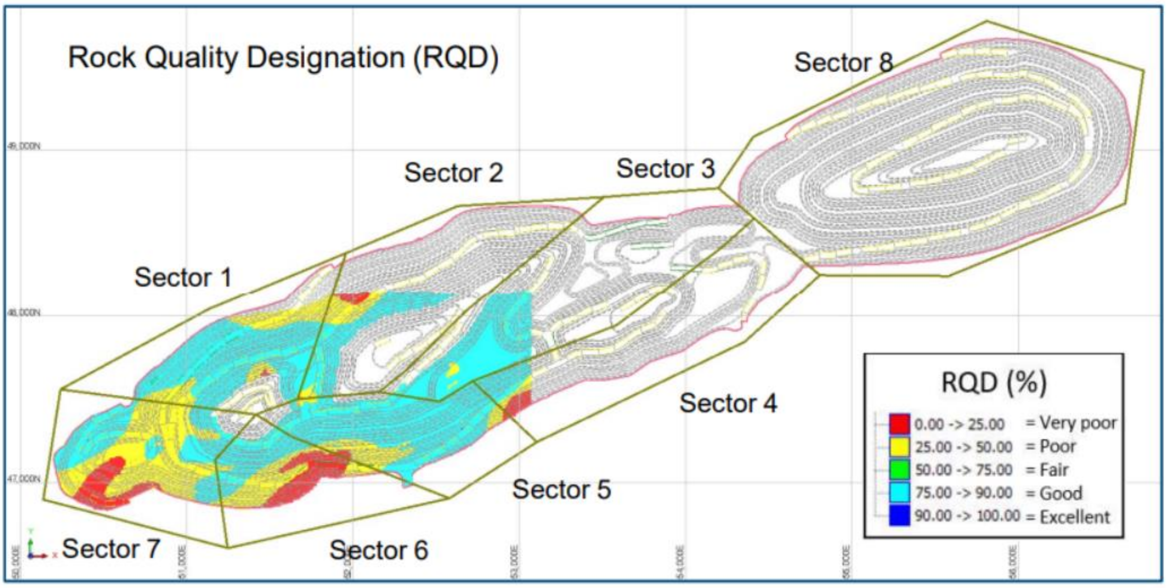


Figure 3.6: RQD of various sectors

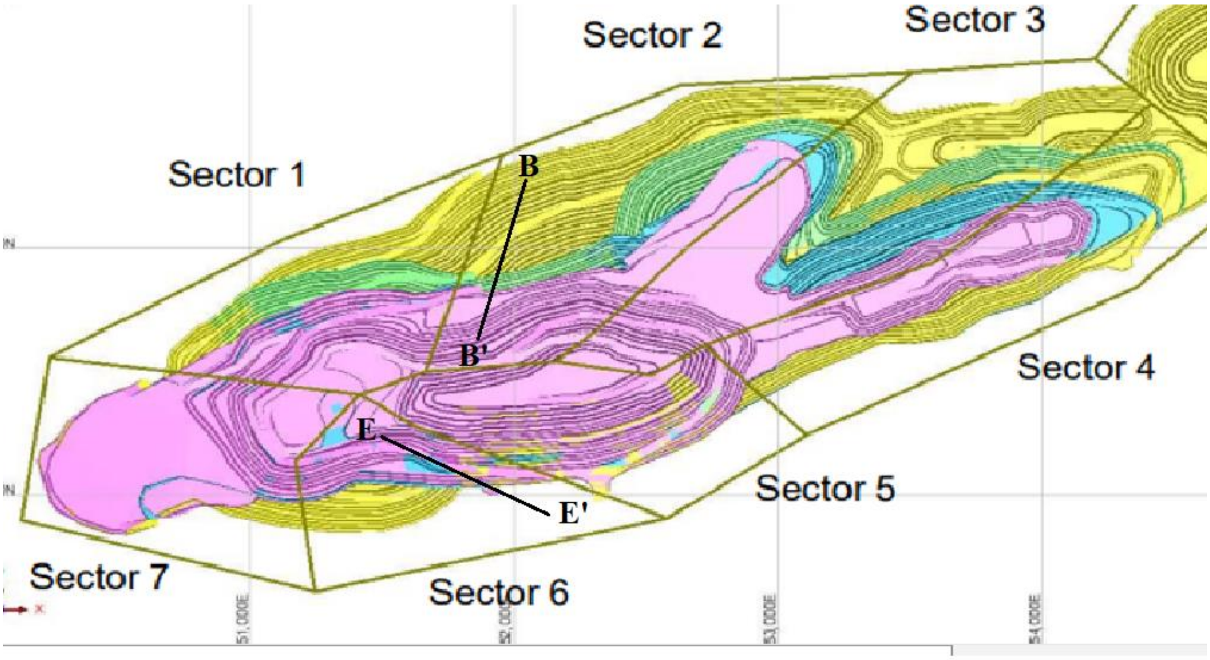


Figure 3.7: The cross-sectional area of sectors 2 and 6 (B-B' and E-E')

# CHAPTER FOUR

## 4. NUMERICAL MODELLING

### 4.1 Introduction

In this chapter, the slope stability analysis was determined by bisecting an area portion of Sectors 2 and 6 containing the preferred slope orientation of the pit. In order to develop the pit models, a two- dimension sections were constructed for each sector using the input parameters provided by the case study area. These input parameters are then commercially incorporated into the Slide 2 mathematical geotechnical and RS 2 simulation models developed by Rocscience Inc. The purpose was to obtain the factor of safety (FoS) on the Slide 2 model and then import it into the RS2 to determine the probability of failures for each case of the geotechnical sectors.

### 4.2 Limit Equilibrium Method with SLIDE 2 (LEM)

Slide 2 is a two-dimensional software that analysed slip surfaces in the pit rock slope using vertical and non-vertical slice limit equilibrium method which Spencer, Bishop, Janbu, and many others developed. It also includes various techniques for analysing steady-state or transient groundwater conditions governed by the piezometric lines. However, the software aids users in simplifying the construction of the pit design, computation, and interpretation to estimate the factor of safety and probability of failure. For the purposes of this study, the Janbu simplified method will be used. The reason for this is because it satisfies the use of force equilibrium better than the Bishop simplified method. This method is also used to assess non-circular failures that may cause slope instability due to multiple formations, complex geometry, seismic forces, and support forces. As a result, when compared to the Bishop simplified method, it produces results for rock engineering project design with fewer convergence issues and more precise factor of safety results. The cross-sectional area such as B-B' and E-E' of the selected sectors is depicted in Fig. 3.7 for the purposes of this study, indicating the various geotechnical domains that will be incorporated using the limit equilibrium method.

### *4.2.1 Procedure of Slope Stability Analysis with Slide 2*

Below are the steps taken to create the model for the selected sectors:

1. The geometry of the model of the selected sectors was delineated based on the pit geometry and topography.
2. After designing the model using the precise coordinates for the bench width, bench height, and slope angles allotted for each of the domains, the model is then closed in a loop for further analysis;
3. Based on the values of the input parameter as illustrated in Table 3.4, the model is therefore assigned according to their boundaries and material properties;
4. The presence of groundwater is then assigned to the model as indicated by the data obtained from the field;
5. Furthermore, the FoS for the sectors are determined by computing the model using the LEM methods, such as Janbu simplified method;
6. The estimation of the FoS for each domain are obtained through back analysis which displays the presence of various slip surfaces provided by the application.
7. The summary of the factor of safety are illustrated in Table 4.1.

### *4.2.2 Results*

Following the analysis of the model, the results of Slide 2 for sectors 2 and 6 and their FoS are illustrated below:

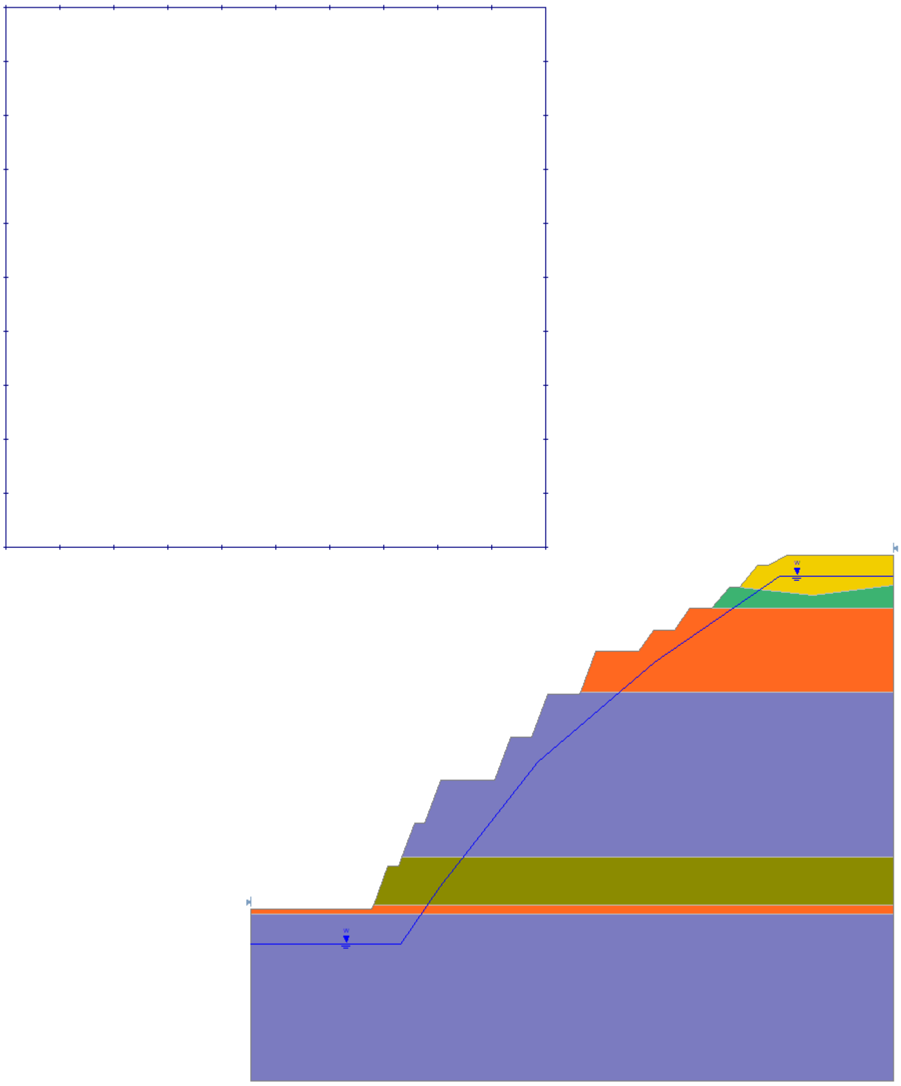
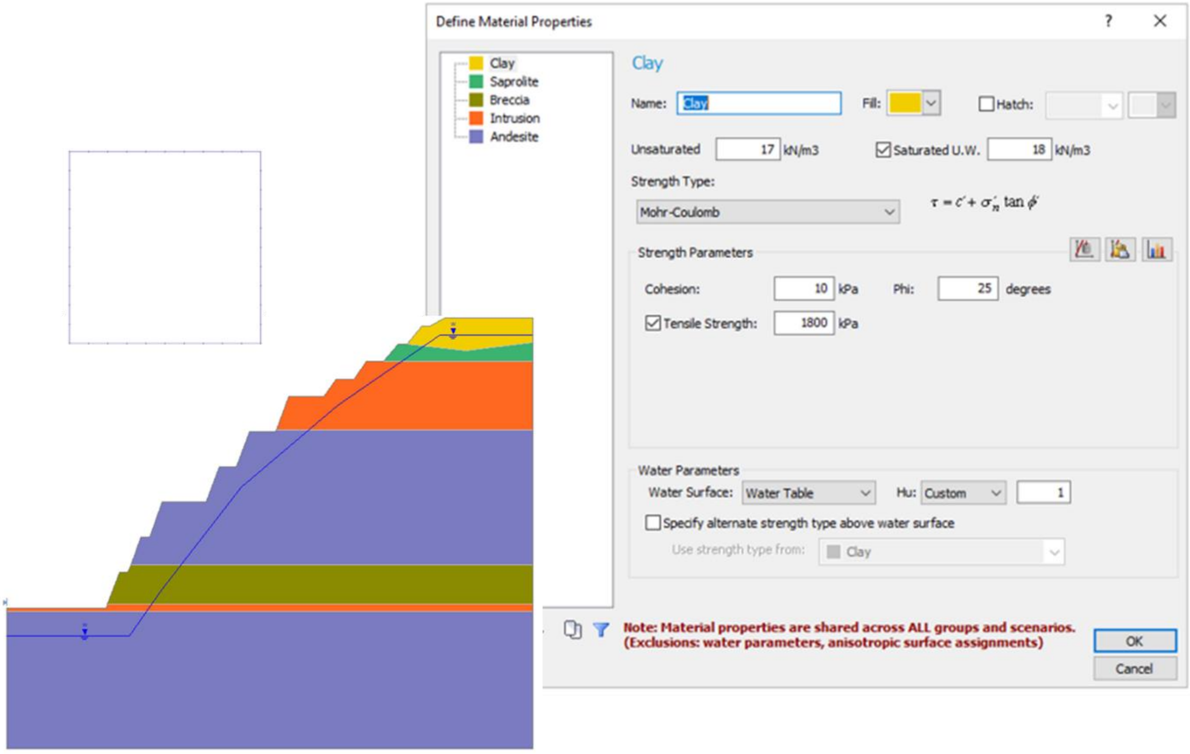


Figure 4. 1:  Automatic grid displayed before analysing the model in sector 6 (E-E')



**Clay**

Name: Clay

Fill:

☐ Hatch:

Unsaturated 17 kN/m3

☑ Saturated U.W. 18 kN/m3

Strength Type: Mohr-Coulomb

$\tau = c' + \sigma'_n \tan\phi'$

Strength Parameters

Cohesion: 10 kPa

Phi: 25 degrees

☑ Tensile Strength: 1800 kPa

Water Parameters

Water Surface: Water Table

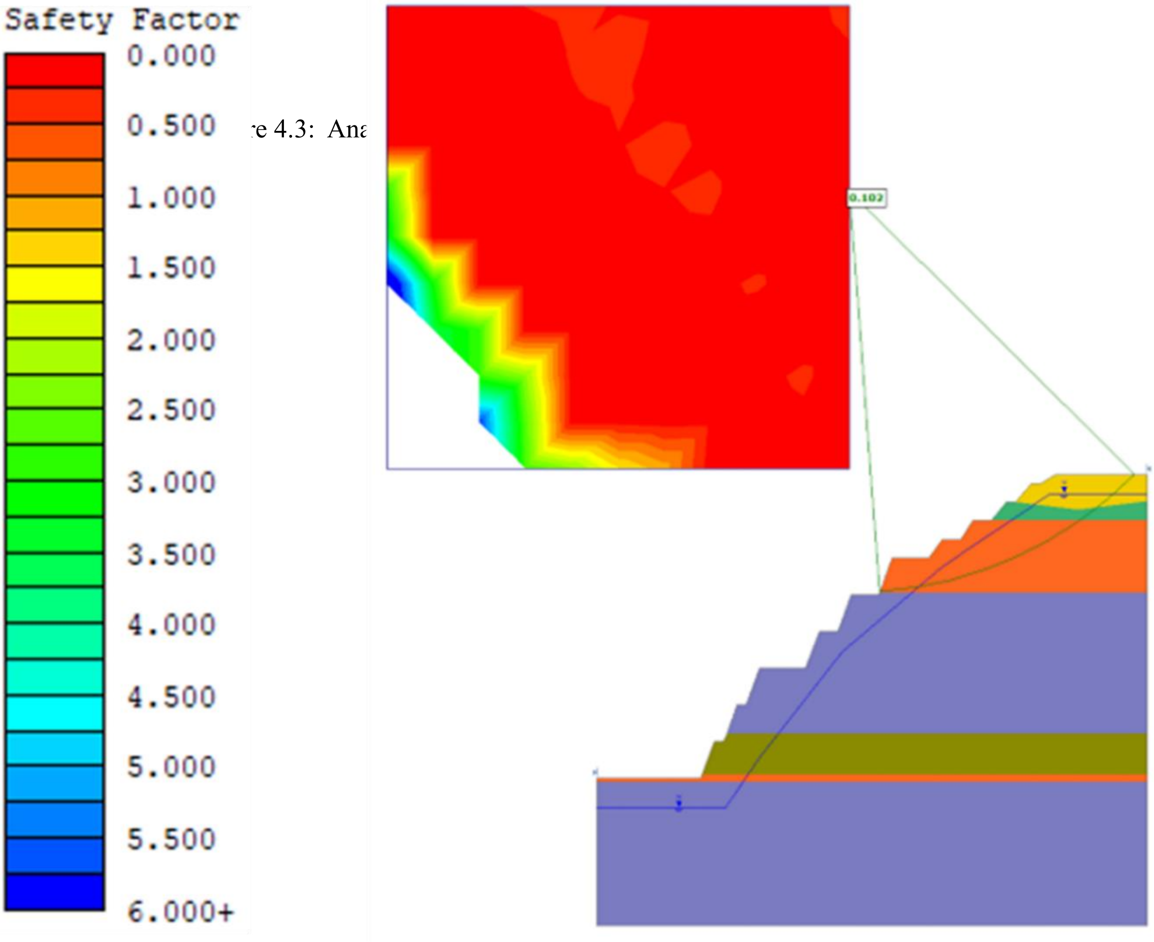
Hu: Custom 1

☐ Specify alternate strength type above water surface

Use strength type from: Clay

**Note: Material properties are shared across ALL groups and scenarios. (Exclusions: water parameters, anisotropic surface assignments)**

Figure 4.2: Material properties assigned to each boundary



e 4.3: Ana

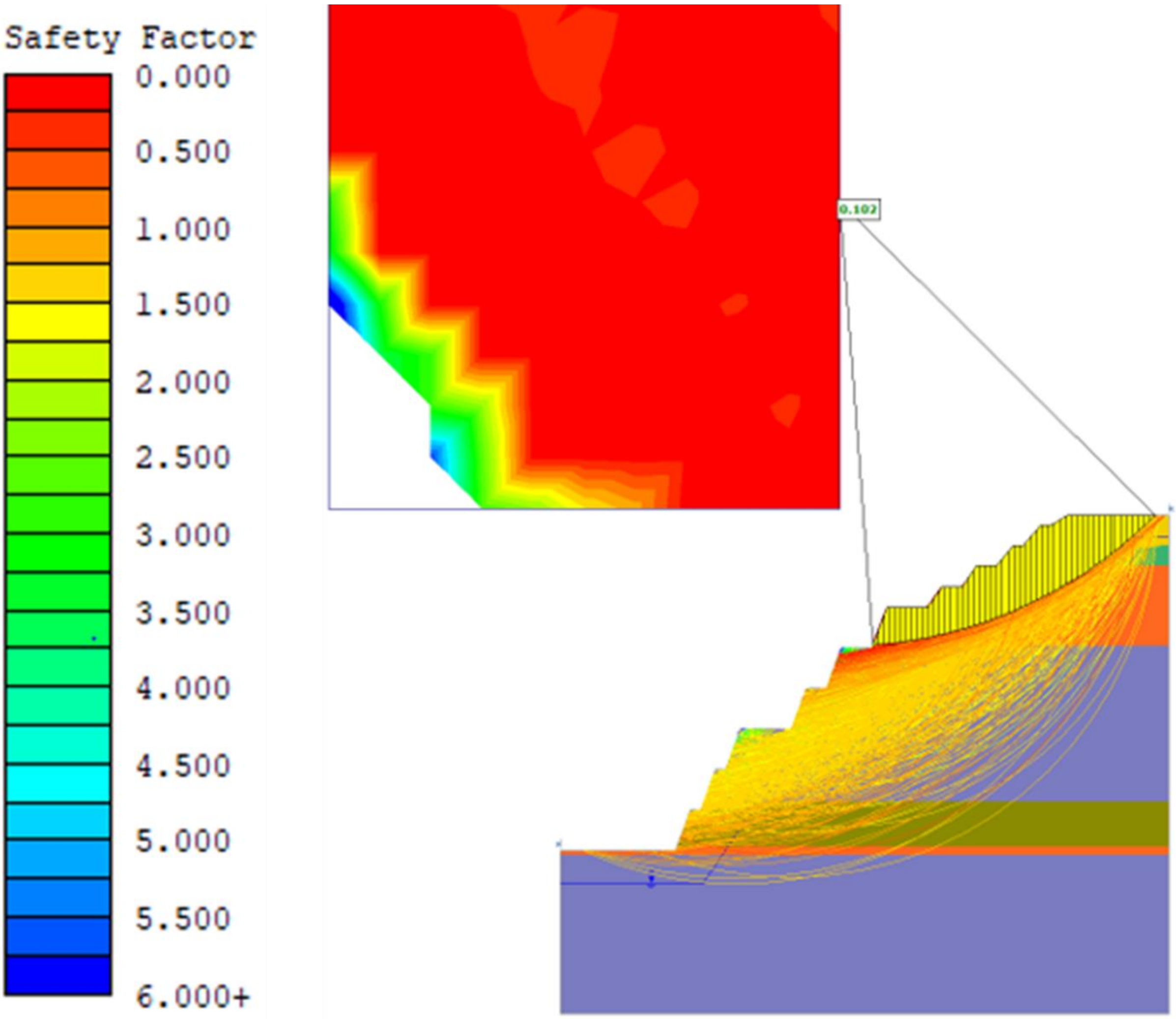


Figure 4.4: Slip surface on the boundaries displayed

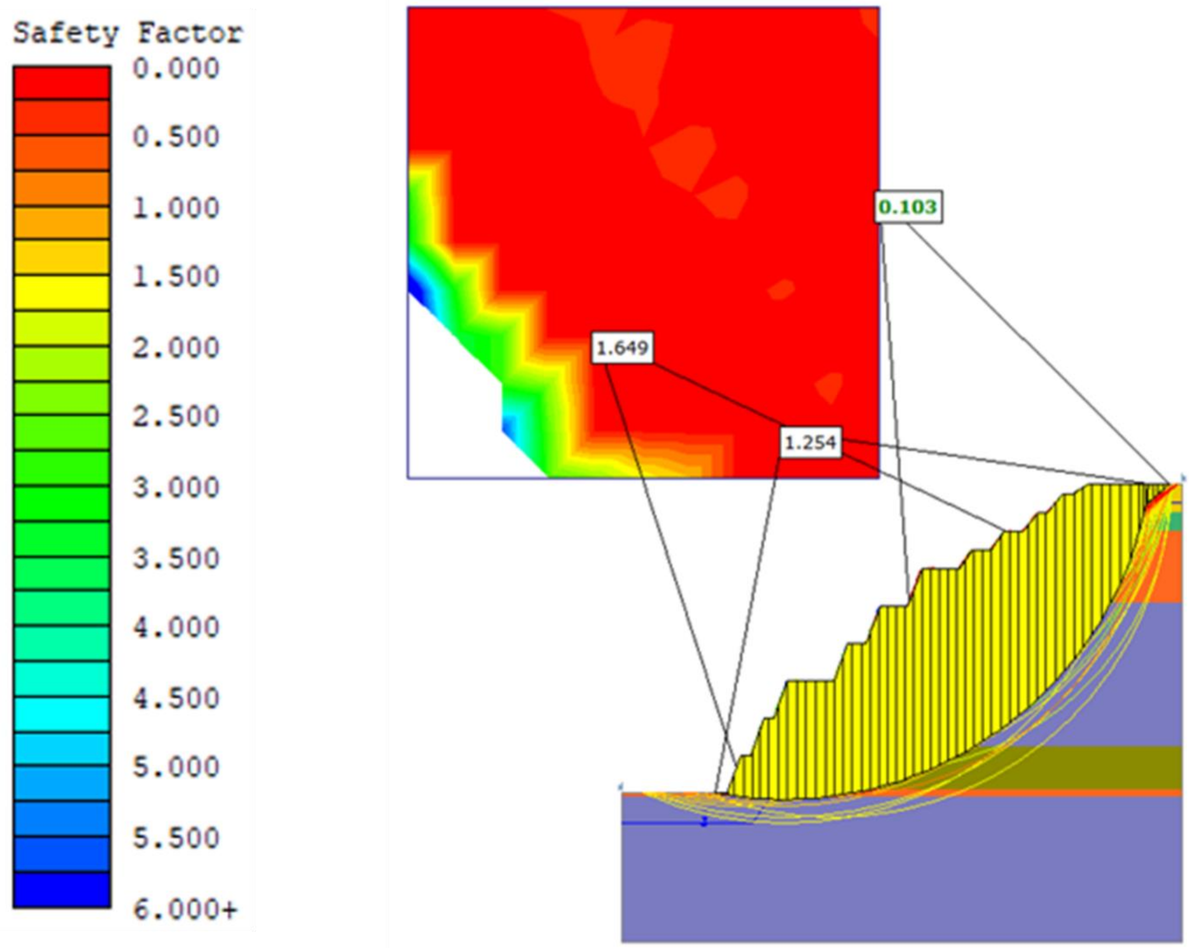


Figure 4.5: Back analysis incorporated showing various FoS on the benches

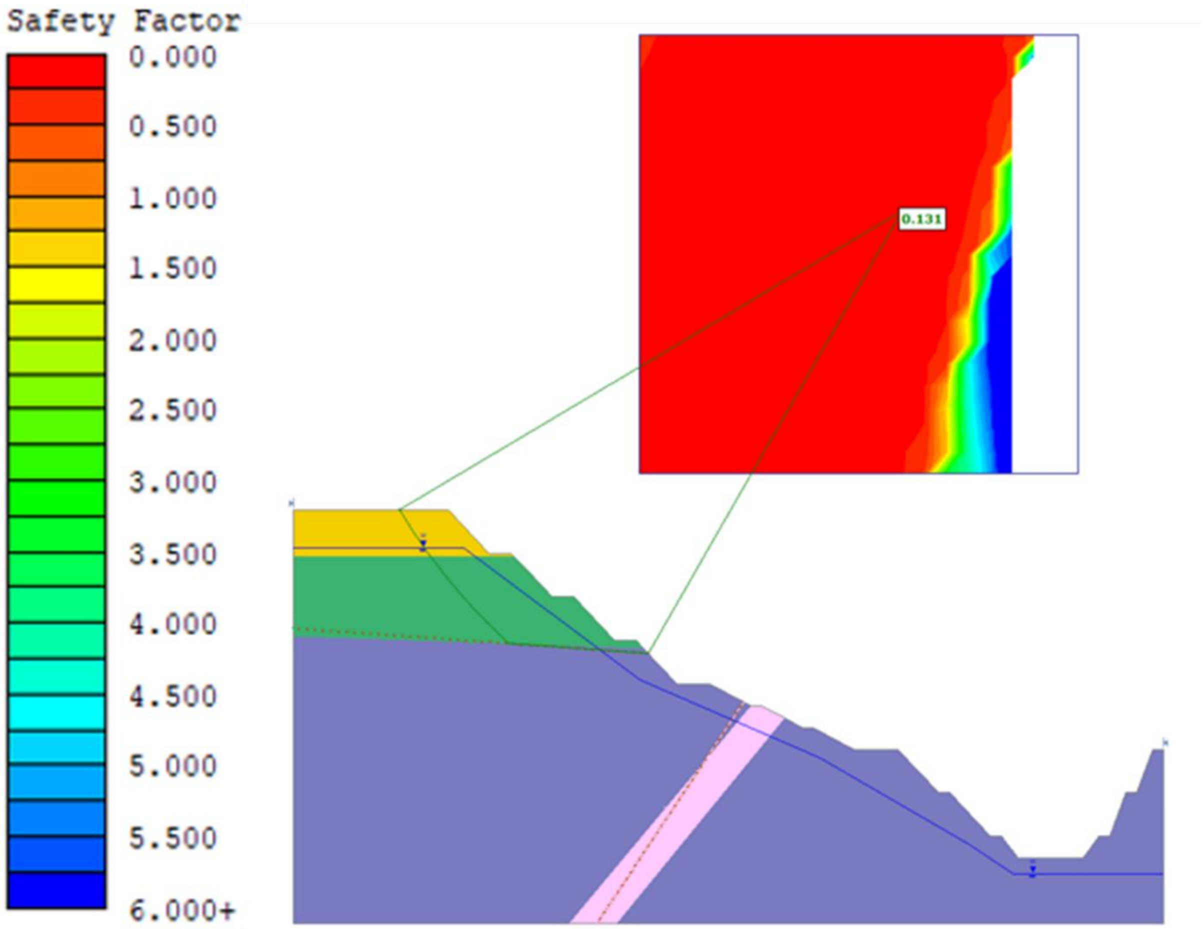
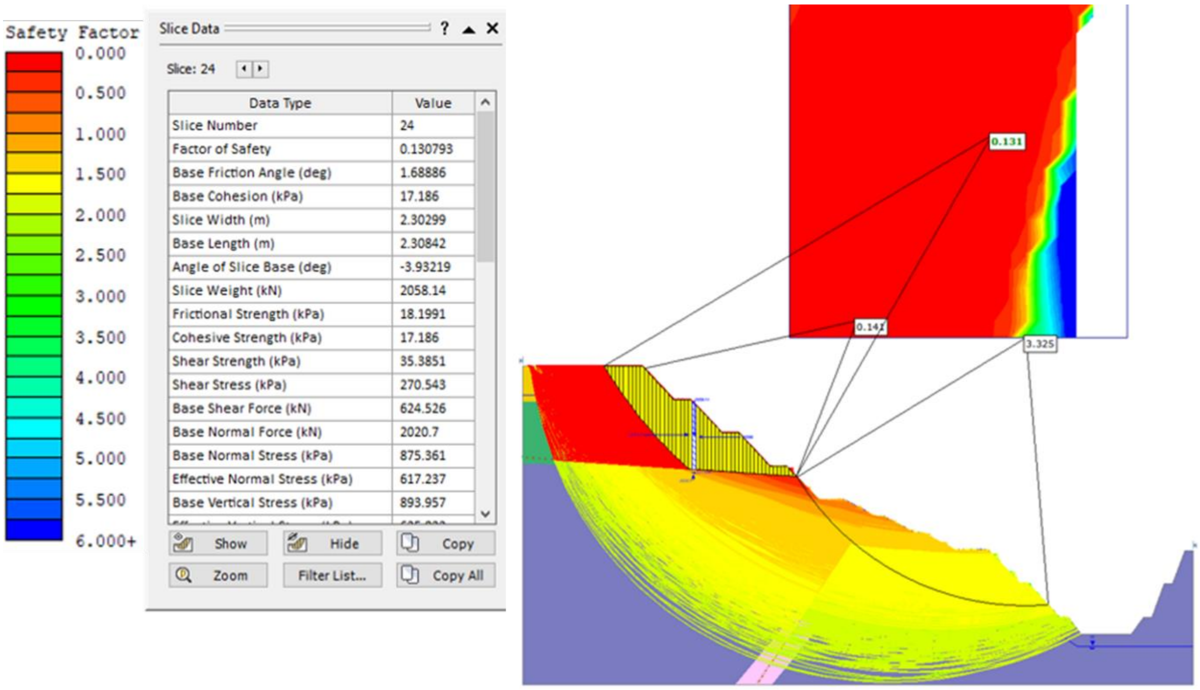


Figure 4. 6: The FoS in sector 2 after computed (B-B')



Slice: 24

| Data Type | Value |
|---|---|
| Slice Number | 24 |
| Factor of Safety | 0.130793 |
| Base Friction Angle (deg) | 1.68886 |
| Base Cohesion (kPa) | 17.186 |
| Slice Width (m) | 2.30299 |
| Base Length (m) | 2.30842 |
| Angle of Slice Base (deg) | -3.93219 |
| Slice Weight (kN) | 2058.14 |
| Frictional Strength (kPa) | 18.1991 |
| Cohesive Strength (kPa) | 17.186 |
| Shear Strength (kPa) | 35.3851 |
| Shear Stress (kPa) | 270.543 |
| Base Shear Force (kN) | 624.526 |
| Base Normal Force (kN) | 2020.7 |
| Base Normal Stress (kPa) | 875.361 |
| Effective Normal Stress (kPa) | 617.237 |
| Base Vertical Stress (kPa) | 893.957 |

Figure 4.7: Back analysis showing various FoS with increased bench height

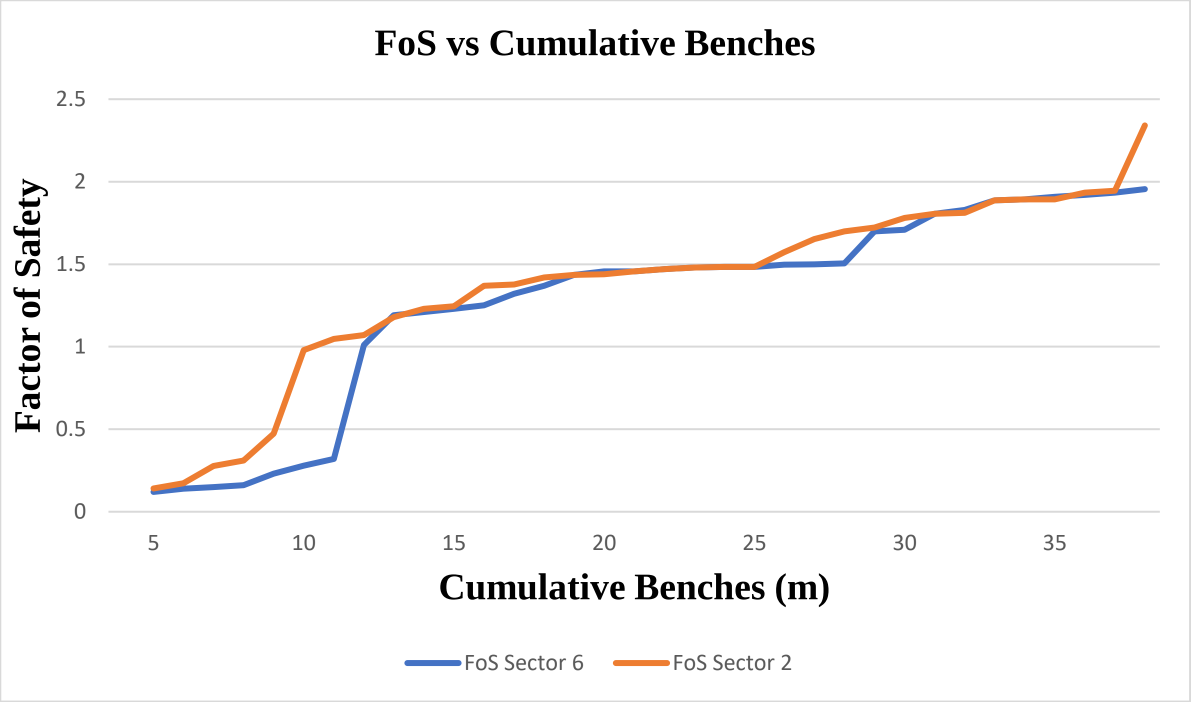


Figure 4.8: The relationship between FoS and Cumulative Benches

Table 4.1: Summary of FoS in sectors 2 and 6

| **Bench Height (m)** | **FoS Sector 6 (Janbu Simplified)** | **FoS Sector 2 (Janbu Simplified)** |
|---|---|---|
| 5 | 0.12 | 0.141 |
| | 0.14 | 0.172 |
| | 0.15 | 0.278 |
| | 0.16 | 0.31 |
| | 0.23 | 0.474 |
| 10 | 0.28 | 0.98 |
| | 0.32 | 1.048 |
| | 1.01 | 1.07 |
| | 1.19 | 1.18 |
| | 1.21 | 1.23 |
| 15 | 1.23 | 1.245 |
| | 1.25 | 1.37 |
| | 1.32 | 1.378 |
| | 1.37 | 1.42 |
| | 1.435 | 1.435 |
| 20 | 1.456 | 1.44 |
| | 1.457 | 1.456 |
| | 1.47 | 1.471 |
| | 1.48 | 1.48 |
| | 1.484 | 1.484 |
| 25 | 1.484 | 1.484 |
| | 1.498 | 1.572 |
| | 1.499 | 1.652 |
| | 1.506 | 1.699 |
| | 1.699 | 1.723 |
| 30 | 1.708 | 1.78 |
| | 1.805 | 1.805 |
| | 1.829 | 1.812 |
| | 1.888 | 1.888 |
| | 1.893 | 1.892 |
| 35 | 1.908 | 1.893 |
| | 1.92 | 1.933 |
| | 1.933 | 1.945 |
| | 1.955 | 2.341 |

### *4.2.3 Discussions*

Figures 4.2 and 4.6 provide a visual representation of the sector 6 and sector 2 models that were used for slope analysis in the current study. However, due to the unavailability of the overall pit geometry, these models were replicated from the monthly report generated by Bozshakol, a reliable source of data. This highlights the importance of using existing data and reports to construct accurate and reliable models for slope analysis. During the process of designing the models, a discrepancy in the depth data between andesite and saprolite in sector 6 was discovered. This inconsistency in the data pointed out the presence of missing information, which was taken into account during the model design to ensure that the resulting model aligns with the pit design parameters. This attention to detail is crucial in developing a robust and accurate model for slope stability analysis, as missing or inconsistent data can significantly impact the reliability of the results.

Table 3.2 summarizes the input parameters assigned to material properties in both sector 6 and sector 2. These parameters are critical in determining the behaviour of the materials under different conditions, and they play a significant role in the stability analysis of the slope. The accurate assignment of material properties is essential in capturing the true behaviour of the slope and providing reliable results. Furthermore, to incorporate the effect of water in the pit, the piezometric lines of the pit geometry indicating the presence of a water table were integrated into the model across material boundaries. This allows for a more realistic representation of the slope behaviour, as the presence of water can significantly affect the stability of the slope.

To compute the model, the surface auto grid was activated, which is a widely used technique to calculate the slope stability and display the potential failure zones. The results of the analysis were then visualized in Figures 4.3, 4.4, 4.5, 4.6, and 4.7, which display the failure zones with slip surfaces. These figures provide crucial information on the potential failure modes and locations, which are essential for identifying areas of concern and planning appropriate mitigation measures. In addition to analysing the slope stability at different failure zones, the model was also queried at various bench heights on the material boundaries to assess the impact of different depths on the factor of safety. The factor of safety is a critical parameter in slope stability analysis, as it indicates the margin of safety against potential failure. The results, as illustrated in Figure 4.8, showed that an increase in depth during back analysis led to an increase in the values of the factor of safety. This information provides valuable insights into the stability of the slope at different depths, which can aid in decision-making and designing appropriate measures to mitigate potential risks.

It was deduced from the analysis that the material boundaries, specifically clay and saprolite, contain weak zones with low rock mass properties. This indicates that these domains are susceptible to failure under external forces, and therefore require special attention from geotechnical engineers to prevent any rock mass displacement. This observation highlights the importance of considering weak zones and their properties during slope stability analysis to accurately assess the stability of the slope and develop effective mitigation strategies. To further validate the findings and support the claim of potential failure in certain sectors, the model was imported into the RS2 software, a widely used software for slope stability analysis, to estimate the probability of failure in each sector. This additional step adds robustness to the analysis and provides a comprehensive understanding of the potential risks associated with the slope stability. In conclusion, the analysis of the sector 6 and sector 2 models for slope stability estimation involved careful consideration of existing data, replication of the models from reliable sources, incorporation of water table information, activation of surface auto grid, visualization of failure zones, and querying of bench heights.

Table 4.2: Input parameter for the Slide 2 Model

| Domain | Sectors | Unit weight | | LEM Generalised Hoek-Brown Parameters | | | | LEM Model | |
|---|---|---|---|---|---|---|---|---|---|
| | | Unsaturated U.W | Saturated U.W | UCS (Mpa) | GSI | Intact Rock Constant (mi) | Disturbance Factor (D) | Friction Angle | Cohesion |
| | | kN/m$^3$ | kN/m$^3$ | | | | | Degree | kPa |
| Clay | 1,6,7 | 17 | 18 | — | — | — | — | 10-17 | 23-30 |
| | 2,3,4,5 | 15 | 17 | — | — | — | — | 15-20 | 22-25 |
| Saprolite | 1,6,7 | 17 | 17.5 | — | — | — | — | 22 | 39 |
| | 2,3,4,5 | 22 | 23 | 3 | 20 | 8 | 0.7-0 | — | — |
| Andesite | All | 29 | 29.5 | 33-66 | 60 | 25 | 0.7-1 | — | — |
| Breccia | All | 28 | 28.5 | 28-70 | 55 | 19 | 0.7-2 | — | — |
| Diorite | All | 28 | 28.5 | 53-86 | 65 | 25 | 0.7-3 | — | — |
| Gabbro | All | 27 | 28 | 8—20 | 45 | 15 | 0.7-4 | — | — |
| Granodiorite | All | 28 | 28.5 | 35-63 | 65 | 25 | 0.7-5 | — | — |
| Sediments | 6 | 20 | 21.5 | 2.5 | 20 | 17 | 0.7-6 | — | — |
| | 2,4,5 | 28 | 28.5 | 22-85 | 30 | 17 | 0.7-7 | — | — |

Table 4.3: Input parameter for the RS2 Numerical Model

| Domain | Sectors | Unit weight | | FEM Input Parameters | | | |
|---|---|---|---|---|---|---|---|
| | | Unsaturated U.W | Saturated U.W | Young’s Modulus (Ei) | Poisson’s Ratio | Porosity | Tensile Strength |
| | | kN/m$^3$ | kN/m$^3$ | | | | Mpa |
| Clay | 1,6,7 | 17 | 18 | 0.05 | 0.37 | 0.05 | 1.8 |
| | 2,3,4,5 | 15 | 17 | 0.05 | 0.37 | 0.05 | 1.8 |
| Saprolite | 1,6,7 | 17 | 17.5 | 0.1 | 0.29 | 0.6 | 0.1 |
| | 2,3,4,5 | 22 | 23 | 0.42 | 0.31 | 0.6 | 0.1 |
| Andesite | All | 29 | 29.5 | 41 | 0.25 | 0.4 | 9 |
| Breccia | All | 28 | 28.5 | 38 | 0.3 | 0.4 | 6 |
| Diorite | All | 28 | 28.5 | 38 | 0.28 | 0.4 | 9 |
| Gabbro | All | 27 | 28 | 14 | 0.24 | 0.5 | 4 |
| Granodiorite | All | 28 | 28.5 | 26 | 0.28 | 0.4 | 5 |
| Sediments | 6 | 20 | 21.5 | 5 | 0.15 | 0.5 | 0.25 |
| | 2,4,5 | 28 | 28.5 | 20 | 0.26 | 0.5 | 6 |

## 4.3 Finite Element Method with SR2

The RS2 model, a numerical model with a two-dimensional approach, was utilized in this study to analyse the extent of deformation and estimate the probability of failures in selected sectors. This application was specifically developed to provide support for slope stability assessments, as well as design considerations for excavation environments, tunnels, and embankments, among other applications. Despite its widespread use in slope stability analysis, the RS2 model is also capable of evaluating deterministic values in terms of the factor of safety through critical shear strength reduction. The shear strength reduction (SSR) technique is employed in slope stability analysis to quantify the reduction in strength of each boundary until total displacement occurs. Moreover, the RS2 model aids users in gaining a comprehensive understanding of the behaviour of various materials in the field of geotechnical engineering.

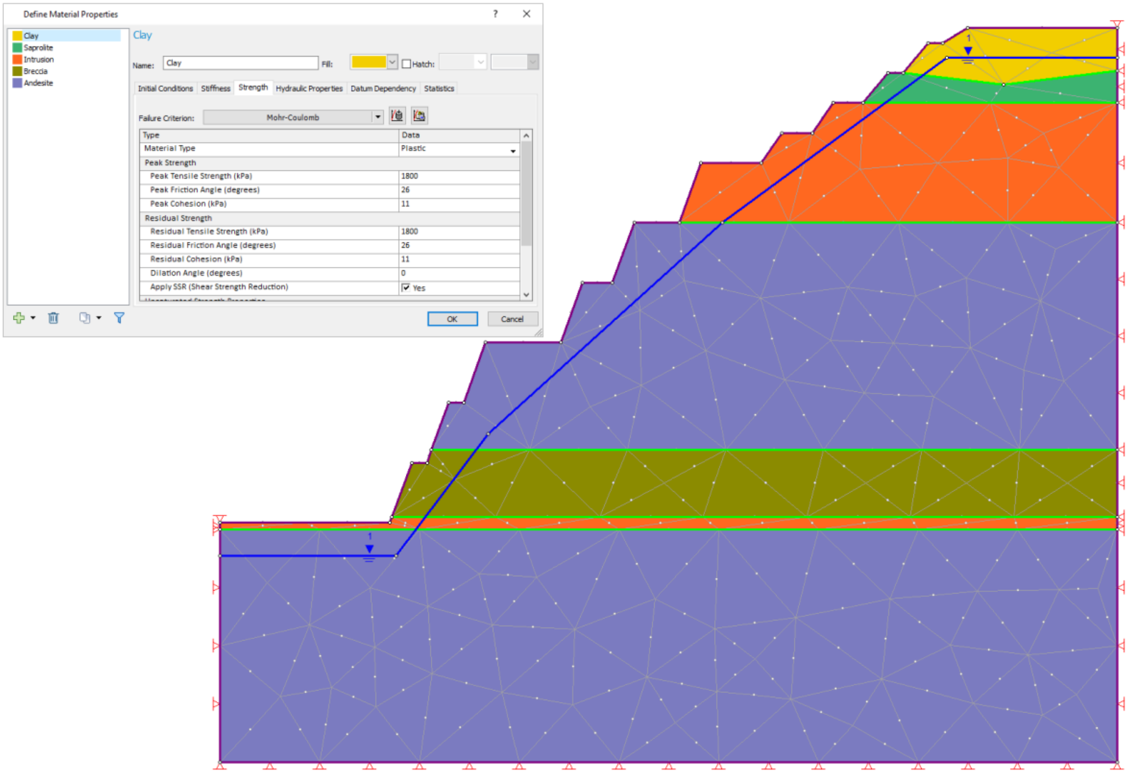
### *4.3.1 Model Set-up*

Following the results produced by the Slide 2 model, the probability of failures in the selected sectors is expected to be estimated. The following are the steps taken to accomplish my results:

1. The Slide 2 model is imported into the RS 2 model;
2. Based on the model interface, the Monte Carlos Simulation are selected from the probabilistic drop down as against the Point Estimation Method;
3. The number of mesh are reduced to 500 to enhance the simulation time;
4. The material boundaries consisting the rock mass as assigned to their properties such as Young's Modulus, Poisson's ratio, cohesion and friction;
5. For the clay domain, the rock cohesion, UCS and friction angle as assigned while the other rock domains are assigned with UCS, Young's Modulus and Poisson's ratio;
6. After computing the model, the results are later analysed.

### *4.3.2 Results*

Below are the representations illustrating the outcomes obtained from the RS 2 numerical simulations, specifically for sectors 2 and 6. These results have been visually depicted in the figures provided.



Name: Clay

Fill:

☐ Hatch:

Failure Criterion: Mohr-Coulomb

| Type | Data |
|---|---|
| Material Type | Plastic |
| Peak Strength | |
| Peak Tensile Strength (kPa) | 1800 |
| Peak Friction Angle (degrees) | 26 |
| Peak Cohesion (kPa) | 11 |
| Residual Strength | |
| Residual Tensile Strength (kPa) | 1800 |
| Residual Friction Angle (degrees) | 26 |
| Residual Cohesion (kPa) | 11 |
| Dilation Angle (degrees) | 0 |
| Apply SSR (Shear Strength Reduction) | ☑ Yes |

Figure 4.9: Sector 6 model imported into the RS2

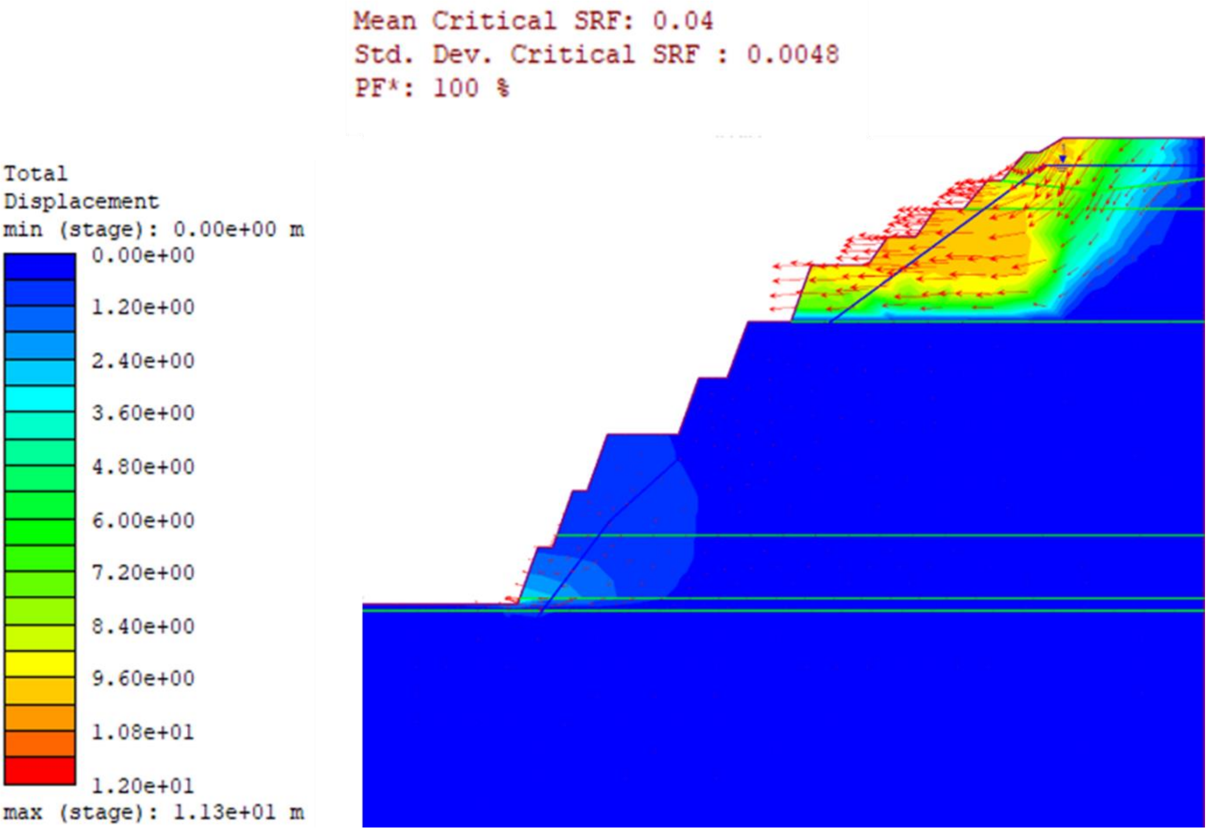


Figure 4.10: Analysed model with SRF at 0.03

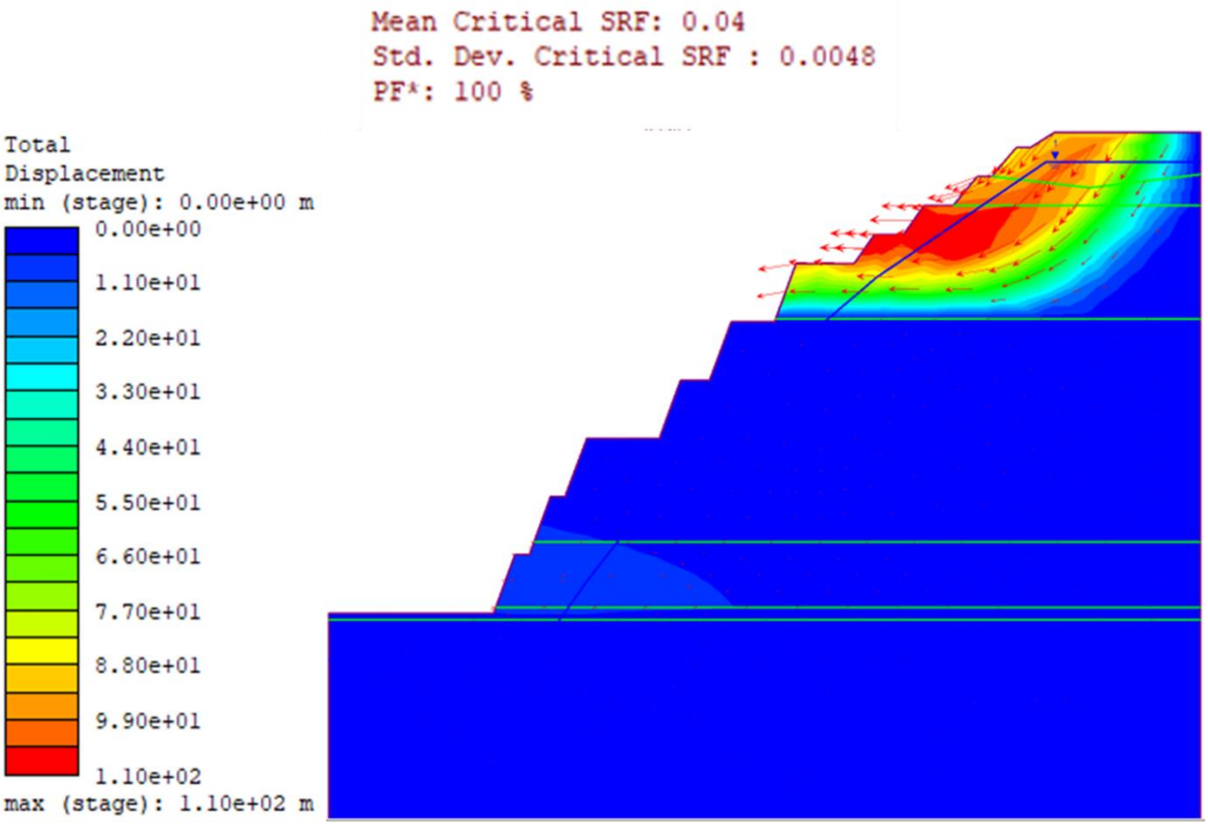


Figure 4.11: Analysed model with SRF at 0.13

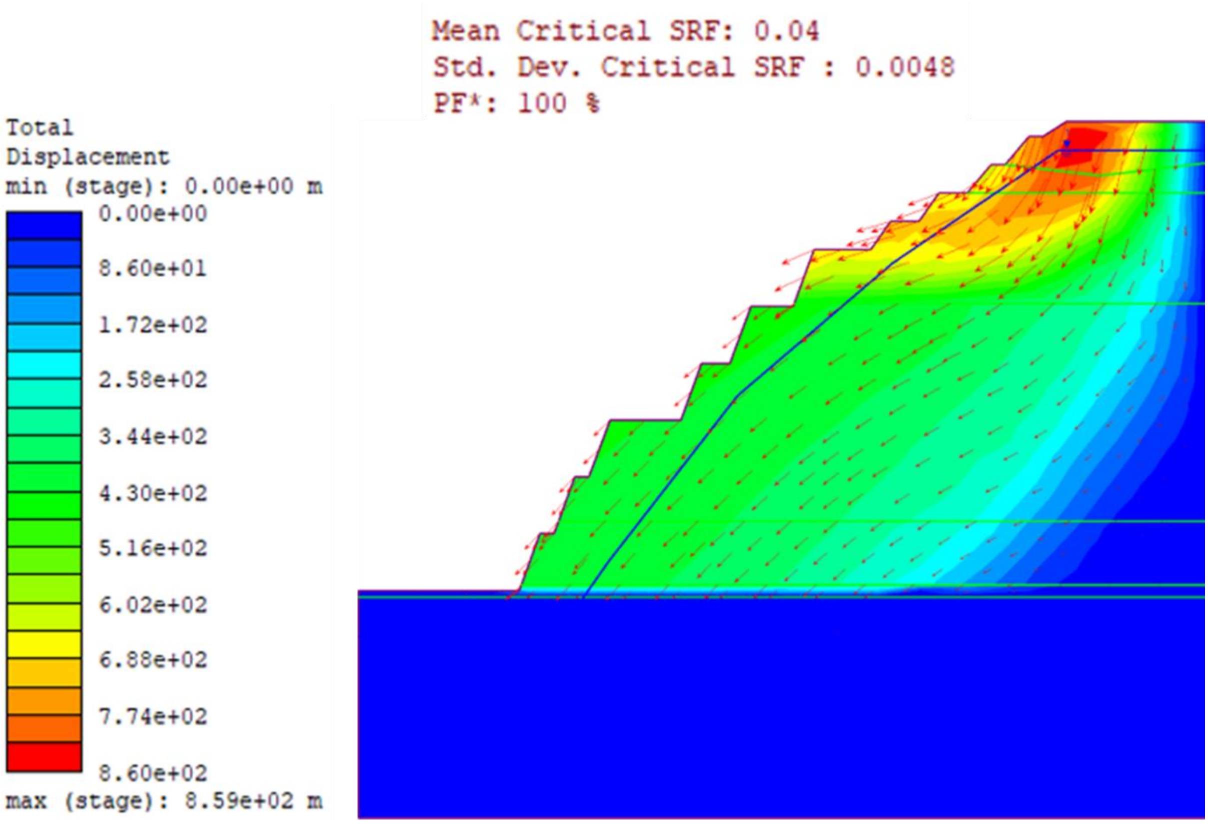


Figure 4.12: Analysed model with SRF at 0.25

### *4.3.3 Discussion*

The figures presented in Section 4.3.2 provide a comprehensive summary of the RS 2 simulation, shedding light on the results obtained. Notably, Figure 4.9 visually showcases the importation of the Slide 2 model, which consists of almost 500 meshes, indicating the complexity of the simulation. Furthermore, Table 3.5 provides valuable information regarding the allocated ranges of values for the input parameters that correspond to various rock properties in the case study area. It is worth mentioning that these values are assumed, as precise data for the geotechnical domain was not available for the simulation.

Upon careful analysis, it is evident that there are significant differences in the geotechnical properties of the rock mass in different domains. For instance, in domain #2, the saprolite exhibits an extremely low tensile strength of 1800 kN/m$^3$, while in domain #1, the clay has a lower cohesion and friction angle, with values of 22 kPa and 200 respectively for sector 2. These findings are further supported by the laboratory reports, which reveal that both the clay and saprolite in both sectors are highly weathered, characterized by poor RQD/Jn values ranging from 7-15%. This information is critical in understanding the behaviour of the rock mass and the potential for slope instability in different domains.

The impact of these geotechnical properties on the stability of the slopes is evident in the simulation results. Figure 4.10 clearly illustrates a total displacement of 1.13 m at a critical strength reduction factor (SRF) of 0.03, revealing the distribution of stresses in domain #1 and domain #2. However, as the critical SRF increases, the simulation results, as depicted in Figure 4.12, show more failure zones within the two domains, indicating a higher probability of slope failure. This highlights the sensitivity of the slopes to changes in the critical SRF and the need for careful consideration of this parameter in slope stability analysis.

Notably, the presence of induced stresses within the rock mass is another crucial factor that influences slope stability. Figure 4.12 also reveals that at a displacement of 1.33 m, induced stresses could extend towards domain #3 and domain #4 at a critical SRF of 0.25. This indicates that continuous mining activities within the geotechnical sectors, without proper pit design and scheduling, could create additional weak zones and trigger slope instability. Moreover, during the winter and rainy seasons when high levels of water saturation are present within these domains, the effects of slope instability could be further exacerbated. The saturation of water could activate ubiquitous joints and structural imprints within the rock mass, leading to increased risks of slope failures.

In conclusion, the RS 2 simulation results provide valuable insights into the geotechnical properties and slope stability of different domains in the case study area. The assumed values of input parameters, based on available data, highlight the need for further investigation and precise characterization of the rock mass. The findings emphasize the significant impact of geotechnical properties, induced stresses, and environmental factors, such as water saturation, on slope stability. Proper pit design, scheduling, and careful consideration of critical SRF are crucial in mitigating the risks of slope instability and ensuring safe mining operations in the study area.

# CHAPTER FIVE

## 5. RELIABILITY ANALYSIS

### 5.1 Introduction

In this section, the sectors 2 and 6 will undergo reliability analysis utilizing the Microsoft Excel spreadsheet developed by Low and Tang (1997). The reliability index of these two sectors will be determined based on the input parameters provided by the company. Subsequently, a comprehensive summary of the results will be analysed and explained in detail.

### 5.2 Reliability Analysis with FORM

The concept of the reliability index was introduced by Hasofer (1974), and it is commonly known as the "first-order reliability method" (FORM). This method involves linearizing each limit state function in standard normal space at the design point, which is the point on the limit state surface that is closest to the origin. The distance between the origin and the limit state surface in standard normal space serves as a representation of the reliability index, denoted as β. Following that, the FORM algorithm is based on the performance function G(X), where X is the fundamental vector of random variables. The probability of failure $(p_f)$ can be expressed when the combined probability density function, $F_X(X)$ of all random variables is defined and expressed as:

$$P_F \equiv P\left(G\left(\mathbf{x}\right) \leq 0\right) = \int_{G(\mathbf{x})\leq 0} p_X\,(\mathbf{x})d\mathbf{x} \qquad (9)$$

Later, Low and Tang (1997) opted to use Eqn. 9 as it offers several advantages over the covariance matrix $C$. The correlation matrix $\boldsymbol{R}$ is easier to construct and provides a clearer illustration of the correlation structure. Additionally, Eqn. 9 can be implemented without programming, making it compatible with EXCEL's built-in functions. This can be accomplished by minimizing the value of β, while imposing the constraint that $G[U] = 0$, and automatically adjusting the values of the random variables, $x_k$, using Microsoft Excel's Solver optimization tool. Another approach, known as the ellipsoid method, utilizes spreadsheets to provide an intuitive ellipsoidal perspective in the initial space of the random variables.

The region of $X$ where $G(X) < 0$ is denoted as $L$, indicating that Eqn. 9 cannot be analytically solved. In the FORM (First Order Reliability Method) approximation, the vector of random variables X is transformed to the standard normal space U. This is achieved by expressing U as a set of independent Gaussian variables with zero mean and unit standard deviation, while the linear function is denoted as G (U).

By formulating an equation-based distance between the origin and the hyperplane, hyperplane $G(U) = 0$, the reliability index, $\beta$ is used to calculate the probability of failure $\left(p_f\right)$.

Below is how the equation is expressed:

$$p_f = 1 - \Phi(\beta) = \Phi(-\beta) \tag{10}$$

hyperplane $G(U) = 0$ $\beta = \min_{x\grave{o}F} \sqrt{\mathbf{n}^T \mathbf{R}^{-1} \mathbf{n}}\ for\ \{X{:}\, G(X) = 0\}$

Assuming that $\Phi$ represents the cumulative distribution function (CDF) of the standard normal variant, the relationship between parameters is considered precise when they follow normal distributions, and the limit state surface is planar otherwise. Low (2014) later demonstrated a method to compute the reliability index in the original space. His approach is based on the Hasofer (1974) reliability index matrix formulation, which involves $\beta\ x_k\ \mu_k\ \sigma_k$.

$$\beta = \min_{x\grave{o}F} \sqrt{\mathbf{n}^T \mathbf{R}^{-1} \mathbf{n}}\ for\ \{X{:}\, G(X) = 0\} \tag{11}$$

Given that **n** is the column vector of $n_x$, which is also equivalent to:

$$\beta = \min_{x\epsilon F} \sqrt{\left(\frac{x-\mu}{\sigma_k}\right)^T (\boldsymbol{R})^{-1} \left(\frac{x_k-\mu_k}{\sigma_k}\right)^T} \qquad for\{X{:}\, G(X) = 0\} \tag{12}$$

## 5.3 Method Set-Up

Based on the geotechnical data obtained from the case study area, a comprehensive analysis was conducted to identify the relevant input parameters for the pit slope design in sectors 2 and 6. These input parameters were carefully selected based on their significance in determining the stability of the slope. The chosen parameters include Rock Quality Designation (RQD), cohesion, joint sets, width to height ratio of the benches (W/H), and friction angle.

To facilitate the estimation of reliability indices (β) for the geotechnical sectors, a FORM (First Order Reliability Method) spreadsheet was developed. The spreadsheet utilizes these input parameters to estimate the stability and reliability of the pit slope design. The designations for the input parameters in the FORM spreadsheet are as follows: RQD/Jn is denoted as P1, the joint friction angle φ as P2, and the width to height ratio of the pit bench (W/H) as P3, as shown in Figure 5.1.

In the FORM spreadsheet, the datasets for P1 (RQD/Jn) are assumed to be normally distributed and are specified in cell A2:A4, as shown in Figure 5.1. The mean values and standard deviations of the input parameters are assigned to cells C2:C4 and D2:D4, respectively. The correlation matrix (R), obtained from Eqn. 12, indicates that the input parameters are not correlated, and this information is illustrated in cell K2:K4 in the spreadsheet. Furthermore, based on Eqn. 11, the column vectors $n_x$ are assigned to cells N2:N4 in the spreadsheet.

To estimate the reliability index (β) for the selected sectors, the design point $x^*$ is initially assigned as the mean values of the input parameters. This allows the SOLVER, a numerical optimization tool in the spreadsheet, to automatically select the required algorithm for reliability analysis. This set-up was successfully implemented for the geotechnical domains in both sectors, and the results are summarized in Table 5.3, Table 5.4, and Table 5.5, along with a graphical representation in Fig. 5.2.

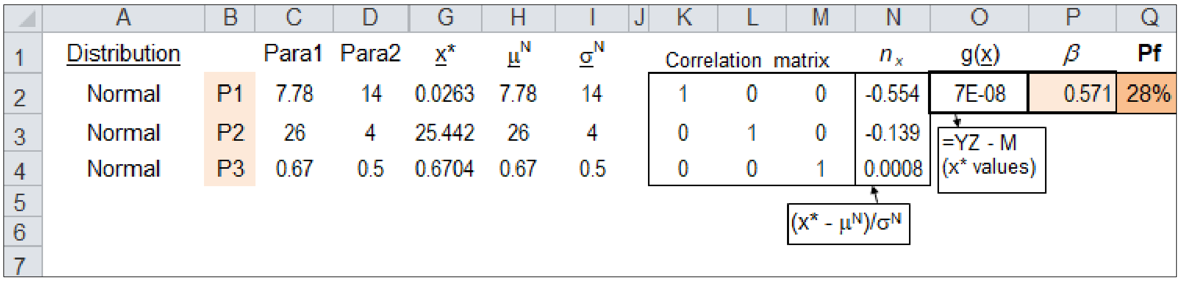
The utilization of the FORM spreadsheet and the selection of input parameters based on the geotechnical data and their correlations are crucial for accurately estimating the reliability indices (β) of the geotechnical sectors in the pit slope design. This information provides valuable insights for decision-making and risk assessment in the overall stability analysis and management of the pit slope, ensuring safe and sustainable mining operations.

Table 5.1: Input parameters for Sector 2

| Sample | | 1 | 2 | 3 | 4 |
|---|---|---|---|---|---|
| Rock domain | | Clay | Saprolite | Sediment | Andesite |
| RQD | [%] | 0 | 0 | 17-35 | 40-55 |
| GSI | [-] | 10 | 17 | 20 | 60 |
| Joint orientation | [°] | 002/20 | 007/27 | 110/30 | 137/45 |
| Joint cohesion | [MPa] | 0.08 | 0.11 | 0.17 | 0.49 |
| Joint friction angle | [°] | 10 | 20 | 25 | 32 |
| UCS | [MPa] | 1 | 3 | 2.5 | 48 |

Table 5.2: Input parameter for sector 6

| Sample | | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|---|
| Rock domain | | Clay | Saprolite | Sediment | Andesite | Breccia |
| RDQ | [%] | 5 | 12 | 19-38 | 65 | 55 |
| GSI | [-] | 11 | 18 | 19 | 57-63 | 52-58 |
| Joint orientation | [°] | 007/10 | 1.5/20 | 115/35 | 270/44 | 202/27 |
| Joint cohesion | [MPa] | 0.15 | 0.24 | 0.57 | 1.16 | 0.9 |
| Joint friction angle | [°] | 15 | 21 | 28 | 34 | 27 |
| UCS | [MPa] | 1 | 3 | 22 | 33 | 28 |



| | A | B | C | D | G | H | I | J | K | L | M | N | O | P | Q |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Distribution | | Para1 | Para2 | $x^*$ | $\mu^N$ | $\sigma^N$ | | Correlation matrix | | | $n_x$ | g(x) | $\beta$ | **Pf** |
| 2 | Normal | P1 | 7.78 | 14 | 0.0263 | 7.78 | 14 | | 1 | 0 | 0 | -0.554 | 7E-08 | 0.571 | 28% |
| 3 | Normal | P2 | 26 | 4 | 25.442 | 26 | 4 | | 0 | 1 | 0 | -0.139 | | | |
| 4 | Normal | P3 | 0.67 | 0.5 | 0.6704 | 0.67 | 0.5 | | 0 | 0 | 1 | 0.0008 | | | |
| 5 | | | | | | | | | | | | | | | |
| 6 | | | | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | | | | |

=YZ - M
($x^*$ values)

$(x^* - \mu^N)/\sigma^N$

Figure 5.1: Reliability index of domain 2 in sector 6

Table 5.3: Summary indicating the range of computed reliability indices and PoF for sector 6

| Domain | #1 | #2 | #3 | #4 | #5 |
|---|---|---|---|---|---|
| Lithology/rock type | **Clay** | **Saprolite** | **Sediment** | **Breccia** | **Andesite** |
| Reliability index range | 1.201-0.09 | 1.43-0.60 | 1.52-0.73 | 1.97-0.86 | 2.051-0.844 |
| Probability of failure range | 2-89% | 5-72% | 8-50% | 3-10% | 1-7% |

Table 5.4: Summary indicating the range of computed reliability indices and PoF for sector 2

| Domain | #1 | #2 | #3 | #4 |
|---|---|---|---|---|
| Lithology/rock type | **Clay** | **Saprolite** | **Sediment** | **Andesite** |
| Reliability index range | 1.25-0.05 | 1.5-0.65 | 1.58-0.78 | 2.56-1.05 |
| Probability of failure range | 2-100% | 4-75% | 10-55% | 1-5% |

Table 5.5: Summary of the probability of failure on selected sectors

| s/n | H/W | PoF (%) | | | |
|---|---|---|---|---|---|
| | | Domain #1 | Domain #2 | Domain #3 | Domain #6 |
| 1 | 0.35 | 5.6 | 10 | 1.24 | 8.2 |
| 2 | 0.53 | 10.8 | 15 | 7.7 | 10.4 |
| 3 | 0.64 | 30 | 32 | 25 | 37 |
| 4 | 0.85 | 78 | 85 | 72 | 92 |
| 5 | 1 | 90 | 95 | 85 | 100 |

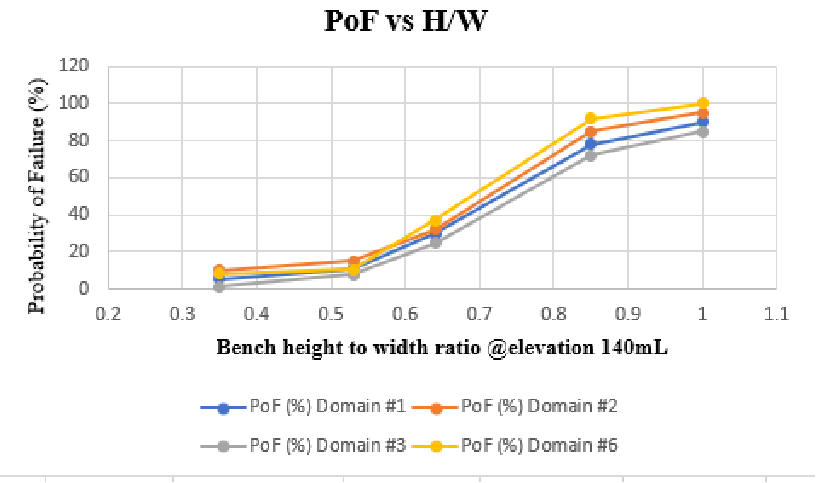
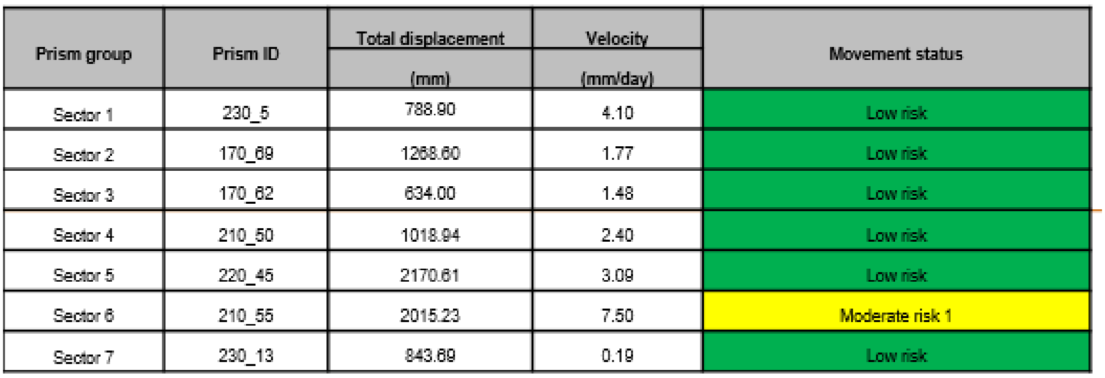


Figure 5.2: An illustration of PoF against H/W

Table 5.6: Monthly report (September) observed in sector 6 (Kazminerals, 2022)



| Prism group | Prism ID | Total displacement (mm) | Velocity (mm/day) | Movement status |
|---|---|---|---|---|
| Sector 1 | 230_5 | 788.90 | 4.10 | Low risk |
| Sector 2 | 170_69 | 1268.60 | 1.77 | Low risk |
| Sector 3 | 170_62 | 634.00 | 1.48 | Low risk |
| Sector 4 | 210_50 | 1018.94 | 2.40 | Low risk |
| Sector 5 | 220_45 | 2170.61 | 3.09 | Low risk |
| Sector 6 | 210_55 | 2015.23 | 7.50 | Moderate risk 1 |
| Sector 7 | 230_13 | 843.69 | 0.19 | Low risk |

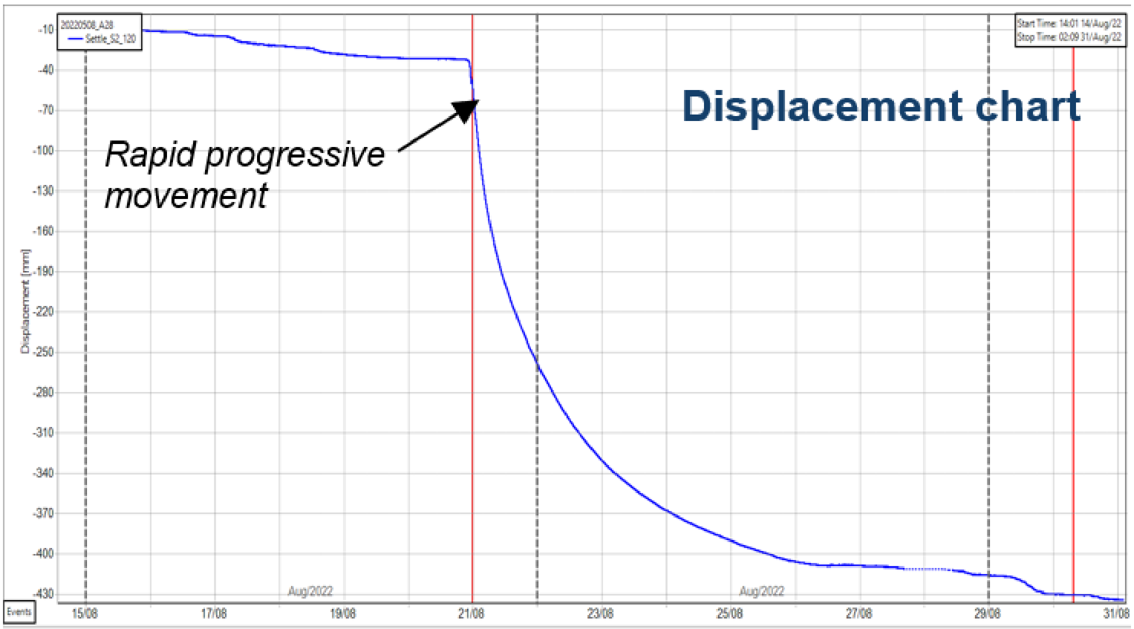


Figure 5.3: Displacement graph showing the initial movement of material in sector 6

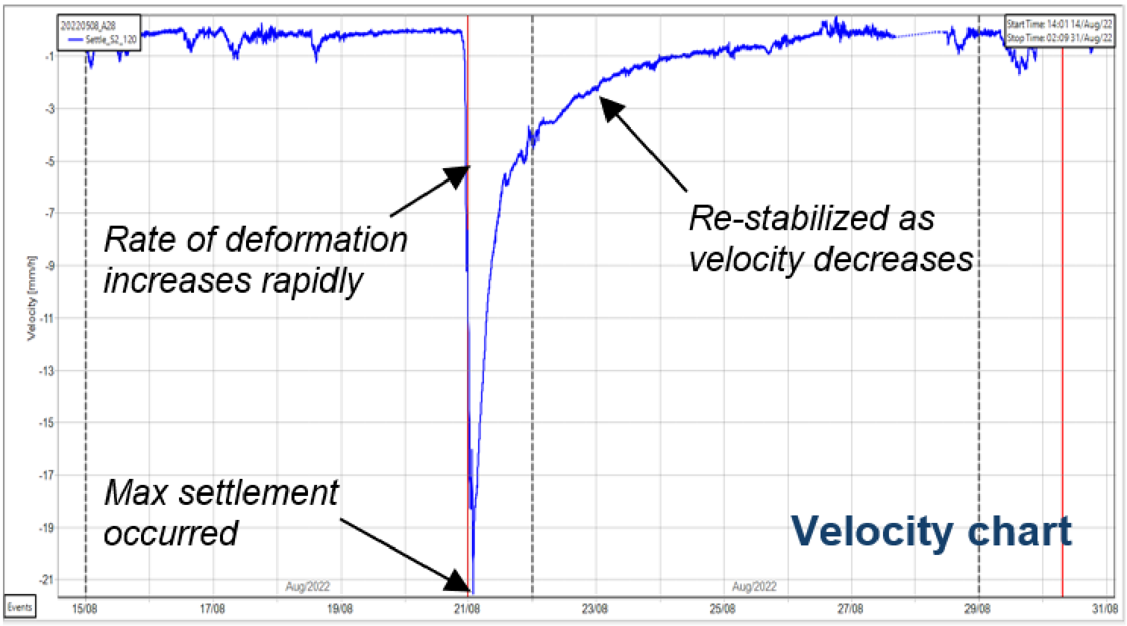


Figure 5.4: Maximum displacement of material in sector later observed

## 5.4 Discussions

The summary of the estimated reliability indices and failure probabilities for sector 6 are presented in a clear and concise manner in Table 5.6, providing valuable insights into the performance of different geotechnical domains within the sector. Figures 5.3 and 5.4 depict a graphical displacement chart used to observe Table 5.3. During mining operations, this study discovered a maximum velocity of 21mm/h at an average velocity of 7.5mm/day by the Georadar A-27 monitoring system located at the South wall of the pit. The monitoring system later detected significant rock movement, revealing obvious crack propagation within the settlement location in sector 6 as illustrated in Fig. 3.4. However, the propagation of this rock mass displacement had no significant effect on the mining activities in the geotechnical sectors as the velocity of the materials tends to restabilizes quickly at 0.65mm/h as shown in Fig. 5.2.

Specifically, domains #1, #2, and #3 are found to have the highest average probability of failure at 70.3 percent, indicating that these domains are more susceptible to slope instability and rock displacement. On the other hand, domains #4 and #5 exhibit a lower average probability of failure at 8.5 percent, suggesting that they are relatively more reliable and less prone to failure.

Similar trends are observed in sector 2, where the first three domains also have a high average probability of failure at 76.7 percent, while domain #4 has a probability of failure of 5 percent, which is comparatively lower. This information highlights the importance of understanding the performance of different geotechnical domains within a sector and considering their unique characteristics in slope design and implementation.

Based on a comprehensive comparison between the two sectors, it is concluded that geotechnical domains with weak rock structures are more likely to experience slope instability and have a higher probability of failure. These weak domains, such as domains #1, #2, and #3, may be influenced by various factors, including mining operations, accumulated energy from natural processes, and the characteristics of the rock mass. Therefore, it is crucial for geotechnical engineers to carefully consider these factors and implement appropriate measures to control and mitigate any potential rock displacement in these domains.

In order to further analyse the relationship between slope stability and geometric parameters, Figure 5.2 depicts a graph of probability of failure versus height-to-width ratio (H/W). The selection of H/W ratio as a parameter is based on its relevance to the pit slope design from Slide 2, and the FoS values obtained from the back-analysis of benches in the Slide 2 model are used to determine the corresponding depth and width. The graph clearly illustrates that as the H/W ratio increases, the probability of failure also tends to increase, indicating that higher slope heights in relation to their widths are associated with increased risk of failure. This finding emphasizes the need for careful consideration of slope geometry and appropriate design measures, particularly in sectors with higher reliability indices, such as sectors 2 and 6.

In conclusion, the findings from the analysis of reliability indices, failure probabilities, and slope geometry suggest that geotechnical engineers should pay close attention to the weak domains with lowest reliability indices range such as, 0.09-1.43 and 0.05-1.5 respectively.

Hence, it is justified that the domains with a reliability index greater than 2.0 is acceptable because it provides a more reliable slope stability than the other domains with moderately high reliability indices range of 1.5-1.99, such as sediment and breccia. In addition, it is essential to consider various triggering factors, implement appropriate measures to control rock displacement, and ensure slope stability in order to mitigate potential risks and ensure safe and reliable mining operations.

## 5.5 Results Comparison

A typical geotechnical risk that is commonly observed during field observations is outlined in detail in Table 5.6. This failure was discovered in close proximity to the North wall of Sector 6, specifically at a bench elevation of 140-120mL. The field report highlighted that the presence of natural discontinuities and low rock strength in the transition zone between fresh rock and clay could potentially have contributed to this failure. These observations are consistent with the results obtained from reliability and numerical analyses, providing valuable insights into the underlying causes of the failure.

Furthermore, laboratory tests conducted on the intact rock mass properties further supported the field observations, strengthening the validity of the findings. In particular, the borehole data from section 6, including GT023, GT024, and GT036, revealed that the rock properties in domain #1 and domain #2 at the elevation of 140-120mL were characterized by weakness, further underscoring the geotechnical risks associated with this location.

Upon further investigation, it was determined that the observed failure from the graphical displacement was a brittle wedge failure, a type of failure mechanism commonly encountered in geotechnical engineering. The recorded total displacement of 2015.23 mm was notably the highest observed, indicating the severity of the failure and the extreme instability of the domains for mining operations.

As a result, based on the geotechnical monitoring feedback obtained from the prism monitoring system, it was concluded that this location poses a low risk for further mining activities. These professional observations and analyses provide critical information for decision-making and risk management in geotechnical engineering, underscoring the importance of thorough field observations, laboratory testing, and monitoring systems in identifying and mitigating geotechnical risks in the field.

# CHAPTER SIX

## 6. CONCLUSIONS AND RECOMMENDATIONS

In this chapter, my dissertation will culminate with a concise summary of the study's scope, leading to a final conclusion. Additionally, I will provide my personal insights and recommendations for future enhancements based on my perspective.

### 6.1 Conclusions

This study provides an overview of the reliability analysis conducted for pit slope design, accounting for various uncertainties in the input parameters. However, due to these uncertainties, the performance of the pit design cannot be accurately determined. Two approaches were employed to estimate the factor of safety (FoS) and probability of failure in pit slope design: deterministic and numerical modelling using Slide 2 and RS 2 software. The input parameters considered in the pit slope design included cohesion, friction angle, RQD, joint orientation, Young's modulus, and Poisson's ratio.

The results from Slide 2 indicated that the factor of safety increases with an increase in bench height, implying that more competent rock domains lead to better and safer slope designs. Subsequently, RS 2 was utilized with Monte Carlo Simulation (MCS) to verify the deterministic approach. MCS was chosen because it could handle a larger volume of input data compared to the Point Estimation Method provided by the software. The analysis deduced that domain #1 and #2 contains weak zones with poor rock quality. This was confirmed after conducting the model, as the estimated probability of failure decreased in each geotechnical domain when encountering competent materials. Consequently, domain #1 and #2 exhibited a maximum prone failure zone within the sector.

Furthermore, reliability analysis was introduced using the First Order Reliability Method (FORM) spreadsheet to determine the reliability of each domain in the sectors. It was confirmed that domain #1 and #2 had the lowest reliability indices, which implies that they are prone to high probability of failure. This was later compared with field reports obtained from the case study area, and it was concluded that these approaches used for slope stability analysis were in agreement with each other.

Upon thorough examination of domains #1 and #2, it was identified that certain factors, such as an increase in saturation, can weaken the rock mass by reducing its shear strength, causing it to become more prone to failure. This can be particularly concerning in areas where the slope is steep or where the rock mass is already compromised due to poor rock quality or joint orientation. Additionally, other factors such as snow fall, weathering, and erosion can also play a role in the weakening of the rock mass in domains #1 and #2. The presence of snow during the winter period, for instance, can cause cracks and fractures in the rock due to the expansion and contraction of water as it freezes and thaws, respectively. Also, which occurs over time due to exposure to atmospheric conditions, can alter the properties of the rock mass and reduce its strength. Furthermore, erosion caused by water or wind action, can remove the protective soil cover from the slope, exposing the rock mass to potential failure mechanisms, leading to the possibility of rock falling from the top bench, which could pose risks to mine personnel and mining activities in the surrounding area.

## 6.2 Recommendations

The geotechnical department must be acutely aware of the limitations of relying solely on a deterministic approach when designing slopes. In slope design, deterministic approaches use fixed values for input parameters to calculate factors of safety and make decisions. However, these fixed values may not accurately represent the true variability and uncertainties of the site conditions, which can lead to unreliable slope designs.

To address this issue, it is advisable to compare the results obtained from deterministic approaches with those obtained from a probabilistic approach. A probabilistic approach takes into account the variability and uncertainties of input parameters by incorporating statistical distributions, allowing for a more comprehensive and realistic assessment of slope stability. By considering a range of possible values for input parameters, rather than fixed values, a probabilistic approach provides a more robust and reliable assessment of the risk of slope failure.

Furthermore, the input parameters used in slope design are often affected by various sources of inconsistency, such as measurement errors, sampling variability, and geologic uncertainties. When using a deterministic approach alone, these inconsistencies can result in recurrent errors in pit slope design, leading to potential slope failures and associated economic losses. Therefore, it is essential to incorporate a probabilistic approach to account for these inconsistencies and reduce the risk of errors in slope design.

The displacement results obtained from the integration of the monitoring system in Bozshakol mine with pit slope design can serve as a valuable benchmark for geotechnical risk assessment. By analysing the rock mass displacement data obtained from the monitoring system, the thresholds of rock mass displacement that can lead to slope instability can be identified. This information can then be used to refine the design parameters and improve the accuracy and reliability of slope stability assessments.

Based on the outcomes obtained from the field observations and approaches used in this study, it can be deduced that there are possibilities of slope instability such as brittle wedge, circular, and planar failures that could propagates concurrently within these domains in the near future. However, since this slope stability analysis is a complex and multi-faceted field, and there are other factors beyond the input parameters that can influence this rock mass instabilities. For example, ground vibrations caused by blasting, changes in rock strength due to weathering or geological processes, and the orientation and characteristics of rock joints can all impact slope stability. For this reason, it is crucial to take note the various types of failure modes to avert for any loss of life or properties during mining activities.

In conclusion, the geotechnical department must be cautious of relying solely on deterministic approaches in slope design, and should instead incorporate probabilistic approaches to account for the variability and uncertainties of site conditions. The results obtained from the integration of monitoring systems with pit slope design can serve as a benchmark for geotechnical risk assessment. Therefore, further research is necessary to better understand the complexities of slope stability analysis and consider other influencing factors to ensure safe and economically viable slope designs in mining and geotechnical engineering practices.

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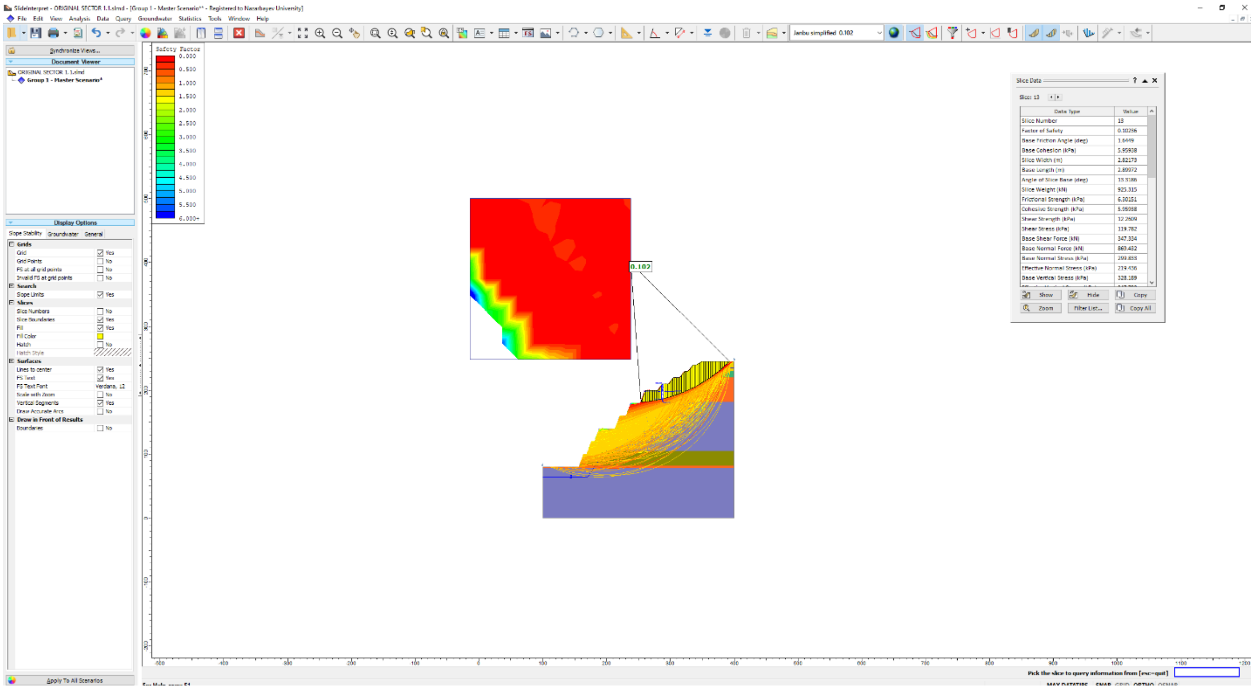
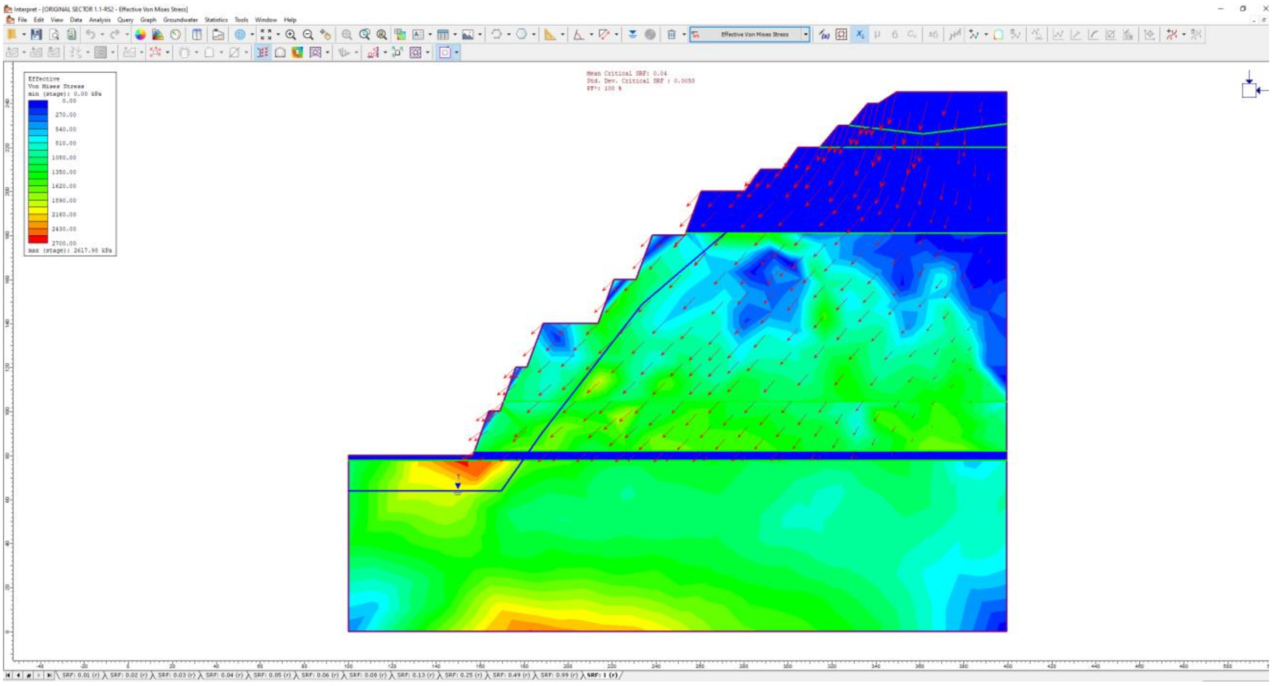
# APPENDICES



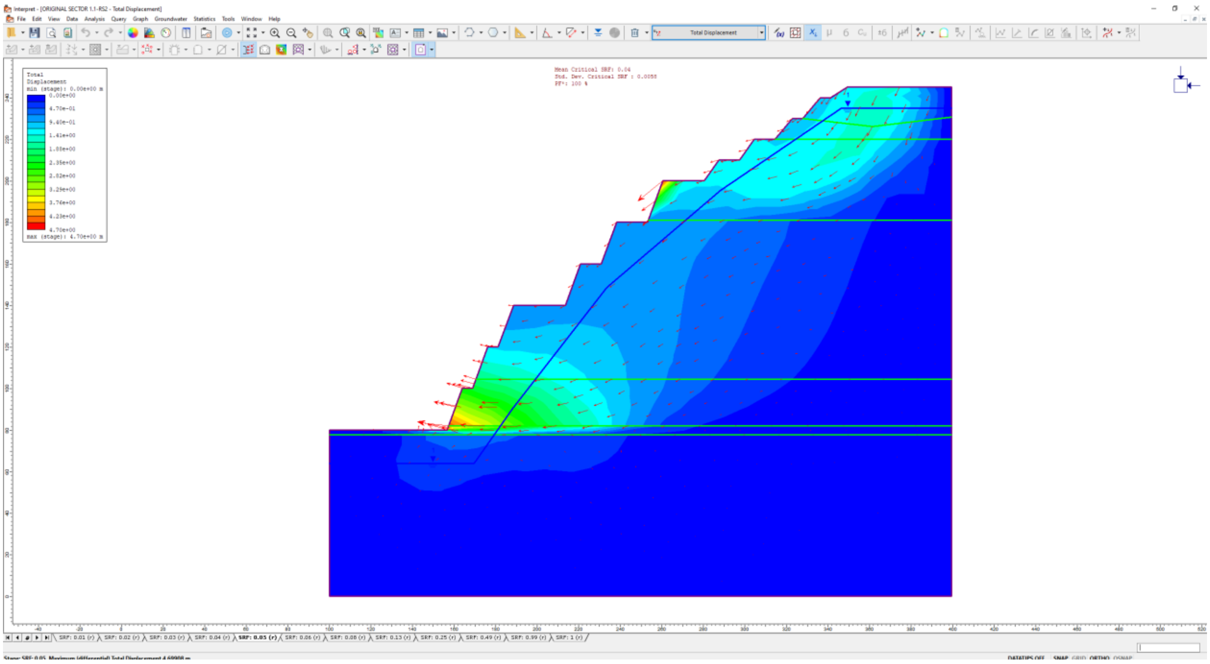
Slice Data

Slice: 13

| Data Type | Value |
|---|---|
| Slice Number | 13 |
| Factor of Safety | 0.10256 |
| Base Friction Angle (deg) | 1.6449 |
| Base Cohesion (kPa) | 5.95938 |
| Slice Width (m) | 2.82173 |
| Base Length (m) | 2.89072 |
| Angle of Slice Base (deg) | 15.3186 |
| Slice Weight (kN) | 925.315 |
| Frictional Strength (kPa) | 6.30151 |
| Cohesive Strength (kPa) | 5.95938 |
| Shear Strength (kPa) | 12.2609 |
| Shear Stress (kPa) | 119.782 |
| Base Shear Force (kN) | 347.334 |
| Base Normal Force (kN) | 869.432 |
| Base Normal Stress (kPa) | 299.893 |
| Effective Normal Stress (kPa) | 219.436 |
| Base Vertical Stress (kPa) | 328.189 |



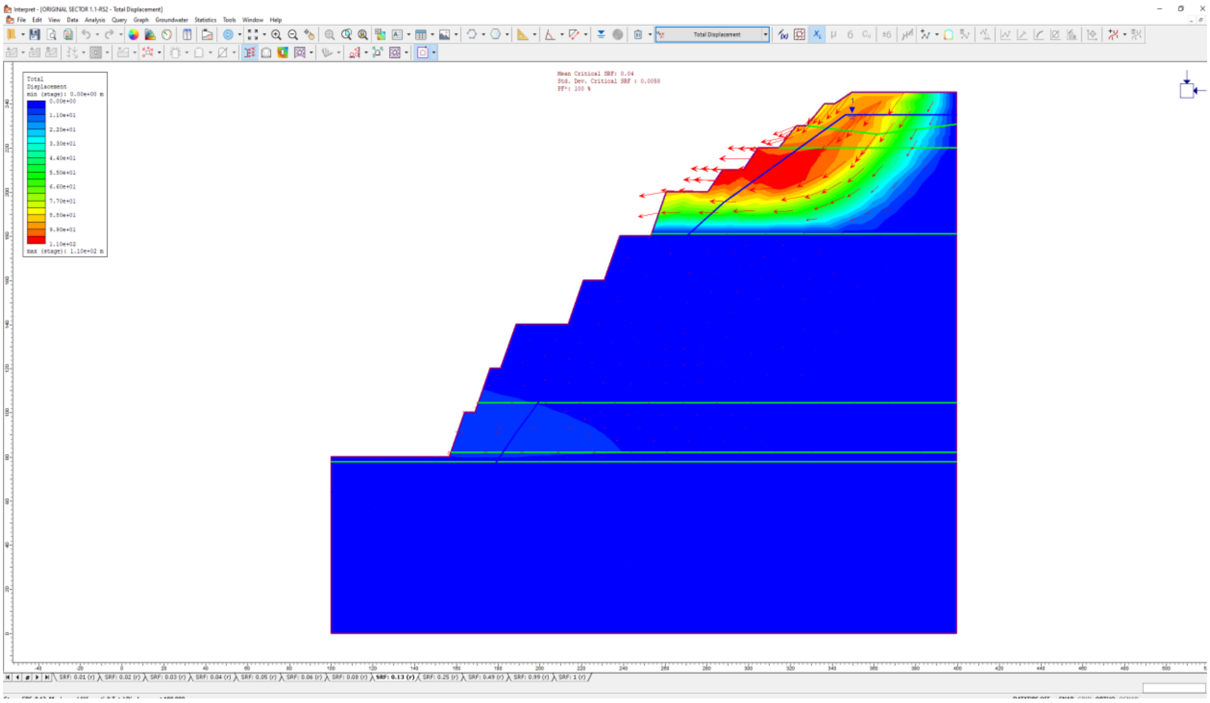
Mean Critical SRF: 0.04
Std. Dev. Critical SRF : 0.0058
RF: 100 %



Total Displacement
min (stage): 0.00e+00 m
0.00e+00
4.70e-01
9.40e-01
1.41e+00
1.88e+00
2.35e+00
2.82e+00
3.29e+00
3.76e+00
4.23e+00
4.70e+00
max (stage): 4.70e+00 m

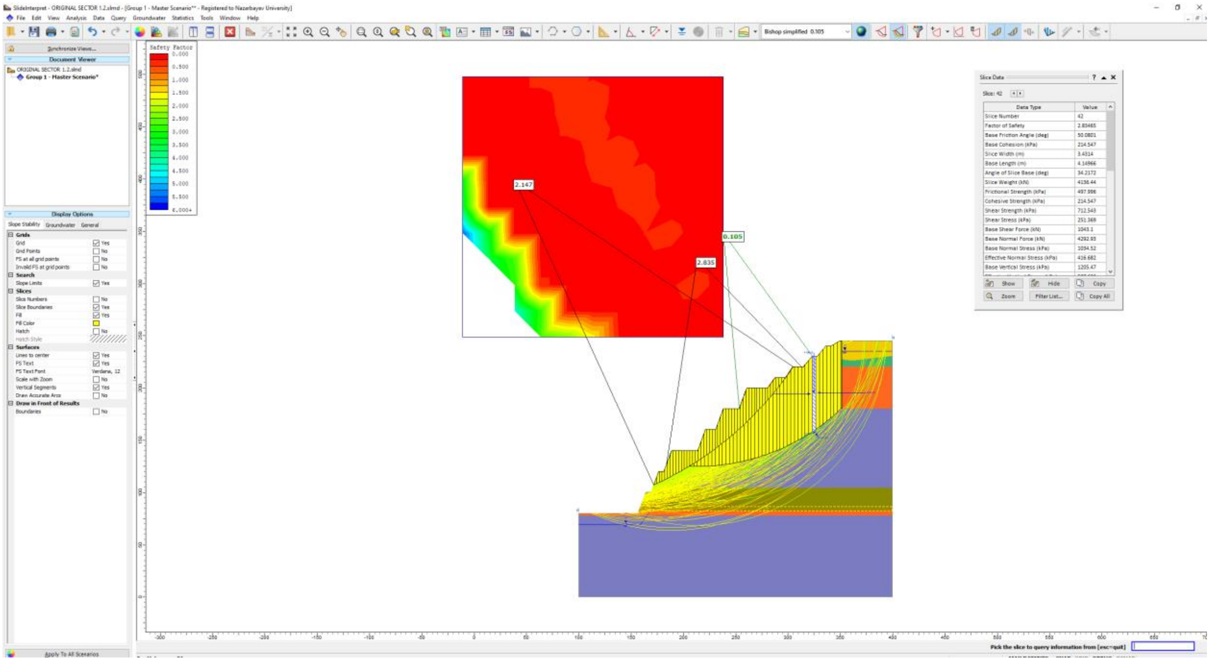
Mean Critical SRF: 0.04
Std. Dev. Critical SRF : 0.0058
PF*: 100 %

Stage: SRF: 0.05, Maximum (differential) Total Displacement 4.69908 m



Total Displacement
min (stage): 0.00e+00 m
0.00e+00
1.10e+01
2.20e+01
3.30e+01
4.40e+01
5.50e+01
6.60e+01
7.70e+01
8.80e+01
9.90e+01
1.10e+02
max (stage): 1.10e+02 m

Mean Critical SRF: 0.04
Std. Dev. Critical SRF : 0.0058
PF*: 100 %



Display Options

Grids

Grid ☑ Yes
Grid Points ☐ No
FS at all grid points ☐ No
Invalid FS at grid points ☐ No

Search

Slope Limits ☑ Yes

Slices

Slice Numbers ☐ No
Slice Boundaries ☑ Yes
Fill ☑ Yes
Hatch ☐ No

Surfaces

Lines to center ☑ Yes
FS Text ☑ Yes
FS Text Font Verdana, 12
Scale with Zoom ☐ No
Vertical Segments ☑ Yes
Draw Accurate Arcs ☐ No

Draw in Front of Results

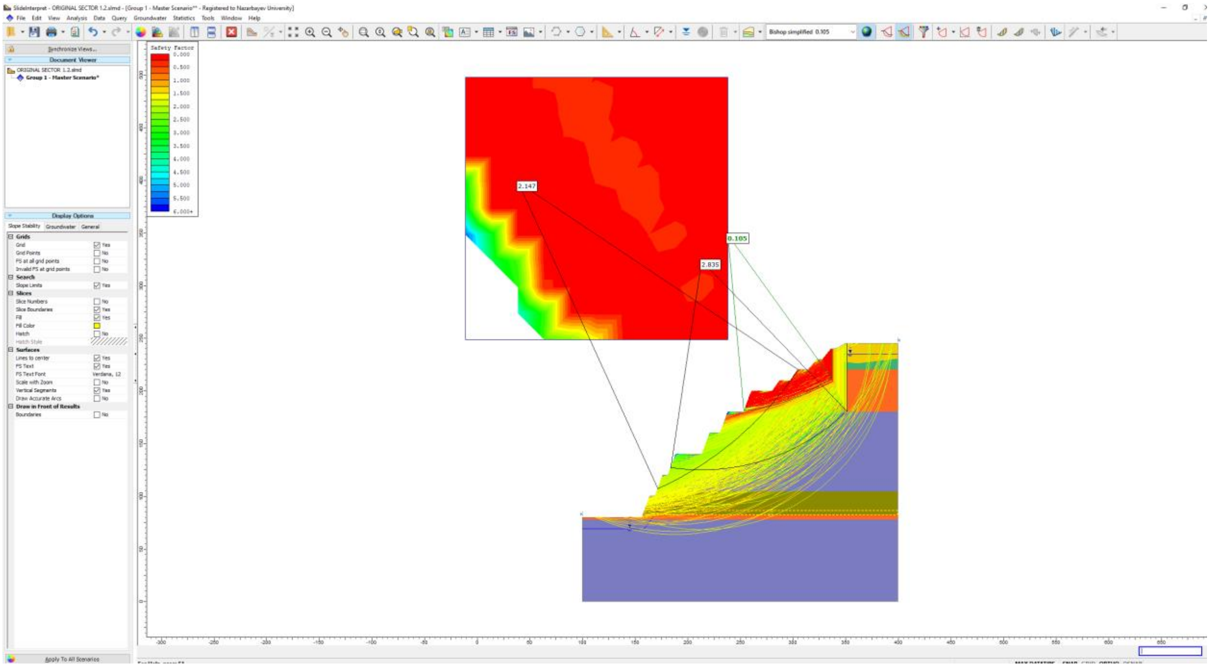
Boundaries ☐ No

Slice Data

Slice: 42

| Data Type | Value |
|---|---|
| Slice Number | 42 |
| Factor of Safety | 2.83465 |
| Base Friction Angle (deg) | 50.0801 |
| Base Cohesion (kPa) | 214.547 |
| Slice Width (m) | 3.4314 |
| Base Length (m) | 4.14966 |
| Angle of Slice Base (deg) | 34.2172 |
| Slice Weight (kN) | 4136.44 |
| Frictional Strength (kPa) | 497.996 |
| Cohesive Strength (kPa) | 214.547 |
| Shear Strength (kPa) | 712.543 |
| Shear Stress (kPa) | 251.369 |
| Base Shear Force (kN) | 1043.1 |
| Base Normal Force (kN) | 4292.93 |
| Base Normal Stress (kPa) | 1094.52 |
| Effective Normal Stress (kPa) | 416.682 |
| Base Vertical Stress (kPa) | 1205.47 |

Pick the slice to query information from [esc=quit]



Display Options

Grids

Grid ☑ Yes
Grid Points ☐ No
FS at all grid points ☐ No
Invalid FS at grid points ☐ No

Search

Slope Limits ☑ Yes

Slices

Slice Numbers ☐ No
Slice Boundaries ☑ Yes
Fill ☑ Yes
Hatch ☐ No

Surfaces

Lines to center ☑ Yes
FS Text ☑ Yes
FS Text Font Verdana, 12
Scale with Zoom ☐ No
Vertical Segments ☑ Yes
Draw Accurate Arcs ☐ No

Draw in Front of Results

Boundaries ☐ No