BACHELOR THESIS

THREE-DIMENSIONAL SLOPE STABILITY ANALYSIS AND VOLUMETRIC ESTIMATION OF SLOPE FAILURE

submitted by

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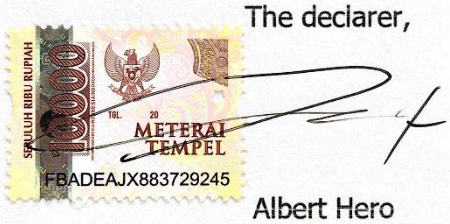

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ABSTRACT

PT X is a nickel ore mining company based in Pomalaa, Southeast Sulawesi. The hauling road of PT X is through mountainous terrain. Slope failure becomes an issue and might be risking the safety of operations and production. Thus, geotechnical analysis is required to analyze the slope behavior. Most geotechnical analyses are conducted in 2D and assume that the slope has infinite dimensions. In actual conditions, all slopes have finite dimensions. Thus, three-dimensional effects should be considered. This research determines the instability mode, evaluates the factor of safety (FoS), and also estimates the volume of failure. The 3D kinematic analysis (KA) is used to get the slope mode of instability, and the 3D limit equilibrium method (LEM) is used to determine the FoS of the slope and estimate the volume of failure. The 3D kinematic analysis determines that the mode of failure for the slope is wedge sliding, with the direction of sliding heading towards N64°E. The LEM analysis is also integrated with the sliding direction and discontinuities based on KA result. The 3D LEM analysis also generates a volume estimation for each calculation method based on the global minimum slip surface. Although the slope stability is relatively stable and reaches the acceptance criteria of the FoS, the potential for wedge sliding should be a concern to have detailed research to determine the best stabilization method.

Keywords: geotechnical investigation, 3D model, 3D structure extractions, drone mapping, semi-automated

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Albert Hero

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CHAPTER I

INTRODUCTION

1.1 Background

The risk of slope failure has attracted worldwide attention, and it is commonly understood that the risk of slope failure involves the failure probability and the resulting consequences (Li and Chu, 2015). Slope stability has a crucial part in an open pit mine's production and safety operations. Slopes must be designed with geotechnical considerations to reduce the risk of landslides, which could disrupt mining operations. (Azizi *et al.*, 2019).

PT X is an Indonesian nickel ore mining company based in Pomalaa, Southeast Sulawesi. This company's project area (port) is around 16 kilometers from the concession area. The only way to get to that port is to take a hauling road that runs through the terrain of a mountain. Due to the inappropriate geotechnical requirement, a slope on one side of the road in some locations requires a geotechnical investigation. It is important to remember that this is the only way to get to the port, and if it fails, the production and safety operations will be seriously impacted.

Slope stability analyses are frequently performed using limit equilibrium methods. However, most of these studies are limited to two dimensions, making it impossible to accurately estimate a failure based on actual three-dimensional (3D) properties (Chen *et al.*, 2001). It is common to assume that the problem can be treated two-dimensional for practical purposes in slope stability analysis and back analysis of slides. The slip surface is assumed to be infinitely wide, and thus the three-dimensional or end effects are ignored. However, all slides have finite dimensions, and where these are such that three-dimensional effects become significant, these should be properly considered (Gens *et al.*, 1988).

The safety factors of two-dimensional analyses determined through various methods are often conservative when compared to those of three-dimensional analyses, with a percentage difference of up to 30%. The back analysis of the parameters of unstable slopes will overestimate the material strength parameters, resulting in dangerous results from the two-dimensional analysis (Su and Shao, 2021).

Since the last decade, 3-dimensional slope stability analysis has developed significantly, and several geomechanical researchers have proposed concepts for optimizing slope design concerning mining operation's economics and safety. The 3-dimensional slope stability analysis methods have addressed the assumption of spatial parameters in determining safety factors and failure probability, allowing for the determination of the volume of failed material and the location of the most critical slopes (Azizi *et al.*, 2020). Due to the direct implications of slope failure, it may result in financial loss. Because the grade of failure result is directly related to the volume of the failure, it is critical to estimate the volume of failure that may occur in order to mitigate slopes (Azizi *et al.*, 2019).

This research is conducting the geotechnical assessment of the slope in the form of a 3-dimensional slope stability analysis. It produces the modes of slope instability, the factor of safety, and estimates the volume of the failure material.

1.2 Problem Statements

Slope stability is a critical aspect of a mine's safety operations and production. To reduce the possibility of landslides, a slope must be designed using geotechnical considerations. However, most of these studies are conducted in two dimensions, making it impossible to predict failure based on actual conditions or three-dimensional

(3D) properties. Based on these issues, this research conducts a slope stability analysis using actual conditions and spatial properties.

The formulation of the problem in this research are as follows.

1. The width of the slope is ignored in the 2D slope stability analysis, which assumes it to be infinite. The analysis is more comprehensive when using the 3D slope stability analysis to assess the slope stability based on the actual conditions.
2. The slope failure can't be estimated in a 2D slope stability analysis due to the assumption that the slope width is infinite. However, it can be estimated using actual conditions in a 3D slope stability analysis.

1.3 Research Objectives

The objectives in this research are as follows.

1. To determine the mode of slope failures using the 3-dimensional kinematic analysis method.
2. To evaluate the value of the slope safety factor using the 3-dimensional analysis method.
3. To estimate the volume of slope failure.

1.4 Research Uses

The results of this research can be used as the basis for further slope analysis, as well as to show how a semi-automated geotechnical investigation can be used to evaluate slope stability.

1.5 Research Location

The research has located at the concession area of PT X, which is in Pomalaa District, Kolaka Regency, Southeast Sulawesi. PT X is a private company that mines lateritic nickel ore deposits. This research is focusing on a slope nearly hauling road with geographical coordinates on 4°13’46.272” S and 121°38’1.428” E (see Figure 1.1). The PT X mining site can be reached via a one-hour flight from Makassar (Sultan Hasanuddin International Airport) to Kolaka (Sangia Nibandera Airport), followed by a 30-minute drive to the PT X office (Dawi-Dawi, Pomalaa District, Kolaka Regency), which is 23 kilometers away. Although the trip from the PT X office to the research site can be completed in 30 minutes by land transportation covering a distance of 8 kilometers.

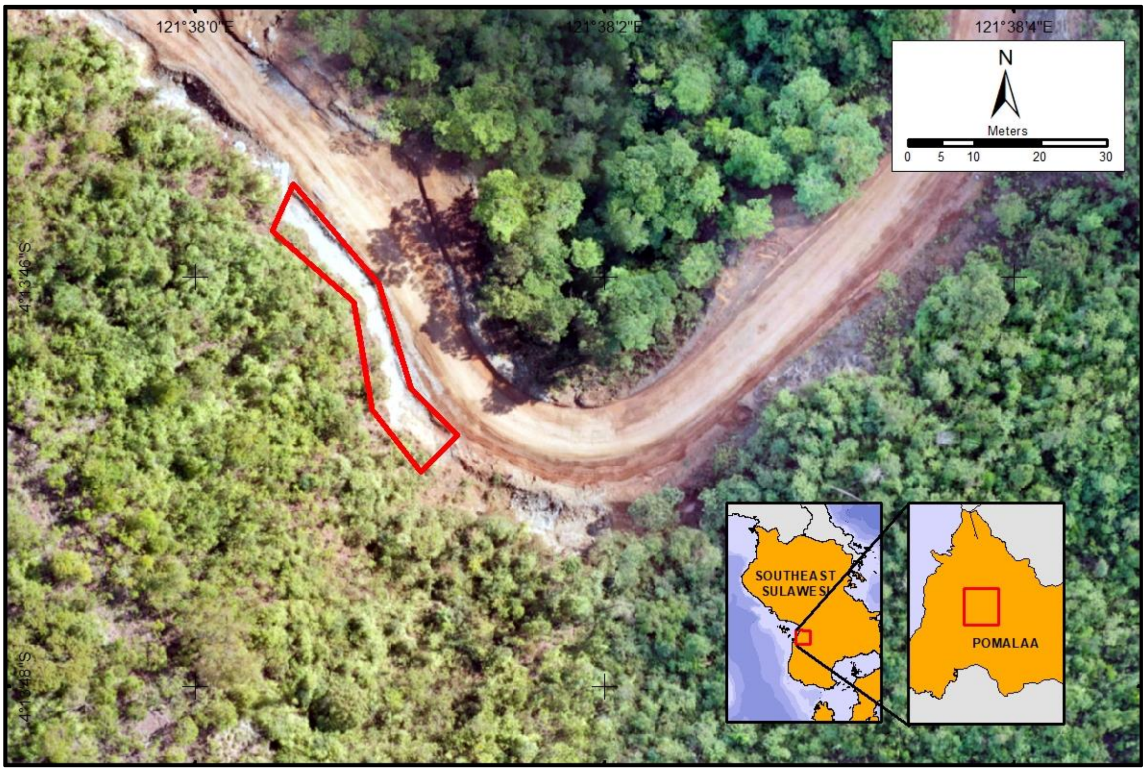

Figure 1.1 Research location

1.6 Research Stages

This research is conduct through several stages, as follows:

1. Topic Determination

   This stage is the initial stage to determine the focus of this research. The predetermined topic serves as a guideline in the discussion and problem-solving in research.

2. Literature Study

   The literature study was carried out to review any publication that related to this research. The references are coming from books, journals, and any kind of resources. The outcomes for these stages are based on knowledge to conduct this research.

3. Formulation of The Problems

   Formulation of the problem is a step in the research process that states the problem as the major topic of discussion. The analysis of the modes of failure and slope safety factors, as well as the volume estimation of landslides that may occur in the research area, are the main topics of discussion in this research.

4. Data Acquisition

   The data for this research comes from PT X, a nickel ore mining company based in Kolaka Regency, Southeast Sulawesi. The datasets obtained are drone imagery and rock properties data. Drone imagery is a set of photos captured by a drone to show the topographic condition of the observation area as well as the structural condition of the slope face. The company also provides rock properties data that is obtained from rock properties tests, which results in physical and mechanical rock properties data.

5. Data Processing

   The processing stages perform two analyses, which are 3D kinematic analysis and 3D limit equilibrium analysis. This process are using secondary data. Drone images are converted to point clouds and a three-dimensional slope model. These models use for those analyses. ROck Slope Kinematic Analysis (ROKA) and Dips Version 7.016 are used for 3D kinematic analysis to determine the modes of slope instability, while Plaxis 3D LE is used for 3D limit equilibrium analysis with lithological data and rock properties as input parameters to determine the slope's safety factor and estimate the volume of slope failure.

6. Thesis Preparation

   The final report is prepared following the completion of data acquisition and processing. Conclusions are derived based on the results of data processing that has been carried out on the problems that have been stated.

7. Thesis Seminar and Submission

   A thesis seminar is held where the thesis is presented to the supervisor and examiner lecturer team. The thesis is then submitted to the library of Hasanuddin University's Mining Engineering Department once it has been revised and presented in the thesis seminar.

CHAPTER II

SLOPE STABILITY ANALYSIS AND SEMI-AUTOMATED GEOTECHNICAL INVESTIGATION

2.1 Slope Stability

A variety of engineering activities necessitate rock excavation, and this man-made excavation disrupted the slope's natural condition. Many components of mining require excavation activities, and slope stability is always a concern during these activities. If the forces that act on a slope are not stable, the instability may manifest itself as displacement, which may or may not be acceptable, or the slope may collapse suddenly or progressively (Wyllie and Mah, 2005), slope failure can occur as schematically shown in Figure 2.1.

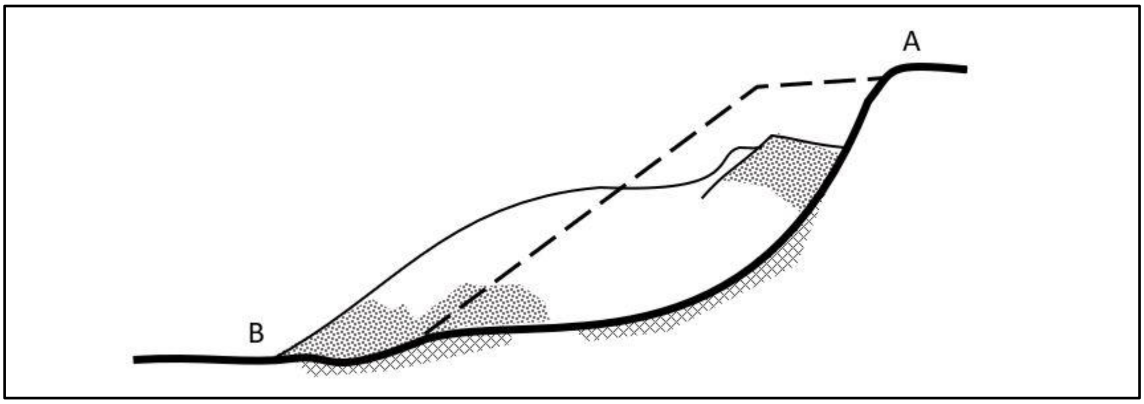

Figure 2.1 Slope failure scheme (Wyllie and Mah, 2005)

The slope face may deform during excavation work on natural slopes, reducing shear strength and increasing the risk of slope failure. The majority of man-made slope failures are due to design errors, which include geometric design, such as slope inclination and height, as well as an inability to anticipate load and soil resistance (Sutejo and Gofar, 2015).

The stability of a slope can be described in some of the following terms based on these concepts of slope stability (Wyllie and Mah, 2005):

1. Safety factor (FS); stability measured by the slope's limit equilibrium, which is stable if FS > 1.
2. Strain; failure defined by the initiation of strains that are great enough to prevent the slope from operating safely, or by the rate of movement exceeding the rate of open-pit mining.
3. Failure probability; stability is measured by the probability distribution of the difference between resisting and displacing forces, which are both expressed as probability distributions.
4. Load and resistance factor design (LRFD); the factored resistance must be more than or equal to the sum of the factored loads to be stable.

In nature, soil and rock are generally in equilibrium, meaning that the stress distribution on the soil or rock is in a steady state. If the soil or rock is subjected to an activity, such as excavation, subsidence, stockpiling, transportation, erosion, or other activities that disrupt the balance, the soil or rock will try to reach a new equilibrium by releasing the load, especially in the form of failures. In principle (see Figure 2.2), on a slope, there are a kind of forces, namely the force that makes the rock mass move (driving force) and the force that holds the rock mass (resistance force). A slope will fail if the driving force is greater than the resistance. Mathematically, slope stability can be expressed in terms of factor of safety (FoS) where (Arif, 2016):

$$Factor\ of\ Safety = \frac{Resistance\ Forces}{Driving\ Forces} \tag{2.1}$$

FoS > 1, the slope is considered stable

FoS < 1, the slope is considered unstable

FoS = 1, the slope is in a state of equilibrium, but it will immediately fail if it gets a little disturbance

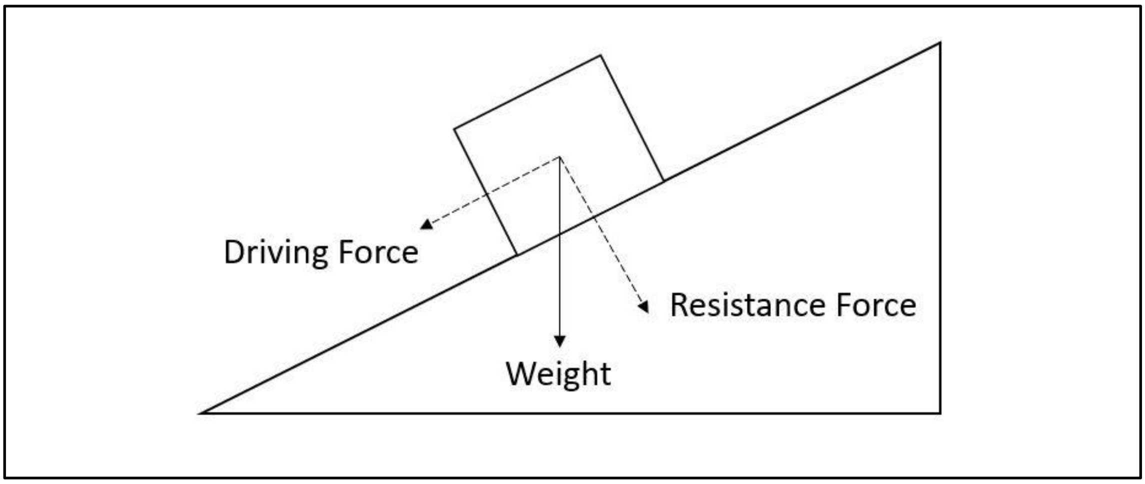

Figure 2.2 Forces that have acted on a slope (Arif, 2016)

2.2 Modes of Slope Instability

Different kinds of slope failure are associated with particular geological structures. Thus, it is critical for the slope designer to be able to identify potential stability issues early in the project. The stereographic projection should be used to identify some of the structural patterns, which should subsequently be examined using pole plots. The stereographic projection provides for the representation and analysis of three-dimensional orientation data in two dimensions. The plot shows the concentrations of poles that represent the orientations of sets of discontinuities (Wyllie and Mah, 2005; Hoek and Bray, 1981).

Figure 2.3 shows the four types of failures considered, as well as typical pole plots of geological conditions that are likely to cause such failures. The following are the main types of block failures in slopes, and also the structural geology conditions that are likely to cause them: (a) plane failure in rock with persistent joints dipping out of the slope face and striking parallel to the face; (b) wedge failure on two intersecting discontinuities; (c) toppling failure in strong rock with discontinuities dipping steeply into the face; and (d) circular failure in the rock fill, very weak rock, or closely fractured rock with randomly oriented fracture (Wyllie and Mah, 2005; Hoek and Bray, 1981).

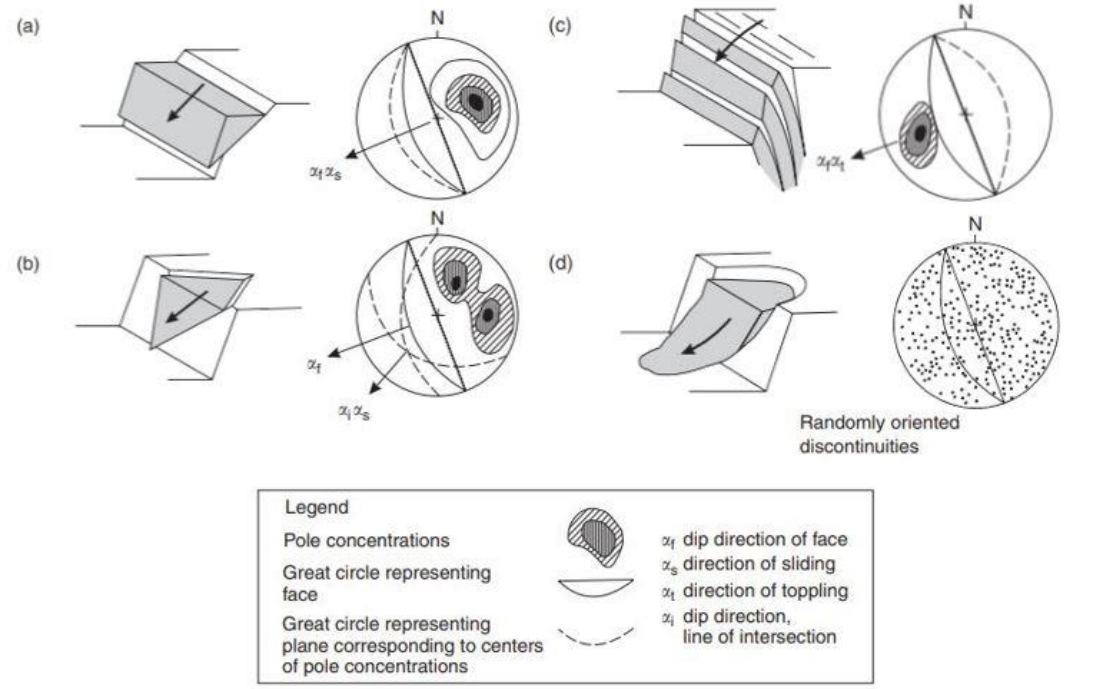

Figure 2.3 Modes of slope instability (Wyllie and Mah, 2004; Hoek and Bray, 1981)

2.2.1 Plane Failure

A plane failure is a rather uncommon problem on rock slopes because all of the geometric conditions required to produce such a failure only occur infrequently on an actual slope. However, it would be a mistake to ignore the two-dimensional case, as there are numerous significant lessons to be learned by studying the mechanics of this simple failure mode. Plane failure is especially useful for highlighting the slope's sensitivity to changes in shear strength and groundwater conditions, which are less visible when dealing with the more complex mechanics of three-dimensional slope failure (Wyllie and Mah, 2005).

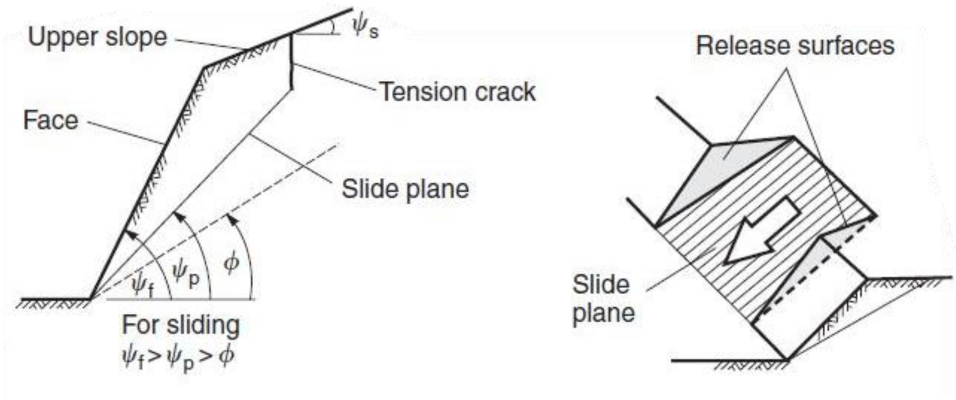

Figure 2.4 Geometry of slope exhibiting plane failure (Wyllie and Mah, 2005)

Figure 2.4 shows a typical plane failure in a rock slope, where a block of rock has slid out of the face on a single plane. The following geometrical requirements must be met in order for this type of failure to occur (Wyllie and Mah, 2005).

a. The plane on which sliding occurs must strike parallel or nearly parallel (within approximately $\pm 20°$) to the slope face.
b. The sliding plane must "daylight" in the slope face, which means that the dip of the plane must be less than the dip of the slope face, that is, $\psi_p < \psi_f$.
c. The dip of the sliding plane must be greater than the angle of friction of this plane, that is, $\psi_p > \phi$.
d. The upper end of the sliding surface either intersects the upper slope or terminates in a tension crack.
e. Release surfaces that provide negligible resistance to sliding must be present in the rock mass to define the lateral boundaries of the slide. Alternatively, failure can occur on a sliding plane passing through the convex "nose" of a slope.

Based on the geometrical requirements the plane is potentially sliding when the condition fit the Equation 2.2.

$$\psi_f > \psi_p > \phi \quad (2.2)$$

2.2.2 Wedge Failure

The failure of slopes with discontinuities that strike the slope face obliquely occurs when a wedge of rock slides along the line of intersection of two such planes. Considering wedge failures can occur under a much wider range of geology and geometric conditions than plane failures, wedge stability research is an important part of rock slope engineering (Wyllie and Mah, 2005).

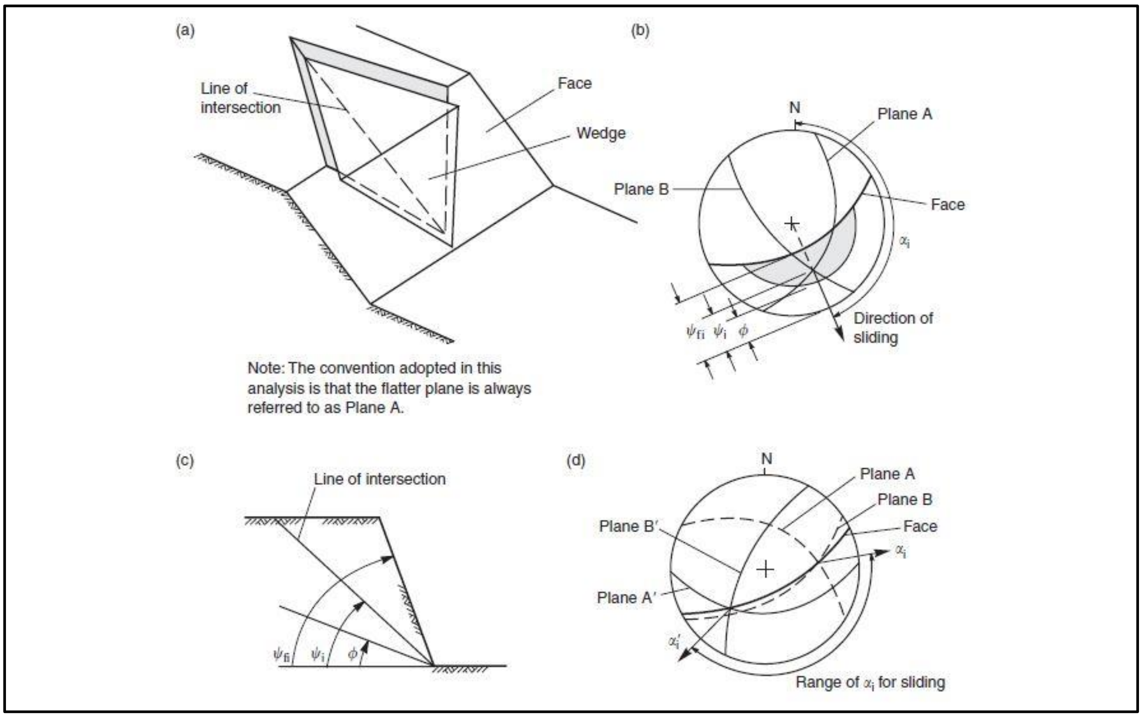

Figure 2.5 Geometric conditions for wedge failure (Wyllie and Mah, 2005)

The illustration of the geometric condition for wedge failure and the orientation of the intersection in pictorial view and also in stereoplot where wedge failure is feasible is shown in Figure 2.5. Based on the geometry as defined before, the general conditions for wedge failure are as follows (Wyllie and Mah, 2005).

a. Two planes will always intersect in a line (Figure 2.5(a)). On the stereonet, the line of intersection is represented by the point where the two great circles of the planes intersect, and the orientation of the line is defined by its trend (α_i) and its plunge (ψ_i) (Figure 2.5(b)).

b. The plunge of the line of intersection must be flatter than the dip of the face, and steeper than the average friction angle of the two slide planes, that is $\psi_{fi} > \psi_i > \phi$ (Figure 2.5(b) and (c)). The inclination of the slope face ψ_{fi} is measured in the view at right angles to the line of intersection. Note the ψ_{fi} would only be the same as ψ_f, the true dip of the slope face, if the dip direction of the line of intersection were the same as the dip direction of the slope face.
c. The line of intersection must dip in a direction out of the face for sliding to be feasible; the possible range in the trend of the line of intersection is between α_i and α_i'.

In general, sliding may occur if the intersection point of the two great circles of the sliding plane is inside the shaded area on Figure 2.5. In other words, the stereonet is revealing whether wedge failure is kinematically possible. The stereonet can be used to identify the trend (α_i) and plunge (ψ_i) of the line of intersection of planes A and B, and also Equations 2.3 and 2.4 can be used to calculate it. Where α_A and α_B are the dip directions and ψ_A and ψ_B are the dips of the planes (Wyllie and Mah, 2005).

$$\alpha_i = \tan^{-1}\left(\frac{\tan\psi_A \cos\alpha_A - \tan\psi_B \cos\alpha_B}{\tan\psi_B \sin\alpha_B - \tan\psi_A \sin\alpha_A}\right) \tag{2.3}$$

$$\psi_i = \tan\psi_A \cos(\alpha_A - \alpha_i) = \tan\psi_B \cos(\alpha_B - \alpha_i) \tag{2.4}$$

2.2.3 Toppling Failure

Toppling failure is a mode of failure that occurs when columns or blocks of rock rotate around a fixed base. Toppling failure can occur in a variety of ways, including block, flexural, block-flexur, and secondary toppling (see Figure 2.6). The importance of distinguishing between types of toppling comes from the fact that there are numerous techniques of stability analysis for toppling failures, and the right analysis must be used in design (Wyllie and Mah, 2005).

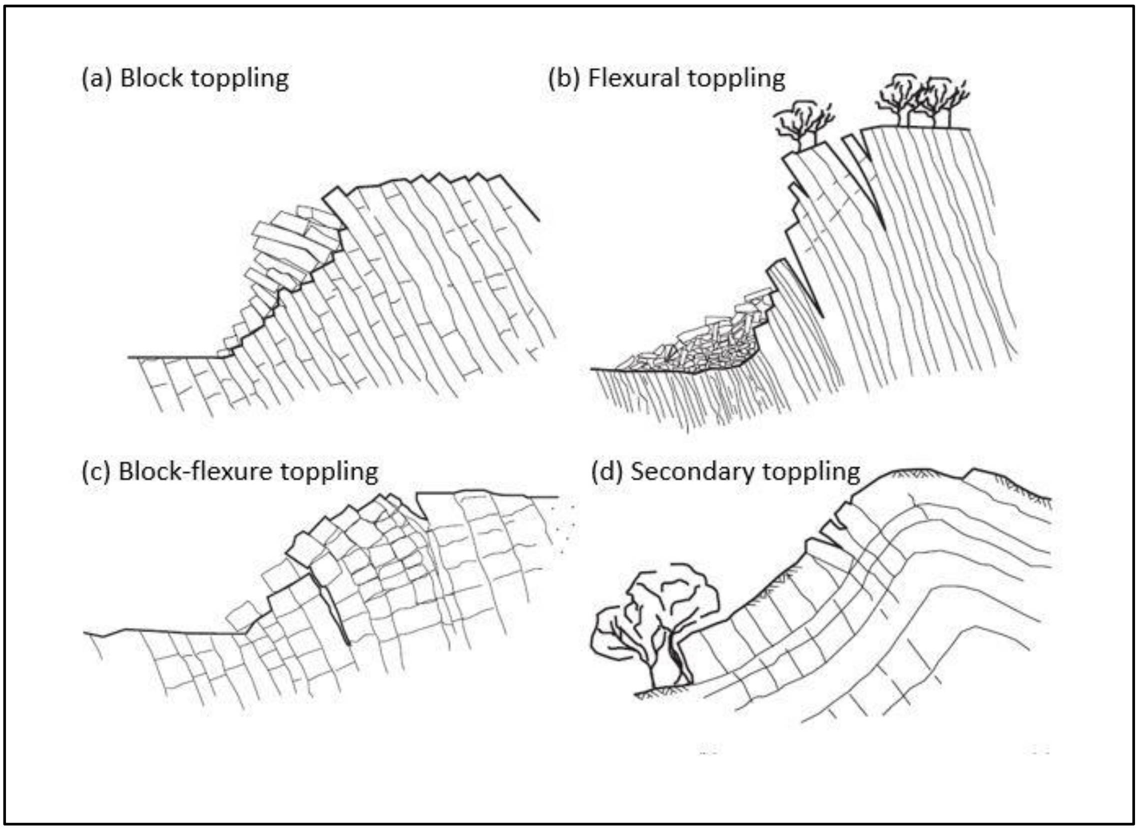

Figure 2.6 Toppling failure types (Wyllie and Mah, 2005)

Observations of topples in the field suggest that instability can occur when the dip direction of the planes forming the blocks' sides is within about 10 degrees of the slope face's dip direction. The potential for toppling can be assessed from two kinematic tests. These tests can be performed at the shape of the block, and the relationship between the dip of the planes that form the slabs and the face angle. The block shape test requires that distinguish stable, sliding, or toppling blocks with a height (y) and width (Δx) on a plane dipping angle (ψ_p) are shown in Figure 2.7. If the friction angle between the block's base and the plane is ϕ_p, then the block will be stable against sliding when the base plane's dip is less than the friction angle (see Equation 2.5) but will topple when the center of gravity of the block lies outside the base (see Equation 2.6) (Wyllie and Mah, 2005).

$$\psi_p < \phi_p \tag{2.5}$$

$$\frac{\Delta x}{y} < \tan\psi_p \tag{2.6}$$

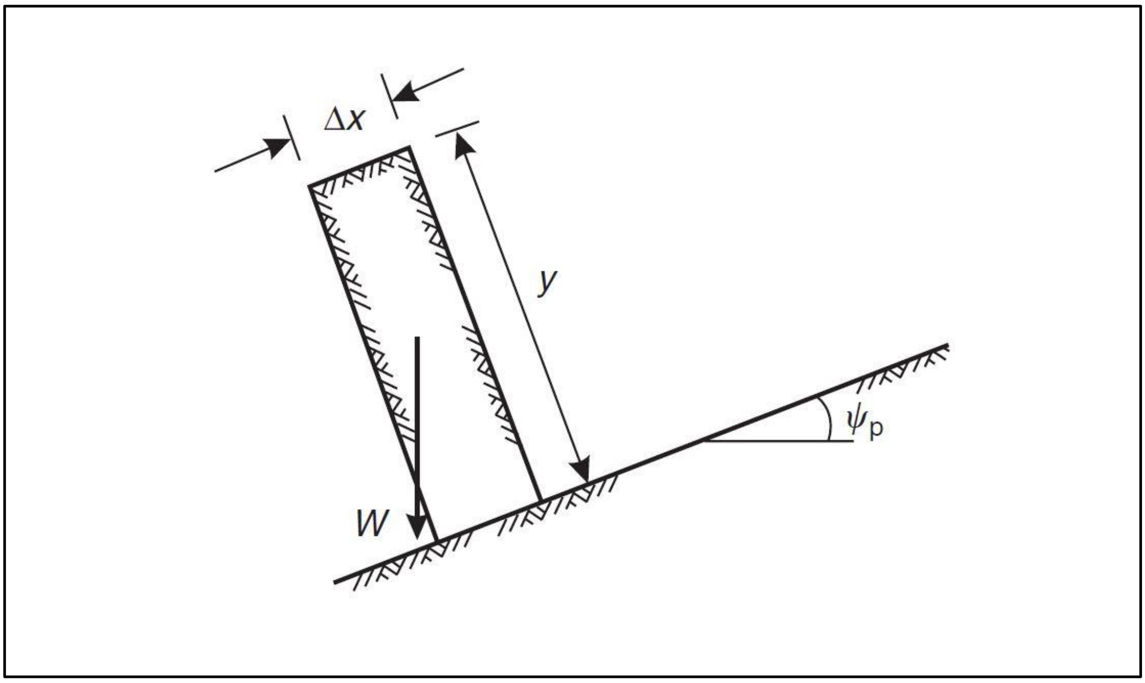

Figure 2.7 Block height/width test for toppling

For toppling to occur, the interlayer slip test requires shear displacement on the face-to-face contacts on the top and bottom faces of the blocks. If the following conditions are met (see Figure 2.8(a)), sliding on these faces will occur. The stress condition near the slope face is uniaxial with the normal stress direction (σ) that aligned parallel to the slope face. When the layers slip past each other, normal stress must be inclined at amount an angle of block's friction angle (ϕ_d) with the normal to the layers. If ψ_f is the slope face's dip and ψ_d is the dip of the planes forming the blocks' sides, then the condition for interlayer slip is given by Figure 2.8(b) (Wyllie and Mah, 2005).

$$\left(180 - \psi_f - \psi_d\right) \geq (90 - \phi_d) \tag{2.7}$$

or for kinematic test as defined on lower hemisphere stereographic projection (see Figure 2.8(c))

$$\psi_d \geq \left(90 - \psi_f\right) + \phi_d \tag{2.8}$$

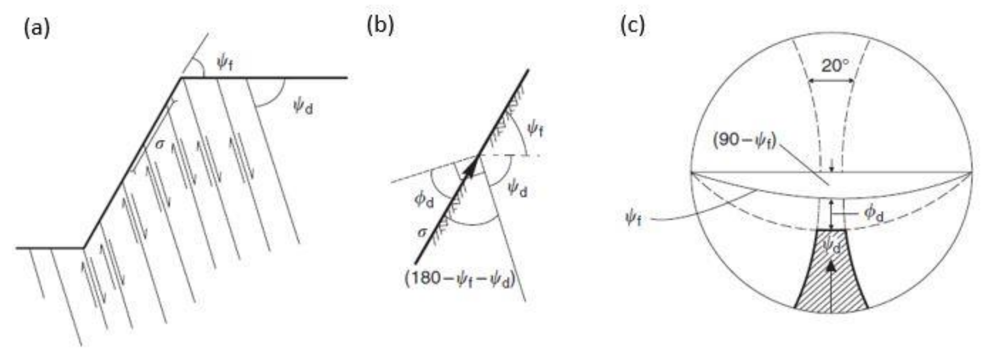

Figure 2.8 Kinematic condition for toppling (Wyllie and Mah, 2005)

Another kinematic condition for toppling is that the planes composing the blocks should strike roughly parallel to the slope face, allowing each layer to topple freely without being constrained by adjacent layers. Observations of topples in the field show that instability is possible when the dip direction of the planes (α_d) forming the sides of the blocks, is within about 10° of the dip direction of the slope face (α_f) as shown in Equations 2.9 (Wyllie and Mah, 2005).

$$\left|(\alpha_f - \alpha_d)\right| < 10° \tag{2.9}$$

2.2.4 Circular Failure

Besides the stability of rock slopes containing well-defined sets of discontinuities, it is also important to design cuts in weak materials such as highly weathered or closely fractured rock and rock fills. Failure occurs at a surface that approaches a circular form in these materials (Wyllie and Mah, 2005).

The concept that geological features such as bedding planes and joints, which divide the rock into a discontinuous mass, affect the failure of rock slopes. The slide surface is generally defined by one or more of the discontinuities. In the case of a closely fractured or highly weathered rock, however, a clearly defined structural pattern no longer exists, and the slide surface is free to find the least resistance through the slope. Due to the absence of well-defined structure, the discontinuity orientation would be randomly plotted on the streonet as shown in Figure 2.9. Most stability theories are based on observations of slope failures in these materials, which suggest that this sliding surface assumes the shape of a circle (Wyllie and Mah, 2005).

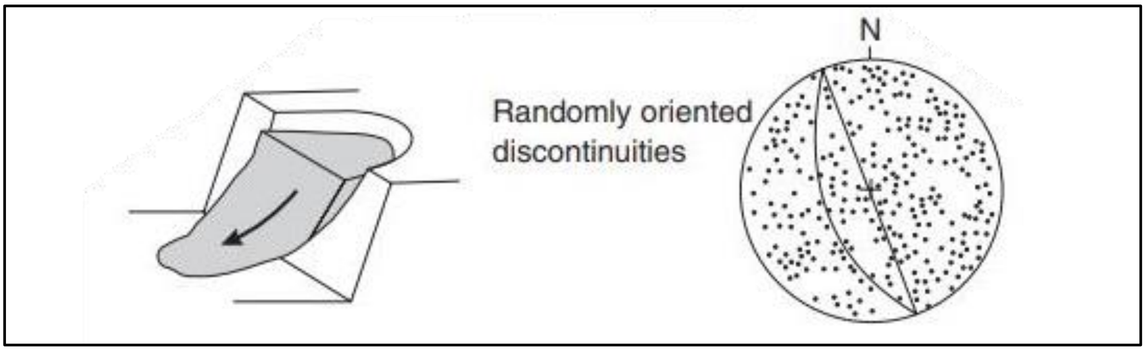

Figure 2.9 Geometric condition for circular failure (Wyllie and Mah, 2005)

2.3 Rock Properties

The main issues in these developments and advances include modeling of rock behavior, design methodologies for rock structures, and rock testing methods. The input factors, such as boundary conditions and material and rock mass properties, have a significant impact on the models created. As a result, experimental investigations and the identification of rock engineering properties are still important in rock mechanics and rock engineering applications (Ulusay, 2015). For geological engineering design and construction, rock physical and mechanical properties are critical. Many geological disasters are caused by misunderstandings of rock mechanical properties in the mining industry (Peng and Zhang, 2007).

It is necessary to have knowledge about material properties in order to conduct slope design. It can be carried out in a variety of ways, including in-situ and laboratory tests. The mechanical test was performed on the rock directly on the field in the in-situ test, whereas the laboratory test used rock samples from the site. Physical properties of rock, such as density, specific gravity, porosity, and void ratio; and mechanical properties of rock, such as compressive strength, tensile strength, shear strength, Young's modulus, and Poisson's ratio, would be determined in the laboratory (Arif, 2016).

2.3.1 Physical properties of rock

The majority of rocks are heterogeneous composites; only monomineralic rocks, such as rock salt or anhydrite, have a single mineral type. If pores and cracks filled with fluids are present, heterogeneity becomes more contrasted. Physical rock properties are influenced by mineral content, porosity/fracturing, and internal rock structure. Physical rock properties, on the other hand, can be used to characterize rocks in terms of properties and parameters of interest (for example geomechanical properties) (Schön, 2015).

Laboratory tests can be used to determine the physical properties of rock for geotechnical applications, and one of the properties that can be determined is the rock's density (Arif, 2016). Density (ρ) is defined as the quontient of mass (m) and volume (V) of material (see Equation 2.10). The International System of Units (SI) for density is kg m^{-3} (Schön, 2015).

$$\rho = \frac{m}{V} \tag{2.10}$$

where

ρ = density (kg/m^3);

m = mass of rock (kg);

V = volume of rock (m^3).

The density of rocks can be tested in a variety of conditions, including while the rock is in its natural state, as well as when it is dry or saturated. The density for each condition can be determined through Equation 2.10 using mass for each condition (Arif, 2016).

The unit weight of a rock, γ, is its specific weight (FL^{-3}) and for the unit it can be pounds per cubic foot (lb/ft^3) or kilonewtons per cubic meter (kN/m^3) (Goodman, 1989). Unit weight (γ) is the weight of rock per unit volume (see Equation 2.11)

$$\gamma = \frac{W}{V} \tag{2.11}$$

where

γ = unit weight (kN/m^3);

W = weight of rock (kN);

V = volume of rock (m^3).

2.3.2 Mechanical properties of rock

The mechanical properties of rocks are concerned with the response of rock to forces. The mechanical structure of rocks is determined by their structural properties such as mineral composition, mineral grain arrangement, and any cracks caused by diagenesis or tectonic forces during their long geological history (Jaeger, 2007). In geomechanics, rock deformation and failure or strength behavior are primary concerns. In most situations, rock mechanical properties in the laboratory are assessed by measuring rock deformation (strain) under the action of a given force on cylindrical samples (Schön, 2015).

The direct shear strength test is one of several types of rock mechanical properties tests. When the stress is so minimal or may be ignored at such a shallow depth, the failure is usually caused by sliding on the discontinuity plane or the rock's shear strength. The cohesion and internal friction angle parameters would be determined by this test and it would be obtained through the Mohr-Coulomb criterion graph (see Figure 2.10). These parameters are critical for designing a stable slope, and the Mohr-Coulomb criterion, which is written on Equations 2.12, is the most commonly used shear failure criterion (Arif, 2016).

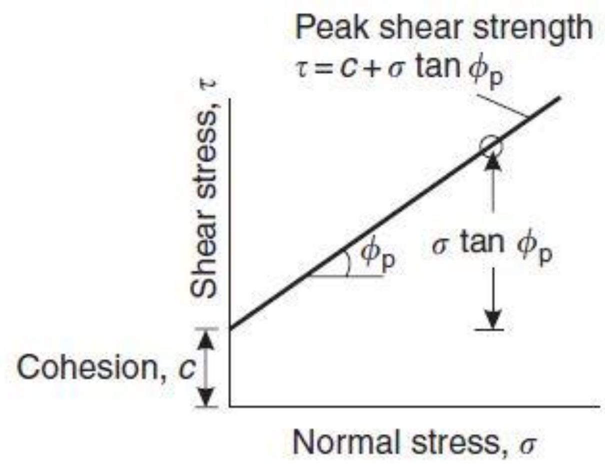

Figure 2.10 Mohr-Coulomb criterion graph of shear strength (Wyllie and Mah, 2005)

$$\tau = c + \sigma_n(\tan \phi) \tag{2.12}$$

where

τ = shear strength (kPa);

c = cohesion (kPa);

σ_n = normal stress (kPa);

ϕ = internal friction angle (°).

2.4 Kinematics Analysis

Kinematic analysis is the process of examining the direction in which a block will slide and providing an indicator of stability conditions after the type of block failure has been determined on the stereonet. While the stereonet analysis provides a good indication of stability conditions, it does not take into consideration external factors such as water pressures or reinforcement consisting of tensioned rock bolts, which can have a substantial impact on stability. Kinematic analysis is typically used in the design process to identify possibly unstable blocks (Wyllie and Mah, 2005).

A geologically induced break in the continuity of a body of rock along which no visible displacement has occurred. A set is a collection of parallel joints, and joint sets intersect to form a joint system. Joints can be open, filled, or healed. Bedding joints, foliation joints, and cleavage joints are all terms for joints that form parallel to bedding planes, foliation, and cleavage (ISRM, 1978).

The potential of unstable conditions or excessive deformations developing is largely controlled by the orientation of discontinuities relative to an engineering structure. The morphology of the individual blocks, beds, or mosaics that form up the rock mass is determined by the mutual orientation of discontinuities (ISRM, 1978). The typical tool for mapping is the geological compass, which is used to read directly in terms of dip and dip direction (Hoek and Bray, 1981).

Structural mapping methods are systematically examining all significant geological features are "line" and "window" mapping. Stretching a tape along the face and mapping any discontinuity that crosses the line, line lengths are typically between 50 and 100 meters. The tape is known as a "scan line". The positions of all discontinuities can be identified if the line's endpoints are surveyed. While window mapping necessitates mapping all discontinuities within a fixed-size representative segment or "window" spaced at regular intervals alongside the exposure (Wyllie and Mah, 2005).

2.4.1 Stereographic analysis

Considering structural geological data commonly occurs in three dimensions with natural scatter, it is important to have an analytical technique that can address these issues in order to use the data in design. The stereographic projection has been found to be an effective tool for this purpose. The stereographic projection provides for the representation and analysis of three-dimensional orientation data in two dimensions. The stereographic projection is made up of a reference sphere with a horizontal equatorial plane and a fixed orientation relative to the north (Figure 2.11). The intersection with the reference sphere is rotated down to a horizontal surface at the base of the sphere in order to generate a stereographic projection of a plane or line (Figure 2.12). In slope stability analysis using stereonet, planes are used to represent both discontinuities and slope faces (Wyllie and Mah, 2005).

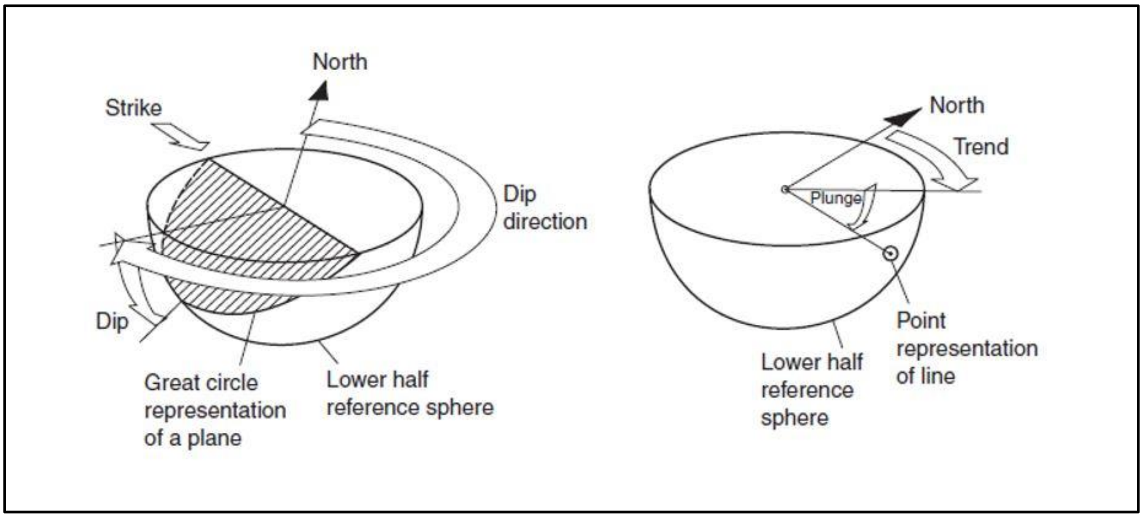

Figure 2.11 Stereographic representation of plane and line on lower hemisphere of the reference sphere (Wyllie and Mah, 2005)

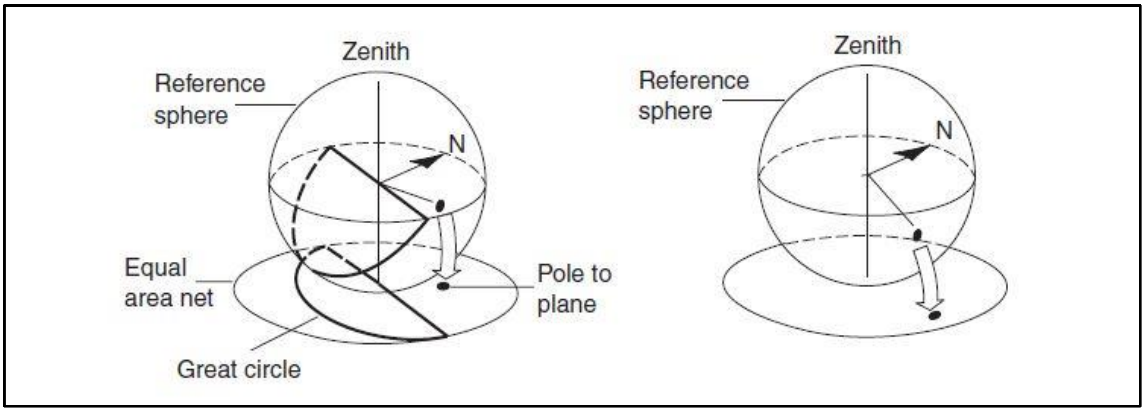

Figure 2.12 Equal area projections of plane and line (Wyllie and Mah, 2005)

The most convenient way to examine the orientation of a large number of discontinuities is to use pole plots, in which each plane is represented by a single point. Pole plots can be generated by hand plotting (manual) or by stereographic computer software. Natural discontinuities have a certain amount of variation in their orientations, which causes the pole plots to scatter. It can be difficult to distinguish between the poles from different sets and to determine the most likely orientation of each set if the plot contains poles from several discontinuity sets. The most densely concentrated areas of poles can be more easily spotted (see Figure 2.13). The contour shows the concentrations of poles that represent the orientations of sets of discontinuities right away (Wyllie and Mah, 2005).

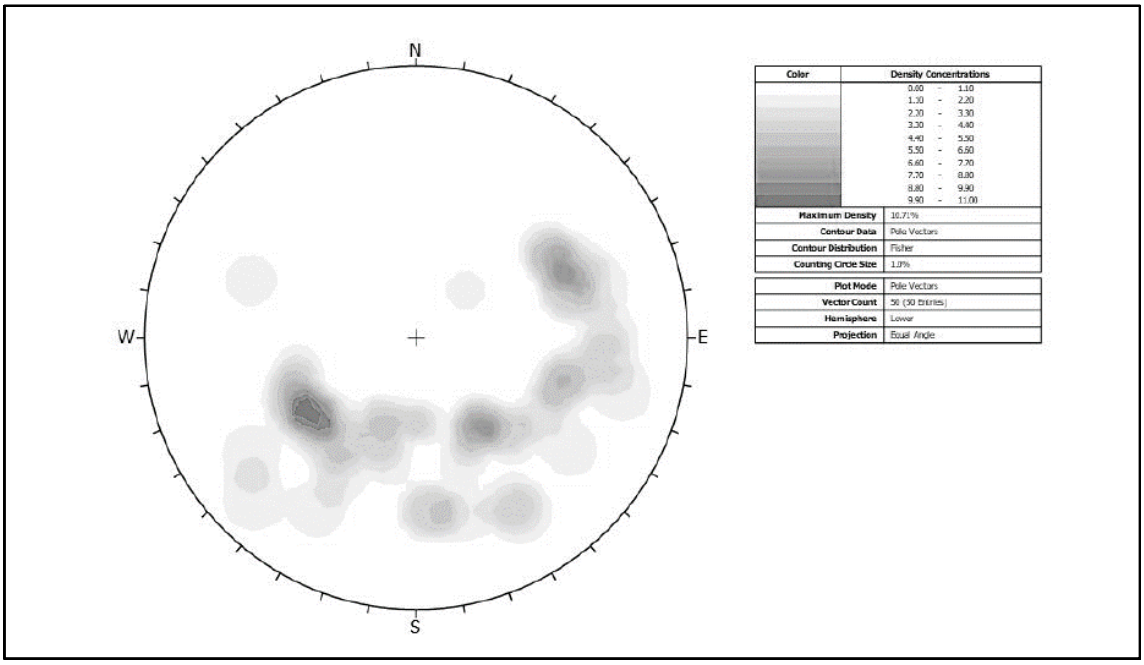

Figure 2.13 Contoured data or pole density

After determining the orientation of the discontinuity sets on the pole plots, as well as critical single discontinuities such as faults, the next stage in the analysis is to see if these discontinuities form potentially unstable blocks in the slope face. Plotting great circles of each of the discontinuity set orientations, as well as the face orientation, is used to do this analysis. The primary objective of plotting great circles of discontinuity sets on a slope is to figure out the geometry of blocks created by intersecting discontinuities as well as the direction in which they might slide. Last but not least, in order to select the potential failure mode, the analysis results must be in line with the general condition of slope instability modes (Wyllie and Mah, 2005).

2.4.2 3D kinematic data acquisition and processing

Mapping visible structural features on outcrops or excavated faces is a time-consuming operation (Hoek and Bray, 1981). There are circumstances when engineers are unable to directly reach a rock face for mapping purposes, such as when there is a risk of rock falls or the face is overhanging in some areas. In these circumstances, terrestrial photography (photogrammetry) can be used to perform the indirect geological mapping. The basic approach entails calculating the orientation of each surface by acquiring the coordinates of at least three points on it and the set up can be seen on Figure 2.14 (Wyllie and Mah, 2005; Hoek and Bray, 1981).

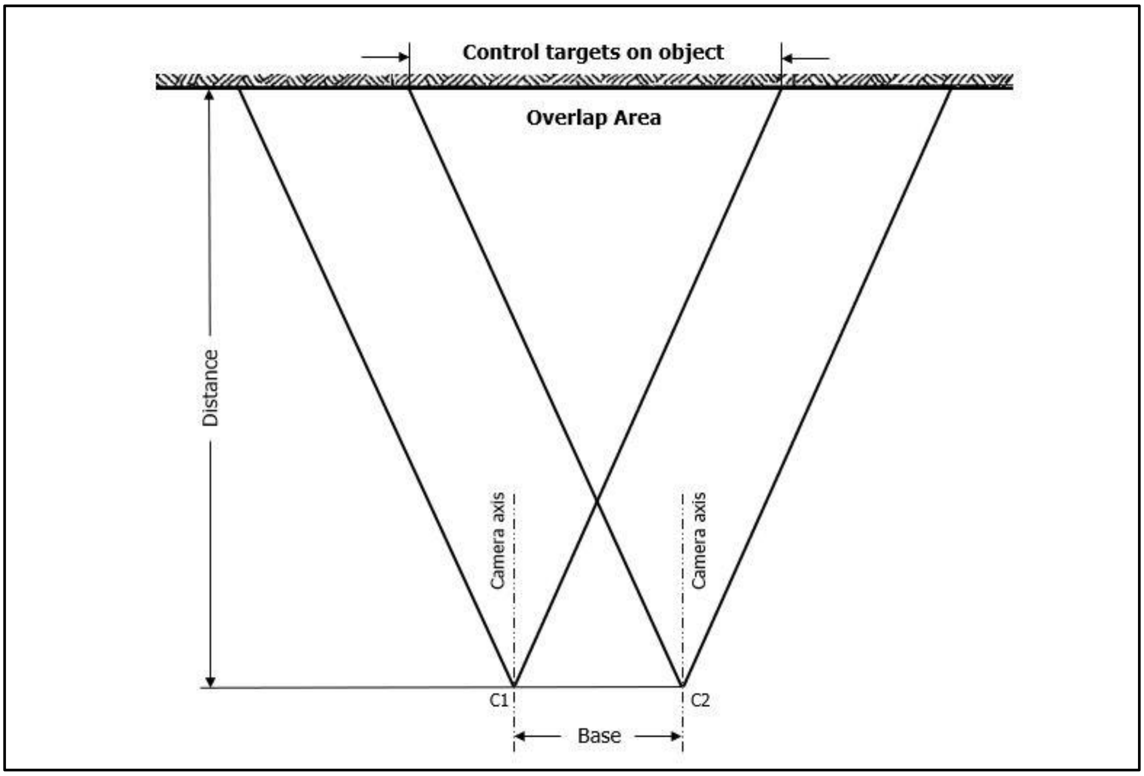

Figure 2.14 Field set up to obtain an overlapping stereopair imagery (Hoek and Bray, 1981)

The development of a new automated technique was made possible by recent advancements in the use of Digital Outcrop Models (DOMs). ROck Slope Kinematic Analysis (ROKA) is an open-source program that performs kinematic analysis utilizing discontinuity measures collected on a 3D DOM. The proposed approach can identify the possible crucial combination of the discovered discontinuities and the slope's orientation as shown in Figure 2.15. Using this method, the algorithm can detect crucial combinations on the slope based on traditional kinematic analysis of planar failure, flexural toppling, wedge failure, and direct toppling modes of failure, and visualize them on DOMs (Menegoni *et al.*, 2021).

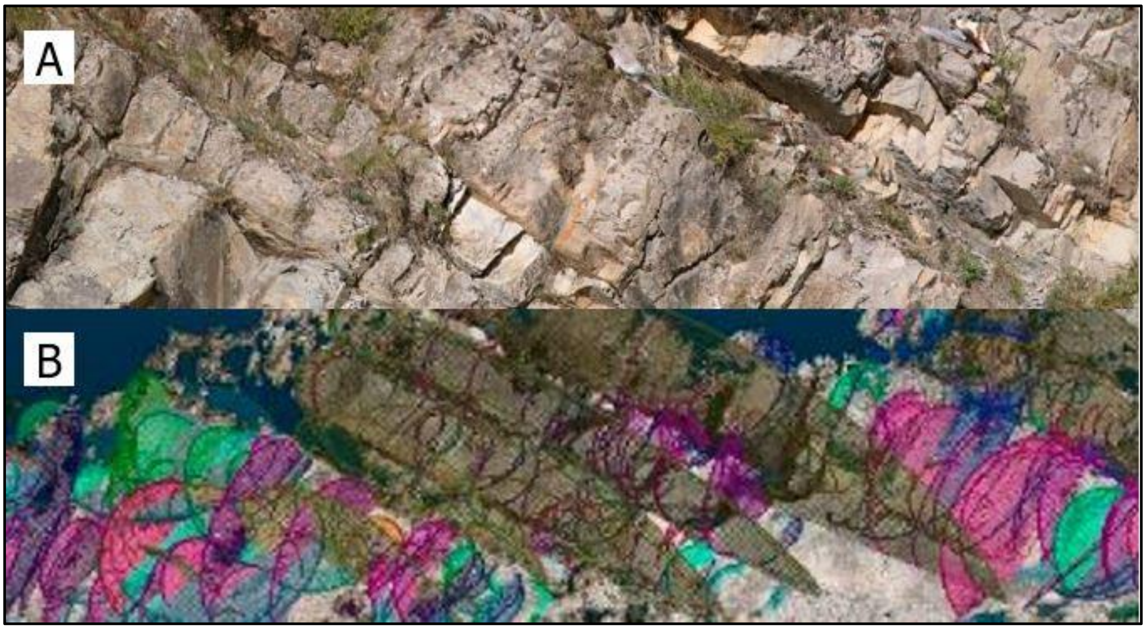

Figure 2.15 Illustration of 3D kinematic analysis using ROKA (Menegoni *et al.*, 2021)

ROKA was created based on three fundamental assumptions. The first and most important assumption is that the discontinuity surface is flat and circular, in accordance with Baecher's disc model. This assumption is commonly used in material mechanics, and it treats discontinuities as discs. As a result, a discontinuity may be characterized using the coordinates of the disc center (x,y,z), the disc radius (r), and the orientation of the 3D plane in which the disc is located (see Figure 2.16). The disc's orientation can be determined by its attitude (e.g., dip/dip direction) or by its normal orientation (N) (Menegoni *et al.*, 2021).

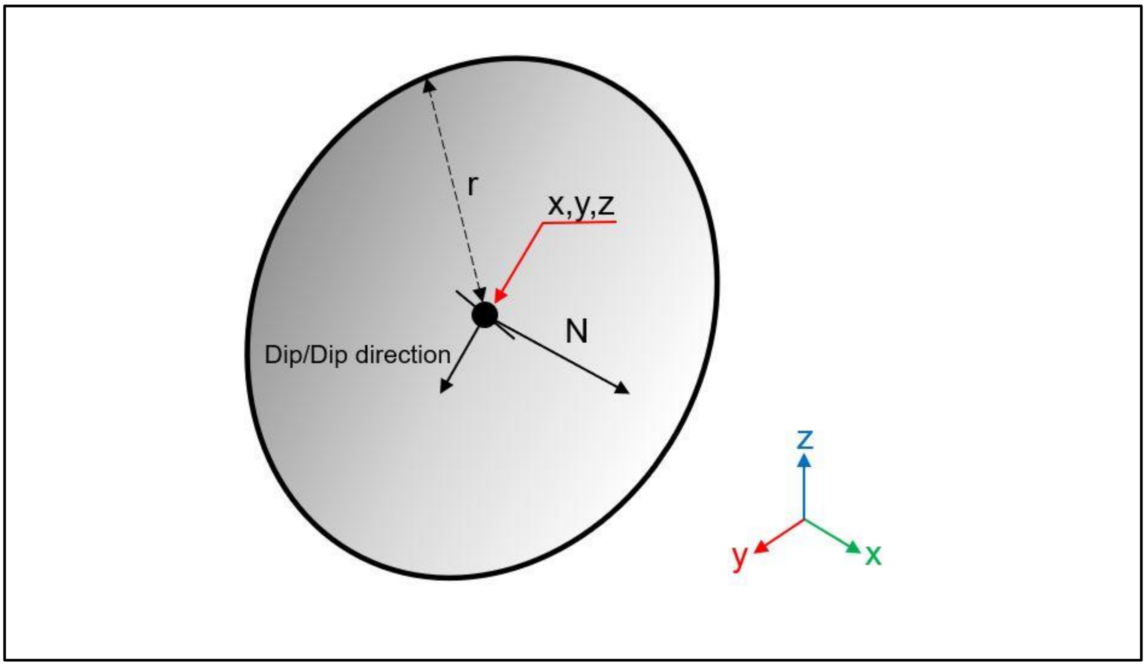

Figure 2.16 Baecher's disc theory

The relationship between N, its components Nx, Ny, and Nz (see Figure 2.17), and the dip or dip direction can be defined by calculating the direction cosines (Menegoni *et al.,* 2021):

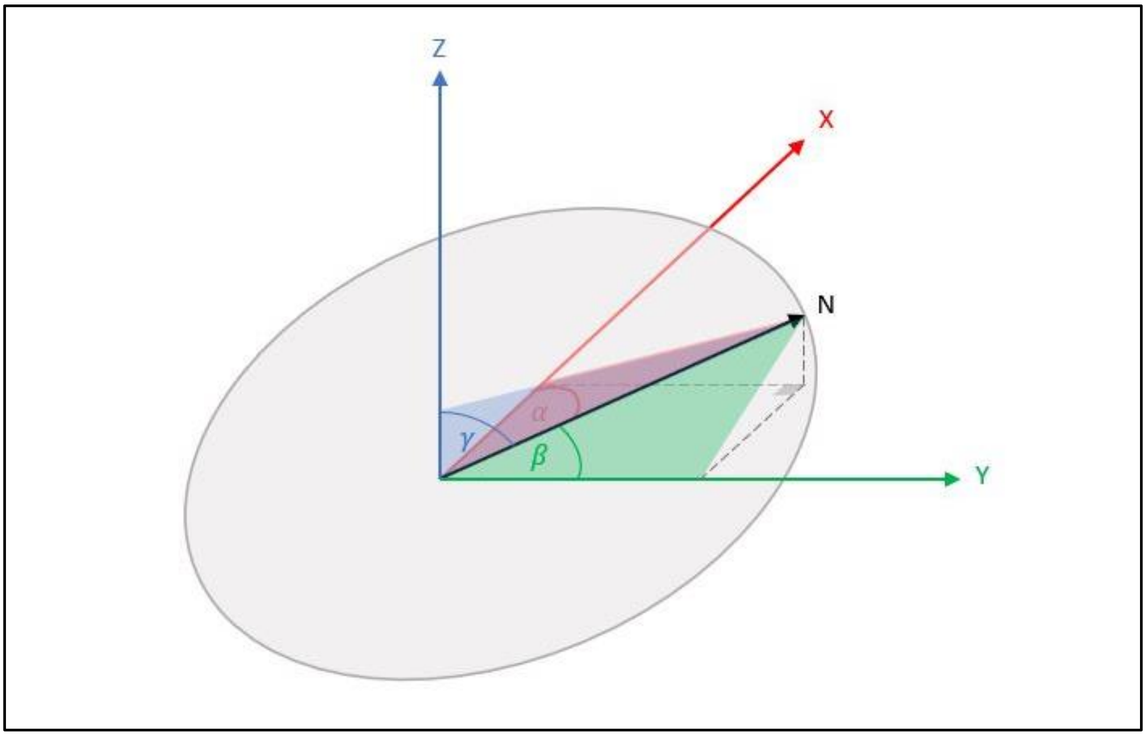

Figure 2.17 Normal orientation and its components

$$\cos\alpha = \frac{N_x}{\overline{N}}; \cos\beta = \frac{N_y}{\overline{N}}; \cos\gamma = \frac{N_z}{\overline{N}} \tag{2.13}$$

The dip angle can be defined as:

$$dip = 90° + \arcsin(\cos\gamma) \tag{2.14}$$

whereas the dip direction can be defined

a. if $\cos\alpha > 0$ and $\cos\beta > 0$:

$$dip\ direction = \arctan\left(\frac{\cos\alpha}{\cos\beta}\right) \tag{2.15}$$

b. if $\cos\alpha > 0$ and $\cos\beta < 0$ or $\cos\alpha < 0$ and $\cos\beta < 0$:

$$dip\ direction = 180° + \arctan\left(\frac{\cos\alpha}{\cos\beta}\right) \tag{2.16}$$

c. if $\cos\alpha < 0$ and $\cos\beta > 0$:

$$dip\ direction = 360° + \arctan\left(\frac{\cos\alpha}{\cos\beta}\right) \tag{2.17}$$

The second assumption is that all of the discontinuities analyzed are just those that are visible and mapped onto the rock slope and that their surfaces are assumed to be circular with a diameter equal to their maximum visible extension. The third assumption is that discontinuities or their intersections are potentially unstable if they cross the slope surface and fulfill the geometric relationships with the slope's local orientation. As the result, the position, dimension, and orientation of the discovered discontinuities on the DOM were exported into an XLSX file. Then, they were plotted onto a stereographic diagram to identify the main sets and perform the kinematic analysis (Menegoni *et al.*, 2021).

2.5 Limit Equilibrium Method

For decades, limit equilibrium is a kind of analysis that has been utilized to analyze stability in geotechnical engineering, and the principles have been widely applied to the stability study of earth slopes. The concept of discretizing a possible sliding mass into vertical slices (Krahn, 2003). Limit equilibrium methods begin by defining a proposed slip surface to be evaluated in order to determine the factor of safety, which is defined as the ratio of available resisting moments to driving moments along the surface (Albataineh, 2006). This method only uses equilibrium static conditions and ignores the stress-strain relations at the slope. Another assumption is that the geometry and modes of the failure plane need to be determined first (Arif, 2016).

Several solution techniques for the slice method have been developed and are widely used. The fundamental difference between these methods is the consideration and satisfaction of static equations, the inclusion of interslice normal and shear forces, and the anticipated relationship between the interslice forces. The criteria for some of the most common approaches are summarized in Table 2.1. This table shows which equations of equilibrium are satisfied, whether the interslice normal is defined, whether the interslice shear is considered, and what the relationship between the interslice normal and shear forces is supposed to be (Krahn, 2003).

Table 2.1 Statics satisfied and interslice forces in various methods (Krahn, 2003)

Method	Moment equilibrium	Horizontal force equilibrium	Interslice normal (E)	Interslice shear (X)	Inclination of X/E resultant
Bishop's simplified	Yes	No	Yes	No	Horizontal
Janbu's simplified	No	Yes	Yes	No	Horizontal
Spencer	Yes	Yes	Yes	Yes	Constant
Morgenstern-Price	Yes	Yes	Yes	Yes	Variable
General Limit Equilibrium (GLE)	Yes	Yes	Yes	Yes	Constant (horizontal)

A general limit equilibrium (GLE) formulation was developed by Fredlund. The GLE formula is based on two safety equation components and provides for a range of interslice shear-normal force conditions. The approaches in the GLE formulation are not limited by the shape of the slip surface. Although the Bishop technique was designed for circular slip surfaces, the same assumptions can be applied to noncircular slip surfaces. The GLE formulation can be used to analyze any kinematically admissible slip surface shape using any of the methods stated in Table 2.1 (Krahn, 2003)

Nowadays, there is a greater demand and need to analyze a slope in 3 dimensions. This is because 2D analysis assumes that the slope's width is infinite, hence the 3D effect is ignored. The width to height ratio is usually insufficient and fluctuates perpendicular to the sliding action. As a result, 3D analysis is thought to be necessary in order to obtain a representative factor of safety (Azizi *et al.*, 2020).

The 3-dimensional model refines the 2-dimensional model by projecting the skid plane into a column and calculating the force and moment from the x, y, and z directions. The mass potential of the slip plane is split into several columns for the 3-dimensional analysis. The column mass might be thought of as a rigid body on the verge of failure (Azizi *et al.*, 2020). There are three possibilities regarding sliding direction (Cheng and Yip, 2007):

1. columns are sliding in the same direction;
2. columns are sliding toward each other; and
3. columns are sliding away from each other.

Considering such limitations, the assumption of a unique sliding direction may be a suitable formulation for analyzing the ultimate limit state, and the current formulation is based on this assumption (Cheng and Yip, 2007).

At the ultimate equilibrium condition, the internal and external forces acting on each column are shown in Figure 2.18. Where a_i is space sliding angle for sliding direction with respect to the direction of slide projected to x–y plane; a_x and a_y is base inclination along x and y directions measure at center of each column (shown at the edge of column for clarity); Ex_i and Ey_i intercolumn normal forces in x and y directions, respectively; Hx_i and Hy_i lateral intercolumn shear forces in x and y directions, respectively; N_i' and U_i is effective normal force and base pore water force, respectively; P_{vi} and S_i is vertical external force and base mobilized shear force, respectively; and Xx_i and Xy_i is vertical intercolumn shear force in plane perpendicular to x and y directions. Weight of material and vertical load are assumed to act at the center of each column for simplicity (Cheng and Yip, 2007).

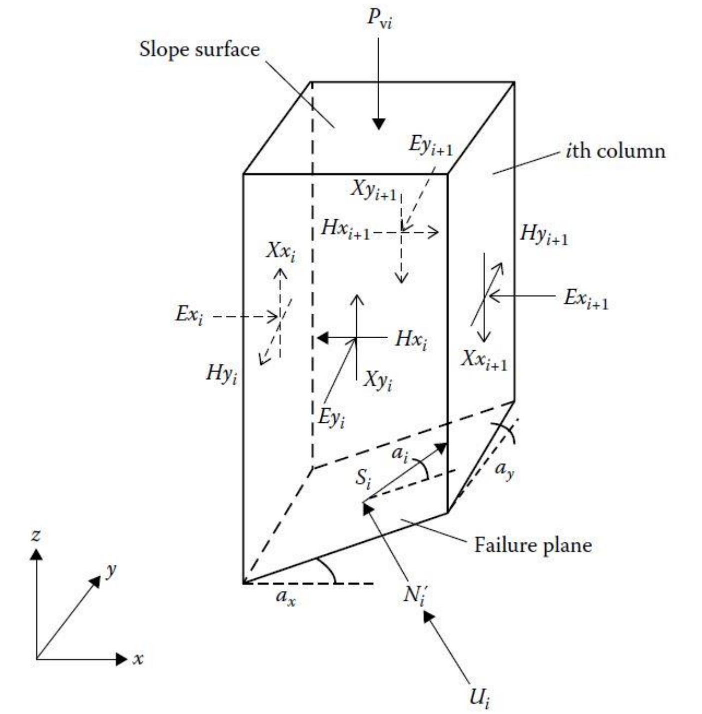

Figure 2.18 External and internal forces acting on soil or rock column (Cheng and Yip, 2007)

The present 3D formulation requires the following assumptions (Cheng and Yip, 2007):

1. Mohr–Coulomb failure criterion is valid;
2. for Morgenstern–Price's method, the factor of safety is calculated based on the sliding direction a', where the force and moment are equal; and
3. sliding direction is the same for all columns.

The global factor of safety, F, is determined as follows using the Mohr–Coulomb criteria:

$$F = \frac{S_{fi}}{S_i} = \frac{C_i + N_i' \tan \phi_i'}{S_i} \tag{2.18}$$

where S_{fi} is ultimate resultant shear force available at the base of column i; N_i' is effective base normal force; $C_i = c'A_i$ where c' and A_i effective cohesive strength and the base area of the column, respectively. The base shear force S_i and base normal force N_i are expressed as the components of forces with respect to x, y, and z directions for column i (Cheng and Yip, 2007).

2.6 Volumetric Estimation of Slope Failure Material

The volume of a landslide is the amount of rock or soil masses that are displaced and accumulate into heaps as a result of the stress release process to rebalance due to disturbances induced by excavation or other external forces (Azizi *et al.*, 2019). As a result, 3D analysis is considered to be necessary in order to obtain a representative factor of safety. Furthermore, 3D analysis can estimate the volume of failure, whereas 2D analysis cannot. If the volume is known, it can be used as one of the factors in making recommendations for failure prevention (Azizi *et al.*, 2020).

For 3-dimensional analysis, failure prediction volume calculations are based on the total volume of each column that slides or indicates the unstable condition, so that from the volume of each column comes to the total volume, which is multiplied by the unit weight of the material's contents to get the total weight of the failure (Azizi *et al.*, 2019).

$$Wf = Vf \times \gamma \tag{2.19}$$

where

Wf = total weight of the failure (kN);

Vf = volume of each unstable column (m^3);

γ = unit weight of material (kN/m^3).

2.7 Shear Strength Reduction Method

Slope stability analysis utilizing the shear strength reduction (SSR) technique is now widely accepted due to the increased computational power and resource availability to geotechnical engineers. The factor of safety (FOS) is defined as the ratio of the actual shear strength of the material to the minimum shear strength required to prevent failure (Maji, 2017). SSR is similar to the limit equilibrium approach in that it considers the shear strength ratio of a material to the driving force. (Azizi *et al.*, 2020).

The principle of the SSR technique is to reduce shear strength gradually until failure conditions are reached. This method is defined through the following equations (Lu *et al.*, 2013).

$$c_f = \frac{c}{SRF} \tag{2.20}$$

$$\phi_f = \arctan \frac{\tan \phi}{SRF} \tag{2.21}$$

where, c and ϕ are the cohesion and angle of internal friction for the Mohr-Coulomb parameters. The c_f and ϕ_f, the factored shear strength parameters, and SRF is the shear strength reduction factor. The SRF is gradually increased in order to reach the state of limiting equilibrium. This indicates that the shear strength deteriorates to the point that a stable condition for a slope is no longer possible (Lu *et al.*, 2013).