



APPLICATION OF DISCRETE FRACTURE NETWORKS (DFN) TO THE DESIGN OF OPEN PIT BENCHES IN ROCK SLOPES

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A dissertation submitted to the Faculty of Engineering and the Built Environment, University of the Witwatersrand, Johannesburg, in fulfillment of the requirements for the degree of Master of Science in Engineering.

Johannesburg, 2021



DECLARATION

I declare that this dissertation is my own, unaided work except where I have explicitly indicated otherwise. I have followed the required conventions in referencing the thoughts and ideas of others. It is being submitted for the Degree of Master of Science in Engineering to the University of the Witwatersrand, Johannesburg and has not been submitted before for any degree or examination to any other University.

A handwritten signature in black ink, appearing to read "Valencia Kuppusamy", is written over a horizontal line.

Valencia Kuppusamy

11th day of October 2021

ABSTRACT

The commonly used phrase, "*we are only as strong as our weakest links*" adequately describes the role of rock mass structures in the stability of open pit benches. The modelling of these structures has always been challenging due to the high degree of variability and uncertainty. Therefore, methods that would more realistically represent these structures would produce results that are more readily observed in nature. This is evident with bench design methods, as the traditional methods involving stereographic kinematic analysis and limit equilibrium analysis are based on conservative assumptions that can be regarded as unrealistic. Although these traditional methods are valuable, quick, and easy to use, the economic impact of their conservatism on bench design is significant enough to warrant the need for a more realistic approach. Therefore, this research involved the application of the discrete fracture networks to rock slopes at bench scale. Discrete fracture network modelling is a methodology used to generate three-dimensional stochastic representations of rock mass structures from statistical distributions that describe their characteristics (e.g., orientation, spacing, size, aperture).

A case study Mine X was selected to consider the implications of the traditional bench design methods and DFN approach. FracMan was used to generate the bench-scale DFN model, and SWedge and DIPS was used to complete the traditional methods. The DFN model generated from sampled field data (i.e., 1D/2D discontinuity data collected from variably oriented rock exposures and boreholes) share the sample statistics and allow for explicit modelling of an individual fracture or simplified fracture sets (Elmo, et al., 2015). The results of this study suggest that the DFN approach using FracMan produces more optimal slope designs as compared to the traditional deterministic and pseudo-probabilistic methods. The traditional method assumes continuous structures and predicts the maximum sized wedge formed from the combination of two joint sets. However, the FracMan rock wedge algorithm results in a higher accuracy of block shape and volume, which is considered more appropriate for the kinematic assessment of block stability.

The DFN approach represents rock mass structures better and can produce more optimal bench designs however it is important to note that the full potential of a DFN model to capture stochastic variability is only achieved if the generated DFN geometry is statistically the same or at least similar (within some tolerance) to the actual fracture network it is meant to simulate. If this is not the case, then this would perhaps be the most inefficient and overly complicated alternative approach to the traditional bench design approach (Poropat & Elmouttie, 2011).



ACKNOWLEDGEMENTS

I would like to express my sincere gratitude to my supervisor, Professor Thomas Stacey for his valued guidance, continuous understanding, patience, and encouragement. Our discussions have always been interesting and your sustained interest and contributions to the rock mechanics community are both appreciated and inspiring to many generations.

A special thanks to my lovely friends, family and Golder colleagues for their ongoing support and motivation. Appreciation is expressed to Rocscience for providing student licence for their suite of software required for the completion of this project.

Lastly, I would like to acknowledge, Golder Associates for providing access to various software packages, and their client for permitting the use of their data for this research project.



Dedicated to my dearest parents, Rosey and Sagren, and my phenomenal sisters, Yogeshni and Livania, for their unconditional love, continuous support, and unwavering belief in me.



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LIST OF SYMBOLS / ABBREVIATIONS

DFN	Discrete Fracture Network
FOS	Factor of Safety
POF	Probability of Failure
RMR	Rock Mass Rating
RQD	Rock Quality Designation
ISRM	International Society for Rock Mechanics
GSI	Geological Strength Index
LE	Limit Equilibrium
ϕ	Friction Angle
P₁₀	Linear Fracture Intensity (#fractures/m)
P₃₂	Volumetric Fracture Intensity (m ² /m ³)
PMC	Palabora Mining Company
BFA	Bench Face Angle
IRA	Inter-ramp Angle
SBW	Spill berm width
BW	Bench Width
BH	Bench Height
MP	Massive Pyroxenite
γ	Unit Weight (kN/m ³)
PR	Persistence Ratio
pdf	Probability Density Function
PIC	Phalaborwa Igneous Complex
MP	Massive Pyroxenite

CHAPTER 1

1.0 INTRODUCTION

The global mining market has become increasingly competitive and as a result design engineers are constantly striving to reduce the overall mining costs. In the open pit mining industry, this translates to an excavation of the steepest possible slope angles that would achieve the lowest stripping ratio while ensuring maximum ore recovery. Therefore, the geotechnical open pit slope design needs to balance the economics with the management of the safety risks; and this design begins at the operational bench level. Bench-scale failures can often lead to:

- Injury or loss of life;
- Production losses and delays;
- Extra stripping costs for recovery and handling of failure material;
- Re-routing haul roads; and
- Machinery repair and replacement costs.

All of which can significantly affect the economic performance of a mine and directly impact the health and safety of the mine personnel and the environment. Hence, the need for better design tools, methodologies and practices that produce more optimal bench design and performance.

Benches are the building block of any open pit slope design and at this scale, the failure mechanisms are typically structurally controlled. Over the years with the advancement in technology, there have been many improvements to the structural data collection methods and instrumentation, which have subsequently improved safety aspects; increased the data collection speed, accuracy, quality, and availability of data relating to rock mass structures. Methods such as photogrammetry, acoustic and optical borehole viewers, and high-resolution geophysics, are widely used in the mining industry (Rogers, et al., 2006). However, incorporating this structural data into the methodology commonly used for the design of rock slopes still remains a challenge for many practitioners. This is particularly evident with the design of benches in hard rock mining environments. The kinematic analysis carried out in the conventional bench design approach includes many assumptions regarding the properties of these structures which may not be realistic. The discrete fracture network (DFN) approach offers an alternative that attempts to address some of these unrealistic assumptions and better utilize the structural data available today.

The modelling of these structures has always been challenging due to the high degree of variability and uncertainty. According to (Elmouttie & Poropat, 2015), rock mass modelling requires 3 measurement and analysis procedures that can address 'total uncertainty', which is a combination of stochastic variability and inherent uncertainty. An example of the stochastic variability would be the distribution of adequately measured orientations of structures, which is generally addressed by assigning statistical representations of the variable. The inherent variability would be those parameters that are not as well

defined, such as discontinuity shape, which often requires the engineer to determine its suitability in the model. The DFN approach has perhaps had the most success with realistically modelling the heterogeneity of the rock mass structures because it allows for the characterization of individual fracture properties such as, position, size, aperture, etc. (Elmouttie & Poropat, 2015).

In light of the importance of benches in open pit mining and rock mass structures being fundamental to their design and performance; this research study involving the ***“Application of discrete fracture networks (DFN) approach to the design of open pit benches in rock slopes”*** was proposed.

1.1 Research Objectives

The primary objectives of this research are:

- To investigate the DFN approach using FracMan and its application to the stability of rock slopes at bench-scale; and,
- To evaluate whether the DFN approach produces more optimal bench designs through more realistic representations of small-scale structures and better predictions of wedge failure volumes;

With these two objectives in mind, this research was aimed at addressing the following related questions:

- Does the DFN approach result in more realistic kinematic analysis?
- Does the application of the DFN method result in improved bench design?
- How does the DFN method compare with the conventional bench design methodology?
- What are the considerations for use of the DFN approach and its application to bench design and rock slope stability?

1.2 Research Methodology

In order to achieve the primary research objectives and address the related questions, the following were required:

- Selection / collection and relevant data processing of all available field mapping data.
- Stereographic analysis of structural data (i.e., interpretation of structural sets and subsequent kinematic analysis).
- Statistical data analysis of structural data available to determine DFN input parameters.
- Development of DFN model using FracMan and validation of DFN model realizations.
- Conventional wedge stability analysis using limit equilibrium software, SWedge (included use of deterministic and probabilistic analysis options available).

-
- FracMan rock wedge stability analysis based on DFN generated.
 - Comparisons of the DFN approach to the conventional bench design approach considering the prediction of wedge volumes.

1.3 Dissertation Outline

This dissertation is presented in eight chapters and five appendices:

- **Chapter 1** is an introductory chapter that presents the problem statement, the research objectives, and an overview of the research methodology.
- **Chapter 2** provides a literature review of the related dissertation content. The chapter begins with the background on rock mass structures and their characteristics; followed by a brief description of the structural data collection methods, structural modelling approaches and associated sampling bias, errors, and uncertainty. The review then concentrates on the DFN modelling approach and then extends to its geotechnical open pit bench design application which forms the basis of this research. This includes an overview of the open pit slope design process, kinematic slope stability analysis and related limit equilibrium analysis with special focus on bench design and wedge stability analyses.
- **Chapter 3** presents the case study. The site description of Mine X is provided together with summaries of the field data collection programme. In addition, an overview of the photogrammetry data processing and subsequent digital mapping using Sirovision is provided.
- **Chapter 4** focuses on the stereographic analyses carried out using the structural data collected from Mine X. The rock mass study area for Mine X is presented here. The subsequent interpretation of the structural data to delineate discontinuity sets that were considered representative of the selected rock mass is summarised. This is then followed by the presentation of results related to the stereographic kinematic assessment carried out for the pit walls at Mine X.
- **Chapter 5** describes the methodology used to create and validate the DFN model. It summarises the DFN model input parameters, model generation and model validation processes used for the Mine X open pit benches within the massive pyroxenite rock masses. The five DFN model realisations for the Mine X open pit benches are presented in this chapter.
- **Chapter 6** provides the results and observations from the limit equilibrium SWedge analysis and FracMan rock wedge stability analysis for Mine X case study and a short comparison of these methods.
- **Chapter 7** presents the bench design considerations in relation to the DFN approach as compared with conventional approach assessed with the aid of SWedge. Bench widths and bench face angles are assessed, and bench design recommendations provided accordingly. The chapter concludes with a brief consideration of the bench design risk.

-
- **Chapter 8** summarises the key findings for each main component of this research study. The research objectives and questions are addressed, and recommendations for future research work are also provided.
 - **Appendix A** provides all stereographic projection plots related to the Mine X case study.
 - **Appendix B** provides all 3D image data processed using Sirovision.
 - **Appendix C** provides a summary of all relevant statistical discontinuity data analysis for Mine X.
 - **Appendix D** provides a summary of the DFN modelling and rock wedge results using FracMan.
 - **Appendix E** provides SWedge deterministic, probabilistic and combination results for various Mine X pit wall bench scenarios.

CHAPTER 2

2.0 LITERATURE REVIEW

This research project involves the use of discrete fracture networks in the design of open pit mine benches in hard rock masses. The related literature presented in this chapter includes: a description of rock mass structures and their characteristics; data collection methods and associated structural modelling approaches; a discussion on the discrete fracture network (DFN) modelling approach; an overview of the open pit design process with specific reference to the slope design of mine benches; a brief summary of the main kinematic failure modes; and lastly a description of the limit equilibrium stability analyses related to wedge type failures.

2.1 Rock Mass Structures

A rock mass is comprised of naturally variable intact rock material and rock mass structures. These rock mass structures are generally referred to as discontinuities. The term “discontinuity” can be defined as any significant mechanical break or fracture of negligible tensile strength in a rock and makes no distinction regarding its origin, age, or geometry (Priest, 1993). The presence of these discontinuities and their characteristics often control the behaviour of hard rock masses. Therefore, an understanding of the methods used to measure, characterise, and analyse these structures is considered fundamental to any rock engineering design or stability evaluation. This section describes the characteristics of discontinuities, summarises the commonly used structural data collection methods and briefly discusses the subsequent structural modelling approaches.

2.1.1 Discontinuity Characteristics

According to (ISRM, 1978), the ten parameters selected to describe discontinuities in rock masses, illustrated in Figure 1, can be defined as follows:

1. **Orientation** that refers to the attitude of the discontinuity in space and is described by the dip direction and the dip of the line of steepest declination on the discontinuity plane;
2. **Spacing** which is the perpendicular distance between adjacent discontinuities (i.e., mean or modal spacing of a discontinuity set);
3. **Persistence** is the discontinuity trace length observed in an exposure and typically gives a rough estimate of the areal extent or penetration length of a discontinuity. Termination types influencing persistence (e.g., termination of a discontinuity in solid rock or against another discontinuity reduces the persistence);
4. **Roughness** refers to the inherent surface roughness and waviness relative to the mean plane of a discontinuity. Roughness and waviness contribute to shear strength and large-scale waviness may result in local dip angle variations;

5. **Wall strength** is the equivalent compression strength of the adjacent rock walls of a discontinuity. It can be lower than the rock strength due to weathering or alteration and is also considered an important component of shear strength if the rock walls are in contact;
6. **Aperture** is the perpendicular distance between adjacent rock walls of a discontinuity in which the intervening space is filled with air or water;
7. **Filling** refers to the material that separates the adjacent rock walls of a discontinuity and is generally weaker than the parent rock (e.g., clay, sand, silt, gouge, mylonite, and thin mineral coatings);
8. **Seepage** considers the water flow and free moisture visible in individual discontinuities or the entire rock mass;
9. **Number of sets** refer to the number of joint sets comprising the joint system. Individual discontinuities can further divide the rock mass; and,
10. **Block size** is the rock block dimensions that result from the mutual orientation of intersecting joint sets and the spacing of individual sets. Individual discontinuities may further influence shape and size of the block.

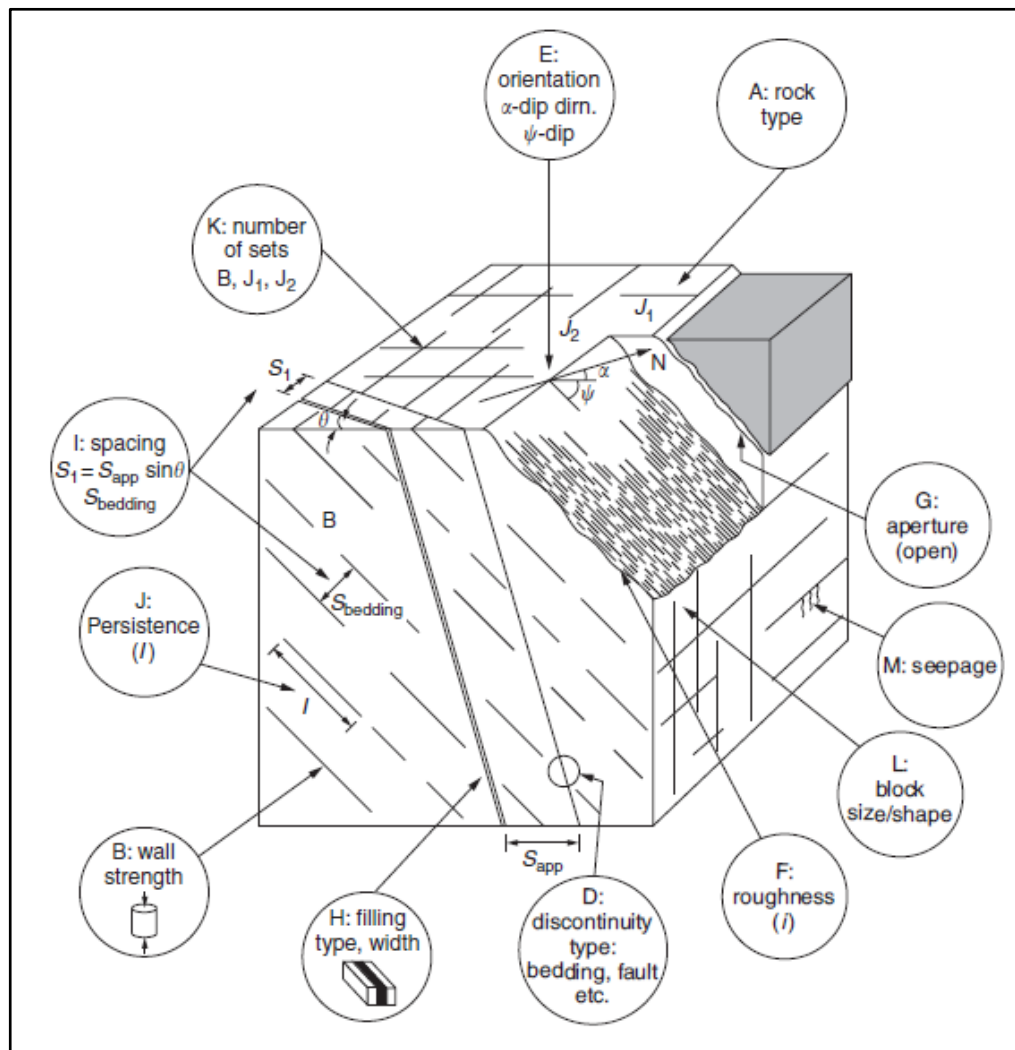


Figure 1: Schematic showing rock mass properties (Wyllie & Mah, 2004).

A discussion on the shear strength models available for rock discontinuities is beyond the scope of this research project, however it is detailed in sources; (ISRM, 1978), (Hoek & Bray, 1981), (Priest, 1993), (Brady & Brown, 2005), (Hoek, 2002), (Wittke, 1990) and (Wyllie & Mah, 2004). This research relates primarily to the geometric characteristics of discontinuities. The following sections describe these types of discontinuities and their geometrical properties: orientation, spacing, intensity, persistence, shape, size, and block size.

2.1.1.1 *Discontinuity Types*

Discontinuity is a collective term used for most structure types, planes of weakness such as bedding and schistosity, weakness zones (e.g., shear zones), and faults (ISRM, 1978). Generally, the natural discontinuities which are of geological and geomorphological origin are often distinguished from artificial discontinuities formed by drilling, blasting and other excavation activities (Priest, 1993). This differentiation between natural and artificial discontinuities, has different uses and implications depending on the industry application (e.g., rock quality designation (RQD) is determined using only natural discontinuities). The type of natural discontinuities often describes the dominant mode of formation and geological history, which have various implications in the exploration industries. Studies have concluded that even a single fracture may have a long complex history of changes in stress, temperature, strain rate, mineralisation and recrystallisation (Priest, 1993). For engineering applications, the following natural discontinuity types are commonly recognized (ISRM, 1978), (Priest, 1993), (Brady & Brown, 2005):

- **Faults** – A fracture or fracture zones along which there has been recognizable displacement, that vary from a few centimetres to a few kilometres. Shear displacement results in striated and/or polished walls. The rock on both sides of the walls is often shattered, altered, or weathered, which results in breccia or gouge infill with thicknesses that vary from millimetres to hundreds of metres. These are important considerations for slope stability as they are either: low shear strength zones or potential release or slip failure surfaces; act as either a conduit for groundwater flow or as an aquitard in which case results in a dangerous build-up of groundwater pressures. Geologically, faults are classified according to their relative displacement (e.g., a fault with general displacement along the strike is classified as a strike-slip fault);
- **Folds** – These are structures in which the orientation of the beds are changed by flexure that results from applied post-depositional tectonic forces. These can occur at larger mining region scales or smaller local scales. Folds are classified based on their geometry and formation method. Folds alter the orientation of beds locally and other associated structures.
- **Shear Zones** – Refer to bands of material in which local shear failure of the rock has occurred. It can be up to several metres thick with fracture surfaces that may be slickensided or coated with low friction material that are produced by the stress relief process or weathering.
- **Dykes** – these are long narrow intrusions of typically fine-grained igneous rocks with steep or vertical and approximately parallel sides. They can be of considerable length, vary in width from

centimetres to metres and can appear as dyke swarms. These structures can form potential seepage paths and zones of low stiffness and shear strength.

- **Bedding planes** – These are highly persistent features that divide sedimentary rocks into beds or strata. These may contain; cross or discordant bedding, partings of different grain size, or be partially healed, which have different shear strength considerations. Preferred orientation of particles results from the depositional process that gives rise to weakness planes parallel to these bedding planes. Bedding is generally horizontal or planar but can be subjected to complex folding and faulting resulting in ranges of orientations.
- **Joints** - A geological break in a body of rock along which there are no visible displacement. A group of parallel or sub-parallel joints are termed joint sets. These joint sets intersect to form a joint system. Joints can be open, filled or healed and are the most common or geotechnically significant discontinuities in rocks. Many studies carried out on the genesis of joints provide insight on the varying mechanisms resulting in specific types of jointing, such as columnar type jointing being formed by stresses induced by the cooling of basalts. According to (Gumede, 2005), jointing is generally a result of rock reaction to the following;
 - Earth stresses responding to the mantle / crustal movements,
 - Rock material or rock mass response to stress changes caused by loading and unloading of the overburden through denudation processes, and,
 - Different expansion and contraction rates within a rock material or rock mass in response to changes in temperature, mineral alterations and variations in moisture content;
- **Veins** – These are mineral infilled joints or fissures, also referred to as cemented joints. These can be tabular, sheet-like, or irregular, infilled with minerals such as quartz or calcite and could be weaker or stronger than the wall rock.
- **Cleavage** – Two broad types of rock cleavage, namely, fracture and flow cleavage have been identified by (Spencer, 1969) and (Whitten & Brooks, 1972). *Fracture cleavage* describes incipient cemented or welded parallel discontinuities that is independent of any parallel mineral alignment and can have six various possible formation mechanisms. It is commonly associated with other structural features such as faults and folds. *Flow cleavage* occurs as slaty cleavage or schistosity and is dependent on recrystallisation and parallel alignment of platy minerals which produces an interleaving or foliation structure. This type is commonly formed by high temperature and/or high-pressure metamorphism of fine-grained rocks and is often visible in slates, phyllites and schists. Although this flow cleavage possesses substantial tensile strength, it can create significant anisotropy in the deformation and strength of these rocks. Cleavage subjective to faulting and folding can result in discontinuities forming along the well-developed planes.

2.1.1.2 *Discontinuity Orientation*

Orientation is one of the most important discontinuity characteristics in most engineering applications. It is a primary consideration in kinematic analysis as it involves assessing the discontinuity orientation relative to the engineering structure or excavation and identifies possible unstable conditions or excessive deformation. The mutual discontinuity orientations determine the shape of the individual blocks that comprise the rock mass (ISRM, 1978).

The orientation of a discontinuity refers to the attitude of the discontinuity in space and is described by the dip direction or azimuth (i.e., measured clockwise from true north) and the dip angle (i.e. dip of the line of maximum declination on the discontinuity plane). Other orientation conventions used include dip angle and strike (i.e., direction of the horizontal line on a planar structure). The orientation of structures is often measured from core and/or rock surface exposures using either one dimensional scanline surveys or two-dimensional window surveys. Further details on the data collection methods are provided in Section 2.1.2.

Orientation data are typically processed using stereographic projection methods which allow representation, and analyses of three-dimensional angular relations of planes and lines, in two-dimensions. Stereographic projection methods were originally a structural geology tool that was extended for use in engineering applications (Brady & Brown, 2005). The principles of these stereographic projection techniques are discussed in (Phillips, 1977) with its application to rock engineering provided in (Priest, 1985), (Hoek & Bray, 1981) and (Goodman, 1976). A brief explanation on the stereographic projection method is provided below with reference to the equal-area, lower hemisphere projection commonly used for engineering purposes.

Stereographic projection consists of a reference sphere in which its equatorial plane is horizontal, and its orientation is fixed relative to north as shown in Figure 2(a). Figure 2(b) shows the planes and lines with a specific trend and plunge projected so that the axis of the feature axis passes through the centre of the reference sphere. A unique line on the surface of the reference hemisphere is defined by the intersection of the feature with the lower half of the reference sphere. This intersection with the reference sphere for a plane is a circular arc referred to as a great circle and for a line, this intersection with the reference sphere is a point. Therefore, to create a stereographic projection of a plane or line, the intersection with the reference sphere is rotated down to a horizontal surface at the base of a sphere [see Figure 2(c) and Figure 2(d)]. These rotated lines and points are unique locations on a stereonet that represent the orientation of a discontinuity. Poles are alternative means of representing the orientation of a plane. It is the point on the surface of the reference sphere that is cut by a radial line in a direction normal to the plane. The pole to a plane allows a single point to represent the orientation of a discontinuity plane. In this way, large number of discontinuities can be easily plotted as poles with their pole concentrations being immediately visible (Wyllie & Mah, 2004).

There are many stereographic computer programs available to process, plot, and contour discontinuity orientation data. Rocscience Dips, one of the more popular commercially available stereographic programs, has been used for this project.

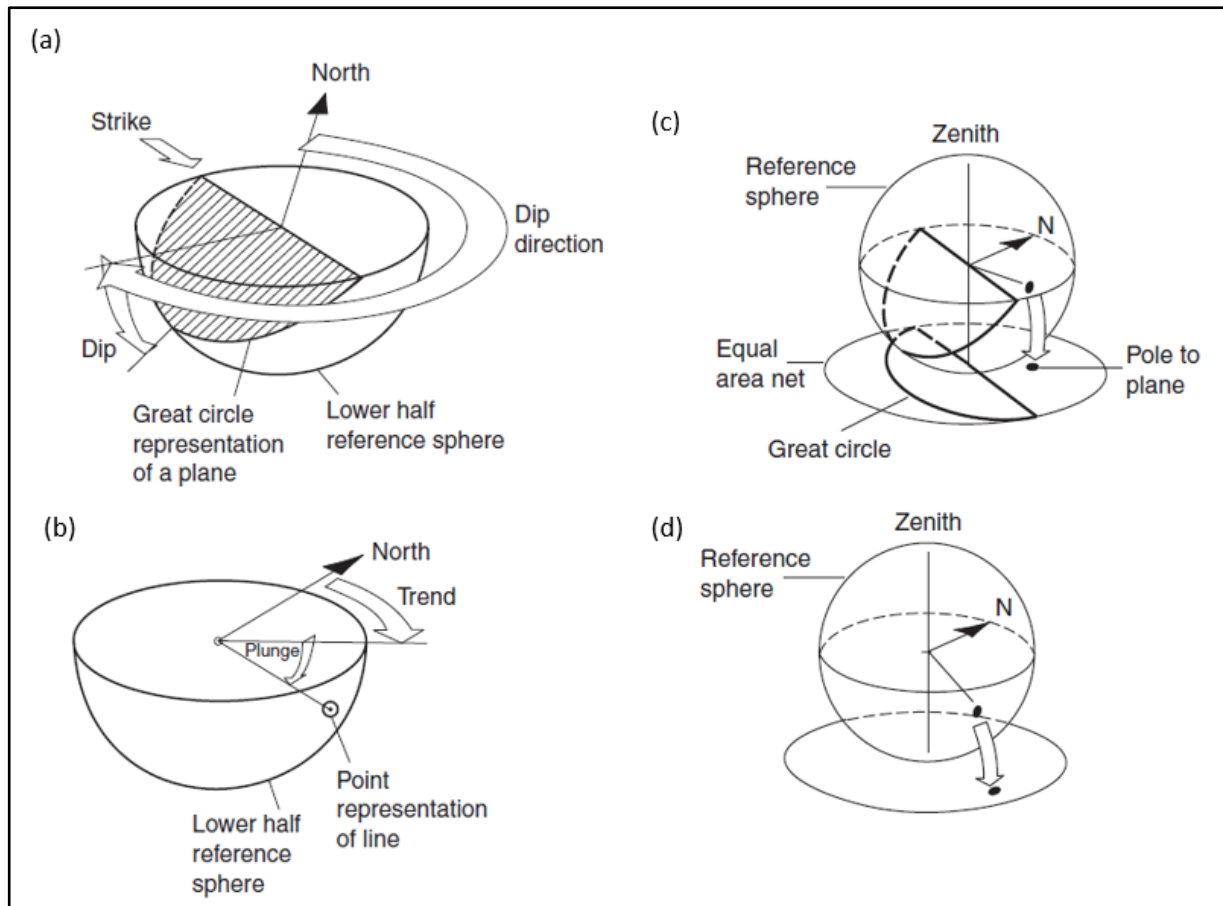


Figure 2: On the left, (a-b) shows a stereographic representation of plane and line on lower hemisphere projection and on the right (c-d) shows an equal area projection of plane and line: (a) plane projected as a great circle; (b) isometric view of line (plunge and trend); (c) plane projected as great circle and corresponding pole; (d) line projected as pole [Modified from (Wyllie & Mah, 2004)].

Contouring of pole densities allows for highly concentrated areas of poles to be more easily identified. It is carried out by counting the number of poles within a sample window area that is 1 % of the area of the projection circle. This allows the maximum pole concentrations to be determined as well as the decreasing concentrations around the major concentrations (Brady & Brown, 2005). Examples of a contour plot and pole plot are shown in Figure 3 and Figure 4 respectively.

The distribution of orientations around the central orientations may be symmetric or asymmetric. There are several probability distributions suggested to represent the discontinuity orientations such as hemispherical uniform, Fisher, bivariate Fisher, Bingham, bivariate normal and bivariate lognormal (Zhang, 2017). The most commonly used is the fisher distribution, in which the symmetric dispersion of orientation data from the mean is represented by a number known as the Fisher constant. The higher the K-value, the less disperse the values are about the mean.

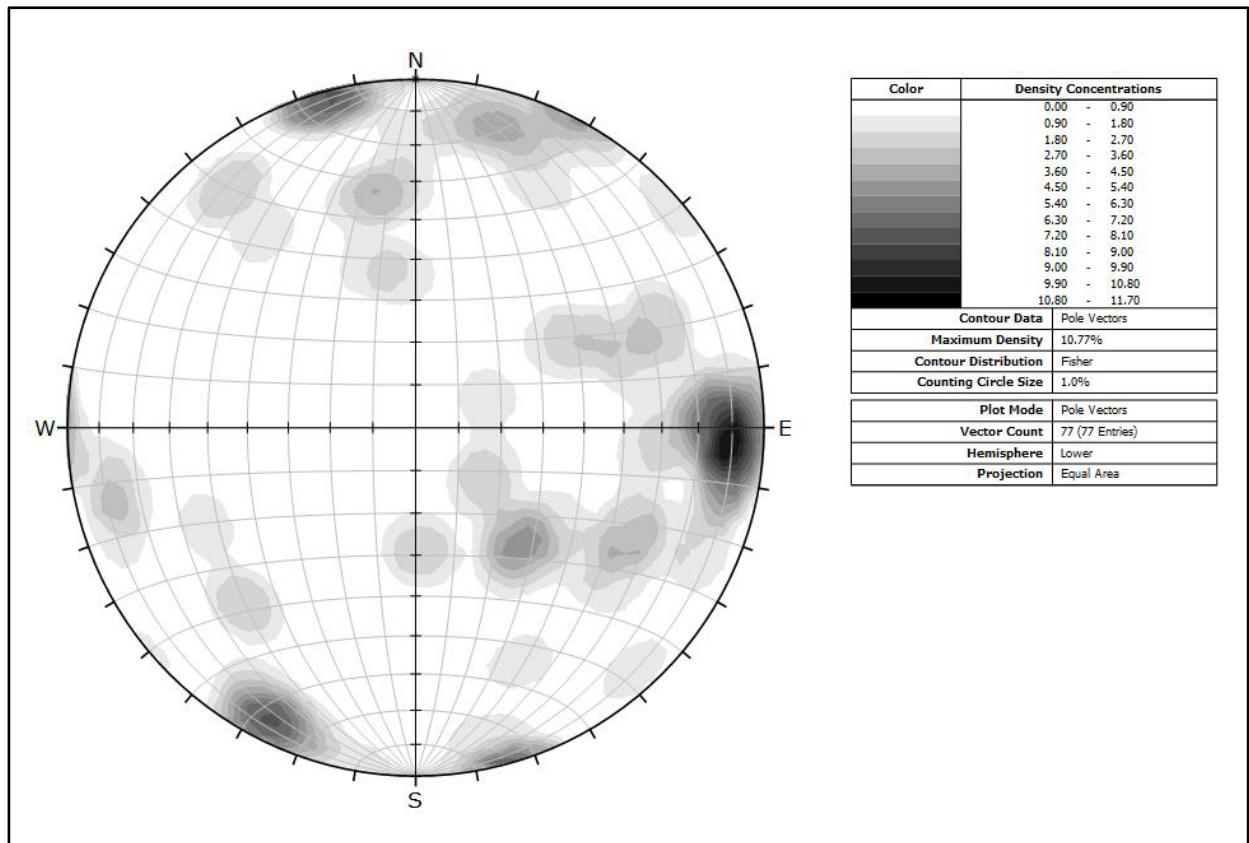


Figure 3: Contours of pole concentrations for a Mine X scanline survey.

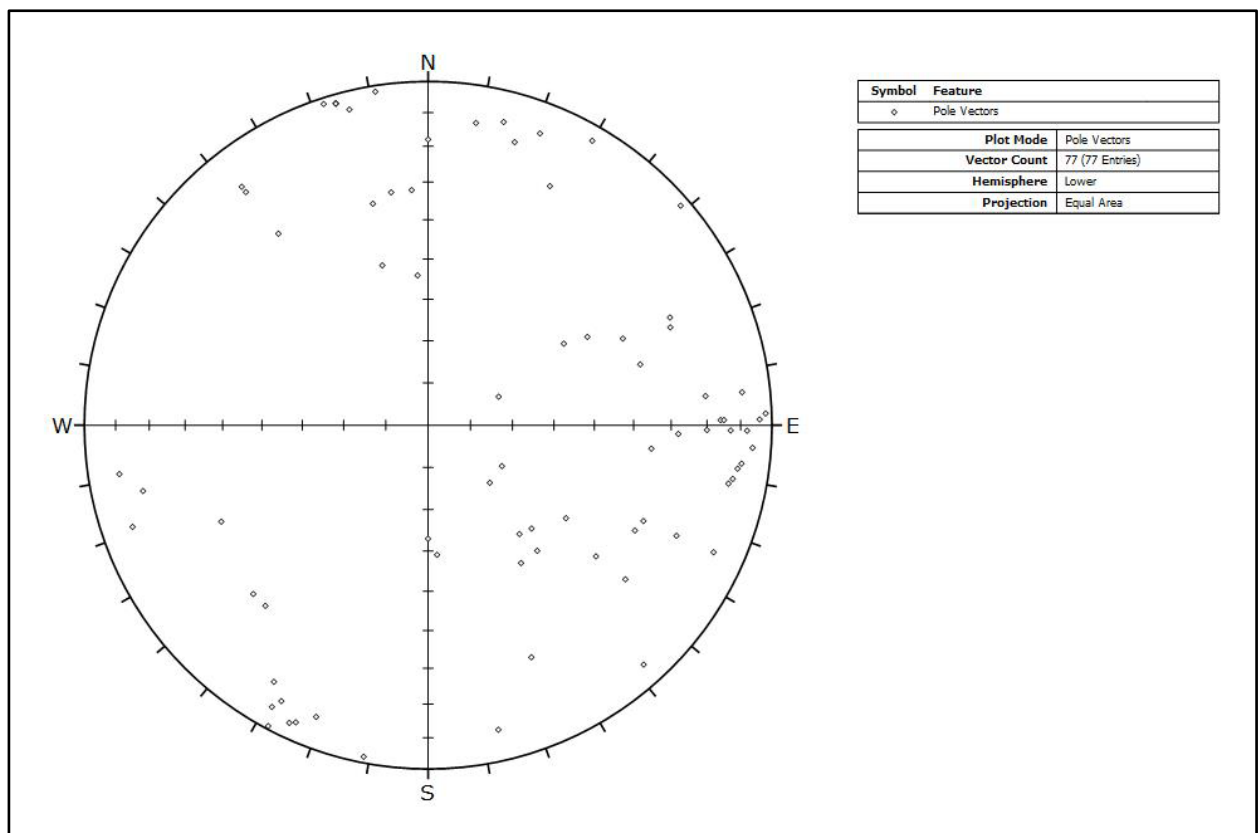


Figure 4: Poles of discontinuities measured at Mine X.

The grouping of poles into discontinuity sets (or clusters) of preferred orientation are often carried out using a combination of contouring and visual inspection with an understanding of the site geological conditions. A number of algorithms based on statistical or fuzzy-set approaches are also available for numerically clustering orientation data, the details of which are described in (Shanley & Mahtab, 1976), (Mahtab & Yegulalp, 1984), (Einstein, et al., 1979), (Harrison, 1992), (Miller, 1983), and (Kulatilake, 1993). Orientation data, like most other characteristics of discontinuity data is subjected to sampling bias. This is discussed further in Section 2.1.2.

2.1.1.3 Discontinuity Intensity

Discontinuity intensity can be expressed in terms of one-dimensional, two-dimensional, and three-dimensional measures which includes discontinuity spacing, linear, areal, and volumetric frequency, rock quality designation (RQD), discontinuity trace length per unit area of rock exposure and discontinuity area per unit volume of rock mass (Zhang, 2017). The expression of intensity used is often influenced by the nature of the exposure and the survey technique. The following subsections provide details on these intensity expressions with general reference made to survey techniques presented in Section 2.1.2.

2.1.1.3.1 Discontinuity Spacing and Linear Frequency

Discontinuity spacing is the perpendicular distance between adjacent discontinuities, as shown previously in Figure 1. Spacing of discontinuities influences the rock mass strength, degree of fragmentation (i.e., block size), permeability, and seepage characteristics and in some extreme cases, the failure mode (e.g., from translational to circular or even flow). It also has an important consideration in various rock mass classification systems, such as the Geomechanics Classification or Rock Mass Rating (RMR) system published by Bieniawski in 1976 (Hoek, 2002).

The minimum, modal and maximum spacing is generally recorded for each discontinuity set observed in rock exposures using the descriptions provided in (ISRM, 1978), (e.g., extremely close spacing is <20mm and extremely wide is >6m). These are processed statistically with the use of histograms for each of the discontinuity sets with the frequency curve providing an indication of the respective modal values and dispersions. According to (Priest & Hudson, 1976), a negative exponential distribution of discontinuity values is a good approximation for the true distribution patterns of any combination of evenly spaced, clustered, and randomly positioned discontinuities, unless there is a large predominance of evenly spaced discontinuities. Later publications on the relative frequency of discontinuity spacings suggest that it can be described by different distribution laws shown in Figure 5. The log-normal or negative exponential distribution laws are predominantly used in the literature with rare use of the normal distribution. There are many ideas put forward to explain these differences, with sampling bias being the most common. However, for unbiased data, (Dershowitz & Einstein, 1988) suggests that independently created joints can produce negative exponential distributions whereas joints that interact can create log-normal distributions. The research by (Sen & Kazi, 1984), presents

procedures that allow unbiased mean discontinuity spacing values to be obtained for exponential and log-normal distributions and suggests that the log-normal probability density function is more flexible because it considers the variance of the discontinuity spacings in any scanline evaluation. Further discussions on spacing distributions are included in (Priest & Hudson, 1981), (Rives, et al., 1992), (Priest, 1993) and (Hudson & Harrison, 1997). The collection methods for discontinuity spacing data and the related sampling biases are presented in Section 2.1.2.

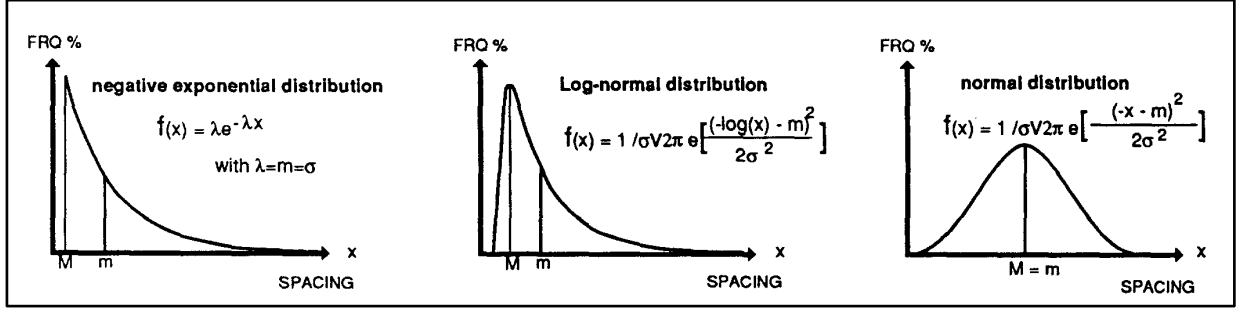


Figure 5: Different types of distribution laws used to describe discontinuity spacing; M-mode; m-mean; σ -standard deviation (Rives, et al., 1992).

Linear frequency refers to the number of discontinuities per unit distance and is the reciprocal of the mean spacing. Linear frequency is generally determined from scanlines and boreholes and varies with the direction of the sampling line. In addition, seismic refraction techniques can be used to determine the frequency based on the reasonably reliable relationships established by several researchers, such as (Palmstrom, 1995) and (Sjogren, et al., 1979).

2.1.1.3.2 Rock Quality Designation (RQD)

Rock Quality Designation (RQD), proposed by (Deere, 1963), is a measure of the quality of borehole core and defined as the percentage of the borehole core consisting of intact lengths of rock greater than or equal to 10cm. It is expressed mathematically as (Hudson & Harrison, 1997):

$$RQD = 100 \sum_{i=1}^n \frac{x_i}{L} \%$$

Where, x_i is the spacing values greater than 10cm, and n is the number of these intersected by a borehole core or scanline length L . This value of 0.1 m is referred to as the threshold value. (ASTM, 1997) provides further details for the determination of RQD.

RQD forms an important parameter to engineering rock mass classifications schemes (e.g., RMR system and Q-system). Correlations between RQD and linear discontinuity frequency have been derived for different discontinuity spacing distributions by researchers such as, (Priest & Hudson, 1976) and (Sen & Kazi, 1984). Seismic velocity measurements have also been used to estimate RQD. This rock quality index is widely used and easy to measure but can be subjected to measurement errors (e.g., counting drilling-induced break or measuring at an oblique angle to the core axis) and significant

bias caused by the direction of the borehole or scanline. Figure 6 shows an example of the RQD percentage varying with the sample direction with further discussions provided in (Palmstrom , 2005).

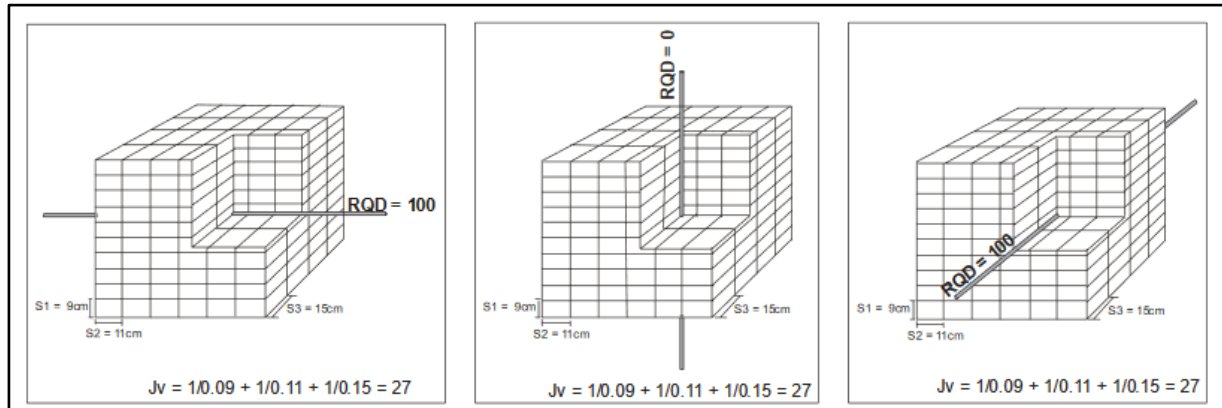


Figure 6: Example of three boreholes within the same rock mass in different directions. RQD can be both 0 and 100 (Palmstrom , 2005).

2.1.1.3.3 Areal and Volumetric Intensity

Volumetric frequency assumes that the occurrence of a discontinuity surface in a rock mass can be represented by the occurrence of a point located at the centroid or any other unique position on the discontinuity. Therefore, the volumetric frequency is the average number of these points per unit volume of rock sample. Direct estimation of volumetric frequency will require a volumetric rock sample dissected in a specific manner which is generally considered expensive and impractical. It is hence preferred to estimate volumetric frequency from areal or linear frequency from rock faces. An individual discontinuity occurrence can be represented by the mid-point of its trace on a rock exposure and the average number of these points per unit area of exposure is referred to as the areal intensity (Priest, 1993). Further discussion on intensity measures is presented in Section 2.2.4.

2.1.1.3.4 Block Size

Block size is another important parameter describing discontinuity intensity. According to (ISRM, 1978), the term block size is a composite description of the rock mass that is influenced by spacing, number of discontinuity sets and orientation. The number of discontinuity sets and the orientation determine the shape of the blocks that can be in the form of cubes, rhombohedrons, tetrahedral, sheets and many others. Figure 7 shows examples of discontinuity patterns where relatively regular jointing exists. However, according to (Palmstrom, 2001), there are no regular jointing patterns in most cases and hence it would be more practical to broadly characterise blocks into four main types only, as shown in Figure 8.

Block size properties together with the interblock shear strength govern the mechanical behaviour of the rock mass under given stress conditions. Block size has been incorporated in rock mass classification systems and is used in numerical and analytical solutions related to rock mass strength and

deformation moduli. There are many types of block size measurements, summarised in Table 1. (Palmstrom, 2001), (Palmstrom , 2005) and (ISRM, 1978) provide more details of these block size estimation methods and their limitations.

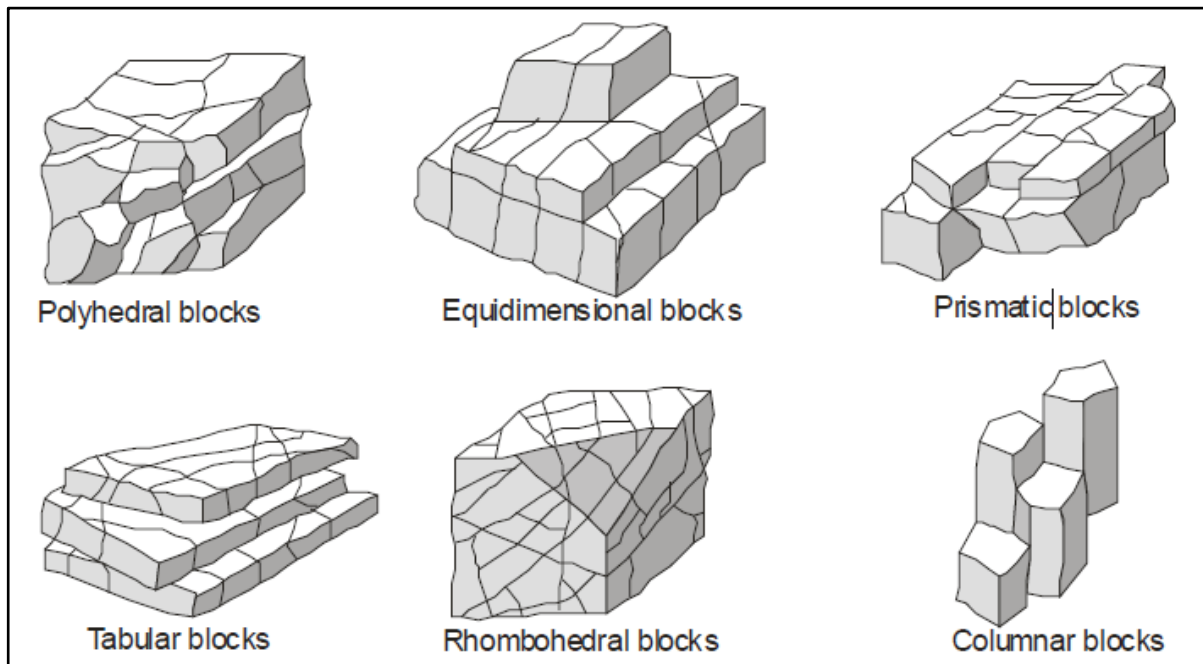


Figure 7: Examples of block shapes (Dearman, 1991).

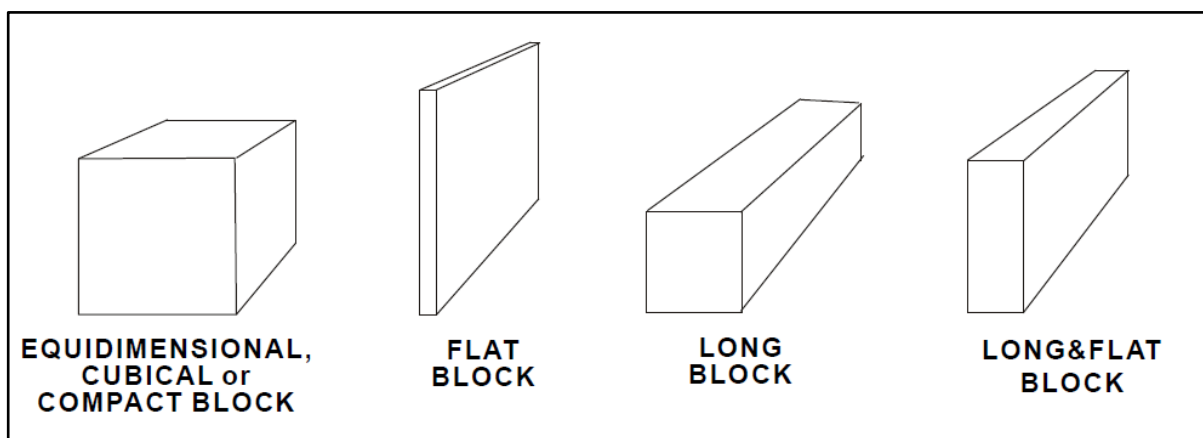


Figure 8: Main types of blocks (Palmstrom, 1995).

Table 1: Methods for measuring block size (Palmstrom , 2005)

Measurements in Rock Surfaces	Measurements on Drill Cores	Refraction seismic measurements
Block size (volume of block) (V_b)	Rock quality designation (RQD)	Sound velocity of rock masses
Volumetric joint count (J_v)	Fracture frequency	
Joint spacing (s)	Joint intercept	
Weighted joint density (wJ_d)	Weighted joint density (wJ_d)	
Rock quality designation (RQD)	Block volume	

2.1.1.4 Discontinuity Persistence, Size and Shape

Persistence refers to the areal extent or size of a discontinuity within a plane. It is difficult to determine due to the intrinsic three-dimensional nature of discontinuities within a rock mass, however, it can be crudely quantified by observing discontinuity trace lengths on exposed rock faces. The persistence of discontinuities is an important parameter as it has a significant influence on shear strength developed in the plane of a discontinuity, and on the fragmentation characteristics, cavability, and permeability of the rock mass. The discontinuity persistence is classified based on the modal trace length of each discontinuity set as detailed in (ISRM, 1978) (e.g., description of very low persistence is less than 1m, and very high persistence that is greater than 20m). In a rock mass, one particular discontinuity set is often more continuous than other sets and minor sets tend to terminate against primary structural features or may terminate within solid rock; hence discontinuities sets are distinguished using terms of persistent, sub-persistent and non-persistent as illustrated in Figure 9. Dip lengths and strike lengths are generally recorded along with the corresponding termination type.

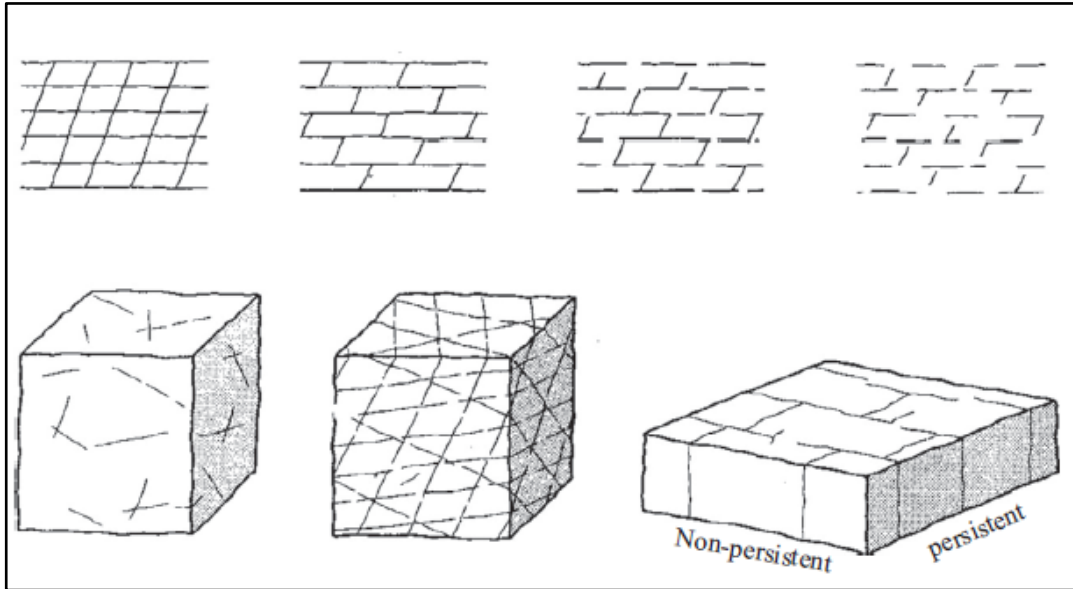


Figure 9: Simple sketches and block diagrams indicating the persistence of various sets of discontinuities (ISRM, 1978).

Discontinuity planes through a rock mass contains a patchwork of discontinuities and intact rock regions known as intact rock bridges. Therefore, the persistence of discontinuities in a rock mass are often described using a persistence ratio (PR). (Einstein, et al., 1983), with reference to a discontinuity plane in a rock mass, defines discontinuity persistence, K (later referred to as PR), as the fraction area that is actually discontinuous expressed as the limit:

$$K = \lim_{A_D \rightarrow \infty} \frac{\sum_i a_{D_i}}{A_D}$$

In which D is the region of the plane with area A_D and a_D is the area of the i_{th} joint in D , refer to Figure 10(a). The summation in the above equation is over all joints in region D . Equivalently, the discontinuity can be expressed as a limit length ratio along a given line on a discontinuity plane, shown in Figure

10(b). Furthermore, (Einstein, et al., 1983) discusses the persistence parameter further and describes a potential failure surface involving a path through the intact rock and discontinuities in a rock mass (also known as critical step-path failures).

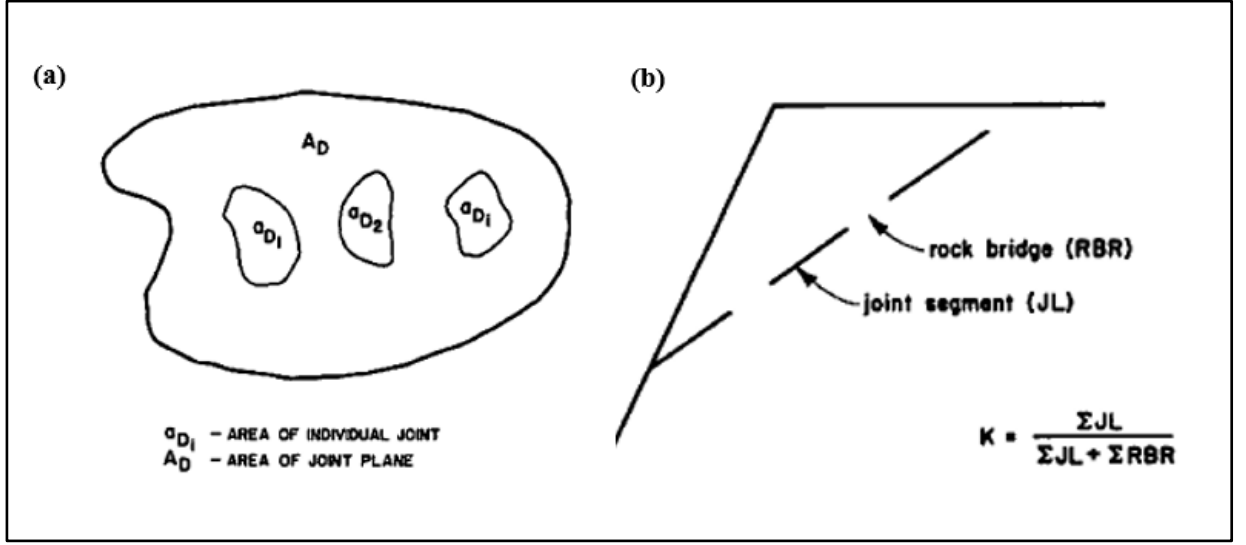


Figure 10: Discontinuity persistence defined as: (a) area ratio, and (b) length ratio (Einstein, et al., 1983).

As mentioned previously, persistence is related to the discontinuity size that is related to the trace length of rock exposures; hence the importance of methods used for the estimation of mean trace lengths of discontinuities. Trace lengths are subject to sampling bias and errors, the details of which are presented in Section 2.1.2.3. The probability distributions of measured trace lengths reported by many researchers are commonly exponential and lognormal with work by (Kulatilake, 1993) recommending the Gamma distribution. It should be noted that only researchers, (Cruden, 1977), (Baecher, et al., 1997) and (Kulatilake, 1993) applied many distributions to find the best fit representative for the trace length data. It is thus recommended by these authors, that suitable distributions for trace lengths of discontinuity sets be assessed using chi-squared and Kolmogorov-Smirnov goodness of fit tests.

(Pahl, 1981) suggested technique to estimate the mean trace length was based on the mathematical technique first used by (Wicksell, 1925). This estimate considers an infinite surface produced by a discontinuity set whose orientation has a single value (i.e. all discontinuities in the set have the same orientation) and uses categorization [i.e. traces with both ends censored (N_0), with one end censored and one end observable (N_1), and with both ends observable (N_2)] of randomly located discontinuities that intersect a rectangular window of a width (a) and height (b) and the traces that make an angle (φ) with the height side, as illustrated in Figure 11 (Zhang, 2017). The mean trace length, μ_l , can be determined by (Pahl, 1981):

$$\mu_l = \frac{ab(N + N_0 - N_2)}{(a \cos \varphi + \sin \varphi)(N + N_0 - N_2)}$$

Where, N is the total number of traces, and is equal to $N_0 + N_1 + N_2$. Pahl's technique was extended by (Kulatilake & Wu, 1984) and (Mauldon, 1998) to discontinuities whose orientations are described by probability distributions.

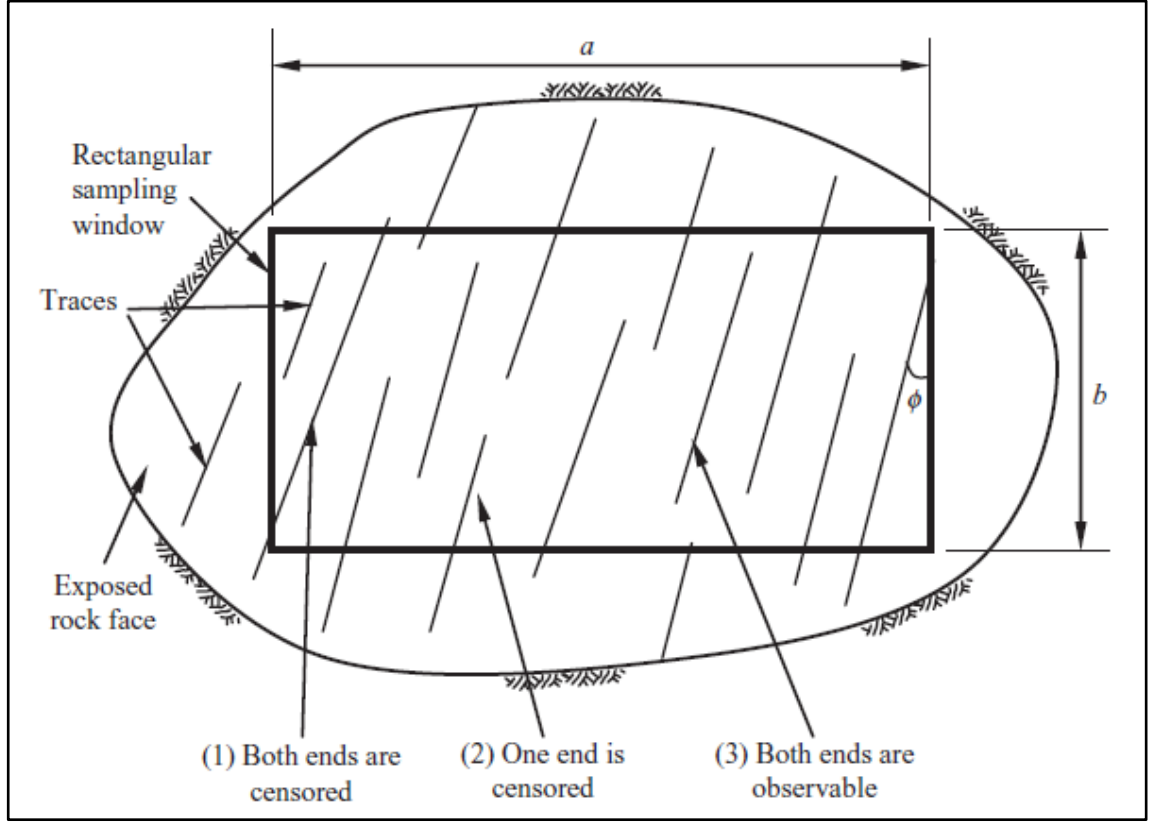


Figure 11: Discontinuities intersecting a rectangular sampling window (Zhang, 2017).

(Zhang & Einstein, 1998), following the methodology by (Pahl, 1981) and (Kulatilake & Wu, 1984) suggested a method that can estimate the trace length from observed trace data in a circular window, without considering the effect of orientation. The following expression was derived:

$$\mu_l = \frac{\pi(N + N_0 - N_2)}{2(N + N_0 - N_2)} c$$

Where c , is the radius of the circular sampling window (refer to Figure 12). It should be noted that the above expression was suggested by (Mauldon, 1998) using a different method.

The method proposed by (Pahl, 1981) and (Zhang & Einstein, 1998) do not contain errors due to censoring and to size bias (i.e. longer traces are more likely to be observed in a sampling area than a shorter trace); does not require trace length measurements or distributions; and assumes all discontinuities are planar.

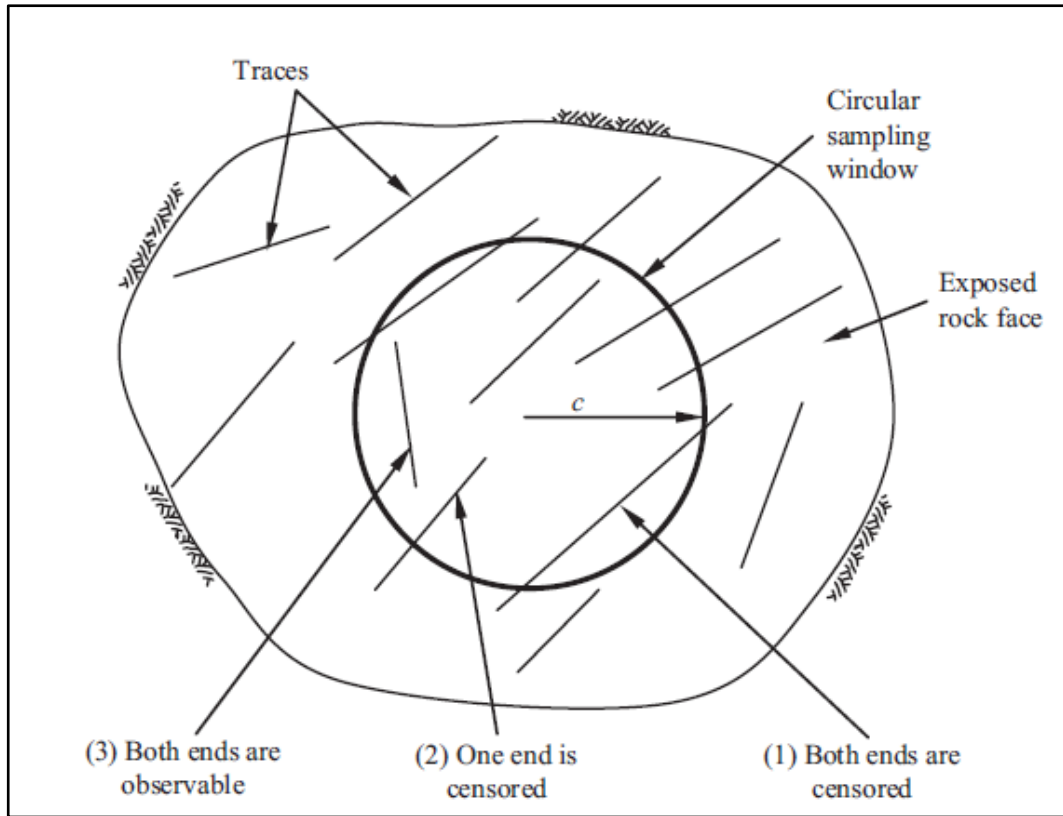


Figure 12: Discontinuities intersect a circular sampling window (Zhang, 2017).

2.1.1.4.1 Discontinuity Shape

Discontinuity shape is one of the more difficult parameters to determine as there is no direct reliable method of measurement and hence is often interpreted from limited shape information using many assumptions. Evidence from laboratory experiments and field observations suggests that the initial shape is circular for a tensile discontinuity in an isotropic homogenous rock. However, the circularity can be modified quickly by successive tectonic events and deformation processes such as faulting, folding and jointing thereby resulting in complex discontinuity shapes (Jing & Stephansson, 2007).

According to (Dershowitz & Einstein, 1988), the shape of discontinuity boundaries can be polygonal, circular, elliptical, or irregular and discontinuity surfaces can be planar or non-planar and the shape should consider this. Seeing that discontinuities are often planar; it is simpler to associate “shape” with the two-dimensional appearance and address non-planarity separately. It is common practice in rock engineering to assume discontinuities are planar (Priest & Hudson, 1976), (Baecher, et al., 1997), (Warburton, 1980a), (Kulatilake, 1993). Further discussions on the planar shape of discontinuities are provided in (Zhang & Einstein, 2010).

Many researchers have inferred discontinuity shape from the study of trace lengths in both the strike and dip directions. According to (Zhang, et al., 2002), the conclusion that discontinuities are equidimensional is drawn only from the fact that the average strike length of a discontinuity set is approximately equal to its average dip length but this equality of the average strike and dip length does

not necessarily mean that the discontinuities of this set are equidimensional but rather the following possibilities exist (Figure 13):

- Discontinuities are non-equidimensional, for example elliptical or rectangular, with their long axes in a single (or deterministic) orientation. However, the discontinuities are oriented so that the average strike length is approximately equal to the average dip length;
- Discontinuities are in fact equidimensional;
- Discontinuities are non-equidimensional, for example elliptical or rectangular, with their long axes randomly oriented. The random discontinuity orientation distribution makes the average strike length approximately equal to the average dip length.

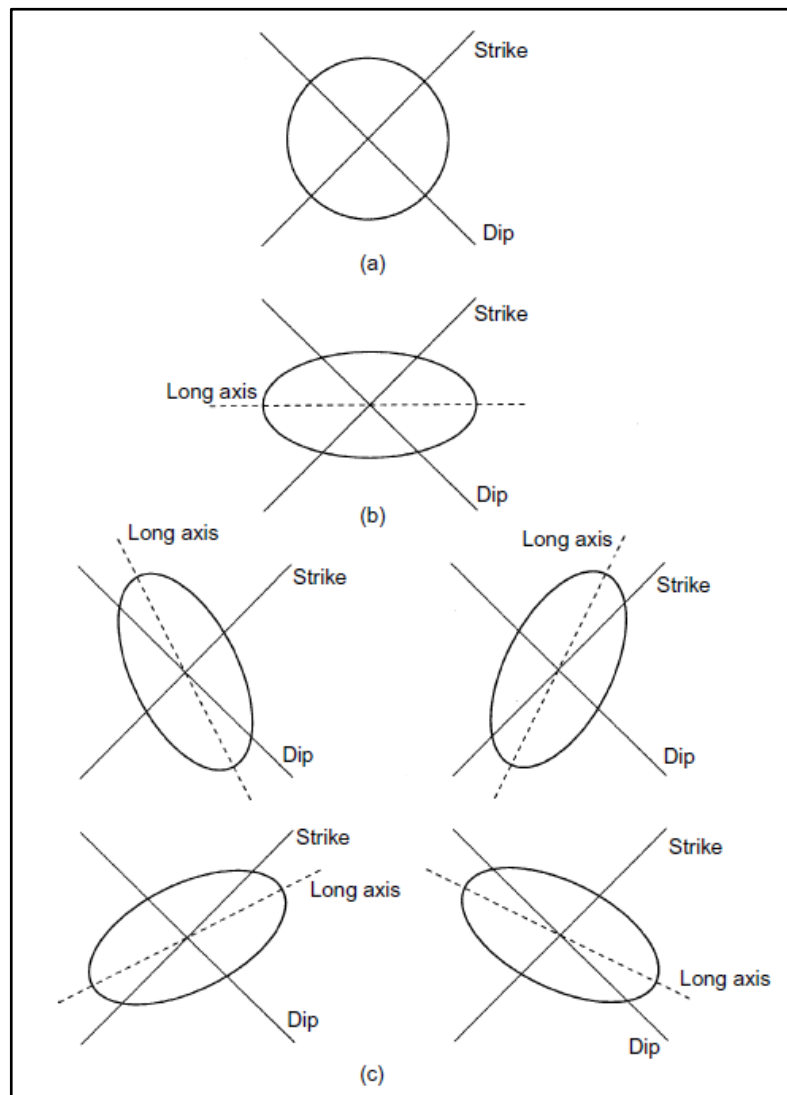


Figure 13: Three possible cases for which the average strike length is about equal to the average dip length. (a) Discontinuities are equidimensional (circular). (b) Discontinuities are non-equidimensional (elliptical), with long axes in a single orientation. (c) Discontinuities are non-equidimensional, with long axes randomly oriented so that the average strike length is about equal to the average dip length (Zhang, et al., 2002).

2.1.1.4.2 Discontinuity Size

Discontinuity size is a fundamental consideration in the evaluation of mechanical and hydraulic properties of rock masses. One of the existing methods for estimating discontinuity size distributions is to use stereological relations between the distributions of discontinuity size and (true) trace length. The true trace length distribution is derived from the measured trace lengths at exposed rock faces with correcting the sampling biases and making assumptions regarding the discontinuity shape.

(Warburton, 1980a), (Warburton, 1980b), (Priest & Hudson, 1981), (Villaescusa & Brown, 1992) (Baecher, et al., 1997), (Zhang & Einstein, 1998), (La Pointe, 2002), (Zhang, et al., 2002), (Priest, 2004), (Lyman, 2003), provide methods for inferring discontinuity size distributions. (Warburton, 1980a) provided the starting point for determining the discontinuity size with a simple model that assumes discontinuities are flat circular discs of negligible thickness, occurring parallel to each other and is centred at random points with volume density having a Poisson distribution. (Warburton, 1980a) derived the following relationship of the trace length distribution and the distribution of discontinuity size:

$$h_A(y) = \frac{y}{m} \int_y^\infty \frac{g(x)}{\sqrt{x^2 - y^2}} dx$$

Where y is the trace length and x is the fracture diameter; $h_A(y)$ is the probability density function (pdf) of trace length, $y:g(x)$ is the pdf of fracture diameter and the subscript A denotes the sampling of trace length measurement; and m is the mean of x , that is:

$$m = \int_y^\infty xg(x)dx.$$

Following the above methodology of (Warburton, 1980a), (Warburton, 1980b), (Zhang, et al., 2002), a general stereological relationship was derived between trace length distributions and discontinuity size (expressed by the major axis length, a , of the ellipse) for area sampling based on the assumption that discontinuities are elliptical in shape (see Figure 14). The probability density function of trace length, l :

$$f(l) = \frac{l}{M\mu_a} \int_{l/M}^\infty \frac{g(a)}{\sqrt{(Ma)^2 - l^2}} da \quad (l \leq aM)$$

where,

$$M = \frac{\sqrt{\tan^2 \beta + 1}}{\sqrt{k^2 \tan^2 \beta + 1}}$$

In which, k is the aspect ratio of the discontinuity (i.e., the ratio of the major to minor axes, the length of the discontinuity minor axis is a/k); β is the angle between the discontinuity major axis and the trace line measured in the discontinuity plane. Hence, β will change for different sampling planes, however for a specific sampling plane, there will only be one β value for a discontinuity set with a deterministic orientation. From this relationship, expressions were derived for calculating the mean and standard deviation of the discontinuity size for log-normal, negative exponential and gamma distributions and is provided in (Zhang, et al., 2002).

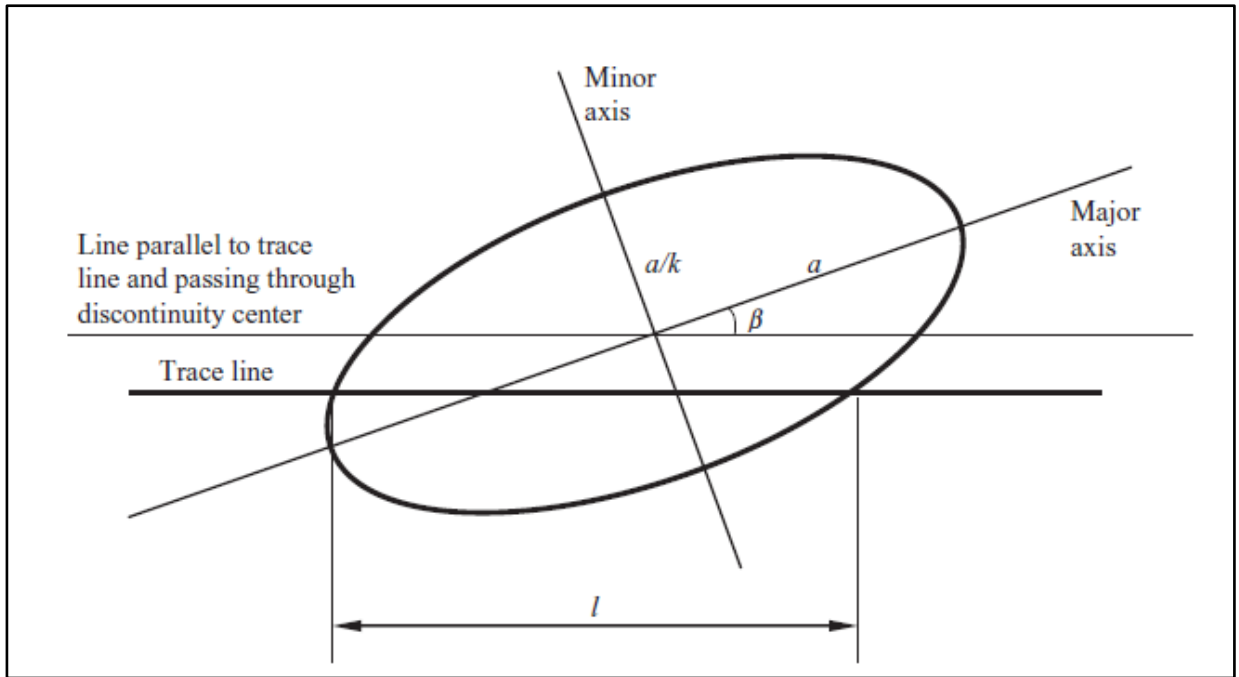


Figure 14: Parameters used in the definition of an elliptical discontinuity [In (Zhang, 2017), after (Zhang, et al., 2002)].

2.1.2 Structural Data Collection Methods

There are various techniques that are applied to the collection of discontinuity data from boreholes or rock exposures. These techniques generally involve one or a combination of the following:

- Traditional manual mapping and logging (e.g., scanline mapping, window mapping; borehole core logging);
- Digital imaging [e.g., photogrammetry, laser scanning/LiDAR, Unmanned Aerial Vehicle (UAV) surveys (e.g., drones) and borehole optical televiewer surveys];
- Surface geophysical techniques (e.g., seismic reflection/refraction methods, gravity, resistivity, and electromagnetic surveys).

Each of these methods and associated data processing have their own advantages (e.g., accuracy, efficiency, safety, cost effective etc.), and disadvantages (e.g., time-consuming, poor quality, expensive, impractical, unreliable etc.). The method selected is dependent on factors such as, the nature of the problem to be solved, accessibility of rock exposures, acceptable level of risk, and available resources (i.e., skilled personnel and finances). The technological advances in the data collection methods have improved the overall quality and quantity of structural data available however there are still uncertainties, errors and bias that need to be considered with their engineering application. An overview of the sampling errors and biases associated with structural data is presented in 2.1.2.3.

For this research project, traditional outcrop mapping methods and digital mapping methods were used to collect the structural data at Mine X. An overview of the data collection methods used is provided below.

2.1.2.1 Scanline Mapping Method

Scanline mapping is a one-dimensional discontinuity sampling technique. It involves fixing a measuring tape along a rock face and measuring all discontinuities intersecting the scanline, as illustrated in Figure 15. Discontinuities that do not intersect the scanline can also be recorded by projecting the scanline intersection points. The parameters generally measured for each discontinuity include the distance along the scanline, orientation, discontinuity type, surface roughness and planarity, spacing, aperture size, infill type and thickness, persistence, and termination types. Additional parameters and/or descriptions are often added by practitioners based on the industry application. It should be noted that the ISRM suggested methods for structural data collection does not adequately consider the application to discrete fracture modelling; however, these guidelines are outlined in (Elmo, et al., 2015).

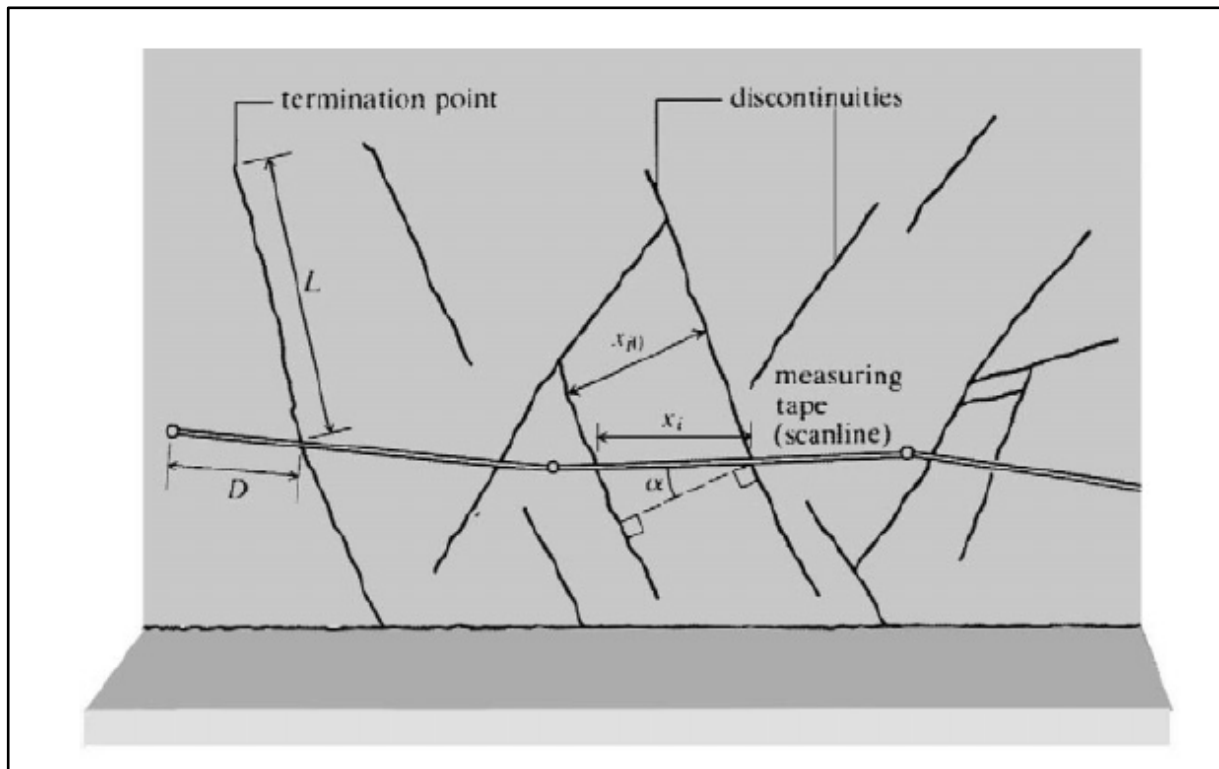


Figure 15: Scanline Survey (Brady & Brown, 2005).

2.1.2.2 Digital Photogrammetry Method

Photogrammetry can be defined as, “the art, science and technology of obtaining reliable quantitative information about physical objects and the environment throughout the process of recording, measuring and interpreting photographic images and patterns of radiant energy derived from sensor systems” (Slama, et al., 1980). Stereo photogrammetry, a common form of photogrammetry, involves the

estimation of an objects position in 3D space by using the relationship between the relative positions of the objects in two or more images of the objects acquired at known locations with known orientations. Using this system, spatial mapping is performed by acquiring two images of the same object/scene from locations separated by a suitable distance. Two cameras can be used to capture the images at the same time or a single camera to capture the images from separate locations at different times. The quality of the photogrammetric mapping is directly related to the following (CSIRO Exploration and Mining, 2002):

- the image quality;
- the geometric relationship between the camera position (s) and the object being photographed;
- the spectral and spatial resolution of the camera;
- the spectral and spatial variance of the object; and
- in some cases, problems can arise from using a single camera because of the change in lighting that can occur between the different times;

The geometry of photogrammetry is summarized in Figure 16. According to (CSIRO Exploration and Mining, 2002), photogrammetry is a process of triangulation using the angle measurements obtained from the images. As shown in Figure 16, the object Point 'P' can be calculated from measurements of the positions of the image(s) of the point in the left and right images, that is the positions of points 'PL' and 'PR'. These measurements combined with; the position of the camera for each image, the orientation of the camera for each image, and the focal length of the camera; the position of point 'P' can be determined. A 3D map can be generated by repeating this process for a set of objects, or points. Most software used to create these 3D image models are based on this concept. Typical photogrammetry software available includes ©Sirovision, ©3G – ShapeMetrix & JointMetrix and ©Adams Technology –3DM Analyst. Errors are commonly introduced by image distortions causing errors in the calculated position of an object at point 'P', and, incorrect specification of the focal length which can cause errors in the position and scale of the 3D map generated. It is thus important to calibrate the camera in order to minimise errors and provide an estimate of mapping accuracy. This camera calibration typically involves determining of the centre of the image (zero position for the image coordinate system), focal length of the camera lens and lens distortion parameters (CSIRO Exploration and Mining, 2002).

Once the 3D image model is created, structural mapping can be carried out using the integrated structural mapping capabilities of the software package (e.g., Sirojoint in Sirovision). This allows the orientations of each discontinuity mapped to be determined by generally fitting a plane to the points located within the discontinuity boundary and then calculating the dip direction and dip angle of the best-fit plane. Further details on these photogrammetry considerations are discussed in (CSIRO Exploration and Mining, 2002) and (Slama, et al., 1980).

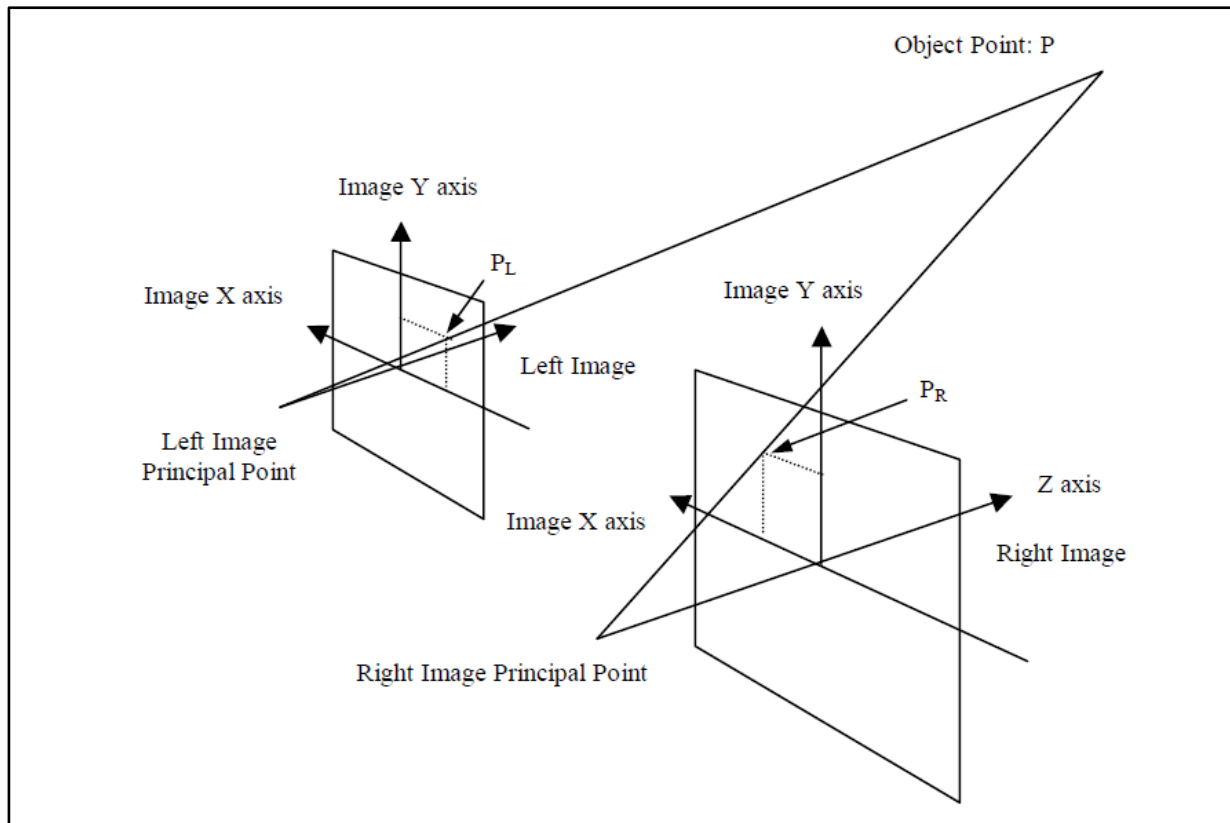


Figure 16: Geometry for the determination of the position of a point in object space (CSIRO Exploration and Mining, 2002).

2.1.2.3 Sampling Bias and Errors

The true characteristics of discontinuities cannot be known exactly however in order to determine the best estimate of discontinuity characteristics, the measurements of these properties should be as unbiased and objective as possible. All discontinuity data collection methods are associated with some bias and sources of errors that are inherent in the methodology and/or introduced by inappropriate or defective instrumentation used or by the subjectivity of different field practitioners themselves. The knowledge of these errors and bias prior to undertaking of an investigation can help avoid, limit, and possibly correct these. According to several sources such as: (Priest & Hudson, 1981), (Einstein, et al., 1983), (Kulatilake & Wu, 1984), (Villaescusa & Brown, 1992), (Mauldon, 1998) (Zhang & Einstein, 1998), (Brown, 2002), and a few others, the following sampling biases and measurement errors are common with discontinuity data:

- **Measurement errors** – these include sample disturbance, random errors, bias errors, and statistical fluctuations. Bias and systematic errors are introduced by the preferential collecting or not collecting of some information. Random errors can generally be reduced with larger sampling sizes. A study carried out by (Ewan & West, 1981) on measurement errors by examining the reproducibility of joint orientation measurements using six different observers performing 10m scanline surveys concluded that the orientation of a particular discontinuity recorded can have an average maximum variation of 10° for dip direction and 5° for the dip angle without there being any

actual difference in the joint orientation. It should be noted that despite these errors, the discontinuity orientation diagram for the collected discontinuity data identified the same major discontinuity sets (Brown, 2002).

- **Orientation bias** – the frequency of discontinuities that will intersect a selected scanline, window or drill core is dependent on the orientation of the sampling window or rock exposure relative to the orientation of the discontinuity set. If the discontinuity set is orientated perpendicular to the sampling window or line, then the probability of intersecting these discontinuities is the highest with the lowest probability associated with the discontinuity set orientated parallel to the sampling window or line. The probability of encountering discontinuities decreases with the angle between the sampling window. Orientation bias is generally corrected with a weighting factor first proposed by (Terzaghi, 1965).
- **Size (or length) bias** – the larger the scale (i.e., surface area or trace length) of the discontinuity, the more likely it is to be sampled by a given drill core, scanline, or mapping window.
- **Truncation bias** – a truncation size or size cut-off is usually used in scanline or window mapping due to the difficulty and practicality of measuring very small trace lengths. According to (Brown, 2002), fractures that are less than 50mm in length may be ignored. It was also noted that such a small cut-off will generally have little effect on the overall discontinuity statistics, however if the aim is to comprehensively describe the distribution of the discontinuity sizes, then the truncation size cut-off must be considered.
- **Censoring bias** – this bias is associated with the artificial boundaries that are imposed when performing rock mapping, long discontinuity traces may extend beyond the visible exposure so that one end or both ends of the discontinuity trace cannot be observed. These limiting boundaries are typically, the bench heights of open pit mines, drive heights in underground mines and the size of the mapping window. Censored trace lengths generally provide a lower bound estimate of the true trace length.

An unbiased structural analysis requires a measurement process that is objective. The appropriate selection of methodologies and sampling sites can greatly reduce errors and bias (e.g., the use of three mutually orthogonal scanlines to reduce the orientation bias). It is more difficult trying to account for errors due to subjectivity than for systematic sampling errors (Brown, 2002). Where it is not possible to account for all biases, it must be removed or reduced using analytical or numerical methods.

The potential errors and sampling biased of the discontinuity data collected has an influence on the subsequent discontinuity analysis and modelling discussed further in the next Section 2.1.3.

2.1.3 Structural Modelling Approaches

The structural data collected are used in the subsequent discontinuity analysis and structural modelling. The main structural modelling approaches and tools used include (Read, 2009):

-
- **Solid modelling** – this involves the three-dimensional modelling of the structural geology using commercial modelling software packages such as Vulcan, DataMine, Rockworks, Gemcom etc. These software packages generally incorporate various statistical interpolation methods, each with their own assumptions and advantages that facilitate the 3D geometric modelling of the geological units and structures.
 - **Stereographic projection** – this is a very powerful method of displaying and interpreting the three-dimensional geometry of line and planes. It allows for quick solutions to complex geometric problems. In the open pit industry, it is generally used for the processing and interpretation of structural data and related kinematic analysis. The most widely used stereographic projection software is Rocscience DIPS. Further details on the use of this method are described in Section 2.4.2.
 - **Discrete fracture network modelling** – this modelling allows explicit representation of how the faults and joints recognized in the structural model are spatially distributed within the rock mass. This has helped visualize how the rock mass deforms and how failure mechanisms develop in slopes, more specifically, sliding failure involving major structures and fractures across an intact block of rock (rock bridges) left between these structures. DFN modelling packages include FracMan, JointStats, 3FLO and SIMBLOC. Additional important applications include fragmentation analysis and determining flow conditions in hard rock masses. Section 2.2 provides more detail on this modelling approach.

2.1.3.1 Modelling Errors and Uncertainty

Modelling of discontinuities has always been challenging due to the high degree of variability and uncertainty. In engineering geology, (Einstein & Baecher, 1982), have identified three major sources of uncertainty and error that are also applicable to discontinuity analysis. These sources and consequences are described below:

1) *Innate spatial variability of geological formations;*

The geology of the earth sub-surface is spatially variable as it is composed of different materials which are stratified, truncated and in other ways separated into more or less discrete zones. Even within a homogeneous body, material properties vary from point to point. Whilst this variability can be addressed with a sufficient number of observations, this is often not the case in the rock engineering industry, hence the uncertainty remains with material properties or classification at certain points that have not been sampled or observed. The important consequences of a limited number of measurements being used to describe the subsurface are that the values for these parameters are imprecisely known and the actual spatial variation may differ from the measured data range used.

2) *Inaccuracies caused by modelling physical behaviour;*

The description of the modelling behaviour of an engineering geologic structure requires the assumption of a model based on theory or empirical relations. These models are often a simplification

of reality and hence can introduce modelling errors. These errors are caused by; uncertainties related to the theory assumed for the applied physical process, the boundary conditions and initial conditions which must be selected; errors introduced by numerical modelling or mathematical approximations, and omissions (i.e. important factors left out of the model). It should be noted that modelling errors and parametric estimation are interdependent.

3) *Errors introduced in measuring and estimating engineering properties;* (i.e., related to sampling and measurement techniques described in section 2.1.2.3).

According to (Elmoultie & Poropat, 2015), rock mass modelling requires measurement and analysis procedures that can address ‘total uncertainty’, which is a combination of stochastic variability and inherent uncertainty. An example of the stochastic variability would be the distribution of adequately measured orientations of structures, which is generally addressed by assigning statistical representations of the variable. The inherent variability would be those parameters that are not as well defined such as discontinuity shape, which often requires the engineer to determine its suitability in the model. In the engineering geology industry, uncertainty is inevitable however one should always attempt to reduce this as much as possible.

2.2 Discrete Fracture Network (DFN) Modelling

The DFN method was introduced in the 1970s and its earliest applications were mainly in the petroleum, nuclear and hydrogeological disciplines. However, over the last few decades, there has been a major increase in the application of the DFN approach to solve geomechanical problems (Elmo, et al., 2015).

Discrete fracture network modelling is a methodology used to generate three-dimensional stochastic representations of rock mass structures from statistical distributions that describe their characteristics (e.g. orientation, spacing, size, aperture). The stochastic nature of the DFN modelling process is such that there are an infinite number of possible realizations of the 3D fracture system based on the sampled field data. The DFN model generated from this sampled field data (i.e., 1D/2D discontinuity data collected from variably oriented rock exposures and boreholes) share the sample statistics and allow for explicit modelling of an individual fracture or simplified fracture sets (Elmo, et al., 2015).

The principles of DFN modelling have been documented by several researchers such as (Dershowitz, et al., 1998), (Jing & Stephansson, 2007), (Dershowitz & Einstein, 1988), (Elmo, et al., 2015), (Grenon & Hadjigeorgiou, 2008) and (Rogers, et al., 2006). Advancement in technology over the years has facilitated the diversification of DFN applications, examples of which include (Lorig, et al., 2015):

■ *Mining Geomechanics;*

- Estimation of rock mass strength, fragmentation and micro-seismicity in caving mines, and to estimate anisotropic rock mass deformability and strength in open-pit mines through use of synthetic rock mass models;
- Kinematic analysis of block stability in open-pit and underground excavations;

- Determination of equivalent porous media for flow estimates and pore pressure distributions;
- Assessment of large-scale slope stability (considering the influence of intact rock bridges);
- *Civil Geomechanics;*
 - Excavation stability.
 - Flow studies for nuclear waste repositories.
 - Grout injection design for fractured rock.

The development of a DFN model requires the definition of several fracture properties summarised in Table 2. Primary properties represent the geometry of the fracture properties and is essential for all DFN modelling. The secondary properties are defined based on the DFN application (i.e., hydraulic or geomechanical).

Table 2: Primary and Secondary Properties for defining a DFN model (Elmo, et al., 2015)

Primary	Secondary
Orientation distribution	Aperture distribution
Fracture length distribution	Fracture shear strength properties
Fracture intensity distribution	Fracture stiffness properties
Spatial variation	Transmissivity distribution
Termination percentage	Storativity distribution
	Termination percentage

Figure 17 illustrates the typical process involved with the generation of a DFN model. Data required for the model development is derived from structural data collection methods discussed previously in section 2.1.2. Typical analysis involves field data processing and statistical assessment of discontinuity data to determine the primary fracture properties (i.e., DFN inputs in the form of statistical distributions or average values). Figure 18 shows a general workflow for the statistical data processing of these DFN inputs that would be completed for each defined discontinuity set.

Model validation is an important aspect in evaluating the suitability of the synthetic fracture network generated. The “validation loop” shown in Figure 17 refers to a refining process that involves testing the developed model against various validation data sets in order to ensure that it is representative of the actual rock mass (Golder Associates, 2006). The DFN model generated is validated by comparing the orientation, intensity, and pattern of the simulated fracture traces to those observed or measured in the field.

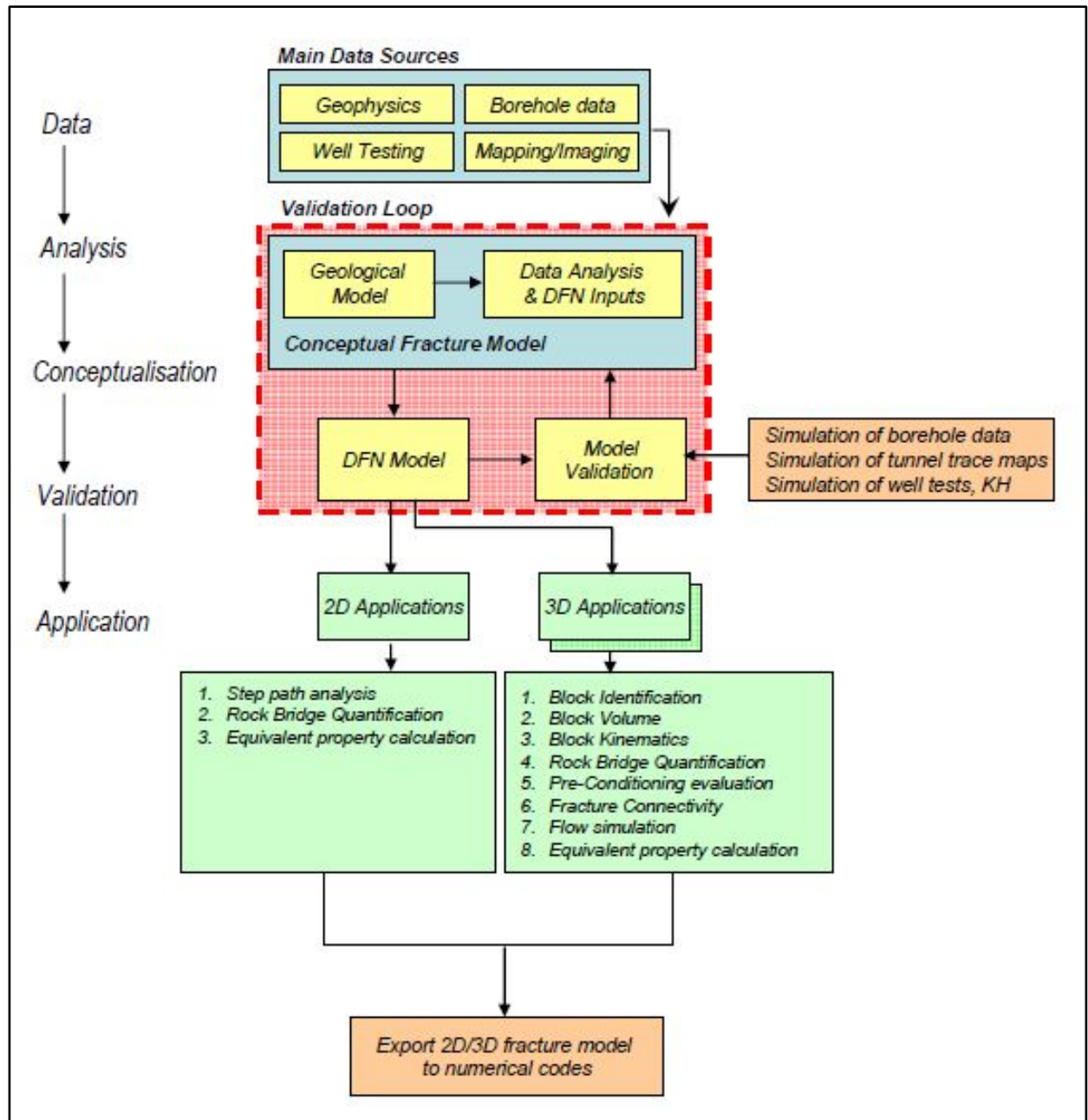


Figure 17: Generalized workflow for the development and application of a DFN model (Rogers, et al., 2007).

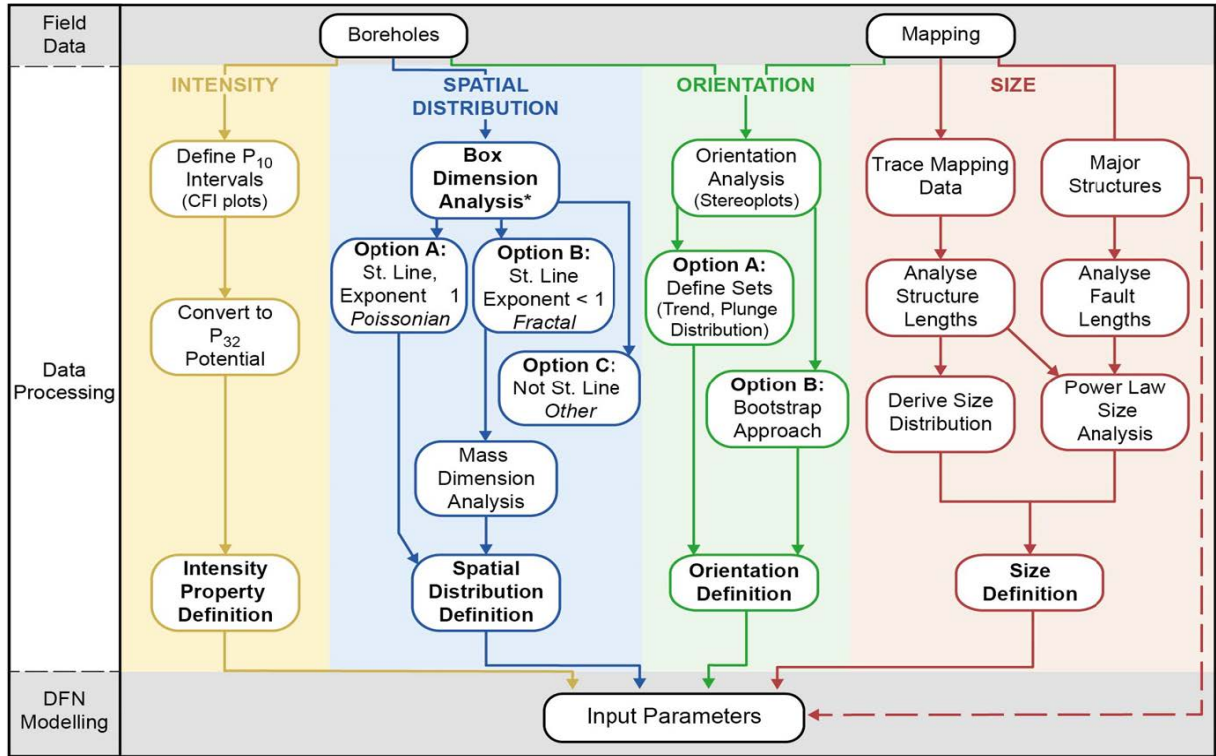


Figure 18: Workflow for statistical data processing to derive the necessary DFN input parameters (Weir & Fowler, 2014).

Each of the DFN model inputs are discussed further in the following sub-sections. These sub-sections are extensions of the discontinuity properties discussed previously in section 2.1.1. It should be noted that the term “fracture” and “discontinuity” are synonyms and are used interchangeably in this dissertation.

2.2.1 Fracture Spatial Models

A spatial model determines the way fractures are generated within a given 3D volume. There are many spatial models that have varying considerations regarding fracture characteristics. The difference between models is generally with the specific distribution laws that governs the simulation of fracture orientation and location. The spatial model selected for the generation of a DFN will be based on the field data, geological observations, scale, and its application. Conventionally, the Poisson DFN model (or Baecher model) has been used and is incorporated in many software packages as it is considered suitable to provide solutions for different applications involving relatively small scale DFNs. However, it does not allow for geostatistical data considerations that may be more appropriate for complex large-scale DFNs (Elmo, et al., 2015). A more detailed summary of the models is provided in (Dershowitz & Einstein, 1988) and (Staub, et al., 2002) and (Fadakar-Alghalandis, 2014).

The FracMan software package includes spatial models such as the Enhanced Baecher, Nearest-Neighbour, and Fractal Levy-Lee model. Figure 19 shows an example of these different spatial models. The fracture location for Enhanced-Baecher model is defined by a regular deterministic pattern or

stochastic process, whereby the stochastic approach assumes that the fracture centres are randomly located in space using the Poisson process (Elmo, et al., 2007). The fractal Levy-Lee model creates fracture centres sequentially and the fracture size is related to its distance from previous fractures (Staub, et al., 2002). The Nearest neighbour model is appropriate to model the tendency of fractures to be clustered around major points and faults that preferentially create new fractures in the proximity of earlier fractures (Dershowitz, et al., 1998).

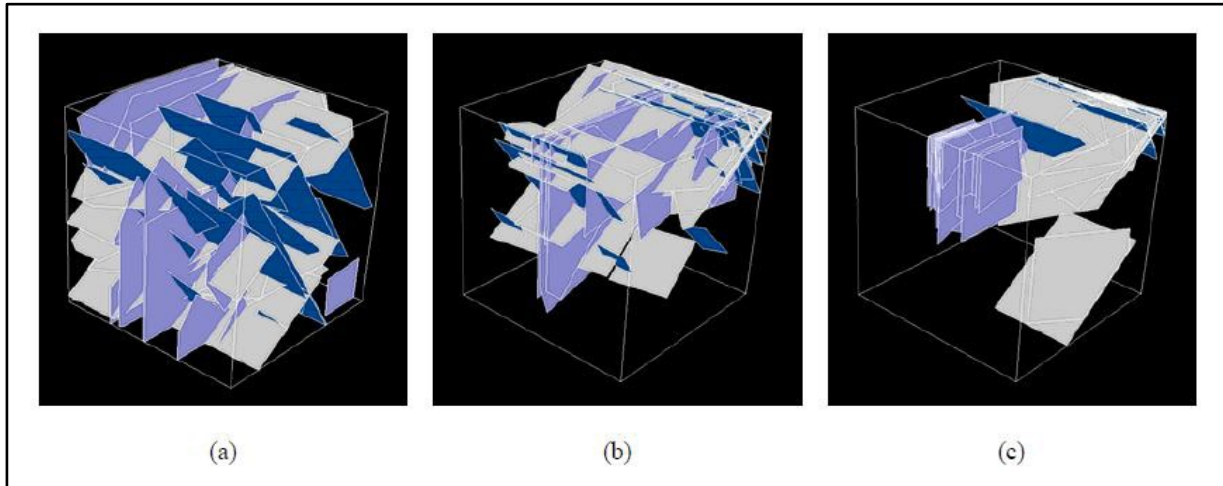


Figure 19: DFN models generated using different fracture spatial models for equivalent fracture orientations and radius distributions. (a) Enhanced Baecher model, (b) Nearest Neighbour and (c) Fractal Levy-Lee model (Elmo, et al., 2007).

2.2.2 Fracture Size

Fracture length or size is perhaps one of the most important parameters as it directly affects the fracture connectivity and rock block sizes. Section 2.1.1.4 provides details on the several methods used for fracture size estimation. The fracture size in the DFN modelling is typically expressed as equivalent radius determined by either a simulated sampling technique or analytical methods. (Mauldon, 1998), (Warburton, 1980a), (Warburton, 1980b), (Pahl, 1981), (Priest & Hudson, 1981), (Villaescusa & Brown, 1992), (Baecher, et al., 1997), (Zhang & Einstein, 1998), (Zhang, et al., 2002), (Priest, 2004), (Lyman, 2003), (Kulatilake & Wu, 1984), provide methods for inferring fracture size. These methods differ depending on the assumed fracture shape, observed trace length distributions and methodology used to treat sampling biases. Lognormal or negative exponential distributions have been used to describe the equivalent radius; however, the power law distribution has been applied to correlate small-scale fractures to larger scale features such as faults.

2.2.3 Fracture Orientation

There are several probability distributions suggested to represent the discontinuity orientations such as hemispherical uniform, Fisher, bivariate Fisher, Bingham, bivariate normal and bivariate lognormal (Zhang, 2017). Section 2.1.1.2 provided details on the processing and assessment of fracture

orientation data. DFN models can be generated using two approaches, namely the aggregate and disaggregate approach. The disaggregate approach involves the application of separate statistical procedures (inclusive of fracture intensity and size) to define fracture sets and subsequently separate DFN models for each fracture set is combined to obtain an overall representation of the fracture network. Conversely, the aggregate approach applies the same statistical distribution for all fractures and relies on the definition of a single fracture intensity and fracture size (Miyoshi, et al., 2018).

FracMan offers the “bootstrap method” to generate an aggregate DFN model to simulate highly dispersed orientation data. This statistical method is based on multiple random sampling with replacement from an original sample that is used to create a pseudo-replicate sample of fracture orientations distribution (Miyoshi, et al., 2018). Figure 20 illustrates the bootstrap approach.

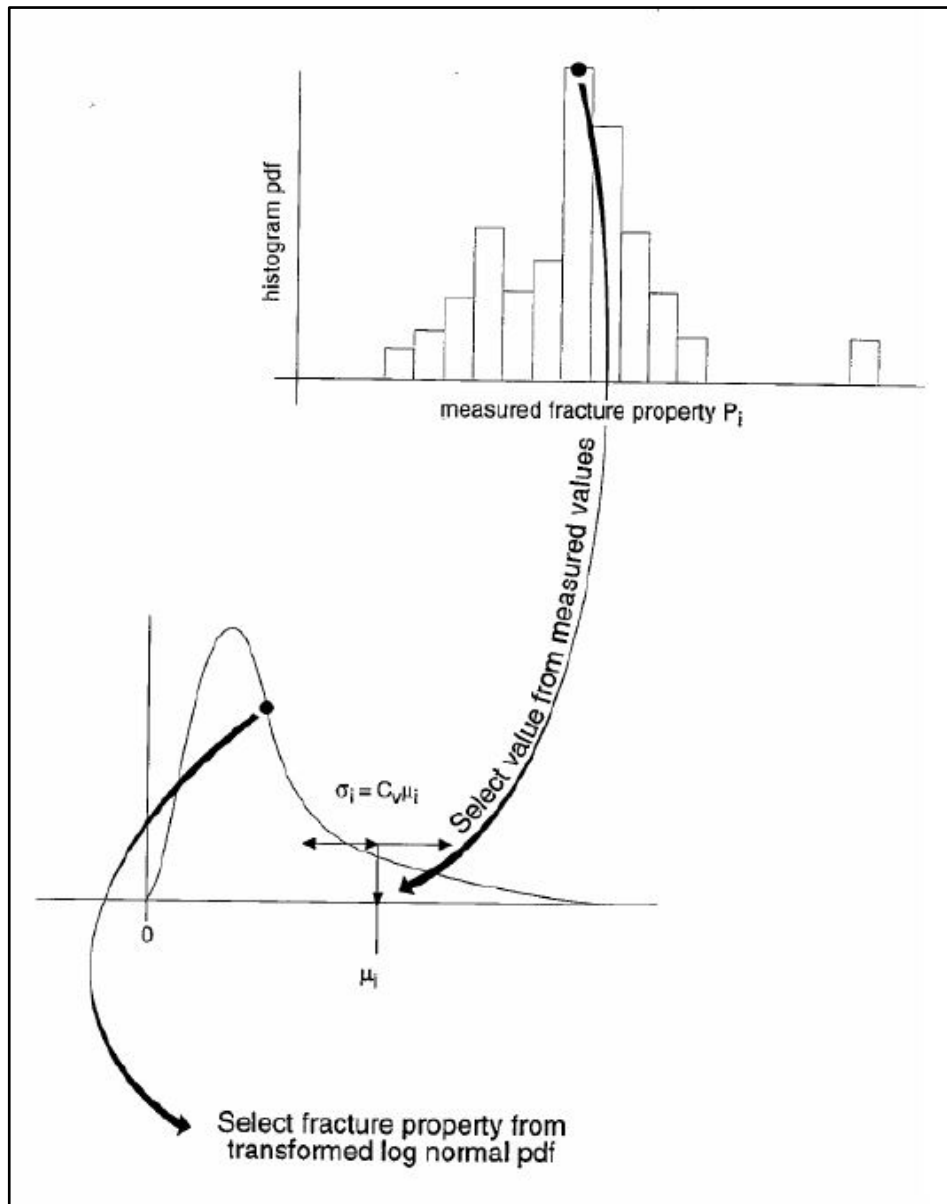


Figure 20: Bootstrap DFN model (Dershowitz, et al., 2004).

2.2.4 Fracture Intensity

Fracture intensity has a wide range of possible measures as discussed in previous section 2.1.1.3. In the DFN community, a unified system (i.e., Pij system) of fracture intensity measures has been developed to provide an easy framework to move between varying scales and dimensions (Rogers, et al., 2009). (Dershowitz & Herda, 1992) expressed fracture intensity as the area of fractures in a volume of rock and developed this Pij system (Figure 21), where the “i” denotes the sample dimension (e.g., borehole, trace map, volume) and “j” denotes the dimension measurement (e.g., count, length, area). For example, fracture intensity from borehole data (i.e., fracture frequency from borehole logging or borehole imaging tools) would be P10 whilst P21 is derived from trace mapping of benches or tunnel walls. Both these sample sources are directionally biased. However, the volumetric fracture intensity, P32 (fracture area/volume) represents a non-directional intrinsic measure of fracture intensity and is generally preferred for a DFN model (Elmo, et al., 2014). P32 cannot be measured directly and hence is inferred using a simulated sampling methodology with the 1D and 2D borehole or mapping data.

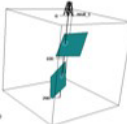
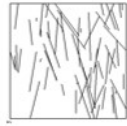
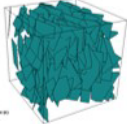
		Dimension of Measurement				
		0D	1D	2D	3D	
Dimension of Sample	1D 	P10 [1/m] No of fractures per unit length of borehole	P11 Length of fractures per unit length			Linear Measures
	2D 	P20 No of fractures per unit area	P21 [1/m] Length of fractures per unit area	P22 Area of fractures per area		Areal Measures
	3D 	P30 No of fractures per unit volume		P32 [1/m] Area of fractures per unit volume	P33 Volume of fractures per unit volume	Volumetric Measures
		Density		Intensity	Porosity	

Figure 21: Fracture Intensity - Pij System [adapted from (Dershowitz & Herda, 1992)].

Fracture frequency from borehole data (P10) is the most commonly used parameter to determine the fracture intensity in DFN modelling. This is carried out using cumulative fracture intensity (CFI) plots, (Figure 22) to identify the zones of the rock mass where P10 remains constant (i.e., gradient of the CFI plot is relatively constant) over a given interval length for each borehole. The interpretation of CFI plots from a large number of boreholes creates a data set of P10 values and interval lengths which provides the basis for fracture intensity extrapolation and any 3D spatial modelling (Rogers, et al., 2014). Alternatively, a DFN model can be generated with P10 conditioning which allows for direct replication of P10 intensity measured along field sampling line to the modelled sampling line. DFN models generated by the P10 conditioning process have been found to best satisfy both the relative spatial intensity and the actual intensity (Weir & Fowler, 2014).

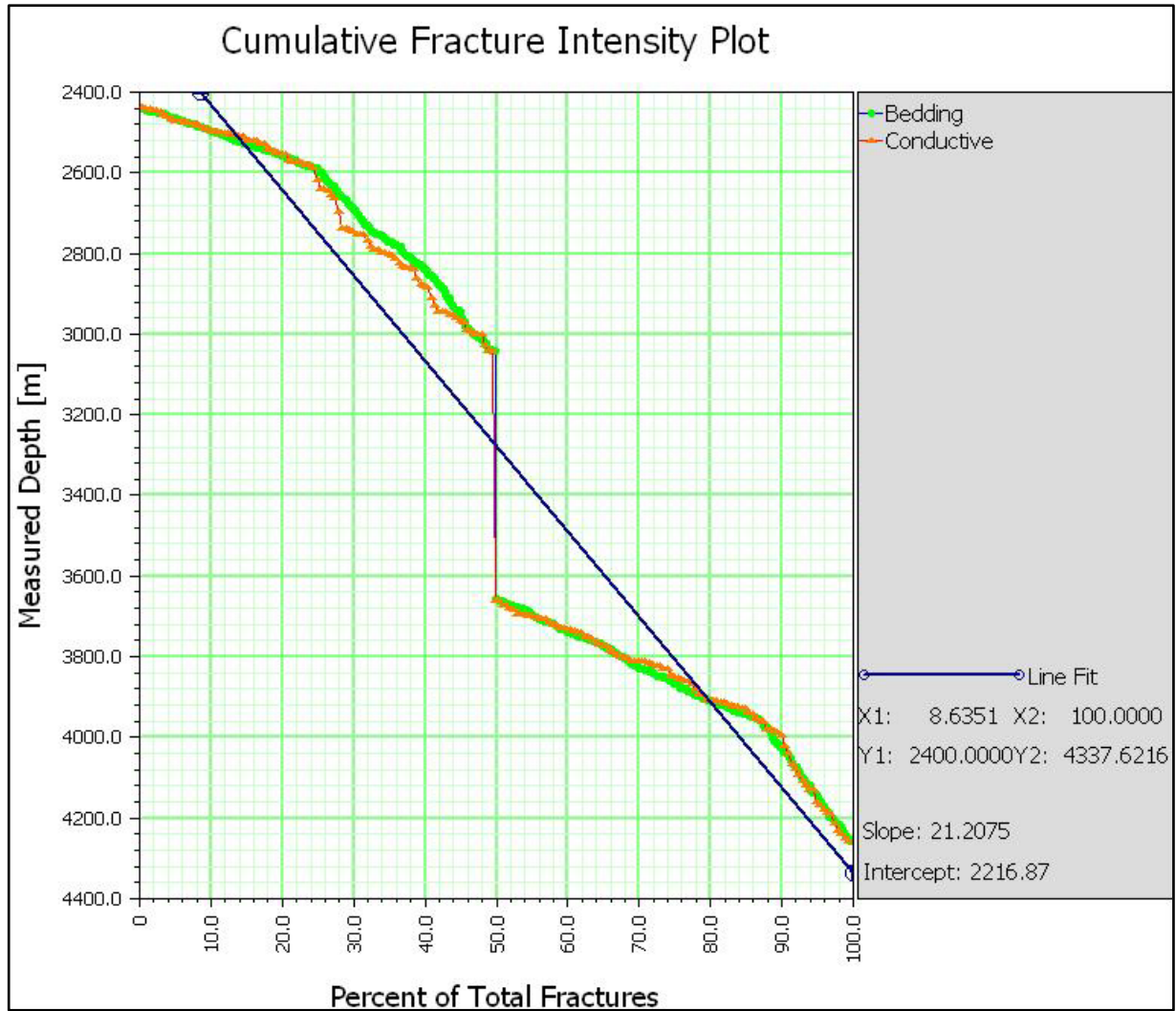


Figure 22: Example of Cumulative Frequency Intensity (CFI) Plot produced by FracMan software (Golder Associates Ltd, 2006).

The P32 can be inferred using a simulated sampling methodology based on the linear correlation with P21 (i.e., the total trace length of fractures per unit area), or with P10 (i.e. represents the total number of fractures along a scanline or borehole) using the following equations (Elmo, et al., 2015):

$$P32 = C32 \times P21$$

$$P32 = C31 \times P10$$

Where, C32 and C31 are constants of proportionality which depend on the relative orientation of the fractures to the orientation of the sampling panel or scanline/borehole. Furthermore, the volumetric fracture intensity is related to the fracture radius, therefore truncation biases would play a significant role in determining the appropriate fracture intensity for the DFN analysis (Elmo, et al., 2015).

2.2.5 Fracture Termination

Fracture terminations provide useful data on the structural character of the rock mass as it relates to fracture connectivity and block formation (i.e., controls the amount of rock bridges between adjacent

fractures) (Elmo, et al., 2007). Fractures could terminate within the intact rock, at another joint and in rock across another joint. This parameter is often used as a calibration tool whereby mapped terminations on exposed surfaces are compared to those generated in equivalent surfaces within the DFN models, as well as for fracture length analysis. The “T-type” termination, shown in Figure 23, is considered important as it relates to the order in which fracture sets occurred in nature. Other termination types do not link to the structural character of the rock mass but rather indicates the censoring bias introduced by the sampling method (Elmo, et al., 2015).

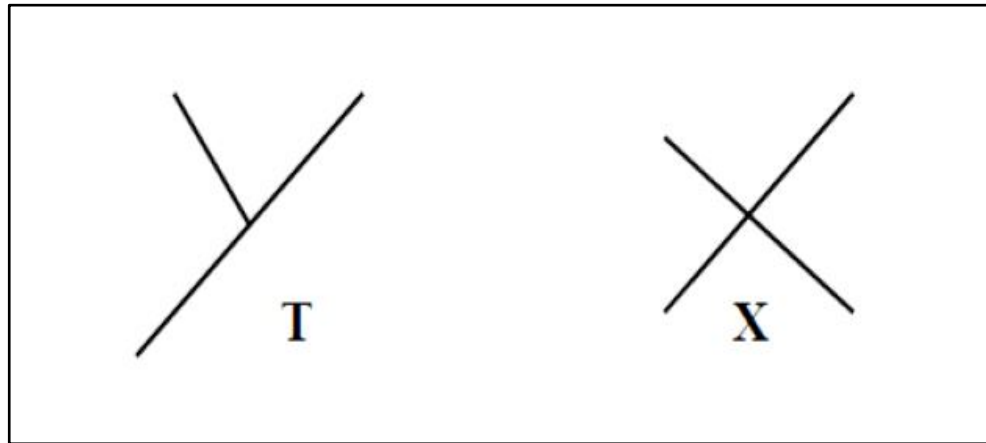


Figure 23: Fracture termination types (Elmo, et al., 2015)

In FracMan, the fracture termination is expressed as a termination percentage which refers to the T-type fractures observed from all observed ends. Fracture networks that contain a higher percentage of T-type fracture terminations generally have a greater potential to form blocks with fewer intact rock bridges retaining the block to the adjacent rock mass (Rogers, et al., 2007).

Modelling of the discontinuity ordering and termination are used to develop hierarchical rock mass models whereby the rock fracture network would strive to satisfy the structural geological history of the rock. This is typically used in the petroleum industry and well as with caving studies (Brown, 2002).

2.3 Overview of the Slope Design Process

The open pit design process is aimed at determining safe and economical slope configurations at bench, inter-ramp and overall slope scales as shown in Figure 24. In the mining and civil engineering industries, the slope design process is relatively standard and typically involves the steps summarised in Figure 25. Each of these steps and models in the slope design process have their own considerations and methodologies; the details of which are well documented in several sources such as (Read & Stacey, 2009), (Wyllie & Mah, 2004), (Kliche, 1999), and (Hoek & Bray, 1981).

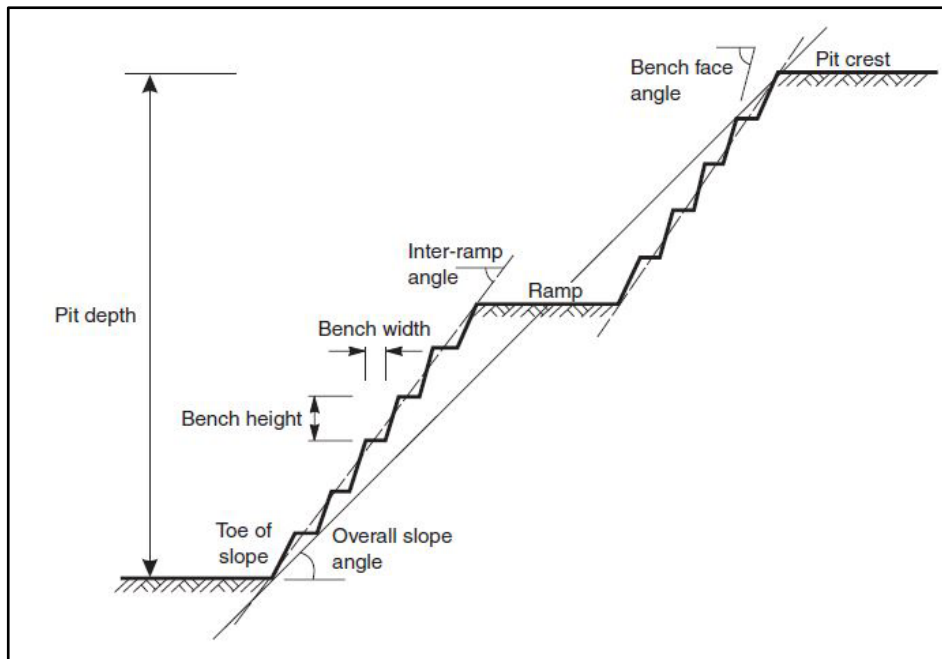


Figure 24: Typical Open Pit Cross-Section (Wyllie & Mah, 2004).

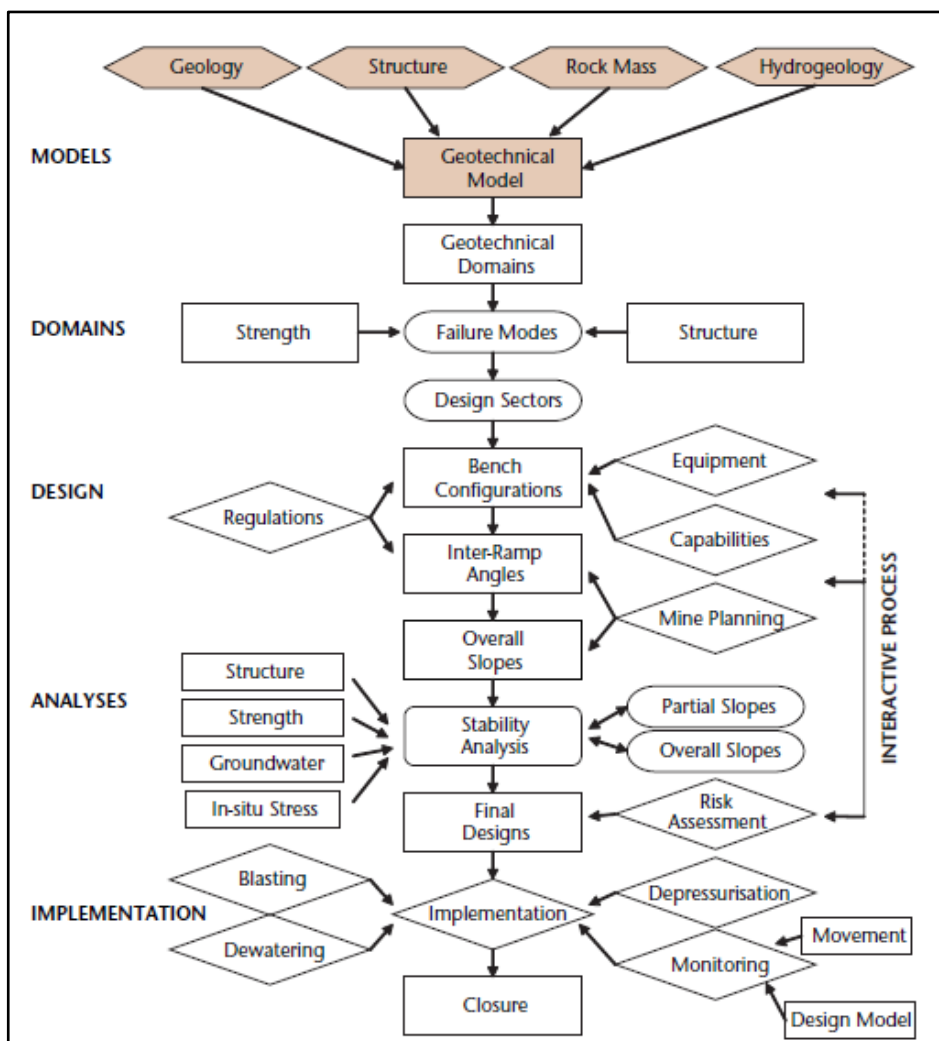


Figure 25: Slope Design Process (Stacey, 2009).

As illustrated in Figure 25, the geotechnical model forms the basis of the slope design and is derived from component models of material properties (rock mass), geology, structure, and hydrogeology. These component models provide an understanding of the following, (Stacey, 2009):

- Material distribution in the pit walls that includes the lithology (ore deposit and waste rock), alteration, weathering grade;
- Rock mass structures at mine-scale, inter-ramp and bench-scale and their properties;
- Rock mass strength;
- Stability controlling factors (e.g., structure, weak materials);
- Groundwater pressure and surface water flow aspects of the hydrogeological regimes that influence slope stability.

The geotechnical model developed is used to evaluate the stability of rock slopes at various scales illustrated in Figure 26. The slope stability methods selected (e.g., empirical, limit equilibrium, numerical and probabilistic methods) are governed by the expected modes of failure, scale of the slope, available data and uncertainty, and level of acceptable risk indicated by the factor of safety or probability of failure. This research focuses on bench scale stability analyses in hard rock mining environments.

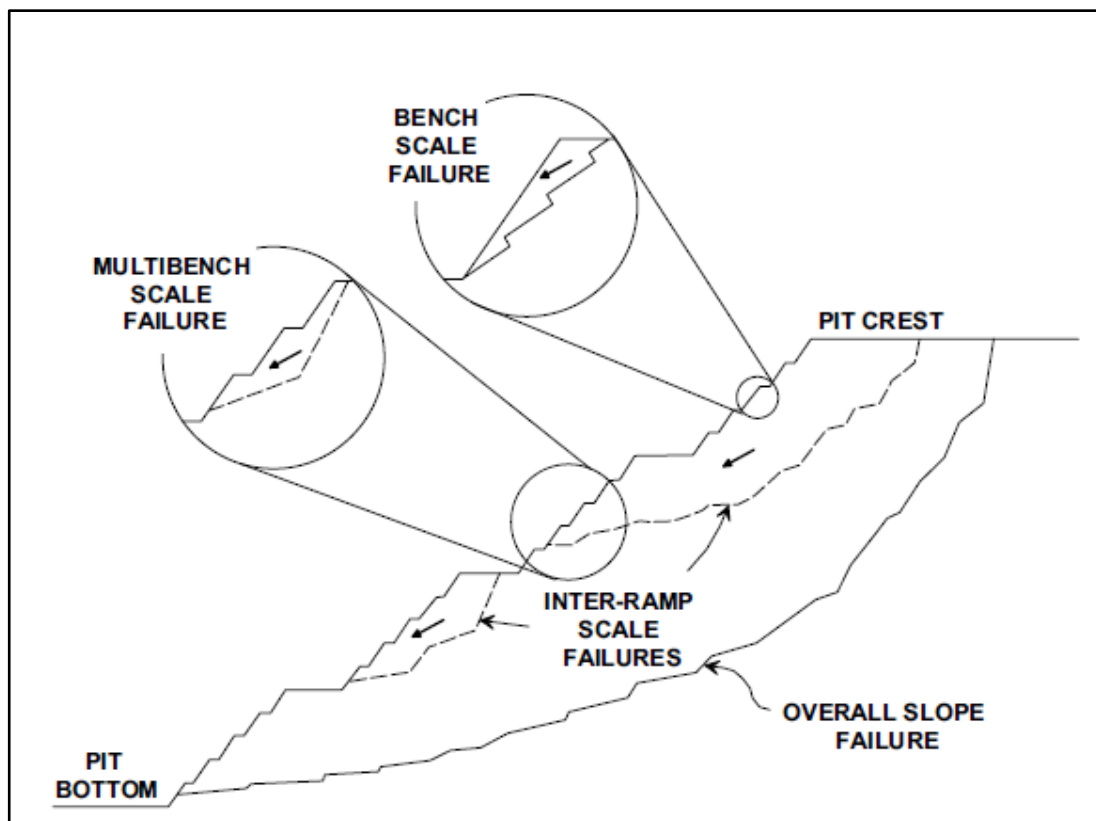


Figure 26: Scope and Scale of Failures (Hawley, et al., 2009).

2.3.1 Bench Design

The bench design process involves completing a stability analysis to establish minable bench face angles, adequate bench widths and, to a limited extent selecting the bench height. The bench geometry shown in Figure 27 defines the steepest inter-ramp slope that can be mined whilst maintaining adequate catch bench widths (Ryan & Pryor, 2001). The primary function of a bench in an open pit mine is to provide a safe working environment for personnel and equipment. This requires stable bench face angles and bench widths capable of mitigating rock failures and containing spillages from the benches above (Lorig, et al., 2009).

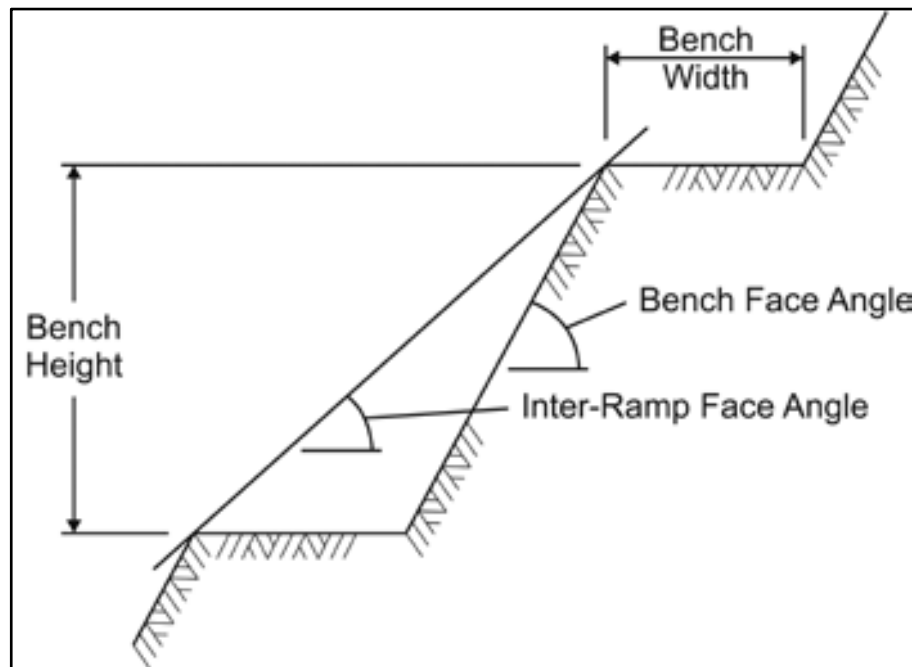


Figure 27: Bench Configuration (Lorig, et al., 2009).

Bench heights are selected based on the excavation methods and mining equipment used. Double benching is commonly practised, improving production, and typically involves stacking of two single benches with a larger catch bench width (Lorig, et al., 2009).

The design of the bench face angles generally follows the process summarised in Figure 28 and is controlled principally by rock mass structures and their properties. Therefore, the structural data available and methods used to characterise these rock mass structures define conditions that would directly influence bench stability. The kinematic analyses based on stereographic projection methods and limit equilibrium techniques are typically used to assess planar sliding and wedge failures that are considered fundamental to bench design. Further discussion on these analyses is provided in Section 2.4.

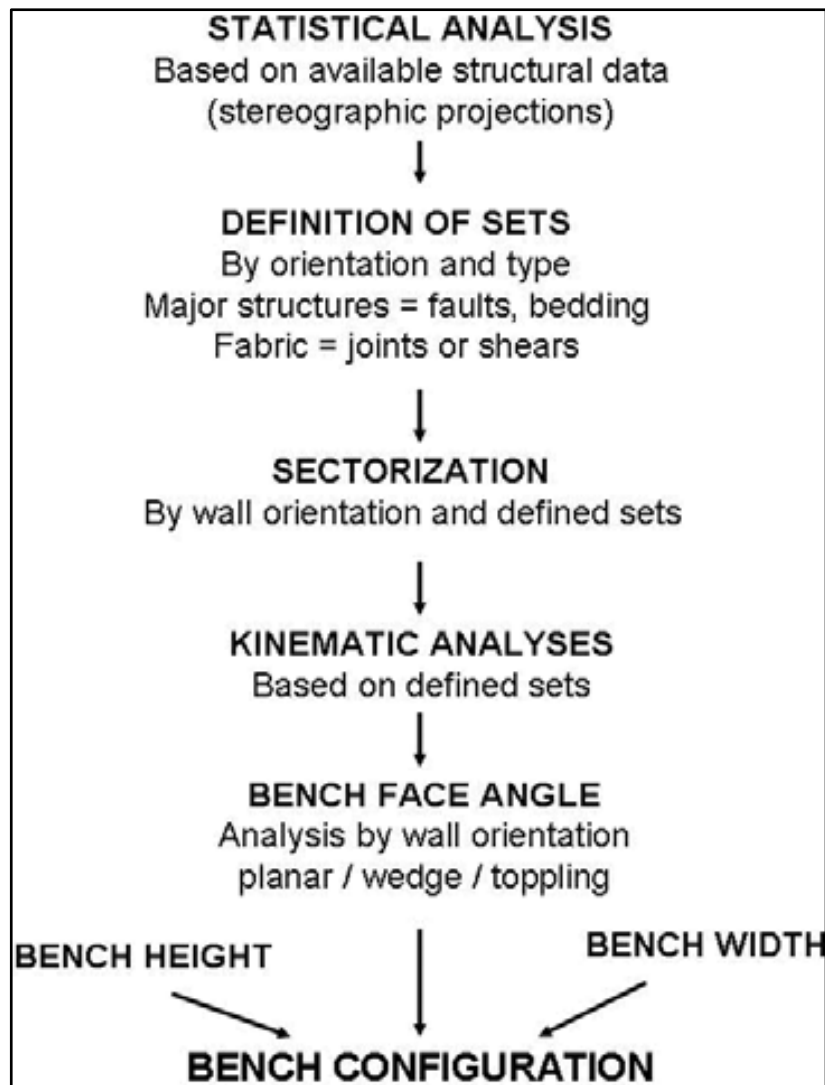


Figure 28: General bench face design process for moderate to strong rock (Lorig, et al., 2009).

The kinematic analysis can also be accompanied by various probabilistic methods in order to determine bench design parameters. The Cumulative Frequency Analyses (CFA) is an example of such a method that is commonly used for bench design (Carvalho, 2002).

Bench widths are required to be wide enough to mitigate against hazardous rockfalls and contain any spillage material from the benches above (Lorig, et al., 2009). The Modified Richie Criteria, developed by Call & Nicholas Inc, was derived from the original work published by (Ritchie, 1963). (Ritchie, 1963), developed width and depth criteria for a ditch at the toe of a slope to protect highways. It was based on the understanding that the falling rocks can roll considerable distances from the toe of the slope because of horizontal momentum and spin, and would impact with the side of the ditch, effectively stopping the horizontal rolling (Call, 1992). Given that the bench height is an important control on the distance a rock will travel when detached from a bench, (Nicholas & Sims, 2001) [in (Ryan & Pryor, 2001)] derived the following empirical relationship between the bench slope height and the average or preferred catch bench width:

$$\text{Bench Width (m)} = 0.2 \times \text{Bench Height} + 4.5\text{m}$$

The Modified Ritchie criterion has been shown to be conservative but the most commonly used criterion as field tests indicate that it is effective in benched mine slopes. According to (Ryan & Pryor, 2001), other researchers have developed rock catchment criteria from both field tests and computer simulation methods; however, due to the complexity of this problem, any one criterion cannot be considered as 100% effective.

The above-mentioned Modified Ritchie rockfall criterion does not consider failure volumes from planar sliding and wedge failures. (Haines, et al., 2006) derived the following formula that can be used to calculate the bench width based on individual wedge failure volume:

$$\text{Bench Width (m)} = \sqrt[3]{\text{Failure Volumes} \times \text{Bulking Factor}}$$

Where, the bulking factor is typically 1.5.

(Gibson, et al., 2006) presented further methods to calculate the volume of the failed material with a conical or pyramidal geometry. (Jermy, et al., 2011) provide additional details on the assumptions and limitations of these common methods used to estimate bench widths in open pit mines. Other methods for estimating bench widths and/or berm widths are described in (Piteau & Martin, 1977)/ (Martin & Piteau, 1977), and (Pierson, et al., 1994), (Hustrulid, et al., 2001), (Holley, et al., 2006) and (Gibson & Paul, 2016). It should be noted that for practical and economical purposes, bench widths are typically expected to retain 70 to 85 % of failed material volumes from sliding plane or wedge failures (Lorig, et al., 2009).

The calculated bench widths cannot always be achieved or maintained in practice due to back break. (Ryan & Pryor, 2001) defines back break as the horizontal distance between the planned toe and actual mined crest of the final bench slope as shown in Figure 29. The amount of back break and the effective bench face angle are controlled predominantly by geological structures and the quality of blasting practices.

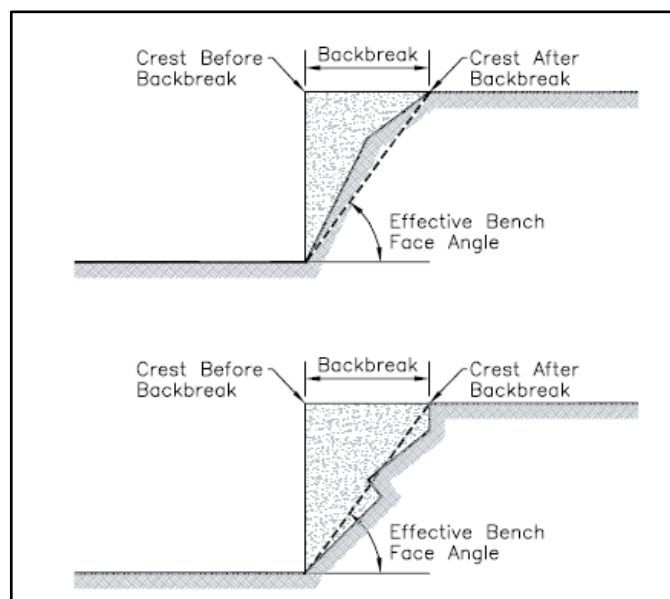


Figure 29: Definition of back break and effective bench face angle (Ryan & Pryor, 2001).

2.4 Kinematic Slope Stability Analysis

This section summarises the main kinematic modes of failures in rock slopes and the related limit equilibrium stability analyses. The limit equilibrium analysis is carried out for wedge type failure mechanisms identified using conventional methods and the discrete fracture modelling approach (i.e. FracMan Rock Wedge Stability Analyses). More advanced numerical methods involving continuum (e.g. finite element and finite difference methods), discontinuum (e.g. discrete element, discontinuous deformation analysis) or hybrid modelling that are also used to evaluate overall stability of rock slopes with perhaps more complex geometries and expected behaviour, are beyond the scope of this project.

2.4.1 Kinematic Modes of Failure

The term “kinematics”, in relation to rock slopes, refers to the motion of rock blocks without reference to the forces that results in their movement. The kinematic analysis identifies potential geometrically unstable blocks in the rock slopes by primarily considering the orientation of weakness planes or discontinuities with respect to the orientation of the rock slope, with minimal reference to their strength properties (Goodman, 1989). Kinematic analysis is typically performed using stereographic projection techniques and is based on a prior evaluation of the rock mass structures.

Structurally controlled failures in hard rock slopes involve one or a combination of the three basic failure mechanisms, namely, planar sliding, wedge, and topping failure. Figure 30 shows the main types of block failures in rock slopes alongside a stereonet with their structural geology conditions likely to cause these failures (Wyllie & Mah, 2004). Circular failure generally occurs in very weak rock or closely fractured rock with randomly oriented discontinuities. (Stacey, 2007), noted that structurally controlled failure mechanisms are more complex than planar, wedge, circular shear or toppling in high, hard rock slopes. These slopes experience progressive failure (i.e. referred to as step-path failure) involving the initiation and progression of failure along existing planes of weakness and intact rock.

(Wyllie & Mah, 2004) based on (Hoek & Bray, 1981), and (Goodman, 1989) provide a comprehensive discussion on the kinematic analysis of planar, wedge and toppling failures. Accordingly, this can be summarised as follows:

- **Planar sliding** occurs when a discontinuity dips out of a rock slope at an angle that is shallower than the inclination of the slope and steeper than the effective angle of friction on the discontinuity. These failures generally only develop to a significant extent if the strike of the geologic discontinuity lies within $\pm 20^\circ$ of the strike direction of the rock slope face.
- **Wedge failure** can occur when two or more discontinuities intersect to form a wedge and the line of intersection of this wedge dips out of the slope at an inclination that is shallower than the inclination of the slope, but steeper than the effective angle of friction along the discontinuities. Wedge failures will only develop to a significant extent if the azimuth of the line of intersection lies within $\pm 45^\circ$ of the dip direction of the rock slope face.

- **Toppling failures** may develop when a rock mass contains multiple, parallel, steeply dipping continuous discontinuities, such as bedding planes, that strike nearly parallel to the strike of the face of the rock slope, and that dip into the face. The strike of the structures is typically within $\pm 10^\circ$ of the strike of the rock slope face. The potential for toppling failure is governed by the spacing, strength, inclination and continuity of the discontinuity sets forming the toppling blocks, the stiffness and strength of the intact material, as well as the rock slope angle. Two main types of toppling failure are distinguished due to their related distinct stability analyses required, namely, block toppling and flexural toppling. Block toppling refers to forward rotation and eventual toppling of distinct columns of rock separated by closely spaced steeply dipping joints, usually with well-defined basal (i.e., shallow dip) discontinuities. Flexural toppling occurs when a weak and fissile rock mass possessing a steeply dipping and closely spaced persistent weakness structure (e.g., banding, foliation, schistosity) bends and eventually may result in overturning of rock debris. The many different types of toppling failure are generally variations between these two main types of toppling failures.

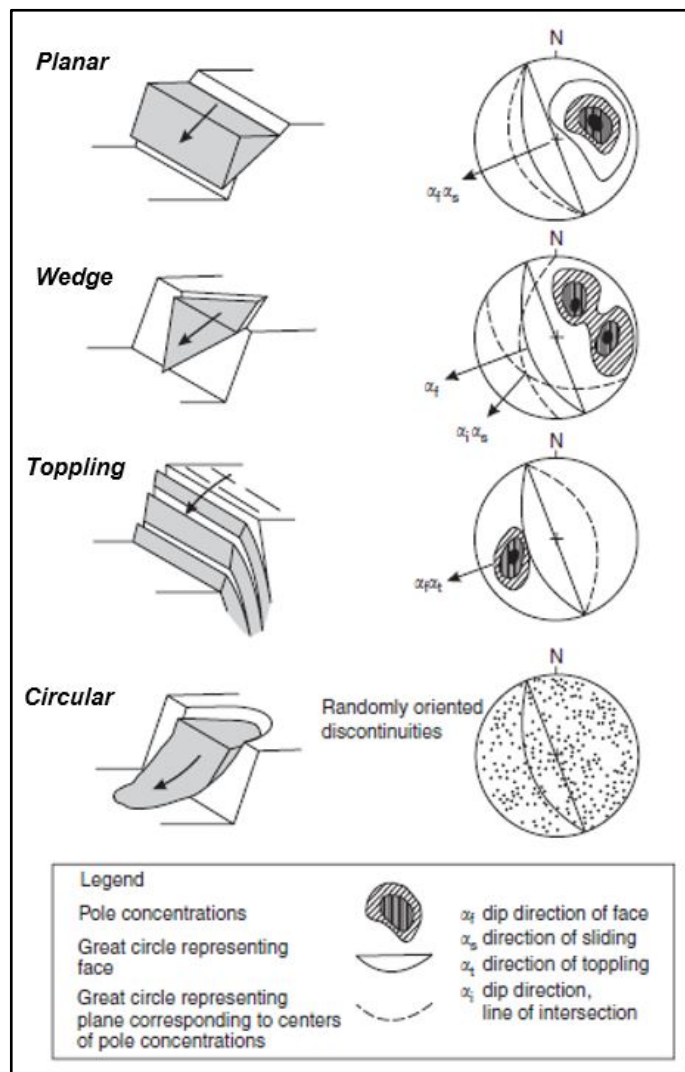


Figure 30: Main types of failures in slopes with associated structural geology condition, [Modified from (Wyllie & Mah, 2004)].

2.4.2 Kinematic Stereographic Projection Method

Structural data makes use of stereographic projections that allow three-dimensional orientation data to be represented and analysed in two dimensions by means of lines and points that signify planes and poles accordingly. Stereographic projections are limited in that they consider only the angular relationships between lines and planes and do not represent the sizes and positions of the structures.

Kinematic analyses using stereographic analyses typically makes use of equal-area, lower hemisphere projections. A detailed evaluation of the structural data is carried out in order to determine discontinuity sets that are representative of the structural fabric of the rock mass. These selected discontinuity sets are used for the kinematic analyses. Section 2.1 provides some detail on discontinuity set selection methods available.

The considerations for the main kinematic failure mechanisms were described in Section 2.4.1 above. Figure 31 summarises the kinematic stereographic method. This method uses failure envelopes that are generated on stereonet considering rock face slope angles and friction cones according to Markland's test, described in (Hoek & Bray, 1981) for the planar and wedge failure, and the work of (Goodman, 1980) for toppling failure. The potential for planar and toppling failure is determined by considering the number of poles that occur within the corresponding failure envelopes. Wedge failure potential is determined by the number of plane intersections or the intersection poles that occur within the defined failure envelope. The larger the number of poles, the higher the potential for the failure mechanism will be. An example of a kinematic analyses of wedge failure using ©Rocscience Dips is detailed in Section 2.4.2.1 below.

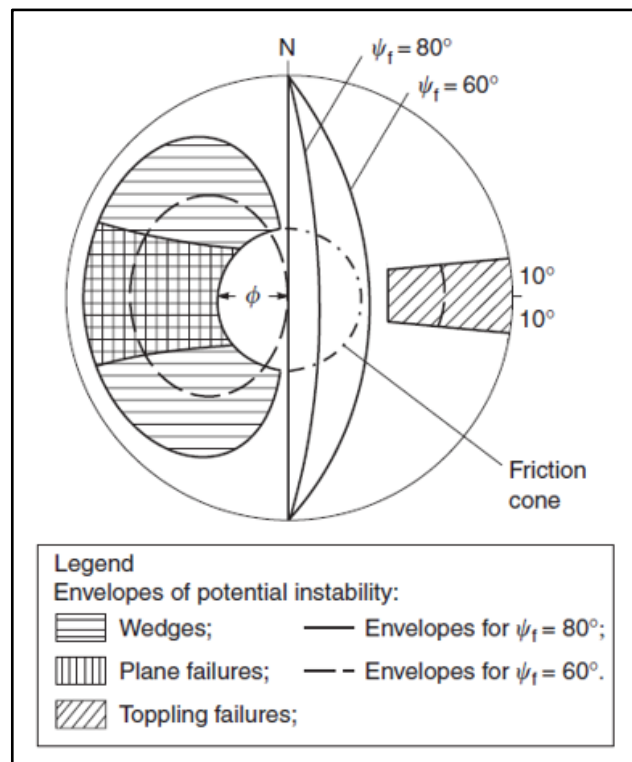


Figure 31: Stereographic projection showing example of combined kinematic analyses (Wyllie & Mah, 2004).

2.4.2.1 DIPS Wedge Failure Analyses

The kinematic sliding conditions described above in Section 2.4.1 have been incorporated into the ©Rocscience DIPS program. DIPS use the analyses of intersections to identify wedges formed and its sliding potential. The dip vectors are considered to determine if sliding would occur on two planes or one plane. The intersection of two planes forms a line in three-dimensional space that is represented as a point on a stereonet; this line is referred to as an intersection line and its orientation denoted by the trend/plunge. If an intersection point satisfies the frictional and kinematic conditions for sliding, it occurs within the critical zone. These critical intersections pose a risk of sliding. Figure 32 illustrates the wedge sliding analyses in DIPS. Two critical zones are distinguished; primary and secondary critical zones. Primary critical zone (shaded in red) is within the plane friction cone and outside the slope plane and intersections that fall within this zone are indicative of wedges that can slide on either two planes or one plane. The secondary critical zone (shaded in yellow) indicates the occurrence of oblique wedges (i.e., intersections are inclined at angles less than the friction angle) that can slide on only one discontinuity plane if the dip vector is greater than the friction angle. Friction cone defines the limits of frictional stability on a stereonet; wedge sliding analysis uses the plane friction cone (Rocscience Inc, 2017).

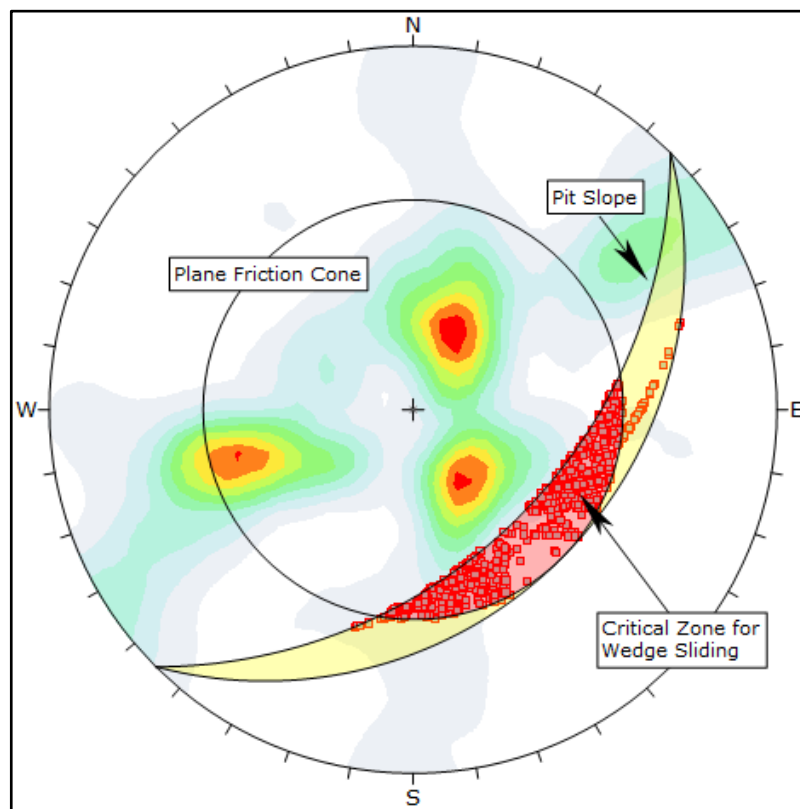


Figure 32: DIPS kinematic analysis of wedge sliding (Rocscience Inc, 2017).

Wedge results include the number of intersections counted within the critical zone and expressed as a percentage. This percentage provides an estimate of the “probability of failure” with respect to the total number of intersections for the selected intersection type (i.e., all set planes, mean and user planes, and all data planes gridded). Generally, critical zones of intersections overlapping with a maximum contour concentration indicate a significant potential risk of failure (Rocscience Inc, 2017).

2.4.3 Limit Equilibrium Analyses

The limit equilibrium analyses for wedges and planes calculates the factor of safety by considering force equilibrium in the direction of sliding. The factor of safety is defined as (Rocscience, 2018):

$$\text{Factor of safety} = \frac{\text{resisting forces (e. shear/tensile strength, support)}}{\text{driving forces (e. g. weight, seismic, water)}}$$

According to (Lorig, et al., 2009), suitable software packages available for limit equilibrium planar sliding and wedge solutions include; SBlock, Swedge, RocPlane, and Siromodel. The stability analyses using limit equilibrium methods in SWedge can be deterministic (i.e., input parameters are known “exactly”, and a single factor of safety is calculated), probabilistic (i.e. input parameters are defined by statistical distributions which produces a safety factor distribution from which a probability of failure is calculated), a combination (i.e. allows entry of a number of discrete joint plane orientations and all possible combinations of two joint which can form a wedge can be analysed), or sensitivity based (i.e. the effect of uncertainty and variability of input values parameters can be explored (Rocscience, 2021). An example of a limit equilibrium solution provided in (Wyllie & Mah, 2004) for a wedge geometry is illustrated in Figure 33.

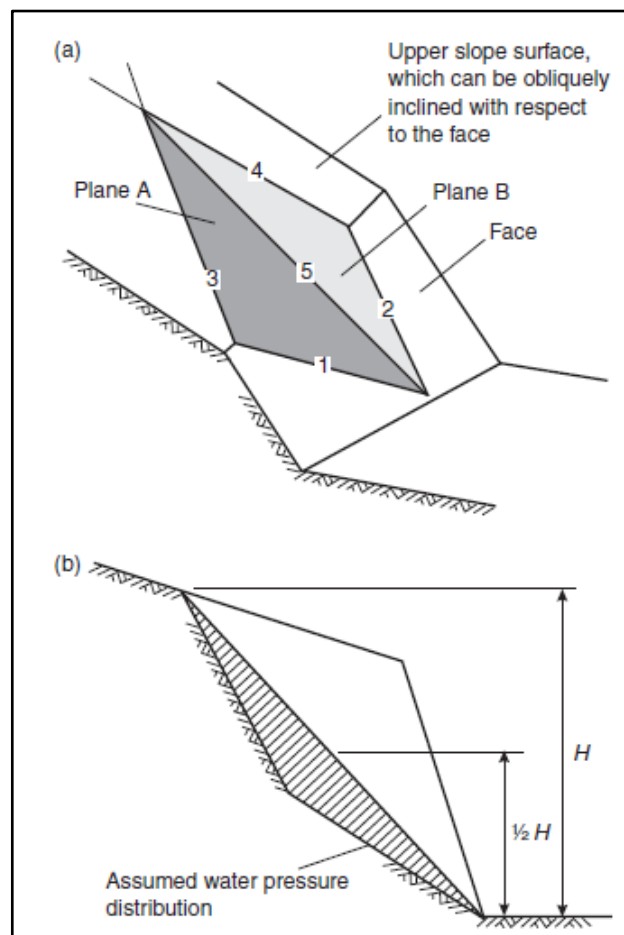


Figure 33: An example of wedge limit equilibrium stability analyses: (a) graphic view of wedge showing the numbering of the intersection lines and planes; (b) view normal to the line of intersection (5) showing wedge height and water pressure distribution (Wyllie & Mah, 2004).

2.4.4 FracMan Rock Wedge Stability Analyses

FracMan software developed by Golder Associates utilises an explicit block search algorithm, referred to as “Rock Wedge Analysis”, to evaluate the rock block stability of surface or underground excavations (FracMan Technology Group , 2016). This allows FracMan to identify 3D blocks defined by the intersection of the underlying DFN model with a specified free surface (e.g., a rock slope face). This involves computing trace maps for the DFN-Slope surface fracture intersections and identifying two dimensional closed blocks in the trace maps. This results in a collection of faces and connections that are processed using an “unfolding algorithm” to create the minimum volume of polyhedron that connects the specified free surface; then, the volumes are calculated based on a 3D tessellation process (i.e. building the blocks by compiling defined trace maps with no overlaps and no gaps) and the assigned rock density (Elmo, et al., 2014).

The stability analysis in FracMan is based on (Goodman & Shi, 1985) block theory. The factor of safety depends on the failure mode of the block and considers cases of free-falling blocks (i.e., factor of safety that is zero), stable blocks, and translational sliding on one or more planes. Stable blocks are assigned a FOS of 100 as these have an effectively infinite factor of safety whereas freefalling blocks have an FOS of zero. Between these extremes, we have the cases of one-sliding plane, two-plane sliding or three-plus sliding surfaces that are not well defined. The FOS for the one and two-plane sliding failure modes can be calculated with either Mohr-Coloumb or Barton-Bandis shear strength criteria (Golder Associates, 2006). Table 3 provides a summary of the FOS categories associated with the FracMan rock wedge analysis.

Table 3: FracMan Rock Wedge FOS Categories

Factor of Safety (FOS)	Failure Mode Description
FOS = 100	Kinematically Inadmissible
FOS > 1.0	Stable Blocks
FOS < 1.0	Unstable Blocks
FOS = 0	Freefall Blocks

The DFN method allows for the generation of multiple statistically possible realizations that allow for a better understanding of the frequency of occurrence for blocks of a certain size or factor of safety. The rock wedge algorithm provides a more accurate block shape and volume, that is more appropriate for the kinematic assessment of block stability.

2.5 Chapter Summary

It is customary with every research study to review the technical literature related to the subject matter and as such, this chapter provides summaries and highlights of the relevant background information useful to readers not familiar with the discipline.

Rock mass structures control the stability of rock slopes at bench-scale. Understanding these structures and their properties, data collection methods and modelling approaches is essential to subsequent stability assessments. The structural data collected for this study involved 1D scanline mapping and 2D digital window mapping techniques using photogrammetry. As is the case with most structural data collection methods, the data measured are subjected to measurement errors, orientation bias, size bias, truncation bias as well as censoring bias. It is therefore imperative to reduce or correct for the bias and errors wherever possible (e.g., appropriate selection of methodologies and sampling sites can greatly reduce errors and bias). Similarly, the structural modelling approaches such as solid modelling, stereographic projection methods and discrete fracture networks, are susceptible to: the innate spatial variability of geological formations, modelling physical behaviour inaccuracies, and errors introduced in the measurement and estimation of engineering properties.

This research is primarily focused on the use of discrete fracture networks and its application to the design of open pit benches. The discrete fracture network modelling approach is a method that generates three-dimensional stochastic representations of rock mass structures from statistical distributions that describe their characteristics (e.g., orientation, spacing, size, aperture). The stochastic nature of the DFN modelling process is such that there are an infinite number of possible realizations of the 3D fracture system based on the sampled field data. The generation of a DFN model involves selection of an appropriate fracture spatial model, statistical assessment of sample data for the derivation and/or inference of fracture size, orientation, and intensity.

Bench design comprises determination of the bench face angle, bench width and selection of the bench height that are structurally controlled by kinematic failure mechanisms as planar sliding, wedge sliding and toppling. This study specifically considers wedge sliding in relation to bench design. The traditional bench design approach relating to the assessment of wedges, generally comprises of a stereographic kinematic assessment followed by a limit equilibrium stability analysis using SWedge. This traditional method serves as a form of comparison for the DFN approach to rock wedge stability assessment for the purpose of determining bench design parameters. FracMan was the software package used to generate the DFN and complete the related limit equilibrium rock wedge stability assessment. This “rock wedge analysis” in the FracMan program is an explicit block search algorithm that is used to evaluate the rock block stability of surface or underground excavations and is based on block theory. It results in a higher accuracy of block shape and volume, which is considered more appropriate for the kinematic assessment of block stability.

CHAPTER 3

3.0 MINE X CASE STUDY: SITE DESCRIPTION

This chapter describes the mine selected for this study and the field data collection carried out. The name of the mine has been omitted due to a confidentiality agreement and will consequently be referred to herein as Mine X. Figure 34 shows Mine X, an open pit pyroxenite mine located within the Ba-Phalaborwa local municipality in the Limpopo province of South Africa. Mine X at the time of the investigation was approximately 200m deep and extended to 1.5 km long in the north-south direction and 850m wide in the east-west direction.



Figure 34: Location of Mine X Open Pit in South Africa (Source: Google Earth 2010)

Phalaborwa is situated in the Lowveld, approximately 400 m above sea level, with gently undulating topography and occasional conical shaped hills. The region forms part of the watershed of the Letaba and Olifants rivers as well as part of the dryer northern sub-region of the Mopane-bushveld, close to the Kruger National Park of the Savanna Bushveld (Golder Associates, 2010).

The climate is predominantly subtropical, with Phalaborwa receiving approximately 450 mm of rain annually. Evaporation during the summer months is very high, with a total evaporation averaging approximately 1 550 mm annually. The prevailing winds are north-east throughout the year, with easterly winds during the summer months and south-westerly winds during winter (Golder Associates, 2010).

3.1 Geological Overview

3.1.1 Regional Geology

The study area forms part of the Phalaborwa Igneous Complex (PIC) which is considered the most important carbonatite complex in South Africa (Roux, et al., 1989). According to (PMC Mine Geological and Mineralogical Staff, 1976), this igneous complex is a vertical pipe that covers an elongated, kidney-shaped area extending roughly 6.5km NS by 2.5km EW in the great lowveld plain of the Limpopo province (i.e. the former Northern Transvaal), (see Figure 35). PIC has resulted from several successive stages of alkaline intrusive cycles that emplaced these variety of rocks that range from ultramafic to peralkaline in character. It has been dated to approximately 2060 Ma using U/Pb isotopes from baddeleyite (Reischmann, 1995).

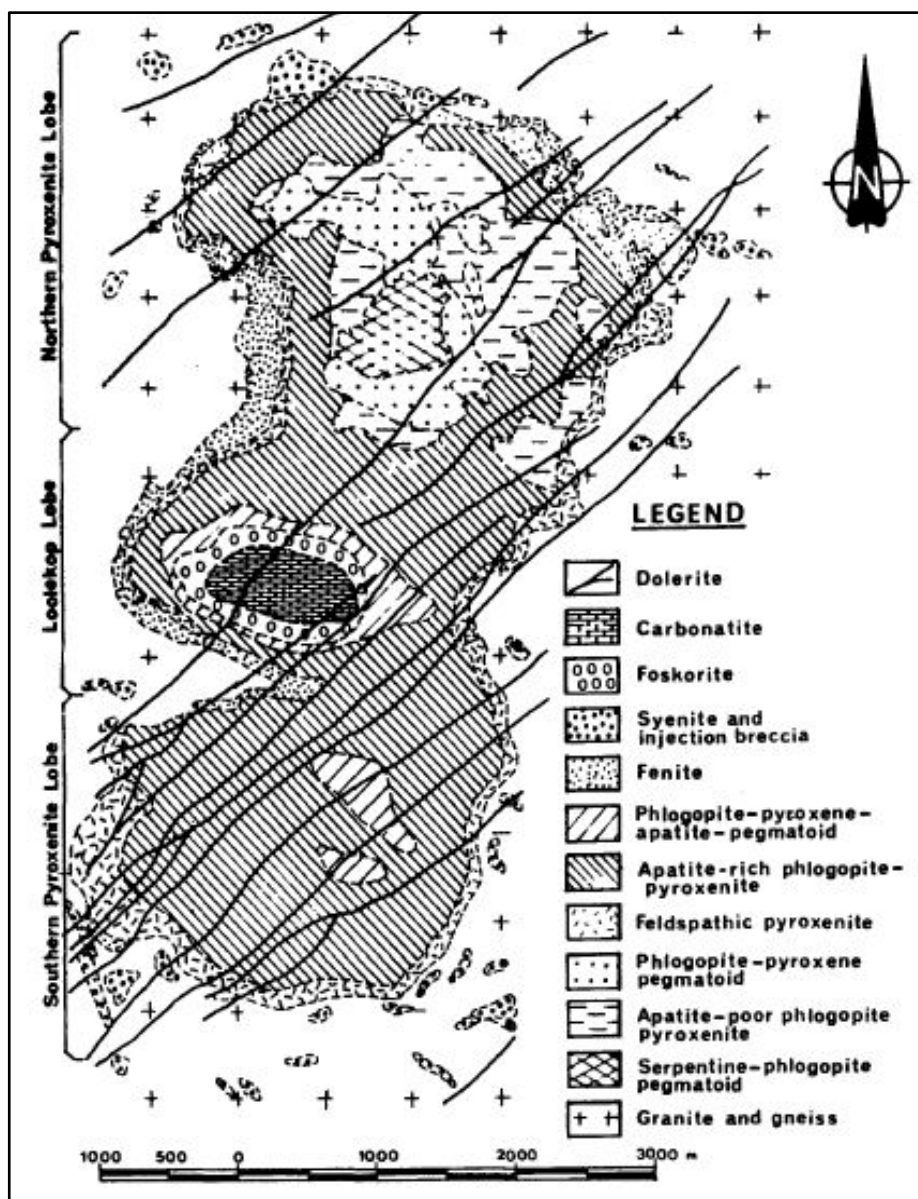


Figure 35: The geology of the Phalaborwa Complex (Roux, et al., 1989).

The geology of Phalaborwa, according to (Southwood & Cairncross, 2017), was documented by many authors over the years with the earliest detailed account provided by (Mellor, 1907) with subsequent interpretations and publications by (Hall, 1912), (du Toit, 1931), (Wilson, 1998), (Shand, 1931), (Russell, et al., 1954), (Forster, 1958), (Lombaard, et al., 1964), (Hanekom, et al., 1965), (PMC Mine Geological and Mineralogical Staff, 1976), (Eriksson, 1982), and (Verwoerd, 1986). The Phalaborwa igneous complex is characterized by Cu-Zr mineralisation that includes predominantly pyroxenites, foskorite and carbonatite. Its geological sequence may be summarised: with the initial intrusion of vertical pipe-like body of many pyroxenites into the Archean gneisses and granites of the Kaapvaal Craton; followed by the intrusion of plug-like syenite bodies into the centres of the periphery of the ultramafic pipe and away from it; a latter lengthy non-violent, metasomatic intrusive stage that resulted in the formation of irregular, vertically disposed ultrabasic pegmatoids at three centres in the pyroxenite pipe, including the emplacement of the foskorite and banded carbonatite in the central pegmatoid body and some fenitization of the surrounding Archean gneisses; and last intrusive phase of younger transgressive carbonatite into the shattered core of the central lobe of the PIC. A series of near-vertical, northeast trending dolerite dykes, (i.e., dolerite swarms), traverses the rock types of the PIC (PMC Mine Geological and Mineralogical Staff, 1976). The three coalescing and concentrically zoned lobes that make up the Phalaborwa igneous complex are referred to as the North Pyroxenite lobe, Loolekop lobe and Southern Pyroxenite lobe as shown in Figure 35. The Loolekop carbonatites and foskorites are economically significant as it hosts the sulphide mineralization, apatite, and titaniferous magnetite. In addition, the ultramafic pegmatoids are a source of commercially important vermiculite (Southwood & Cairncross, 2017).

3.1.2 Local Geology

The local geology around Mine X open pit is shown in Figure 36. The pyroxenite ore consists of three minerals namely, diopside, phlogopite and apatite from which phosphorous rock concentrate is extracted: apatite being the only economic phosphate mineral in the Phalaborwa Igneous Complex. The ore body is bounded by the granite contact on the north and western side of the pit and the apatite-poor pyroxenite, on the eastern side of the pit.

Mine X forms part of the North Pyroxenite lobe of the PIC and consists of five or less concentric zones, in sequence from the centre: Serpentine-phlogopite pegmatoid; Phlogopite-pyroxene pegmatoid; Apatite-poor phlogopite pyroxenite; Apatite-rich phlogopite pyroxenite; and Feldspathic pyroxenite. The inner three zones contain very little or no apatite, whilst the outer two zones contain appreciable quantities of apatite and form the economic ore zone in this lobe. Both the apatite-rich and apatite-poor pyroxenitic zones are heterogeneous and consist of several irregularly distributed subzones. The latter vary in composition from massive pyroxenite through phlogopitic pyroxenite to glimmerite, consisting essentially of phlogopite. Intrusive contacts between the subzones indicate that more than one generation of pyroxenite is present (De Jager, 1990).

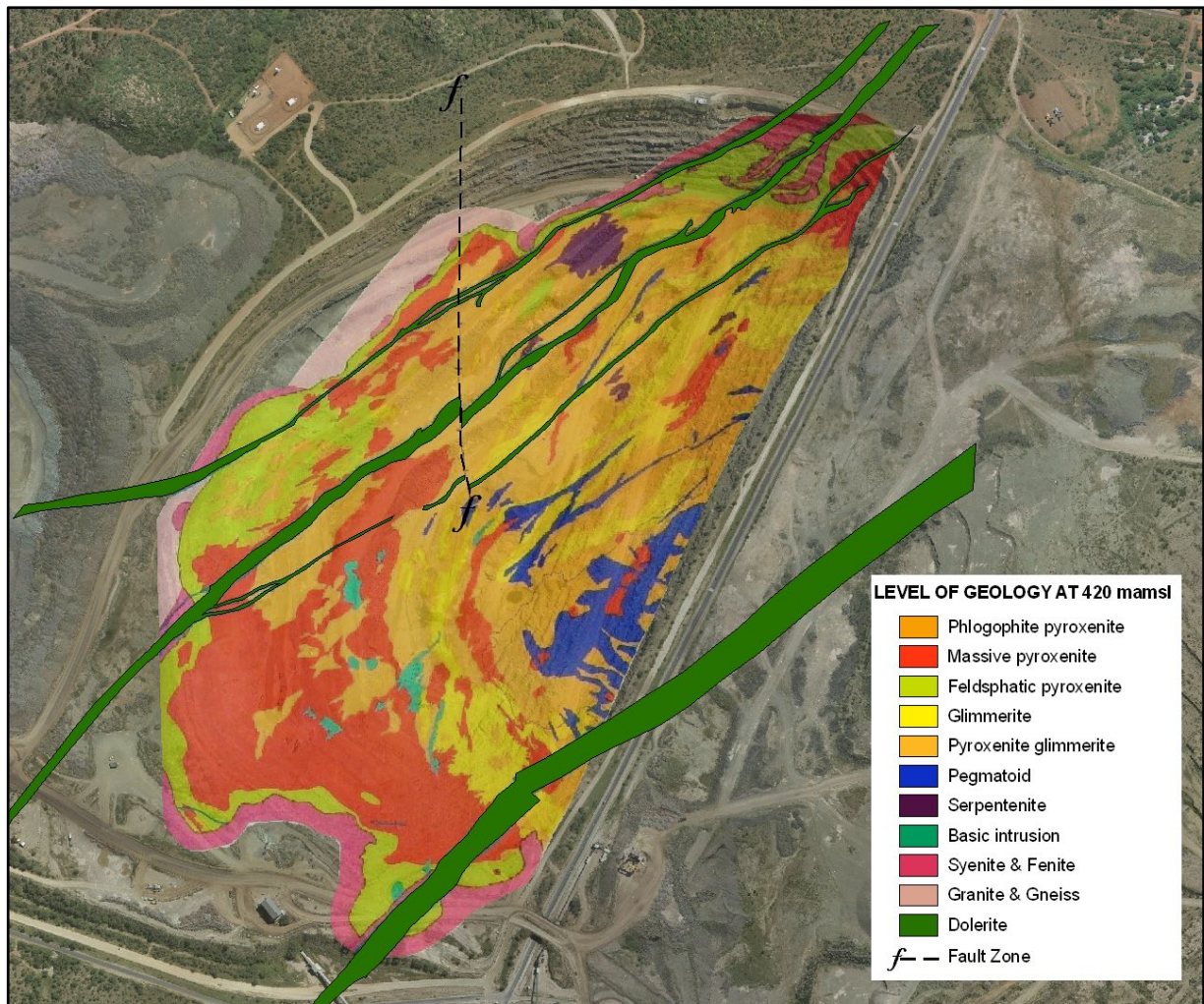


Figure 36: Geological Map of the pyroxenite Mine X (Golder Associates, 2010).

There are three main types of geological structures that traverse the pit, namely dolerite dykes, a fault and shear zones. The steeply dipping dolerite dykes strike in a north-east to south-west direction. The shear zones strike in a north-south direction across the pit. North-south striking faults towards the north west sector of the pit. Figure 37 shows a cross section through the Northern Pyroxenite, illustrating the vertical pipe-like trends of the different pyroxenite types and the steeply dipping dykes.

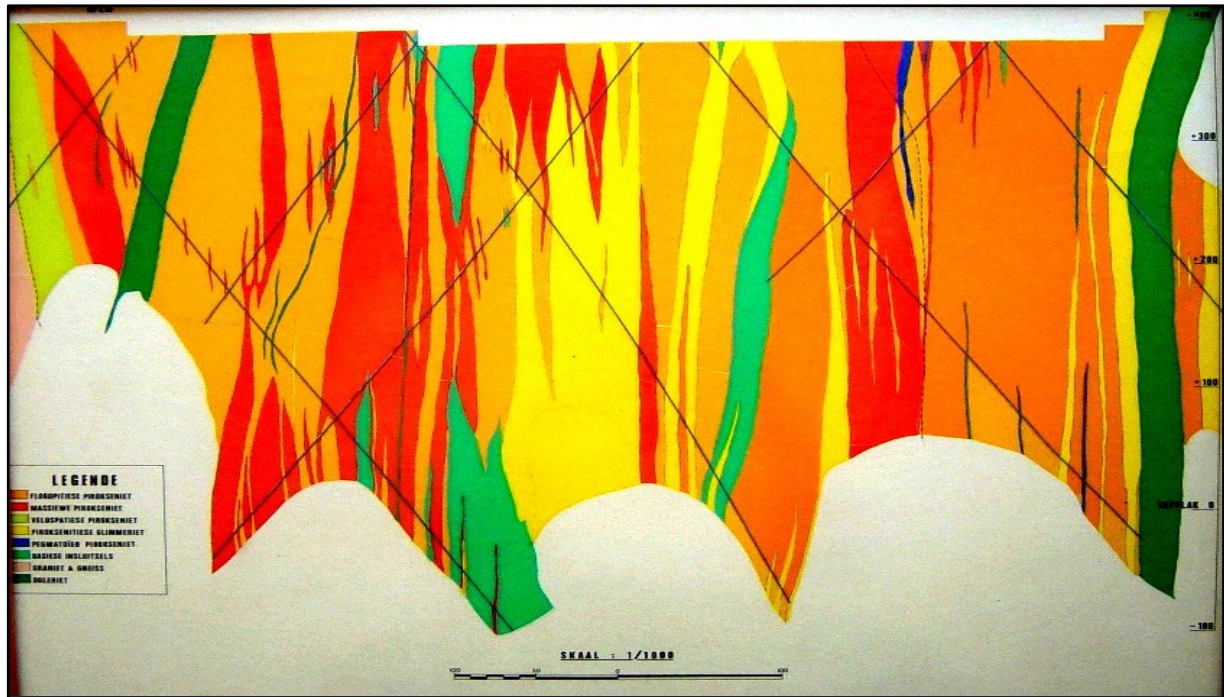


Figure 37: Geological cross-section for Mine X open pit (Golder Associates, 2010).

3.1.3 Hydrogeology

According to (Golder Associates, 2010), the large SW-NE striking dolerite dykes dominate the hydrogeology of the pit. The dykes are permeable at shallow depths, where they are weathered, but become hard and impermeable at greater depths, giving rise to compartmentalization of the Phalaborwa aquifer. The dyke contact zones with the host rock exhibit very little seepage and the fractured areas in the host rock have yields between 0.1 – 0.3 l/s. Approximately 30 l/s of water is being pumped out the sump on a continual basis. The groundwater flow in the study area is towards the pit, but primarily to the south and towards the Selati River.

3.2 Summary of Field Data Collection

The available data from four historical geotechnical projects completed by the researcher were reviewed and a local South African open pit pyroxenite mine, referred to as Mine X, was selected as the case study for this research project. The data available is limited to structural mapping data collected by the researcher from several window and scanline mapping traverses at Mine X. Although the field investigation was completed prior to the commencement of this research project, the review of available data and related re-processing was completed as part of this project in accordance with the objectives outlined. A summary of the relevant data collected and processed for this study is provided in sections 3.2.1 and 3.2.2 below. The data processing and analyses are discussed in subsequent chapters.

3.2.1 Scanline Mapping Surveys

A total of twenty-two scanline mapping surveys were carried out on multiple benches at Mine X open pit. Table 4 provides details of the scanline surveys shown in Figure 38. A total of 935 structural orientation measurements were collected. The scanline surveys recorded the following geotechnical parameters according to standard ISRM methods (ISRM, 1978): discontinuity type (e.g. joint, bedding etc.); persistence; infilling; roughness; joint roughness coefficient (JRC) (Barton, 1987); termination index; aperture; rock type; and groundwater condition. Figure 39 shows an example of the scanline mapping on bench level 11. Figure 40 shows a stereographic contour plot with the poles of all structures mapped during scanline bench mapping exercise. APPENDIX A provides all stereographic projections for the individual scanline mapping surveys.

Table 4: Mine X Scanline Mapping Surveys

Line Survey ID	Bench Level	Bench Height (m)	Bench Face Orientation		Lithology	Scanline Length (m)
			Dip Angle (°)	Dip Direction (°)		
B10-L1	11	12	80	132	Granite	30
B10-L2	11	12	80	151	Granite	30
B10-L3	11	12	80	151	Granite	30
B10-L4	11	12	80	151	Granite	30
B10-L5	11	12	80	151	Granite /Dolerite	30
B10-L6	11	12	80	150	Dolerite	30
B10-L7	11	12	80	150	Dolerite	30
B10-L8	11	12	80	138	Dolerite	30
B10-L9	11	12	80	137	Dolerite	30
B10-L10	11	12	80	117	Granite /Dolerite	30
B10-L11	11	12	80	118	Granite	30
B10-L12	11	12	80	116	Granite	30
B10-L13	11	12	80	093	Granite	30
B12-L14	13	12	80	038	Massive Pyroxenite	30
B12-L15	13	12	80	038	Massive Pyroxenite	30
B12-L16	13	12	80	038	Massive Pyroxenite	30
B12-L17	13	12	80	038	Massive Pyroxenite	30
B15-L18	16	15	80	290	Feldspathic Pyroxenite	30
B15-L19	16	15	80	290	Massive Pyroxenite	20
B10-L20	11	12	80	150	Dolerite	30
B10-L21	11	12	80	150	Dolerite /Granite	30
B10-L22	11	12	80	294	Massive Pyroxenite	30

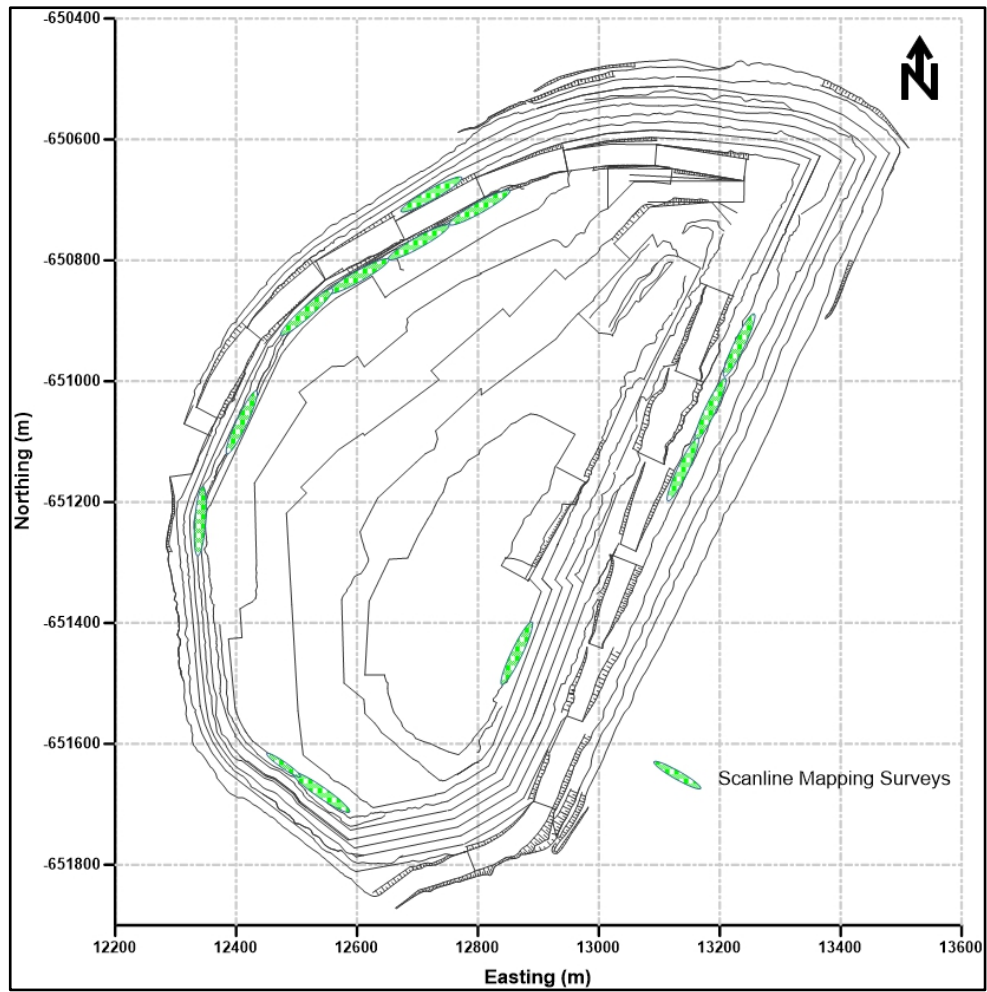


Figure 38: Scanline mapping surveys carried out at Mine X open pit.



Figure 39: Example of bench face mapped using scanline survey method.

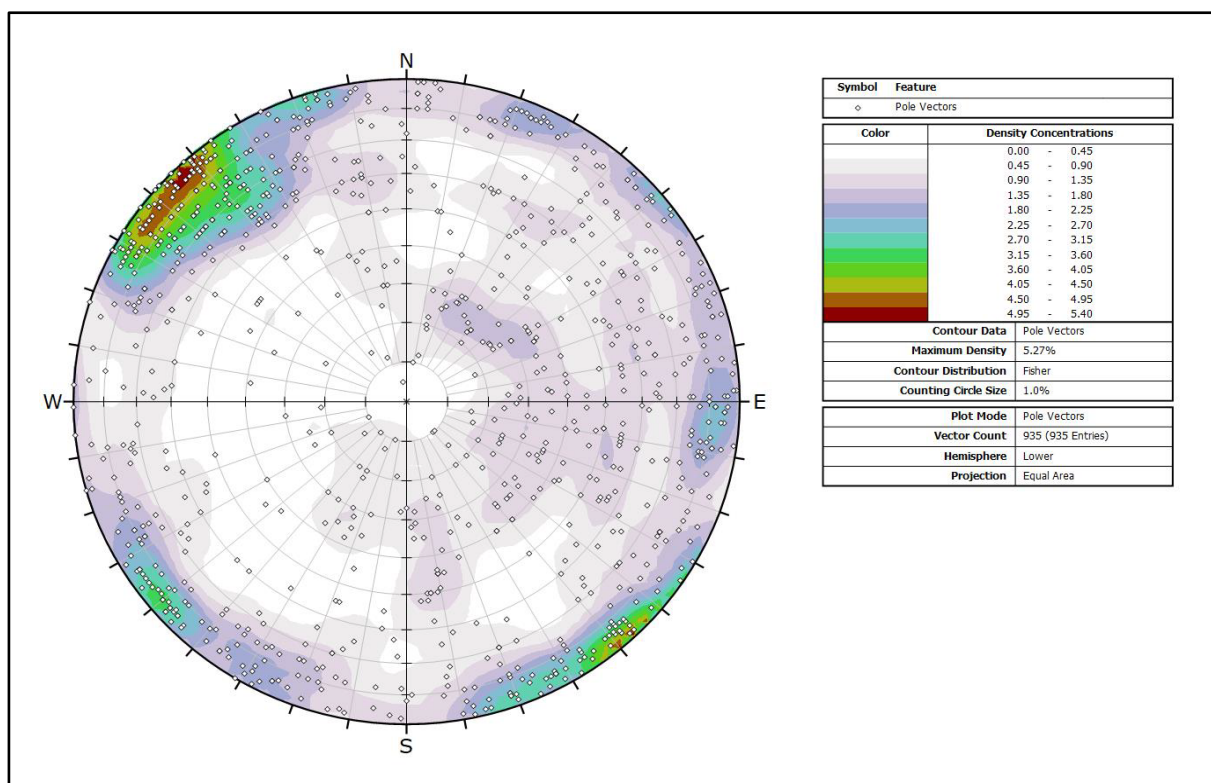


Figure 40: Stereographic projection showing major concentration contours with poles of all scanline bench mapping surveys.

3.2.2 Window Mapping using Photogrammetry

Photogrammetry methods were used to map benches that were inaccessible and/or considered too dangerous to complete conventional bench mapping surveys. This method typically involves taking overlapping digital photographs of exposed bench face with reference positions surveyed (i.e., two camera positions and at least one control point for each photo pair), and subsequent image processing in a suitable program (e.g., ©Sirovision, ©3G ShapeMetrix & JointMetrix, and ©Adams Technology 3DM Analyst) to create a spatially correct 3D image from which accurate measurements of structures can be made. This project makes use of the Sirovision software developed by CSIRO in Australia, a high precision 3D photogrammetry and 3D imaging technology for the mapping of surfaces. This software consists of two programs namely, Siro3D which is used to create the 3D outcrop model and SiroJoint used for structural mapping and analyses.

The general workflow followed to create these 3D images using Sirovision is presented in Figure 41, and Figure 42 shows an example of a 3D outcrop model created for Mine X. Table 5 provides a summary of the window mapping surveys completed using photogrammetric methods. Figure 43 shows the locations of the control points and areas mapped within the Mine X open pit. A stereographic projection showing all photogrammetry bench structures mapped is provided in Figure 44; with all 3D image models and related stereonet presented in APPENDIX B and APPENDIX A respectively.

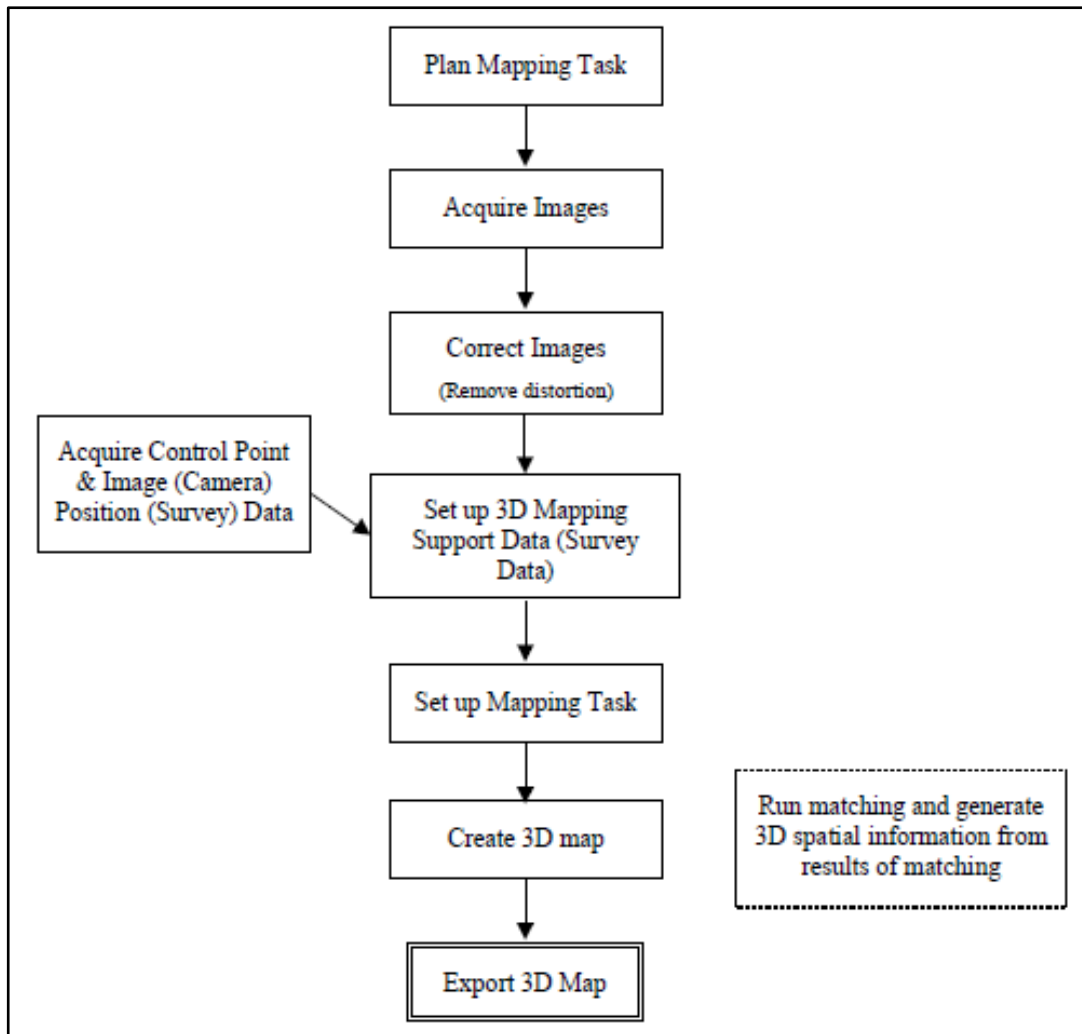


Figure 41: Workflow required for creation of 3D image models using Sirovision software (CSIRO Exploration and Mining, 2009).

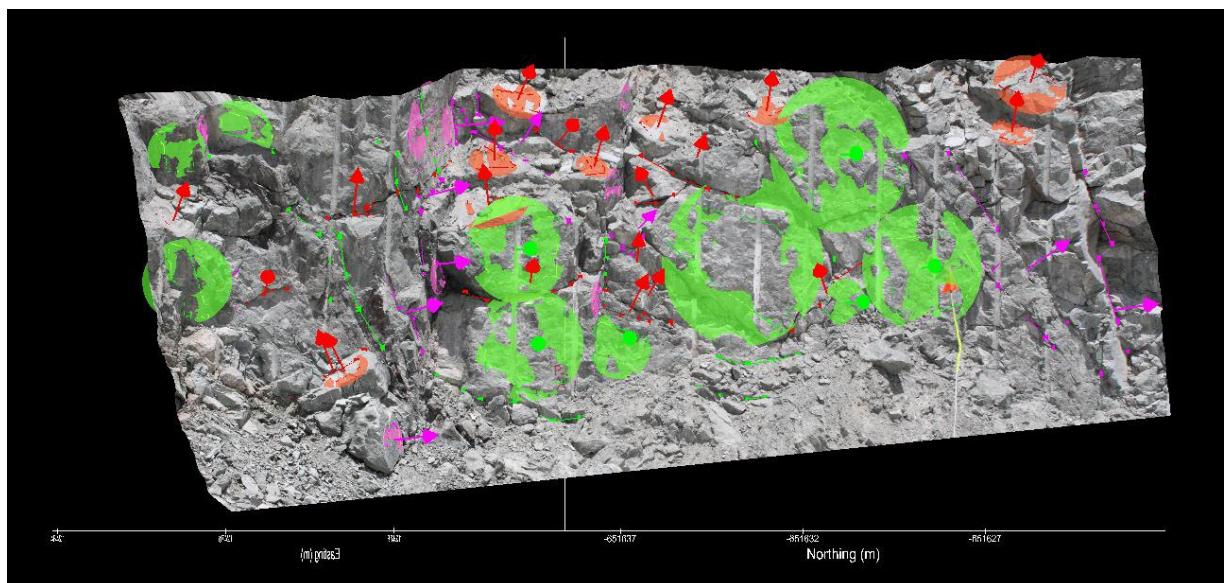


Figure 42: Example of Sirovision 3D outcrop model.

Table 5: Mine X Window Mapping Surveys using Photogrammetry

Window Mapping Survey ID / 3D Model Name	Bench Level	Control Points	Bench Face Orientation		Lithology
			Dip Angle (°)	Dip Direction (°)	
BF12-L01_c_3D_Image	13	P1	80	042	Massive Pyroxenite
BF12-L02_c_3D_Image	13	P2	80	042	Massive Pyroxenite
BF12-L03_c_3D_Image	13	P3	80	042	Massive Pyroxenite
BF12-L04_c_3D_Image	13	P4	80	042	Massive Pyroxenite
BF12-L05_c_3D_Image	13	P5	80	042	Massive Pyroxenite
BF12-L06_c_3D_Image	13	P6	80	042	Massive Pyroxenite
BF12-L07_c_3D_Image	13	P7	80	042	Massive Pyroxenite
BF12-L08_c_3D_Image	13	P8	80	042	Massive Pyroxenite
BF12-L09_c_3D_Image	13	P9	80	042	Massive Pyroxenite
BF12-L10_c_3D_Image	13	P10	80	042	Massive Pyroxenite
BF12-L11_c_3D_Image	13	P11	80	042	Massive Pyroxenite
BF12-L12_c_3D_Image	13	P12	80	345	Massive Pyroxenite
BF12-L13_c_3D_Image	13	P13	80	345	Massive Pyroxenite
BF12-L14_c_3D_Image	13	P14	80	345	Massive Pyroxenite
BF15-L16_c_3D_Image	16	P16	80	065	Massive Pyroxenite
BF15-L17_c_3D_Image	16	P17	80	065	Massive Pyroxenite
BF15-L18_c_3D_Image	16	P18	80	042	Massive Pyroxenite
BF15-L19_c_3D_Image	16	P19	80	042	Massive Pyroxenite
BF15-L20_c_3D_Image	16	P20	80	042	Massive Pyroxenite
BF15-L22_c_3D_Image	16	P22	80	042	Massive Pyroxenite
BF13-L25_c_3D_Image	14	P25	80	290	Massive Pyroxenite
BF13-L26_c_3D_Image	14	P26	80	290	Massive Pyroxenite
BF13-L27_c_3D_Image	14	P27	80	290	Massive Pyroxenite
BF13-L29_c_3D_Image	14	P29	80	345	Massive Pyroxenite
BF13-L30_c_3D_Image	14	P30	80	345	Massive Pyroxenite
BF13-L31_c_3D_Image	14	P31	80	345	Massive Pyroxenite
BF13-L32_c_3D_Image	14	P32	80	042	Massive Pyroxenite
BF13-L33_c_3D_Image	14	P33	80	042	Massive Pyroxenite
BF13-L34_c_3D_Image	14	P34	80	042	Massive Pyroxenite
BF15-L37_c_3D_Image	16	P37	80	290	Feldspathic Pyroxenite
BF15-L38_c_3D_Image	16	P38	80	290	Feldspathic Pyroxenite
BF15-L39_c_3D_Image	16	P39	80	290	Feldspathic Pyroxenite
L42-100_c_3D_Image	8	P42	80	290	Massive Pyroxenite
L43-100_c_3D_Image	8	P43	80	290	Massive Pyroxenite
L44-100_c_3D_Image	8	P44	80	290	Massive Pyroxenite
BF15-L23_c_3D_Image	16	P23	80	042	Massive Pyroxenite
BF15-L24_c_3D_Image	16	P24	80	042	Massive Pyroxenite
BF15-L36_c_3D_Image	16	P36	80	290	Feldspathic Pyroxenite

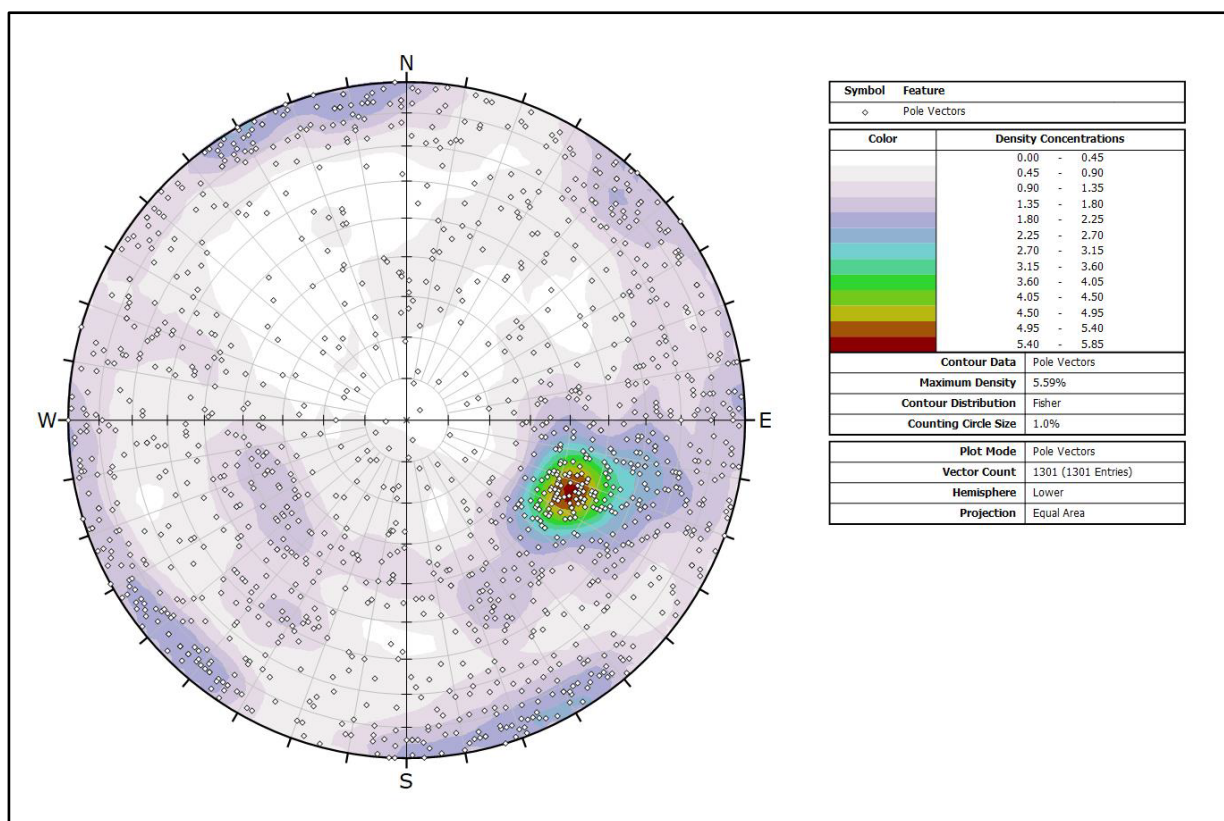


Figure 44: Stereographic projection showing major concentration contours with poles of all photogrammetry window bench mapping surveys.

3.3 Chapter Summary

After the review of all available data from several projects, a case study was selected based on the amount of structural data available. A pyroxenite pit at a local South African phosphate mine in Phalaborwa was selected. The study area forms part of the Phalaborwa Igneous Complex (PIC) which is considered the most important carbonatite complex in South Africa (Roux, et al., 1989). The field data collection program involved scanline and window mapping of Mine X pit benches exposures across various lithological units, namely, granite, dolerite, massive pyroxenite and feldspathic pyroxenite. Photogrammetry methods were used to map structures within benches that were inaccessible and/or considered unsafe to complete conventional bench mapping surveys. Mapping data was the only source of structural data available; no borehole information was provided. The data was processed and the statistical analysis of the available discontinuity data is summarised in APPENDIX C; with APPENDIX A providing all the stereoplots for the Mine X structural data; and APPENDIX B, summarising the image data processing methodology and digital mapping with Sirovision. This is a hard rock mining environment with structures that contribute predominantly to the observed slope instabilities.

CHAPTER 4

4.0 STEREOGRAPHIC ANALYSES OF STRUCTURAL DATA

The discontinuity orientation data collected from mapping exercises conducted at Mine X open pit, described in the previous Chapter 3.0, was processed and analysed using statistical stereographical methods with the ©Rocscience Dips program. This chapter presents the stereographic analyses carried out for Mine X open pit, which includes the initial structural data interpretation and the kinematic assessment.

4.1 Interpretation of Structural Data

The purpose of this structural evaluation was to identify discontinuity sets that would best represent the structural fabric of the rock masses prevalent in the pit walls of Mine X. The conventional equal area, lower hemisphere, Fisher contoured stereographic projections were used. The mapping orientation data was corrected for, sampling biased using Terzaghi's weighting (Terzaghi, 1965), and magnetic declination in the Phalaborwa area.

The scanline mapping surveys, and digitally mapped window surveys data covered bench-scale exposures across various lithological units, namely, granite, dolerite, massive pyroxenite and feldspathic pyroxenite. Structures mapped included joints, shears, and faults. This structural orientation data was analysed using various data sets created based on the different lithological units, discontinuity types, pit wall locations, mapping data source and combinations/subsets thereof, (stereographic plots included in APPENDIX A). The contoured pole densities for each of the various structural sets created were visually assessed and compared to regional structural trends in order to identify dominant local structural trends. Examples of the data set created per discontinuity type and per lithological unit are shown in Figure 45 and Figure 46 respectively. In general, the structural trends identified from the mapping data collected correspond with the regional structural trends observed. All available mapping data were processed and assessed; and a subset of this data was selected for further use in this research project. Section 4.1.1 presents the selected discontinuity data and its structural interpretation.

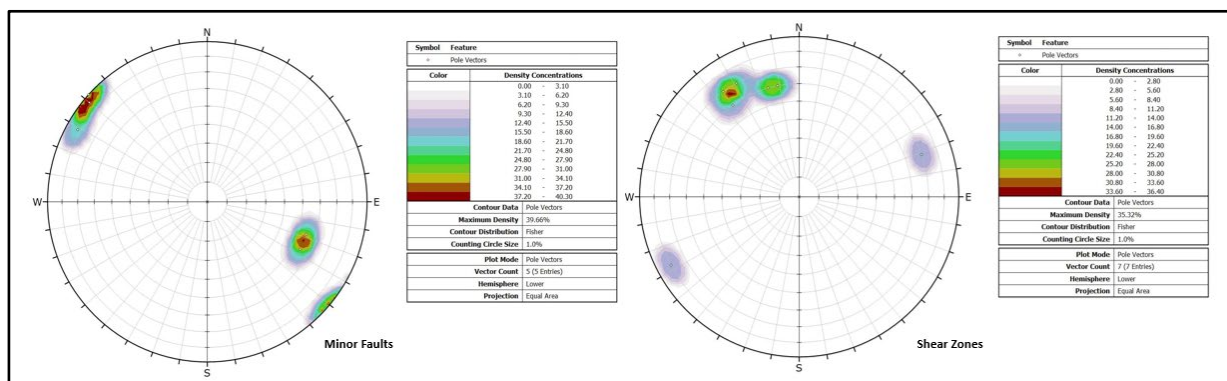


Figure 45: Stereonet plots for discontinuity types: minor faults and shear zones.

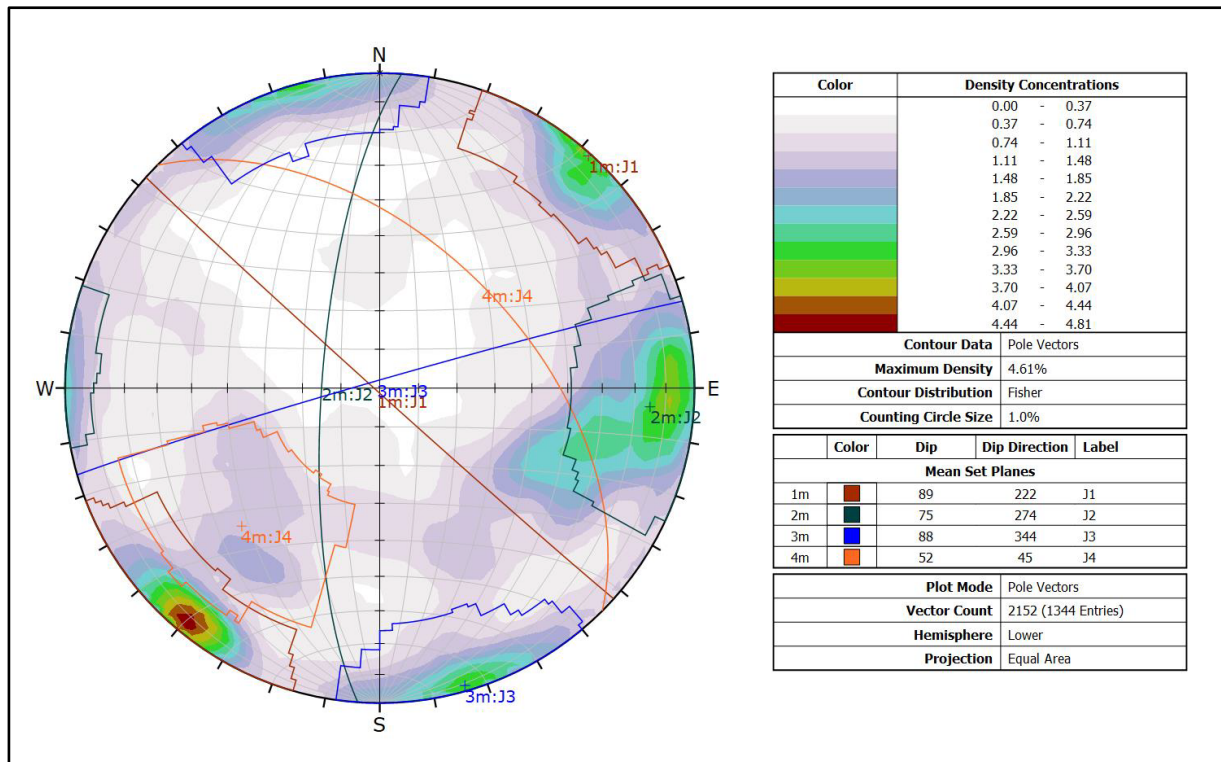


Figure 47: Stereographic projection plot showing discontinuity sets identified for the massive pyroxenite rock.

4.2 Kinematic Analyses

The Mine X open pit has been subdivided into pit sectors based on the pit wall orientations and similar geotechnical conditions, summarised in Table 7 and shown in Figure 48. The kinematic analyses carried out assessed the potential for planar and wedge sliding, based on the following:

- The discontinuity sets identified in Table 6, which were assumed to be representative of the rock mass fabric;
- A friction angle of 30 degrees;
- Massive pyroxenite benches;
- A 90-degree slope face [i.e. bench face angle (BFA)];
- Pit sectors and their corresponding average pit wall directions summarised in Table 7.

Table 7: Mine X Pit Sectors

Pit Sector	North East Sector	East Sector	South Sector	South West Sector	West Sector	North West Sector
Pit Wall Mean Dip Direction	218°	295°	040°	352°	085°	147°

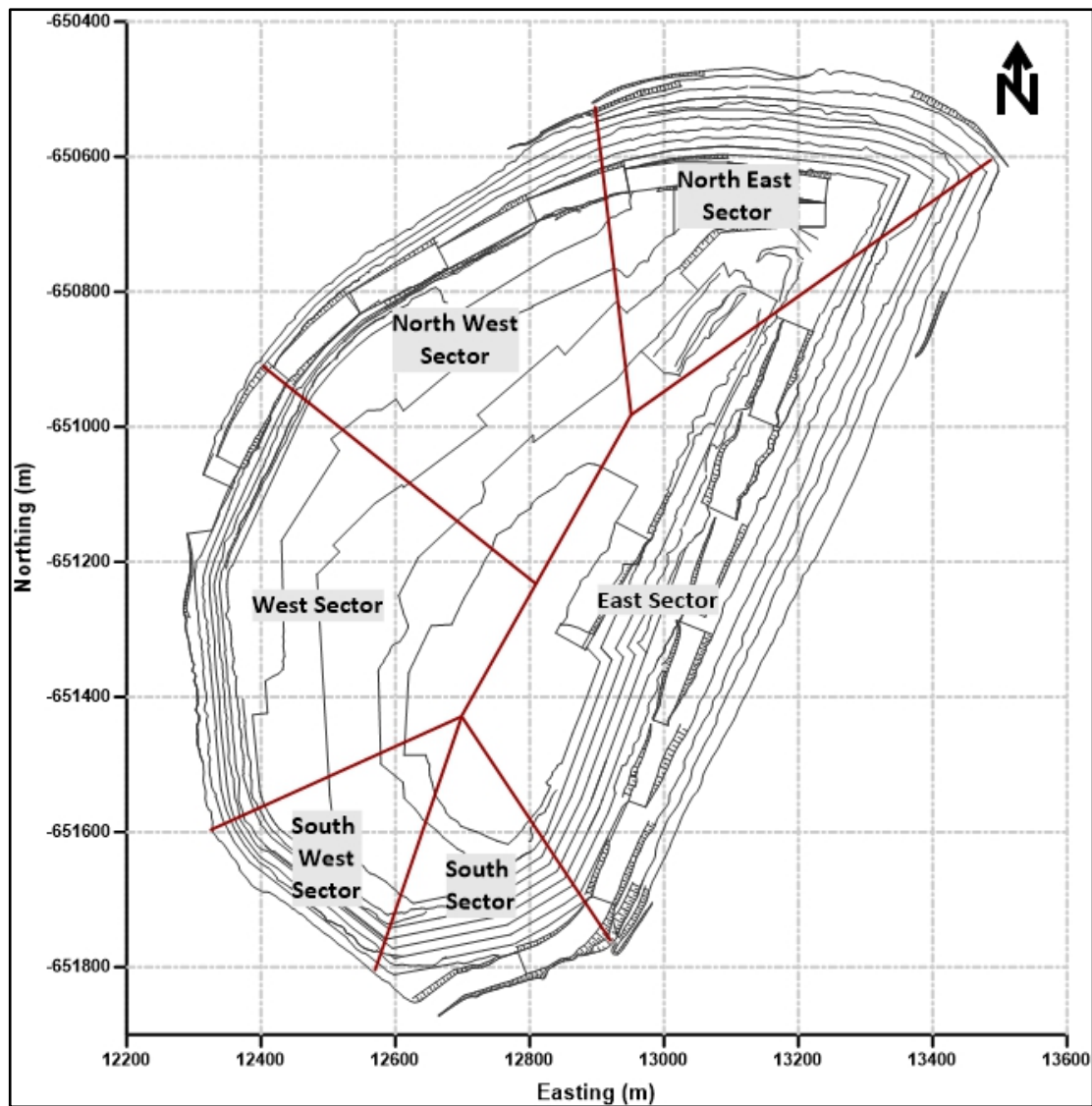


Figure 48: Mine X Pit Sectors

The results for the kinematic analyses completed for the Mine X MP benches is summarised in Table 8. Examples of the wedge sliding and planar sliding kinematic assessment using DIPS are shown in Figure 49 and Figure 50, respectively. APPENDIX A provides the kinematic stereographic projections for all pit sectors analysed.

The east, south west and north west pit sector walls are more susceptible to wedge type failures with several joint set combinations identified (refer to results Table 8). Planar sliding is more prevalent in the east, south and south west sector pit wall benches. Based on all data planes intersection type, the highest (i.e., 50.5%) “probability of failure”, for wedge sliding, is expected to occur in the east pit sector and the lowest (i.e. 29.2%) expected in the north west pit sector walls. The east sector pit walls show the highest probability of failure, expressed as a percentage, is 17.5% for planar sliding based on all gridded data.

The wedge failures identified during this kinematic analysis for the Mine X pit sectors will be further assessed deterministically and probabilistically using limit equilibrium methods to a better ascertain the kinematic failure mode potential for the various pit sectors. This is presented in Chapter 6.0.

Table 8: Summary of Planar and Wedge Kinematic Results for Mine X MP Benches

Pit Sector	Mean Pit Wall Dip Direction (°)	Planar Failure Potential		Wedge Failure Potential		
		Joint Sets / Gridded Data Planes	% Poles	Joint Set Combination/ Intersection Type		% Poles
East Sector	295	All Data Planes	17.5	All Data Planes		50.5
		J2	47.6	All Set Planes		62.8
				User & Mean Set Planes		66.8
				J1	J2	
				J1	J3	
				J2	J3	
				J2	J4	
South Sector	040	All Data Planes	15.7	All Data Planes		41.5
		J1	48.6	All Set Planes		55.6
		J4	78.4	User & Mean Set Planes		33.3
				J2	J4	
				J3	J4	
North East Sector	218	All Data Planes	9.8	All Data Planes		33.3
		J1	44.2	All Set Planes		37.6
				User & Mean Set Planes		50.0
				J1	J3	
				J1	J4	
				J2	J3	
North West Sector	147	All Data Planes	6.2	All Data Planes		26.7
		J3	23.6	All Set Planes		27.3
				User & Mean Set Planes		16.7
				J3	J4	
West Sector	085	All Data Planes	7.4	All Data Planes		29.2
		J1	0.7	All Set Planes		31.6
		J2	7.3	User & Mean Set Planes		16.7
		J4	12.0	J3	J4	
South West Sector	352	All Data Planes	11.8	All Data Planes		44.8
		J3	48.2	All Set Planes		56.9
				User & Mean Set Planes		66.7
				J1	J2	
				J1	J3	
				J2	J4	
				J3	J4	

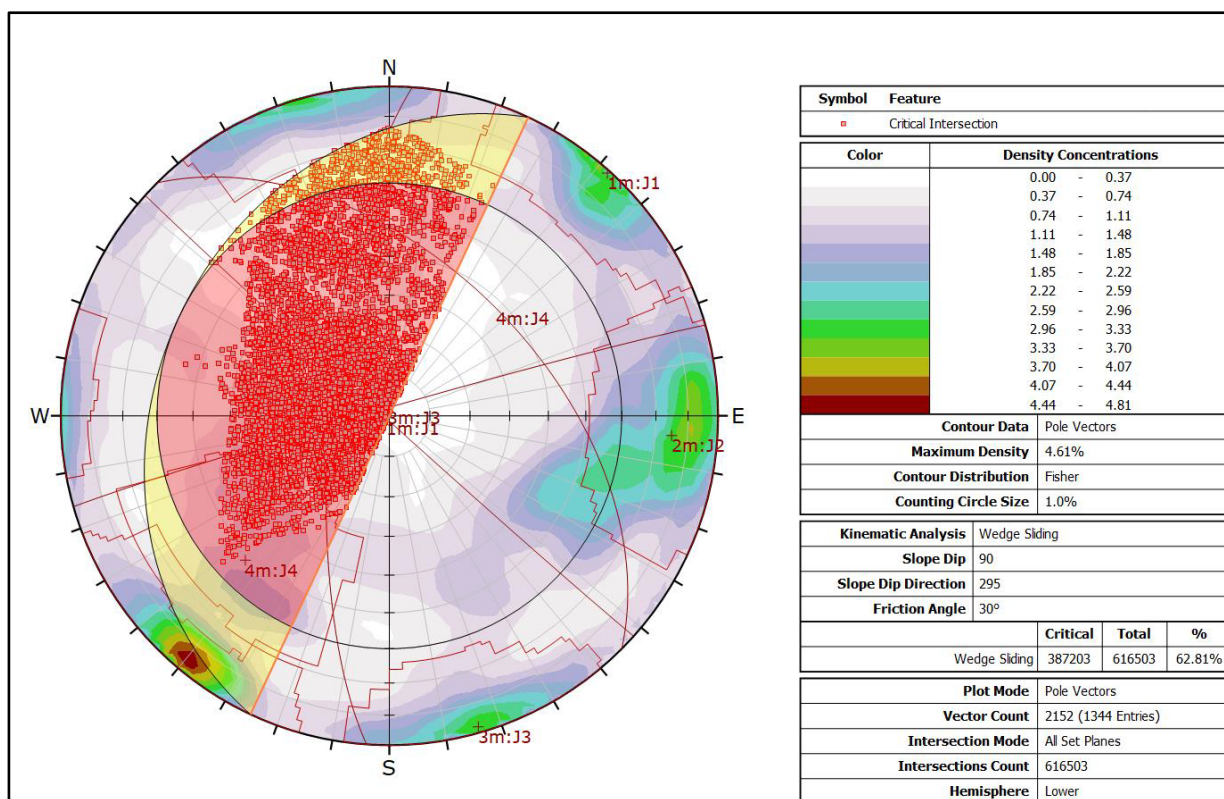


Figure 49: Stereographic projection showing kinematic wedge sliding analyses for Mine X, east sector pit wall benches.

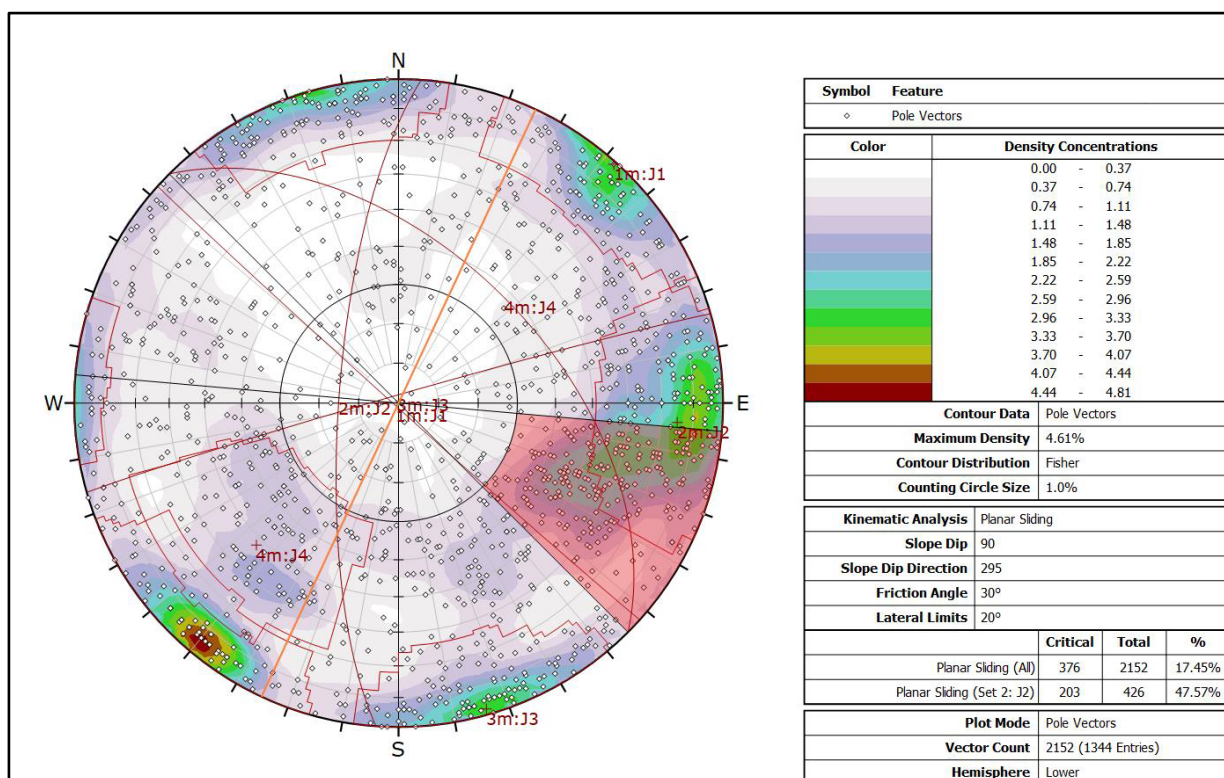


Figure 50: Stereographic projection showing kinematic planar sliding analyses for Mine X, east sector pit wall benches.

It should be noted that the kinematic analysis is considered deterministic if only mean set discontinuities are considered. Generally, considering only the mean set discontinuities is representative of a small or tightly clustered structures that has not much variation and or is limited to the user set window.

If we considered the deterministic kinematic results only (i.e., mean set data), the following would be observed:

- The west sector would be most susceptible to planar sliding as indicated by the highest number of sets planes.
- The south and east sector would be most susceptible to wedge sliding as indicated by the highest number of wedges formed.

Whilst the highest potential for wedge sliding is identified in the east sector considering all results; for planar sliding, the greatest number of set planes susceptible to planar failures is identified in the west sector as compared to the east sector as shown by all data. This observation reminds us that considering mean set planes only can create misleading perceptions as indicated by the planar sliding results. Hence, we should be cognisant that interpreting kinematic failure mode potential from first pass deterministic stereographic kinematic analysis can be misleading and further assessment is required to better ascertain the pit walls susceptibility to a particular mode of failure.

4.3 Chapter Summary

A structural evaluation of all data was carried out and the massive pyroxenite data was selected for this study. This subset of data was analysed in order to identify discontinuity sets that would best represent the structural fabric of the rock masses prevalent in the pit walls of Mine X. A total of four discontinuity sets were identified for the massive pyroxenite unit. These were used for the subsequent stereographic kinematic analysis and as input for the DFN modelling. The kinematic analysis considered six pit wall orientations from various pit sectors. Based on the number of wedge combinations; the north east, east, and south west pit sector walls would expect more wedge type failures with several joint set combinations. Planar sliding more prevalent in the east, south and south west sector pit wall benches. The highest probability of wedge sliding is expected to occur in the east pit sector and the lowest expected in the west pit sector walls. The east sector pit walls show the highest probability of failure for planar sliding. It is important to understand the results of the kinematic assessment to avoid creating misleading perceptions on the potential for kinematic failure modes in the pit wall.

CHAPTER 5

5.0 DFN MODEL DEVELOPMENT AND VALIDATION

DFN models are statistical representations of the rock mass structural fabric and are based on the structural properties such as persistence, orientation, and intensity. This chapter presents a summary of the DFN model properties and the methodology used to develop and validate the DFN model generated for the Mine X massive pyroxenite rock mass fracture system. A total of five DFN model realizations were generated at bench-scale for the Mine X case study with several pit wall orientations considered for the subsequent rock wedge stability analysis. This chapter is supplemented with DFN modelling details provided in APPENDIX D.

5.1 DFN Model Generation

In order to use DFN models for any particular analysis, a DFN model will need to be generated and validated. The steps typically required to generate a simple DFN model involve:

- Field data collection (summarised in chapter 3.0).
- Discontinuity data characterization (presented in 4.0, APPENDIX A and APPENDIX C);
- DFN model generation; and
- DFN model validation.

The FracMan software developed by Golder Associates was used for this research due to its availability and frequent use within the DFN community. This software generates stochastic simulations of fracture networks based on user-defined inputs that are generally obtained through the statistical analysis and interpretation of fracture data.

The FracMan program allows for the following types of fracture network constructions (Golder Associates, 2008):

- *Geocellular* – based on the generation of fractures within a series of finite volume cellular elements;
- *Stratigraphic* – fractures are created in layers, where each layer is defined by a pair of grid layers;
- *Trace Map* – this method generates fractures based on a trace map; and
- *Geometric* – basic method that uses a bounding region for the fracture generation area.

FracMan offers three spatial models (i.e. Enhanced Baecher, Nearest Neighbour and Levy Lee) that can be used to specify the location of individual fractures belonging to each fracture set.

For this research project, the geometric network construction method was used. This method for generating a DFN model requires the identification and characterization of individual fractures sets. The methodology used to determine the DFN primary properties were discussed in chapter 2.0, sub-section

2.2 and the related Mine X case study analyses provided in 3.0 and 4.0. APPENDIX C summarises the statistical analysis for each fracture set identified for the Mine X case study.

As described in chapter 4.0, four fracture sets were identified to represent the massive pyroxenite rock mass smaller-scale structures at Mine X open pit. The orientations of each of these fracture sets were shown in Figure 47. The statistical summaries for the individual fracture sets (provided in APPENDIX C) were used to establish the input parameters for the Mine X DFN models. The primary properties used for the DFN models are summarised in Table 9.

Table 9: DFN Model Input Parameters

Fracture Set ID		J1	J2	J3	J4
Spatial model		Enhanced Baecher	Enhanced Baecher	Enhanced Baecher	Enhanced Baecher
Orientation		Fisher	Fisher	Fisher	Fisher
		K = 35.1	K = 25.8	K = 28.9	K = 23.1
P32 (1/m)		0.20	0.20	0.20	0.20
Fracture Size	Distribution	Lognormal	Lognormal	Lognormal	Lognormal
	Mean (m)	2.0	2.2	1.9	1.7
	Standard Deviation (m)	8.0	6.4	5.8	4.2
Termination %		0% assumed for all sets.			

The P32 intensity (i.e., area of fractures per unit volume) was estimated using the P32 simulation method based on P21 (i.e. length of fractures per unit area) data derived from photogrammetric surveys as described by (Rogers, et al., 2017). Fracture size parameters were determined from trace length statistical distributions and converted to an equivalent fracture radius using analytical methods provided by (Zhang, et al., 2002). The Fisher distribution was selected to model the orientation dispersion and the Enhanced Baecher model applied to the spatial distribution of fractures. Fracture termination was not assigned for this modelling exercise so that the fracture generation would not be conditioned to this. A four-sided polygon was selected to represent the fracture shape for the DFN model. The fracture networks were generated within an outer rock mass box model region and clipped to the bench-scale model region, shown in Figure 51 to reduce boundary effects. The shear strength criterion was kept constant for all discontinuity sets (i.e., Mohr-Coulomb criterion used with an assumed zero cohesion and a friction angle of 30°) in order to maintain the research focus on geometric characteristics related to wedge formation and stability. Similarly, fracture stress was also not considered for the stability analysis.

Generally, the industry practice is to generate 5 to 10 model realizations for a model of this scale as this is considered to produce fairly consistent results. Therefore, a total of five realizations were generated for the Mine X DFN bench-scale model. It should be noted that no deterministic structures such as faults and /or shear zones were explicitly considered in the generation of the DFN models.

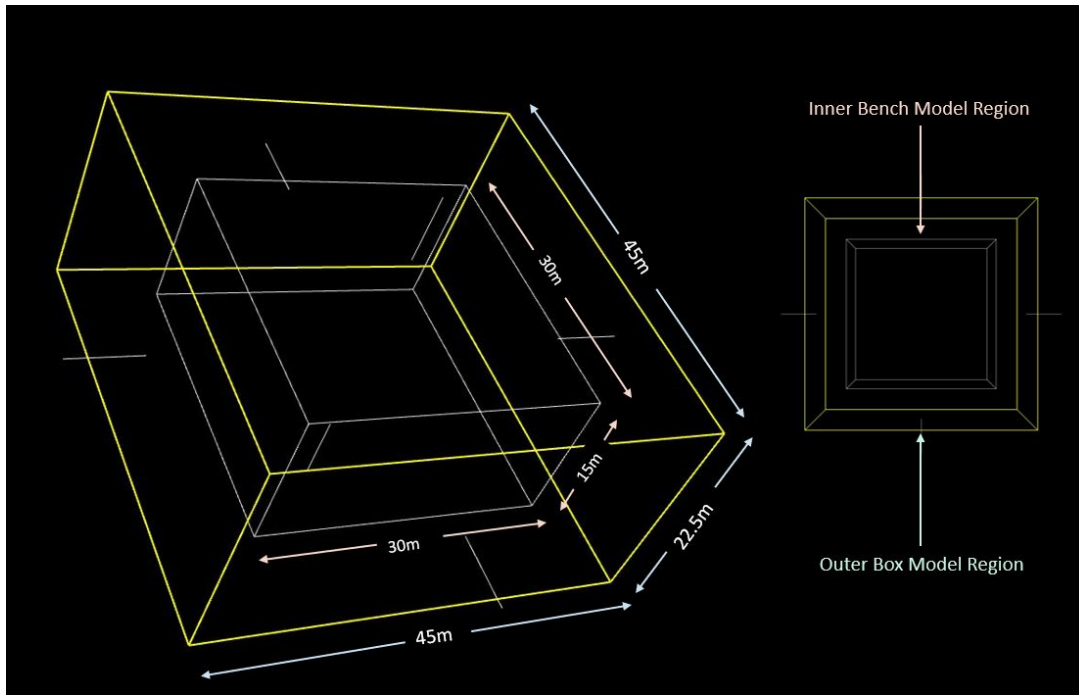


Figure 51: FracMan DFN Model Regions.

5.2 DFN Model Realizations

Each of the five DFN model realizations shown in Figure 52, Figure 53, Figure 54, Figure 55, and Figure 56 are considered as one possible representation of the massive pyroxenite rock mass structure at Mine X pit. These DFN model realizations were validated as described in section 5.3 and APPENDIX D. These DFN realizations are used for the rock wedge limit equilibrium stability assessment using FracMan.

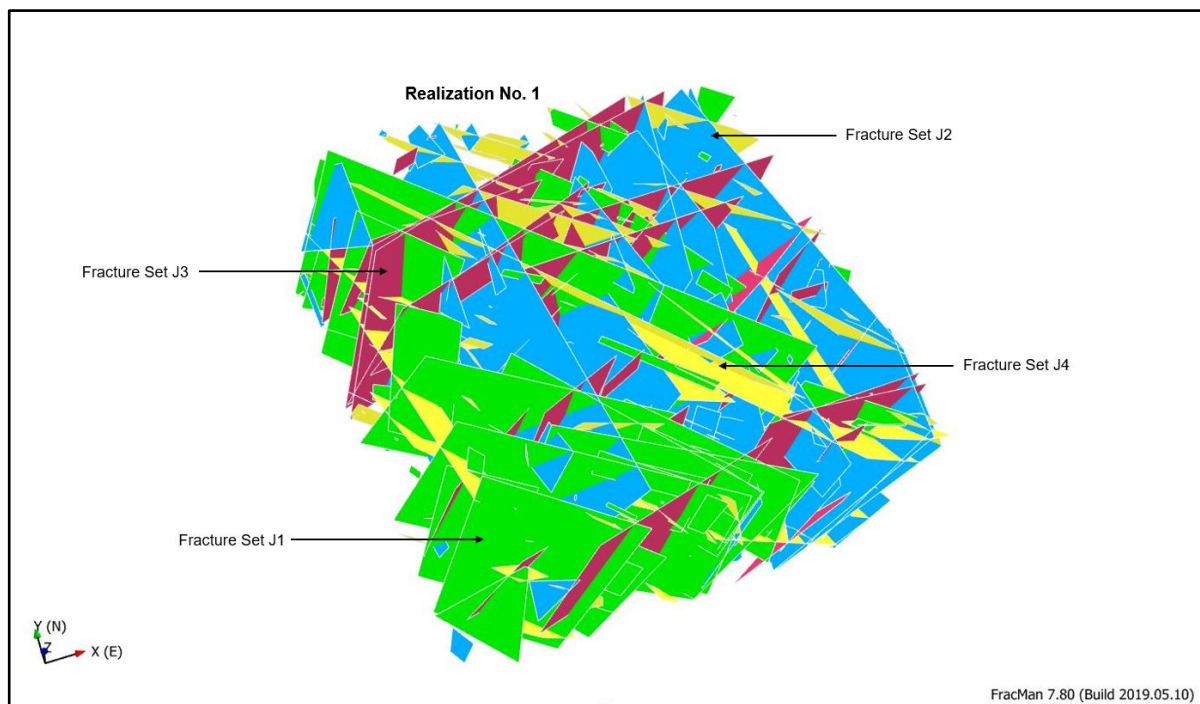


Figure 52: DFN Model Realization 1.

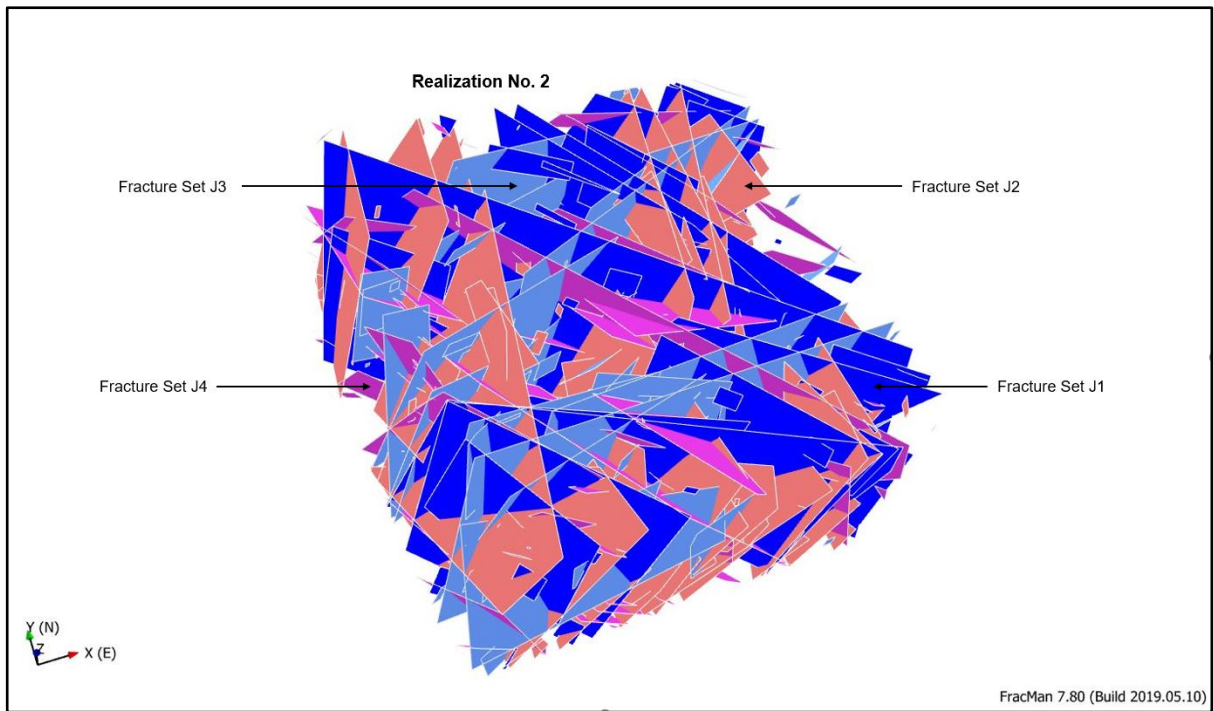


Figure 53: DFN Model Realization 2.

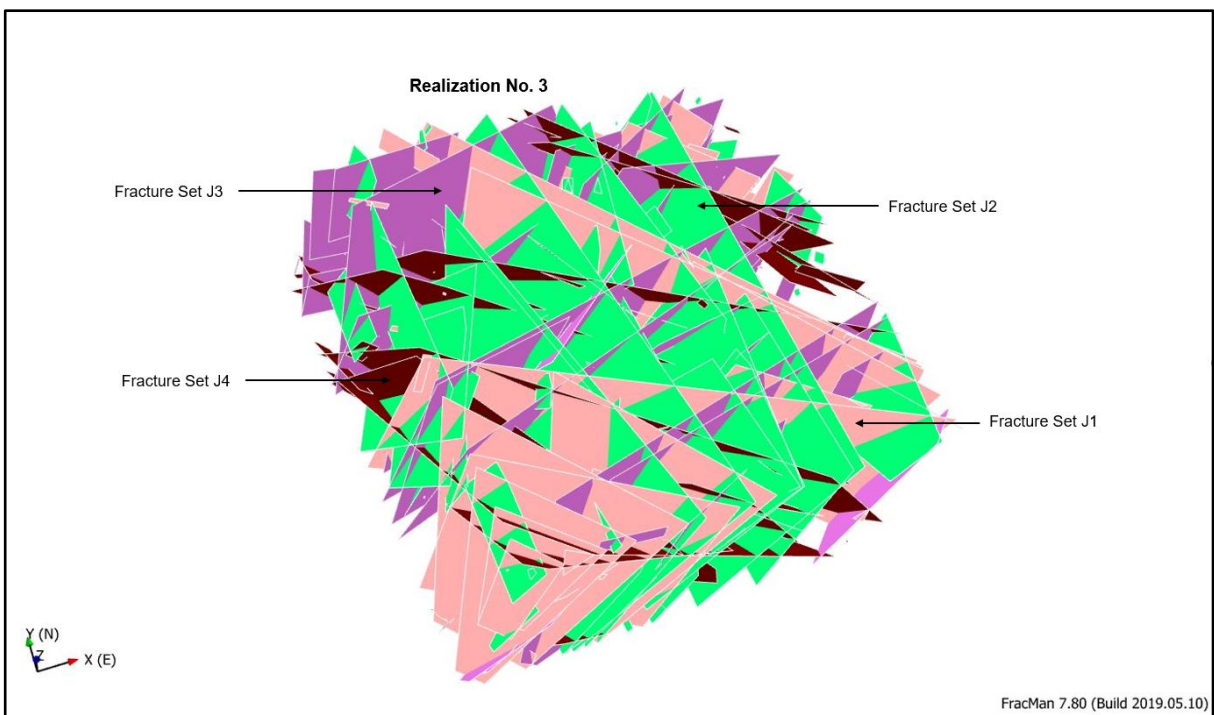


Figure 54: DFN Model Realization 3.

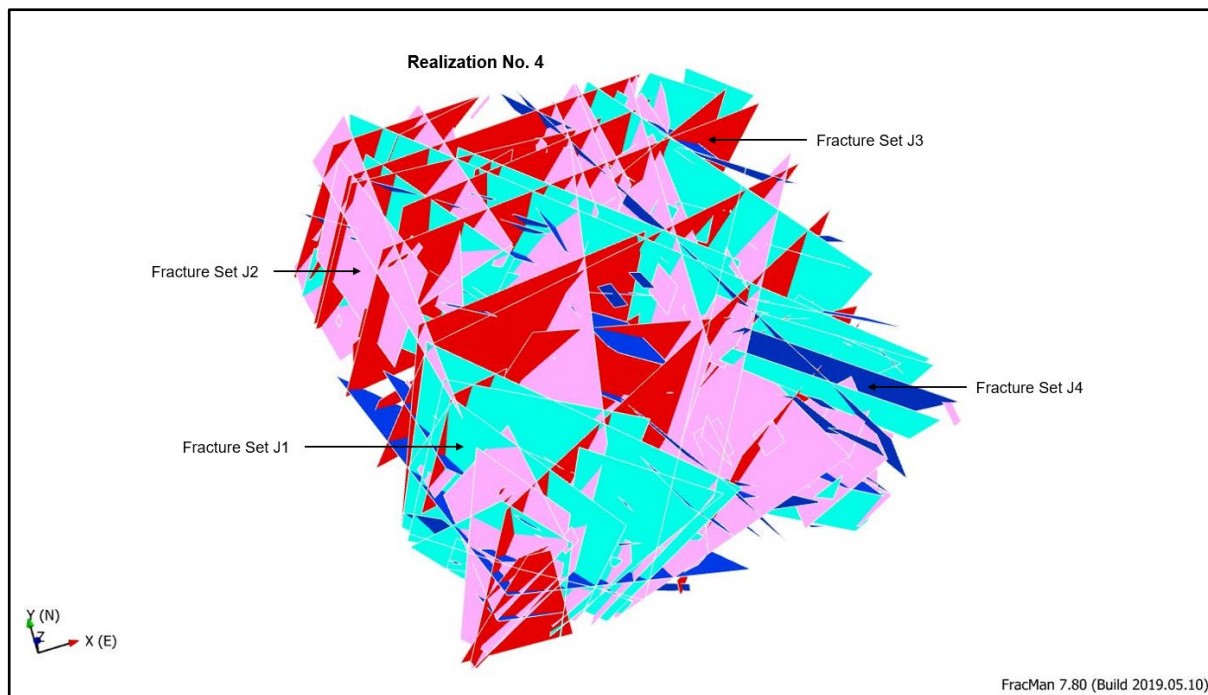


Figure 55: DFN Model Realization 4.

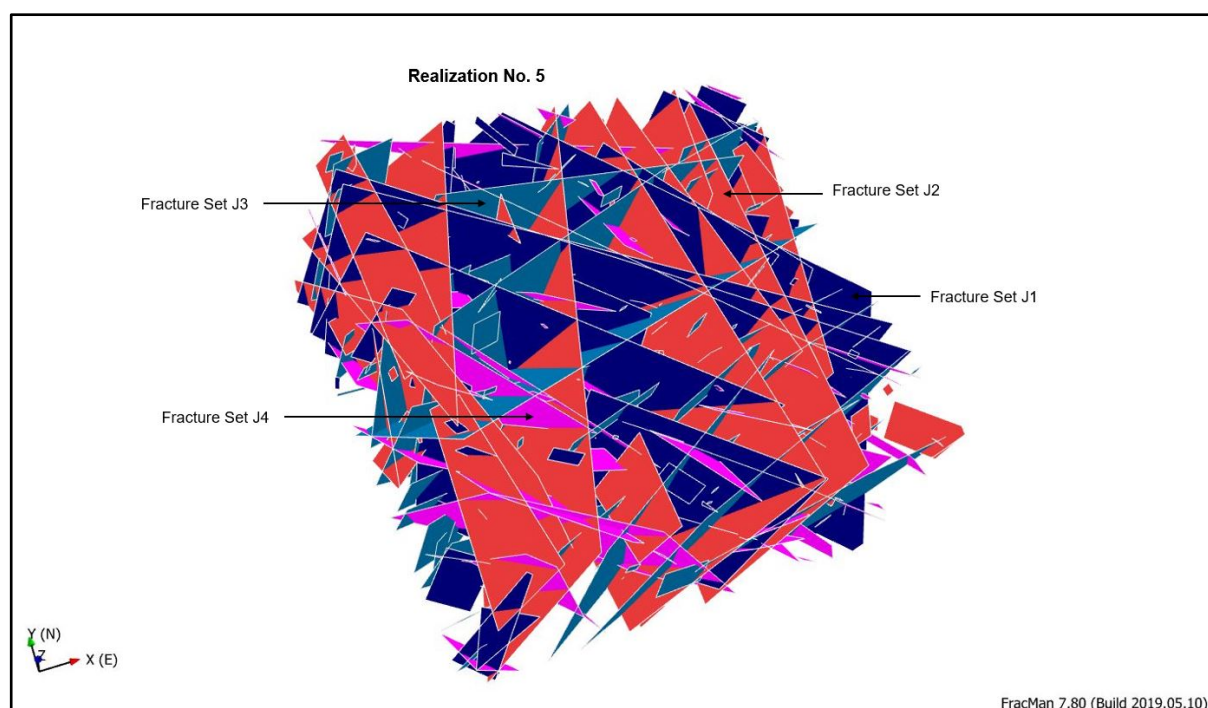


Figure 56: DFN Model Realization 5.

5.3 DFN Model Validation

Model validation is critical in evaluating the suitability of the synthetic fracture network and its intended application. The DFN realizations generated with FracMan produces statistical derived models that were validated by comparing models with the field data and observations. In general, the DFN model validation process has shown a reasonable level of agreement between the DFN model and the source data for Mine X. The following were carried out during the model validation process:

- Comparisons of the orientation of discontinuity sets observed in the field with the simulated fractures for each DFN realization. This includes comparisons of orientation of individual outcrops mapped with simulated fractures intersecting the trace plane orientated parallel to the bench outcrop mapped.
- Assessment of the fracture intensity by comparison of the number of fractures per area the window mapping field data with the corresponding trace plane for each of the DFN realizations. It should be noted that the P32 intensity was generated by simulation using P21 data calculated from field mapping data.
- Evaluation of the trace length distributions of simulated fractures intersecting the trace plane and the field data trace length statistics. Comparison of the simulated fractures trace length data with the field structures mapped.
- Visual inspections, that served as either “reality checks” or “virtual ground truthing” was also carried out to ensure that the DFN model is reflective of the field observations. This included comparisons of the spatial distribution and fracture intensity of mapping outcrops with their corresponding DFN trace maps for each model realization.

APPENDIX D provides the validation of the Mine X DFN model realizations. Stereoplots showing orientation data comparison of mapped structures and simulated fractures are shown in Figure 57, Figure 58, Figure 59, Figure 60, Figure 61 for DFN model realizations 1 to 5. Figure 62, shows a comparison of mapped and simulated fracture orientation for an individual mapped outcrop. As shown by these stereoplots and those provided in APPENDIX D, the pole clusters observed in the mapped structures are also similarly observed in the simulated fractures. This indicates that the fracture orientations are suitably represented in the DFN models generated.

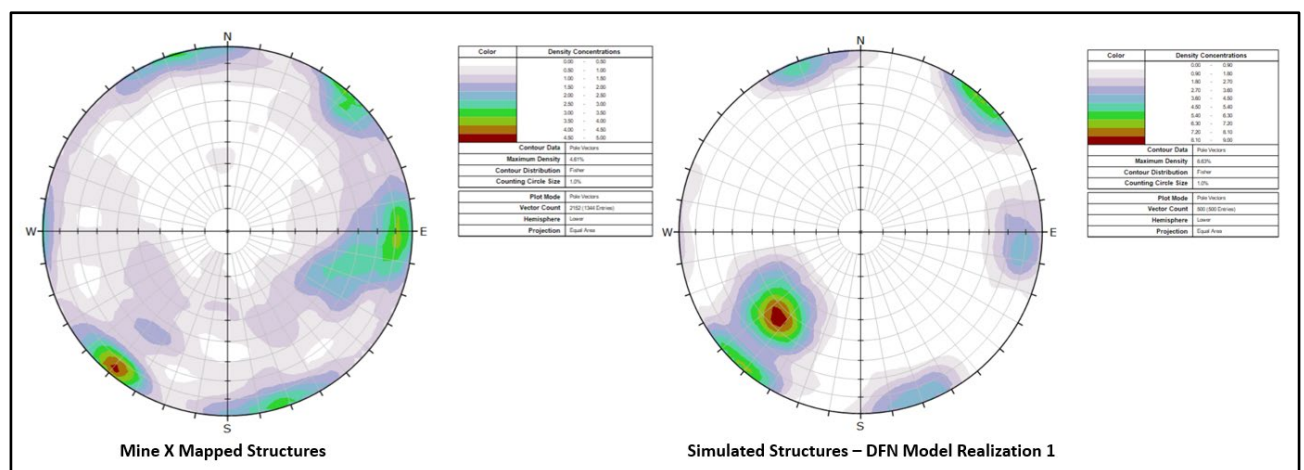


Figure 57: Orientation data comparison of mapped structures with the simulated structures from DFN Model realization no. 1.

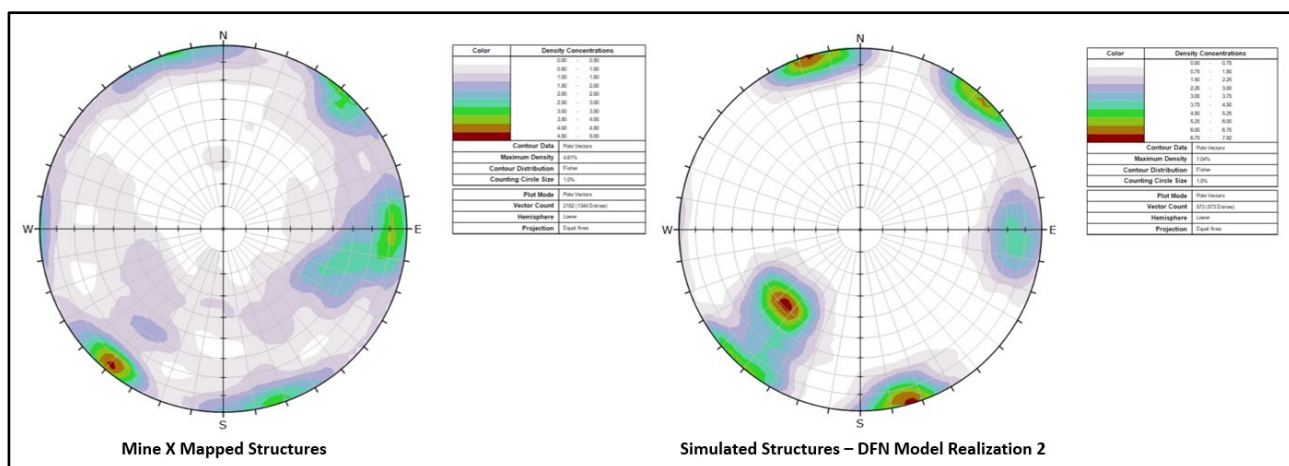


Figure 58: Orientation data comparison of mapped structures with the simulated structures from DFN Model realization no. 2.

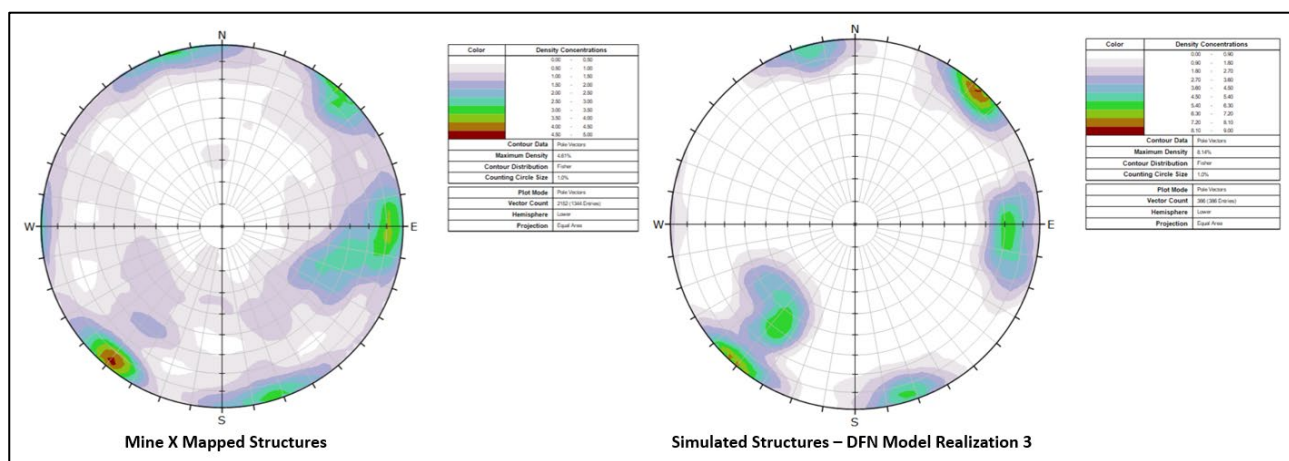


Figure 59: Orientation data comparison of mapped structures with the simulated structures from DFN Model realization no. 3.

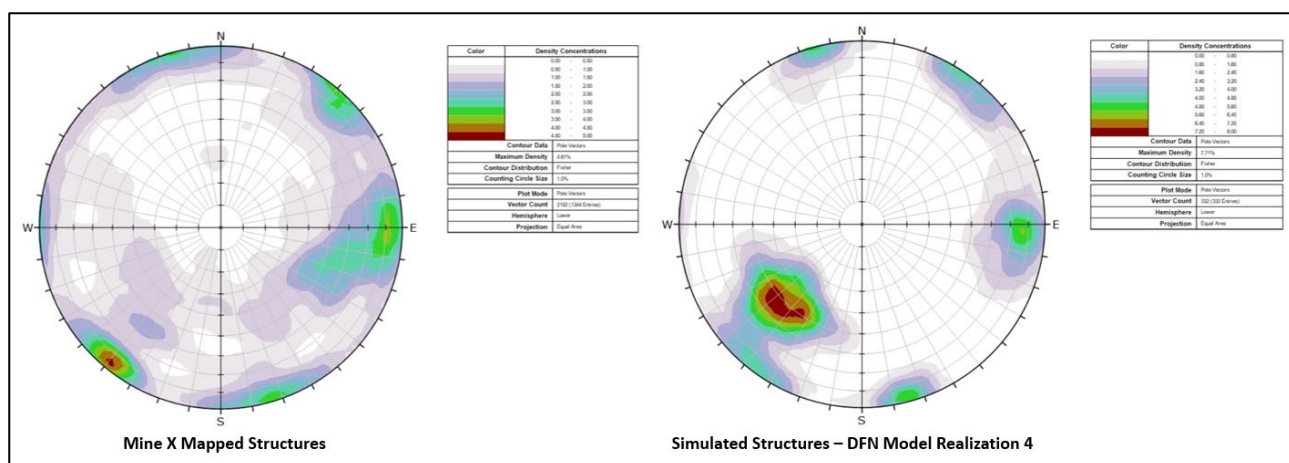


Figure 60: Orientation data comparison of mapped structures with the simulated structures from DFN Model realization no. 4.

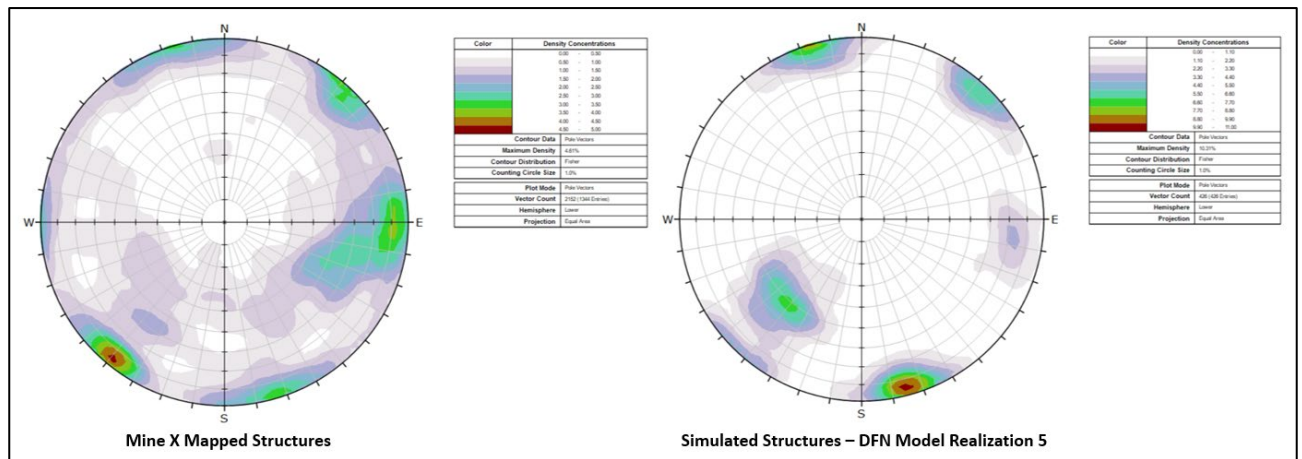


Figure 61: Orientation data comparison of mapped structures with the simulated structures from DFN Model realization no. 5.

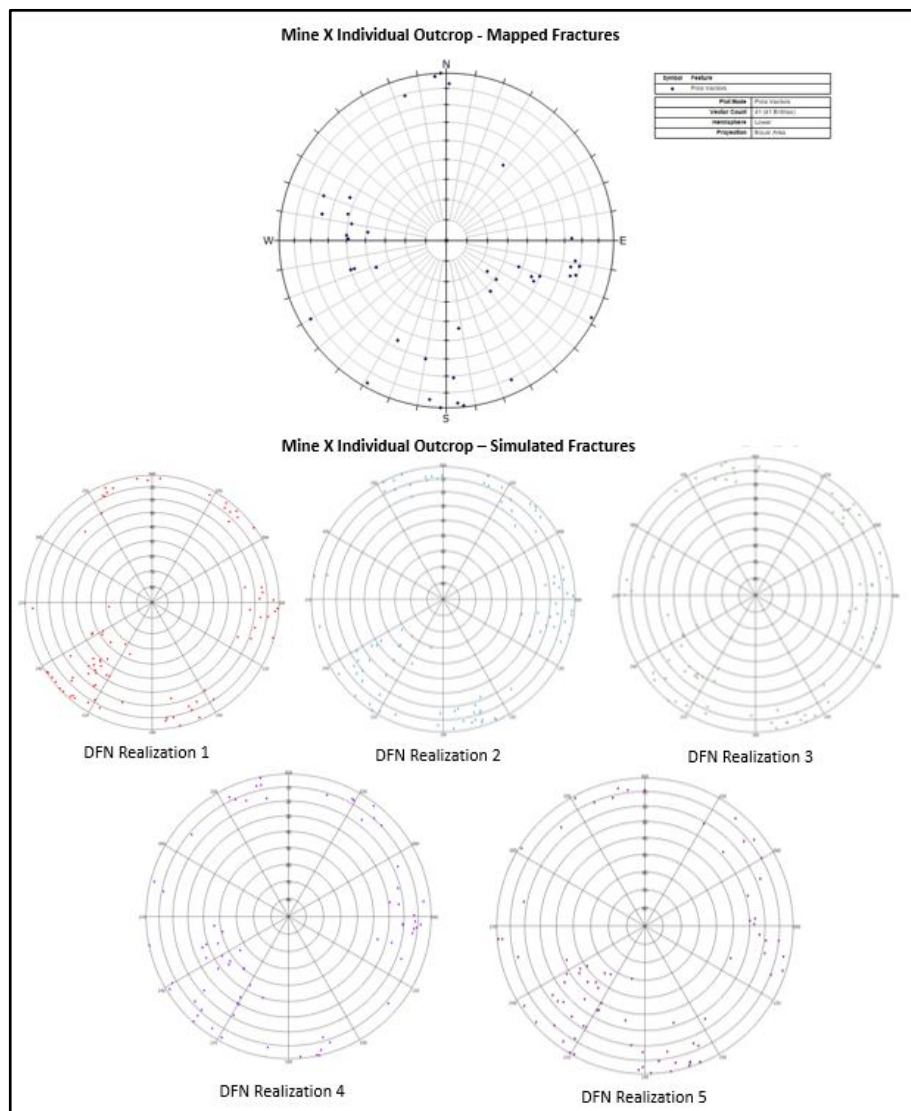


Figure 62: Orientation of mapped fractures from Mine X individual outcrop compared to the DFN simulated fractures – Trace Plane orientation of 80°/065°.

Figure 63 shows an example of the trace length distributions for each discontinuity set that was derived from (DFN realization 1) simulated fractures intersecting the trace plane oriented parallel to a field outcrop mapped. Although the type of distribution fit for the field data and simulated trace length data are the same, and the trace length ranges are similar; the mean values for the simulated trace lengths are higher as compared to the field mapping structures. This could be attributed to censoring bias and possible measurement errors during the data collection phase. It should be noted that the mean equivalent radius for the simulated fractures is similar to the mean derived from field data.

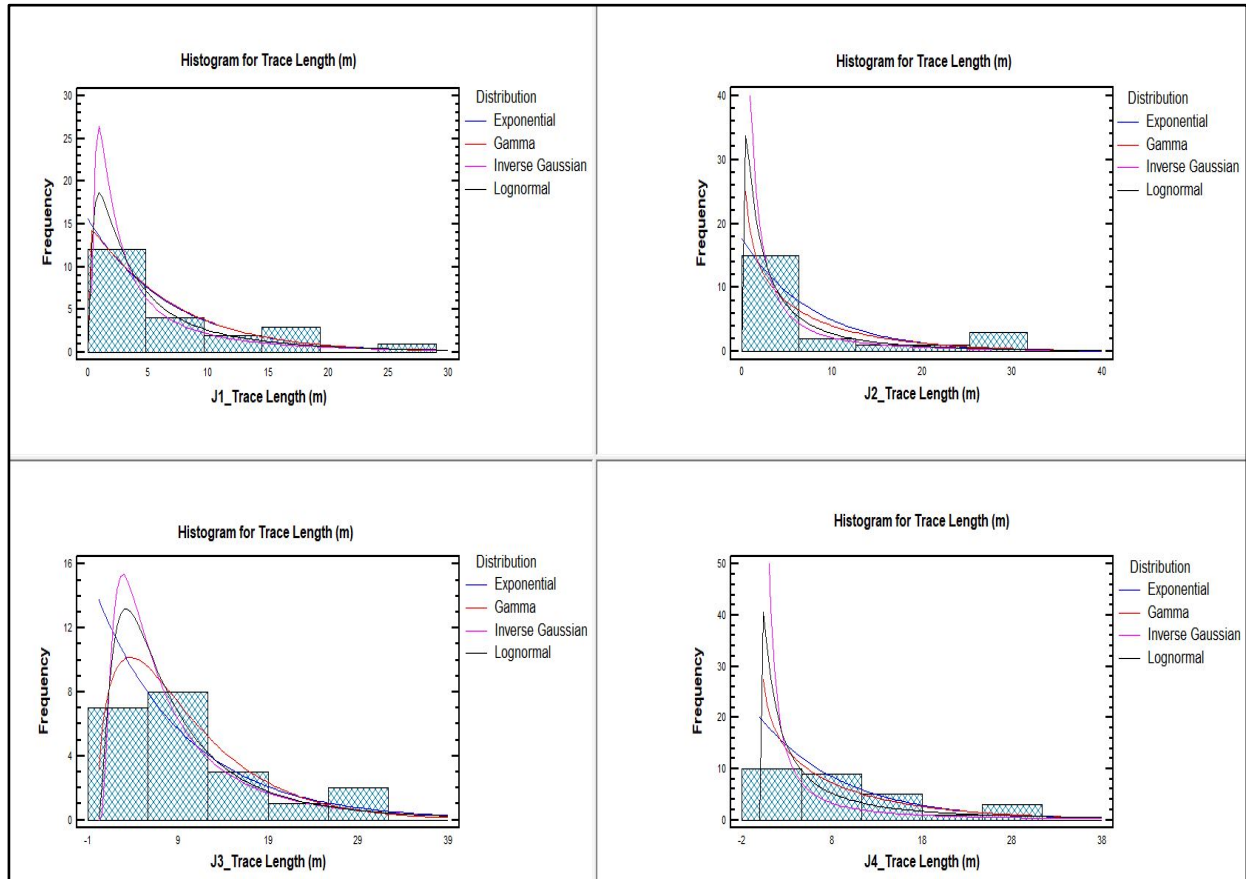


Figure 63: Histogram of Trace length for DFN Realization 1 – Trace plane oriented 80°/042°.

Figure 64 and Figure 59 are examples of graphs comparing the number of fractures per area calculated from the window mapping field data with corresponding trace plane. Fracture frequency assessments and visual comparisons of the spatial distribution of simulated fractures from sample trace planes orientated parallel with the corresponding bench face outcrop mapped would suggest that the DFN simulated fracture intensities are similar to that of the field mapping outcrops. However, it is noted that there are several field outcrops that have a higher fracture intensity as compared to their corresponding simulated fracture trace maps and vice versa. The increased fracture intensity observed in the field could be due to the presence of blasting induced fractures.

Furthermore, it's worth noting that the DFN block forming potential is very sensitive to the input fracture intensity parameter. The variation in the observed and simulated fracture intensity could influence the number of complete blocks that form in the pit slope and the related wedge stability analyses.

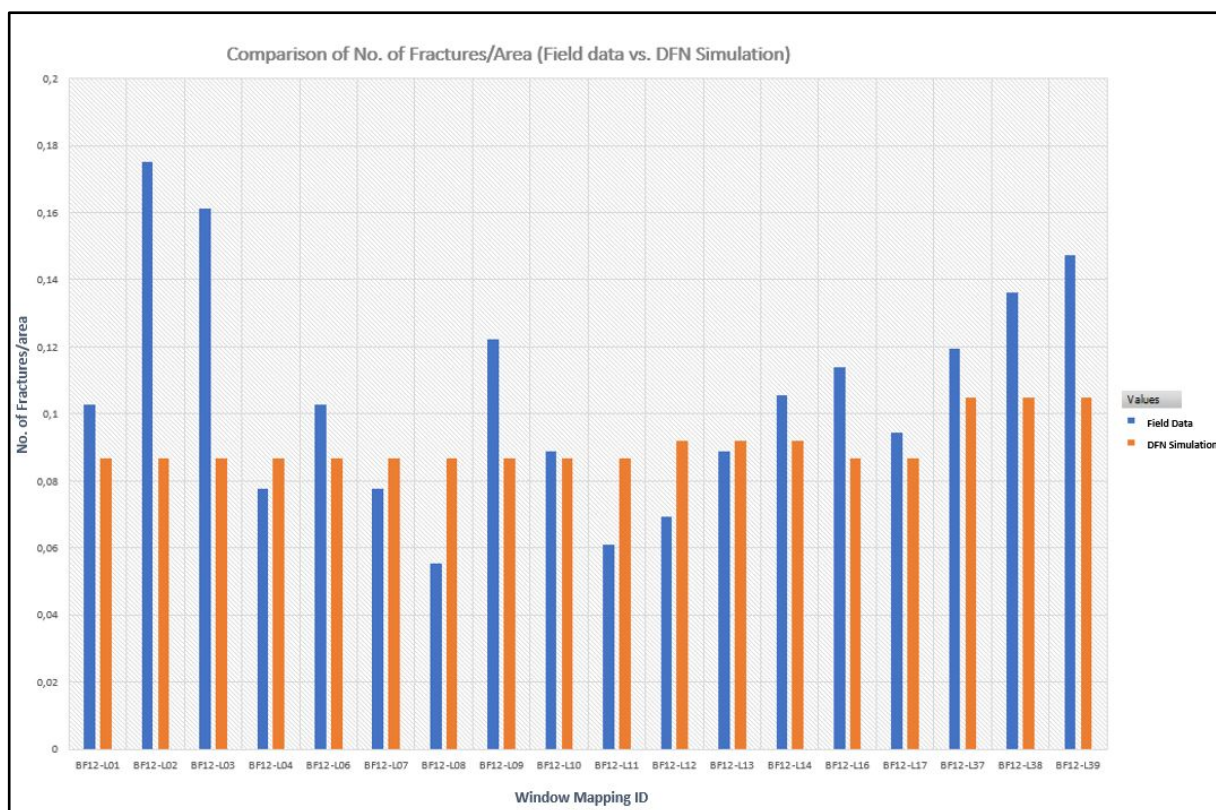


Figure 64: No. of Fractures/area – Comparison of Field data and DFN Simulations (DFN Realization 2).

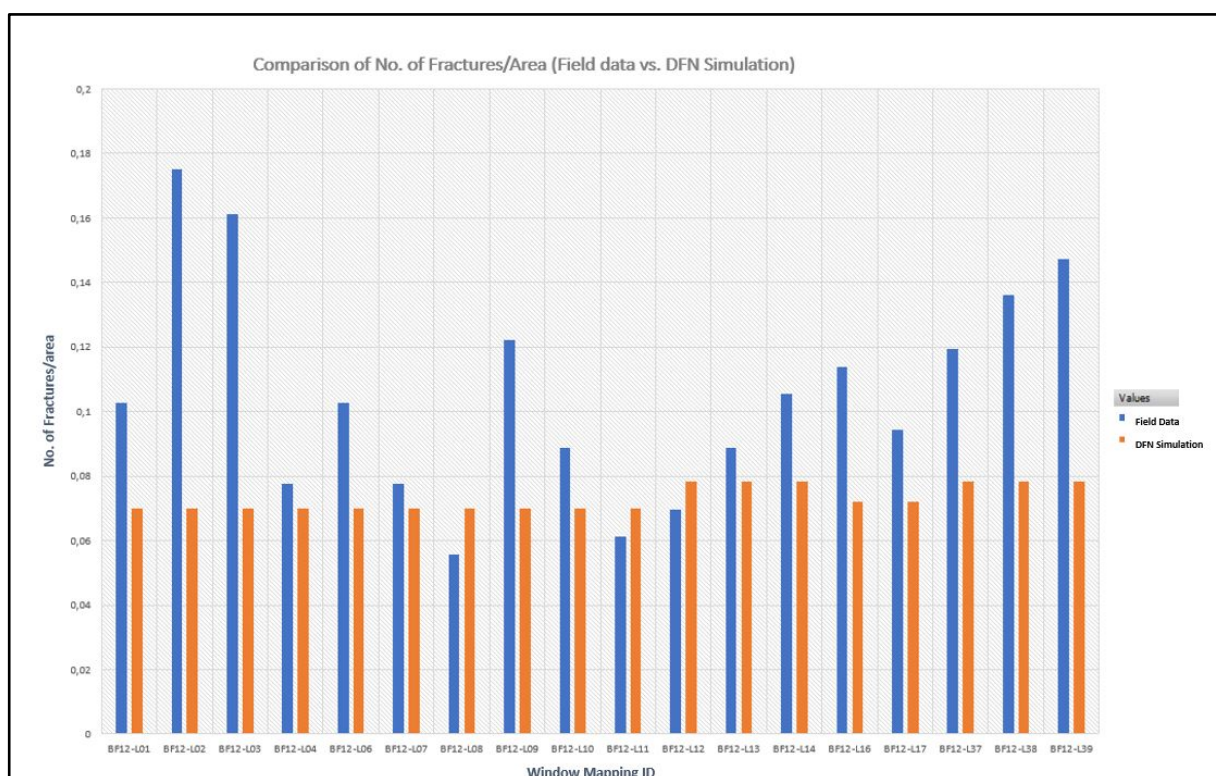


Figure 65: No. of Fractures/area – Comparison of Field data and DFN Simulations (DFN Realization 5).

5.4 Chapter Summary

The statistical analysis of the structural data was used as inputs for the generation of the DFN model. The trace length data from the Mine X mapping exercises can be described by a lognormal distribution. Fracture size parameters were determined from trace length statistical distributions and converted to an equivalent fracture radius using analytical methods. P32 intensity was derived from P21 data through simulation. The Fisher distribution was selected to model the orientation dispersion and the Enhanced Baecher model applied to the spatial distribution of fractures. Four sided polygons were selected to represent the fracture shape.

The DFN model was generated in FracMan using the basic geometric modelling method. A total of five DFN model realizations were generated using FracMan based on the assumed and derived primary DFN parameters. Bench-scale model regions is 30m x 30m x 15m. The simulated fractures in this bench-scale model region were validated by comparing the orientations, intensities, and trace length with field data. Although some variations were observed between the simulated data and field mapping data, in general, the DFN model is in reasonable agreement with the Mine X field observations.

This chapter summarises the DFN model generation and validation process and is supplemented with details provided in APPENDIX D.

CHAPTER 6

6.0 WEDGE STABILITY ANALYSES AND RESULTS

This chapter presents the wedge stability limit equilibrium analyses carried out using ©Rocscience SWedge and ©Golder FracMan software packages for Mine X pit benches. The wedges that were identified during the kinematic stereographic assessment presented in chapter 4.0, section 4.2 were further assessed deterministically and probabilistically using Swedge. The DFN models described in Chapter 5.0 were used to identify and assess the wedges using the FracMan rock wedge function described in Section 2.4.4. The wedge stability analyses using SWedge are also compared to that of the FracMan DFN's rock wedge results. This chapter summarises the results for each wedge stability analysis. It should be noted that the implications of the wedge results on the bench design is provided in the next chapter (Chapter 7.0).

6.1 SWedge Stability Results

SWedge by ©Rocscience was used to calculate the factor of safety and critical wedge volumes for the potential wedges that could form by the intersection of two discontinuity sets in the pit walls at Mine X. The limit equilibrium wedge stability analysis using SWedge considered the following:

- Bench face angles of 90, 80 and 70 degrees.
- Bench height of 15m;
- Bench scale slope surfaces with slope face directions corresponding to the east, west, south, southwest, northeast, and northwest pit wall sectors of Mine X.
- Constant shear strength parameters (Mohr-Coulomb model, $C = 0\text{kPa}$, $\phi = 30^\circ$).
- Unit weight of 2700 kg/m^3 .
- No water pressure, seismicity or tension crack considered.
- Deterministic and probabilistic analyses.

The deterministic analysis results for Mine X case study, considering a bench face angle of 90 degrees is summarised in Table 10. Based on the deterministic results, the largest potential unstable wedge shown in Figure 66, (formed by J3 & J4), was identified in the South sector pit wall of Mine X. The main sliding of this asymmetric wedge is expected to occur along one joint plane with release planes on either end. This is consistent with the slope instabilities observed in the field at the South sector of the pit.

Table 10: Swedge Deterministic Results (BFA of 90 degrees)

Pit Sector	Pit Wall Mean Dip Direction	Wedge Information					
		Joint Set Combination		Trend (°)	Plunge (°)	Volume (m³)	FOS
East Sector	295°	J1	J2	309	72	130.17	0.37
		J1	J3	293	87	1.99	0.06
		J2	J3	261	75	102.06	0.15
		J2	J4	352	38	625.88	1.25
South Sector	040°	J2	J4	352	38	3549.56	1.25
		J3	J4	72	49	3769.41	0.45
North East Sector	218°	J1	J3	293	87	1.61	0.01
		J1	J4	132	04	1638.67	24.96
		J2	J3	261	75	31.49	0.20
North West Sector	147°	J3	J4	072	49	84.87	0.75
West Sector	085°	J3	J4	072	49	468.99	0.75
South West Sector	352°	J1	J2	309	72	33.74	0.37
		J1	J3	293	87	3.68	0.02
		J2	J4	352	38	214.67	1.25
		J3	J4	072	49	108.01	0.45

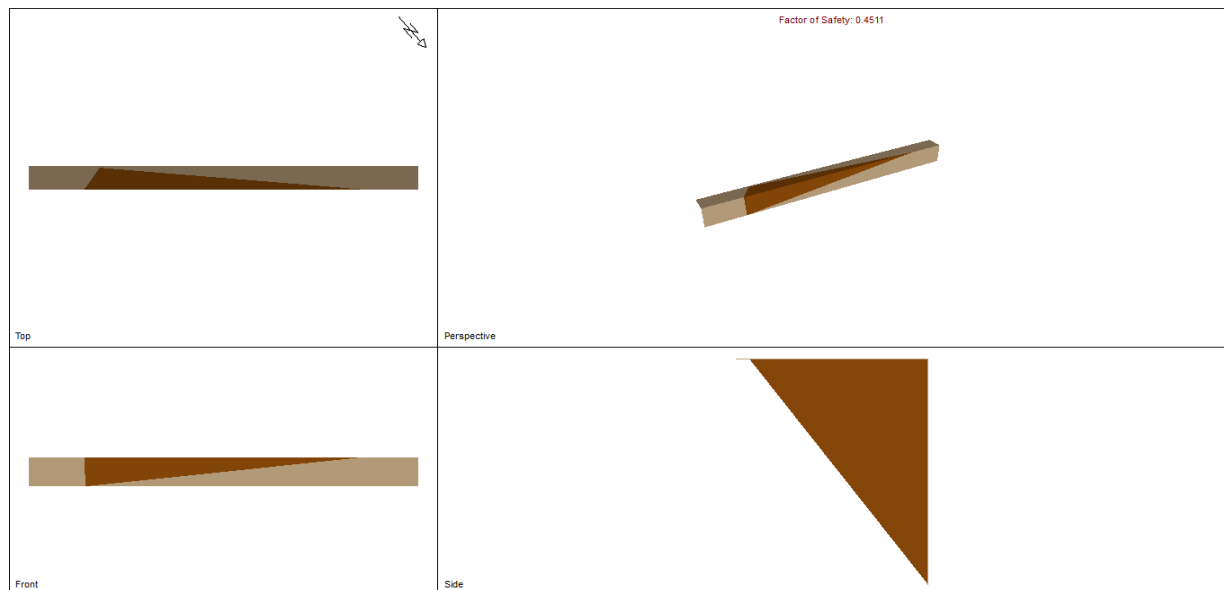


Figure 66: Bench-scale wedge formed by J3 and J4 within the south pit wall (slope dip direction 040°).

The probabilistic option available in SWedge© allows for the input of statistical data that can account for the uncertainty in joint orientation and joint strength values. The probability of failure is calculated from the factor of safety distribution. For the Mine X probability assessment, the shear strength values were kept constant and only the joint orientation were defined as a random variable. An example of the probabilistic result, Figure 67 shows the factor of safety distribution and the calculated probability of failure with all remaining results provided in APPENDIX E. Table 11 summarises the SWedge

probabilistic results for Mine X case study for a BFA of 90 degrees. The probabilistic results indicate that most of the wedges formed have a high probability of failure.

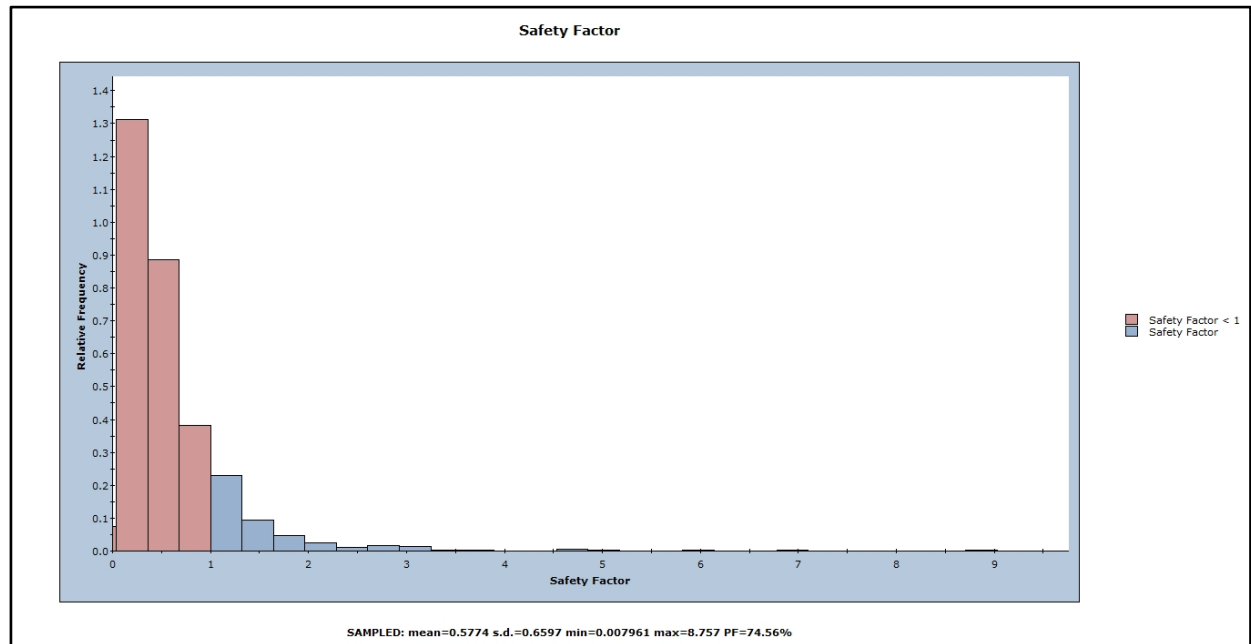


Figure 67: Probabilistic result: FOS Histogram for joint set combination J1 and J2 within the east pit wall (slope dip direction 295°) considering a FOS < 1.

Table 11: SWedge Probabilistic Results (BFA of 90 degrees)

	Pit Wall Mean Dip Direction	Wedge Information				Probability of Failure (PoF)				
		Joint Set Combination		Trend (°)	Plunge (°)	FOS < 1.0	FOS < 1.1	FOS < 1.2	FOS < 1.3	FOS < 1.5
East Sector	295°	J1	J2	309	72	0.75	0.77	0.79	0.81	0.83
		J1	J3	293	87	0.56	0.56	0.57	0.57	0.57
		J2	J3	261	75	0.89	0.89	0.90	0.90	0.90
		J2	J4	352	38	0.32	0.39	0.46	0.51	0.59
South Sector	040°	J2	J4	352	38	0.47	0.51	0.55	0.59	0.65
		J3	J4	72	49	0.76	0.81	0.84	0.86	0.89
North East Sector	218°	J1	J3	293	87	0.51	0.51	0.51	0.51	0.52
		J1	J4	132	04	0.04	0.04	0.04	0.04	0.04
		J2	J3	261	75	0.76	0.77	0.77	0.77	0.78
North West Sector	147°	J3	J4	072	49	0.49	0.54	0.58	0.60	0.66
West Sector	085°	J3	J4	072	49	0.68	0.75	0.78	0.81	0.86
South West Sector	352°	J1	J2	309	72	0.64	0.67	0.69	0.70	0.72
		J1	J3	293	87	0.56	0.56	0.56	0.56	0.57
		J2	J4	352	38	0.35	0.40	0.47	0.52	0.61
		J3	J4	072	49	0.73	0.74	0.78	0.81	0.86

The combination analysis in SWedge© allows the user to consider all possible combinations from joint orientation data (Rocscience, 2021). An example of these results for the combination analysis are provided in APPENDIX E.

6.2 FracMan Rock Wedge Results

The rock wedge analysis was carried out using the discrete fracture networks modelled with defined geometric and spatial characteristics. The “Rock Wedge Analysis” function available in FracMan was used to evaluate the wedge stability for the various bench slope faces assessed for Mine X. The rock wedge limit equilibrium analysis was carried out considering the following:

- Bench face angles 90, 80 and 70 degrees.
- Bench scale slope surfaces with slope face directions corresponding to the east, west, south, southwest, northeast, and northwest pit wall sectors of Mine X.
- All DFN model realizations (1 to 5).
- Constant shear strength parameters (Mohr-Coulomb model, $C = 0\text{kPa}$, $\phi = 30^\circ$).
- Unit weight of 2700 kg/m^3 .
- No water pressure and seismicity considered.
- Fracture stress not applied.

The FracMan wedge stability modelling was carried out with the same input considerations as Swedge in section 6.1 for each DFN realization. APPENDIX D provides all results from the FracMan rock wedge analysis for each DFN realization. The rock wedge stability results are presented as follows:

- Graphs showing the number of total blocks formed in the selected bench slope face and the number of stable, unstable, and freefalling blocks. Examples are shown in Figure 68, Figure 69, Figure 70, Figure 71, and Figure 72 for all DFN model realizations and a BFA of 90 degrees.
- Graphs showing the FOS category results, expressed as a relative percentage for each bench scenario and realization. Examples are shown in Figure 73, Figure 76, Figure 77, Figure 74, and Figure 75 for DFN model realizations 1 to 5 considering a BFA of 90 degrees.
- Graphs showing the FOS categories and the maximum wedge volume for each bench scenario and realization. Examples are shown in Figure 78, Figure 79, Figure 80, Figure 81, and Figure 82 for all DFN model realizations considering a BFA of 90 degrees.
- FracMan rock wedge analysis result image that shows stable and unstable blocks with the trace map overlayed for each bench and pit wall evaluated. Figure 83, Figure 84, Figure 87, Figure 88, Figure 89, Figure 90, Figure 91, Figure 92, Figure 93, Figure 94, Figure 85, Figure 86 and Figure 95 show examples of these results.

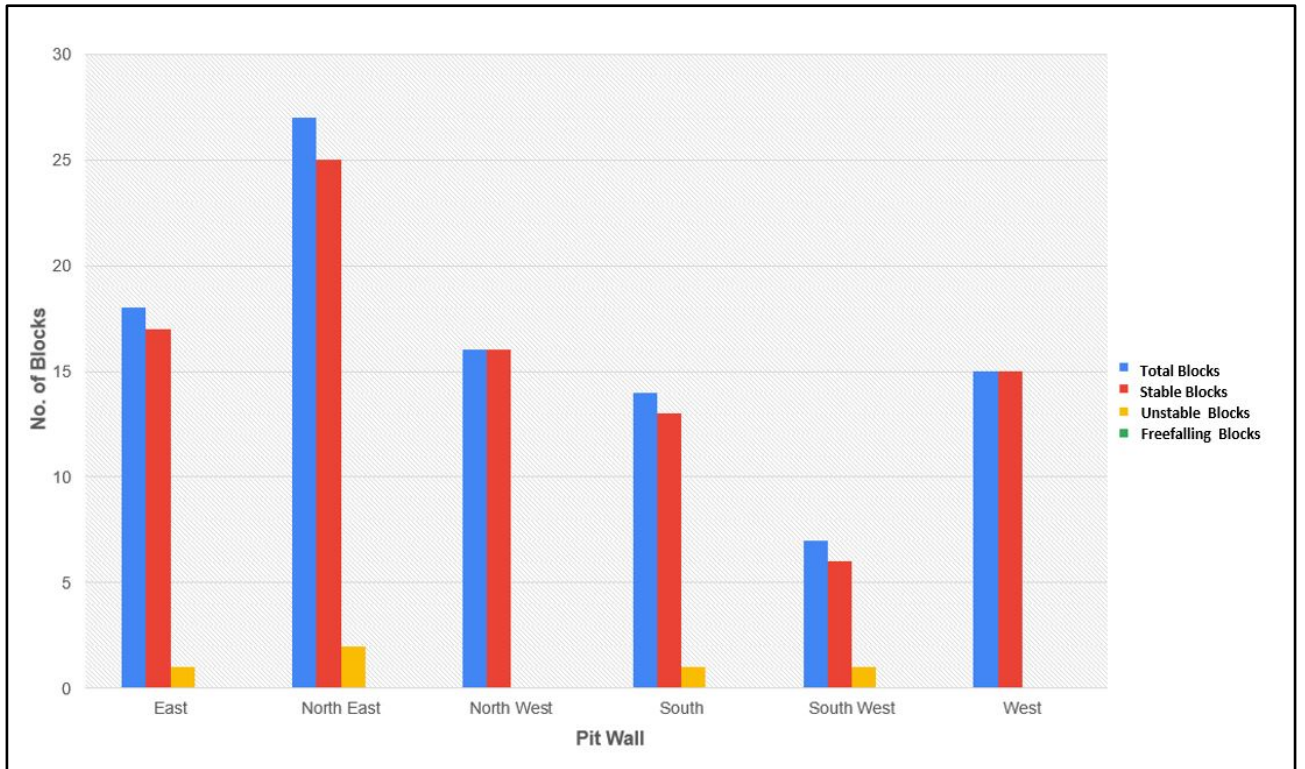


Figure 68: DFN Realization 1 - Graph showing the no. of blocks identified within the various pit wall benches with a BFA of 90 degrees.

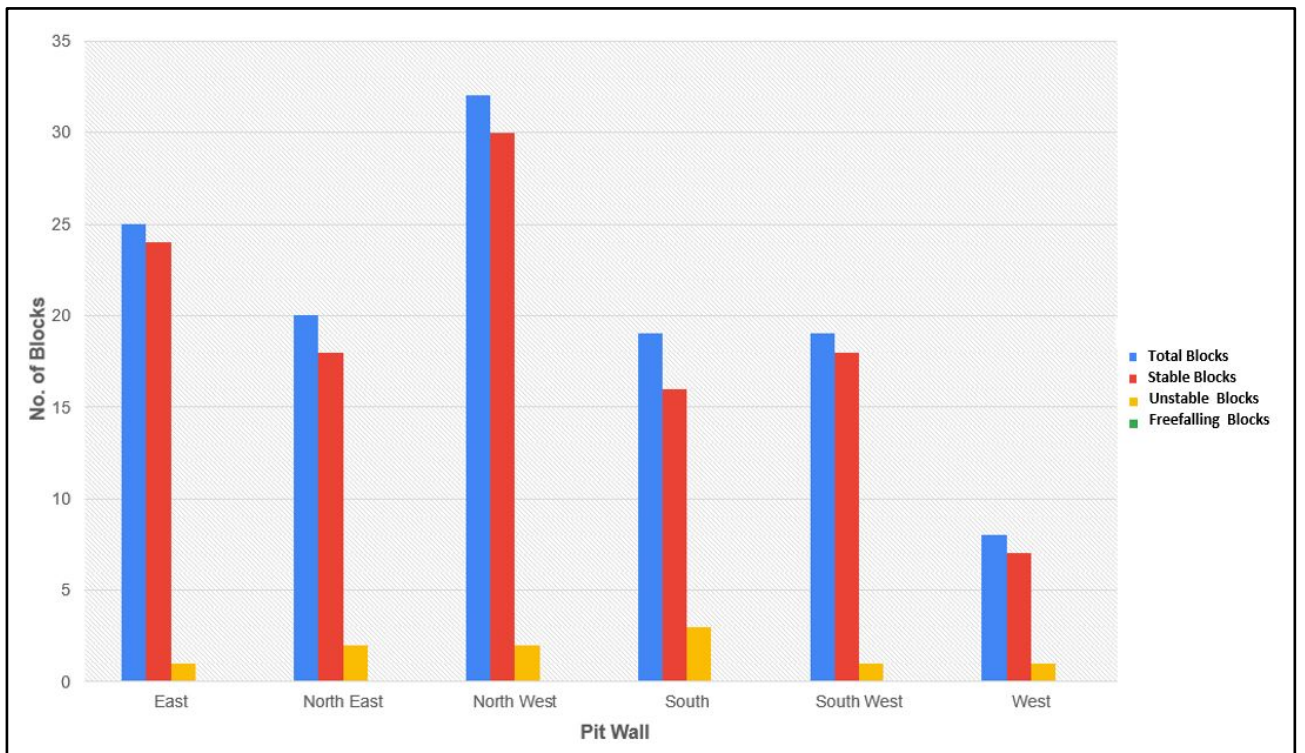


Figure 69: DFN Realization 2 - Graph showing the no. of blocks identified within the various pit wall benches with a BFA of 90 degrees.

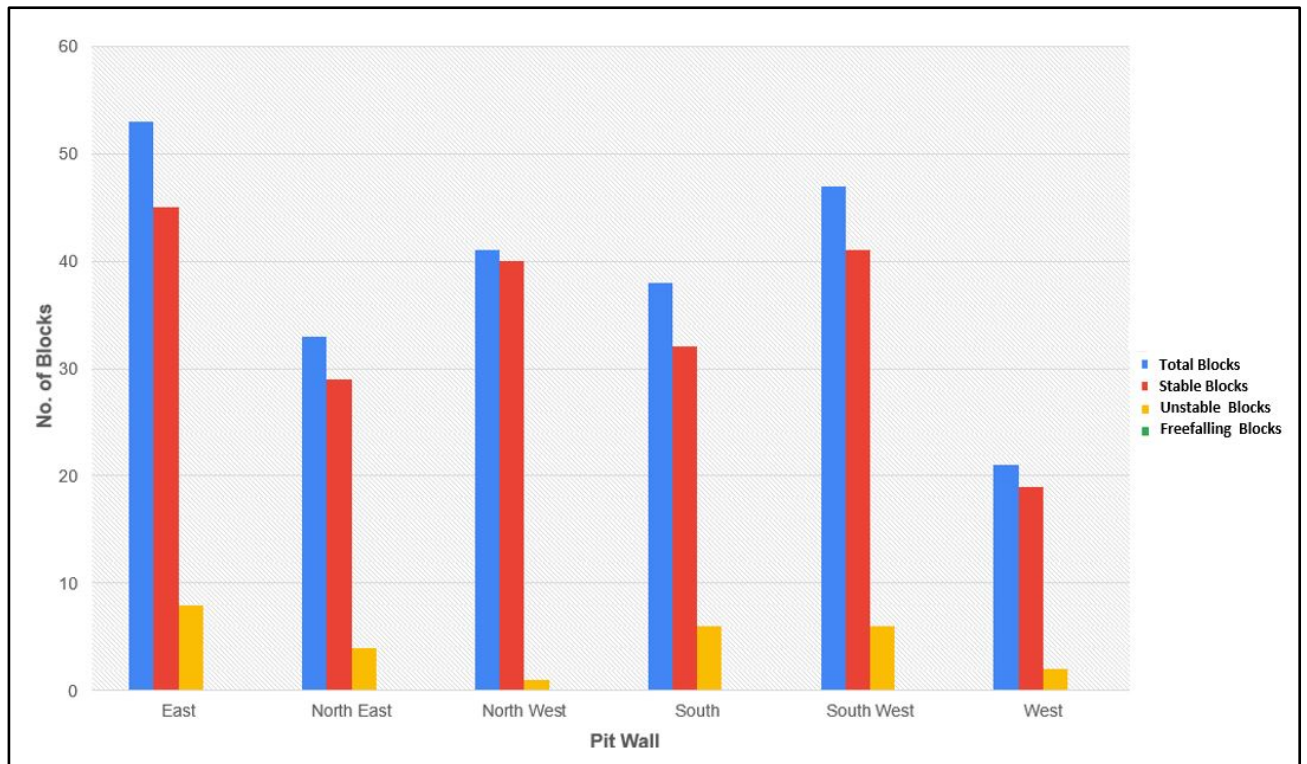


Figure 70: DFN Realization 3 - Graph showing the no. of blocks identified within the various pit wall benches with a BFA of 90 degrees.

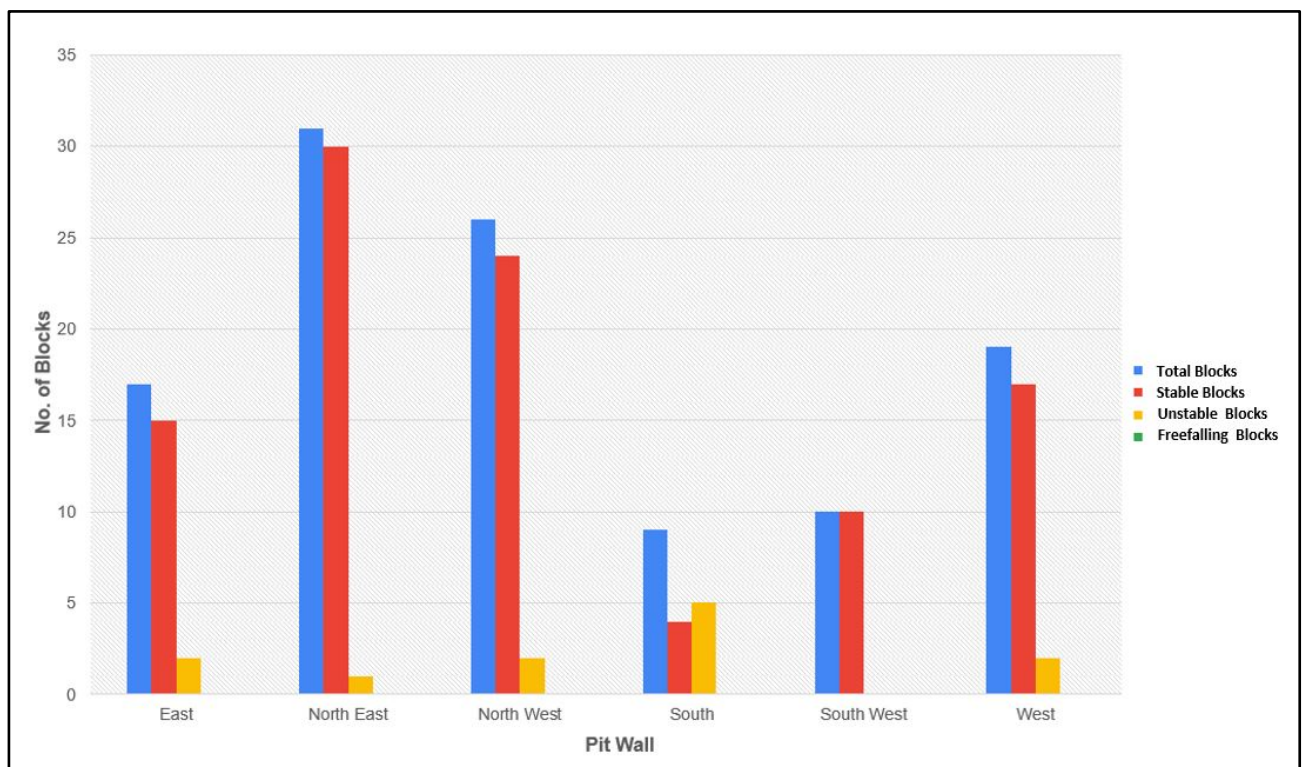


Figure 71: DFN Realization 4 - Graph showing the no. of blocks identified within the various pit wall benches with a BFA of 90 degrees.

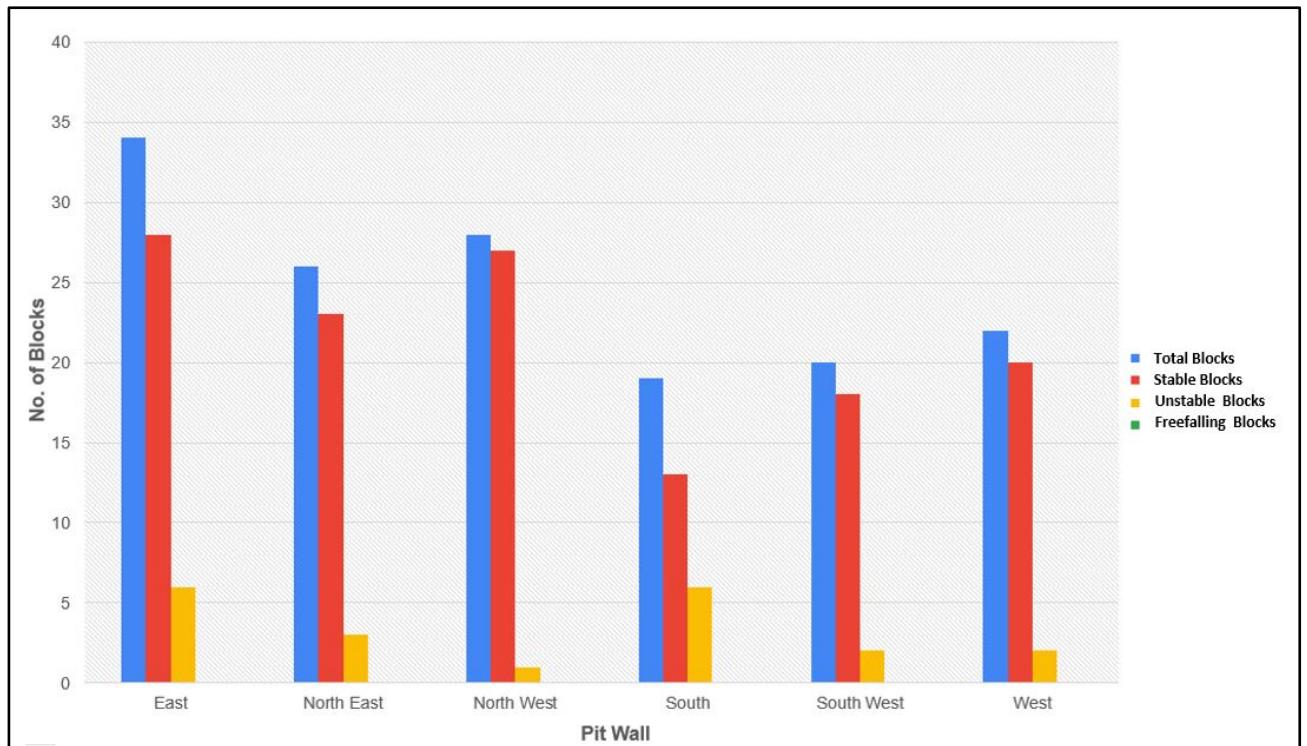


Figure 72: DFN Realization 5 - Graph showing the no. of blocks identified within the various pit wall benches with a BFA of 90 degrees.

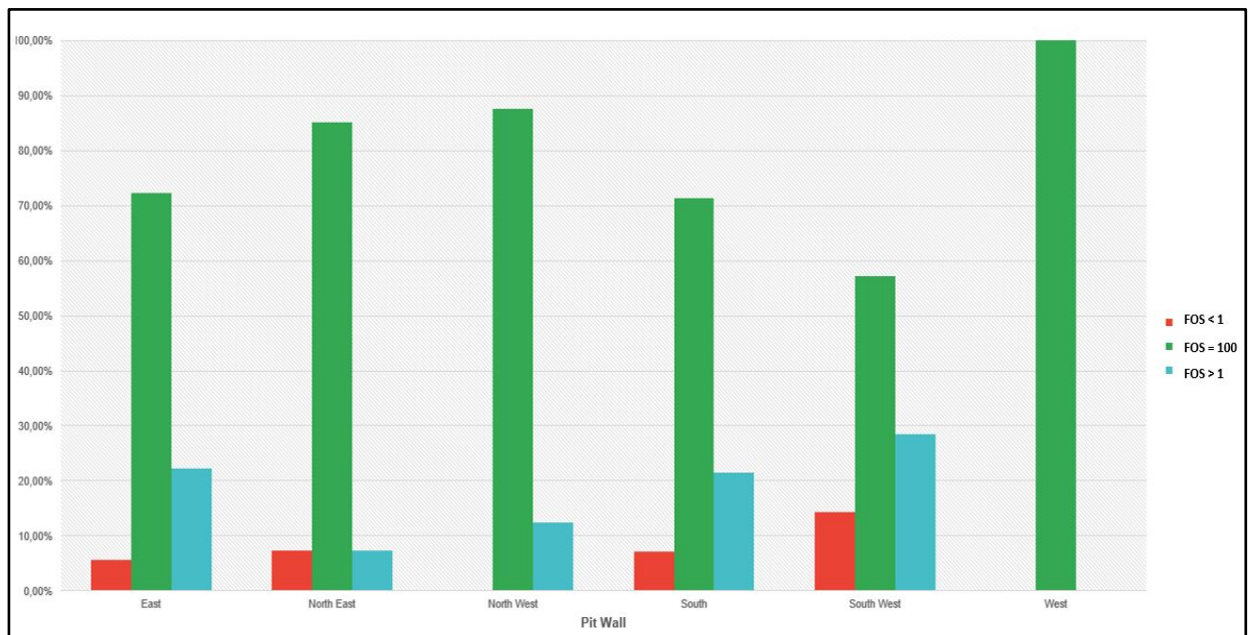


Figure 73: DFN Realization 1 - Graph showing the FOS category results that indicate the stability of blocks identified within the various pit wall benches with a BFA of 90 degrees.

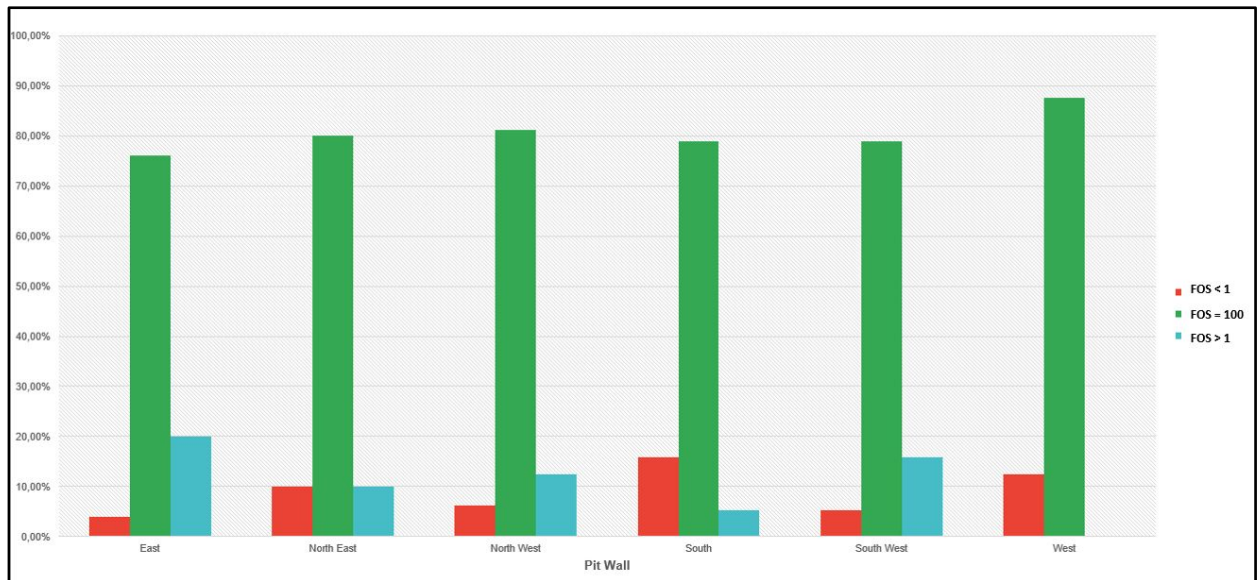


Figure 74: DFN Realization 2 - Graph showing the FOS category results that indicate the stability of blocks identified within the various pit wall benches with a BFA of 90 degrees.

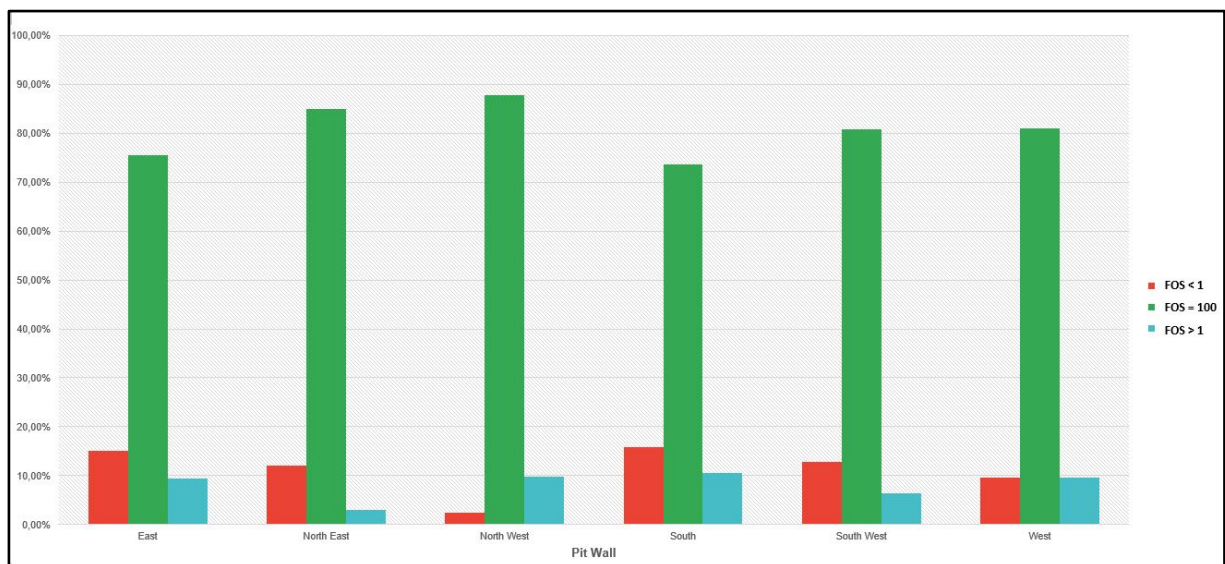


Figure 75: DFN Realization 3 - Graph showing the FOS category results that indicate the stability of blocks identified within the various pit wall benches with a BFA of 90 degrees.

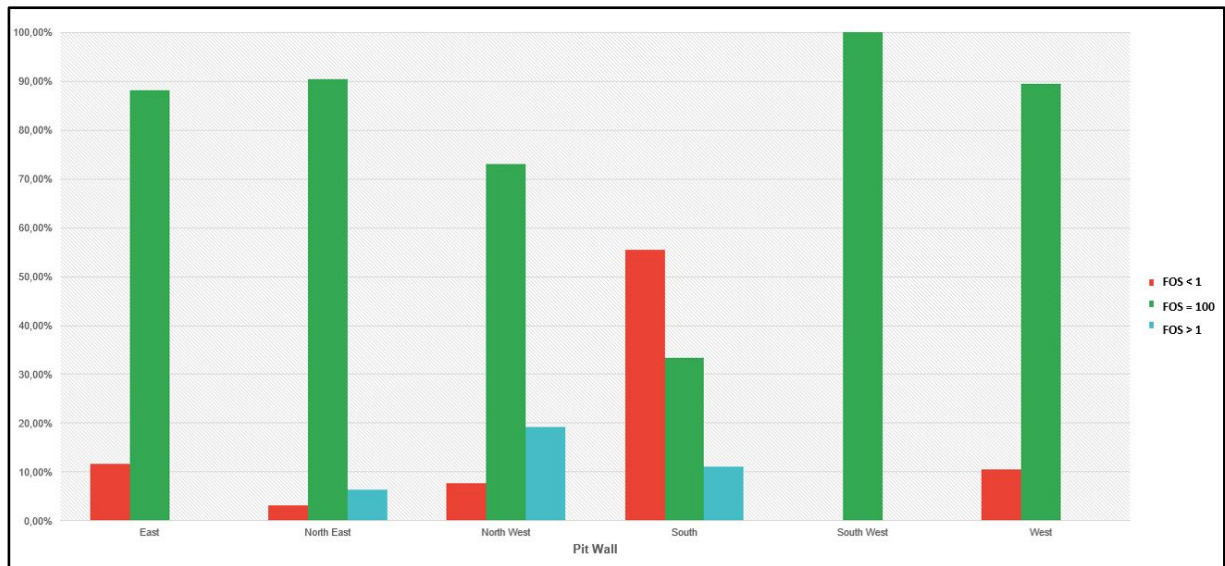


Figure 76: DFN Realization 4 - Graph showing the FOS category results that indicate the stability of blocks identified within the various pit wall benches with a BFA of 90 degrees.

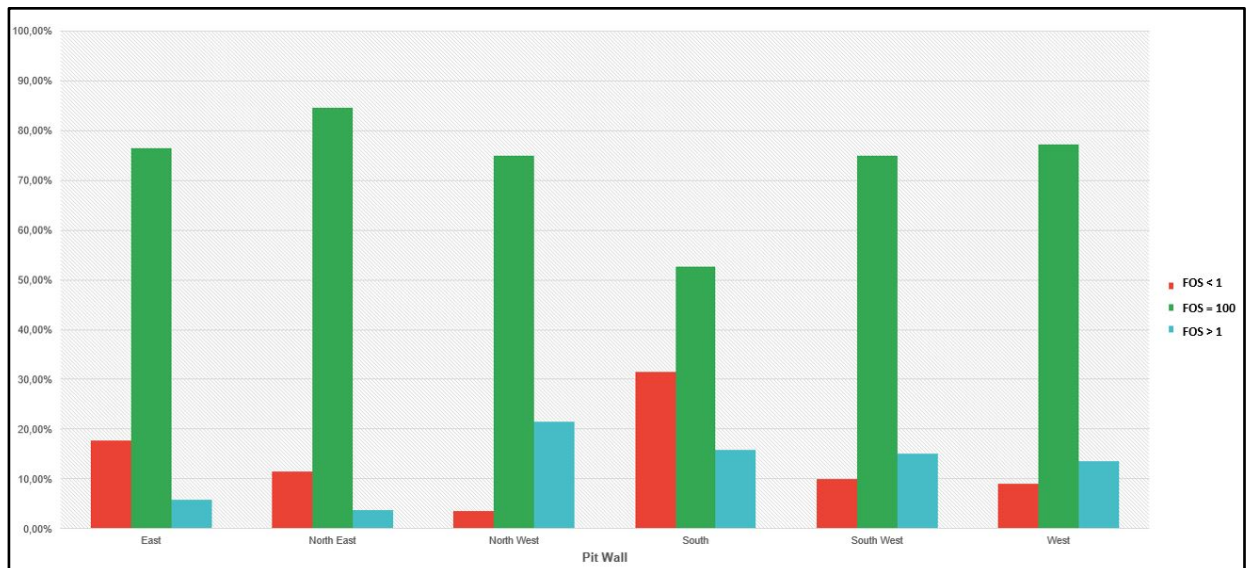


Figure 77: DFN Realization 5 - Graph showing the FOS category results that indicate the stability of blocks identified within the various pit wall benches with a BFA of 90 degrees.

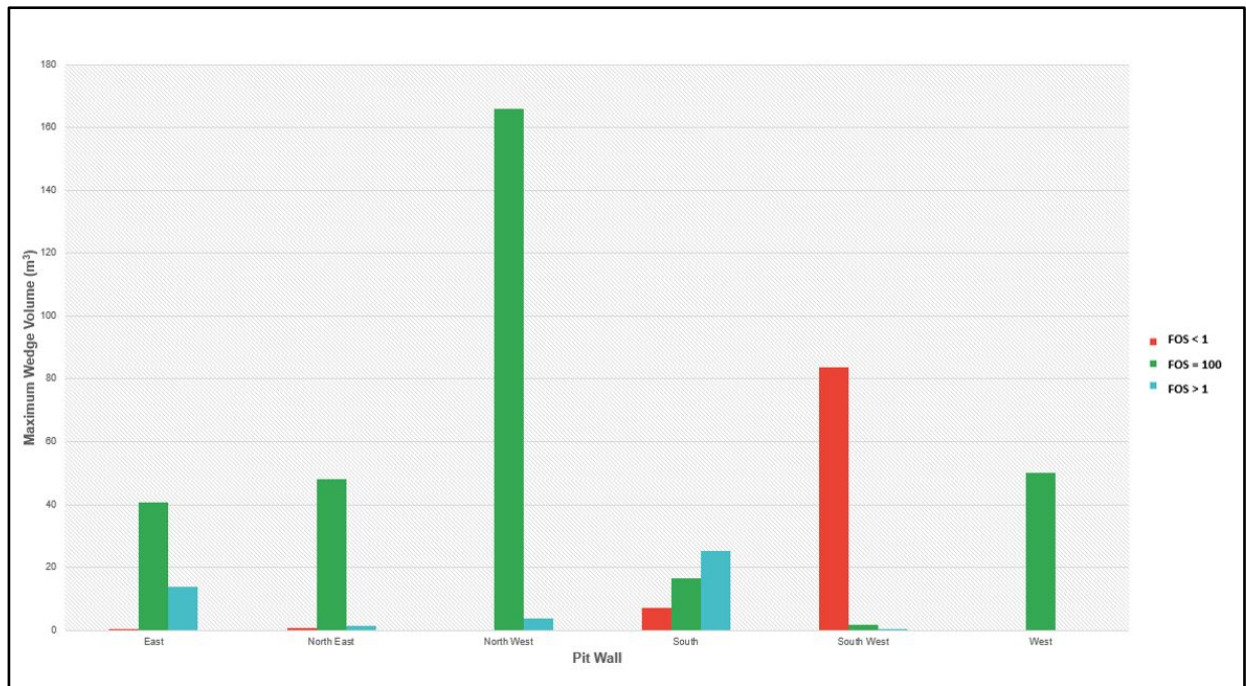


Figure 78: DFN Realization 1 - Graph showing the FOS category results with the maximum wedge volume for the various pit wall benches considering a BFA of 90 degrees.

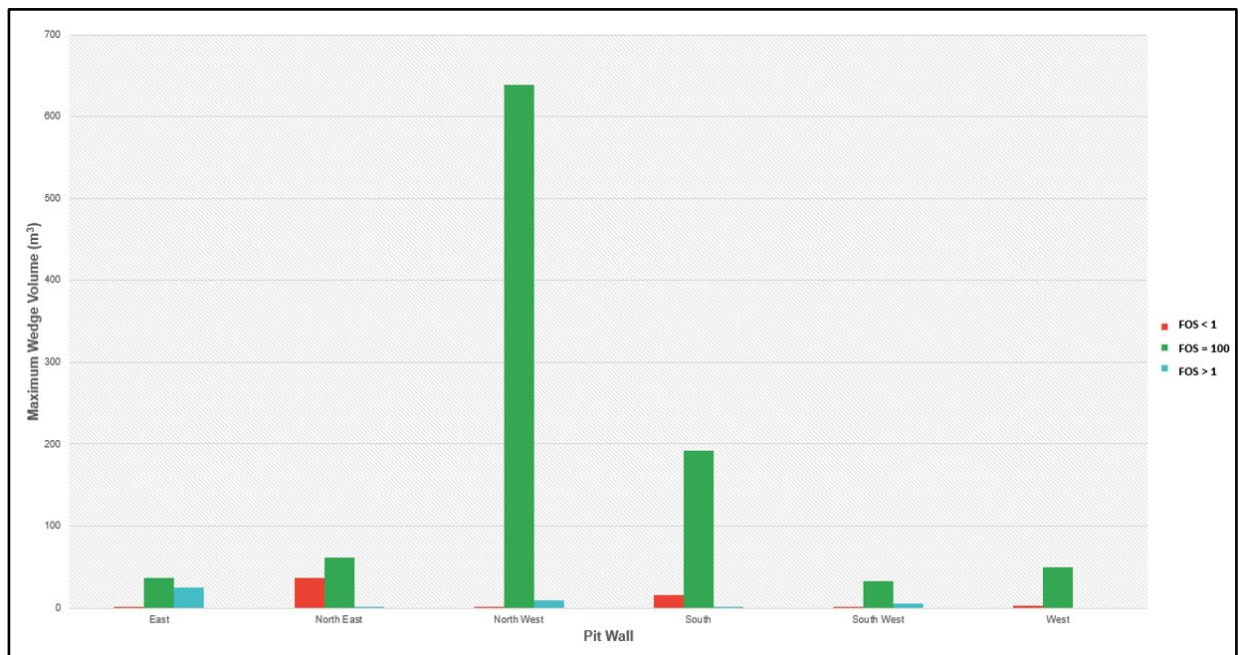


Figure 79: DFN Realization 2 - Graph showing the FOS category results with the maximum wedge volume for the various pit wall benches considering a BFA of 90 degrees.

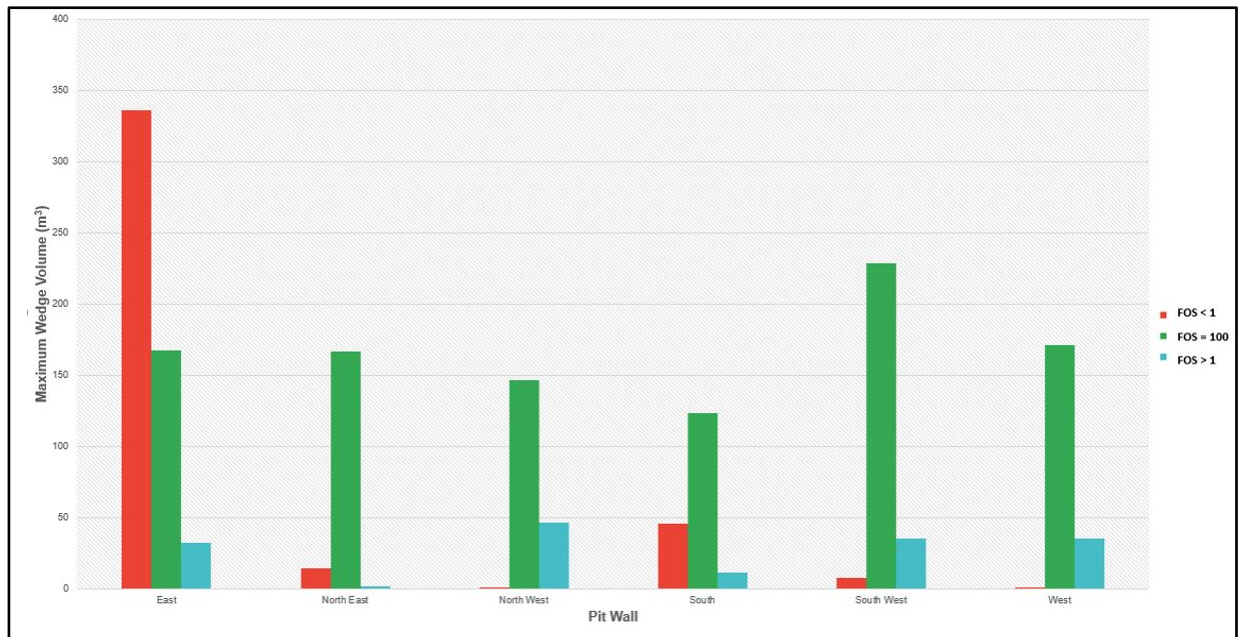


Figure 80: DFN Realization 3 - Graph showing the FOS category results with the maximum wedge volume for the various pit wall benches considering a BFA of 90 degrees.

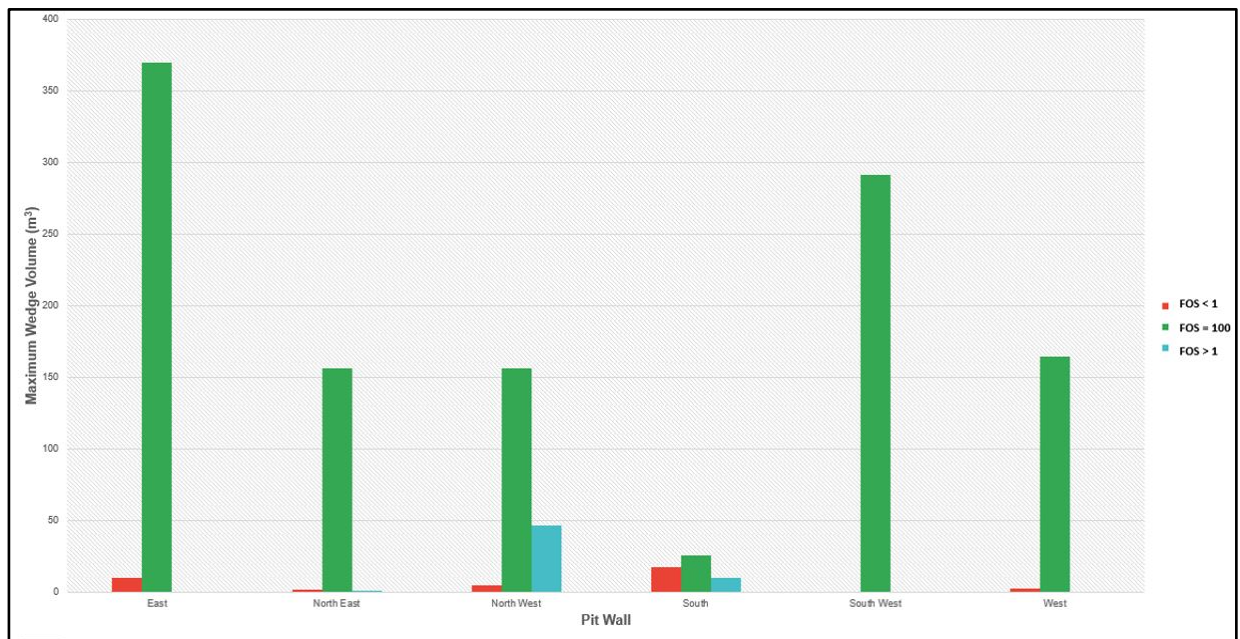


Figure 81: DFN Realization 4 - Graph showing the FOS category results with the maximum wedge volume for the various pit wall benches considering a BFA of 90 degrees.

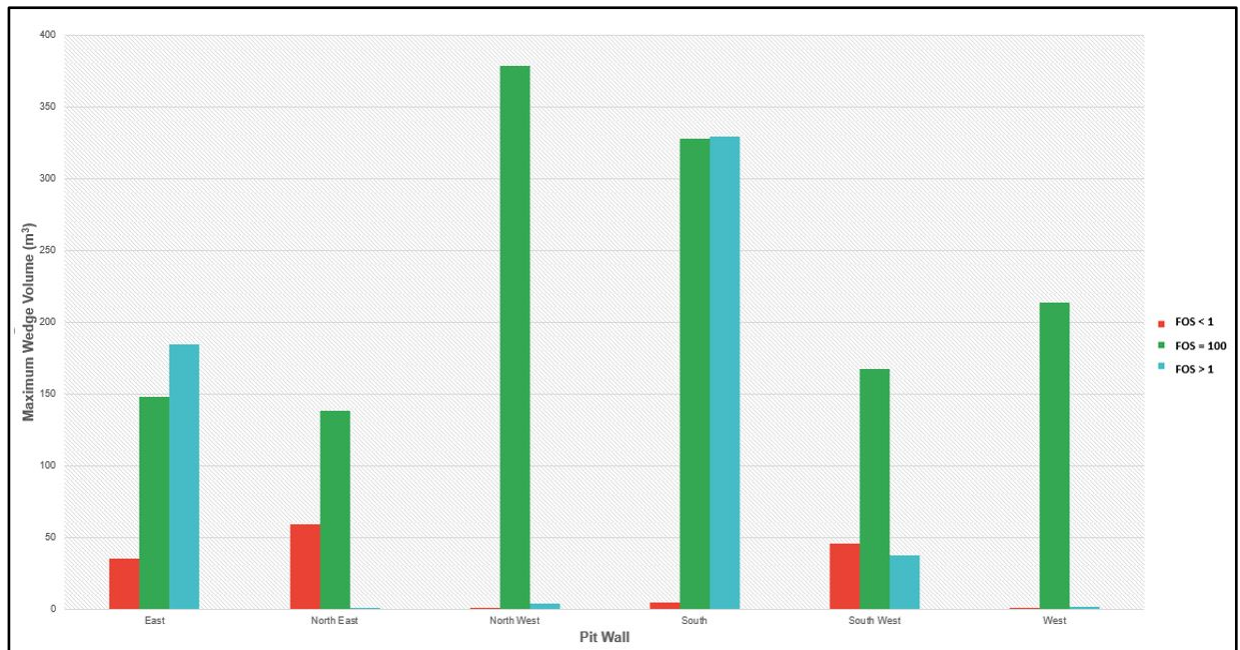


Figure 82: DFN Realization 5 - Graph showing the FOS category results with the maximum wedge volume for the various pit wall benches considering a BFA of 90 degrees.

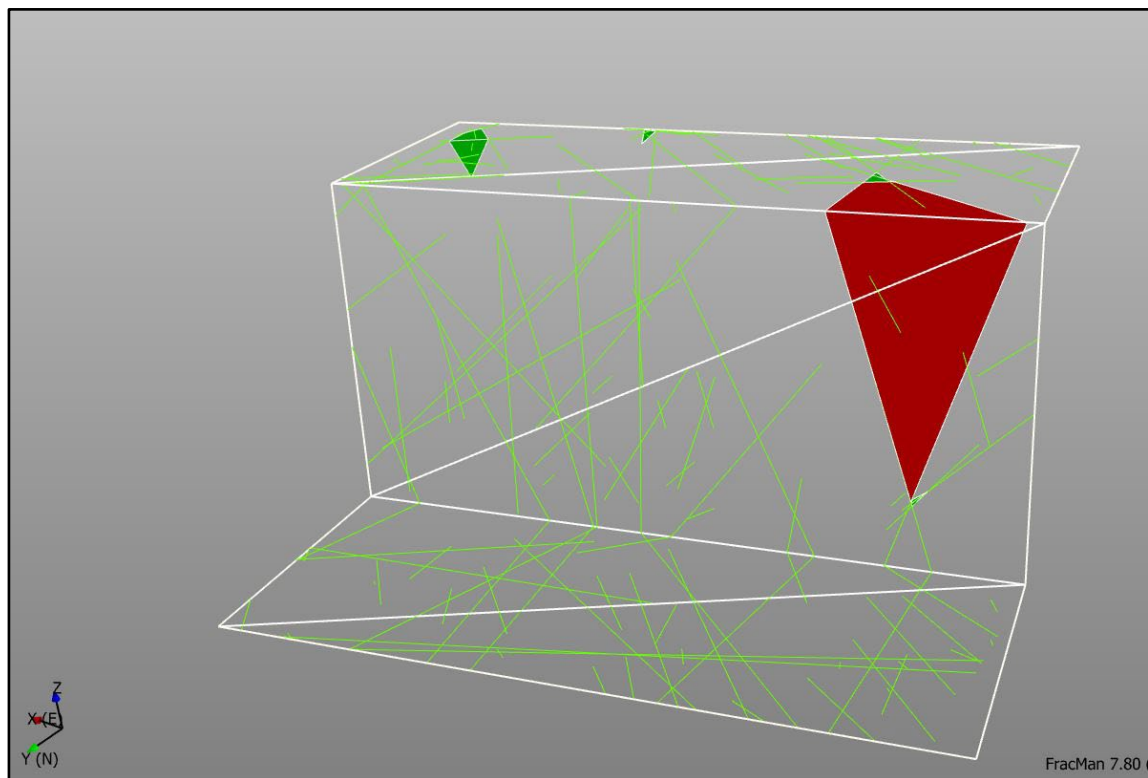


Figure 83: Rock Wedge Results: DFN Realization 1, Southwest Pit Wall, BFA of 90 degrees.

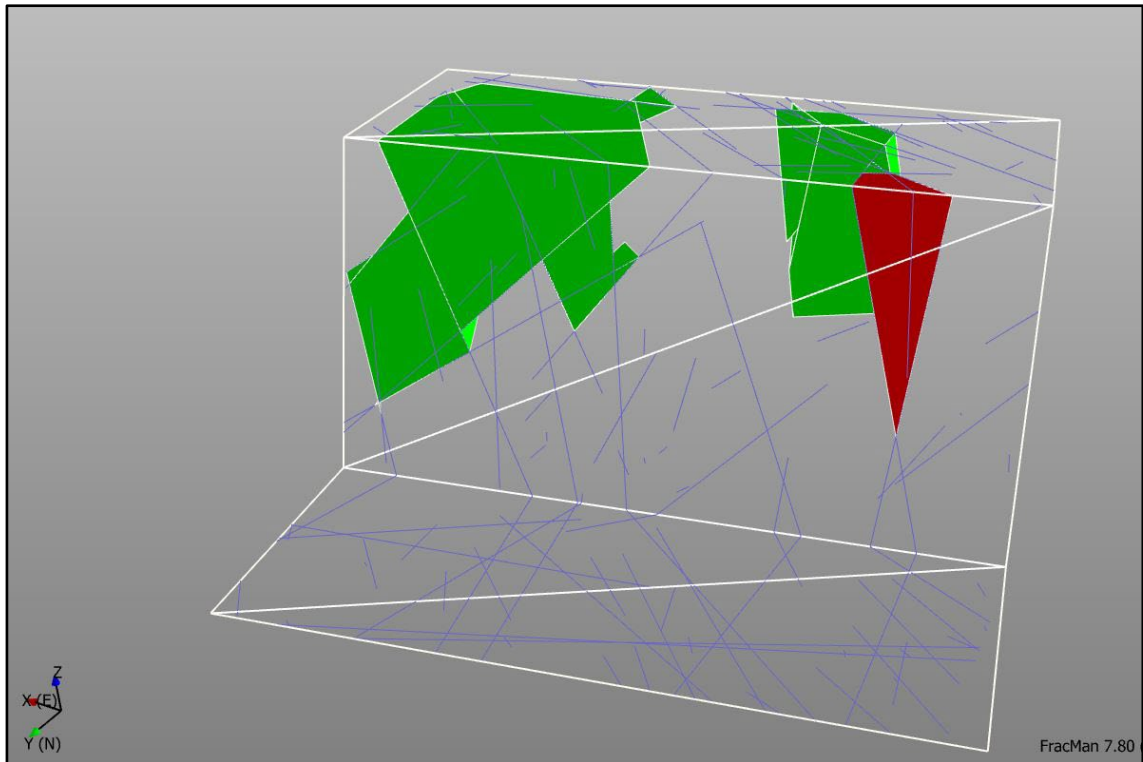


Figure 84: Rock Wedge Results: DFN Realization 1, Southwest Pit Wall, BFA of 80 degrees.

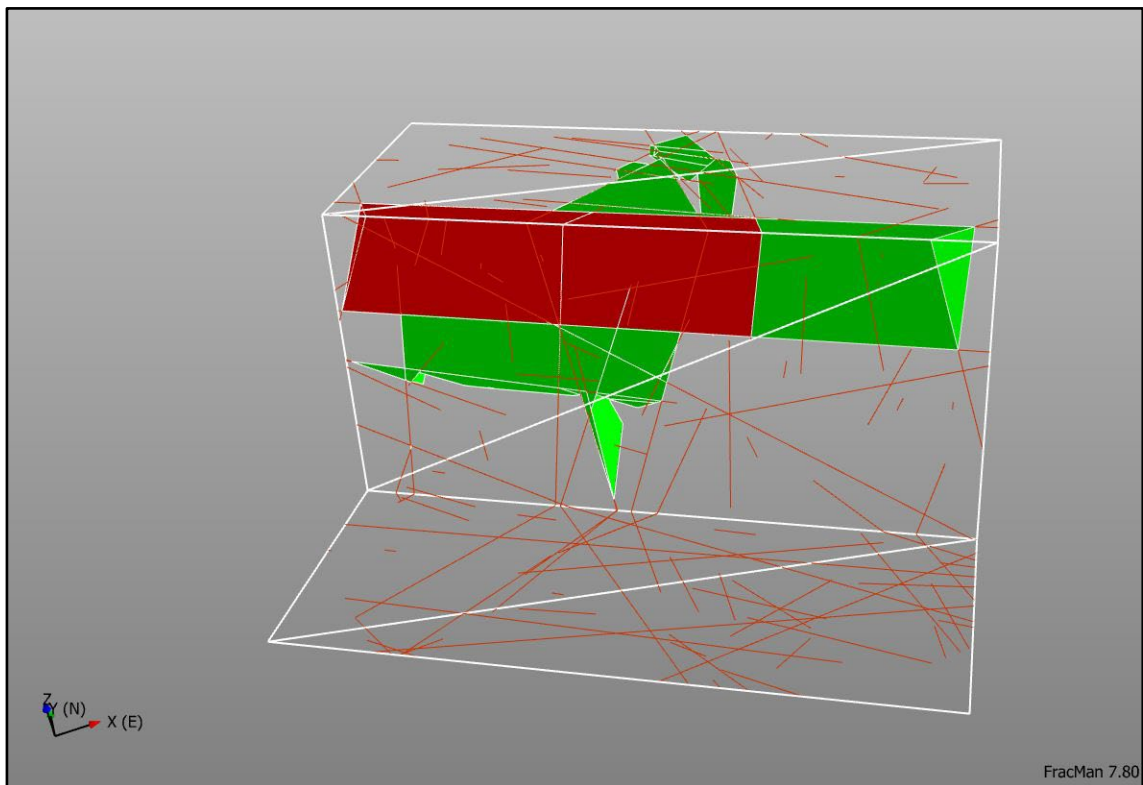


Figure 85: Rock Wedge Results: DFN Realization 2, Northeast Pit Wall, BFA of 90 degrees.

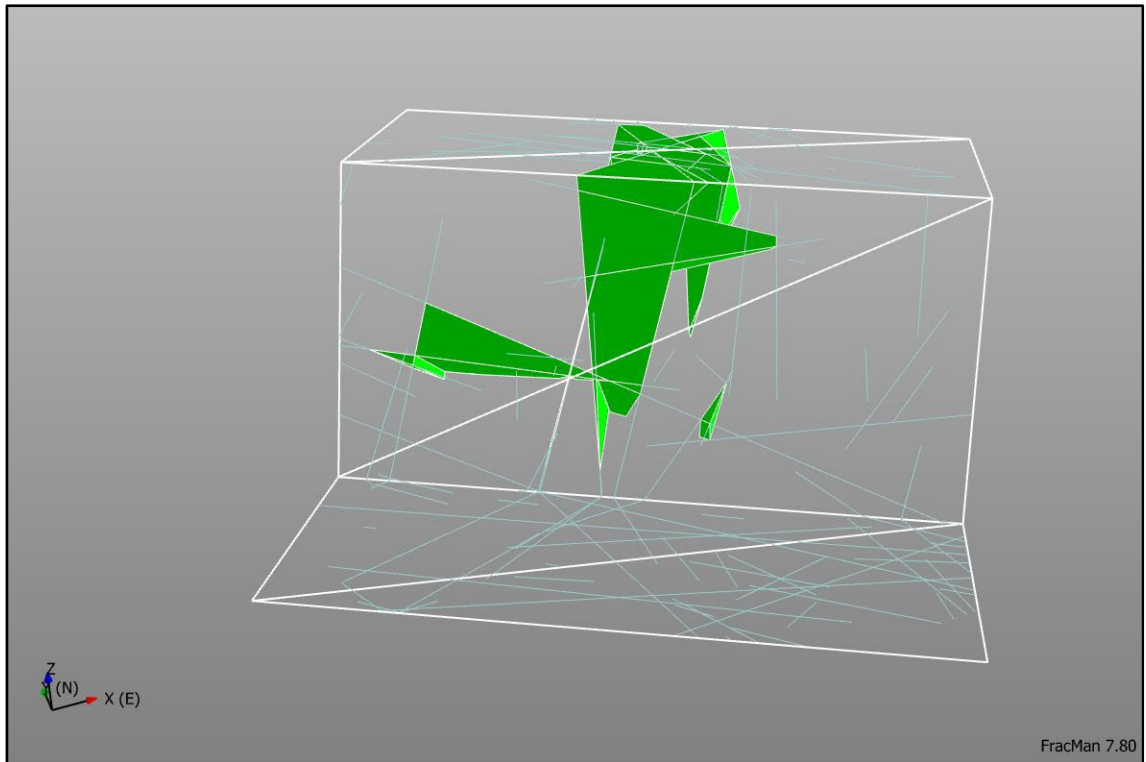


Figure 86: Rock Wedge Results: DFN Realization 2, Northeast Pit Wall, BFA of 80 degrees.

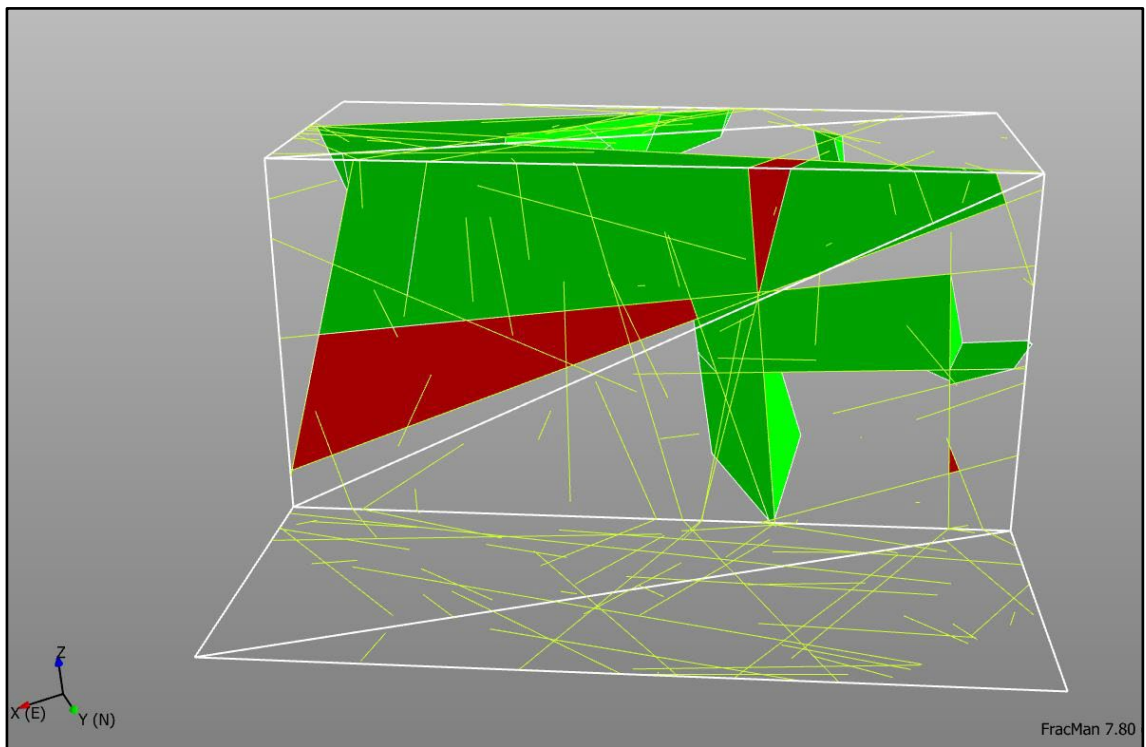


Figure 87: Rock Wedge Results: DFN Realization 2, South Pit Wall, BFA of 90 degrees.

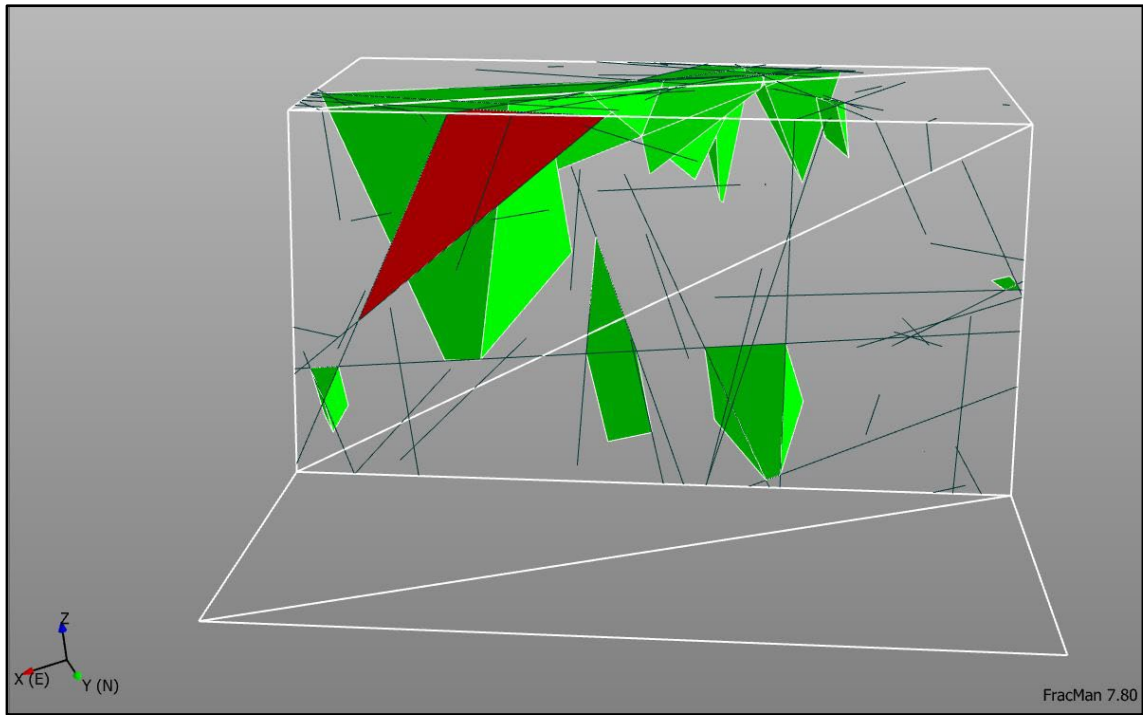


Figure 88: Rock Wedge Results: DFN Realization 2, South Pit Wall, BFA of 80 degrees.

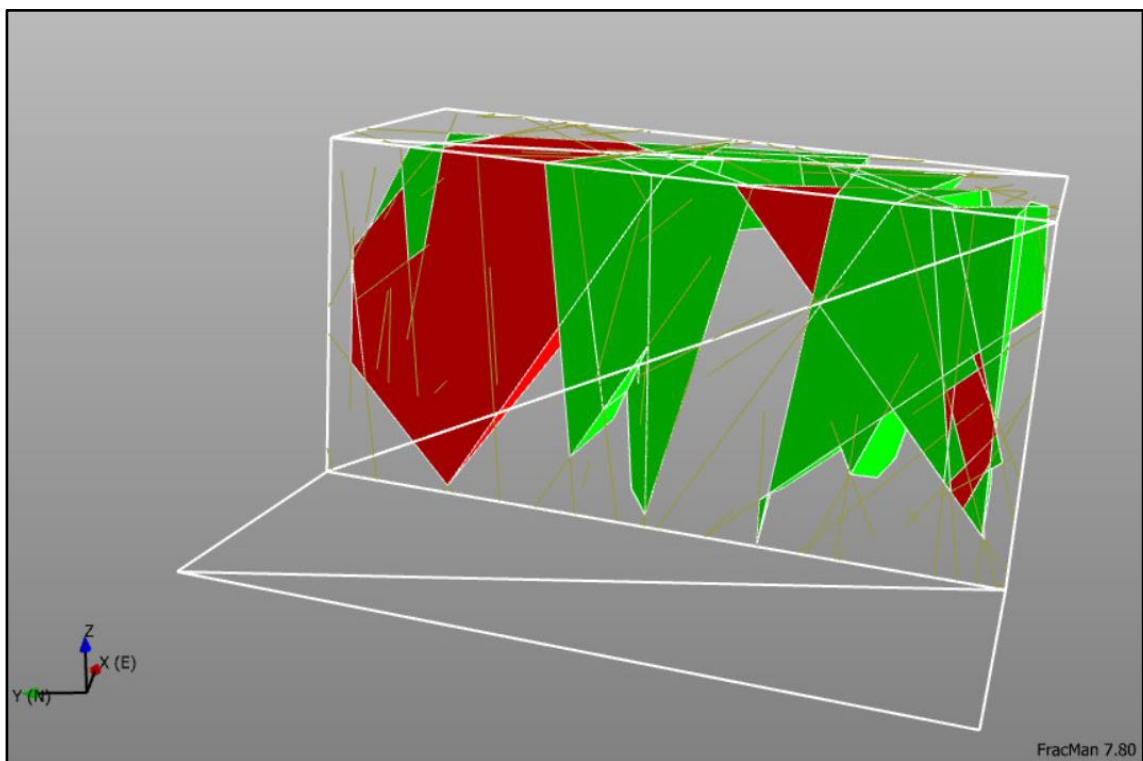


Figure 89: Rock Wedge Results: DFN Realization 3, East Pit Wall, BFA of 90 degrees.

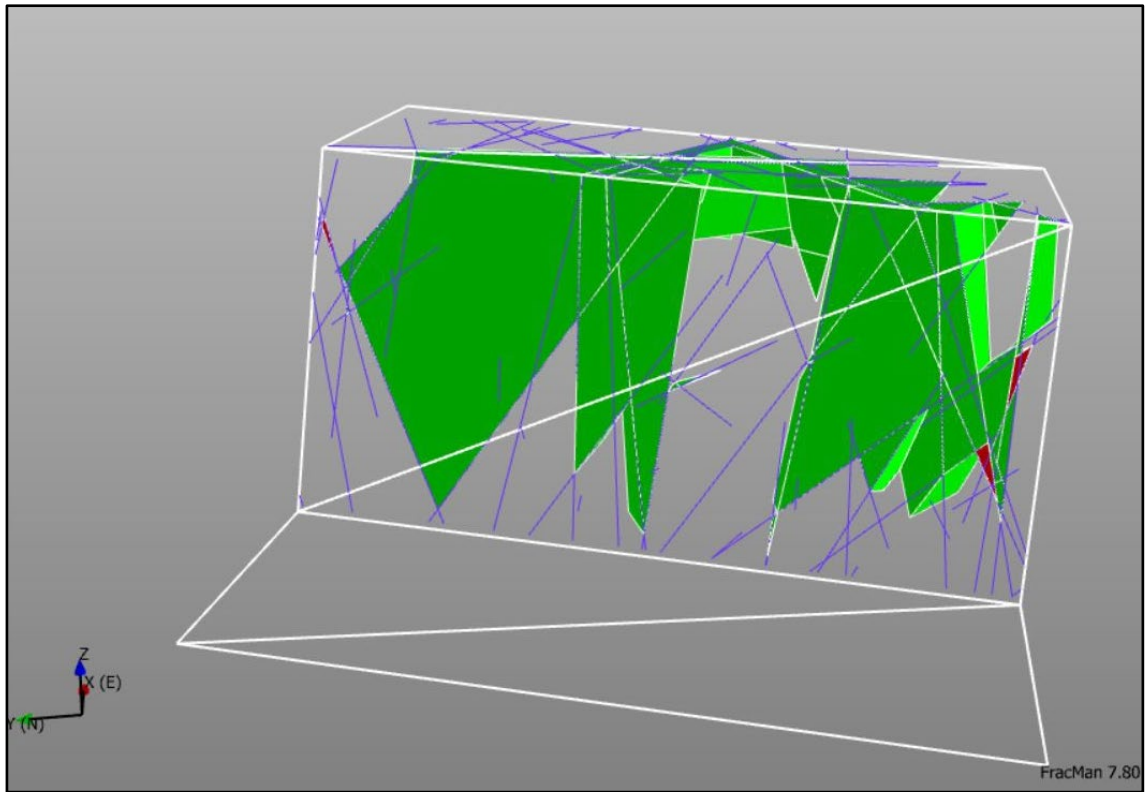


Figure 90: Rock Wedge Results: DFN Realization 3, East Pit Wall, BFA of 80 degrees.

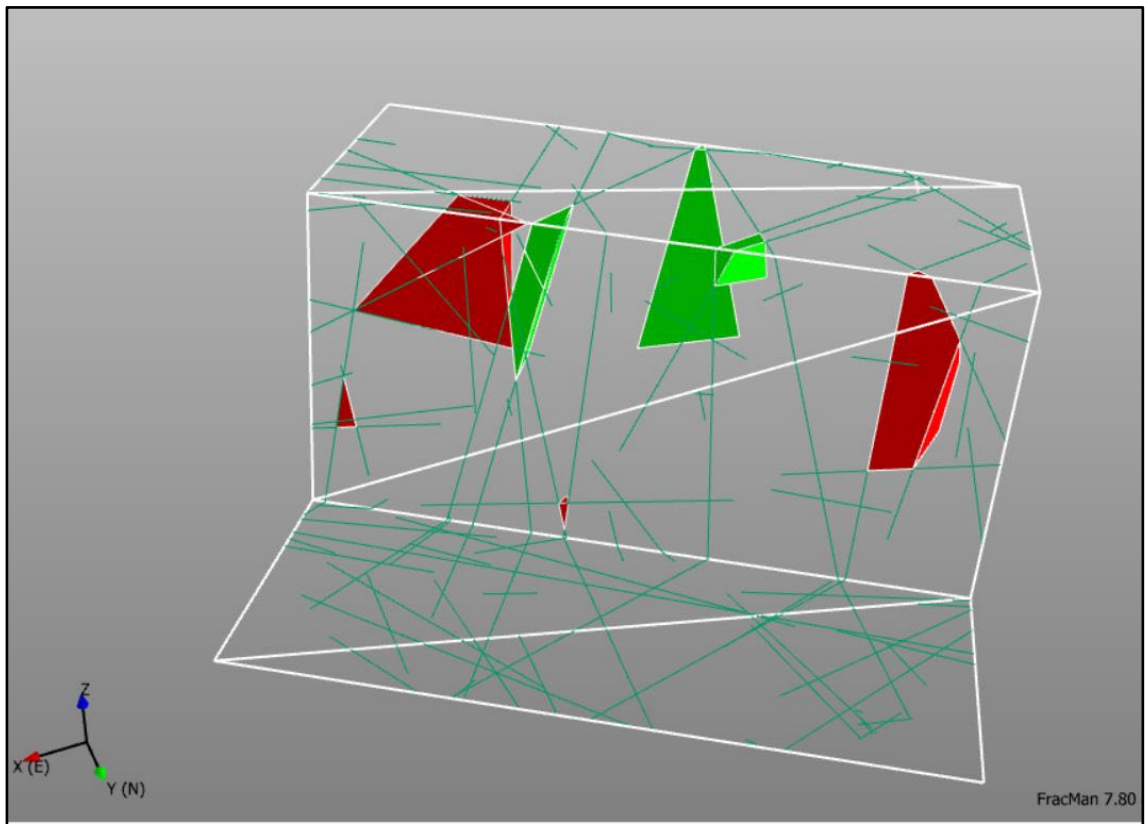


Figure 91: Rock Wedge Results: DFN Realization 4, South Pit Wall, BFA of 90 degrees.

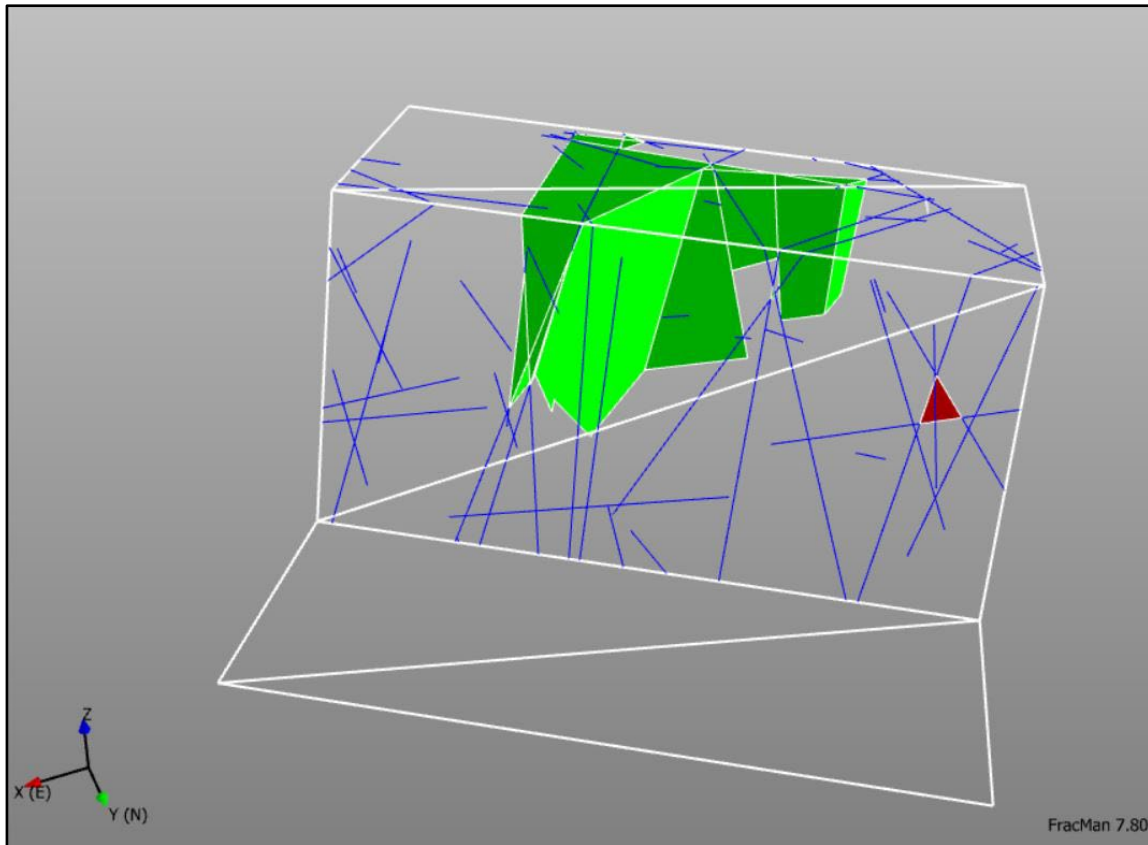


Figure 92: Rock Wedge Results: DFN Realization 4, South Pit Wall, BFA of 80 degrees.

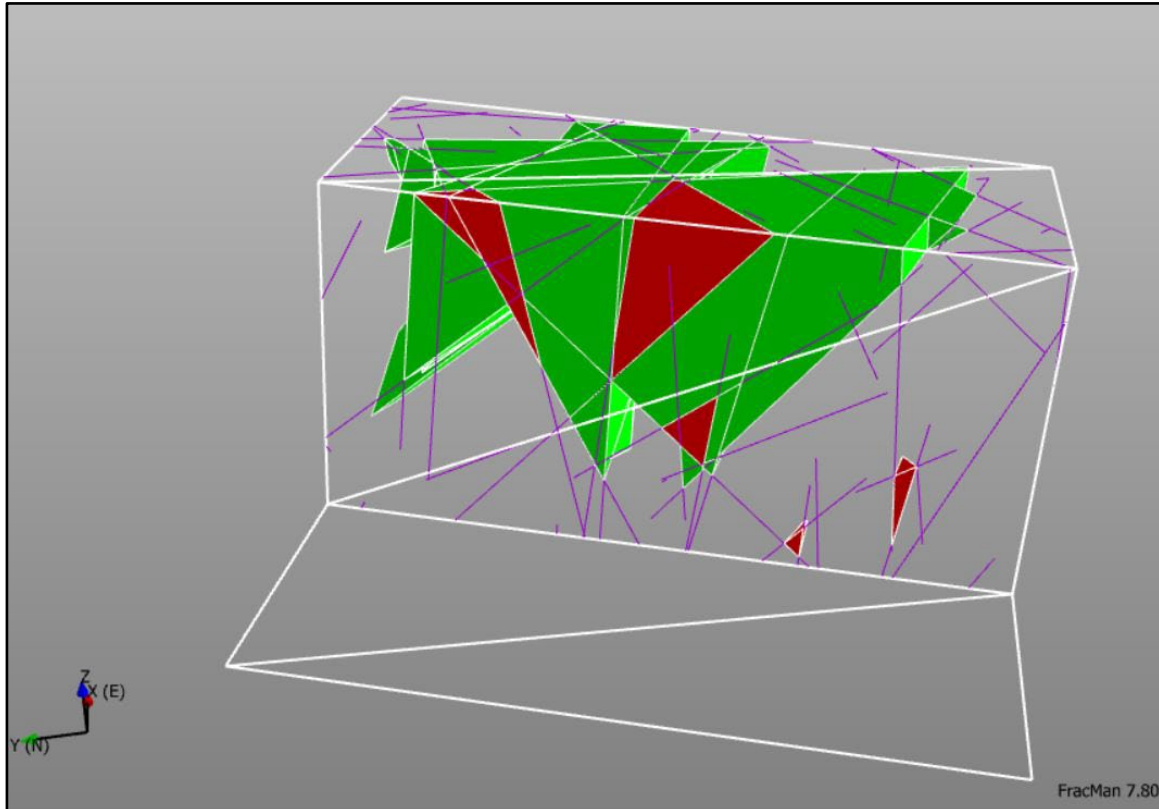


Figure 93: Rock Wedge Results: DFN Realization 5, East Pit Wall, BFA of 90 degrees.

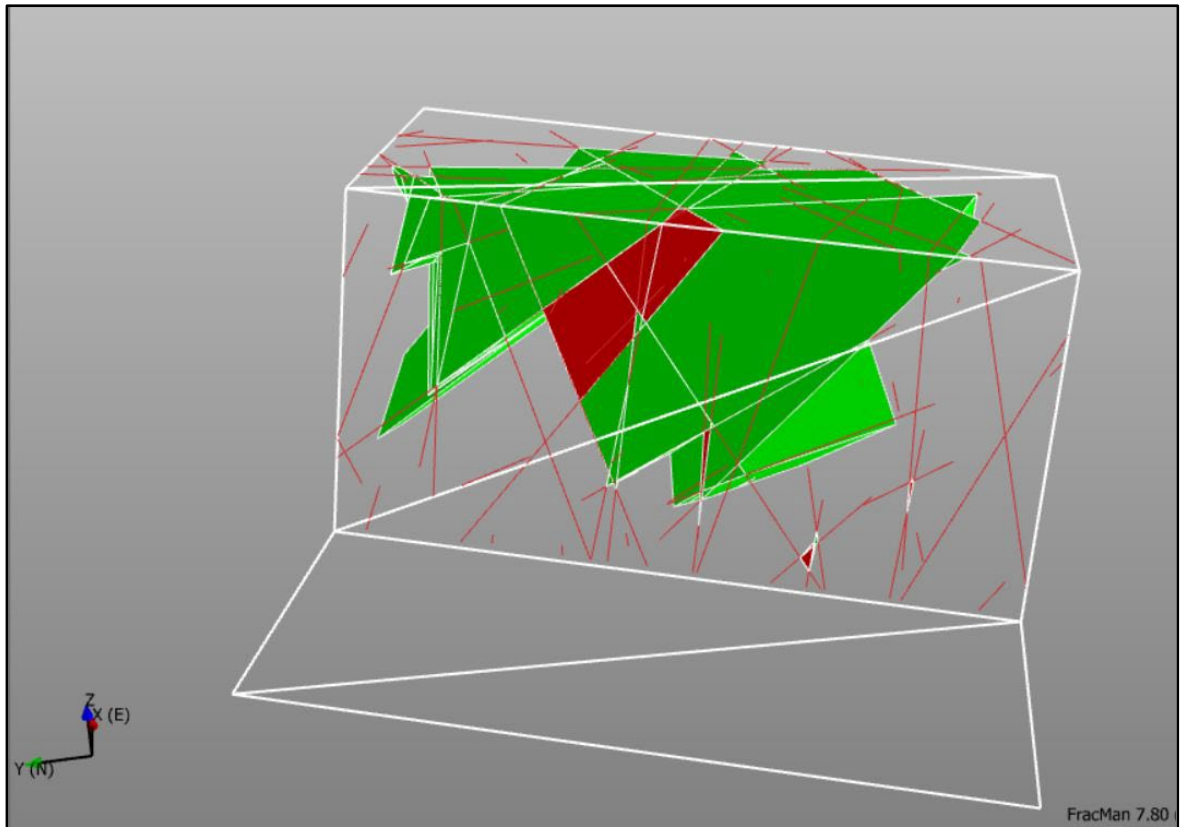


Figure 94: Rock Wedge Results: DFN Realization 5, East Pit Wall, BFA of 80 degrees.

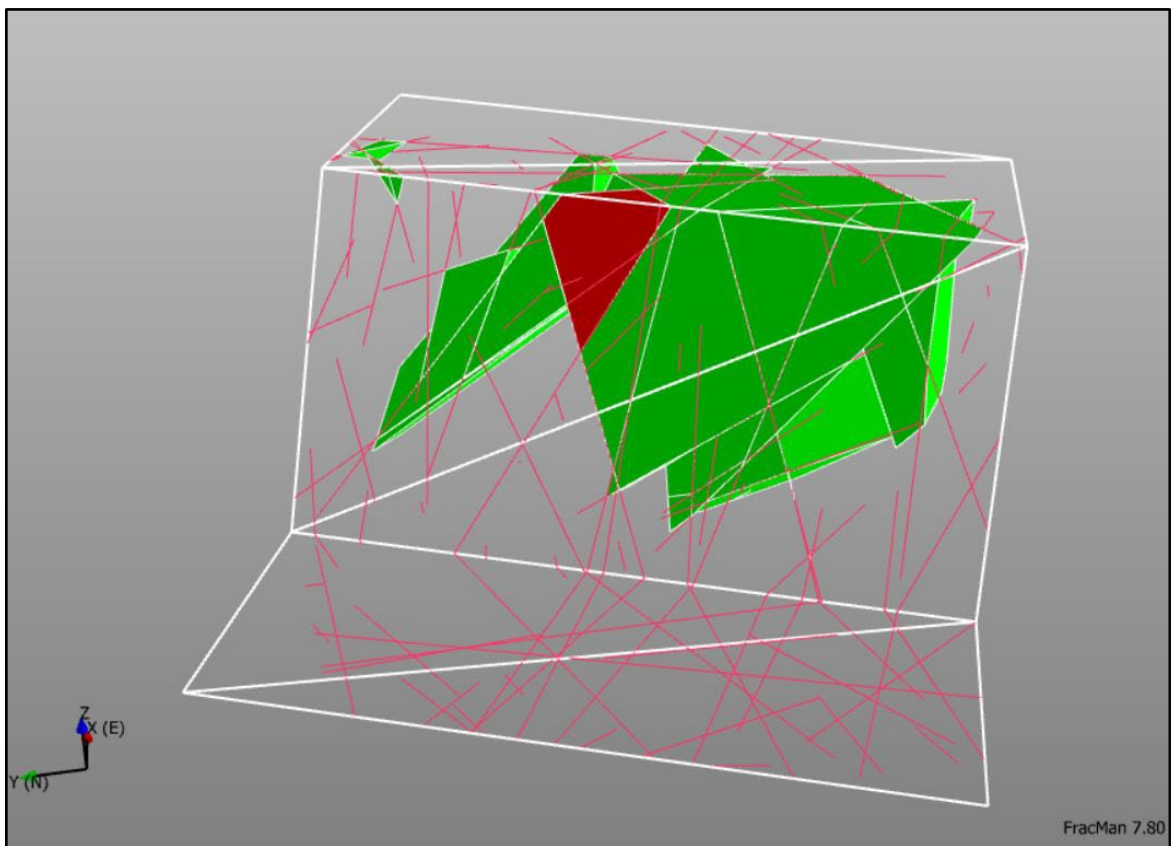


Figure 95: Rock Wedge Results: DFN Realization 5, East Pit Wall, BFA of 70 degrees.

The following were observed from the FracMan rock wedge results:

- Based on the combined five DFN realizations, the largest unstable wedge volume was identified in the east pit wall, considering a BFA of 90°.
- As shown in Figure 87, and Figure 88, we can see that the change in BFA identifies new and different potentially unstable wedges.
- There is a reasonable overall consistency of results between the different DFN model realizations.
- **DFN Model Realization 1:**
 - The highest number of unstable blocks have been identified in the north east pit wall, considering a 90° BFA.
 - The greatest number of unstable blocks have been identified in the west pit wall, considering a BFA of 80°.
 - No unstable blocks have been identified in the west pit wall considering a BFA of 70°.
 - Considering the FOS categories, the highest relative percentage of potential unstable blocks are expected in the south west wall (BFA of 90°). Similarly, for BFA of 80° this is expected in the west wall.
 - The number of unstable blocks is not reduced with a decrease in bench face angle. For example, the west sector considering BFA of 90° has no unstable blocks identified whilst the BFA of 80° has several unstable blocks identified. However, it should be noted that the volumes of the wedges identified for BFA of 80° is relatively small.
 - The south west pit wall 90° bench has the largest unstable wedge volume as compared to the other pit wall benches assessed. The wedge volume for the south west wall considering a BFA of 80° has resulted in a 77% reduction in the unstable wedge volume (refer to Figure 83 and Figure 84).
- **DFN Model Realization 2:**
 - The greatest number of unstable blocks have been identified in the south pit wall, considering a 90° BFA.
 - The highest number of unstable blocks have been identified in the south west pit wall, considering a BFA of 80°.
 - No unstable blocks have been identified in the west pit wall considering a BFA of 70°.
 - Considering the FOS categories, the highest relative percentage of potential unstable blocks are expected in the south wall (BFA of 90°). Similarly, for BFA of 80° this is expected in the south west wall.

-
- The north east pit wall 90° bench has the largest unstable wedge volume as compared to the other pit wall benches assessed (refer to Figure 85). As shown in Figure 86, reducing the BFA to 80° eliminates the unstable wedge identified previously, thereby lowering the expected failure volume for a BFA of 80°.

■ **DFN Model Realization 3:**

- The highest number of unstable blocks have been identified in the east pit wall, considering a 90° BFA.
- The greatest number of unstable blocks have been identified in the east and south pit walls, considering a BFA of 80°.
- The most number of unstable blocks have been identified in the east pit wall, considering a 70° BFA.
- Considering the FOS categories, the highest relative percentage of potential unstable blocks are expected in the south wall (BFA of 90°). Similarly, for BFA of 80° this is expected in the south wall and for BFA of 70° this is expected in the east wall.
- The east pit wall 90° bench has the largest unstable wedge volume (refer to Figure 89) as compared to the other pit wall benches assessed. As shown in Figure 90, reducing the BFA to 80 degrees eradicates the previous identified unstable wedges in BFA of 90 degrees.

■ **DFN Model Realization 4:**

- The greatest number of unstable blocks have been identified in the south pit wall, considering a 90° BFA.
- The highest number of unstable blocks have been identified in the south west pit wall, considering a BFA of 80°.
- No unstable blocks have been identified in the west pit wall considering a BFA of 70°.
- Considering the FOS categories, the highest relative percentage of potential unstable blocks are expected in the south wall (BFA of 90°). Similarly, for BFA of 80° this is expected in the south west wall.
- The south pit wall 90° bench has the largest unstable wedge volume as compared to the other pit wall benches assessed (refer to Figure 91).

■ **DFN Model Realization 5:**

- The highest number of unstable blocks have been identified in the south and east pit walls, considering a 90° BFA.
 - The greatest number of unstable blocks have been identified in the east pit wall, considering a BFA of 80°.
-

-
- The most unstable blocks have been identified in the south, east and south west pit wall, considering a 70° BFA.
 - Considering the FOS categories, the highest relative percentage of potential unstable blocks are expected in the south wall (BFA of 90°). Similarly, for BFA of 80° and 70° this is expected in the south wall.
 - The north east pit wall 90° bench has the largest unstable wedge volume as compared to the other pit wall benches assessed.

6.3 Comparison of FracMan and SWedge Results

Based on the Mine X case study, the following observations can be made from the comparison of the Swedge and FracMan rock wedge results:

- Generally, the results from both SWedge and FracMan identify similar wedge forming potential in the pit wall. However, the stability results are often better for wedges identified using the DFN approach as compared to SWedge results. This was expected, given that SWedge identifies the maximum sized wedge.
- The SWedge predicted maximum volume for a single wedge is substantially larger than the corresponding scenario result using FracMan. From the SWedge results, the south sector (90° BFA) was identified as having the largest wedge volume (greater than 3000m³), whilst the DFN model realizations predicted a maximum unstable block volume of $\pm 45\text{m}^3$ for all wedges considered. This is due to the DFN modelling being based on more realistic assumptions as compared to SWedge (i.e., discontinuity lengths are assumed to be continuous).
- As expected, the number of unstable blocks predicted is higher for the DFN approach as compared to the deterministic approach. This is attributed to the DFN considering the trace length distribution and spacing of structures consideration. This probably increases the small block forming potential with the discrete fracture network approach.
- Many unstable wedges were identified during the FracMan rock wedge analysis that were not identified with the traditional stability results (SWedge results). This is due to the fundamental differences between the two methods and their considerations (e.g. single wedge versus multiple wedges, continuous versus discontinuous fracture length). However, it should be noted the result of a simplistic experiment that compares the wedge considering two joint sets in FracMan and Swedge resulted in wedge volume that is consistent between these two methods.
- Predicted failure volumes do not necessarily decrease with the decrease in bench face angles as there are cases where the DFN model has identified new wedges. This is understandable given the stochastic nature of the DFN approach.
- The FracMan rock wedge maximum volumes predict comparatively lower sized maximum wedges.

-
- FracMan analyses multiple wedges at once as compared to SWedge that assesses only a single wedge combination at a time.
 - The spacing of discontinuities cannot be accounted for using SWedge whereas the DFN does consider this in the form of fracture intensity.

6.4 Chapter Summary

This chapter provides the rock wedge results from both SWedge and FracMan. Based on the deterministic results, the largest potential unstable wedge within the South sector of Mine X whilst the highest number of wedge combinations are associated with the east, south west and north east pit wall sectors. The DFN realizations identify the north east sector as the most susceptible to wedge failure as this is associated with the largest wedge volumes. This chapter concludes with initial observations made from the comparisons between the SWedge and FracMan wedge stability results. It should be noted the wedge stability results were considered for the design of benches presented in chapter 7.0.

CHAPTER 7

7.0 BENCH DESIGN CONSIDERATIONS

The optimization of slope design plays a crucial role in the economics and safety of open pit mines. Bench design components contribute to the inter-ramp geometry which comprises the overall slope design. As discussed in 2.3.1 and illustrated in Figure 27, a bench design comprises of the bench height, bench width and bench face angle. Bench heights are generally fixed by the operational machinery; the bench face angles are selected by considering the results of the kinematic failure modes in conjunction with the level of risk accepted by the operating mine. This research involves the use of DFN's and its application to rock wedge failures (and their consequence) predominantly related to the bench width design parameter and to some extent, bench face angles. This chapter presents the bench design considerations based on the DFN approach and its comparison with the traditional wedge stability approach.

7.1 Slope Design Acceptance Criteria

It is common knowledge that the risk of bench failures cannot be eliminated completely but can be reduced and managed through design, monitoring and appropriate slope management plans. (Sullivan, 2006) explains that benches are always nearly the first element of design for an open pit and generally only once this bench design is completed, then both inter-ramp and overall slopes are evaluated. As shown in Table 12, bench design influences both safety and economics, although safety is the primary concern.

Table 12: Pit Slope Design Elements and Risk Areas

Slope Design Element	Component	Main Controlling for Influencing Factor			Risk Area	
		Geotechnical Issues	Mining Systems	Environmental Factors	Safety	Economics
Bench	Bench Angle	*A	*A	*B	*A	
	Bench Height	*A	*A	*B	*A	*B
	Berm Width	*A	*A	*B	*A	*B
Inter-ramp	Inter-ramp Angle	*A	*B		*1	*A
	Inter-ramp Height	*A			*1	*A
Overall	Overall Slope Angle	*A	*B		*1	*A
	Overall Slope Height	*A			*1	*A
*A – Principal factor or main concern. *B – Minor factor or concern. *1 Notes: Assumes a sound pit slope management procedure is in place.						

Although the objectives of this study do not include any formal risk analyses, the typical acceptance criteria applicable to slope designs was selected for the purpose of understanding the influence of the DFN approach on the bench design considerations. The use of acceptance criteria is a means by which mine owners, mine operating companies and mine investors can establish the required performance of pit slopes with respect to safety, ore recovery and financial return (Read & Stacey, 2009). For this project, the acceptance criteria provided in Table 13 would be used to assess the bench design considerations and potential risk associated. It should be noted that although the acceptance criteria provided in Table 13 calibrate the performance of pit slopes, it does not quantify the risks associated with slope failure (Read & Stacey, 2009).

Table 13: Typical FOS and PoF acceptance criteria values (Read & Stacey, 2009).

Slope Scale	Consequence of Failure* ²	Acceptance Criteria* ¹		
		FOS (min) (static)	FOS (min) (dynamic)	POF (max) P [FOS≤1]
Bench	Low – High	1.1	NA	25 – 50%
Inter-ramp	Low	1.15 – 1.2	1.0	25%
	Medium	1.2	1.0	20%
	High	1.2 – 1.3	1.1	10%
Overall	Low	1.2 – 1.3	1.0	15 – 20%
	Medium	1.3	1.05	5 – 10%
	High	1.3 – 1.5	1.1	≤10%
*1 Needs to meet all acceptance criteria.				
*2 Semi-quantitatively evaluated				

7.2 Bench Width Estimation

Bench widths and/or berm widths can be: estimated empirically using (Ryan & Pryor, 2001), (Pierson, et al., 1994), (Hustrulid, et al., 2001); calculated using (Haines, et al., 2006), (Gibson, et al., 2006), (Holley, et al., 2006), (Gibson & Paul, 2016) based on several different considerations; and, selected based on various probabilistic and reliability analysis. The (Haines, et al., 2006)/ (Holley, et al., 2006) method was used to estimate the berm/bench widths based on wedge failure volumes predicted from both SWedge and FracMan results. Bench widths estimated from the SWedge deterministic and probabilistic results are provided in Table 14, and Table 15, with the FracMan DFN bench width estimates provided in Table 16.

Generally, the pit slope benches are designed to be sufficiently wide to retain 75-80% of debris falling from the bench above (Read & Stacey, 2009). Berm width estimates based on the maximum volume and an 80% design threshold are provided in Table 14 and Table 15. The minimum bench widths provided in Table 15 are estimated using the bench design option available in SWedge for a design threshold of 100% and 80% that takes into consideration the distribution of persistence values for each

joint set. (Rocscience Inc, 2019) provides detail on the theory behind the bench design methodology available in SWedge.

Table 16 presents bench width estimates based on the maximum unstable block volume. For comparative purposes, the Modified Ritchie method, (Ryan & Pryor, 2001), was used to estimate a minimum bench width of 7.5 m considering a bench height of 15m as this is generally considered as an adequate minimum bench width for many mines (also is independent of wedge failure volumes). This minimum bench width together with a maximum bench face angle of 90°, for a bench height of 15m would produce a maximum inter-ramp angle (IRA) of 63°. For this exercise, it was assumed that this was the accepted bench design configuration, and it would be used for the comparisons of proposed bench design recommended based on the different wedge stability results (see section 7.4.1).

Table 14: Bench Width Estimation based on SWedge Deterministic Results

Pit Sector	Pit Wall Mean Dip Direction	Wedge Information				Bench Face Angle (°)	Bench / Spill Berm Width	
		Joint Set Combination		*1Volume (m³)	FOS		*2SBW / Bench Width (m)	*3Minimum Bench Width (m)
East Sector	295°	J1	J2	130,17	0.37	90	5.8	5.4
		J1	J3	1,99	0.06	90	1.4	1.3
		J2	J3	102,60	0.15	90	5.3	5.0
		J2	J4	625,88	1.25	90	9.8	9.1
		J1	J2	25,68	0.37	80	3.4	3.1
		J2	J3	5,32	0.15	80	2.0	1.9
		J2	J4	351,80	1.25	80	8.1	7.5
		J2	J4	146,28	1.25	70	6.0	5.6
South Sector	040°	J2	J4	3549,56	1.25	90	17.3	16.2
		J3	J4	3769,41	0.45	90	17.9	16.5
		J2	J4	2884,12	1.25	80	16.3	15.1
		J3	J4	2194,53	0.45	80	14.9	13.8
		J2	J4	1532,27	1.25	70	13.2	12.3
		J3	J4	983,59	0.45	70	11.4	10.6
North East Sector	218°	J1	J3	1,61	0.01	90	1.3	1.2
		J1	J4	1638,67	24.96	90	13.5	12.5
		J2	J3	31,49	0.20	90	3.6	3.4
		J2	J3	0,44	0.2	80	0.9	0.8

Pit Sector	Pit Wall Mean Dip Direction	Wedge Information				Bench Face Angle (°)	Bench / Spill Berm Width	
		Joint Set Combination		*1Volume (m³)	FOS		*2SBW / Bench Width (m)	*3Minimum Bench Width (m)
North West Sector	147°	J3	J4	84,87	0.75	90	5.0	4.7
				3,61	0.75	80	1.8	1.6
West Sector	085°	J3	J4	468,99	0.75	90	8.9	8.3
				355,12	0.75	80	8.1	7,5
				185,10	0.75	70	6.5	6.1
South West Sector	352°	J1	J2	33,74	0.37	90	3.7	3.4
		J1	J3	3,68	0.02	90	1.8	1.6
		J2	J4	24,47	1.25	90	7.2	3.1
		J3	J4	108,01	0.73	90	5.5	5.1
		J1	J2	2,31	0.37	80	1.5	1.4
		J2	J4	214,67	1.25	80	6.9	6.4
		J2	J4	298,49	1.25	70	7.7	7.1
*1 Maximum wedge volumes.					*3 Minimum bench width for 80% design threshold.			
*2 Based on maximum wedge volumes.								

Table 15: Bench Width Estimation based on SWedge Probabilistic Results

Pit Sector	Pit Wall Mean Dip Direction	Wedge Information				Bench Design Parameters			
		Joint Set Combination		*2Mean Volume (m³)	PoF (FOS < 1.0)	Bench Face Angle (°)	*3Bench Width (m)	*1Bench Width (m)	SWedge *4Bench Width (m)
East Sector	295°	J1	J2	-	0.29	70	-	-	16.3
				3.14	0.43	75	1.5	1.7	16.8
				25.68	0.56	80	1.8	3.4	16.7
				68.27	0.66	85	2.2	4.7	16.5
				130.17	0.75	90	2.5	5.8	16.5
		J1	J3	-	0.13	70	1.8	-	5.0
				-	0.22	75	2.2	-	6.4

Pit Sector	Pit Wall Mean Dip Direction	Wedge Information				Bench Design Parameters			
		Joint Set Combination		*2Mean Volume (m³)	PoF (FOS < 1.0)	Bench Face Angle (°)	SWedge *3Bench Width (m)	*1Bench Width (m)	SWedge *4Bench Width (m)
East Sector	295°			-	0.33	80	2.5	-	7.8
				-	0.45	85	2.8	-	9.5
				1.99	0.57	90	3.1	1.4	11.3
		J2	J3	-	0.26	70	1.8	-	5.0
				-	0.42	75	2.0	-	6.5
				5.32	0.62	80	2.3	2.0	8.8
				38.87	0.78	85	2.7	3.9	10.5
				102.06	0.89	90	3.1	5.3	12.0
		J2	J4	146.28	0.20	70	2.7	6.0	8.6
				240.37	0.25	75	3.1	7.1	10.6
				351.82	0.29	80	3.8	8.1	12.8
				480.10	0.31	85	4.7	9.0	14.8
				625.88	0.33	90	5.4	9.8	16.7
South Sector	040°	J3	J4	983.59	0.67	70	2.1	11.4	6.3
				1545.35	0.72	75	2.3	13.2	7.1
				1499.16	0.75	80	2.4	13.1	7.7
				1296.31	0.76	85	2.5	12.5	8.5
				1143.88	0.79	90	2.7	12.0	9.1
North East Sector	218°	J2	J3	-	0.26	70	2.2	-	7.2
				-	0.39	75	2.5	-	8.9
				0.44	0.52	80	2.8	0.9	10.6
				9.95	0.64	85	3.2	2.5	12.1
				31.49	0.75	90	3.7	3.6	13.5
North West Sector	147°	J3	J4	-	0.10	70	1.7	-	5.0
				-	0.18	75	2.0	-	5.9
				3.61	0.27	80	2.1	1.8	7.2
				31.18	0.37	85	2.4	3.6	8.5
				84.87	0.46	90	2.6	5.0	9.5

Pit Sector	Pit Wall Mean Dip Direction	Wedge Information				Bench Design Parameters			
		Joint Set Combination		*2Mean Volume (m³)	PoF (FOS < 1.0)	Bench Face Angle (°)	SWedge *3Bench Width (m)	*1Bench Width (m)	SWedge *4Bench Width (m)
West Sector	085°	J3	J4	185.10	0.59	70	2.2	5.6	9.3
				265.25	0.63	75	2.5	7.4	9.8
				355.13	0.66	80	2.8	8.1	10.3
				454.78	0.67	85	3.0	8.8	11.4
				468.99	0.68	90	3.2	8.9	12.4
South West Sector	352°	J1	J2	-	0.22	70	2.1	-	2.0
				-	0.33	75	2.5	-	2.5
				2.31	0.44	80	3.0	1.5	3.0
				13.55	0.54	85	3.5	2.7	3.5
				33.74	0.64	90	3.9	3.7	3.9
		J3	J4	-	0.20	70	1.5	-	5.9
				-	0.33	75	1.8	-	6.5
				-	0.48	80	2.1	-	7.2
				20.90	0.59	85	2.4	3.2	8.5
				108.01	0.45	90	2.6	5.2	9.8
		J2	J4	298.49	0.33	70	4.0	7.7	4.0
				270.61	0.34	75	4.5	7.4	4.5
				248.47	0.35	80	5.0	7.2	5.0
				230.21	0.35	85	5.4	7.0	5.4
				214.67	0.35	90	5.7	6.9	5.7
		J1	J3	-	0.04	70	1.1	-	3.5
				-	0.11	75	1.3	-	4.6
				-	0.22	80	1.6	-	5.7
				-	0.37	85	1.8	-	6.8
				3.68	0.54	90	2.1	1.8	8.2
*1 Calculated based on the wedge volumes (Haines, et al., 2006)						*3 Minimum bench width for 80% design threshold.			
*2 Based on maximum wedge volumes.						*4 Minimum bench width for design 100% threshold.			

Table 16: Bench width estimation based on FracMan Rock Wedge Results

Pit Sector	Pit Wall Mean Dip Direction	DFN Realization	PoF [FOS<1]	Bench Face Angle (°)	Maximum Unstable Volume (m ³)	Bench / Berm Width (m)
East Sector	295°	No. 1	0.06	90	0.004	0.2
			-	80	-	-
			-	70	-	-
		No. 2	0.04	90	1.42	1.3
			0.05	80	1.78	1.4
			-	70	-	-
		No. 3	0.15	90	335.92	8.0
			0.06	80	0.09	0.5
			0.05	70	1.13	1.2
		No. 4	0.12	90	9.89	2.5
			0.08	80	19.91	3.1
			-	70	-	-
		No. 5	0.18	90	35.43	3.8
			0.16	80	21.50	3.2
			0.11	70	16.46	2.9
South Sector	040°	No. 1	0.07	90	6.96	2.2
			-	80	-	-
			-	70	-	-
		No. 2	0.00	90	15.44	2.9
			0.06	80	7.70	2.3
			0.16	70	0.00	0.0
		No. 3	0.16	90	45.97	4.1
			0.08	80	2.11	1.5
			0.03	70	0.52	0.9
		No. 4	0.56	90	16.94	2.9
			0.09	80	0.30	0.8
			-	70	-	-

Pit Sector	Pit Wall Mean Dip Direction	DFN Realization	PoF [FOS<1]	Bench Face Angle (°)	Maximum Unstable Volume (m³)	Bench / Berm Width (m)
South Sector		No. 5	0.32	90	4.76	1.9
			0.24	80	2.44	1.5
			0.17	70	26.43	3.4
North East Sector	218°	No. 1	0.07	90	0.61	1.0
			-	80	-	-
			-	70	-	-
		No. 2	0.10	90	37.19	3.8
			-	80	-	-
			-	70	-	-
		No. 3	0.12	90	14.63	2.8
			-	80	-	-
			-	70	-	-
		No. 4	0.03	90	1.74	1.4
			-	80	-	-
			-	70	-	-
		No. 5	0.12	90	58.76	4.5
			0.04	80	2.80	1.6
			0.04	70	0.48	0.9
North West Sector	147°	No. 1	-	90	-	-
			-	80	-	-
			-	70	-	-
		No. 2	0.06	90	2.03	1.4
			0.03	80	0.005	0.2
			-	70	-	-
		No. 3	0.02	90	0.13	0.6
			-	80	-	-
			-	70	-	-
		No. 4	0.08	90	4.42	1.9
			0.00	80	0.00	0.0

Pit Sector	Pit Wall Mean Dip Direction	DFN Realization	PoF [FOS<1]	Bench Face Angle (°)	Maximum Unstable Volume (m³)	Bench / Berm Width (m)
North West Sector		No. 5	-	70	-	-
			0.04	90	0.11	0.5
			-	80	-	-
			-	70	-	-
West Sector	085°	No. 1	-	90	-	-
			0.17	80	0.32	0.8
			-	70	-	-
		No. 2	0.13	90	2.55	1.6
			0.05	80	0.11	0.5
			-	70	-	-
		No. 3	0.10	90	1.11	1.2
			0.09	80	0.01	0.2
			0.03	70	0.06	0.4
		No. 4	0.11	90	2.57	1.6
			0.05	80	0.29	0.8
			-	70	-	-
		No. 5	0.09	90	0.001	0.1
			0.21	80	30.73	3.6
			0.08	70	14.22	2.8
South West Sector	352°	No. 1	0.14	90	83.69	5.0
			0.07	80	19.28	3.1
			0.14	70	0.00	0.0
		No. 2	0.05	90	0.02	0.3
			0.12	80	0.003	0.2
			-	70	-	-
		No. 3	0.13	90	7.27	2.2
			0.04	80	5.14	2.0
			0.03	70	0.14	0.6
		No. 4	-	90	-	-

Pit Sector	Pit Wall Mean Dip Direction	DFN Realization	PoF [FOS<1]	Bench Face Angle (°)	Maximum Unstable Volume (m ³)	Bench / Berm Width (m)
South West Sector	352°		0.08	80	0.013	0.3
			-	70	-	-
		No. 5	0.10	90	45.36	4.1
			0.19	80	13.59	2.7
			0.10	70	0.40	0.8

It is clear from the wedge volumes predicted, that the deterministic approach produces consistently more conservative results as compared to the DFN approach. In general, the probabilities of failure based on the FracMan DFN wedge results are significantly lower than the SWedge probabilistic results. If we consider a bench height of 15m, a BFA of 90° in the South Sector, the wedge results are as follows:

- SWedge Deterministic Result, the predicted maximum unstable volume of 3769.41m³ is associated with wedge formed by the combination of sets J3 & J4 (FOS = 0.45). A minimum bench width of 16.5m would be required to retain roughly 80% of the wedge volume using (Holley, et al., 2006). The expected maximum IRA based on this minimum bench width is 42°. Assuming 63° is the maximum IRA required, the bench design based on this result would produce a 33% decrease in the maximum IRA.
- SWedge Probabilistic Result, the predicted maximum unstable volume of 1143.88m³ is associated with wedge formed by the combination of sets J3 & J4 (POF [FOS<1] = 0.79). Based on the (Holley, et al., 2006), a minimum bench width of 11.1m would be required to retain roughly 80% of the wedge volume. Assuming 63° is the maximum IRA required, the bench design based on this result could produce a 10% decrease in the maximum IRA. The SWedge bench design option estimates the minimum bench width of 2.7m, which would result in an increase of the maximum IRA by 27% considering only a single wedge combination scenario.
- FracMan Rock Wedge Result, the predicted maximum unstable volume of 45.97m³ is associated with DFN realization no. 3 (FOS = 0.42). A minimum bench width of 3.8m would be required to retain 80% capacity based on the (Holley, et al., 2006) formula. The maximum IRA will be 76° based on this DFN realization's selected minimum bench width. Assuming 63° is the maximum IRA required, the bench design based on this result would produce a 13° increase in the maximum IRA (i.e., 21% increase) considering all unstable wedges in this sector.

The SWedge predicted minimum bench width using the "bench design" option provides minimum bench widths that appear to be relatively small given the associated predicted wedge volume. This is related to persistence analysis being based on the "small (ubiquitous)" selection. This resultant minimum bench width is more comparable to bench widths predicted from DFN approach however there is little consistency with the wedge volumes predicted and the minimum bench width results produced.

7.3 Bench Face Angle Selection

As described in 2.3.1, bench face angles are controlled by the rock mass structures and their properties. The change in bench angles and/or inter-ramp angles has a strong economic impact on the mining business, particularly the cash flow. The selection of the bench face angle is dependent on the results of the kinematic analysis and the subsequent stability analyses in relation to the acceptable level of risk. Based on the results shown in section 6.0, 7.2, and 7.3 and considering the South sector, the selection of a BFA would be:

- SWedge Deterministic Result: a BFA of 70° would be selected as this would require the lowest bench width (i.e., 11.4m) to retain 80% of the predicted failure volumes.
- SWedge Probabilistic Result, considering the upper limit of the acceptable bench PoF of 50%, wedges produced by combination of J3 & J4 have a high susceptibility to failure for all bench face angles considered. A BFA of 70° would be selected as this would result in the minimum bench width (10.6m) required to retain the bench failure volume predicted. Assuming 63° is the maximum IRA required, the bench design given this BFA, and bench width will result in a 32% decrease in the maximum IRA. A selection of an 80° BFA would require further management to increase their risk acceptance and may involve more implementation of suitable mitigation measures.
- FracMan Rock Wedge Result, a maximum BFA of 90° would be selected due to the smaller predicted volumes and low probabilities of failure considering all unstable wedges in this sector. Given the minimum bench width required to retain the predicted failure volumes based on all realizations in this sector, the desired design is achievable and there is potential for bench design optimization. The maximum IRA could be increased by 19% as the bench width could be reduced by 45%.

It is evident from this exercise that the BFA selection would be more conservative if we considered only the SWedge stability analyses.

7.4 Bench Design Recommendations

The influence of the wedge stability results from the different methods were assessed in terms of the bench design parameters for each pit design sector for Mine X. Table 17 summarises the proposed recommended bench design parameters for the massive pyroxenite rock mass units at Mine X based on the following:

- From the SWedge deterministic results, wedges that exceed the accepted design criteria (i.e., FOS <1.1) were considered and an 80% design threshold was applied to these predicted wedge volumes.
- From the SWedge probabilistic results, wedges that exceed the accepted design criteria (i.e., PoF [FOS<1] between 25-50%) and the 80% design threshold were applied to the corresponding

wedge volumes predicted. Both the SWedge predicted minimum bench width and the (Haines, et al., 2006) berm width was provided.

- FracMan rock wedge results considered the “worst case” scenario DFN realization with the 80% debris volume applied.
- Based solely on wedges controlling the stability of benches and does not consider planar back break effects nor effective bench face angles.

As shown in Table 17, the consideration of each wedge stability result separately, does have an influence on the recommended design parameters. This is most prevalent with the bench width estimation and related estimated maximum inter-ramp angle.

Table 17: Mine X MP Bench Design Recommendations

Slope Design Element	Wedge Stability Results	Pit Design Sector					
		East Sector	South Sector	North East Sector	North West Sector	West Sector	South West Sector
		295°	040°	218°	147°	085°	352°
Bench Height	All	15m	15m	15m	15m	15m	15m
Bench Face Angle	SWedge Deterministic	90°	70°	90°	90°	80°	90°
	SWedge Probabilistic	90°	70°	90°	90°	80°	90°
	FracMan Rock Wedge	90°	90°	90°	90°	90°	90°
Minimum Bench Width	SWedge Deterministic	5.3m	10.6m	3.4m	4.7m	7.5m	5.1m
	SWedge Probabilistic	3.1m *5.0m	2.1m *10.6m	3.7m *3.4m	2.6m *4.7m	2.8m *7.5m	2.6m *3.4m
	FracMan Rock Wedge	7.3m	3.8m	4.1m	1.8m	3.3m	4.6m
Maximum Inter-ramp Angle	SWedge Deterministic	71°	43°	77°	73°	56°	71°
	SWedge Probabilistic	78° *72°	63° *43°	76° *77°	80° *73°	70° *56°	80° *77°
	FracMan Rock Wedge	64°	76°	75°	83°	78°	73°
*Based on (Haines, et al., 2006) bench width estimation method							

Assuming the Mine X current slope design consists of a bench face angle of 80°, bench height of 15m and a minimum bench width of 7.5m for all pit design sectors, this would indicate the following based on the above results and observations:

- East Sector – The current design is acceptable and achievable considering all wedge stability results.
- South Sector – The current design is not acceptable based on the SWedge results and considering the acceptable design criteria for bench slopes. FracMan rock wedge results indicate that the design is acceptable and perhaps could be optimized.
- North East Sector - The current design is acceptable and achievable considering all wedge stability results. All bench design recommendations based on the different wedge results are similar to each other.
- North West Sector - The current design is acceptable and achievable considering all wedge stability results.
- West Sector – Current bench design can be achieved based on both the SWedge and FracMan rock wedge results. However, the FracMan rock wedge results suggest a potential to steepen the BFA.
- South West Sector - The current design is acceptable and achievable considering all wedge stability results.

7.4.1 Bench Design Risk

For the purpose of this study, we consider the worst-case scenario from the wedge stability results and their probabilities of failure using the below equation that quantifies the risk analysis of the open pit bench-scale slope (Read & Stacey, 2009),

$$\text{Risk} = \text{Probability of Failure} \times \text{Consequence of Event}$$

with Table 18 and/ or Figure 96 qualitative risk matrix for personnel safety to briefly evaluate the potential risk associated with the wedge stability methods and their influence on the bench design, albeit somewhat subjective. Assuming a current bench design that comprises of an 80° BFA, 15m bench height and a 7.5m bench width, and considering the wedge stability assessments for each pit sector, the associated risk is summarised in Table 19.

Table 18: Qualitative Risk Matrix for Personal Injury (Read & Stacey, 2009)

Consequences of Personal Injury					
Level		Descriptor		Example of Description	
1		Insignificant		No injuries	
2		Minor		First aid treatment	
3		Moderate		Medically treated injury	
4		Major		Extensive injuries/permanent disability	
5		Catastrophic		Fatality	
Qualitative Measure of Likelihood					
Level		Descriptor		Example of Description	
A		Almost certain		Is expected to occur in most circumstances	
B		Likely		Will probably occur in most circumstances	
C		Possible		Might occur at some time	
D		Unlikely		Could occur at some time	
E		Rare		May occur only in exceptional circumstances	
Qualitative Risk Matrix					
Consequences					
Likelihood	Insignificant	Minor	Moderate	Major	Catastrophic
A	H	H	E	E	E
B	M	H	H	E	E
C	L	M	H	E	E
D	L	L	M	H	E
E	L	L	M	H	H
E: Extreme risk – immediate action required; unacceptable risk					
H: High risk – Senior management attention required; unacceptable risk without action					
M: Moderate risk – management responsibility; acceptable with control measures					
L: Low risk – manage by routine procedures; acceptable risk					

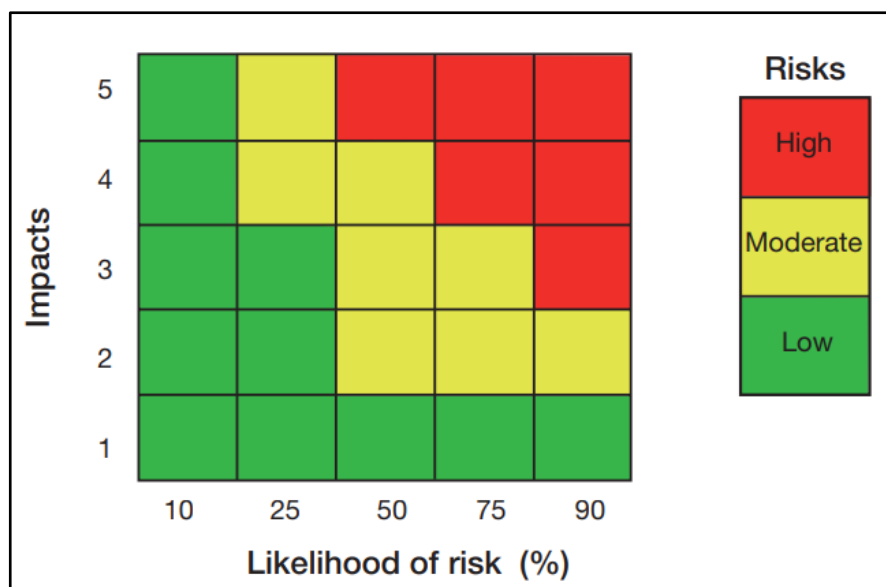


Figure 96: A semi-quantitative risk matrix (Read & Stacey, 2009)

Table 19: Current risk based on wedge stability assessments.

Slope Design Parameter	Wedge Stability Results	Pit Design Sector					
		East Sector	South Sector	North East Sector	North West Sector	West Sector	South West Sector
		295°	040°	218°	147°	085°	352°
Bench Face Angle	SWedge Deterministic	M	E	L	H	E	L
	SWedge Probabilistic	M	E	M	L	E	M
	FracMan Rock Wedge	L	L	L	L	M	M
Bench Width	SWedge Deterministic	L	E	L	L	E	L
	SWedge Probabilistic	L	E	L	L	H	L
	FracMan Rock Wedge	L	M	L	L	L	L

As shown in Table 19, the assumed current Mine X bench design risk profile varies for each sector considering the probability of bench failures and inadequate bench widths predicted by the different wedge stability assessments. However, it is apparent by the risk profile associated with the DFN approach wedge stability, that the current bench design is generally at a low to moderate risk which is acceptable and managed by routine procedures; and/or, acceptable with control measures. In theory,

depending on the mine accepting a higher level of risk, the bench design based on FracMan rock wedge results implies that the bench design could be eligible for optimization.

Based on current assumed bench design risk profile associated with the SWedge results, the level of risk can be considered, unacceptable without action, and/or, unacceptable requiring immediate actions. The experience with Mine X relative to this risk comparison, the current level of risk can be described as acceptable and manageable. This is more aligned with the predictions of the DFN approach as compared to the traditional deterministic approach.

7.5 Chapter Summary

The optimization of a slope design plays a crucial role in the economics and safety of open pit mines. Bench design components (i.e., bench face angle, bench height and bench width) contribute to the inter-ramp geometry which comprises the overall slope design. Bench heights are generally fixed by the operational machinery; the bench face angles are selected by considering the results of the kinematic failure modes in conjunction with the level of risk accepted by the operating mine. This chapter investigated the influence on bench design parameters considering the wedge stability assessments carried out using both SWedge and FracMan. Bench widths were estimated based on wedge volumes using the (Haines, et al., 2006)/ (Holley, et al., 2006) method and the bench face angle selection was discussed based on the FracMan and SWedge results.

Although the bench design recommendations based on the results from the different wedge analyses are similar for several pit design sectors; the results associated with SWedge analysis produce more conservative bench designs as compared to the DFN approach. Based on the DFN approach there are opportunities for further optimization of the bench face angles as well as the adequate bench widths.

Based on the assumed current bench design and a semi-quantitative and qualitative risk assessment, the risk profile associated with the DFN approach wedge stability is generally at a low to moderate risk level, which is acceptable and managed by routine procedures; and/or, acceptable with control measures. This is consistent with the acceptable risk experience at Mine X.

CHAPTER 8

8.0 CONCLUSIONS AND RECOMMENDATIONS

The use of discrete fracture networks is becoming increasingly common in geomechanics, [as noted by many such as (Dershowitz, 1984), (Dershowitz & Einstein, 1988), (Grenon & Hadjigeorgiou, 2012), (Elmo, et al., 2015), (Rogers, et al., 2007), (Stacey, et al., 2014)], and whilst this approach has been applied to large-scale slope stability problems, its application to smaller scale problems is comparatively more limited. One such application of limited use, is in the design of open pit benches. Bench stability is structurally controlled, and the traditional bench design methods are often based on assumptions that may be too conservative or perhaps not realistic. Considering the significant economic impact of a conservative design or bench-scale failures, these conservative assumptions should be addressed with other tools available in the open pit design discipline. This research investigated the DFN method as a tool to assist practitioners with bench design in hard rock environments. The main components of this study involving the Mine X case study included:

- Development, validation of a bench-scale DFN model using FracMan and subsequent rock wedge stability analysis.
- Wedge stability assessments based on the traditional approach involving the stereographic kinematic analysis followed by the deterministic and pseudo-probabilistic wedge stability assessment.
- Application of these two approaches to assess its influence on selected aspects of the bench design process: as well as a comparison between these approaches.

The key findings from this research study are summarised below for each of these main components of the study.

FracMan DFN Modelling

- The true characteristics of discontinuities cannot be known exactly. However, to determine the best estimate of discontinuity characteristics, the measurements of these properties should be as unbiased and objective as possible. Therefore, one should be cognisant of the errors and bias associated with structural data collection and structural modelling techniques and attempt to reduce or prevent these accordingly.
- The DFN approach is considered relatively successful with realistically modelling the heterogeneity of the rock mass structures because it allows for the characterization of individual fracture properties such as, position, size, aperture, intensity, and orientation.
- The primary input parameters are determined by statistical analysis of structural data characteristics (e.g., orientation, spacing), assumptions (e.g., fracture shape) and conversions of data (e.g., trace length data to fracture size) in order to generate three-dimensional stochastic

representations of rock mass structures or fractures. The DFN models generated from this sampled field data (i.e., 1D/2D discontinuity data collected from variably oriented rock exposures and boreholes) share the sample statistics and allow for explicit modelling of an individual fracture or simplified fracture sets (Elmo, et al., 2015).

- Development and validation of a DFN model can be challenging using data that was not originally intended for DFN use. Understanding the data collection methods, bias and errors can help reduce and or prevent this at the field investigation phase. It should be noted that the ISRM suggested methods for structural data collection do not adequately consider the application to discrete fracture modelling; however, guidelines outlined in (Elmo, et al., 2015) attempts to address some of these inadequacies.
- Model validation is an important aspect in evaluating the suitability of the DFN model generated. The DFN model generated is validated by comparing the orientation, intensity, and pattern of the simulated fracture traces to those observed or measured in the field. Model validation is essential to ensure that the DFN model does not misrepresent the behaviour of the rock mass in any type of analysis it may be used for (e.g., stability analysis, hydrogeological study, etc.)
- DFN fracture intensity parameter P32 is exceptionally sensitive and has the potential to generate fracture networks that may not be representative nor easily validated. This serves as a reminder that the full potential of a DFN model to capture stochastic variability is only achieved if the generated DFN geometry is statistically the same or at least similar (within some tolerance) to the actual fracture network it is meant to simulate. If this is not the case, then this would perhaps be the most inefficient and overly complicated alternative approach to the traditional bench design approach (Poropat & Elmouttie, 2011).
- The FracMan rock wedge algorithm results in a higher accuracy of block shape and volume, which is considered more appropriate for the kinematic assessment of block stability. This is primarily due to the DFN approach considering the interaction of multiple discontinuity sets and variability of discontinuity properties.

SWedge Stability

- SWedge deterministically predicts the maximum sized wedge formed by a single combination of two intersecting discontinuity sets. The probabilistic option allows for the input of statistical data that can account for uncertainty in joint orientation and joint strength values but can only produce probability of failure results based on individual wedge set combinations.
 - The related kinematic analysis only considers defined sets and thus the structures considered in this assessment are limited to these set clusters.
 - Probabilistic wedge stability results in a more conservative bench design that can be optimized using the bench design and scale wedge options available in SWedge.
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Bench Design - Traditional Approach vs. DFN Approach

- The traditional method involves the stereographic kinematic analysis and the limit equilibrium wedge stability method using SWedge. This traditional approach has its advantages, namely:
 - Approach is relatively easy to use and understandable.
 - The computational power required for the software involved (e.g., DIPS and SWedge) is low and the results are obtained almost immediately.
- The stereographic projections are limited in that they consider only the angular relationships between lines and planes and do not represent the sizes and positions of the structures, unlike the FracMan rock wedge analysis.
- The SWedge stability results assume infinitely continuous fracture lengths. The DFN approach can address these conservative assumptions of the traditional method by considering the sizes and locations of fractures.
- The learning curve for using FracMan to generate a DFN model can be long for a novice user.
- The DFN approach makes better use of all available structural data that technology has made easily available today.
- DFN approach allows for a more realistic wedge stability approach as it considers multiple wedges as opposed to a single wedge combination.
- Rock wedge volumes predicted are more realistic as compared to the traditional approach. This is because the DFN approach better captures what is observed in the field (e.g., interaction of multiple discontinuity sets and discontinuous fracture lengths).
- DFN approach can result in more optimized bench design that can result in significant economic savings and better management of benches in hard rock mining environments.
- Based on this study, the DFN approach produces more realistic stability results, and it does not assume the fracture lengths are infinitely continuous. This could imply that the presence of intact rock bridges does have an influence on stability at bench scale even though this is not considered to be a practical consideration at this scale.

Having provided the key findings related to the main components of this research study, we conclude by briefly addressing the research objectives and related questions. These are as follows:

- 1) ***Objective 1: To investigate the DFN approach using FracMan and its application to the stability of rock slopes at bench-scale.***
 - a. The DFN approach certainly produces more realistic representations of structures and block stability results which would ultimately lead to more realistic and optimal bench designs.

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- b. The DFN approach is relatively more complicated than the traditional approach. However, given the value addition to the bench design and management, and overall economics of the mine, it may be justified.
 - c. A DFN model applicability is only as good as its validation.
- 2) **Objective 2:** *To evaluate whether the DFN approach produces more optimal bench designs through more realistic representations of small-scale structures and better predictions of wedge failure volumes.*
- a. The more realistic assumptions associated with the DFN approach (e.g., variability of fracture properties, and fracture location) were demonstrated in the Mine X case study. The DFN predictions of unstable block volumes were more comparable to field observations than the wedge volumes predicted by traditional methods. Consequently, the design based on this DFN approach would be more optimal.
- 3) **Question 1:** *Does the DFN approach result in more realistic kinematic analysis?*
- a. Yes, it does because the FracMan wedge analysis considers discontinuous fractures with variable locations, sizes, orientations as well as the interaction of these multiple discontinuity sets. The Mine X FracMan wedge stability results indicate a greater level of stability as compared to the traditional method.
- 4) **Question 2:** *Does the application of the DFN method result in improved bench design?*
- a. Yes, as indicated by the bench design considerations presented in this study. An example discussed involved a potential to reduce the bench width by 50% and the bench face angle by 10 degrees.
- 5) **Question 3:** *How does the DFN method compare with the conventional bench design methodology?*
- a. DFN methodology is more complex however the value provided by considering realistic assumptions justifies the effort required.
- 6) **Question 4:** *What are the considerations for use of the DFN approach and its application to bench design and rock slope stability?*
- a. Considering the heterogeneity of the rock mass structures, the DFN approach produces a stability result that is comparatively more realistic. For example, the stability analysis with the DFN approach can consider the influence of intact rock bridges.
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Recommendations for Further Research

The opportunities for future research related to the use of discrete fracture networks are plentiful. The following are some ideas relating to the use of DFNs for bench design that is perhaps worth further exploration:

- i) Investigate the influence of intact rock bridges on bench stability. Intact rock bridges at bench scale may not be considered practical however it does have an influence on bench stability and therefore requires further research. This is particularly relevant considering that double benching practices is substantially increasing the scale of bench slopes.
- ii) Compare the various DFN model realizations, its block stability results and the associated bench configuration to better understand its variability and its influence on the recommended bench design. Compare DFN predicted failure volumes with any observed failure volumes and rock mass fragmentation.
- iii) Investigate the influence of planar back break and different blasting practices on bench stability using DFN methods.
- iv) Complete a P32 sensitivity analysis in the DFN generation to determine its influence on the rock wedge stability analysis.
- v) Standardize the structural data collection methods for DFN use.
- vi) Bench design using reliability analysis and FracMan DFN models.
- vii) Quantify the risk and the economic impact from bench designs derived from the traditional and DFN approach considering different mining environments.

9.0 REFERENCES

- ASTM, 1997. *Standard Test Method for Determining Rock Quality Designation (RQD) of Rock Core*, West Conshohocken, United States: ASTM International.
- Baecher, G. N., Lanney, A. N. & Einstein, H. H., 1997. *Statistical description of rock properties and sampling*. Colorado, 18th U.S. Symposium on Rock Mechanics, pp. 1-8.
- Barton, N., 1987. Discontinuities. In: F. G. Bell, ed. *Ground Engineers' Reference Book*. London: Butterworths.
- Brady, B. H. & Brown, T. E., 2005. *Rock Mechanics for Underground Mining*. 3rd ed. United States of America: Springer Science.
- Brown, E. T., 2002. *Block Caving Geomechanics*. 1st ed. Queensland, Australia: Julius Kruttschnitt Mineral Research Centre.
- Call, R. D., 1992. Slope Stability. In: H. I. Hartman, ed. *SME Mining Engineering Handbook*. Littleton, Colorado: AIME, pp. 881-896.
- Carvalho, L. J., 2002. *Slope Stability for Open Pits*, Mississauga, Canada: Rocscience Rock Newsletter.
- Cruden, D. M., 1977. Describing the Size of Discontinuities. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstract*, Volume 14, pp. 133-137.
- CSIRO Exploration and Mining, 2002. *Photogrammetry for Mining Applications*, Canberra, Australia: CSIRO Publishing.
- CSIRO Exploration and Mining, 2009. *SIRO3D: 3D Imaging/Pit Mapping System FAQ V4*, Canberra, Australia: CSIRO Sirovision.
- De Jager, D., 1990. Phosphate resources in the Palabora Igneous Complex, Transvaal, South Africa. In: A. Notholt, R. Sheldon & D. Davidson, eds. *Phosphate Deposits of the World*. Cambridge: Cambridge University Press, pp. 267-271.
- Dearman, R. W., 1991. *Engineering geological mapping*. 1st ed. Oxford: Butterworth - Heinemann Ltd.
- Deere, D. U., 1963. Technical Description of Rock Cores for Engineering Purposes. *Rock Mechanics and Engineering Geology*, 1(1), pp. 16-22.
- Dershowitz, S. W., 1984. *Rock Joint Systems*, Cambridge, Massachusetts: Massachusetts Institute of Technology (unpublished PHD thesis).
- Dershowitz, S. W. & Einstein, H. H., 1988. Characterizing Rock Joint Geometry with Joint System Models. *Rock Mechanics and Rock Engineering*, Volume 21, pp. 21-51.
- Dershowitz, W. S., La Pointe, P. R. & Doe, T. W., 2004. *Advances in discrete fracture network modeling*. Portland, US EPA/NGWA fractured rock conference.
- Dershowitz, W. S. & Herda, H. H., 1992. *Interpretation of fracture spacing and intensity*. Santa Fe, New Mexico, 33rd US Rock Mechanics Symposium.
- Dershowitz, W. S., Lee, G., Geier, J. & LaPointe, P. R., 1998. *FracMan: Interactive discrete feature data analysis, geometric modelling and exploration simulation - User Documentation*. Seattle, Golder Associates Inc..
- du Toit, A., 1931. The genesis of the pyroxenite-apatite rocks of Phalabora, Eastern Transvaal. *Transactions of the Geological Society of South Africa*, Volume 34, pp. 107-127.
- Einstein, H. H. & Baecher, G. B., 1982. Probabilistic and statistical methods in engineering geology. I. Problem statement and introduction to the solution.. *Rock Mechanics*, Volume 12, pp. 47-61.
-

-
- Einstein, H. H., Baecher, G. B., Veneziano, D. & al., e., 1979. *Risk analysis for rock slopes in open pit mines, Parts I–V*, H.H., Baecher, G.B., Veneziano, D., et al.,: USBM Technical Report J0275015..
- Einstein, H. H., Veneziano, D., Baecher, G. B. & O'Reilly, K. J., 1983. The effect of discontinuity persistence on rock slope stability. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, 20(5), pp. 227-236.
- Elmo, D., Rogers, S., Stead, D. & Eberhardt, E., 2014. Discrete Fracture Network approach to characterise rock mass fragmentation and implications for geomechanical upscaling. *Institute of Materials, Minerals and Mining and The AusIMM*, 123(3), pp. 149-161.
- Elmo, D., Stead, D. & Rogers, S., 2015. *Guidelines for the Quantitative Description of Discontinuities for use in Discrete Fracture Network Modelling*. Vancouver, Canada, International Symposium on Rock Mechanics .
- Elmo, D., Yan, M., Stead, D. & Rogers, S. F., 2007. *The Importance of Intact Rock Bridges in the Stability of High Rock Slopes: Towards a Quantitative Investigation Using an Integrated Numerical Modelling – Discrete Fracture Network Approach*. Perth, Australia, Proceedings of the 2007 International Symposium on Rock Slope Stability in Open Pit Mining and Civil Engineering, Australian Centre for Geomechanics.
- Elmouttie, M. K. & Poropat, G. V., 2015. *Uncertainty Propagation in Structural Modelling*. Vancouver, Canada, International Symposium on Rock Slope Stability in Open Pit Mining and Civil Engineering.
- Eriksson, S., 1982. *Aspects of the petrochemistry of the Palabora Igneous Complex, South Africa*, Johannesburg, South Africa: Unpublished PhD Thesis, University of Witwaterstrand.
- Ewan, V. J. & West, G., 1981. *Reproducibility of joint orientation measurement in rock. TRRL Supplementary Report 702.*, Crowthorne, England.: Department of the Environment and Department of Transport.
- Fadakar-Alghalandis, Y., 2014. *Stochastic modeling of fractures in rock masses.*, Adelaide: University of Adelaide.
- Forster, I., 1958. Paragenetical ore mineralogy of the Loolekop-Phalabora Carbonatite Complex, Eastern Transvaal. *Transactions of the Geological Society of South Africa*, Volume 61, pp. 359-365.
- FracMan Technology Group , 2016. *FracMan Geomechanics Edition*. [Online]
Available at: www.fracman.com
[Accessed 2016].
- Gibson, H. W., de Bruyn, I. A. & Walker, D. J., 2006. Considerations in the optimization of bench face angle and berm width geometries for open pit mines. *South African Institute of Mining and Metallurgy (SAIMM) Int Symp on Stability of Rock Slopes in Open Pit Mining and Civil Engineering*, Issue 44, pp. 557-578.
- Gibson, W. H. & Paul, J., 2016. *Comparison for estimating volume of failed wedges*, Perth: SRK Consulting (Australasia).
- Golder Associates Ltd, 2006. *Fractures, FracMan and Fragmentation: Applications of DFN Models to Block and Panel Caving*, Burnaby: Golder Associates Ltd.
- Golder Associates, 2006. *FracMan - Interactive discrete fracture network (DFN) analysis for representing key discrete fractures in 3D space, geometric modelling and exploration simulation*, Seattle: Golder Associates Inc..
- Golder Associates, 2008. *FracMan 7: Interactive Discrete Data Analysis, Geometric Modelling and Exploration Simulation*, s.l.: Golder Associates.
-

-
- Golder Associates, 2010. *Geotechnical Study of the Mine X*, Johannesburg: Golder Associates Report No. 12603-9564-1.
- Golder Associates, 2010. *Mine X Phase I - Hydrogeological Investigation*, Johannesburg: Golder Report No. 12609-9510-1.
- Goodman, R. E., 1976. *Methods of Geological Engineering in Discontinuous Rocks*. New York: West Publishing.
- Goodman, R. E., 1980. *Introduction to Rock Mechanics*. Toronto, Canada: John Wiley.
- Goodman, R. E., 1989. *Introduction to Rock Mechanics*. 2nd ed. Toronto, Canada: John Wiley & Sons.
- Goodman, R. E. & Shi, G. H., 1985. *Block Theory and its Application to Rock Engineering*. New Jersey: Prentice-Hall.
- Grenon, M. & Hadjigeorgiou, J., 2008. A design methodology for rock slopes susceptible to wedge failure using fracture system modelling. *Engineering Geology*, Volume 96, pp. 78-93.
- Grenon, M. & Hadjigeorgiou, J., 2012. Application of fracture system models in mining and civil rock engineering design. *International Journal of Mining, Reclamation, and Environment*, 26(1), pp. 55-73.
- Gumede, H., 2005. *Development of data sets on joint characteristics and consideration of associated instability for a typical South African gold mine. (MSc Dissertation)*, Johannesburg, South Africa: University of the Witwatersrand.
- Haines, A., Voulgaris, P., Walker, D. & de Bruyn, I., 2006. Geotechnical design considerations for the proposed Oyu Tolgoi open pits, Southern Mongolia. *The South African Institute of Mining and Metallurgy International Symposium on Stability of Rock Slopes in Open Pit Mining and Civil Engineering*, Issue 44, pp. 133-154.
- Hall, A., 1912. The crystalline metamorphic limestone of Lulukop and its relationship to the Palabora Plutonic Complex. *Transactions of the Geological Society of South Africa*, Volume 15, pp. 255-262.
- Hanekom, H., van Staden, C., Smit, P. & Pike, D., 1965. The geology of the Phalabora Igneous Complex. *Geological Survey of South Africa*, Volume memoir 54.
- Harrison, P. J., 1992. Fuzzy objective functions applied to the analysis of discontinuity orientation. In: J. A. Hudson, ed. *ISRM Symposium, EUROCK 92, Rock Characterization*. Chester, UK: s.n., pp. 25-30.
- Hawley, M., Marisett, S., Beale, G. & Stacey, P., 2009. Performance assessment and Monitoring. In: J. Read & P. Stacey, eds. *Guidelines for Open Pit Slope Design*. Collingwood, VIC, Australia: CSIRO Publishing, p. 337.
- Hoek, E., 2002. *Practical Rock Engineering*. [Online]
Available at: www.rocscience.com
[Accessed 2002].
- Hoek, E. & Bray, J. W., 1981. *Rock Slope Engineering*. 3rd ed. London: The Institute of Mining and Metallurgy.
- Holley, K., Skayman, P. & Zhiwei, H., 2006. *Geotechnical design for the open pits at Tanjianshan, China*. Johannesburg, The Southern African Institute of Mining and Metallurgy International Symposium on Stability of Rock Slopes in Open Pit Mining and Civil Engineering.
- Hudson, J. A. & Harrison, J. P., 1997. *Engineering Rock Mechanics: An Introduction to the principles*. 1st ed. London: Elsevier Science Ltd..
- Hustrulid, W. A., McCarter, M. K. & Van Zyl, D. J., 2001. *Slope Stability in Surface Mining*. Englewood: Society for Mining, Metallurgy, and Exploration.
-

-
- ISRM, 1978. Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses. International Society for Rock Mechanics: Commission on Laboratory and Field Tests. *International Journal of Rock Mechanics and Mining Science & Geomechanics*, Volume 15, pp. 319-368.
- Jermey, C. A., Kuppusamy, V., Fietze, C. P. & Hornsby, P. K., 2011. *Open Pit Bench Failure Volume Analysis and Berm Design – Is it Realistic?*. Vancouver, Canada, International Symposium on Rock Slope Stability in Open Pit Mining and Civil Engineering.
- Jing, L. & Stephansson, O., 2007. *Fundamentals of Discrete Element Methods for Rock Engineering: Theory and Applications*. Volume 85 ed. s.l.:Elsevier B.V..
- Kliche, C. A., 1999. *Rock Slope Stability*. Littleton: Society for Mining, Metallurgy, and Exploration, Inc. (SME).
- Kulatilake, P. H., 1993. *Application of probability and statistics in joint network modeling in three dimensions*. Canberra, Australia, Probabilistic Methods in Geotechnical Engineering.
- Kulatilake, P. H. S. W. & Wu, T., 1984. Estimation of mean trace length of discontinuities. *Rock Mechanics and Rock Engineering*, 17(4), pp. 215-232.
- La Pointe, P. R., 2002. Derivation of parent fracture population statistics from trace length measurements of fractal fracture populations. *International Journal of Rock Mechanics & Mining Sciences*, Volume 39, pp. 381-388.
- Lombaard, A., Ward-Able, N. & Bruce, R., 1964. The exploration and main geological features of the copper deposits in carbonatite at Loolekop, Palabora Complex. In: S. H. Haughton, ed. *The geology of some ore deposits in Southern Africa*. s.l.:Geological Society of South Africa, pp. 315-337.
- Lorig, L. J. et al., 2015. Application of discrete fracture networks in mining and civil geomechanics. *Mining Technology*, 124(4), pp. 239-254.
- Lorig, L., Stacey, P. & John, R., 2009. Slope Design Methods. In: P. Stacey & J. Read, eds. *Guidelines for Open Pit Slope Design*. Collingwood, VIC, Australia: CSIRO Publishing, pp. 235-242.
- Lyman, G. J., 2003. Rock fracture mean trace length estimation and confidence interval calculation using maximum likelihood methods. *International Journal of Rock Mechanics & Mining Sciences*, Volume 40, pp. 825-832.
- Mahtab, M. A. & Yegulalp, M. T., 1984. *A similarity test for grouping orientation data in rock mechanics*. Evanston, Illinois, 25th U.S. Symposium on Rock Mechanics.
- Martin, D. C. & Piteau, D. R., 1977. Select berm width to contain local failures. *Engineering and Mining Journal*, 178(6), pp. 161-164.
- Mauldon, M., 1994. Intersection Probabilities of Impersistent. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, 31(2), pp. 107-115.
- Mauldon, M., 1998. Estimating mean fracture trace length and density from observations in convex windows. *Rock Mechanics and Rock Engineering*, Volume 31, pp. 201-216.
- Mellor, E., 1907. *The geology of the district about Haenertsburg, Leydsdorp and the Murchison Range*, Transvaal: Transvaal Mines Department, report of the Geological Survey.
- Miller, M. S., 1983. A statistical method to evaluate homogeneity of structural populations. *Journal of the International Association for Mathematical Geology*, 15(2), pp. 317-328.
- Miyoshi, T., Elmo, D. & Rogers, S., 2018. *A Discrete Fracture Network approach to study the Variability of the Geological Strength Index*. Seattle, Washington, ARMA, American Rock Mechanics Association.
-

-
- Nicholas, D. E. & Sims, D. B., 2001. Collecting and Using Geologic Structure Data for Slope Design. In: W. A. Hustrulid, M. K. McCarter & D. J. Van Zyl, eds. *Slope Stability in Surface Mining*. Littleton, Colorado: Society for Mining, Metallurgy, and Exploration, Inc. (SME), pp. 11-23.
- Pahl, J. P., 1981. Estimating the Mean Length of Discontinuity Traces. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, Volume 18, pp. 221-228.
- Palmstrom, A., 2005. Measurements of and correlations between block size and rock quality designation (RQD). *Tunnelling and Underground Space Technology*, 21(6), pp. 365-377.
- Palmstrom, A., 1995. *RMI—A Rock Mass Classification System for Rock Engineering Purposes*. , Oslo: PhD Thesis, University of Oslo.
- Palmstrom, A., 2001. Measurement and characterization of rock mass jointing. In: V. M. Sharma & R. K. Saxena, eds. *In-situ characterization of rocks*. Rotterdam: A. A. Balkema, pp. 49-97.
- Phillips, F. C., 1977. *The Use of the Stereographic Projection in Structural Geology*. 3rd ed. London: Edward Arnold.
- Pierson, L., Davis, S. A. & Pfeiffer, T. J., 1994. *The nature of rockfall as the basis for new fallout area design criteria for 0.25:1 slopes*, Salem: Oregon Department of Transportation.
- Piteau, D. R. & Martin, D. C., 1977. *Slope stability analysis and design based on probability techniques at Cassiar mine*, Vancouver: Canadian Institute of Mining, Metallurgy and Petroleum.
- PMC Mine Geological and Mineralogical Staff, 1976. The Geology and the Economic Deposits of Copper, Iron, and Vermiculite in the Palabora Igneous Complex: A Brief Review. *Economic Geology*, Volume 71, pp. 177-192.
- Poropat, G. V. & Elmouttie, M. K., 2011. *Uncertainty Propagation in Structural Modelling*. Vancouver Canada, International Symposium on Rock Slope Stability in Open Pit Mining and Civil Engineering.
- Priest, D. S., 1985. *Hemispherical Projection Method in Rock Mechanics*. London: Allen & Unwin.
- Priest, S. D., 1993. *Discontinuity Analysis for Rock Engineering*. London: Chapman & Hall.
- Priest, S. D., 2004. Determination of Discontinuity Size Distributions from Scanline Data. *Rock Mechanics and Rock Engineering*, 37(5), pp. 347-368.
- Priest, S. D. & Hudson, J. A., 1976. Discontinuity Spacings in Rock. *International Journal of Rock Mechanics and Mining Science & Geomechanics*, Volume 13, pp. 135-148.
- Priest, S. D. & Hudson, J. A., 1981. Estimation of discontinuity spacing and trace length using scanline. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, 18(3), pp. 183-197.
- Read, J., 2009. Structural Model. In: J. Read & P. Stacey, eds. *Guidelines for Open Pit Slope Design*. Collingwood, VIC, Australia: CSIRO Publishing.
- Read, J. & Stacey, P., 2009. *Guidelines for Open Pit Slope Design*. 1st ed. Collingwood, VIC, Australia: CSIRO Publishing.
- Reischmann, T., 1995. Precise U/Pb age determination with badelyite (ZrO₂), a case study from the Palabora Igneous Complex, South Africa. *South African Journal of Geology*, Volume 98, pp. 1-4.
- Ritchie, A. M., 1963. Evaluation of rockfall and its control. *US Highway Research Board*, Issue 17, pp. 13-28.
- Rives, T., Razack, M., Petit, J. P. & Rawnsley, K. D., 1992. Joint spacing: analogue and numerical simulations. *Journal of Structural Geology*, 4(8/9), pp. 925-937.
-

Rocscience Inc, 2017. *Dips 7.0 Manual: Graphical and Statistical Analysis of Orientation Data*, Toronto: Rocscience Inc.

Rocscience Inc, 2019. *SWedge Theory Manual - Persistence and Bench Design*, s.l.: Rocscience.

Rocscience, 2018. www.rocscience.com. [Online]

Available at: https://www.rocscience.com/help/swedge/#t=theory%2FSwedge_Theory.htm

[Accessed 10 December 2019 December 2018].

Rocscience, 2021. *SWedge*. [Online]

Available at: <https://www.rocscience.com/software/swedge>

[Accessed 6th October 2021 2021].

Rogers, S., Elmo, D., Webb, G. & Catalan, A., 2014. Volumetric fracture intensity measurement for improved rock mass characterization and fragmentation assessment in block caving operations.. *Rock Mechanics and Rock Engineering*, 48(2), pp. 633-649.

Rogers, S. F., Bewick, R. P., Codelco, A. B. & Gaudreau, D., 2017. *Intergrating photogrammetry and discrete fracture network modelling for improved conditional simulation of underground wedge stability*. Perth, Australian Centre for Geomechanics.

Rogers, S. F., Dershowitz, S. W. & van As, A., 2007. *Characterising the insitu fragmentation of a fractured rock mass using a discrete fracture network approach*. London, Taylor & Francis Group, pp. 137-143.

Rogers, S. F., Elmo, D., Beddoes, R. & Dershowitz, B., 2009. *Mine scale DFN modelling and rapid upscaling in geomechanical simulations of large open pits*. Santiago, Chile, International Symposium on Rock Slope Stability in Open Pit Mining and Civil Engineering.

Rogers, S. F., Kennard, D. K., Dershowitz, W. S. & Van As, A., 2007. *Characterising the In Situ Fragmentation of a Fractured Rock Mass Using a Discrete Fracture Network Approach*. Vancouver, Canada, ARMA, American Rock Mechanics Association.

Rogers, S. F., Moffitt, K. M. & Kennard, D. T., 2006. *Probabilistic Slope and Tunnel Block Stability Analysis using Realistic Fracture Network Models*. Colorado, American Rock Mechanics Association (ARMA), 41st U.S Symposium on Rock Mechanics (USRMS).

Roux, E. H. et al., 1989. Phosphate in South Africa. *Journal of South African Institute of Mining and Metallurgy*, 89(5), pp. 129-139.

Russell, H., Hiemstra, S. & Groenewald, D., 1954. Mineralogy and petrology of the carbonatite at Loolekop, Eastern Transvaal. *Transactions of the Geological Society of South Africa*, Volume 57, pp. 197-208.

Ryan, T. M. & Pryor, P. R., 2001. Designing Catch Benches and Interramp Slopes. In: W. A. Hustrulid, M. K. McCarter & D. J. Van Zyl, eds. *Slope Stability in Surface Mining*. Littleton, Colorado: Society for Mining, Metallurgy, and Exploration, Inc. (SME), pp. 27-72.

Sen, Z. & Kazi, A., 1984. Discontinuity Spacing and RQD Estimates from the Finite Length Scanlines. *International Journal of Rock Mechanics, Mining Science and Geomechanics*, 21(4), pp. 203-212.

Shand, S., 1931. The granite-syenite-limestone complex of Phalabora, Eastern Transvaal, and the associated apatite deposits. *Transactions of the Geological Society of South Africa*, Volume 34, pp. 81-106.

Shanley, R. J. & Mahtab, A. M., 1976. Delineation and Analyses of Clusters in Orientation Data. *Mathematical Geology*, 8(1), pp. 9-23.

Sjogren, B., Ovsthus, A. & Sandberg, J., 1979. Seismic classification of rock mass qualities. *Geophysics Prospect*, Issue 27, pp. 409-442.

-
- Slama, C. C., Hendrikson, S. W. & Theurer, C., 1980. *Manual of Photogrammetry*. 4th ed. Falls Church, Virginia: Manual of Photogrammetry.
- Southwood, M. & Cairncross, B., 2017. The Minerals of Palabora, Limpopo Province, South Africa. *Rocks and Minerals*, 92(5), pp. 426-453.
- Spencer, E. W., 1969. *Introduction to the Structure of the Earth*. New York: McGraw-Hill.
- Stacey, P., 2009. Fundamentals of Slope Design. In: J. Read & P. Stacey, eds. *Guidelines for Open Pit Slope Design*. Collingwood, VIC, Australia: CSIRO Publishing, pp. 1-9.
- Stacey, T. R., 2007. *Slope Stability in High Stress and Hard Rock Conditions*. Perth, International Symposium on Rock Slope Stability in Open Pit Mining and Civil Engineering, Australian Centre for Geomechanics, pp. 187-200.
- Stacey, T. R., Armstrong, R. & Terbrugge, P. J., 2014. *Experience with the development and use of a simple DFN approach over a period of 30 years*. Vancouver, International Discrete Fracture Network Engineering Conference.
- Staub, I., Fredriksson, A. & Outter, N., 2002. *Strategy for a Rock Mechanics Site Descriptive Model Development and testing of the theoretical approach.*, Sweden: SKB Report R-02-02. Golder Associates AB. SKB.
- Sullivan, D. T., 2006. *Pit Slope Design and Risk - A view of the current state of the art*. South Africa, The South Africa Institute of Mining and Metallurgy, International Symposium on Stability of Rock Slopes in Open Pit Mining and Civil Engineering.
- Terzaghi, R. D., 1965. Sources of error in joint surveys. *Geotechnique*, Volume 15, pp. 287-304.
- Verwoerd, W., 1986. Mineral deposits associated with carbonatites and alkaline rocks. In: C. Anhaeusser & S. Maske, eds. *Mineral deposits of Southern Africa*. s.l.:Geological Society of South Africa, pp. 2173-2191.
- Villaescusa, E. & Brown, E. T., 1992. Maximum Likelihood Estimation of Joint Size from Trace Length Measurements. *Rock Mechanics and Rock Engineering*, Volume 25, pp. 67-87.
- Warburton, P. M., 1980a. A Stereological Interpretation of Joint Trace Data. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, Volume 17, pp. 181-190.
- Warburton, P. M., 1980b. Stereological Interpretation of Joint Trace: Influence of Joint Shape and Implications for Geological Surveys. *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*, Volume 17, pp. 305-316.
- Weir, F. M. & Fowler, M., 2014. *An Introduction to Discrete Fracture Network Modelling and its Geotechnical Applications*. Sydney, AUSROCK.
- Whitten, D. G. A. & Brooks, J. R., 1972. *A Dictionary of Geology*. Harmondsworth: Penguin.
- Wicksell, D. S., 1925. The corpuscle problem: A mathematical study of a biometric problem. *Biometrika*, Volume 17, pp. 84-89.
- Wilson, M., 1998. Copper. In: M. Wilson & C. Anhaeusser, eds. *The mineral resources of South Africa*. Pretoria: Council for Geoscience, pp. 209-227.
- Wittke, W., 1990. *Rock Mechanics Theory and Applications with Case Histories*. Berlin: Springer-Verlag.
- Wyllie, D. C. & Mah, C. W., 2004. *Rock Slope Engineering: Civil and Mining*. 4th ed. London: Spon Press.
- Zhang, L., 2017. *Engineering Properties of Rocks*. 2nd ed. Cambridge, USA: Elsevier Ltd..
-

Zhang, L. & Einstein, H. H., 2010. The Planar Shape of Rock Joints. *Rock Mechanics and Rock Engineering*, Volume 43, pp. 55-68.

Zhang, L., Einstein, H. H. & Dershowitz, W. S., 2002. Stereological relationship between trace length and size distribution of elliptical discontinuities. *Geotechnique*, 6(52), pp. 419-433.

Zhang, Z. & Einstein, H. H., 1998. Estimating the Mean Trace Length of Rock Discontinuities. *Rock Mechanics and Rock Engineering*, 31(4), pp. 217-235.

APPENDIX A

Stereographic Projection Plots for Mine X Case Study

APPENDIX B

Sirovision 3D Image Data Processing

APPENDIX C

Statistical Analysis of Discontinuity Data

APPENDIX D

FracMan DFN Modelling

APPENDIX E

SWedge Results