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Using discrete fracture networks to understand wedge failures for open pit slope design

JW Watton *PSM, Australia*

FM Weir *PSM, Australia*

Abstract

A wedge failure is defined as the sliding of a rock block on the line of intersection of two discontinuities. This failure mechanism can occur over a wide range of geological and geometric conditions. This paper presents the ground conditions and existing slope performance at a case study site where bench and multi-bench slope stability is controlled by wedge failures.

The study compares the probability of wedge failure from mapping and photogrammetry data with the existing performance of the current pit slope using industry-accepted methods. The existing slope performance was assessed using photogrammetry techniques built from aerial drones to estimate the proportion of pit slope that failed.

Slope stability was also assessed using a discrete fracture network approach, with the resultant predicted area of the failed slope compared against the current slope performance measured by photogrammetry. The study highlights the importance of understanding size and spatial relationships between defect sets for wedge formation and the implications for bench and inter-ramp scale slope design.

Keywords: *wedge, discrete fracture network, open pit slope design*

1 Introduction

Wedge failure, characterised by the sliding of a rock block along the line of intersection of two discontinuities, is influenced by various geological and geometric factors. While this failure mechanism can manifest under diverse conditions, its occurrence is contingent upon the size and spatial relationships of the intersecting defect sets. Existing industry-accepted statistical methods for evaluating the likelihood of structural instability often rely on assumptions regarding defect length and occurrence, which tend to err on the side of caution. However, significant deviations between these assumptions and the actual conditions may lead to variations in the performance of constructed slopes compared to those predicted.

This paper presents a comparison of industry-accepted approaches to pit slope design in structurally controlled environments with those derived from a discrete fracture network (DFN) approach. By evaluating the performance of the existing slope, this study aims to shed light on the effectiveness and reliability of different design methods.

2 Case study site

2.1 Setting

The study site comprises an existing 150 m-high pit slope in a high-strength igneous rock mass from a large copper porphyry mine. The rock mass has an adopted intact strength of about 105 MPa. This relatively high strength means that pit slope performance is structurally controlled at the scale assessed.

The rock mass is intersected by moderately to steeply dipping major structures that are multiple benches in length (i.e. greater than about 50 m), which are the structure set controlling slope performance. The slope aspect faces 285°. The adopted defect friction angle for the major structures is 20° and was based on the observed defect conditions. All major structures dip steeper than the adopted friction angle.

2.2 Structural data

Two structural datasets were available for analysis:

1. Dataset A: photogrammetry mapping of major controlling structures (77 structures).
2. Dataset B: pit mapping of major controlling structures (45 structures).

Defect orientations of both datasets are shown in Figure 1. Dataset A is from photogrammetry mapping of multi-bench to inter-ramp scale structures exposed in the pit face. Dataset B is comprised of field spot-mapping observations from multi-bench to inter-ramp scale structures exposed in the pit face.

Wedge failure (including sliding along one defect, with side release) was identified as the critical failure mechanism based on kinematic assessment. Kinematic assessment utilises the mean orientation of the controlling defect sets and the adopted design shear strengths for the pit wall to evaluate the controlling failure mechanisms and targets for statistical assessment. This was supported by field observations of slope performance.

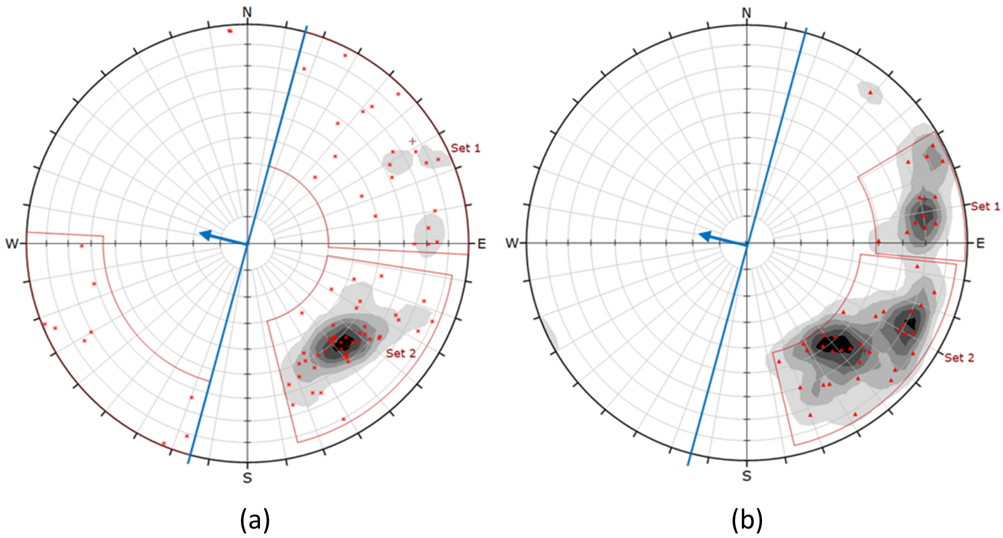


Figure 1 Equal area stereographs showing poles to measured fractures and identified fracture sets: (a) Dataset A; (b) Dataset B. The pit slope aspect is indicated by the blue line and arrow

3 Probability of wedges undercutting the slope

3.1 Theory

Probability of undercutting (PoU) is a statistical, risk-based analysis which accounts for the distribution of orientations and lengths within a defined area compared with the mean orientation used in kinematic assessment. In this way, the sensitivity to change in slope angle is assessed. The likelihood of failure, (P_f), is calculated using the following equation from McMahon (1974):

$$P_f = P_u \times P_e \tag{1}$$

where:

P_u = the percentage of defects in unstable orientations, or, in other words, the chance a slope is undercut by a defect or wedge intersection.

P_e = the percentage of defects that are sufficiently extensive and continuous to result in failure for the height of slope analysed.

For this study, defects are assumed to be continuous over the slope height under consideration; that is, $P_e = 1$. Therefore P_f is equal to the percentage of defects undercutting the slope $\left(P_f = P_u\right)$. This assumption is conservative as it assumes all defects can contribute to structural failure and does not consider the spatial distribution of defects.

Conventional open pit slope design assumes a level of slope performance which is commonly expressed as the probability of failure for inter-ramp and bench scale performance not exceeding, say, 5 and 30% of the slope area respectively (e.g. Wesseloo & Read 2009). These values are typically used as a guide and are applied using geological and geotechnical judgement and experience.

3.2 Statistical analysis results

The results of the statistical analyses for both Datasets A and B are shown in Table 1. A range of results are provided to assist with comparison to the existing slope performance, as discussed in Section 4. The results can also be shown graphically, with an increasing proportion of wedge intersections as the slope dip increases for a given slope aspect, as shown in Figure 2.

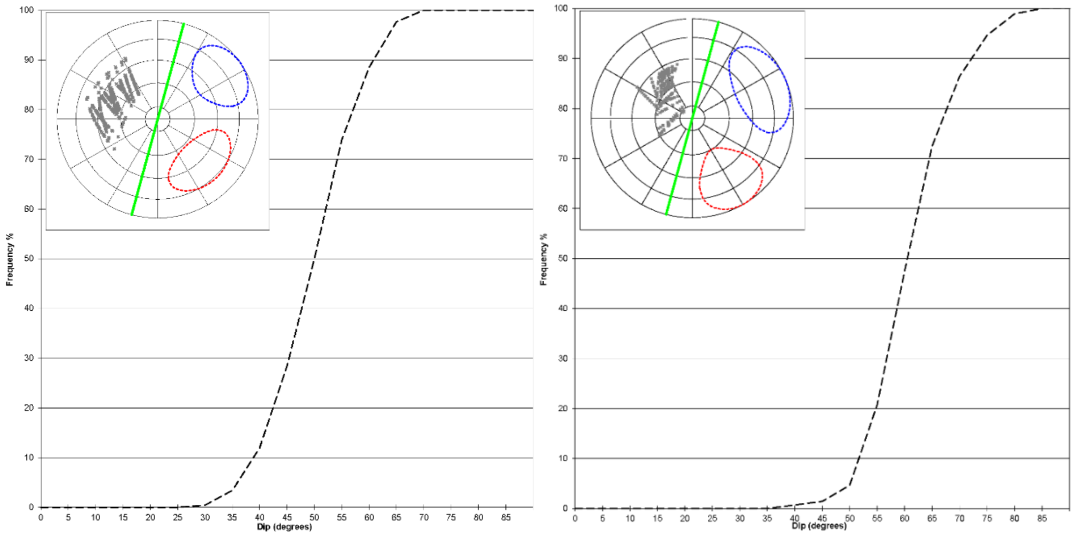
The results indicate the following range of approximate pit slope angles depending on the accepted probability of failure and the dataset used for design:

- Bench face angles: 40° to 60°.
- Inter-ramp angles: 35° to 50°.

There are implications in terms of slope stability and economics, particularly with the range of inter-ramp angles. These results typically form a guide, and that assessment of other factors is required to evaluate the recommended slope design parameters, including defect persistence, spacing, intensity of the set, operational requirements and engineering judgement based on experience with similar rock types.

Table 1 Summary of statistical analysis results

Dataset	Indicative slope angle (°)			
	3% PoU	5% PoU	10% PoU	30% PoU
Dataset A: photogrammetry and structure model	35	36	39	45
Dataset B: pit mapping of major structures	47	50	52	57



(a) (b)

Figure 2 Probability of wedge failure statistical analysis results: (a) Dataset A; (b) Dataset B

3.3 Data bias

In the datasets used for this study the main biases and their impacts are as follows:

- Dataset A: photogrammetry mapping under-samples defects that form two-dimensional traces as these are more difficult to sample compared to defects that form three-dimensional (3D) faces. These under-sampled defects can be in any orientation and are typically either shallow or steeply dipping. Photogrammetry also under-samples defects that are subparallel to the slope.
- Dataset B: pit slope face mapping under-samples defects that are subparallel to the slope, and sub-horizontal defects. As this dataset is major structure mapping it also intentionally under-samples shorter defects.

4 Existing slope performance

4.1 Photogrammetry assessment

The as-built performance of the pit was evaluated to compare against the results of statistical analysis. The evaluation was undertaken using a 3D photogrammetry model. The methodology used is detailed in Table 2.

Table 2 Photogrammetry assessment methodology

Assessment stage	Detail
Data collection and modelling	Detailed pit photographs were collected using drones, with a photogrammetry model built using Pix4D software. The resulting model is shown in Figure 3.
Inter-ramp and bench scale assessment	Built surface areas and angles were measured for both the bench and inter-ramp scale, providing the total bench face surface areas at both scales. Areas with wedge instabilities were assessed and measured in the photogrammetry model to measure the unstable bench face surface area. Inter-ramp scale instability was defined as a wedge instability causing total loss of berm (i.e. the wedge intersection dip is equal to or undercuts the inter-ramp slope). The total and unstable areas were used to assess the proportion of the slope bench face undercut by wedges for each area.
Design slope angle assessment	The photogrammetry assessment of the existing pit was then compared to the statistical analysis results presented in Section 3.

4.2 Existing slope performance results

The results of the slope performance assessment shown in Figure 3 found that the proportion of failed slope was:

- Bench scale: between 6 to 22% of bench faces, averaging 15%.
- Inter-ramp scale: 0 to 3%, averaging 2%.

These proportions of failed slope are within industry-accepted amounts of slope instability (Wesseloo & Read 2009).

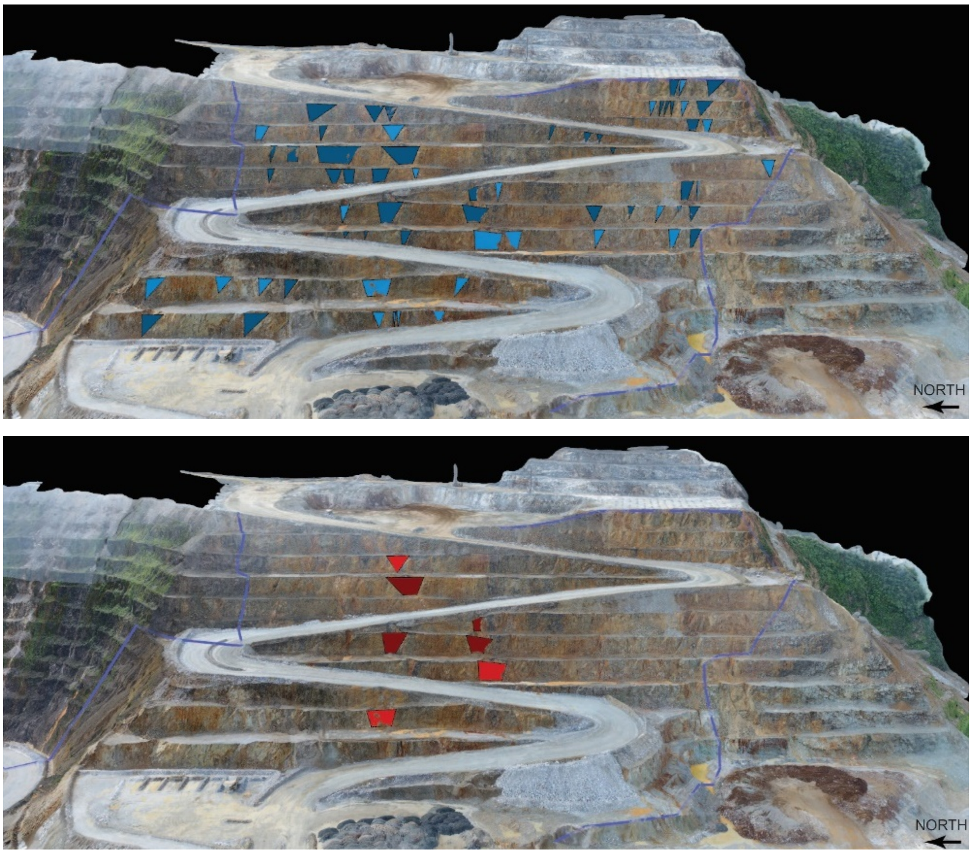


Figure 3 Oblique view (facing east) of the photogrammetry model showing unstable areas: bench scale (blue); inter-ramp scale (red)

4.3 Comparison to the statistical assessment

The results from the statistical analysis and the existing pit slope assessment are compared in Figure 4 and Table 3. The comparisons show that the proportion of the existing pit slope undercut by wedge instability is lower than predicted by the statistical analysis.

This result is due to both the data analysed and the assumptions required for statistical analysis. The datasets analysed contained a relatively low number of defects. In addition, the statistical analysis assumes that the defects being analysed:

- Can occur anywhere along the slope.
- Are persistent enough to contribute to instability.
- Are spatially distributed such that they will form a wedge.

In reality, Set 2 defects are persistent over the inter-ramp scale and are relatively closely spaced. In contrast Set 1 defects are less persistent and are more widely spaced, such that they do not always intersect the slope at locations that cause an instability.

This translates into the relatively infrequent bench scale and rare inter-ramp scale instabilties observed in the existing pit wall exposures as shown in Figure 3.

Table 3 Comparison of statistical results and existing performance

Dataset	Indicative slope angle (°)			
	3% PoU	5% PoU	10% PoU	30% PoU
Dataset A: photogrammetry and structure model	35	36	39	45
Dataset B: pit mapping of major structures	47	50	52	57
Existing performance (approximate)	42	50	60	65

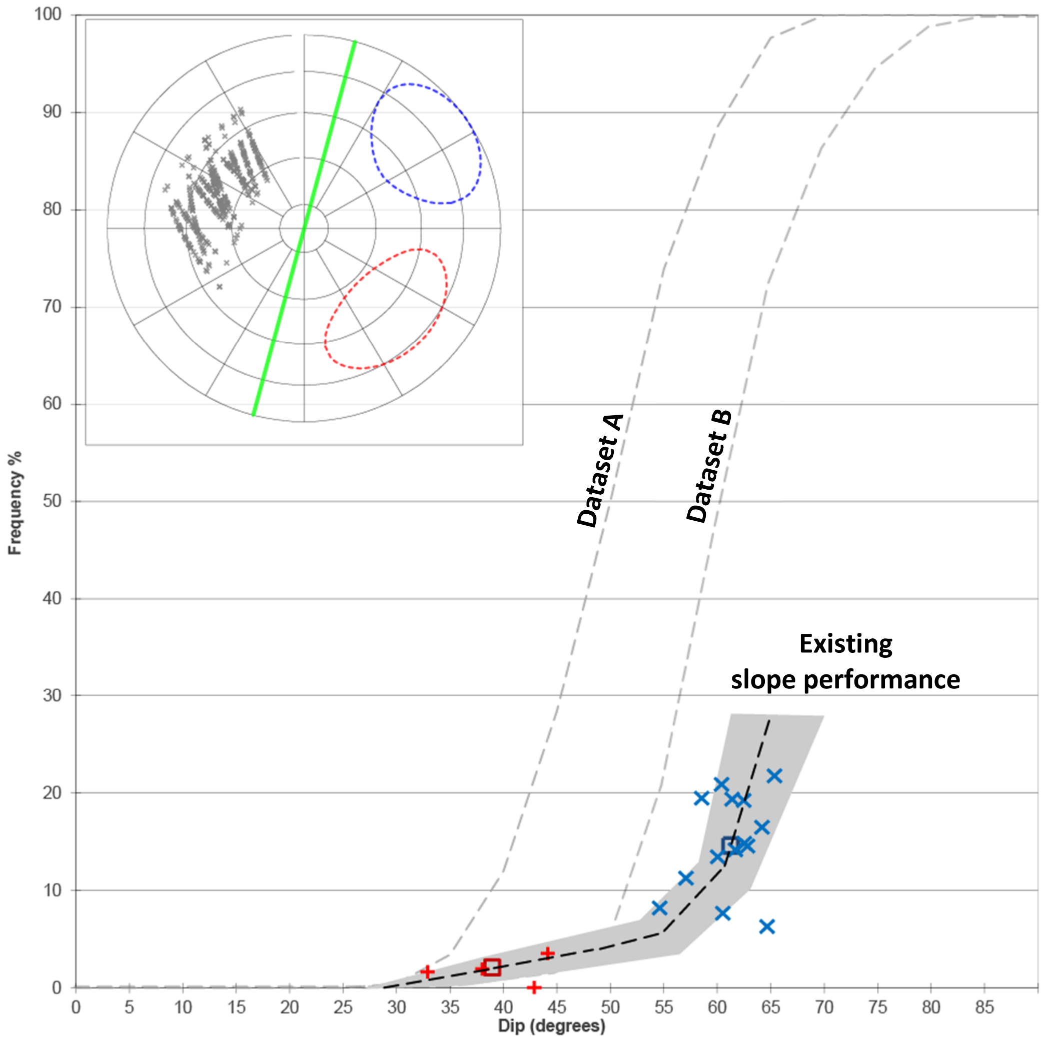


Figure 4 Comparison of statistical analysis results and existing slope performance. Statistical probability of failure results shown as grey dashed lines for both datasets. Failed proportions of the existing slope are shown for bench (blue) and inter-ramp (red) scales. Open square symbols show area weighted average values

5 DFN modelling

5.1 Introduction

DFN models are those that represent a body of rock as an assembly of rock blocks separated by discontinuities. They are stochastic models for the representation of fracture systems. Stochastic analyses are probabilistic simulations based on a set of input parameters and modelled functions to describe the data distributions. Stochastic models provide a powerful means of representing fracture systems based on available structural data. There are numerous examples of where DFN modelling has been applied to rock slope engineering (e.g. Grenon & Hadjigeorgiou 2008; Fowler et al. 2012; Rogers et al. 2006, 2009, 2018; Valerio et al. 2020).

This section presents the DFN modelling that was undertaken for the study. The DFN modelling was undertaken to provide an additional comparison with the statistical analyses and existing slope performance.

5.2 Model inputs

5.2.1 Introduction

The choice of input variables to generate a DFN depends on the purpose of the modelling and the complexity of the geological model. The development of a fracture network for geotechnical applications (such as rock block analysis) requires five primary inputs: orientation, intensity, spatial distribution, size and terminations.

The following sections set out the approach for each of these. The DFN modelling of this study was undertaken using the FracMan® software. Separate DFN models were built for each dataset.

5.2.2 Fracture orientations

Fracture orientation data was taken from Dataset A (photogrammetry and structure model) and Dataset B (pit mapping of major structures). Stereographic analysis showed some clustering of the data, as presented in Figure 1. Two defect sets were identified for each dataset; however, the total number of defects was limited and there was some scatter, resulting in lower Fisher distribution values than anticipated.

Fracture orientations were input into the DFN model using 'bootstrapping' due to the limited number of fractures and scatter observed. Bootstrapping uses orientation data which was imported into the model. Variability is defined by a 'concentration' value. Concentration is similar to a Fisher distribution, however, where a Fisher distribution refers to variability around the average orientation of a defect set, the concentration value applies to variability around the orientation of individual points from the imported data.

5.2.3 Spatial distribution

The spatial distribution of fractures is typically estimated from spacing measurements along sampling lines such as boreholes or mapping scanlines. For this study, analysis of the spatial distribution was attempted on a set-by-set basis using box dimension analyses of the existing slope performance photogrammetry model. The number of defects in each scanline were typically too low for a valid spatial dimension analysis to be undertaken. Priest (1993) suggests that in the absence of a strong clustering pattern, a random fracture location model based on a Poisson process be adopted, consequently the Enhanced Baecher model was adopted for each defect set.

5.2.4 Fracture intensity

Fracture intensity controls the number of fractures in the generated model. It can be quantified using several methods depending on available data. For this study, the number of fractures per unit length of scanline (P_{10}) was adopted to control intensity. The P_{10} was calculated from scanlines along the bench faces in the photogrammetry model. Fourteen scanlines through the photogrammetry model were assessed for P_{10}, with average values and ranges shown in Table 4.

Table 4 Results of P_{10} assessment from scanlines of existing pit

Defect set	No. defects	Total length (m)	Min P_{10}	Max P_{10}	Mean P_{10}
Set 1	31	3,065.5	0	0.02606	0.01011
Set 2	65	3,065.5	0.01037	0.06195	0.02120

5.2.5 Fracture sizes

Fracture size is typically the most difficult parameter to quantify. Some studies have used published fracture lengths for the same lithology at different sites (Weir & Watton 2022), while others have estimated size using the empirical relationship between fracture length and aperture for similar rock types (e.g. Starzec & Andersson 2002). Fracture lengths were recorded for Dataset B (pit mapping), however, these length estimates were typically truncated by bench heights. An assumed equivalent radius and standard deviation of 50 and 25 m, respectively, was adopted. A minimum length of 20 m was adopted following the development of several initial calibration models. A log-normal size distribution was adopted. Fractures were not terminated.

5.3 Stochastic fracture models

5.3.1 Generation

A detailed methodology for the construction of a DFN is provided by several authors, including Dershowitz et al. (2015), Rogers et al. (2009), Starzec & Anderson (2002) and Weir & Fowler (2014). The model details are shown in Figure 5. The generation region for the fracture network was a rectangular prism with dimensions of 1,500 × 2,000 × 2,000 m. This was larger than was required to minimise edge effects. Scanlines with orientations, lengths, and relative locations matching those from the photogrammetry model (used to assess the input P_{10} values, Section 5.2.4) were also added to the model in Figure 5.

Fractures were generated until the average P_{10} values for fractures intersecting the scanlines matched those measured from the existing slopes photogrammetry model scanlines in Table 4. An example of one DFN model realisation is shown in Figure 6.

DFN modelling is an iterative process, with refinement of input parameters undertaken until an acceptable agreement with field measurements is obtained. For this study, the iterative review process was extensive; a total of 15 initial DFN models were generated, with inputs such as orientation and fracture distribution refined before stochastic generation was undertaken. For example, fracture sets were generated and orientations assessed to compare generated orientations from individually defined sets against the bootstrap sampling.

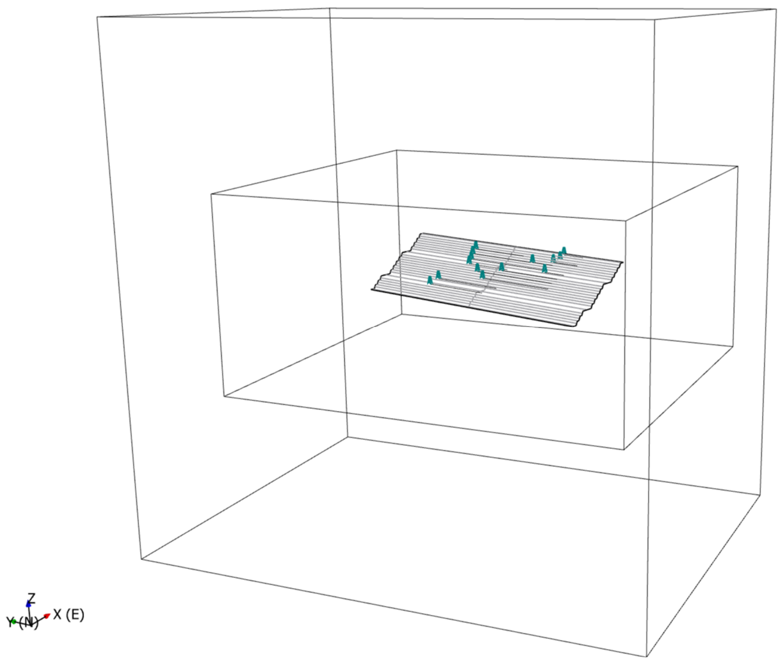


Figure 5 Oblique view of the fracture generation and clipping regions, with pit surface, and scanlines. Lower image shows zoomed view of the pit surface and scanlines

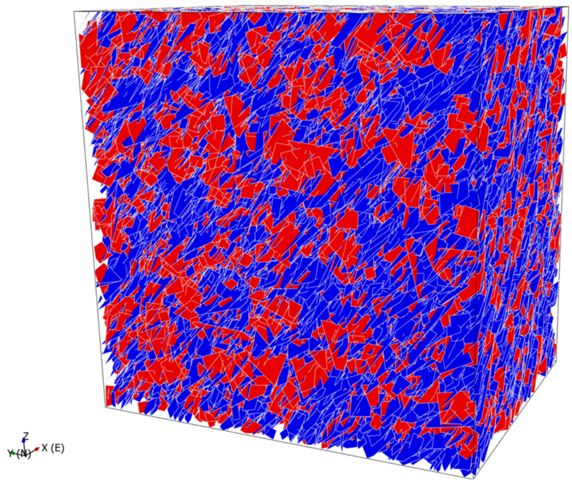


Figure 6 Oblique view of one DFN model realisation, with Set 1 (red) and Set 2 (blue) fractures shown

5.3.2 Model validation

DFN model validation is critical to evaluating the appropriateness of the generated fracture networks. While complex statistical simulations may be developed, the result must be critically verified before stochastic modelling is undertaken. This validation was achieved through comparison of the initial input statistics and those of the generated DFN model.

As part of the validation process the generated network was intersected with the input mapping scanlines. Stereographs of the measured and simulated joints (from one DFN realisation) are shown in Figure 7. The simulated fracture orientations and intensities are considered reasonable and an acceptable match. Vertical cross-sections through the DFN model were also created as a geological sense check.

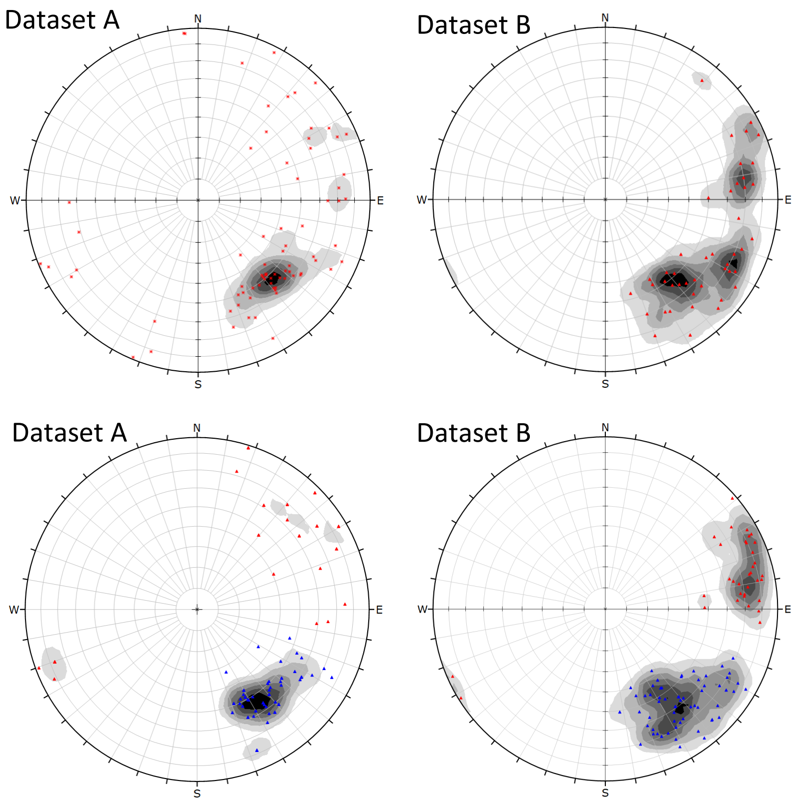


Figure 7 Equal area stereographs showing poles to measured fractures. Upper images are measured data and lower images are DFN-simulated data intersecting scanlines

5.4 Slope stability analysis

5.4.1 Introduction

Following validation of a representative DFN, 100 fracture networks were generated for each dataset for stochastic stability analyses. The rock wedge module of the FracMan software was used to simulate block stability for a representative pit surface. Both stable and unstable blocks are found by the rock wedge analysis. It is important to note that the program will only find blocks that are perfectly formed by fractures. It also only considers block faces that intersect the slope face when evaluating possible sliding faces and reports the maximum Factor of Safety (FoS) from all possible sliding faces.

It should be noted that the rock block calculations are based on a stochastic fracture model, thus the derived block locations are not real block positions. Instead, the analysis provides an indication of the number and size of the blocks that may be expected for a particular slope angle and aspect, which must be related back to the actual length of the pit slope.

5.4.2 Analysis inputs

A representative pit slope was built as a surface in the DFN software, with pit slope parameters presented in Table 5. The pit slope aspect is 285° and it has two 50 m wide ramps that were included to create a pit slope representative of the as-built surface, with examples shown in Figures 5 and 8. Rock mass and defect strength parameters are summarised in Section 2.1. No water pressure was assumed.

Table 5 Analysed pit slope geometry

Bench face angle (°)	Bench height (m)	Berm width (m)	Inter-ramp angle (°)	Overall slope height (m)	Overall angle (°)	Analysed slope length (m)
65	15	10	41	180	32	1,000

5.4.3 DFN analysis results

The software calculates several attributes for each block, including volume, mass, failed surface area and a FoS based on a stability analysis.

The results from the rock block analysis are summarised in Table 6 and an example output from a single model from each dataset is provided in Figure 8. The results of the analysis were considered both in terms of the number of blocks and the area of pit face failed, with graphs presented in Figure 9.

Table 6 DFN analysis results

Dataset	Number of total blocks			Number of unstable blocks			Area of pit face failed (%)		
	Min	Max	Mean	Min	Max	Mean	Min	Max	Mean
A	12	147	50	5	51	19	0.04	1.7	0.4
B	18	119	52	3	27	11	0.01	0.6	0.2

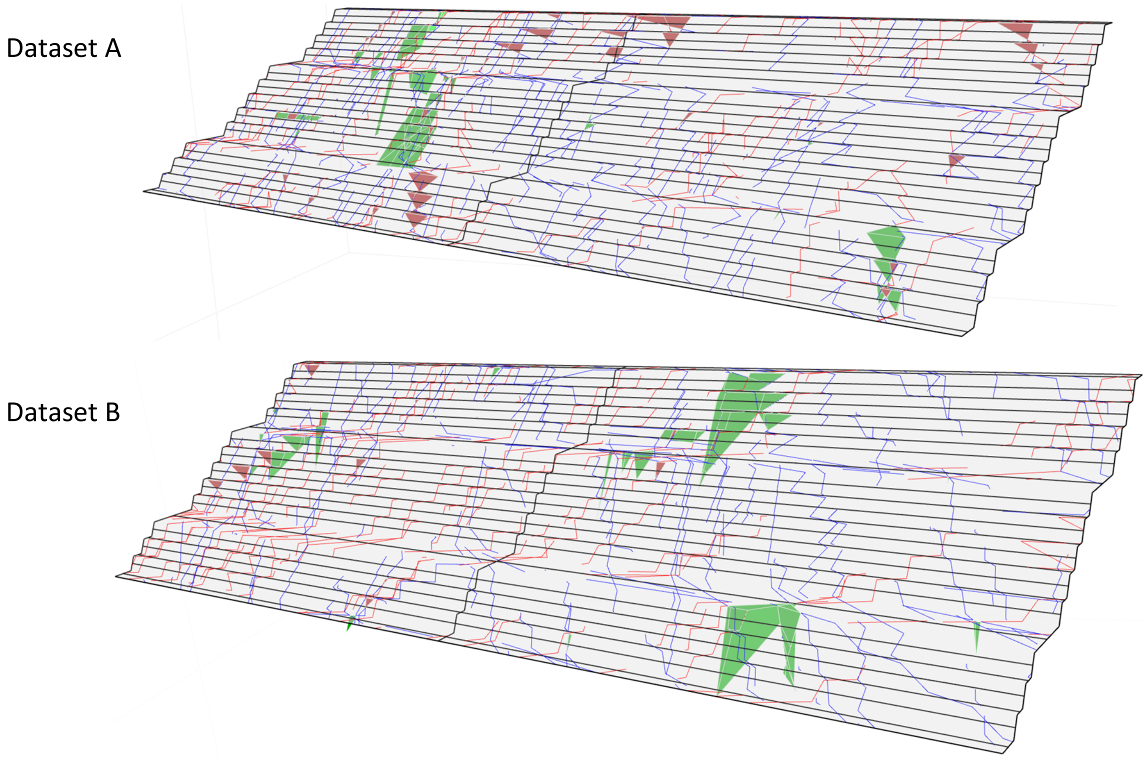


Figure 8 **Examples of blocks identified by the analysis. Stable blocks are green and unstable blocks are red. The section is viewed towards north and the pit slope is 180 m high**

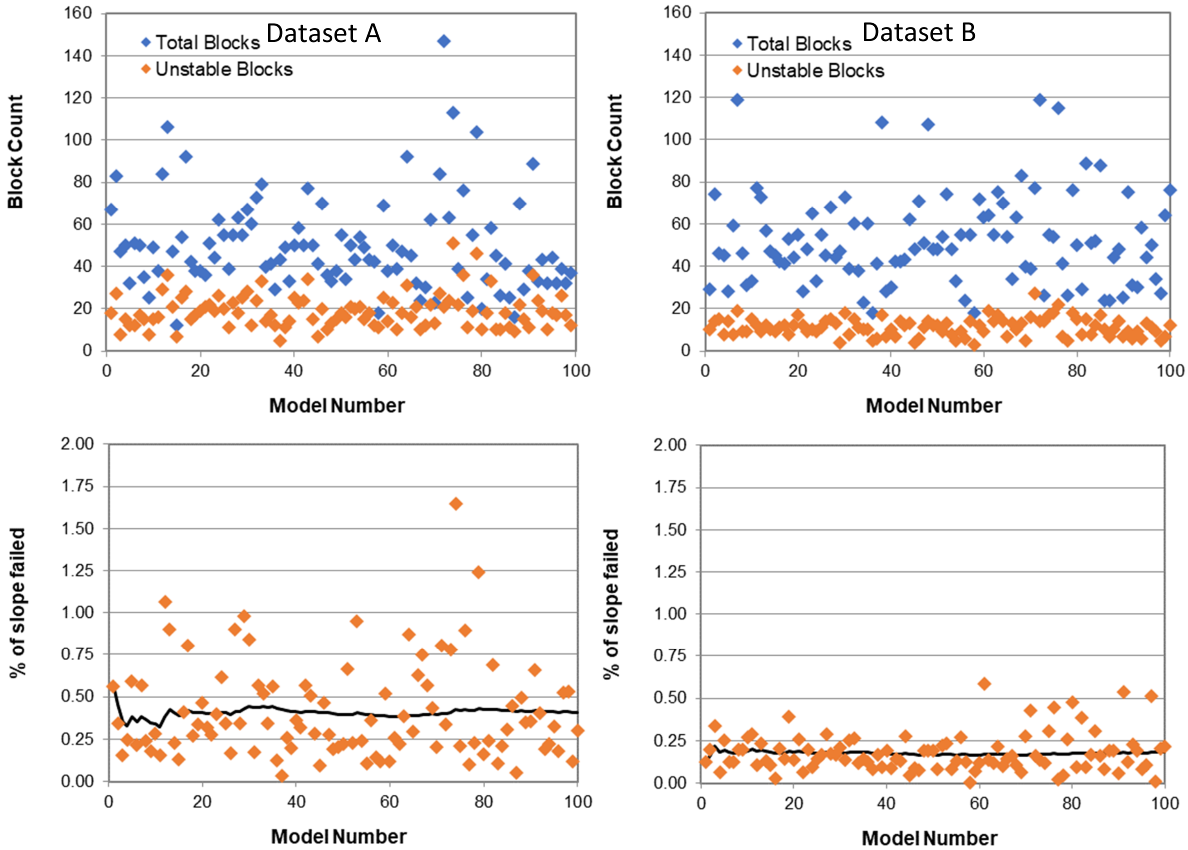


Figure 9 **Results from the rock wedge analysis. Upper graphs show total and unstable blocks. Lower graphs show the area of pit slope failure**

The results show that the area of slope failure is less than 2% and the failures are relatively small. Dataset A has a larger number of unstable blocks and failed pit slope area compared to Dataset B. The average area of pit slope failed is 0.2 to 0.4%. This is lower than the as-built assessment.

The limited number of unstable blocks (and consequently failed areas of the pit slope) may be attributed to the following:

- Data bias in the input data.
- Fracture size assumptions – Defect length data was limited and fracture sizes were assumed. An increased standard deviation may have allowed for increased variability in fracture sizes (i.e. a small number of large structures), which in turn may have increased the amount of pit slope failed. Assessment of defect lengths using Dataset A photogrammetry data would also be possible.
- The analysis methodology, which necessitates blocks to be defined by fully intersecting structures and the pit face – A small intact bridge can prevent blocks from being recognised and therefore they are not being assessed by the rock wedge module. There are some new software packages that can assess the stability of these rock bridges; however, these were not considered in this paper.
- Inclusion of ramps, reducing the overall angle of the pit slope assessed.
- Exclusion of water pressure.

The above points (excluding data bias in the input data) are to be addressed in future work.

5.5 Comparison

The DFN results show that the area of slope failed is less than 2% and the failures are relatively small. The average area of pit slope failed is 0.2 to 0.4%. In comparison with the statistical assessment and observed slope performance:

- The DFN models indicate a lower area of slope is likely to fail at similar slope angles than the statistical assessments undertaken using the same datasets: i.e. the results of the DFN modelling are less conservative. This is as to be expected as defect lengths and spatial distribution are considered in the DFN models.
- Similar (up to 2%) but typically less instability than the existing slope performance, possibly due to the reasons noted in Section 5.4.3. Further work addressing shortcomings associated with defect size is proposed.

6 Conclusion

Through a case study, this paper aimed to compare current industry-accepted methods for pit slope design in structurally controlled settings with the outcomes obtained from a DFN approach. The performance of the existing slope was used as a benchmark for evaluating these different methods.

Additionally, this paper delved into the inputs, development, validation and generation of multiple stochastic fracture models within the DFN framework. These models were subsequently employed in stability analyses using a representative pit slope, enabling an assessment of the potential variations in slope performance and stability based on a given set of input fracture statistics.

The findings revealed that the industry-accepted methods for pit slope design, owing to certain required assumptions, tend to be conservative. Consequently, they indicated higher probabilities of failure compared to what was observed in the existing slope. On the other hand, the DFN models produced a range of results that were closer to the performance of the existing slope. However, it is worth noting that most of the DFN models indicated lower proportions of pit slope failure than what was actually observed in the existing slope.

To address the observed variations between the DFN models and the existing slope, further investigation is warranted. Future works will focus on identifying the potential reasons underlying these differences and include additional case studies. By refining and expanding our understanding of slope stability in structurally controlled settings, the development of more accurate and reliable pit slope design methodologies that better align with real-world performance can be advanced.

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