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Using discrete fracture networks to assess how defect size and other variables affect the formation of wedge failures in open pits

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Abstract

Wedge failure is the sliding of a rock block on the line of intersection of two discontinuities oriented to form a wedge, and is influenced by geological and geometric factors. This paper compares traditional industry-accepted pit slope design methods with a discrete fracture network (DFN) approach. The geotechnical model and observed performance from a case study site are presented and used to benchmark the analysis results.

Major discontinuities measured from photogrammetry and pit face mapping data from the case study site were used for the analysis, and the observed performance was assessed from a photogrammetry point cloud created using drones.

Both the industry-accepted and DFN methods indicated that the probability of failure increases with steeper slope angles, however, the industry-accepted methods overestimated this probability compared to the observed slope performance. The DFN models were more closely aligned with the observed slope performance.

The study highlights that discontinuity size and the spatial distribution of discontinuity sets are key drivers of the potential for wedge instability and the resulting slope performance, which have flow-on impacts for developing reliable pit slope designs.

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

1 Introduction

Wedge failure, defined by the sliding of a rock block along the intersection of two discontinuities, is controlled by a range of geological and geometric factors. Although this type of failure can occur under a range of conditions, it typically depends on the size, orientation, and spatial distribution of the intersecting discontinuity sets.

Commonly used industry-accepted statistical methods for assessing the probability of structural instability typically require assumptions about defect length and distribution. Variation between these assumptions and the actual structural conditions can lead to differences between the predicted and observed slope performance. These differences could manifest as excavated pit slopes that are aggressive or conservative, which can have either safety or economic impacts.

This paper compares an industry-accepted approach to pit slope design in structurally controlled environments to a discrete fracture network (DFN) approach. This paper also aims to evaluate how defect size and other variables affect the formation of wedge failures in open pits by building on earlier work (Watton & Weir 2023). The words 'discontinuity' and 'fracture' are used interchangeably.

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2 Initial discrete fracture network modelling

Initial DFN modelling (Watton & Weir 2023) was undertaken as a preliminary comparison of industry-accepted analysis methods for pit slope design in structurally controlled settings with a DFN approach. The performance of an existing slope was used as a benchmark for evaluating these different methods. This work found that the industry-accepted method was conservative, predicting higher probabilities of failure compared to the observed slope performance. The DFN models, however, mostly predicted lower probabilities of failure compared to the existing slope performance. These lower probabilities in the initial modelling were attributed to the following possible reasons:

- Fracture size assumptions.
- Inclusion of ramps, reducing the overall angle of the pit slope assessed.
- Data bias in the input data. This is inherent in geotechnical data collection and can be understood but not changed. Data bias is discussed further in Section 3.3.
- The analysis methodology, which requires blocks to be bound by intersecting discontinuities and the pit face. A small intact bridge can prevent blocks from being formed and assessed by the analysis software. There are some new software packages that can assess the stability of these rock bridges, however, they are not considered in this paper.
- Exclusion of water pressure.

Fracture sizes and slope angles are addressed in this paper. The possibility of small intact rock bridges was partially mitigated for this paper by increasing the number of models analysed. The impact of water pressures was not assessed and should be considered in future work.

3 Case study site

3.1 Setting and geotechnical model

The study area is a 150 m high existing pit slope excavated in a high-strength igneous rock mass at a large copper porphyry mining operation. The intact rock strength is approximately 100 MPa, indicating that slope stability is structurally controlled at the scale assessed.

The rock mass is intersected by major discontinuities that are moderately to steeply dipping and persist over multiple benches (i.e. over 50 m in length). These discontinuities are the main discontinuity sets influencing slope performance. The slope faces an azimuth of 285°. A friction angle of 20° was adopted for the major discontinuities based on observed conditions. All major discontinuities dip at angles steeper than the adopted friction angle. For this study, major discontinuities are multi-bench to inter-ramp scale geotechnical structures, and are typically faults and shears.

3.2 Structural data

Two structural datasets were utilised in the analysis:

- Dataset A: photogrammetry mapping of 77 major discontinuities.
- Dataset B: pit mapping of 45 major discontinuities.

Defect orientations of both datasets and the identification of two discontinuity sets for each dataset are shown in Figure 1. Both datasets comprise multi-bench to inter-ramp scale discontinuities exposed in the pit face and the data was collected from the same pit area.

Kinematic analysis was undertaken for the pit slope. Kinematic analysis is a screening tool that uses the pit wall azimuth, the average orientations of the main discontinuity sets influencing slope behaviour, and their adopted defect shear strengths. These inputs are used to evaluate the likely failure mechanisms and the relevant conditions for subsequent statistical analysis.

The kinematic analysis identified that wedge failure was the critical instability mechanism, which was consistent with field observations of slope performance. For this case study, wedge failure includes sliding along a single discontinuity with lateral release.

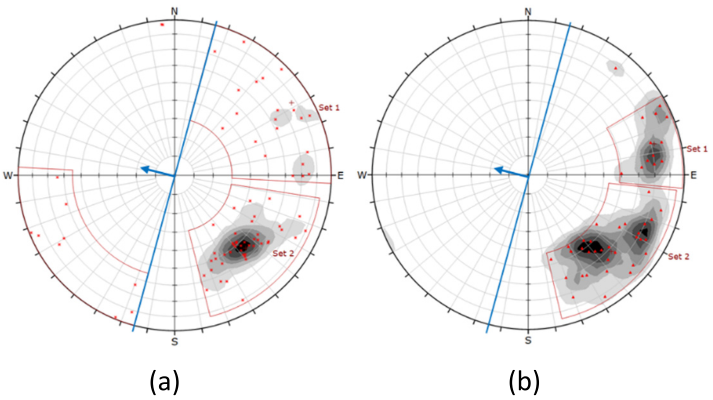


Figure 1 Equal area stereographs showing poles to measured major discontinuities and identified sets. (a) Dataset A; (b) Dataset B. The pit slope azimuth of 285° is indicated by the blue line and arrow

3.3 Data bias

Each data collection method used has inherent data biases. The main data biases and their associated impacts on each of the datasets used for this paper are summarised below:

- Dataset A (photogrammetry mapping): this method under-samples discontinuities that appear as a line (two-dimensional trace) as they are difficult to measure during interpretation of the data, unless they extend over more than one bench. The under-sampled discontinuities can be in any orientation but are mostly normal to the pit slope. Additionally, photogrammetry data under-samples discontinuities that are sub-parallel to the pit slope.
- Dataset B (pit face mapping): this method also under-samples discontinuities that are sub-parallel to the pit slope, as well as sub-horizontal discontinuities.

These under-sampled areas are also known as 'blind zones'. A stereographic example of the discontinuity blind zones for each dataset is presented in Figure 2. As both datasets were targeted at major discontinuities, they intentionally under-sample shorter discontinuities. A good explanation of the various biases associated with different data collection methods and orientations relative to the mapped face are presented in Fowler (2013) and Laing (2005).

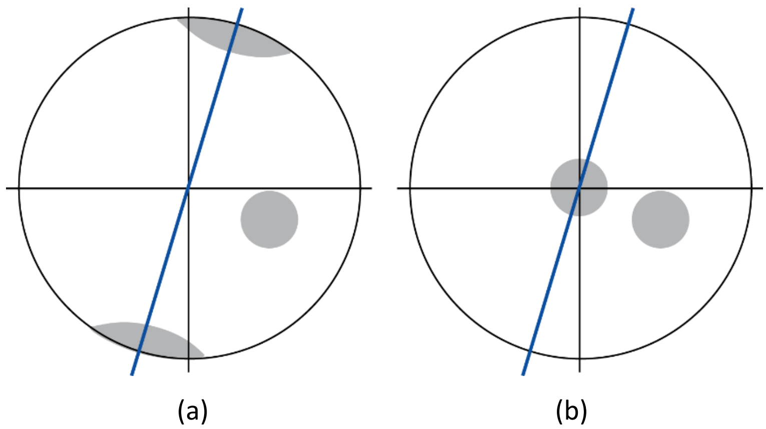


Figure 2 Theoretical blind zones assuming a slope azimuth of 285°. (a) Dataset A: photogrammetry mapping; (b) Dataset B: pit face mapping (after Fowler 2013 and Laing 2005)

4 Probability of wedges undercutting the slope

4.1 Theory

Probability of undercutting (Pou) is a statistical, risk-based approach that considers the variability in discontinuity orientations and lengths within a defined area, in contrast to the average orientations used in kinematic analysis. This method evaluates how changes to the slope angle impact the analysis results. 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 proportion of discontinuities in unstable orientations; that is, the chance a slope is undercut by an individual discontinuity or wedge intersection

P_e = the percentage of discontinuities that are sufficiently persistent to result in failure for the slope height analysed.

For this study, discontinuities are assumed to be continuous over the slope height being analysed; that is, $P_e = 1$. As a result, P_f is equal to the proportion of discontinuities that undercut the slope $P_f = P_u$. This assumption is conservative as it assumes all discontinuities included in the analysis can contribute to structural instability and does not account for their spatial distribution.

Conventional open pit slope design typically targets an acceptable slope performance criterion, which can be expressed as an acceptable probability of failure of 5% for inter-ramp scale and 30% for bench scale performance (e.g. Wesseloo & Read 2009). These values are used as a guide and are adjusted using engineering judgement based on experience in similar geotechnical settings.

4.2 Statistical analysis results

The statistical analysis results for Datasets A and B are presented in Table 1. A range of indicative slope angles are provided for comparison with the observed slope performance, as discussed in Section 5. The results are also illustrated graphically in Figure 3, which shows that the proportion of wedges increases with steeper slopes, for a given slope azimuth.

The results suggest a range of approximate pit slope angles based on the accepted probability of failure and the dataset used to inform the design. The angles range from 35 to 50° (inter-ramp angles) and 40 to 60° (bench face angles). These ranges have implications for both slope stability and economics, particularly with respect to inter-ramp angles; where a range of 15° will have an impact on the proportion of slope that may fail, the crest and toe positions, and the amount of material required to be excavated.

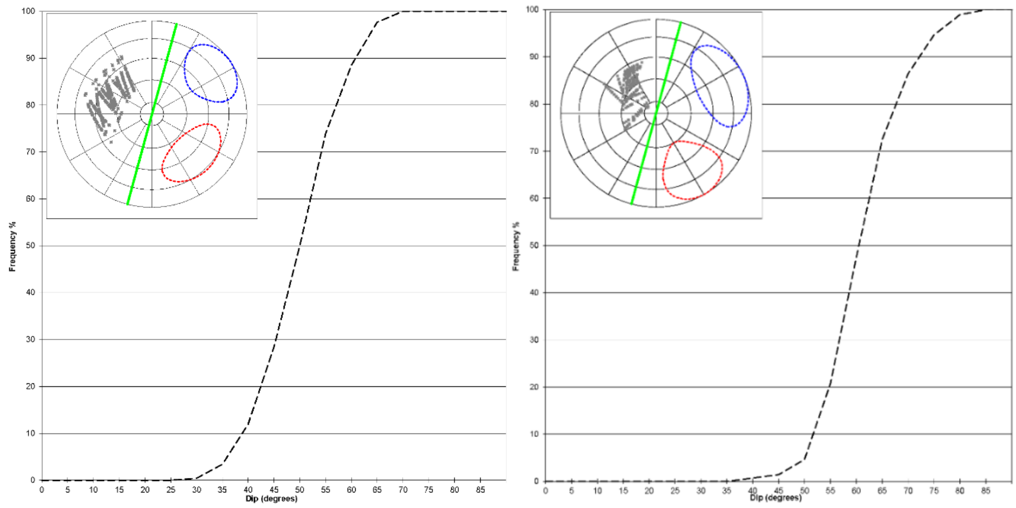
The indicative slope angles are a guide and the recommended slope design geometry includes the consideration of other factors, including:

- discontinuity spacing, intensity, and persistence
- any operational constraints (e.g. mining equipment or infrastructure requirements)
- engineering judgement based on experience in similar geotechnical settings.

Table 1 Summary of statistical analysis results

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

PoU = probability of undercutting



(a) (b)

Figure 3 Probability of wedge failure statistical analysis results. (a) Dataset A; (b) Dataset B (Watton & Weir 2023)

5 Existing slope performance

5.1 Photogrammetry assessment

The observed performance of the pit slope was assessed to enable comparison against the statistical analysis results. The assessment was undertaken using a 3D photogrammetry model. The methodology used is outlined in Table 2.

Table 2 Photogrammetry assessment methodology

Assessment stage	Detail
Data collection and modelling	High-resolution imagery of the pit was acquired using a drone, and a photogrammetry point cloud model subsequently developed using Pix4D software. The resulting model is presented in Figure 4.
Total and failed pit slope surface area assessment of observed slope performance	Surface areas and slope angles were measured at both the bench and inter-ramp scale to enable calculation of the total pit slope surface areas. Areas exhibiting wedge instabilities were identified and the failed pit slope surface area measured using the photogrammetry model. Inter-ramp scale instabilities were defined as wedge failures that resulted in complete loss of berm widths; that is, where the wedge intersection dip is flatter than the bench face angle The total and failed pit slope surface areas were used to assess the proportion of the pit slope undercut by wedges.
Comparison to statistical analysis	The photogrammetry-based observed slope performance was then compared to the statistical analysis results presented in Section 4.

5.2 Failed pit slope surface area

The observed slope performance assessment, as illustrated in Figure 4, indicated the following proportions of failed slope areas:

- Inter-ramp scale: from 0 to 3%, with a weighted average of 2%.
- Bench scale: from 6 to 22%, with a weighted average of 15%.

These proportions of failed slope areas are within and slightly below industry-accepted slope performance criteria (Wesseloo & Read 2009).

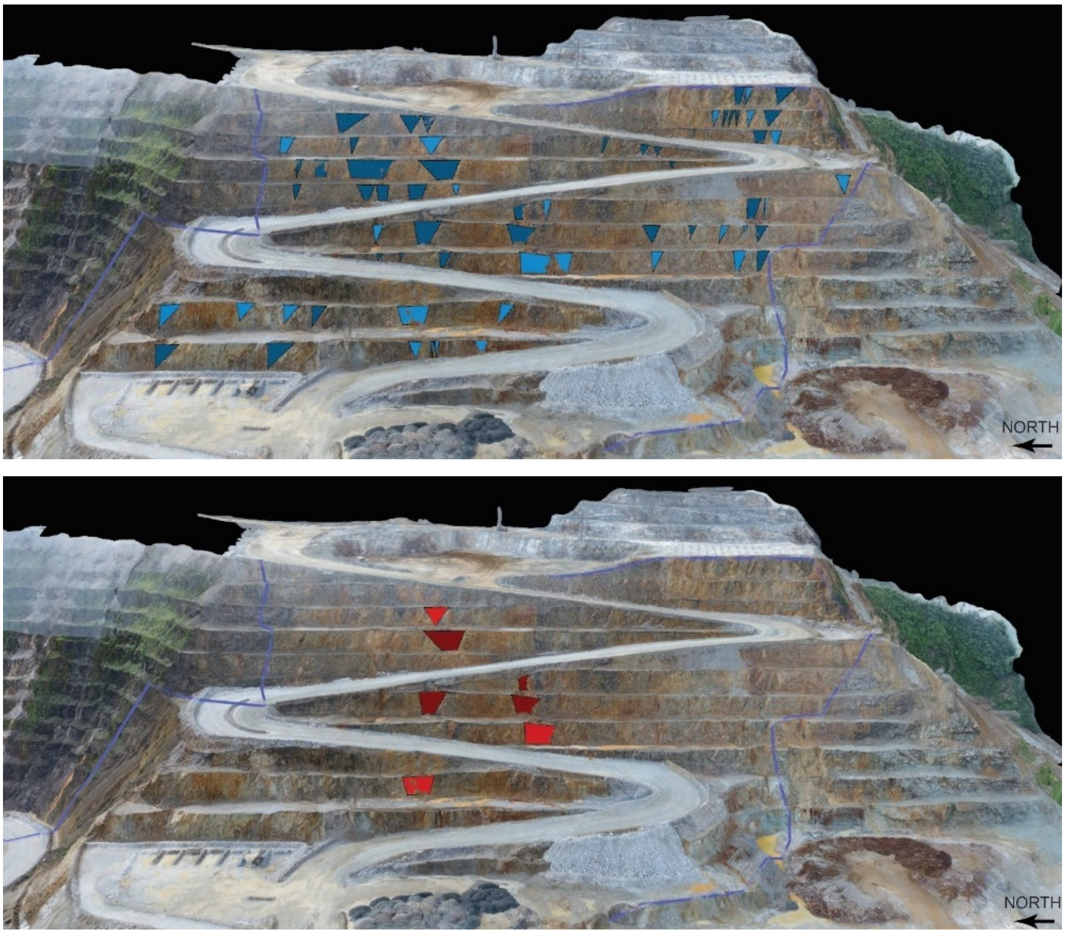


Figure 4 Oblique view (facing east) of the photogrammetry model showing unstable areas: bench scale (blue) and inter-ramp scale (red) (Watton & Weir 2023)

5.3 Comparison to the statistical assessment

A comparison between the statistical analysis results and the observed pit slope performance is presented in Figure 5 and Table 3. The comparison indicates that the proportion of slope undercut by wedge instability in the existing pit is less than that predicted by the statistical analysis.

This is due to both the characteristics of the input data and the assumptions used in the statistical analysis. The datasets used in the analysis contained a relatively limited number of structural measurements. Additionally, the PoU method assumes that the discontinuities selected for analysis:

- occur anywhere along the slope
- are sufficiently persistent to contribute to pit slope instability
- are spatially distributed in a manner that allows wedge formation.

In the excavated slope, discontinuities from Set 2 (Figure 1) are more closely spaced and more frequently observed in the pit slope, whereas discontinuities from Set 1 (Figure 1) are more dispersed and form a more diffuse set. As a result, the discontinuities from both sets do not consistently intersect the slope in positions that contribute to wedge instability.

This translates into the relatively infrequent instabilities observed in the excavated pit wall exposures, particularly for inter-ramp scale instabilities as shown in Figure 4.

Table 3 Comparison of statistical results and existing performance

Dataset	Indicative slope angle (°)			
	3% PoU	5% PoU	10% PoU	30% PoU
Dataset A: photogrammetry model	35	36	39	45
Dataset B: pit face mapping	47	50	52	57
Observed performance (approximate)	45	50	60	70

PoU = probability of undercutting

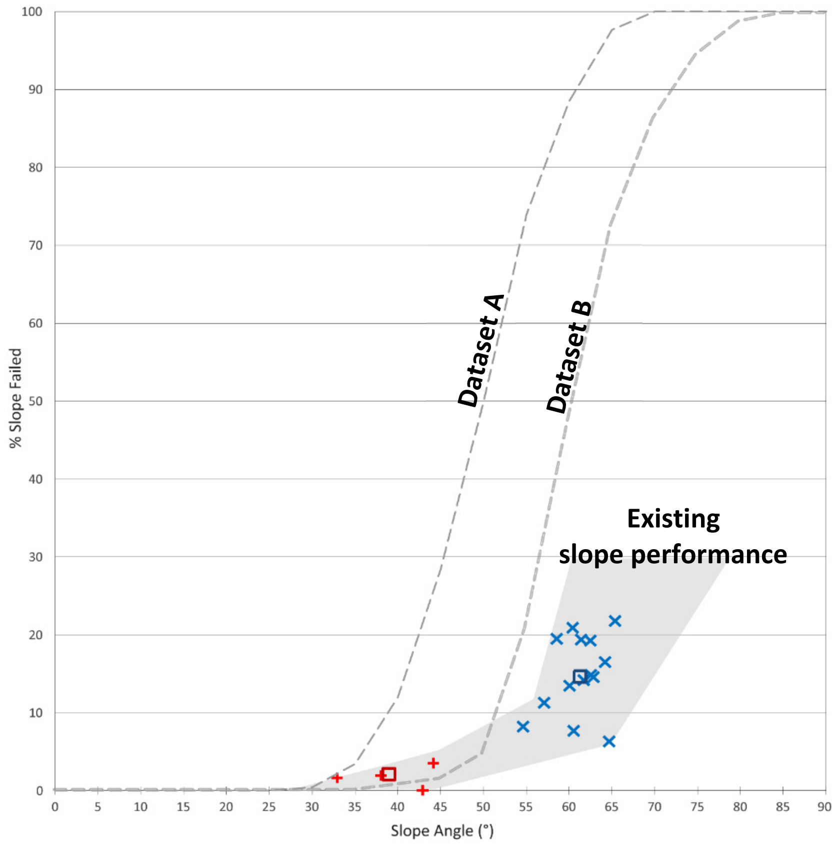


Figure 5 Comparison of statistical analysis results and excavated slope performance. Statistical probability of failure results shown as grey dashed lines for each dataset. Failed proportions of the excavated pit slope are shown for bench (blue) and inter-ramp (red) scales. Open square symbols show area weighted average values. Grey shading shows the range of excavated slope performance measurements

6 Discrete fracture network modelling

6.1 Introduction

DFN models represent rock masses as a collection of discrete rock blocks bounded by discontinuities. These are stochastic models that simulate fracture systems using probabilistic methods, based on defined input parameters and statistical distributions. DFN modelling offers a robust approach for characterising fracture networks using available structural data and is widely applied in rock slope engineering (e.g. Grenon & Hadjigeorgiou 2008; Fowler et al. 2012; Rogers et al. 2006, 2009, 2018; Valerio et al. 2020). This section outlines the DFN modelling conducted for this case study. The objective of the modelling was to provide an additional comparison to the observed slope performance and the industry-accepted statistical method of structural analysis. The modelling also aimed to address some of the limitations noted in Section 2.

6.2 Model inputs

The input parameters used for generating the DFN models were guided by the goals of the modelling (i.e. modelling inter-ramp scale geotechnical structures) and the geotechnical model complexity. Five key inputs are required for DFN models used in geotechnical applications such as rock block analysis: discontinuity orientation, intensity, spatial distribution, size, and terminations. This section describes the methodology used in developing each of these input parameters. DFN modelling for this study was undertaken using FracMan® software, with separate models constructed for each dataset. The input parameters for the DFN modelling are summarised in Table 4.

Table 4 Summary of discrete fracture network input parameters

Parameter	Data	Approach	Adopted parameter and comments
Fracture orientation	Orientations and sets from each dataset as shown in Figure 1	Typically uses mean orientations and Fisher distribution values	Controls the orientation of discontinuities Limited number of discontinuities resulted in lower Fisher distribution values than expected. Bootstrapping was subsequently adopted
Spatial distribution	Observed fractures in photogrammetry model, measured along 14 horizontal scanlines	Typically assessed using box dimension analyses	Controls the distribution or clustering of discontinuities Number of discontinuities were too low for a valid analysis. Adopted random discontinuity locations based on Poisson process per Priest (1993). Enhanced Baecher model adopted
Fracture intensity	Observed fractures in photogrammetry model, measured along 14 horizontal scanlines	Quantified using P_{10} values (number of fractures per unit length of scanline)	Controls the number of discontinuities in the model. Adopted mean and ranges below Set 1: P_{10}: 0.01011 (0 to 0.02606) Set 2: P_{10}: 0.02120 (0.01037 to 0.06195)
Fracture sizes	Lengths not recorded. The data was collected for other purposes, and accurate recording of defect lengths was omitted	Others have used published lengths (e.g. Weir & Watton 2022), or empirical relationships between length and aperture (e.g. Starzec & Andersson 2002)	Controls discontinuity size. Typically, the most difficult parameter to quantify Four different equivalent fracture radii (ER) analysed to assess the impacts of discontinuity length. Standard deviation was assumed as half the ER: ER: 25, 50, 100 and 200 m Minimum length of 20 m based on calibration models Log-normal size distribution
Fracture terminations	Terminations not recorded	–	Fractures were not terminated

6.3 Stochastic discrete fracture network models

6.3.1 Generation

A detailed methodology for building DFN models is outlined by others including Dershowitz et al. (2015), Rogers et al. (2009), Starzec & Anderson (2002), and Weir & Fowler (2014). The model setup used in this study is shown in Figure 6. The DFN generation volume was a rectangular prism measuring 1,500 × 2,000 × 2,000 m. This was intentionally larger than the pit slope to minimise boundary effects. Scanlines were added to the DFN model that had the same orientations, lengths, and relative positions as those from the photogrammetry point cloud model. The DFN scanlines are shown in Figure 6.

Fractures were stochastically generated within the DFN generation volume until the average P_{10} values for the DFN scanlines matched the average P_{10} values from the photogrammetry point cloud model, i.e. the fracture intensity of the DFN model matches that measured from the excavated pit slope. An example of the fractures generated for one DFN model and from two different fracture radii are shown in Figure 7.

The DFN input parameters (Table 4) were iteratively refined until the fractures generated were consistent with the observed input data. Throughout this review process, inputs such as fracture orientation and distribution were checked and adjusted before finalising the DFN input parameters and commencing the stochastic generation of many DFN models and undertaking rock wedge analyses. A total of 15 iterative models were generated for this study. An example of this process is the comparison of DFN-generated fracture orientations and sets against the bootstrapped input data.

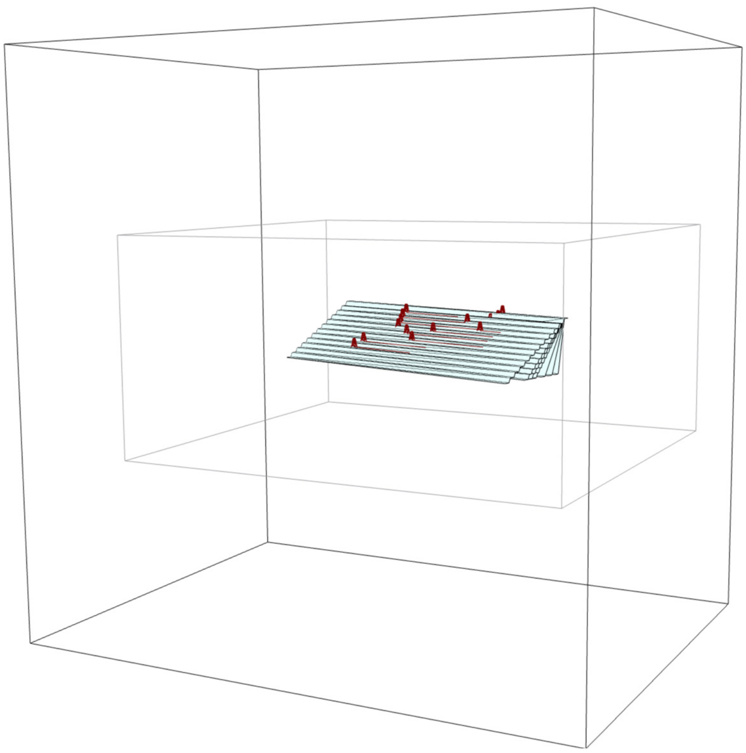
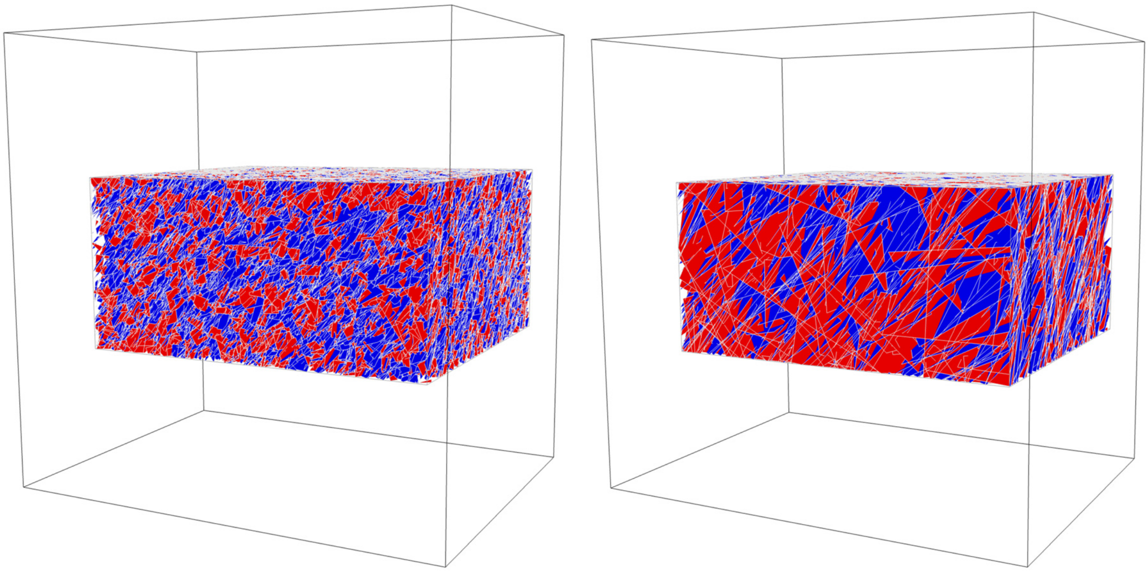


Figure 6 Oblique view of the fracture generation (black) and clipping (grey) regions, with scanlines, and pit surfaces



(a) (b)

Figure 7 Oblique view of fractures generated for one DFN model and from two different fracture radii: (a) 25 m; (b) 200 m. Set 1 (red) and Set 2 (blue) fractures shown

6.3.2 Model validation

Validation of the DFN model was essential to ensure that the generated fractures accurately represented the measured discontinuities from each dataset. Although advanced statistical simulations can be developed, the outputs must be critically assessed before proceeding with stochastic analyses. In this study, validation was undertaken by comparing the statistics of the generated DFN with the input datasets.

As part of this process, the DFN-generated fractures were intersected with the DFN scanlines. Stereographs comparing discontinuity orientations from the input data and one DFN realisation are presented in Figure 8.

The discontinuity orientations and intensities from the input datasets and the DFN model were considered an acceptable match. Additionally, cross-sections through the DFN were produced to provide a geological sense check and confirm the plausibility of the modelled fracture geometry.

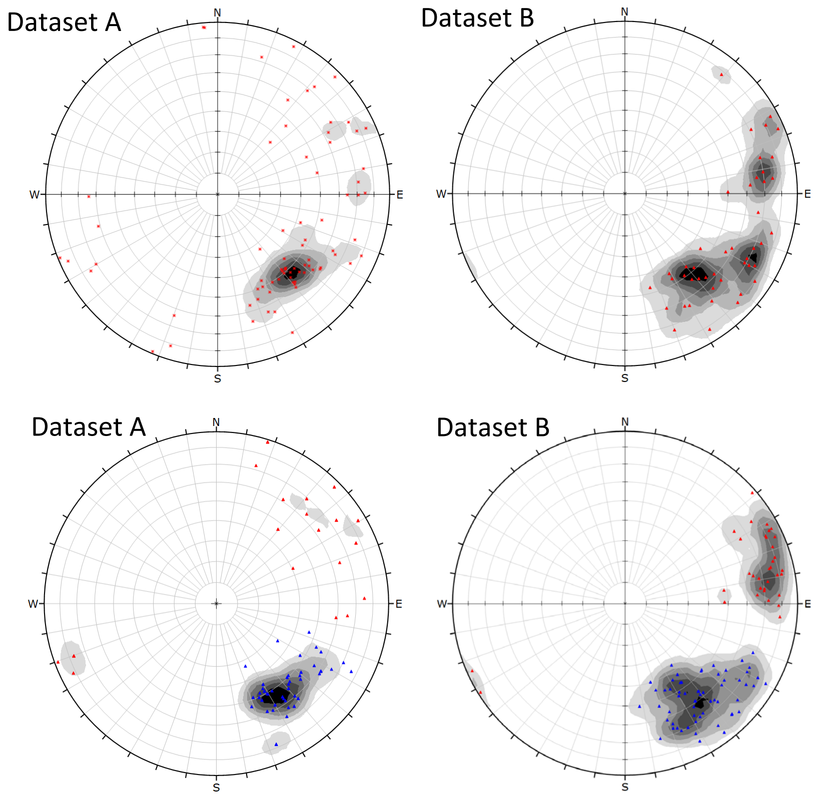


Figure 8 Equal area stereographs showing poles to measured discontinuities. Upper images are measured data and lower images are DFN-simulated fractures intersecting scanlines (Watton & Weir 2023)

6.4 Slope stability analysis

6.4.1 Introduction

After DFN model validation and confirmation of the DFN input parameters, 200 fracture networks were generated for each dataset and fracture size to undertake stochastic stability analysis. Block stability was assessed using the Rock Wedge module within FracMan. The analysis identifies both stable and unstable blocks, however, it is limited to blocks that are fully bound by intersecting fractures. Only those block faces that intersect the pit slope were considered when evaluating potential sliding, and the software reports the maximum Factor of Safety (FoS) across all feasible sliding surfaces for each block.

The modelled block positions do not represent in situ block locations as the rock wedge calculations are based on stochastic fracture models. Instead, the analysis indicates a probabilistic estimate of the likely number, size, and pit face area of structurally bound blocks for given slope azimuths and angles, which is used with the total area of the pit slope analysed to calculate the proportion of failed slope. The fracture networks were clipped to a smaller rectangular prism (Figure 7) measuring 1,350 × 1,550 × 750 m to reduce computation times.

6.4.2 Analysis inputs

Seven pit slopes were built as surfaces in the DFN software, with slope geometry parameters presented in Table 5. The slopes face an azimuth of 285° and exclude ramps. Examples of the pit slope surfaces are shown in Figures 6 and 10. Benches were not included for the steeper slope surface geometries. No water pressure was included in the analysis.

Table 5 Analysed pit slope geometries

Inter-ramp angle (°)	Bench face angle (°)	Bench height (m)	Berm width (m)	Overall slope height (m)	Analysed slope length (m)
30	65	15	19		
41	65	15	10		
45	65	15	8		
49	65	15	6	180	1,000
60	–	–	–		
70	–	–	–		
80	–	–	–		

6.4.3 Discrete fracture network analysis results

A total of 11,200 rock wedge analyses were undertaken, comprising 200 DFN models each analysed with 4 fracture radii, 7 slope geometries, and the 2 datasets.

The software calculates several attributes for each block, including volume, mass, failed surface area, and a FoS based on the rock wedge stability analysis. The results from the rock wedge analysis are summarised in Table 6 and presented as box and whisker plots in Figure 9. An example of the blocks identified by the DFN analysis are presented in Figure 10.

The results were evaluated based on the failed area of the pit slope to allow comparison to the as-built performance. The results indicate the following:

- For both datasets, the average area of slope failed ranges from 0.1 to 58% and increases with both increasing discontinuity size and slope angle.
- However, for shorter discontinuities (i.e. a 25 and 50 m radius), the average area of slope failed remains low for all slope angles analysed.
- Dataset A typically has a higher average area of failed slope area compared to Dataset B for similar fracture sizes and slope angles.
- Some of the results are a reasonable match for the observed slope performance. Section 6.6 provides further comparison and discussion.

Figure 9 shows that some of the outlier points indicate over 100% failed area of the pit slope (e.g. Figure 9, 200 m radius, 80° slope angle). This is due to an artefact in the modelling that impacts area measurements for blocks identified in the upper portion of the slope. It is not anticipated to have a significant impact on the results but will be checked in further work. The following aspects could also impact the results:

- Data bias in the input data, and the relatively limited number of mapped points. It is possible that lower average proportion of failed slope area for Dataset B is due to the relative under-representation of shallow discontinuities compared to Dataset A. This could be due to the data biases noted in Section 3.3.
- The analysis methodology, which requires blocks to be bound by intersecting discontinuities and the pit face. A small intact rock bridge can prevent blocks from being formed and assessed by the analysis software. There are some new software packages that can assess the stability of these rock bridges, however, they are not considered in this paper. The possibility of small intact rock bridges was partially mitigated by increasing the number of models analysed.
- Exclusion of water pressure.

Table 6 Discrete fracture network analysis results (results over 5% highlighted red)

Slope angle (°)	Average area of pit face failed (%), for a given discontinuity radius and dataset							
	25 m disc. radius		50 m disc. radius		100 m disc. radius		200 m disc. radius	
	Dataset A	Dataset B	Dataset A	Dataset B	Dataset A	Dataset B	Dataset A	Dataset B
30	0.4	0.2	0.7	0.2	1.0	0.3	1.4	0.5
41	0.3	0.2	0.6	0.2	1.5	0.4	3.6	0.5
45	0.3	0.2	0.7	0.3	2.1	0.4	5.8	0.7
49	0.3	0.2	0.7	0.3	3.1	0.5	8.6	1.3
60	0.2	0.1	0.9	0.4	7.5	3.0	21.1	8.6
70	0.3	0.2	1.9	1.4	13.1	8.8	39.0	26.9
80	0.4	0.4	2.8	3.1	20.7	19.5	57.6	52.0

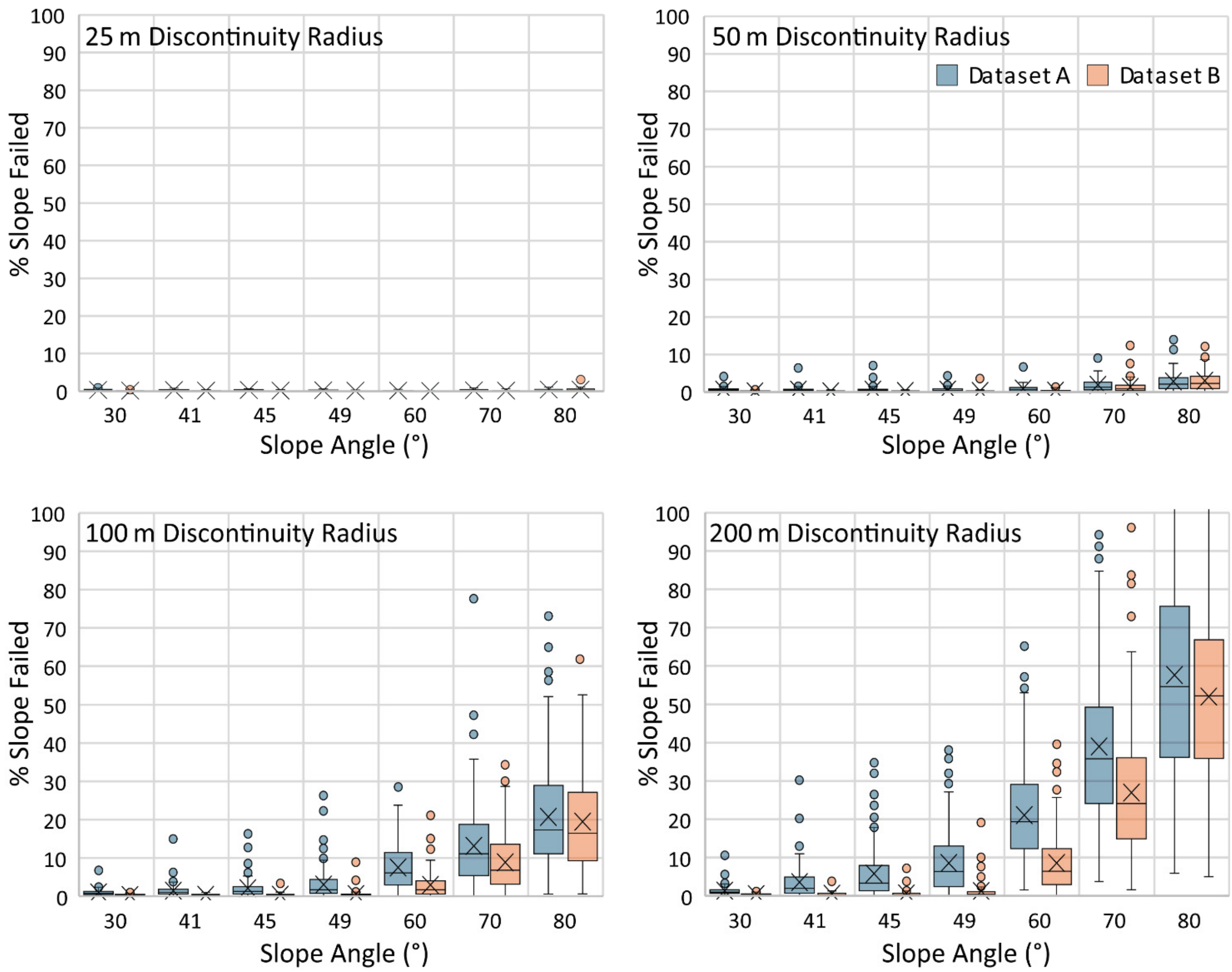


Figure 9 Results from the rock wedge analysis showing the proportion of slope failed depending on the fracture size, dataset, and slope angle analysed

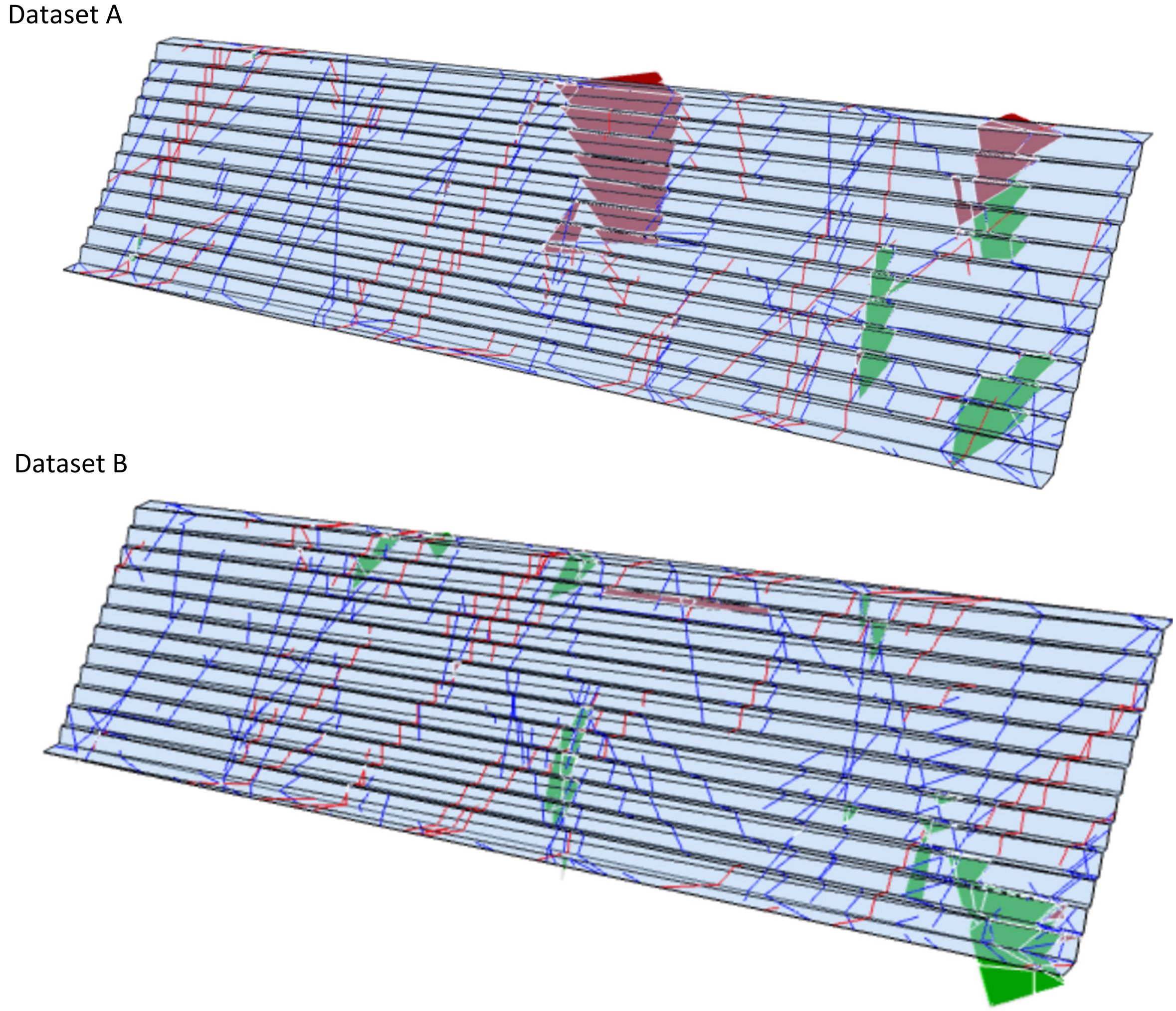


Figure 10 Examples of blocks identified by the analysis of one discrete fracture network model, discontinuity size, and slope angle. Stable blocks are green and unstable blocks are red. Traces for discontinuities intersecting the pit face are shown in red (Set 1) and blue (Set 2)

6.5 Comparison to observed slope performance

The DFN results show that the average area of slope failed ranged from 0.1 to 58%. The DFN results are compared to the statistical assessment and the observed slope performance in Figure 11. The comparison shows that:

- At similar slope angles, the DFN models indicate that less slope area is likely to fail compared to the statistical assessments, i.e. the results of the statistical assessments are more conservative than the DFN models. This is expected as the DFN models consider discontinuity lengths and distribution and varied discontinuity length (compared to the statistical assessment which does not).
- Dataset A (photogrammetry mapping) indicates similar proportions of slope instability compared to the observed slope performance, for discontinuity radii between 100 and 200 m. This is interpreted by the authors as a reasonable discontinuity size for the case study site.
- Dataset B (pit face mapping) indicates less instability than the observed slope performance at typical inter-ramp scale probabilities of failure (approximately 5%), possibly due to the relative under-representation of shallow discontinuities compared to Dataset A.

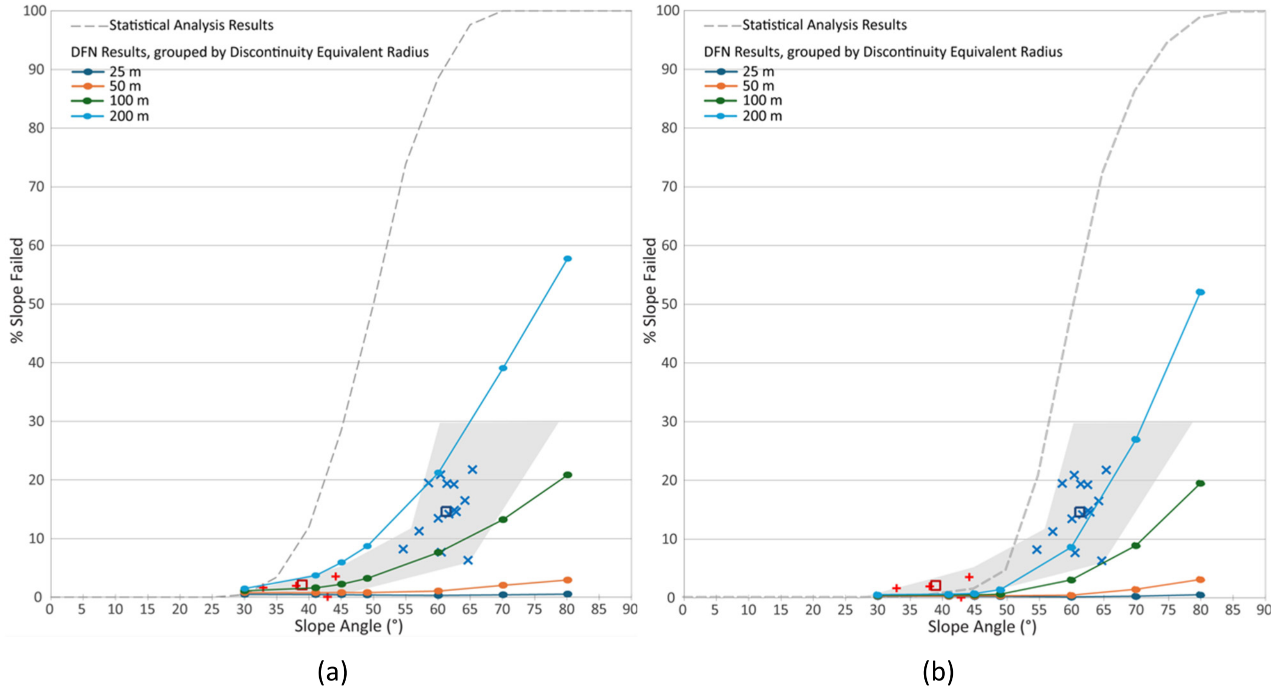


Figure 11 Comparison of statistical analysis results, excavated slope performance, and discrete fracture network modelling average failed slope area. (a) Dataset A; (b) Dataset B. Failed proportions of the excavated pit slope are shown for bench (blue) and inter-ramp (red) scales. Open square symbols show area weighted average values. Grey shading shows the range of excavated slope performance measurements

7 Conclusion

This paper presents a case study comparing industry-accepted methods for pit slope design in structurally controlled rock masses with the results from a DFN approach. The observed performance of an excavated slope was used as a reference point to evaluate these different methods.

This paper also outlined the development of DFN models, including the selection of input parameters, model construction, validation, and the generation of multiple stochastic discontinuity models. Stability analyses were then undertaken on these models, using varied discontinuity lengths and representative pit slopes with a range of slope angles, to assess how these variables impacted the amounts of predicted slope instability.

The results showed that the industry-accepted methods for pit slope design are typically conservative because of the underlying assumptions used. Consequently, they indicated higher probabilities of failure compared to what was observed in the existing slope. The DFN models produced a range of results for the average failed slope area, and a selection of the DFN cases analysed with reasonable discontinuity sizes were a good match to the observed slope performance.

The results indicate that the analysis method is likely to impact the adopted slope design geometry, and that using a DFN-based approach can result in a slope design with increased confidence, and resulting as-built performance closer to what was indicated by the analysis, compared to industry-accepted methods. This may be a less conservative (i.e. steeper) design.

The study also showed that the DFN analysis results are significantly influenced by discontinuity size, which is typically difficult to measure. Given this impact on results, care should be taken during pit mapping to accurately record discontinuity lengths. Data biases in the input datasets can also impact the analysis results, and care should be taken during data collection.

By improving our understanding of slope stability in structurally controlled settings, we can enable the progression of pit slope design methods that more reliably correlate with actual slope performance.

8 Acknowledgement

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