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Evaluation of inter-ramp scale non-daylighting wedges using a discrete fracture network-based method

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Abstract

Open pit slopes that consist of medium to strong, moderately to highly fractured rock mass present challenges when inter-ramp scale analysis and design are considered. Conventional kinematic analyses fail to capture the structural complexity of the slope over stacks of benches. Limit equilibrium approaches can incorporate a simple rock mass fabric, but they are typically not able to represent multiple structural sets or complex rock block geometries. Numerical stress models provide a more rigorous approach but are time consuming to set up, typically have long simulation times and are impractical for evaluating the impact of inherent structural uncertainty on slope stability. It is this last aspect that is often critically important, yet difficult to evaluate.

Discrete fracture network (DFN) methods provide an effective solution for handling structural complexity and uncertainty, with a stochastic modelling approach that allows complex rock block geometries to be generated and the kinematics evaluated. However, most of the multi-bench-scale blocks formed comprise non-daylighting wedges (NDWs) that are stable because they occur behind or beside a buttress of rock mass that stabilises the block, rendering the simple kinematic solution ineffective. To overcome this challenge, a custom approach was developed for evaluating the stability of these NDWs. The approach can efficiently analyse a wide range of discontinuity strengths, rock mass strengths and pore pressure conditions, allowing critical controls on slope stability to be identified and opportunities for slope design optimisation to be evaluated based upon Probability of Failure (PoF) criteria.

This paper provides an update on the DFN-3DPOF method, based upon several years of application. It describes the DFN approach and the block stability solution, when to use this method and when not to, the handling of NDW geometries and how the approach has contributed to advancing inter-ramp analysis and design.

Keywords: *inter-ramp slope stability, discrete fracture network, non-daylighting wedges, 3D probabilistic analysis*

1 Introduction

Open pit slopes that consist of medium to strong, moderately to highly fractured rock mass present challenges when inter-ramp scale analysis and design are considered. Unlike soil or weak rock mass slopes, analysis of fractured rock masses must consider composite failure mechanisms that involve both shearing through rock mass and along discontinuities. Despite best efforts, there is considerable uncertainty with regards to many of the key inputs, including rock mass structure and fabric, rock mass and discontinuity strengths, and pore pressures (Read & Stacey 2009). Over the years, a number of different approaches have been developed to try to best capture key aspects of this heterogeneous, anisotropic rock system in order to provide a basis for reliable and effective slope designs. The main methods are summarised in Figure 1.

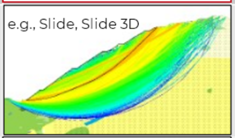
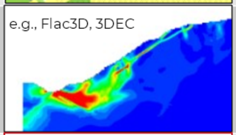
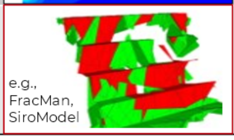
	Method		Advantages	Disadvantages
1	**Kinematic Methods**		• Simple and fast • Widely used • Low cost	• Largely not representative of inter-ramp failure mechanisms • Does notconsider rock mass component • Oversimplify wedge geometry at inte-ramp scale
2	**Limit Equilibrium Methods (methods of slices)**		• Simple and fast • Widely used • Mechanisms involving both rock mass and structure	• Not adequate for assessing variations in persistence, spacing, and orientation of multiple structural sets
3	**Numerical Methods**		• Sophisticated method • Mechanisms involving both rock mass and structure • Strain/deformation analysis	• Additional data requirements • High computational demands • Relatively simple structural model • Not practical for probabilistic analysis
4	**DFN Based Methods**		• Fracture network realistically defined based on actual data • More adept at conducting probabilistic stability analysis	• Cannot directly simulate the rock mass shear component of failure

Figure 1 Summary of the main methods of analysis associated with inter-ramp slope design

The most straightforward methods are simple kinematic approaches, which are best suited for bench-scale analysis but can also be applied to multi-bench stacks (e.g. RocScience 2023a, 2023b). There are a variety of different approaches that can be used, but a key element of most is the need to increase the strength of structures to represent a rock bridge component that would be present when considering an inter-ramp stack that could be several hundred metres high. These methods are very efficient but have the limitation of simplifying potential wedge geometry at the inter-ramp scale and not reasonably capturing the likely failure mechanisms.

Limit equilibrium (LE) solutions are well-known tools for inter-ramp and overall slope-scale analysis, largely in 2D but increasingly in 3D despite some limitations (e.g. Slide2D, Slide3D; RocScience 2023c, 2023d). LE solutions provide fast and streamlined solutions, directly accounting for both rock mass and structural components. However, their ability to incorporate a heterogeneous structural pattern comprising a range of structure types, orientations and lengths is limited.

A higher level of sophisticated analysis is provided by various numerical modelling methods (e.g. 3DEC, Flac3D). These approaches provide rigorous strain and deformation analysis, can be fully coupled, incorporate both rock mass and structural components and, for back-analysis, provide high-quality calibration. However, without a high degree of automation, building the models can be time consuming, additional data requirements are needed, and the main structural model is often simplified and cannot be replaced easily with an alternative model. The computational costs of numerical analysis can be very high, resulting in long run times and a limited ability to perform probabilistic analysis.

The last of the main approaches is based around discrete fracture network (DFN) methods. The DFN method seeks to build realistic networks of structures using a stochastic modelling approach, based around observed distributions of fracture orientation, size and intensity (e.g. Rogers et al. 2006; Elmouttie et al. 2010; Grenon & Hadjigeorgiou 2003; Grenon et al. 2015; Merrien-Soukatchoff et al. 2012). The stochastic nature of these models makes them highly applicable to probabilistic stability analysis, where wedge kinematics can be evaluated on complex composite polyhedral blocks developed from both deterministic and stochastic structures. However, the kinematic solutions for DFN models are limited to a simple sliding model, without taking into account rock mass components.

From this summary of analysis methods, it is clear that while there are many different approaches for inter-ramp stability analysis and design, they all have a range of shortcomings that limit their applicability. To address these shortcomings, a hybrid approach has been developed (Valerio et al. 2020), with the goal of achieving the following key demands: handle structural complexity and uncertainty; consider both rock mass and structural components; have relative short run times; be probabilistic in nature, allowing

Probability of Failure (PoF) to be defined; and allow a range of key properties to be analysed without imposing significant computational overhead.

This paper describes an approach to deliver on these targets, based around a DFN description of slope structures, the identification of complex rock blocks forming in the slope, and the evaluation of their stability considering both kinematic sliding and rock mass strengths. The approach is referred to as DFN-3DPOF. As will be shown, most of these blocks are various forms of non-daylighting wedges (NDWs), namely fully formed blocks defined by structures but held in place by the presence of a buttress of rock mass that prevents the block from sliding. This paper summarises the DFN approach along with block identification, describes how the stability of those blocks is determined, and discusses how these results are then used to evaluate slope stability.

2 The DFN-3DPOF workflow

2.1 Introduction

The DFN-3DPOF workflow comprises the following key steps:

- DFN model development.
- Block identification.
- Block stability assessment.
- Post-processing of Factor of Safety (FoS) results.

This section of the paper describes the key elements of this workflow, how the elements connect, and how the results are processed to evaluate slope stability.

2.2 DFN model development

The DFN approach is a modelling method that seeks to explicitly describe the rock mass fracture system through a combination of deterministic wireframed structures and statistical analysis, by building a series of discrete fracture objects based upon field observations of fracture properties such as size, orientation and intensity (Rogers et al. 2009).

DFN methods have several key advantages over continuum-based methods in that they are better at describing local-scale structural problems and mechanisms because of their ability to capture the discrete fracture properties more accurately than larger-scale continuum approaches. Their ability to represent the known structural system and to extrapolate to the inferred structural system provides a useful method for describing the slope structures, along with their inherent uncertainty.

When using DFN methods to describe bench-scale problems, the focus is on the joint-scale fabric with length scales of approximately tens of metres. However, at the inter-ramp and overall slope scales, there is greater concern with larger structures at the scale of a bench and upwards (i.e. tens to hundreds of metres). Therefore, the characterisation process needs to be aimed at developing the stochastic inputs to define the key geometric parameters of the fault system of structural size distribution, orientation and intensity (Valerio et al. 2020). The key data sources used to constrain inter-ramp and overall slope-scale models are summarised in Figure 2.

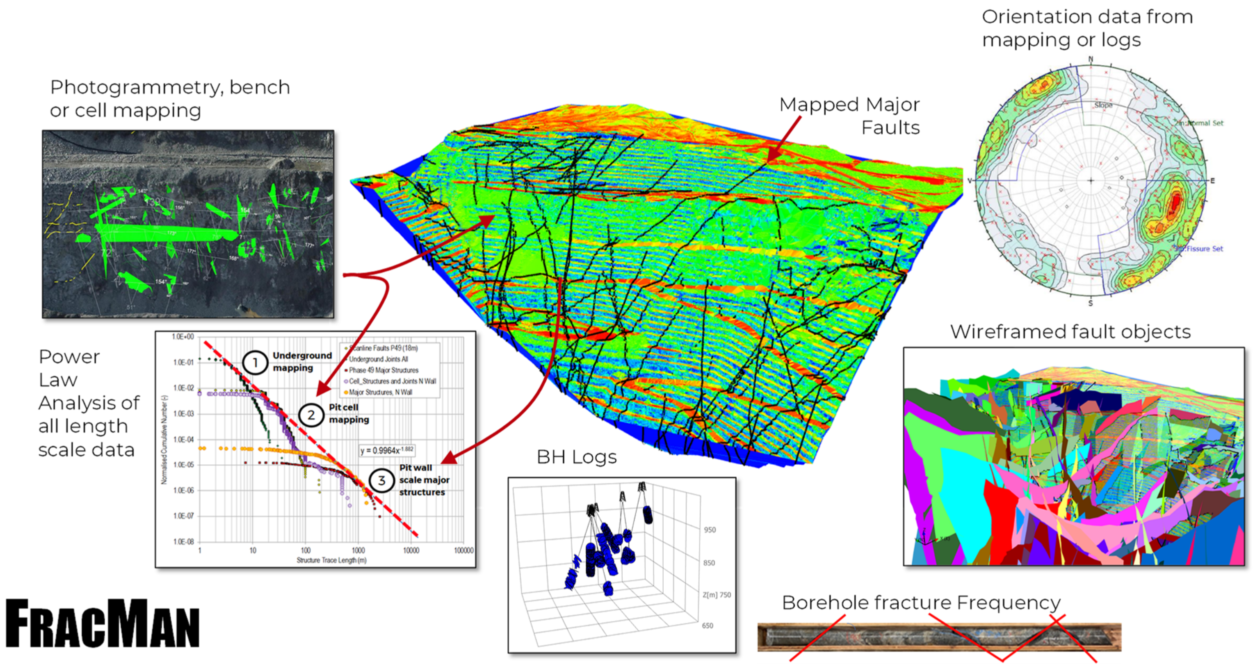


Figure 2 The key data sources that define the geometry of a DFN model. Wireframed or mapped structures, when integrated with the photogrammetry, help define the length-scale properties of structures. Orientation properties are developed from mapped data as well as borehole logs and televiewers, which also provide information on fracture intensity

When building DFN slope models, two different scales of analysis are useful – namely, the smaller sector-scale model and the full wall-scale model. The sector-scale model is akin to a multi-bench-scale kinematic assessment, where the stochastic properties of the different fault families are built and analysed in an ex situ model. This means it is representative of a design sector but does not incorporate explicit wireframed structures because it is the generality of the structural fabric and slope design that is being tested, rather than the possibility of block formation around specific major structures. The sector-scale models need to be sufficiently large enough to allow multi-bench-scale blocks to form. Valerio et al. (2020) report design sector models with an overall height of approximately 210 m (14 × 15 m benches) and a width of approximately 200 m. The purpose of the sector-scale models is to allow DFN fabric to be built and tested rapidly for a range of different inter-ramp angles (IRAs) and slope directions that can inform slope design recommendations.

In contrast to the ex situ sector-scale models, in situ wall-scale models can also be built. These can vary in size, ranging from a small stack of benches to a full wall in height. The principal difference, however, is the desire to not only investigate the behaviour of a slope against the representative structural fabric, but also to directly include major structures (e.g. faults and dykes), many of which may also be boundaries between design sectors. For these models, each design sector has specific DFN inputs representing the local representative fabric. This can comprise a simple average fabric for a domain or can be more geologically constrained, such as with properties being controlled by distance to major faults (e.g. Rogers et al. 2016) or following deformed surfaces such as bedding (e.g. Valerio et al. 2020). Some DFN codes provide a powerful geomodelling environment, allowing these geological controls to be efficiently captured within the workflow.

2.3 Block searching within the DFN model

Having built the DFN models, either sector-scale or wall-scale, the next step is to identify all blocks that form within the slope. For each realisation of the DFN model and slope configuration, FracMan® (WSP 2023) identifies all blocks (multi-faceted wedges) that are enclosed by discontinuities and/or the pit surface. This operation is typically the most time-consuming operation within the DFN-3DPOF workflow. The most efficient way to reduce computational time is to filter out the smallest fractures and not include them in the block search. This is a common discrete fracture modelling technique that can be tested to ensure no significant impact on the identified block geometry (Rogers et al. 2018). Once the blocks have been identified, their kinematic stability is evaluated. An example of a sector-scale model is shown in Figure 3.

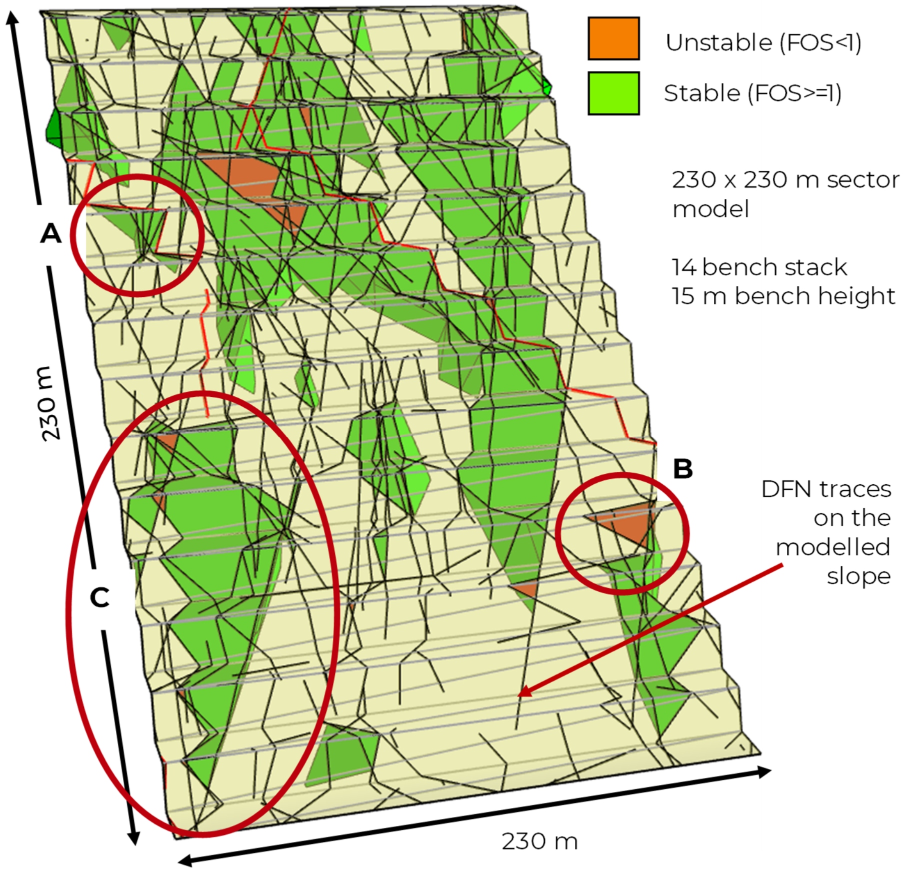


Figure 3 Identified rock blocks from a sector-scale DFN model, 230 × 230 m in size. Blocks coloured green have a FoS ≥1 and red blocks have a FoS <1. The analysis shows three distinct types of blocks: (a) 1–2 benches in size and stable, (b) 1–2 benches in size and unstable (conventional bench-scale instabilities) and (c) multi-bench-scale composite wedges that are stable

As seen in Figure 3, several different types of identified blocks have been highlighted, namely:

- Simple wedges or blocks, with a size between 1 and 2 benches in height and kinematically stable.
- Simple wedges or blocks, with a size between 1 and 2 benches in height and found to be kinematically unstable; these would be considered conventional bench-scale instabilities.
- Complex multi-bench-scale composite wedges that are typically stable.
- Whilst not actually shown here, it is possible to have large multi-bench-scale composite wedges that are unstable, but the geometry of these composite wedges means they are typically locked in.

FracMan and other DFN codes can readily evaluate the stability of the identified blocks in the first two cases, using conventional LE or force balance approaches (Carvahlo 2002). However, the more-complex blocks, often with variable geometries that do not daylight at the pit surface, are identified to be kinematically locked in and therefore excluded from the analysis. These NDWs are typically held stable though a buttress of rock mass either at the toe or side of the wedge or by an irregular geometry along its basal sliding plane. Therefore, to evaluate the stability of these wedges, both the resistance to sliding along structures as well as the resistance provided by the rock mass components must be considered. This is achieved by applying the 3DPOF tool.

2.4 Evaluation of non-daylighting wedge stability

To understand the rock mass component of the stability calculation better, consider a NDW in more detail, in a similarly sized sector-scale model (Figure 4).

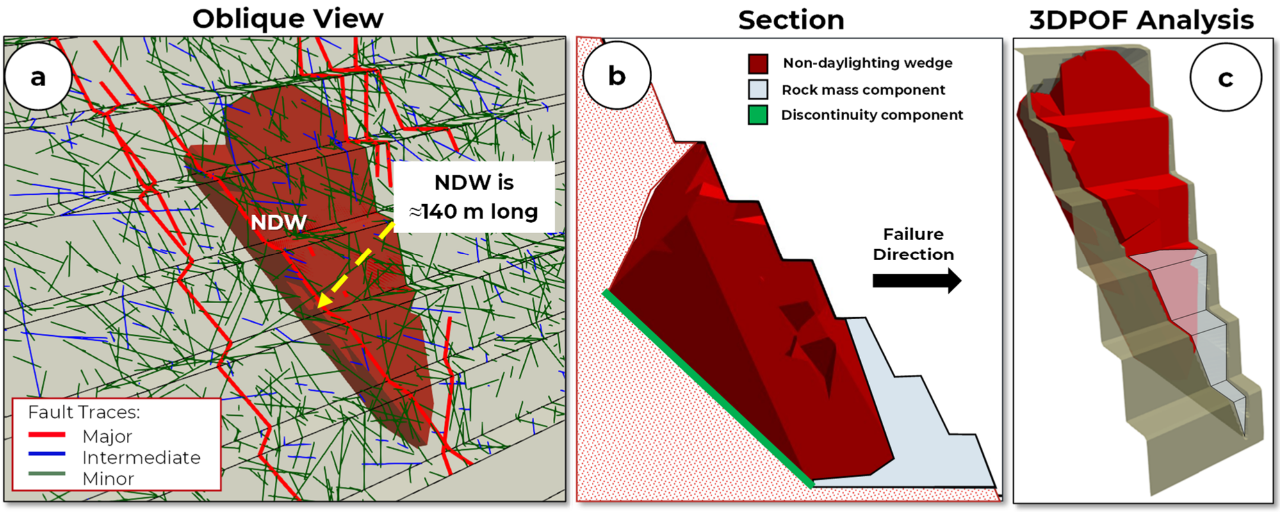


Figure 4 (a) An example of a large NDW 140 m long, formed on a double-bench stack, bench height 30 m, showing how it is formed from a combination of the red deterministic structures and smaller stochastic fabric; (b) In section, the NDW is seen to be stable behind a buttress of rock mass (blue) that needs to fail for the wedge to move; (c) The NDW in 3DPOF, showing the blue rock mass buttress

Figure 4 shows a NDW, approximately 140 m in length, formed from a combination of deterministic major structures and a fabric of intermediate-scale structures. When the NDW is viewed in section, the buttress of rock mass at the toe can be seen clearly. Evaluating the stability of the NDW is different from evaluating purely kinematic wedges. First, FracMan identifies the complex and irregular polyhedral shapes, and each wedge is analysed using the 3DPOF code. Within 3DPOF, an algorithm is used to identify a fully daylighted slip surface honouring, within an acceptable tolerance, the complex nature of the slip surface formed by the intersecting discontinuities. The algorithm is based upon developing an optimised lower convex hull for the complex geometries, extended (daylighted) to form a continuous basal sliding surface. Regions of failure through rock mass or along discontinuities are differentiated along the failure surface to permit a FoS assessment based upon their independent strength properties (i.e. no composite rock mass bridge strengths are necessary). The slip surface method is detailed in Lawrence et al. (2020).

The FoS along the slip surface, defined as the resisting force to driving force ratio, is calculated as:

$$\mathrm{FoS} = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{\sum\,(c_i + \sigma_{n_i} \tan\varphi) A_i}{\sum \tau_{\,i} A_i} \tag{1}$$

where c and φ are the cohesion and friction angle describing the shear strength of the rock mass or discontinuity, σ_{n} and τ are the normal and shear stresses acting at a point on the slip surface, and A is the area of the regions of the slip surface.

An important aspect of the evaluation of these complex blocks is that they can have multiple different failure mechanisms, each with its own FoS. The complex geometries of blocks mean that they can contain internal structures that provide alternative failure planes as well as the identified basal plane. The 3DPOF code searches through each block to identify different potential failure modes, as illustrated in Figure 5.

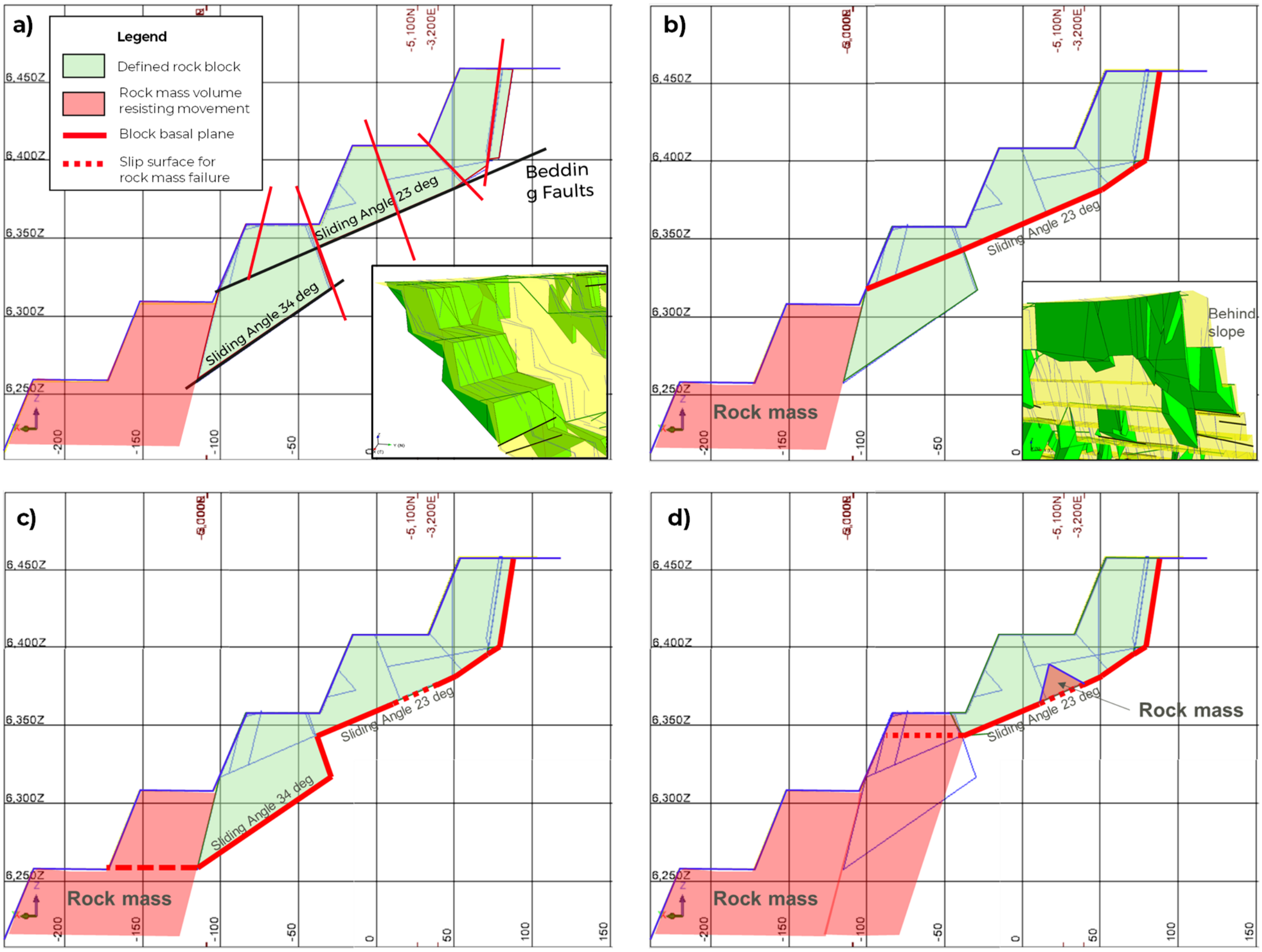


Figure 5 **Example of a composite block and how the 3DPOF code searches to estimate its stability; section through a block shown in inset from the front in (a) and from the rear in (b); note how the composite block is defined by a number of structures, many of which are internal to the overall block; (b), (c), (d) show different failure paths, including different combinations of basal sliding failure and rock mass failure, including the presence of a rock mass notch in (c)**

The example shown in Figure 5 represents a three-bench-high complex block, showing a variety of different failure modes, as defined by the presence of potential basal sliding surfaces, release surfaces, and rock mass components. The example in Figure 5b shows how failure is assessed as a purely kinematic block, sliding on a bedding fault, with a back release plane. In Figure 5c, the red slip surface shows failure of the block through a combination of kinematic sliding and through a rock mass buttress at the toe, representing the entire block volume. Lastly, in Figure 5d, the red slip surface shows failure of the block through a combination of a shorter kinematic slip surface and failure through a rock mass buttress at the toe, of similar size to the buttress shown in Figure 5c. 3DPOF repeats this analysis for all blocks identified from all realisations (typically n = 100), resulting in the generation of a large array of block stability FoS calculations of the order of 10^5–10^6 in number.

In defining properties for the FoS calculations, Mohr–Coulomb strengths are used for discontinuities, and equivalent (instantaneous) Mohr–Coulomb strengths are used for Hoek–Brown characterised rock mass. Thus, for each domain, the properties shown in Table 1 need to be defined.

Table 1 Key properties needed for the 3DPOF analysis

Component	Strength defined by	Parameters
Structures	Mohr–Coulomb failure criterion	Friction angle, ϕ Cohesion, c
Rock mass	Hoek–Brown failure criterion	GSI σ_{ci} m_i D
Pore pressure	–	Ru

Note: GSI = geological strength index; σ_{ci} = intact unconfined compressive strength; m_i = Hoek–Brown material constant for intact rock; D = disturbance factor; Ru = pore pressure ratio.

Calculation of each block's FoS is very fast, in the order of milliseconds. As outlined in Section 1, a key objective is to be able to evaluate a range of different properties. Thus, when 3DPOF analysis is run, a number of different strength cases are evaluated at the same time. These may include scenarios such as base case and lower bound rock mass strengths, base case and lower bound joint strengths, and cohesionless joint strengths. Additionally, all cases are evaluated for a range of pore pressures from dry to fully saturated. 3DPOF uses the Ru definition of pore pressure (GeoSlope 2023), with each strength scenario being evaluated with an Ru, ranging from 0 (dry) to 0.4 (approximately fully saturated), in steps of 0.05. This ability to evaluate a wide range of conditions for every block, almost instantaneously, allows the impact of critical properties to be identified efficiently. Consider the case of the single NDW seen in Figure 4. Under dry conditions, it has a FoS of two. However, as pore pressure increases, there is a reduction in the strength of the rock mass buttress at the toe of the wedge, as well as a reduction in the effective normal stress on the basal plane, resulting in a progressive reduction in FoS (Figure 6).

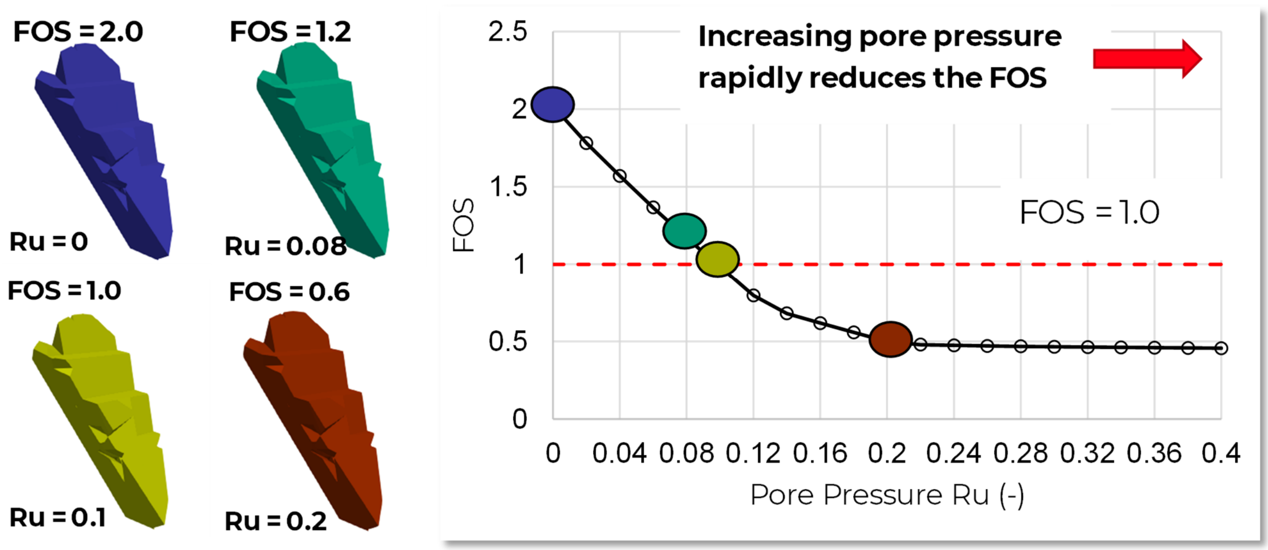


Figure 6 Example of the change in stability (FoS) of a single NDW (from Figure 4) with increasing pore pressure (Ru)

While Figure 6 shows the stability of a single NDW, it is the consolidated results of all wedges that are of primary interest. This can be approached in a number of ways:

- To evaluate overall slope performance, the FoS calculations from all blocks and from all realisations are aggregated to develop a simulated PoF. For this DFN-based analysis, the PoF is defined as the ratio of the sum of the mass of all blocks with a FoS < 1.0 to the sum of the mass of all blocks. For the sector-scale models, the results from each different slope IRA and slope dip direction can be compiled on a chart.
- Alternatively, the consolidated results, whether PoF or failure volume, can be plotted against pore pressure (Ru), allowing a variety of different scenarios to be displayed together. Figure 7 shows an example of the simulated failure volume for four different strength cases (base case, low discontinuity strength, low rock mass strength, and low discontinuity strength and rock mass strength) being plotted against Ru. Across the chart in a dashed red line is the observed failure volume, providing strong evidence that it is low rock mass strength and moderate pore pressure that control stability in this sector.

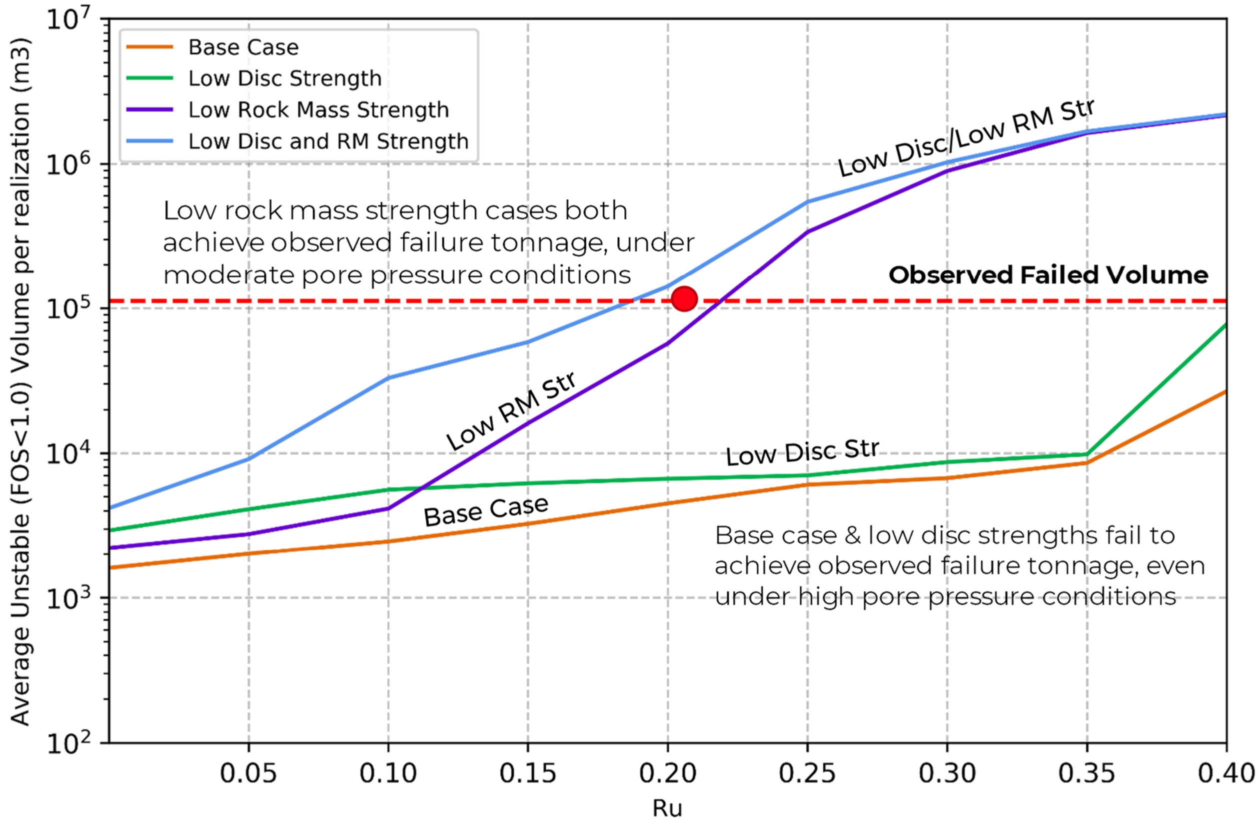


Figure 7 Graph showing average unstable volume as a function of pore pressure (Ru) for four different strength scenarios. The red dashed line shows the estimated observed failure volume, clearly indicating that only the two low rock mass strength cases result in simulated failure volume equivalent to the observed volume

To help identify areas of the slope that result in higher probability of block formation, the results of multiple realisations are accumulated together by mapping all of the formed blocks, regardless of FoS, in a 3D grid and then colouring that grid by the number of blocks present at that location (Rogers et al. 2016). This grid is then displayed as a heat map, exhibited in Figure 8, with areas that show more common block formation seen as red (hot) colours and areas where there is no specific driver seen as blue (cold) colours. Finally, the stability of these complex blocks over the extent of the pit is evaluated using the same processes outlined in the sector-scale analysis. The generation of DFN models and identification of the complex blocks is more computationally intensive using the larger pit-scale model, but the LE component of the DFN-3DPOF approach is unchanged. Statistics are still collated, though now referencing the true region of the pit instead. Heat maps, similar to the example presented in Figure 8, are developed to differentiate regions of the pit where there is a higher probability of inter-ramp stability. These heat maps and unstable mass estimates are then used to inform risk and financial mitigation efforts.

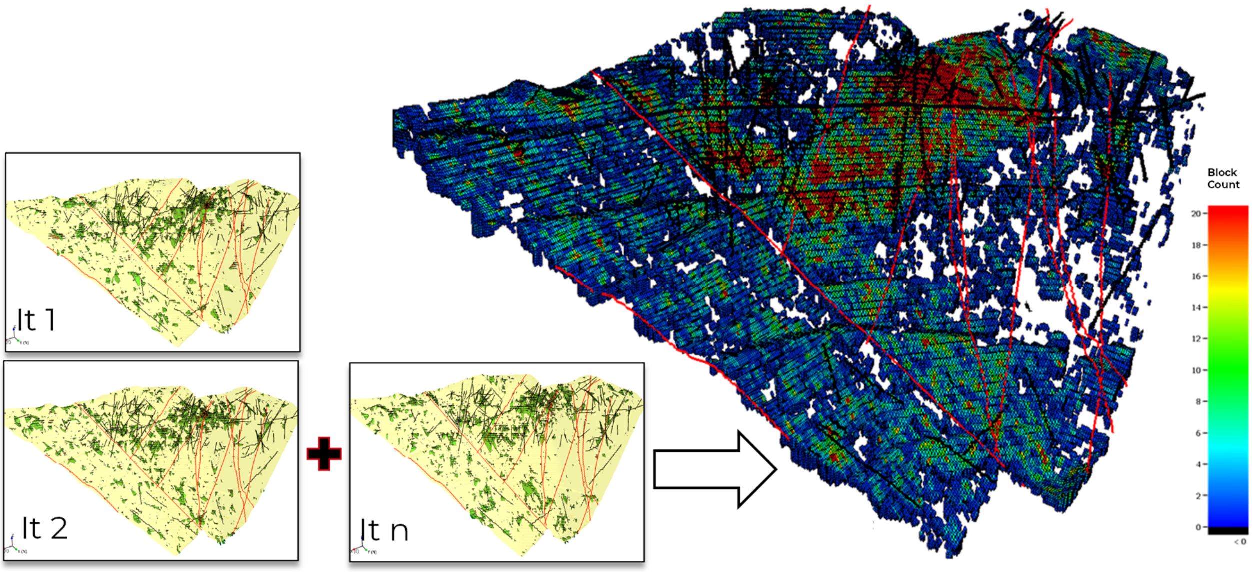


Figure 8 Heat map of block formation, with high probability of block formation shown in red and low probability shown in blue

3 Discussion

At the heart of the method described herein is a consideration about the propensity of a rock mass to form large multi-bench-scale NDWs and the role of these large, contained rock mass volumes in slope instability. Based on modelling carried out to date, it is believed that most of the larger NDWs are formed from the interaction between known major structures as well as the stochastic fabric of intermediate-scale structures. A comparison between block formation with only deterministic structures and block formation with both deterministic and stochastic structures shows a significant increase in NDW formation. This is also supported by reviews of larger slope failures that can partially describe the failure in terms of a deterministic fault but are seldom able to fully describe the total failure without considering smaller scale structures.

An advantage of the DFN approach is that, in addition to determination of the slope PoF, the probability of occurrence can also be calculated. This provides an insight into the likelihood of the formation of the NDWs, allowing data acquisition to be targeted towards parts of a slope where the geometric combination of slope orientation and structural fabric have an elevated risk of NDW formation.

Several DFN-related input properties are seen to strongly influence the formation of slope-scale blocks. These are summarised in Table 2 and typically represent properties that increase the overall connectivity of the network of structures.

As a result of the DFN-3DPOF analysis not requiring structures to be meshed like in more-complex numerical simulations, both the deterministic structural model and the stochastic structural fabric can be readily replaced and re-analysed with minimal effort. This means that issues associated with structural uncertainty can be readily tackled using this approach.

A key question that is often asked about the DFN-3DPOF workflow is, 'Where should it be applied?' As a starting point, it is aimed at medium to strong, moderately to highly structured rock masses. As described above, the method is particularly applicable to a slope comprising a rock mass structural fabric that results in the formation of multi-bench-scale blocks. If stability was controlled by a limited number of isolated structures, other methods such as conventional LE solutions would be applicable. Additionally, the rock mass component needs to be sufficiently strong that composite failure mechanism comprising both structure and rock mass are possible. If the rock mass was too weak, a predominantly rock mass failure would occur, with limited structural failure, and the DFN solution would not be needed. Different slope failure mechanisms are summarised in Figure 9, highlighting which ones are more applicable to the DFN-3DPOF approach.

Table 2 Key DFN properties that influence slope block formation

Parameter	Impact on block formation
Fault intensity (P21)	The higher the intensity of mapped faults included in the model (defined by P21† intensity), the more ubiquitous the rock blocks become
Stochastic intermediate structure intensity (P32)	The higher the intensity of intermediate-scale structures included in the model (defined by P32‡ volumetric intensity), the more ubiquitous the rock blocks become
Structure orientation variation	The more distinct sets with a higher angle of intersection, the more blocks that will form. Higher dispersion within one set will increase block formation, but not as much as with multiple sets
Structural size	Longer structures typically form more blocks than do shorter structures. When using a power law description of length, the lower the power law exponent, the more blocks. This is because it results in a higher relative ratio of longer structures to shorter structures. As the exponent increases, there will be relatively more shorter structures than longer structures

Note: † = Fault length per unit area (Dershowitz & Herda 1992); ‡ = Fault area per unit volume (Dershowitz & Herda 1992).

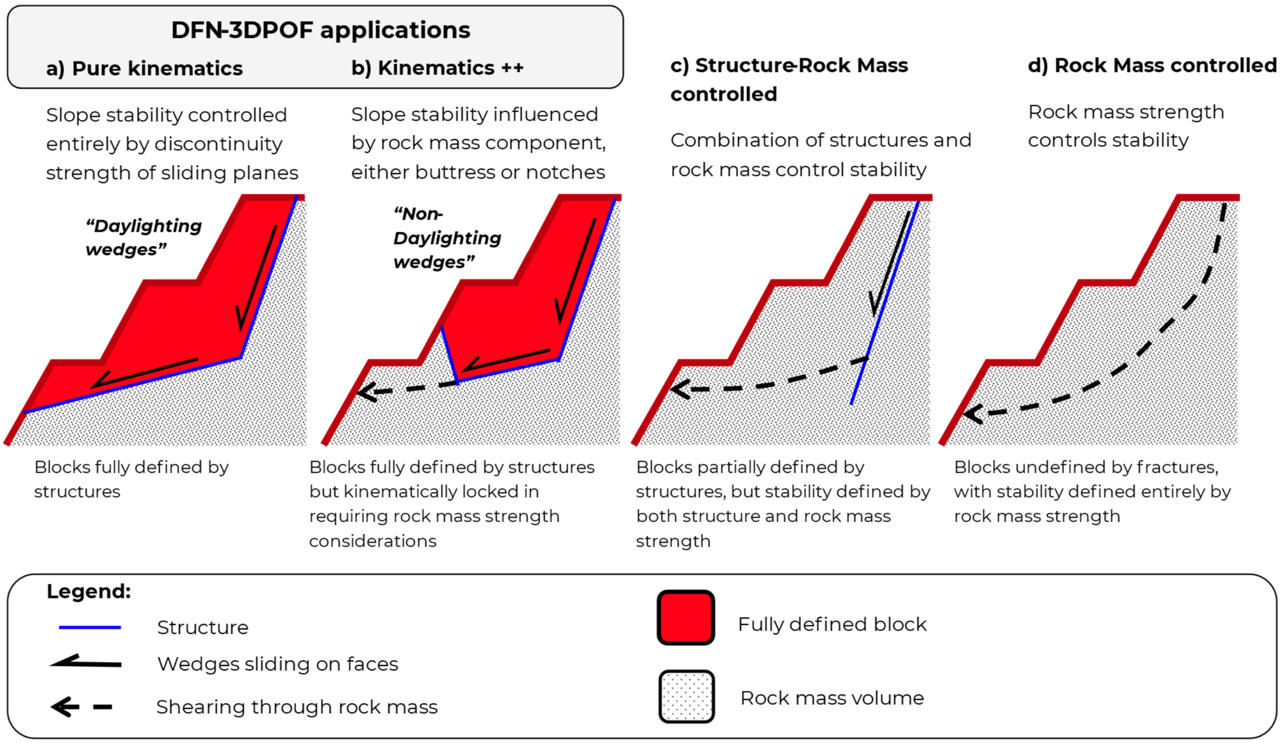


Figure 9 Styles of slope instability, indicating those that are most applicable to the DFN-3DPOF approach

A significant strength of the overall approach is the ease with which a wide range of property sensitivities can be rapidly evaluated, providing a useful screening of potential controls on slope stability. In particular, the DFN-3DPOF method provides a practical and straightforward tool to explore the influence of pore pressure on NDW stability. This has typically been found to be the most significant control on stability and reinforces the need on many slopes for active water management, including drainage and depressurisation.

To date, applications of the DFN-3DPOF approach have been varied, covering a wide range of issues from a short stack of benches to a full slope. Key applications are summarised in the following list:

- Front-end evaluation of potential pit-scale instabilities: the use of an early structural model (wireframes), in conjunction with stochastic structures developed from a combination of the structural model and borehole data, allowed the identification and stability evaluation of potential inter-ramp scale wedges and NDWs in an early life open pit.
- Back-analysis of multi-bench inter-ramp failures: the identification of sector-scale blocks, testing of potential failure mechanisms and calibration of modelled instability volume and PoF evaluation.
- Inter-ramp design: The testing of DFN fabric against a range of slope IRAs to develop PoF versus IRA relationships and select optimal slope angles based on appropriate design acceptance criteria.
- Wall-scale analysis and slope sector hazard ranking: the building of multi-sector DFN models and the evaluation of sector PoF, including property sensitivities, to identify critical slope controls for each sector.
- Evaluation of ground support options: the evaluation of ground support options for a high value slope through the application of a support face pressure in 3DPOF.

The applications to date with 3DPOF have shown that this method, combining DFN analysis and an LE solution for block stability, provides a flexible approach to the assessment of structured rock masses. As more experience is gained, it is believed that more applications and a deeper understanding of inter-ramp slope failure mechanisms might be learned by examining how the DFN models capture slope structure and form blocks and what the critical controls on block stability are.

4 Conclusion

The DFN-3DPOF approach has yielded a unique inter-ramp stability assessment approach that has allowed a far more structurally robust method to be brought to the analysis. The DFN modelling provides a greater degree of structural reality to capture structure at the inter-ramp scale, including uncertainty in key input parameters. The DFN kinematic engine allows the direct assessment of block formation within the slope, and these blocks represent a precursor to potential instability. The DFN-3DPOF method rapidly performs probabilistic analysis of composite failure mechanisms comprising both structural and rock mass components for all identified slope blocks, including property sweeping for sensitivities. The combination of these steps represents a step change in the assessment of medium to strong, moderately to highly fractured rock masses.

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