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The Importance of Intact Rock Bridges in the Stability of High Rock Slopes — Towards a Quantitative Investigation Using an Integrated Numerical Modelling; Discrete Fracture Network Approach

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Abstract

As large open pit rock slopes reach increasingly greater depths and more frequently involve interaction with underground mines the need to consider intact rock fracture becomes ever more important. This paper emphasizes the importance of brittle rock failure propagation through intact rock bridges in high rock slopes, with reference to both large open pits and their natural analogues, high mountain slopes. Quantification of discontinuity persistence and intact rock bridges within rock slopes is a critical component of rock slope research and requires not only the use of new data collection techniques but also data interpretation through rigorous fracture network analysis. Using probability density functions to represent discontinuity orientation, spacing and persistence, the Discrete Fracture Network (DFN) approach is shown to represent an ideal numerical tool with which to synthesise realistic fracture network models from digitally and conventionally mapped data. Integration of a DFN model with a hybrid geomechanics code that is able to model fracture propagation allows the simulation of the interaction between naturally occurring discontinuities and brittle fracture through intact rock bridges. Having discussed the applications and limitations of a 2-D discrete numerical approach, simulations are presented based on DFN models of a large conceptual rock slope and incorporating varied failure mechanisms. Simulations demonstrate the importance of considering both realistic fracture mechanisms and the ability to model complex failure paths involving sliding along discontinuities, dilation, and intact rock fracture. The paper concludes with a discussion on the definition of intact rock bridges for a 3-D analysis. Significant future challenges presented by the incorporation of inherently 3-D rock structures in large-scale numerical models are emphasised.

1 Introduction

Fundamental aspects of numerical modelling of high rock slopes in both civil and mining engineering include: definition of an accurate geological model, geotechnical data collection, assessing the role of major geological structures and determination of rock mass properties. Recent advances in the field of data capture and synthesis allow the derivation of more accurate 3-D models of naturally jointed rock masses, thus overcoming some of the limitations inherent in an infinite ubiquitous joint approach. In general, any model conceptualisation should attempt to reflect the true discontinuous, inhomogeneous and anisotropic nature of the rock mass. In relation to the intrinsic discontinuous nature of rock masses, numerical models should also incorporate a characterisation of both pre-existing fractures and fractures induced by changes in the original state of stress. The physical processes, and the modelling techniques chosen, may eventually dictate the extent to which these features can be incorporated into a model. Parametric characterisation and its association with sample size, representative elemental volume and homogenisation/upscaling represent fundamental problems faced in realistic modelling. For this reason, any modelling and subsequent rock engineering design will, by necessity, include some component of subjective judgement.

The simulation of intact rock fractures has gained significant impetus with the development of deeper large open pits and their interaction with underground mines. The concept of intact rock bridge failure is a critical part of the process of step-path analysis. Jennings (1970) was the first to fully document the importance of intact rock bridge fracturing for open pit mining. This led to the development of several probabilistic limit equilibrium methods incorporating step-path analysis into rock slope design. To date, however, comparatively little research has focused on the field description of step-paths and fracturing of intact rock bridges. The importance of accounting for intact rock failure is emphasised by the fact that even when they occupy only a very small percentage of the discontinuity-coplanar area, intact rock bridges may provide internal or self-supporting load carrying capacity equivalent to conventional underground support systems (Diederichs, 2003). Failure of intact rock bridges in the field may be involved in a wide variety of complex failure mechanisms (e.g. planar, wedge and toppling failure). Fracture coalescence patterns for step-path failures are highly influenced by the fracture geometry, which can be characterized in terms of fracture length, overlap, spacing and fracture angle (Figure 1). Quantification of discontinuity persistence (i.e. fracture length) and intact rock bridges within rock slopes is a critical component of rock slope research, requiring both the use of new data collection techniques and rigorous data interpretation through fracture network analysis.

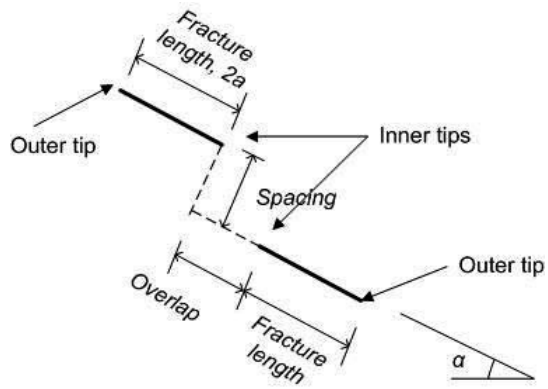


Figure 1 Factors influencing fracture coalescence patterns for step-path failures; positive overlap is defined by overlying fractures (Yan et al., 2007)

Discrete modelling methods are increasingly being used to investigate the importance of intact rock bridges in the stability of high rock slopes. Using probability density functions to represent discontinuity orientation, spacing and persistence, the Discrete Fracture Network (DFN) approach is shown to represent an ideal numerical tool with which to synthesise realistic fracture network models from digitally and conventionally mapped data. By using specific fracture spatial models, the DFN approach allows the definition of geological models which include large (deterministic) structures as well as stochastically generated fracture systems. Integration of a DFN model with a geomechanics code capable of modelling fracture propagation allows the simulation of the interaction between naturally occurring discontinuities and brittle fracture through intact rock bridges. Such an approach provides an opportunity to quantitatively characterise the potential load carrying capacity of rock bridges within high rock slopes. Being inherently 3-D, a DFN model should ideally be coupled with a 3-D geomechanics analysis. However, 3-D modelling of large-scale discrete problems is currently limited by excessive memory requirements and computational times. The level of detail which is necessary to define 3-D intact rock bridges currently limits the analysis to small-scale 3-D problems. As an alternative, it is common practice to consider 2-D sections produced from 3-D DFN models. This paper discusses the limitation of such an approach for any 2-D discrete analysis of a fractured rock body.

A comprehensive discussion of numerical approaches used in the investigation of step-path failure and intact rock bridge fracturing exceeds the scope of this paper. Stead et al. (2006a, 2006b, 2007) and Yan et al. (2007) provided a review on brittle fracture analysis in rock slopes. Elmo in Stead et al. (2006a) undertook integrated geomechanics-DFN modelling of rock slopes using the DFN code FracMan (Golder, 2006; Dershowitz et al., 1998) coupled with the hybrid Finite/Discrete code ELFEN (Rockfield, 2007). This work is a prerequisite of current large open pit-underground mine modelling research being undertaken at Simon Fraser University (Elmo et al., 2007).

2 Principles of discrete fracture network (DFN) modelling

The basis of the DFN approach is the characterisation of each discontinuity set within a structural domain using statistical distributions to describe variables such as orientation, persistence and spatial location of the discontinuities. Because rock mass properties are inherently three-dimensional (3-D), fractures should be characterised in 3-D space. Since a direct examination of rock mass structure in 3-D is not possible, the DFN approach maximises the utility of discontinuity data from mapping of exposed surfaces and boreholes or any other source of spatial information. Discontinuity data sampled from exposures in variably oriented outcrops (2-D) and boreholes (1-D) can be used to synthesize a 3-D stochastic discontinuity model that shares the statistics of the samples and allows for the incorporation of specific (deterministic) discontinuities.

Relatively undisturbed rock core samples can be obtained by high quality drill coring; however, specific sampling and analytical techniques are needed to measure the orientation of the sampled discontinuities within the rock mass. Observations of exposed rock faces, at or near the project site, have the advantage of allowing direct measurements of discontinuity orientation, spacing, and persistence and the identification of discontinuity sets. Other large-scale geometrical and structural features can be readily observed. Different techniques can be employed, including: scanline survey (linear mapping), rectangular window mapping and circular window mapping. Increasingly, digital photogrammetry and laser scanning techniques (LiDAR) provide an alternative technique for fracture characterisation.

The typical process involved in the generation of a DFN model is graphically illustrated in Figure 2, including: (i) fracture spatial model, (ii) fracture orientation distribution, (iii) fracture terminations, (iv) fracture radius distribution and (v) fracture intensity. Validation of the DFN model is achieved by comparing the orientation, intensity and pattern of the simulated fracture traces with those measured in the field. The stochastic nature of the process is such that there are an infinite number of possible realisations of the 3-D fracture system based on the mapped data. Indeed, the mapping process is itself random by the nature of how fractures are presented in available windows. With the exception of fully explicit modelling of an individual fracture or simplified fracture sets, the stochastic approach provides the best option for creating realistic geometric models of fracturing.

2.1 Fracture spatial model

A key parameter in the synthesis of a specific DFN model is the definition of a fracture spatial model (Figure 3). The main difference between DFN models is a function of the way fracture characteristics are considered (Dershowitz and Einstein, 1988; Staub et al., 2002; Rogers et al., 2007). Most of the models involve the same considerations for specific fracture characteristics, such as shape (generally polygons), size and termination at intersections. Fracture spatial models can be grouped according to the specific distribution laws utilised to simulate fracture orientation and fracture location. The choice of a specific fracture spatial model is typically based on assumptions made from field data and geological observations. The code FracMan used in the current study allows for the use of three different fracture spatial models: the Enhanced Baecher, the Nearest-Neighbour and the Fractal Levy-Lee models. In the Enhanced Baecher model fracture location may be defined by a regular (deterministic) pattern or a stochastic process. The stochastic approach assumes that the fracture centres are randomly located in space using a Poisson process. The Nearest-Neighbour model is particularly suited to model the tendency of fractures to be clustered around major points and faults by preferentially producing new fractures in proximity of earlier fractures (Dershowitz et al., 1998). The Levy-Lee model is a fractal model whose key features are that fracture centres are created sequentially and the size of a fracture is related to its distance from previous fractures (Staub et al., 2002).

2.2 Fracture orientation

DFN models can be generated separately for each fracture set and then combined to obtain the overall representation of the fracture network. The application of separate statistical procedures to define fracture sets and, consequently the separate DFN models for each is known as a *disaggregate* approach. Distributions such as Fisher, Bingham, bivariate Fisher and bivariate Bingham can be used to represent fracture orientation. Alternatively, field data that do not conform to straight forward statistical methods (i.e. characterised by a highly dispersed scatter), can be analysed using a *bootstrap* approach, whereby a statistical method based upon multiple random sampling with replacement from an original sample is used to create a pseudo-replicate sample of fracture orientations (Rogers et al., 2007).

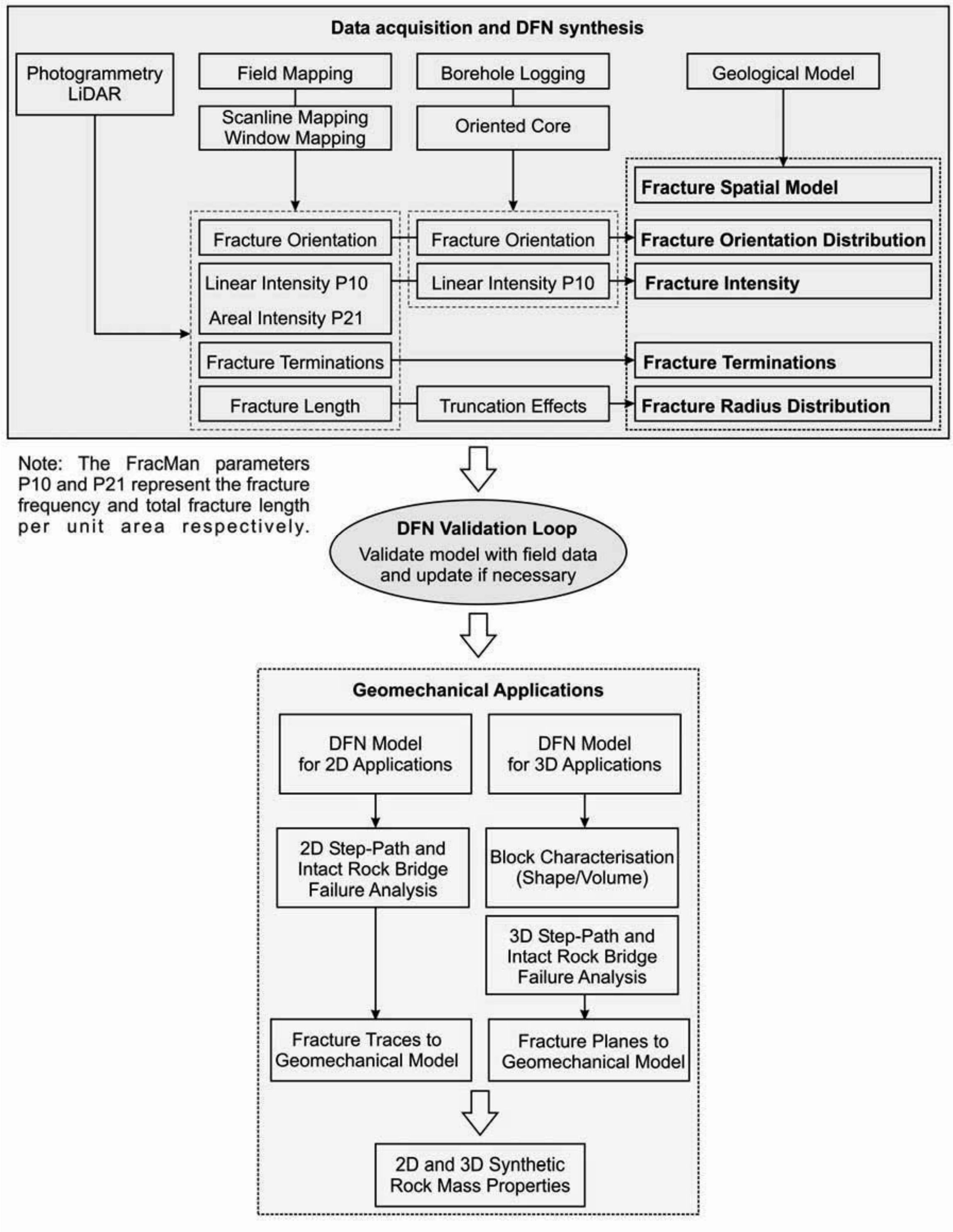
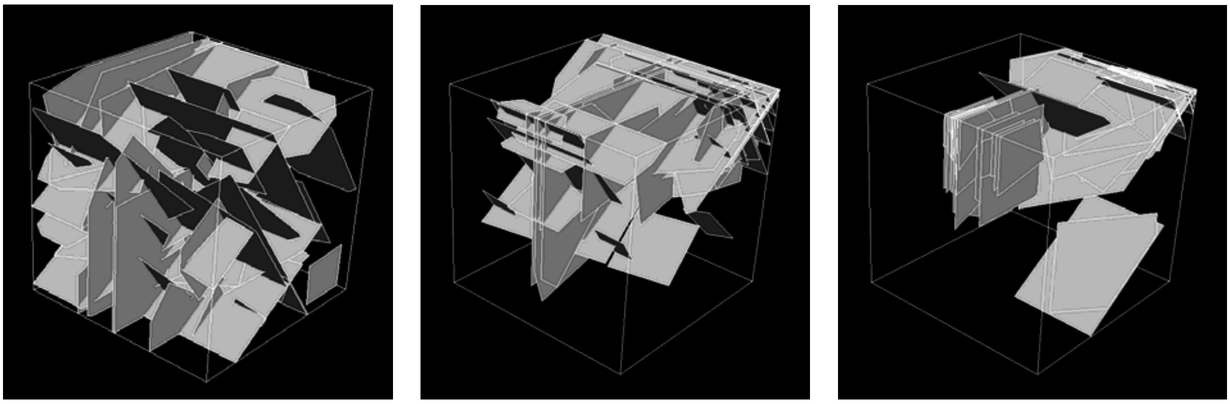


Figure 2 Methodology for the generation of a DFN model and geomechanical applications

2.3 Fracture terminations

Interdependence of fracture sets and chronology of fracture formation can be defined by using a fracture terminations percentage. Ultimately, this parameter controls the amount of rock bridges between adjacent fractures, and thus the potential for having fully formed blocks.



(a) (b) (c)

Figure 3 Example of DFN models generated using different fracture spatial models for equivalent fracture orientation and radius distributions. (a) Enhanced Baecher model, (b) Nearest-Neighbour model and (c) Fractal Levy-Lee model

2.4 Fracture radius distribution

Fracture size is another critical parameter controlling the definition of a blocky geometry. Both fracture shape and size can be assessed by analytical methods applied to data sampled on exposed rock faces (Mauldon, 1998; Zhang and Einstein, 1998 and 2000). The process is based on several assumptions:

- Fractures are considered to be planar.
- For mathematical convenience, in the current DFN approach fractures are assumed to be polygons with n sides and their size is defined by the radius of a circle of equivalent area.

Measured trace lengths are typically biased, therefore there is a need to correct sampling bias in order to derive true trace lengths, which are then employed to estimate fracture size distribution. The analytical method proposed by Zhang and Einstein (2000) for circular window mapping has the advantage over scanline and rectangular window mapping of automatically eliminating orientation bias (Figure 4). Elmo (2006) described an application of the Zhang and Einstein method in which the required numerical calculations were solved for Lognormal and Negative Exponential distribution forms. Recent developments of digital photogrammetry and LiDAR will provide an alternative source from which to estimate fracture size, particularly for large exposed rock faces with limited accessibility.

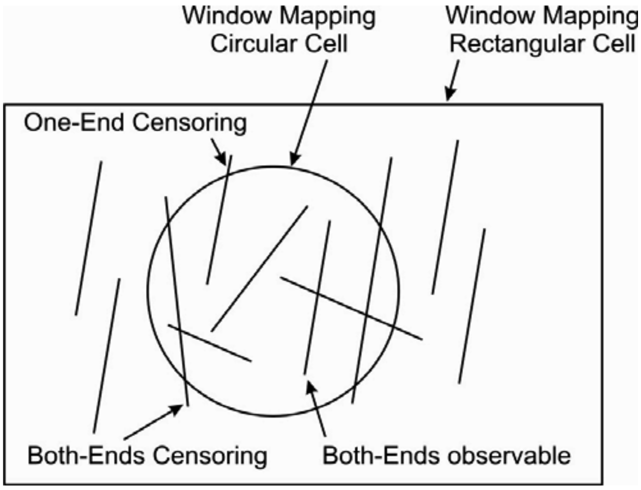


Figure 4 Schematic representations of truncation effects for fracture traces in a circular window (modified after Zhang and Einstein, 1998)

2.5 Fracture intensity

The generation of a DFN model requires the estimation of a parameter known as fracture intensity. More specifically, the volumetric fracture intensity is defined as the ratio of total fracture area to unit volume. This parameter, termed P_{32} (m^2/m^3), can be calculated on the basis of a linear correlation (Dershowitz and Herda, 1992) with P_{21} (m/m^2), which is the total trace length of fractures per unit area, or with P_{10} (m^{-1}), which is the total number of fractures along a scanline or borehole (i.e. fracture frequency). The constants of proportionality depend on the orientation and radius size distribution of the fractures and on the orientation of the sampling panel or scanline/borehole. Alternatively, the volumetric fracture intensity can be indirectly assumed by conditioning the DFN model to a direct replication of the number of fractures intersected along a scanline/borehole.

3 Importance of fracture persistence for a discrete modelling approach

The advantage of using a DFN approach for characterising naturally bounded blocks is demonstrated in Figure 5, and compared with the generally erroneous assumption of a blocky geometry based upon the assumption of infinite long fractures. The proprietary code FracMan (Golder Associates, 2006; Dershowitz et al., 1998) was used in the current analysis. All of the DFN models shown in Figure 5 share the same definition parameters with the exception of the fracture radius distribution, which is defined according to a lognormal distribution function whose descriptive parameters are listed in Table 1. The models in Figure 5 illustrate a simulated slope height of 20 m; the parent DFN models are generated within a 20 x 20 x 20 m box region.

Table 1 Descriptive parameters for the fracture radius distribution used in the DFN models shown in Figure 5

Model ID	Mean Length (m)	Standard Deviation	Distribution Form
A	1	0.5	Log normal
B	2	1	Log normal
C	5	2.5	Log normal
D	20	-	Continuous fractures

The FracMan code contains a geomechanics option allowing the definition of blocks intersecting 3-D surfaces (e.g. tunnel and slopes) and thus facilitating a kinematic stability analysis. However, the scope of the current modelling was not to define the kinematic nature of these blocks, rather it focused on their number, volume and relative continuity within the geological model. The latter is clearly related to the geometry and number of intact rock bridges. Since only bounded blocks daylighting on the outer simulated slope surface are detected, the blocks in Figure 5 were generated by using a sequential sampling technique (i.e. block analysis is performed repeatedly moving the simulated slope surface towards the box centre for an assumed incremental distance). This technique requires the incremental distance to be small relatively to the problem scale, the accuracy of the analysis increasing for very small incremental distances. The preliminary results illustrated in Figure 5 demonstrated a 10–stage sequential analysis, with an incremental distance of 0.5 m and this shows the blocks which are formed within a volume of rock which extends from the outer simulated slope surface inwards for a distance of 5 m.

Although the block analysis was limited to a portion of the total slope volume, Figure 5 clearly shows that the continuity of the blocks (i.e. the occurrence of intact rock bridges) is dependent on fracture persistence. Fewer and sparser rock blocks were defined for a DFN model with relatively shorter fractures. In contrast, very persistent fractures resulted in blocks which extended outside the assumed 5 m-wide sampling volume and it is reasonable to conclude that continuous fractures would ultimately result in a fully blocky structure (sugar-cube type structure). Finally, the analysis also demonstrated that, for each sampling stage, the average volume of the block intersecting the simulated slope is a function of the fracture radius distribution (Figure 6).

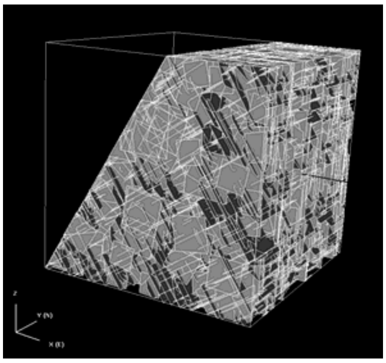
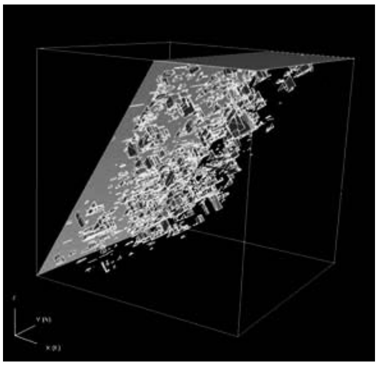
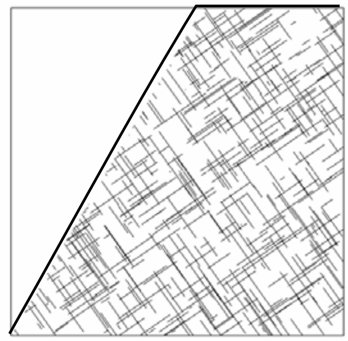
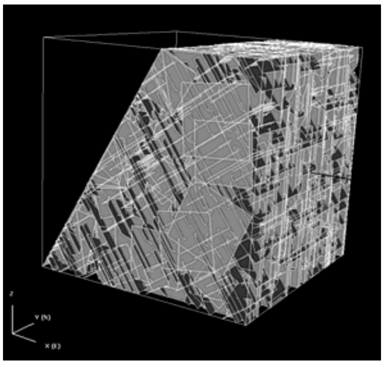
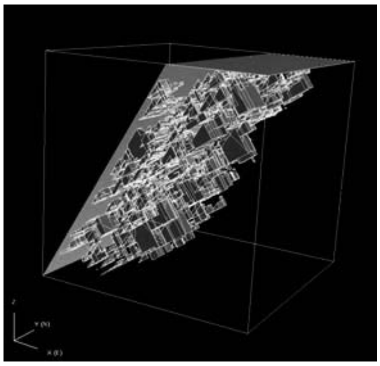
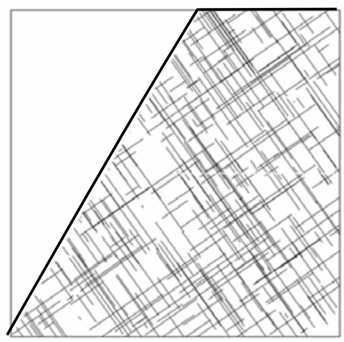
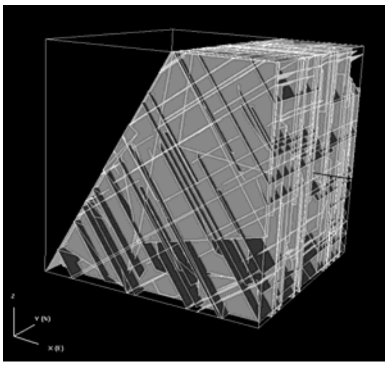
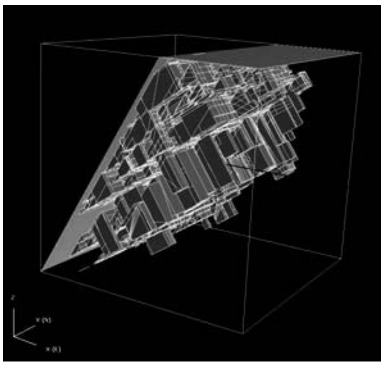
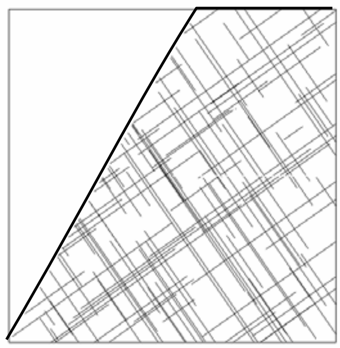
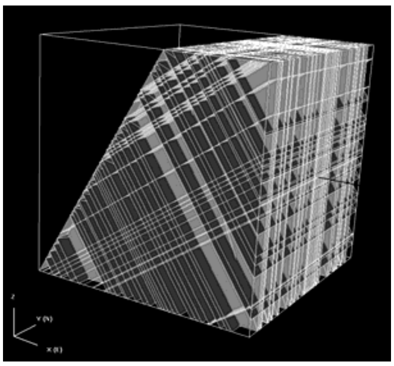
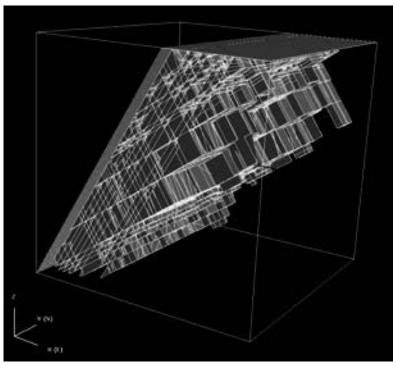
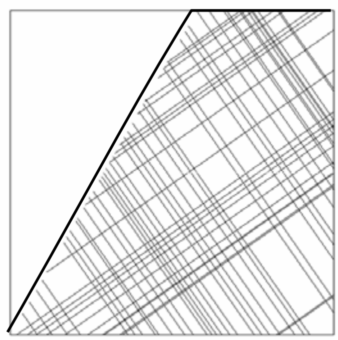
ID	Fracture Network	Blocks Intersecting Surface	2-D Section Normal to Surface
A 1m			
B 2m			
C 5m			
D 20m			

Figure 5 Comparison between DFN models simulated by varying the fracture radius distribution and using the values reported in Table 1. Models A, B, C and D have a mean fracture length of 1, 2, 5 and 20 m respectively

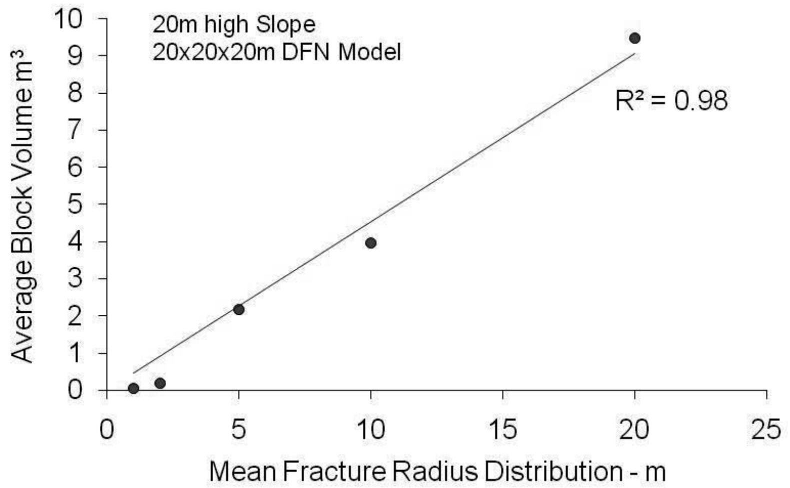


Figure 6 Average joint block volume for the 10-stage sampling analysis performed on the simulated rock slope shown in Figure 5

4 2-D Characterisation of discontinuity persistence and intact rock bridges

The quantification of fracture persistence and intact rock bridges represents a critical component of rock stability analysis. Definitions of fracture persistence have been given by many authors (e.g. Jennings, 1970; ISRM, 1981 and Einstein et al., 1983). For a fracture plane, persistence is conventionally defined as the fraction of discontinuous plane area; using fracture traces, persistence is defined as the ratio between the sum of fracture segments along a line and the total length of the line. Jennings (1970) explicitly expressed the total length of the line as the sum of fracture segments and intact rock bridges. Regardless of the definition adopted, limitations of field mapping as an accurate source for fracture persistence data are widely recognised; additionally, most discrete analysis techniques assume fracture planes to be fully persistent, i.e. with no discontinuous fraction on the plane, though the actual side length of the fracture plane may be finite.

The question of what constitutes a rock bridge is not a trivial issue. For instance, in Figures 7(a) and 7(b) the rock bridge can be easily defined as the shortest distance between two fractures. Figure 7(c), which is the more common situation, shows that more than one potential rock bridge exists based on the previous definition. Clearly, for a highly fractured rock mass the definition of a rock bridge would not be unique, varying from a purely geometrical distance to a more complex zone of influence within which a critical path is defined by extension and coalescence of existing fractures. A condition of minimum shear resistance is achieved along the critical path, which ultimately makes the definition of the effective rock bridge a complex interaction between geometry, material properties and applied field stress.

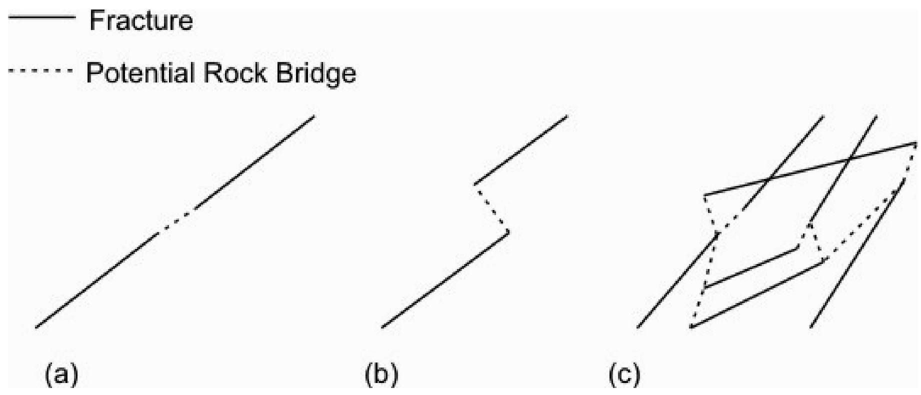
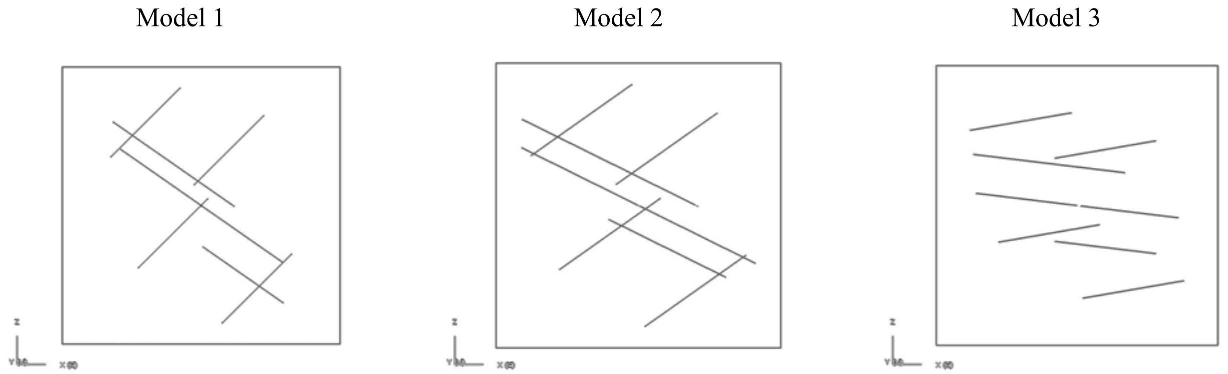
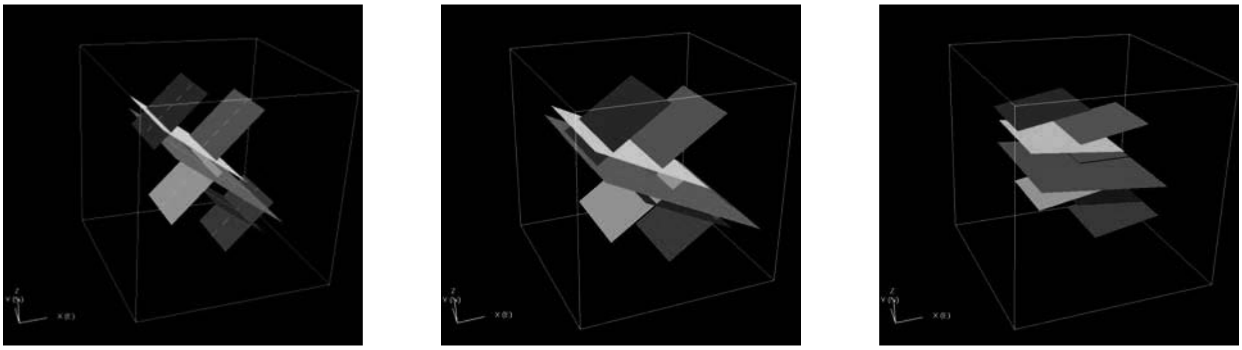


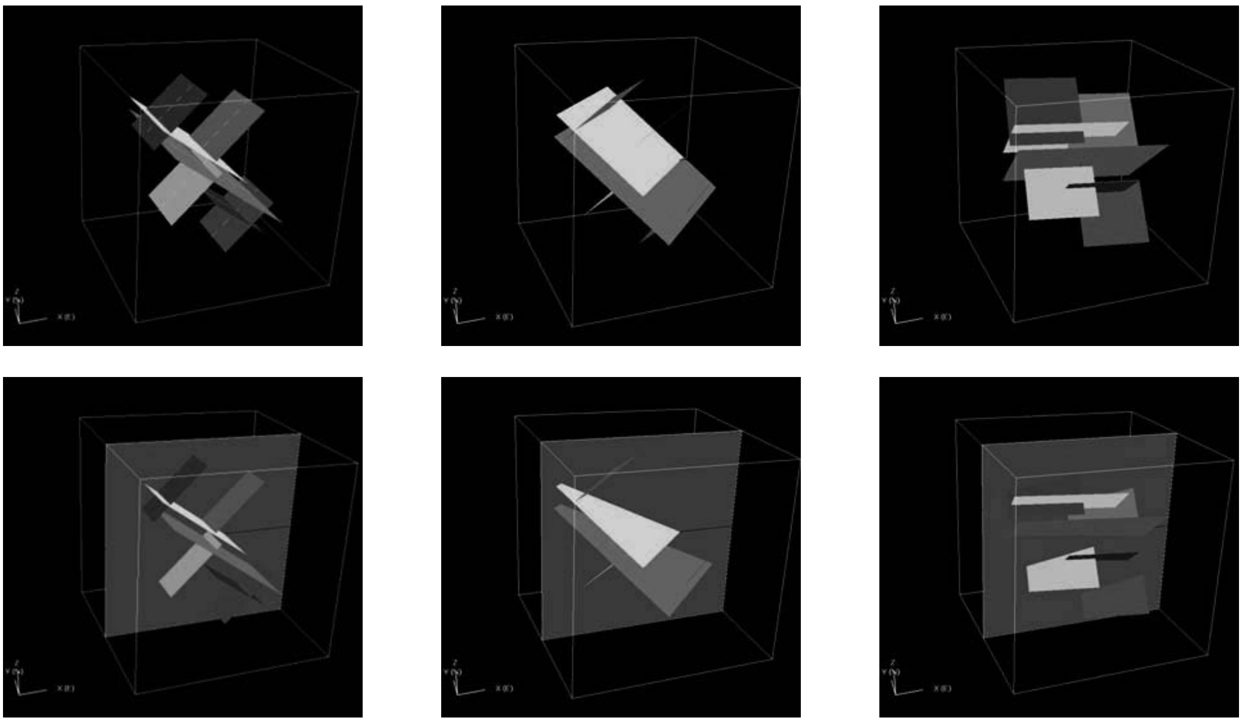
Figure 7 Definitions of a varying 2-D intact rock bridge



(a) 2-D sections showing different fracture intersection patterns and potential intact rock bridges



(b) 3-D fracture networks corresponding to the 2-D sections above and assuming out-of-plane continuity of the fractures according to 2-D *plane strain* conditions



(c) 3-D fracture networks simulated in FracMan used to derive the 2-D sections shown in (a)

Figure 8 Use of a 3-D DFN model as the source of fracture traces for a 2-D analysis: application and limitation

4.1 Application of a DFN approach: characterisation of rock bridges for 2-D analysis

The definition of a rock bridge given in the previous section is only relevant to 2-D analysis. The use of a fully 3-D DFN model as the source of fracture traces for a 2-D analysis should be exercised with caution. Figure 8(a) shows 2-D sections taken from different 3-D DFN models. The equivalent 3-D geometry of such 2-D parent models, extended in the out-of-plane direction, is shown in Figure 8(b) and compared in Figure 8(c) with the actual 3-D DFN source models used.

Some important conclusions can be made:

- The application of the typical 2-D definition of intact rock bridges is clearly limited to very specific cases (e.g. Model 1 in Figure 8a).
- Using a fully 3-D fracture model as the source of 2-D fracture traces for a 2-D geomechanics discrete element analysis may yield results unrepresentative of the actual geological model, altering the mode and characteristics of potential kinematic instability. Fictional rock bridges (e.g. Model 3 in Figure 8a) may also be introduced in the 2-D geomechanical model when importing fracture traces. For these reasons, a filtering process should be applied to any initial 3-D fracture model prior to use in a 2-D analysis, which for instance includes synthetising only fracture sets whose dip direction is parallel to sub-parallel to the assumed 2-D section (e.g. Moffitt et al., 2007).

4.2 Quantitative characterisation of 2-D intact rock bridges based on DFN fracture intensity parameters

A fundamental step towards a more realistic rock slope analysis is the quantitative characterisation of 2-D intact rock bridges as a function of fracture intensity parameters. The analysis presented here is based on the 3-D DFN models shown previously in Figure 5. Fracture intensity parameters used P_{21} (m/m^2) and P_{20} (m^{-2}), which are the total length of fractures per unit area and total number of fractures per unit area respectively, with the analysis results presented in terms of the ratio P_{20}:P_{21}. All DFN models in Figure 5 were generated according to an arbitrarily assumed P_{10} value of 1 (P_{10} is the fracture frequency, i.e. the inverse of fracture spacing). A 2-D section plane was taken normal to the simulated slope surface and its relative orientation with respect to the joint sets used in the DFN model guaranteed that the limitations described in Figure 8 were fully accounted for.

Figure 9 shows the relationship between the mean fracture length for the DFN models illustrated in Figure 5 and the P_{20}:P_{21} ratio calculated for the corresponding 2-D fracture traces. This relationship provides a quantitative indication of the potential occurrence of rock bridges: the lower the P_{20}:P_{21} ratio the longer the fractures and accordingly the fewer the rock bridges. However, it cannot be used as an effective quantitative index of the actual physical dimensions of the rock bridges. As an alternative, it is proposed to characterise the rock bridges in a quantitative form using this relationship and linking it with current rock mass classification systems. The proposed approach offers the possibility of a quantitative representation of the effects of rock bridges in terms of equivalent continuum rock mass properties. For instance, RMR - Rock Mass Rating - (Bieniawski, 1989) values were back calculated for each one of the 2-D sections shown in Figure 5 assuming specific ratings for intact rock strength, RQD, fracture spacing and joint conditions (Table 2). The relationship between RMR values and mean fracture length was then compared with the estimated relationship with the P_{20}:P_{21} ratio (Figure 10) showing an overall good correlation and therefore highlighting the potential applications of the proposed approach. Research is currently undergoing to validate the proposed methodology, extending it also to other rock mass classification/characterisation systems, including the GSI - Geological Strength Index - (Hoek et al., 2002) and Q index (Barton et al., 2002). At this stage of the research, the ratings assumed for RMR in Table 2 focused on the effects of varying joint persistence; however, it is argued that further studies will need to allow for both covariance and scale issues, since joint roughness, spacing and RQD would potentially differ as a function of joint length (e.g. longer fractures are typically wider spaced than shorter ones).

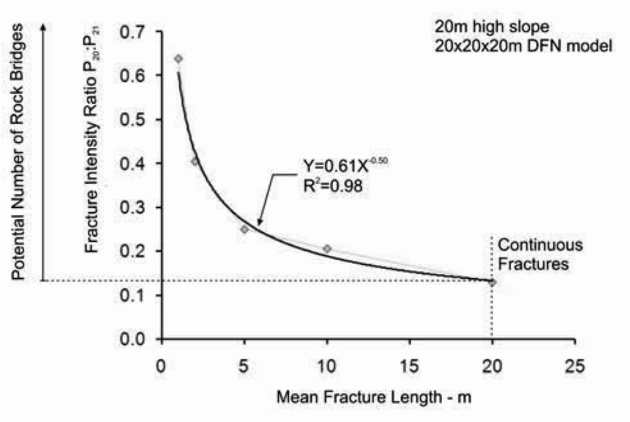


Figure 9 Relationship between mean fracture length for the DFN models illustrated in Figure 4 and the P20:P21 ratio calculated for the corresponding 2-D fracture traces

Table 2 RMR values back calculated for each of the 2-D sections shown in Figure 5

Description	1 m model	2 m model	5 m model	10 m model	20 m model
Rock strength	12	12	12	12	12
RQD	20	20	20	20	20
Spacing	10	10	10	10	10
Joint length	6	4	2	1	0
Joint aperture	5	5	5	5	5
Joint roughness	1	1	1	1	1
Joint infill	6	6	6	6	6
Joint weathering	5	5	5	5	5
Water	15	15	15	15	15
TOTAL	80	78	76	75	74

4.3 Preliminary modelling results of a conceptual 2-D open pit using simulated fracture traces derived from a DFN analysis

This section presents preliminary results of a modelling approach that combines 2-D DFN analysis with a hybrid Finite Element/Discrete Element numerical code. Simulations are presented based on DFN models of a large conceptual rock slope and incorporating varied failure mechanisms, demonstrating the importance of both considering realistic fracture mechanisms and the ability to model complex failure paths involving sliding along discontinuities, dilation, and intact rock fracture. A conceptual model was used in the current study, representing excavation of a 400 m open pit. In subsequent research, the study will be extended to include larger (i.e. deeper) open pit geometries. The proprietary code FracMan was used to develop a 3-D Discrete Fracture Network (DFN) model from which 2-D sections were taken and fracture traces then imported into ELFEN via a dedicated interface. This allowed consideration of regularly defined joint sets and a parametric study on the effects of varying fracture orientation and intensity. The DFN model included two orthogonal joint sets, inclined at 30 and 60 degrees respectively.

The fracture intensity parameter used in the current FracMan model determines what portion of the natural occurring fractures will be modelled. Since not all fractures are represented by the model, the unfractured rock in the model would actually have some degree of fracturing in the field. In order to represent this fracturing, equivalent fractured rock mass properties were defined for the initial conceptual models presented in this paper; details of the model input are reported by Elmo et al. (2007).

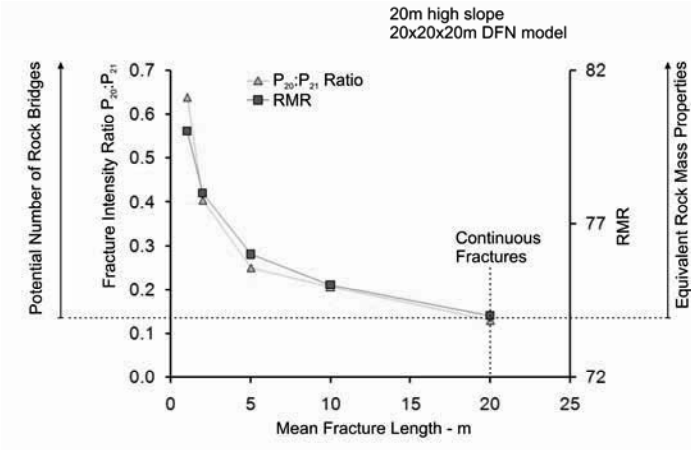


Figure 10 Relationship between RMR values and mean fracture length compared with the estimated relationship with P_{20}:P_{21} ratio shown in Figure 9

The fracture insertion capability of the ELFEN code was exploited in order to simulate stress concentrations at the tips of pre-inserted fractures. This provided an indicator of the rock mass structural integrity and its subsequent response to changes in stress and material properties. Under either tensile or compressive loading conditions, pre-inserted fractures act as the source of further cracking/fracturing of the rock due to their coalescence, resulting in an overall mechanism of brittle rock slope failure. As an example, numerical modelling was clearly able to reproduce extension of existing fractures and step-path failure through fracturing of intact rock bridges (Figure 11). However, it should be noted that the current model only accommodates fracturing through pure Mode I (i.e. tensile fracturing).

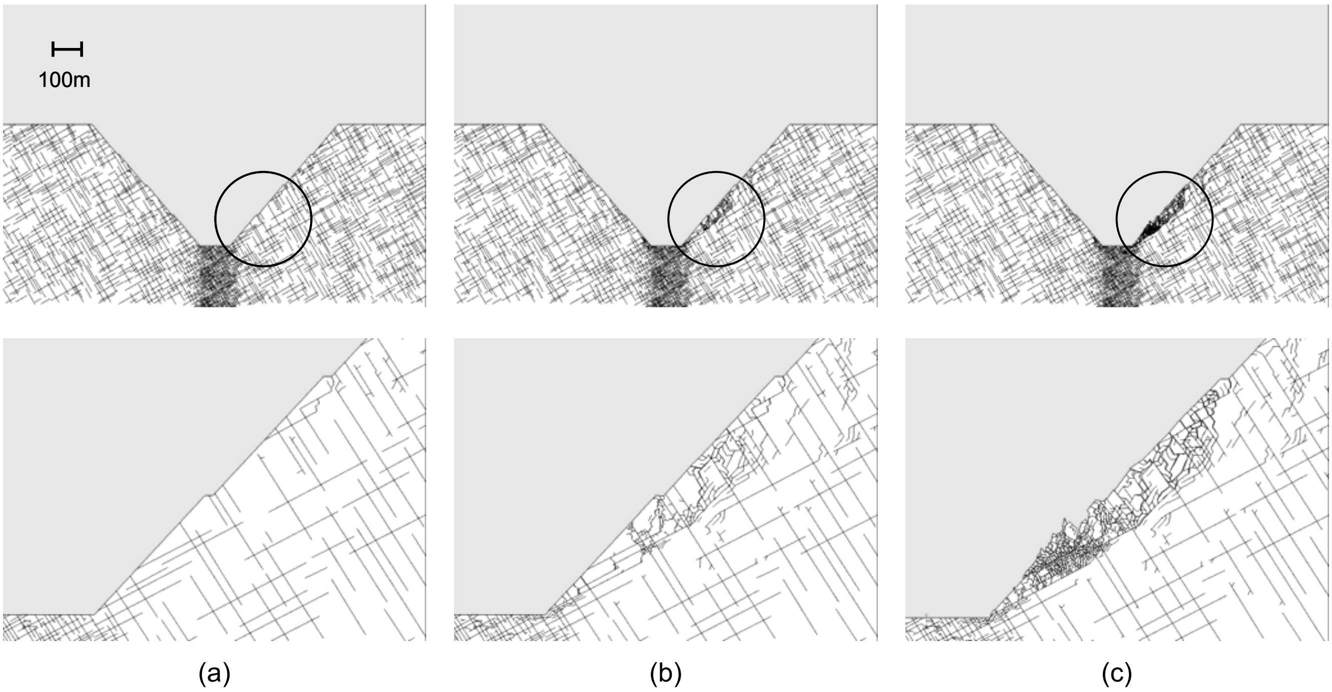
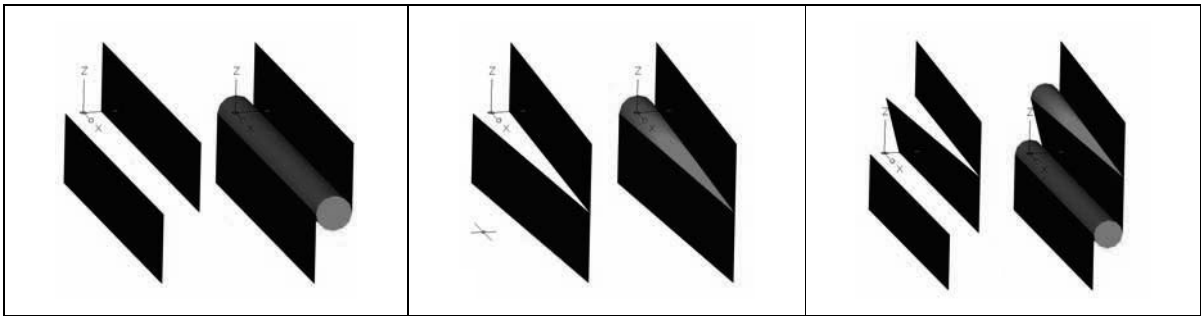


Figure 11 Simulations based on DFN models of a large conceptual rock slope, demonstrating the importance of both considering realistic fracture mechanisms and the ability to model complex failure paths involving sliding along discontinuities, dilation, and intact rock fracture. (a) Full 400 m pit excavated, (b) simulations of step-path and failure of intact rock bridges as a result of induced stresses and (c) simulations of varied failure mechanisms, including sliding and toppling mechanisms

5 Towards a characterisation of rock bridges in a 3-D space

The previous numerical examples demonstrated the effective combination of a *corrected* 3-D DFN approach for 2-D characterisation of intact rock bridges. In order to characterise intact rock bridges in a 3-D space, we first need to define what constitutes a rock bridge in a 3-D fracture model. The geometrical definition of a rock bridge given in section 4, with the exception of the case illustrated in Figure 8 (Model 1), is too simplistic for the 3-D case. The concepts of *cylindrical* and *conical* zones of influence are introduced in this paper, as the basis of an undergoing research project focusing on the characterisation of the strength of intact rock bridges for 3-D modelling (Figure 12). Clearly, the principle of superimposition suggests that the basic concepts of cylindrical and conical zones of influence would result in complex geometrical entities representing a hypothetical rock bridge in 3-D. Accordingly, the concept of step-path failure would also be more complex to analyse. Currently, 2-D discrete analysis allows the study of relatively large slope stability problems and incorporates numerical simulation of macro-scale fracture processes through intact rock bridges. However, large-scale 2-D problems and furthermore fully 3-D large scale (discrete) analysis of fracturing processes is limited by the memory and processing capacity of existing computer platforms (Stead et al., 2007).



(a) (b) (c)

Figure 12 Diagrammatic illustrations of the concepts of (a) cylindrical, (b) conical and (c) composite zone of influence to characterise intact rock bridges in a 3-D space

In an analogous way to the method described in Section 4.2, it is proposed to characterise 3-D intact rock bridges indirectly in terms of joint block volume and volumetric fracture parameters in FracMan. The proposed approach would resolve the effects of intact rock bridges as a function of equivalent rock mass properties, therefore not providing an exact measurement of their geometrical extent and shape. Further studies are underway to validate the use of the relationship between average joint block volume and mean fracture length shown in Figure 6 as the basis of a possible correlation with GSI, and more specifically linking this to recent quantification of GSI based on fracture spacing and blocks volume (Cai et al., 2004).

6 Discussion and conclusion

This paper has demonstrated the effectiveness of a discrete modelling approach in assessing the importance of intact rock bridges in the stability of high rock slopes. It is clear that fracture persistence represents an important parameter in the characterisation of a fracture model. Areal fracture intensity parameters and average joint block volume were found to be directly linked to the mean fracture length. Having discussed applications and limitations of the use of a 3-D DFN model as the source of fracture traces for a 2-D geomechanics analysis, a new approach for characterisation of 2-D intact rock bridges was introduced. This is based on a correlation between fracture intensity parameters and existing rock mass classification systems in order to define equivalent rock mass properties for a fractured rock mass. The proposed approach is independent of the geometrical characteristics of the rock bridges. In an analogous way, it was proposed to characterise 3-D intact rock bridges in terms of joint block volume and volumetric fracture parameters in FracMan.

Preliminary results of 2-D modelling of a 400 m open pit show the importance of both considering realistic fracture mechanisms and the ability to model complex failure paths involving displacement and rotation along discontinuities. However, large-scale 2-D problems, and furthermore, fully 3-D large scale (discrete) analysis of fracturing processes is currently limited by the memory and processing capacity of existing computer platforms. Extending the discussion to the more general 3-D scenario, it is argued that the typical definition of a rock bridge is possibly too simplistic for the 3-D case and in this context this paper suggests the use of *cylindrical* and *conical* intact rock fracture influence zones to characterise 3-D rock bridges.

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