Applications of fracture system models (FSM) in mining and civil rock engineering design

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ABSTRACT
Engineering design in rock must, implicitly or explicitly, take into consideration the influence of small and large scale geological fractures. The complexity of a jointed rock mass is best captured using 3D fracture system model based on quality field data. In this article, we describe on-going work in developing and implementing fracture system models (FSM) to solve three engineering problems using the developed stochastic fracture modelling tool, Fracture-SG. The first case study uses field data from 53 mine sites to demonstrate the advantages of using FSM, as compared to empirical classification indices to quantify the structural complexity of a rock mass. The second case describes the determination of a structural representative elemental volume (REV) along a rock slope, and the third case study describes the use of FSM as an integral part of the stability analysis of a slope subject to structural failures.

KEYWORDS
fracture system models; rock engineering; slope stability; REV; mining

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1 INTRODUCTION

The design of structures in rock requires a quantification and understanding of the behaviour of rock masses under load. In this context, a rock mass is defined as a network of fractures imbedded in an intact rock matrix. While the properties of intact rock are well documented, and can be assessed with standardised laboratory tests, this is not the case for fractured rock masses.

The traditional source of structural data to describe a rock mass is structural mapping. A review of available mapping options suggests that the choice of a suitable mapping technique depends on existing field conditions and intended use of data. It has been demonstrated that most statistical techniques, defining the structural fabric of a rock mass, are only applicable when a sufficient amount of data is available [1]. If the objective is to determine consistent estimates for discontinuity spacing, it may be acceptable to employ production mapping techniques, i.e. recording fractures greater than a certain cut-off trace length. If, however, the intention is to gain a better understanding of the structural regime of a rock mass, it is necessary to record the position, dip, dip direction, trace length, end points, terminations, planarity and roughness of all discontinuities intersecting a scanline

[1]. Digital imaging, laser-based imaging, image and stereo-vision hardware and software are becoming increasingly popular for the characterisation of rock exposures.

These techniques offer several advantages in comparison with manual discontinuity sampling methods: (i) reduction of time and effort required for mapping; (ii) limited exposure of operators to potentially unsafe conditions; (iii) development of a permanent geomechanical database that can be consulted at any time, for example, after excavation or lining of the exposures and (iv) greater quantity of collected data, resulting in more representative and accurate values of fracture network properties [2].

The corollary is that, if considerable effort is invested in collecting geological structural data, it makes sense to maximise the use of such data. In this context, the use of stochastic models is attractive as it provides a powerful means of representing fracture systems. Fracture system modelling employs borehole, line mapping or face mapping field data to generate representative models of the prevailing structural conditions. While most of the work in the last 20 years has been associated with nuclear waste sites and the modelling of fractured hydrocarbon reservoirs, the authors have been working towards the implementation of fracture system modelling in the design of rock engineering structures in fractured rock masses. This article summarises the evolution of fracture system modelling approaches developed by the authors in the last 15 years and presents three case studies where fracture system model (FSM) was used and the derived benefits from its application.

2 FRACTURE SYSTEM MODELLING

Dershowitz and Einstein [3] presented the fundamentals of stochastic fracture modelling and demonstrated how different models, including the Orthogonal, Baecher, Veneziano, Dershowitz and Mosaic Tessellation, can represent the rock mass geometry as an entity and can be used to quantify the spatial variability of fracture geometry. Staub et al. [4] have updated this information and described the more recently developed models. Table 1 is a slightly modified summary of the models reviewed by Dershowitz and Einstein [3] and Staub et al. [4]. The listed models assume that fractures are planar, and any location or autocorrelation process is possible. In most cases fracture locations are stochastic. Fracture size refers to trace length on two-dimensional surfaces or as the surface area of individual fractures. Fracture sizes are usually stochastic, either specified directly or indirectly through stochastic location and orientation. Bounding of fractures implies that fractures smaller than the region under consideration can be represented. Field observations suggest that fractures can either terminate at the intersection with other fractures or against intact rock. This is recognised in most models. Coplanarity implies that a number of fractures can be located on the same plane.

The majority of the fracture system models have not been adequately verified for engineering applications. In practice, the choice of the model will depend on how it can be related to the available field data and to the engineering needs of the project.

Recent years have seen the development of several fracture system generators, incorporating different FSM, of varying complexity and ease of use [5–9].

Table 1. Main features of different fracture system models, modified from Staub et al. [4].

Model	Fracture characteristics considered in model				
	Fracture shape	Fracture size	Termination at intersection	Co-planarity	Orientation of sets
Orthogonal	Rectangle	Bounded	No	–	Parallel
		Unbounded	Yes	Yes	Parallel
		Unbounded	No	Yes	Parallel
Baecher	Circle Ellipse	Bounded	No	No	Stochastic
Veneziano	Polygon	Bounded	In fracture planes only	Yes	Stochastic
Dershowitz	Polygon	Bounded	Yes	Yes	Stochastic
Mosaic Tessellation	Polygon	Bounded	Yes	Yes	Regular stochastic
Enhanced baecher	Polygon	Bounded	Yes	No	Stochastic
Baecher algorithm, Revised terminations	Polygon	Bounded	Yes	No	Stochastic
Ivanova	Convex Polygon	Bounded	Yes	Yes	Stochastic
Poisson rectangle	Rectangle	Bounded	Yes	No	Stochastic
Box fractal model	Polygon	Bounded	Yes	No	Stochastic (random)
Geostatistical	Polygon	Bounded	Yes	No	Stochastic
War zone	Polygon	Bounded	Yes	No	Stochastic
Non-Planar zone	Polygon	Bounded	Yes	Yes	Binary
Levy-Lee fractal	Polygon	Bounded	Yes	No	Stochastic
Nearest neighbour	Polygon	Bounded	Yes	No	Stochastic
Fractal POCS model	Polygon	Bounded	Yes	No	Stochastic

2.1 Development of simple fracture modelling tools

Early work by the authors led to the development of the Stereoblock fracture generator based on the Baecher model [10]. In the Baecher model [11], joints are represented by disks. Over the years, Stereoblock has been modified, in order to respond to a series of engineering applications, taking advantage of algorithms presented by Villaescusa [12], Lessard [13], Grenon [14] and others.

Applications of Stereoblock have included the integration of fracture systems to the analysis of underground drifts and stopes [15,16], to provide characteristic block size distributions for a cave mining project, [17] etc.

In due time, the Stereoblock code was retired in favour of the Fracture-SG code. The following section presents the employed methodology in the development and recent modifications to the Fracture-SG code [18]. The Fracture-SG was coded under the Matlab environment [19]. The new code uses a fracture system generator based on the Veneziano model that uses polygons to represent fractures. Some of the differences in these codes were the transition from the C++ platform for Stereoblock to the use of Matlab for Fracture-SG and eventually the new suite of software based on Fracture-SG. This has allowed much more programming flexibility and has improved the implementation time. Furthermore, it allowed easy visualisation of the fractures, which is of great importance when validating the analysis and presenting the results to decision makers.

Work in parallel to the development of Fracture-SG has resulted in a fully integrated module, Fracture-SL, which can be used to investigate the stability of tetrahedral wedges formed at the crest of rock slopes. Fracture-SL can be used to determine the factor of safety (FS) and the probability of failure (PF) of wedges considering the presence of tension cracks, reinforcement elements, etc. but more importantly it can accommodate as input data multiple FSMs. This has allowed a much more realistic stability analysis of rock slopes.

Another module, Fracture-UN [20] can be used to determine the stability of tetrahedral wedges formed at the periphery of underground excavations, also accounting for the impact of both reinforcement elements and surface support.

It has already been recognised that the accuracy or pertinence of FSM depends on the quality of the sampled structural data. This necessitates some degree of flexibility in how a fracture generator can accept input data. The developed FractureSG code may accommodate fracture sampling conducted by borehole, scanline mapping and face mapping. It has also been used with data collected from laser scanning. Depending on data availability, Fracture-SG considers fracture intensity, size, orientation, co-planarity, termination and hierarchy.

The first step in the modelling process in Fracture-SG is volume definition. Volume size is a function of the engineering problem analysed. To avoid boundary effects, the volume size must be larger than the scale of the investigated engineering problem. In the mid71990s, when the authors were first employing FSM in engineering problems, volume size was constrained by computer limitations and execution times. This has now been largely overcome and we have been successful in running large scale models.

A major challenge in rock engineering is accounting for non-homogeneity. The engineering approach is to consider a rock mass as comprised of multiple structural domains. Each domain is a region with homogenous structural properties as observed in situ. When an engineering surface or underground excavation is intercepted by several structural domains, it is necessary to divide the rock mass volume using domain delimitation planes, which can be located anywhere within the volume of interest. This is easily handled in Fracture-SG.

Smaller structural fractures, such as joints, cannot be discretely modelled within a rock mass volume. Based on field observations, a stochastic process is needed to generate fractures within a rock mass volume, on a fracture set by set basis. Fracture-SG is based on a modified Veneziano model [21]. The original Veneziano model is described by Dershowitz and Einstein [3]. The model relies on the generation of a Poisson network of planes in 3D space followed by a secondary process of tessellation by a Poisson line process and marking of polygonal fractures. The resulting polygonal shape fractures, and the implication that fractures produced on the same plane during the primary generation process remain coplanar after the secondary tessellation process, are often perceived as the main limitations of the Veneziano model. Nevertheless, the Veneziano model is conceptually simple and the required input parameters can be easily inferred from field data. In the latest version of Fracture-SG, modifications were made to the Veneziano model to consider non-coplanar fractures, multiple structural domains and fracture hierarchy. This allows the definition of structural domains that better capture the complexity of a given rock mass.

Another important consideration in large scale excavations is the presence of major structural features such as faults and geological contacts. Such features can in fact be the critical factors that dictate the behaviour of a fractured rock mass. If the location and shape of these major structures is known with a degree of confidence, these discrete features can be positioned at any desired location within the FSM volume. These discrete major structures may, or may not, be geometrically controlled by the domain delimitation planes and can also coincide with a domain delimitation plane.

The primary process in Fracture-SG is the generation of a Poisson network of planes in 3D space based on a fracture set, by set basis. Plane orientation is defined by randomly selecting a fracture orientation value from a probability density function (pdf) defined by a set mean dip, mean dip direction and Fisher constant, with planes randomly located within the 3D volume. A required input parameter is fracture intensity P32 (fracture area per unit volume) with the primary process repeated until the following criterion has been met:

$$\sum_i A_i = V \cdot P_{32} \quad (1)$$

where A_i is the area defining the generation plane i and V the volume of the FSM.

The secondary process is the tessellation by a Poisson line process, and marking of polygonal fractures. The Poisson line process intensity 'λ' is proportional to the inverse of the square root of the mean fracture area E[A] [21].

$$\lambda = \sqrt{\frac{\pi}{E[A]}} \quad (2)$$

A marking rule is employed to establish which fractures on a generation plane are the most acceptable or plausible from a geological perspective. Dershowitz and Einstein [3] have suggested that circular shape fractures have been produced in the laboratory and have been observed in the field. Baecher et al. [11] cited the work of Robertson [22], which analysed structural data on strike length and dip length and showed that they were approximately equal. This was interpreted as evidence that joints are either equidimensional or that their major axis is randomly oriented.

Fracture-SG makes use of the marking rule proposed by Ivanova [9] to select, as fractures, only polygons that are equidimensional or slightly elongated. The implemented rule requires that:

- a polygon has at least four vertices;
- the angle between two sides of a polygon is at least 608;
- polygon elongation is not greater that 1.6.

The third process implemented in Fracture-SG addressed some of the limitations of the original Veneziano model. During the secondary generation process, all fractures on a given generation plane are coplanar. Field observations suggest that this is not always a valid assumption. A solution to this problem has been provided by Meyer [23], whereby fractures on a given generation plane can be translated by the following rule:

$$dz'_{max} = C \frac{E[R'_e]}{[R'_e]} E[R'_e] \quad (3)$$

where C is the coplanarity factor ranging between [0, 1] $E[R'_e]$ is the mean value of the equivalent radius of all polygons; $E[R'_e]$ is the equivalent radius of the fracture to be translated.

Factor C is a user defined input value based on field observations. If fractures are strongly coplanar the Veneziano model is deemed adequate and assigned a 'C' value of 0. If fractures are not coplanar, Meyer's approach allows for translation from the generation planes as a function of the C variable and the fracture equivalent radius. In this approach, larger fractures are translated to a greater distance from the initial plane than smaller fractures.

The fourth process in Fracture-SG consists of marking the structural domains and establishing a fracture set hierarchy. The structural domain marking process, involves sampling all polygonal fractures retained through process 1 to 3. If a fracture set is defined over the entire volume, all generated fractures are considered valid. A fracture set can also be constrained. It can be constrained by stating that it exists only on one side of a given delimitation plane. Furthermore, the location of a given fracture set can be also constrained by multiple delimitation planes. All fractures that do not respect these conditions are removed from the generated FSM.

All fractures that intersect the delimitation planes can either be truncated by that plane or allowed to keep their original dimensions. The applied criterion can be identical for all fractures, or it can be randomly assigned based on a given probability of occurrence (p_{sd}). Events that are not likely to happen have a probability near 0, and events that are likely to happen have probabilities near 1. The objective in Fracture-SG is to provide the best match between field observations and the modelled system.

If field observations of mapped fractures suggest that a given set terminates on another set with a probability (p_{ss}), this probability can be used as another marking rule to better represent the field conditions in set hierarchy.

The adequacy and reliability of a constructed fracture system model is difficult to evaluate, [24]. The validation process used by the authors for stochastic modelled fractures relies on forward modelling. An initial fracture system is generated, on a set by set basis, using fracture orientation data obtained by field sampling, a fix fracture intensity value (P_{32}) and an arbitrary value for fracture size (area).

The fracture size obtained is then compared with the field values. For example, it is possible to compare fracture trace length obtained on a plane oriented in the same direction as the sampling surface sampled in the field. Based on the compared results, the input fracture size is modified. The process is repeated until modelled and field values are statistically equivalent.

Once fracture size is deemed acceptable, the model is calibrated for fracture intensity. Using the initial intensity values, modelled results are compared to in situ values. The validation process differs according to the initial data collection. If the structural data were collected using borehole or scanlines, then fracture frequency is used for data validation. However, if areal mapping was performed, the total discontinuity trace length per plane unit area (P_{21}) is a better validation tool. This

iterative process is repeated until an acceptable statistical agreement is reached between modelled and field data.

Further fracture properties, such as hierarchy, can also be assessed in the model and compared to field values. Once the validation process is completed, the calibrated input data can then be used to generate possible representations of the fracture system at the desired engineering design scale.

3 FSM APPLICATIONS IN MINING AND CIVIL ROCK ENGINEERING

It has already been established that FSM have received limited attention as part of the design process of structures in fractured rock masses. This may have been due to the evolving nature of FSM, but also to the misconception that the methodology was extremely labour intensive and time consuming. Another possibility may have been a lack of understanding of the powerful potential of FSM. This article presents three case studies where FSM modelling was successfully used by the authors. The first two case studies report on how FSM was used to establish a rigorous characterisation of structural rock mass properties, while the latter focus on the stability analysis of a rock slope.

3.1 Rock mass characterization

Quite often, the design of underground excavations in hard rock mines relies on empirical methods. A basic premise of empirical methods is that the quality of a rock mass can be quantified by means of a unique index that groups together a series of geomechanical parameters. These limitations of traditional rock mass classification systems and their constitutive parameters have been discussed by Palmstrom and Broch [25].

This section focuses on how structural properties are quantified during rock mass characterisation using various popular systems. It further addresses the question of whether there is a direct link between the various techniques that define structure and if it is justified to consider the diverse tools of quantifying structure as equivalent. Finally, it demonstrates that defining the structural regime with FSM enables a truly satisfactory representation of the three-dimensional nature of the in situ structural data.

3.1.1 Rock mass structural data

Structural mapping was undertaken at five Canadian underground hard rock mines located within the Canadian Precambrian Shield. The Abitibi Greenstone belt is a volcano-sedimentary region. Extensive scanline mapping and rock mass classification were performed to adequately characterise the dominant rock types in various regions of the mines. A region was defined as a portion of a mining stope, or a section of a drift, where the rock mass properties were relatively uniform. A total of 43 homogeneous regions or structural domains were sampled.

3.1.2 Characterisation results

Once the regions were adequately sampled it was possible to quantify their discontinuous nature (Figure 1). Based on scanline mapping the following structural indices were evaluated:

- fracture frequency,
- volumetric joint count (the number of fractures in a unit volume) [26],
- RQD, Deere [27]

Furthermore, the structural properties of two of the most common rock characterisation systems were also recorded:

- RQD/J_n of the Q system, where J_n is an index based on joint sets. RQD/J_n according to Barton et al. [28] is an index of block size.
- The sum of the index for RQD (parameter B) and the index for the spacing of discontinuities (parameter C) in the RMR system [29].

Finally, the results of scanline mapping were also used to generate three-dimensional FSM using the Stereoblock code and the following parameters were also determined:

- the number of discontinuities per unit area (P_{20}),
- the number of discontinuities per unit volume (P_{30}),
- the total discontinuity trace length per plane unit area (P_{21}),
- the total discontinuity area per unit volume (P_{32}),
- In situ mean block size (ISBD).

While the evaluation of the majority of these parameters was straightforward, the number of discontinuities per area (P_{20}) and trace length of discontinuities per area (P_{21}), evaluated through the FSM, are dependent on the evaluation plane orientation. To somewhat correct for this bias, two orthogonal plane orientations were chosen. The definition of in situ block size is another important application. In this work, we adopted the approach by Kleine [30] where in situ block size (ISBD) is related to the mean non-fractured distance between discontinuities along randomly oriented scanlines. The results are presented in Figure 1.

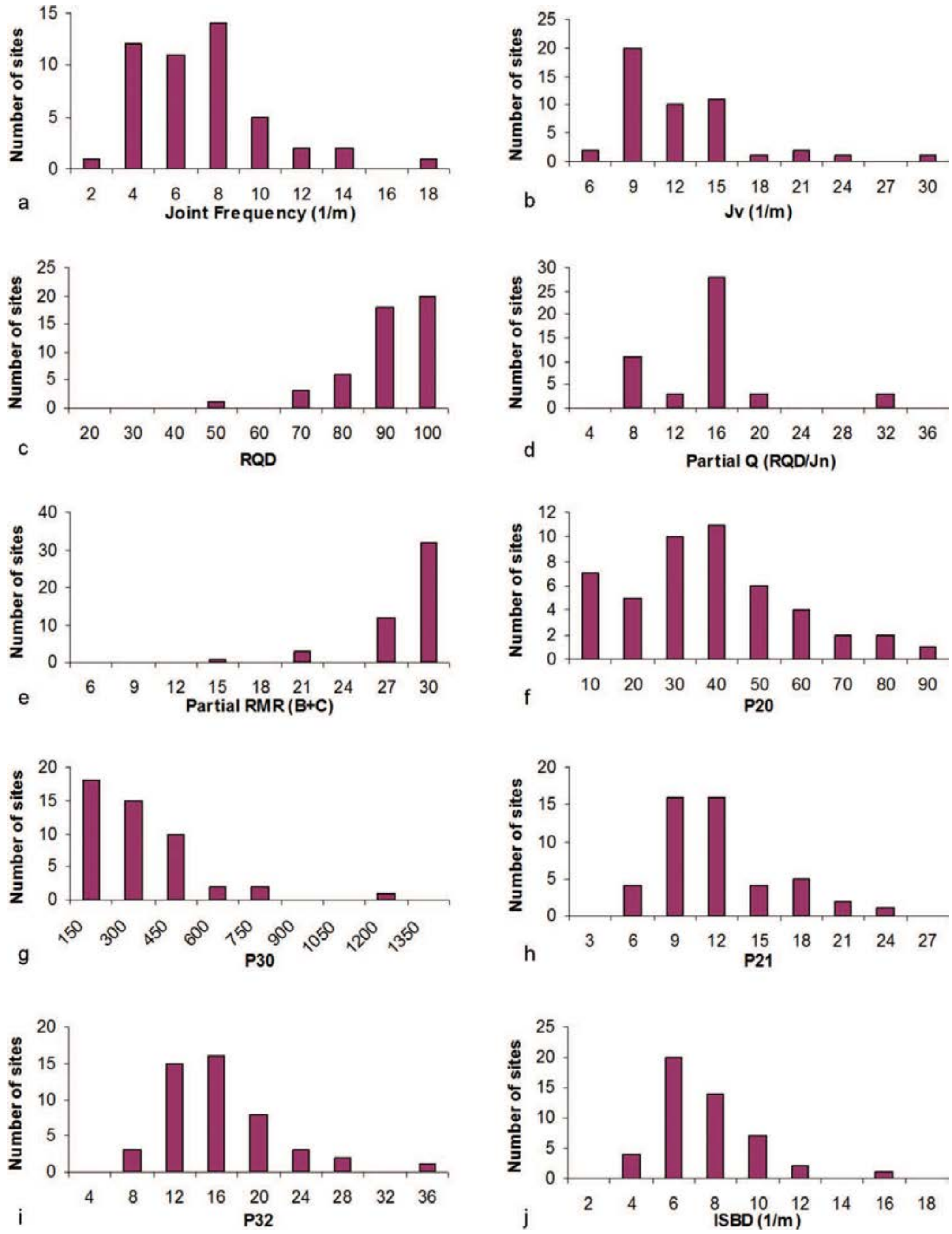

Figure 1. Rock mass characterisation results for various indexes.

Referring to Figure 1a, the reported joint frequency varies from 2.0 to 18 joints per metre. This suggests a close to moderate joint spacing, as defined by Brady and Brown [31]. It is recognised that frequency measurement is strongly influenced by sampling orientation.

The volumetric joint count was determined according to the recommendations of the ISRM [32]. The results shown in Figure 1b display a range of 5.4–29 (1/m). The summation of normal frequencies for non-orthogonal joint sets does not provide a reasonable estimate of the number of joint centres per unit volume. Nevertheless, it provides a useful way to quantify the discontinuous nature of a rock mass by correcting orientation bias resulting from the sampling technique.

Figure 1c summarises RQD values. As the reported values vary from 49 to 98 it is clear that the field data only represents 50% of the possible RQD range. The sensitivity of RQD values is limited when the rock mass is moderately fractured. One has to keep in mind that RQD values are a function of the total fracture frequency, which is highly sensitive to sampling line orientation.

The geometric term of the Q system (RQD/J_n) was evaluated and reported in Figure 1d. RQD was computed as described above and the number of joint sets was determined based on stereonets analysis. RQD/J_n varies from 5 to 32. Barton et al. [28] has argued that RQD/J_n is an indicator of block size in centimetre. Despite claims that the non-dimensional ratio (RQD/J_n) is an indication of block size, this is not obvious from the obtained results.

Within the rock mass rating (RMR) system, the discontinuous nature of the rock mass is considered through RQD (parameter B) and the minimal discontinuity set spacing (parameter C). Referring to Figure 1e, the summation of B and C varies from 13 to 30, with 40 of the 43 values between 27 and 30. This represents only 7.5% of the total possible spectrum and would suggest that the structural component of RMR is not sensitive to the natural variations in this database.

The FSM code was used to estimate the remaining properties, namely the number of discontinuities per unit area (P20), number of discontinuities per unit volume (P_{30}), discontinuity trace length summation per unit area (P21), discontinuity area summation per unit volume (P_{32}) and finally ISBD. One can notice that the majority of the histograms (Figure 1f–j) defining these properties are somewhat similar in shape.

All the results showed in Figure 1 take into account the presence of random joints that are not included in a particular joint set. In these 43 case studies, random fractures represent on average 36% of the total number of discontinuities. Neglecting them would be ignoring a valuable source of information. Furthermore, it would lead to erroneous quantification of the discontinuous nature of the rock mass, since they represent more than one-third of the fracture population.

3.1.3 Discussion

Forty-three regions were characterised using various analysis methods. The size and nature of this database is not adequate to establish the superiority of any particular classification scheme. Of interest, was how to quantify variations in the determined geometrical properties of the fracture systems. This section explores the existence of statistical trends between various geomechanical parameters. This was done by using basic regression analysis whereby the coefficient of correlation was used.

Table 2 presents the correlations between the standard methods of quantifying the discontinuous nature of the rock mass. The results show that RQD/J_n (the partial evaluation of the Q system) and the partial summation of RMR cannot be correlated to any of the other evaluation schemes. It can also be argued that the two approaches do not result in similar results.

The best correlation observed in Table 2, links RQD to J_v. Although there is a relation between the parameters as observed by Palmstrøm [26], the large scatter around the regression line somewhat limits the usefulness of any perceived correlation.

Table 3 presents the relations between the properties that were evaluated with the FSM. Even if one accounts for the limitations of the employed software tool, the results indicate that there is a good linear relation between intensity measurements (P_{21}, P_{32}) and ISBD. It would appear that these three methods provide a consistent way of defining the geometrical nature of the structural regime.

Table 2. Relations between the standard structural methods.

	Frequency	RQD	Q	RMR	J_v
Frequency	–	–	–	–	–
RQD	1.00	–	–	–	–
Q	0.03	0.04	–	–	–
RMR	0.39	0.45	0.18	–	–
J_v	0.67	0.70	0.14	0.67	–

Table 3. Relations between the properties that were evaluated with FSM.

	ISBD	P_{30}	P_{32}	P_{20}	P_{21}
IBSD	–	–	–	–	–
P_{30}	0.41	–	–	–	–
P_{32}	0.95	0.47	–	–	–
P_{20}	0.68	0.71	0.72	–	–
P_{21}	0.95	0.44	0.97	0.71	–

Further analysis has shown that the results obtained for the volumetric joint count (J_v) show a well-correlated trend with the results obtained for P21 and P32 intensities and ISBD (Table 4). It should be noted that there is a poor correlation between Jv and P30 although they are supposed to quantify the same parameter.

The four parameters behaving similarly in Table 4 account for orientation biases. This was not the case for all methods summarised in Tables 2 and 3. Total trace length of joint per area (P_{21}) is susceptible to orientation bias, however since the present analysis considered multiple sampling planes orientations this was minimised. An interesting observation is that the correction for orientation bias is a dominant factor in insuring a similar representation of the rock mass. The results show that FSM enables an unbiased means of assessing the structural properties of a rock mass by fully considering fracture orientation and intensity. This work has clearly demonstrated the limitations of traditional rock mass characterisation schemes, in that they fail to provide consistent results.

Table 4. Relations between J_v and other parameters.

	ISBD	P_{32}	P_{21}	P_{30}	J_v
ISBD	–	–	–	–	–
P_{32}	0.95	–	–	–	–
P_{21}	0.95	0.97	–	–	–
P_{30}	0.41	0.47	0.44	–	–
J_v	0.86	0.91	0.82	0.49	–

3.2 Characterising REV properties of a rock exposure

A representative elemental volume (REV) is the volume for which the size of the tested sample contains a sufficient number of the inhomogeneities for the 'average' value to be reasonably consistent with repeated testing [33]. Defining representative properties for a fractured rock mass at the engineering scale is a difficult task. FSM is arguably a useful approach that can be used to assess the REV size for structural properties of a rock mass [34]. There is an appropriate REV for different excavation or engineering structure scales. Evaluated properties at a REV scale can be used as inputs for a series of analytical and numerical tools.

3.2.1 Rock mass structural data

A data collection campaign was undertaken on a rock slope located beside the Quebec Bridge, Quebec City, Canada where slope instabilities have been a source of concerns. Scanline mapping was performed along a 25.5-m long traverse oriented at a trend of 190° and a plunge of 00°. The observed slope was oriented at a dip of 75° and a dip direction of 100°. Analysis of the structural data identified four fracture sets (Table 5).

Table 5. Structural, field data for the Quebec Bridge road cut.

Fracture set	Dip (°)/dip direction (°)	68% confidence interval cone (°)	Fracture frequency (1/m)	Mean trace length (m)
1	69/198	15	0.36	5.0
2	65/007	15	0.48	5.5
3	54/114	13	0.68	7.5
4	89/246	13	0.2	3.0

3.2.2 REV size assessment

FSM was used to generate a 50 m x 50 m x 50 m calibrated fracture system using the Fracture-SG code. The FSM was subsequently randomly sampled to obtain FSM samples of various sizes. The samples were rectangular prisms with a square base and a height to width ratio of 2. Eighteen sample sizes were selected with a square-base apex length ranging from 0.05 to 20 m. Twenty samples of every size were taken for a total of 360 samples. Figure 2 illustrates a selection of various sample sizes.

As illustrated in Figure 2, for small size samples, very few fractures were located within the control volume. Furthermore, not all fracture sets are located within a given small size sample. These limitations are overcome by the use of larger size samples.

The fracture intensity P_{32} (fracture area per unit volume) results for all 360 samples are plotted in Figure 3. The P_{32} values for all samples are represented by blue crosses, while red dots stand for the mean P_{32} value for the 18 samples sizes. A large variation typifies the P_{32} values for the smaller sample sizes. This variability decreases as the sample size increases. This is consistent with the REV concept. A formal means of assessing variability is achieved using the coefficient of variation (CV). The CV is the ratio of the standard deviation to the mean value. Using the right y-axis of Figure 3, CV for all sample sizes is plotted using a green square. The CV values are decreasing from 1.8 to 0.035 for samples size ranging from 0.05 to 20 m thus quantifying that the variability is decreasing with sample size.

It is common to choose an 'acceptable variation' to identify when the variability between samples are within an acceptable limited range. In this study, the acceptable variations for CV were selected to be between 0.20 and 0.10. Reviewing the P_{32} results in reference to the selected CV criteria suggests that the structural REV is reached at a sample size of 3 m x 3 m x 6.0 m for CV threshold of 0.20, and 10.0 m x 10.0 m x 20.0 m for CV threshold of 0.10. The corresponding P_{32} mean and standard deviation values are 4.36/0.827 and 4.48/0.389.

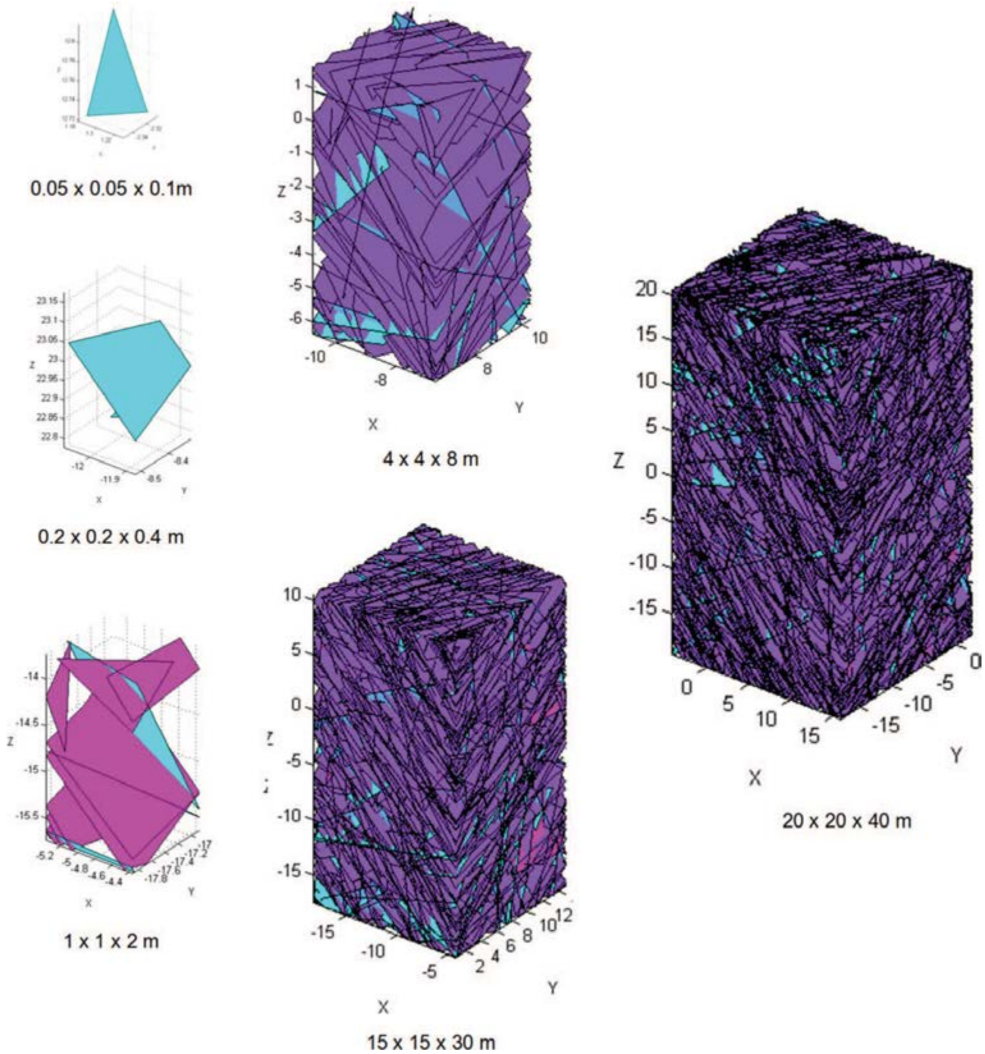

Figure 2. Selection of fracture system samples extracted from the initial FSM. (Not to scale.)

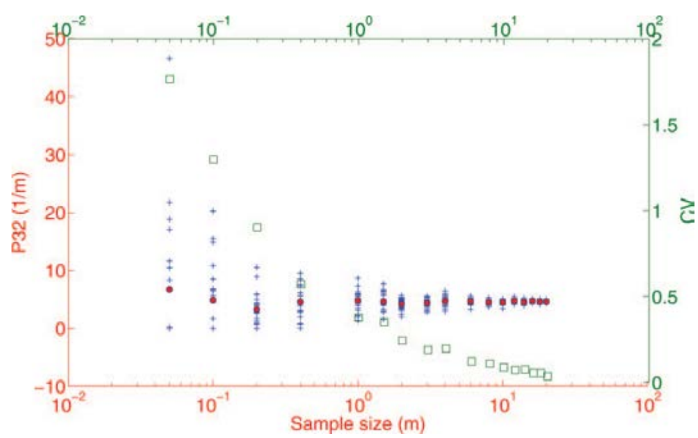

Figure 3. P_{32} and CV vs. sample size for various rock samples sizes.

3.2.3 Discussion

REV is very difficult to establish using standard rock mechanics methods. This section presented a formal approach in defining structural REV using FSM. Having defined such REV properties for the rock mass, one could use these properties to represent the jointed rock mass with adequate continuous properties representative of the field conditions.

3.3 Probabilistic structural slope stability analysis

Assessing the probability of structural failure in rock slopes is becoming increasingly popular. Commonly used probabilistic approaches are based on Monte Carlo sampling using pdfs to represent the distributive nature of selected structural parameters. These are used to assess a PF for any given instability modes. Nevertheless, the majority of current approaches do not rely on the more complete FSM representation of the structural regime and do not adequately address wedge instability size and frequency potential along a rock cut.

We have employed FSM and probabilistic Limit Equilibrium Analysis (LEA) to back analyse the stability of a rock cut that experienced two wedge failures (0.7 m^3 and 9 m^3) at the crest of a rock slope during excavation of a road cut along Highway 610 in southern Québec, Canada (Figure 4). The back analysis compared the modelled instability size, frequency and PF with field observations.

Figure 4. Partial view of the Fleurimont road cut.

3.3.1 Rock mass structural data

The results of scanline mapping are presented in Table 6 [35]. The rock mass was defined by three fracture sets, and for the purposes of the analysis, fractures were assumed to be cohesionless, having an angle of friction of 30°. The rock slope was inclined at 65°/166°. At its highest point, the slope is 8 m in height and about 350-m long. The distance between the rock slope and the highway shoulder was approximately 12 m.

Table 6. Fracture set data for the Fleurimont road cut.

Fracture characteristics	Set #1	Set #2	Set #3
Dip (°)	72	87	35
Dip direction (°)	130	226	318
K	33	16	36
Spacing (m)	1.8	3.5	17.5
Trace length (m)	2.3	4.1	1.8

3.3.2 Probabilistic rock cut stability analysis

Based on the collected field data, the Fracture-SG code was used to create 200 possible FSM. All generated fracture systems were 100 m x 40 m x 40 m in size. The integrated wedge-stability-FSM approach developed by Grenon and Hadjigeorgiou [36] was used for this case study. This approach permits the introduction of a slope configuration in the fracture system and can be used to identify all kinematically possible wedges by their location, size, etc. It is then possible to determine the stability of every wedge formed at the crest of a slope using limit equilibrium techniques. The analysis was facilitated by the Fracture-SL code [37].

The inherent variability of structural properties can be accounted for by selecting the appropriate pdf. In this study, joint geometrical parameters were defined by pdf, while material strength parameters were considered deterministic values.

The next step in this investigation involved the random introduction of 10 slope localisations in every one of the 200 generated fracture systems. Along the crest of these geometrically identical slopes, more than 6000 wedges were created. Figure 5a presents the histogram for these created wedges. As the model accounts for orientation variability, it is possible that wedges can be formed by fractures of the same set (1-1 and 2-2). These wedges are characterised by an elongated shape and only a very small percentage of these wedges are unstable, as shown in the histogram in Figure 5a. On the other hand, the probability of sliding associated with fracture sets 1–2 is high (70%).

A probabilistic assessment of wedge size is of critical importance from an engineering perspective. It can be determined using a scatter plot of wedge size versus fracture set combinations (Figure 5b). This graph reveals that the wedge volume is generally small and that the largest unstable wedges are created by intersection of sets 1 and 2. Several wedges of 1 m^3 may form along the crest of the slope as a result of the combination of sets 1–2, but only few wedges have a volume greater than 3 m^3.

A more quantitative approach of predicting size of wedges along the crest of a slope is provided in Figure 5c. It is noted that 90% of generated wedges will be smaller than 0.1 m^3, 95% smaller than 0.20 m^3 and 99% smaller than 0.75 m^3. Grenon and Hadjigeorgiou [37] have demonstrated that this type of chart can be used to develop reliability criteria for specific rock engineering applications.

3.3.3 Discussion

The starting point for this investigation was the two large wedges identified along the 350 m of slope crest. Both wedges were formed by the intersection of set 1 and 2. The first wedge had a volume of 0.7 m^3 while the second had a volume of 9.2 m^3. The wedge analysis, based on FSM provided valuable information on the failure potential of wedges formed along the slope crest and on the size of these wedges.

Based on these results one may conclude that the two instabilities observed in the field were a very improbable event, although possible. A combined FSM–LEA analysis provides a unique means of assessing the probability of occurrence and PF of rock wedges along a rock slope which is not possible using the more common probabilistic LEA analysis.

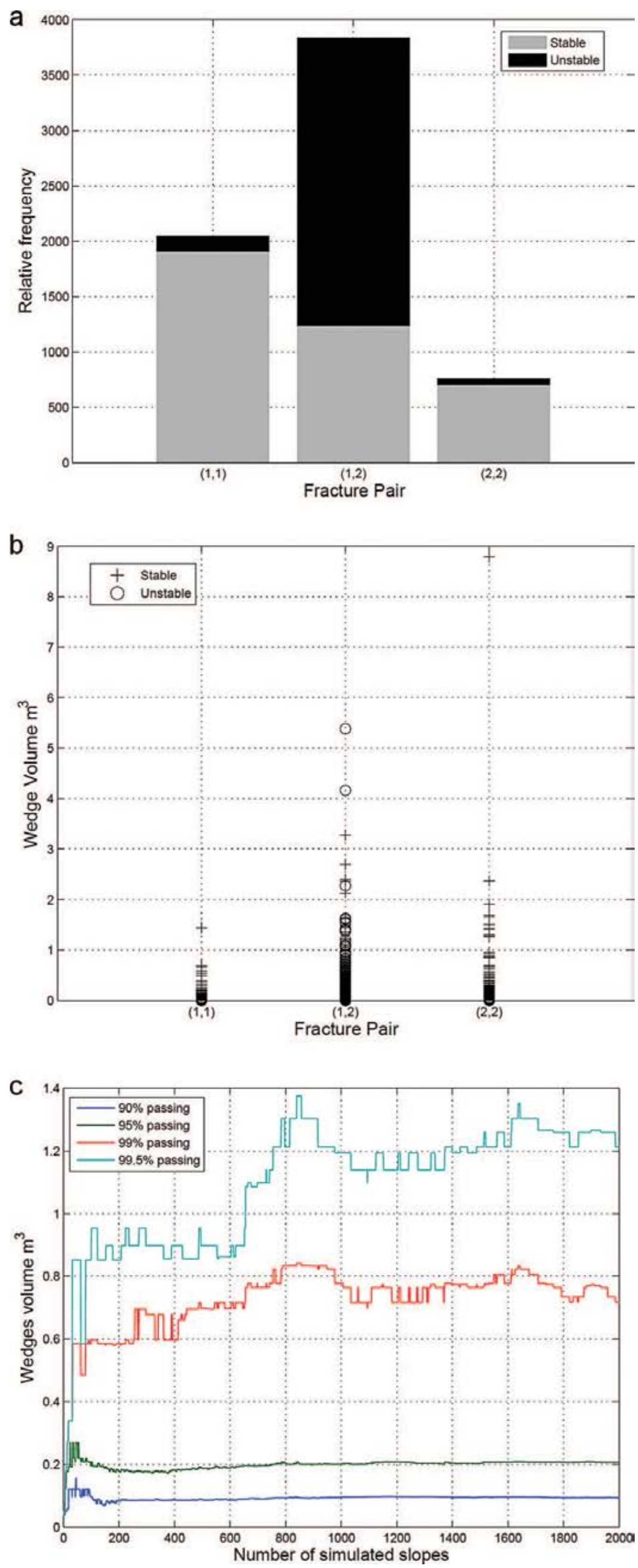

Figure 5. Slope stability. (a) Histogram of the wedges formed at crest, (b) Scatter plot of wedges volumes at crest and (c) Wedge size cumulative distribution.

4 CONCLUSIONS

This article has reviewed the evolution of FSM as an engineering tool for the analysis of structures in jointed rock. A main focus of this article is the algorithms developed and implemented in a working tool, Fracture-SG in response to past limitations in generating realistic fracture systems. The success of a FSM is dependent on the availability of quality field data and the use of appropriate 3D representations in a FSM generator. Finally, this article justifies the need for collecting comprehensive structural data, by demonstrating how such information can be used in the analysis of excavations in rock and the increased benefits from such an approach as compared to conventional methods.

The three presented case studies illustrate the potential and flexibility of the approach. The first case study provided a direct comparison of the use of fracture systems and other empirical rock characterisation tools. The study was based on field data from several underground mines and was interesting, not only for demonstrating the potential of different methods but also for identifying the inconsistencies in results obtained by different empirical methods.

The second case addressed the well-known problem of establishing an appropriate REV. Data collected along a rock slope were used to determine the structural REV for that location. Properties evaluated at REV can be used as input parameters, when using continuum approach based design.

The last example tackled a comprehensive engineering problem. The FSM approach was used in connection with rock slope stability analysis to address important issues, such as not only the number of unstable wedges, but also wedge size and reliability criteria.

These examples demonstrated the benefits that can be achieved from the use of FSM at every stage of the design process in mining and civil rock engineering design. The application of FSM in liaison with analytical tools, that can integrate the structural complexity of jointed rock, can maximise the use of available structural data and improve the design process.

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