STUDY OF SCALE EFFECTS OF ROCK QUALITY DESIGNATION (RQD) MEASUREMENTS USING A DISCRETE FRACTURE NETWORK APPROACH

by

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Abstract

RQD measurements are performed on the assumption that the drilling cores of the rock mass would be representative of in situ geological conditions. This thesis focuses on the use of Discrete Fracture Network (DFN) modelling to study the influence of core length on RQD measurements for synthetic “homogeneous” rock masses. An homogeneous rock mass is considered to have a measurable global volumetric intensity and representing a single geotechnical domain, without the occurrence of shear zones, fault zones and closely spaced weakness planes. For a given fracture intensity, the results show that the variability of RQD measurements decreases with increasing core length size, which is consistent with the concept of Representative Elementary Volume (REV). Furthermore, an attempt is made to demonstrate the link between DFN based fracture intensity indicators (i.e. Linear Intensity, P_{10} and Volumetric Joint Count, P_{30}) and RQD measurements. The analysis is repeated using field data collected at two different room-and-pillar mines, and the results further demonstrate the existence of a Representative Elementary Length (REL) for RQD measurements, analogue to the concept of REV. In this research, the REL of geometrical property P_{21}, which is the length of fracture traces per unit area of sampling plane, is compared to that of RQD. Using an implicit block search algorithm, the blockiness character of the synthetic rock masses is also studied with given fracture intensities used to measure RQD values.
Lay Summary

RQD (Rock Quality Designation) is a parameter that describes rock mass quality in terms of percentage recovery of core pieces greater than 10 centimeters. The RQD represents a basic element of several classification systems. In this paper, scale effects for RQD measurements are studied using synthetic rock masses generated by numerical models. RQD measurements are performed for rock masses with varying fracture intensities and by changing the orientation of the simulated boreholes to account for orientation bias. The objective is to demonstrate the existence of a Representative Elementary Length (REL, 1D analogue of a 3D Representative Elementary Volume, or REV) above which RQD measurements would represent an average indicator of rock mass quality. For the synthetic rock masses, calculation of RQD measurements were also performed using empirical relationships and compared to the simulated RQD measurements along the boreholes.
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List of Principal Symbols and Abbreviations

DFN: Discrete Fracture Network.

E: elastic modulus.

FSM: Fracture System Model.

GSI: Geological Strength Index.

\( J_n, J_r, J_a, J_w \) and SRF: Parameters used in the Q-index classification system.

\( J_v \): Volumetric Joint Count (i.e. \( P_{30} \)).

\( P_{10} \): FracMan notation for number of fractures per unit length of sampling borehole.

\( P_{21} \): FracMan notation for length of fracture traces per unit area of sampling plane.

\( P_{30} \): FracMan notation for number of fractures per unit volume of rockmass.

\( P_{32} \): FracMan notation for area of fractures per unit volume of rockmass.

\( Q \)-index: Q index from the Barton et al. (1974) rock mass classification.

REV: Representative Element Volume.

REL: Representative Element Length.

RMR: Rock mass rating index.

RQD: Rock Quality Designation.

\( t \): RQD threshold value.

UCS: uniaxial compressive strength.

\( \lambda \): Linear Discontinuity Frequency (i.e. \( P_{10} \)).
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1. Introduction

1.1 Background and Problem Statement

The process of characterising the rock mass is a crucial aspect in rock engineering, as design, operation, and safety assessments are all influenced by the character of the rock mass, interpreted in terms of fracture orientation, fracture size, fracture length and fracture terminations. Rock mass characteristics process is illustrated in Figure 1.1.

Figure 1.1. Rock mass characterization process (Elmo 2017, modified from Palmstrom et al., 2000).

It would be convenient if the rock mass could be described using standard descriptors and quantitative parameters associated to those descriptors, as engineers feel more confident with numbers (Hoek, 1999). In this context, rock mass classification systems represent an attempt to “put numbers to geology”. For instance, the Rock Quality Designation (RQD) (Deere, 1964 and
Deere et al., 1967) is one of the most frequently used classification systems in rock engineering, and it forms an important component of the parameters in other industry standard classification systems such as rock mass rating system (RMR; Bieniawski 1989), Q-index (Barton et al., 1974) and the geological strength index (GSI) (Hoek et al., 1995; 2013).

RQD is defined as the percentage of the sum of all intact core pieces greater than 10 cm across the drilled core length. The success of the RQD is largely due to its simple definition; however, there are limitations that need to be considered. The reliability of RQD may decrease if proper consideration is not given to geological conditions, and RQD only represents a small part of the overall rock mass evaluation process (Deere, 1989). The same author recommends using core interval no longer than 1.5m to measure RQD, arguing that this approach is better at identifying weak zones. One of the objectives of this research is to explore the limitation of recommending such a short sampling interval, which may not necessarily take into account scale effects (Figure 1.2).

The concept of scale effects is strictly related to Representative Elementary Volume (REV), and the reduction of the range of a given rock mass properties with increasing volume up to the REV. The size of the REV is not unique, and different REVs may be applied to different locations for the same rock mass; likewise, the size of the REV may also vary depending on which property is being considered. The concept of the REV is crucial to the understanding of fractured rock masses and in prediction of effective properties (Xia et al., 2016).
Since small-scale laboratory experiments are not always representative of naturally fractured rock masses, and large-scale in situ experiments are not practical, numerical modeling provides an alternative method to study fractured rock masses and determine how given properties may change up to the REV.

Figure 1.2. Scale effects and rock mass classification (modified from Elmo and Stead, 2016).

Numerical simulation tools also offer the advantage to generate realistic synthetic rock masses representative of in situ conditions. In this context, Discrete Fracture Network models (DFN models) can be used to produce synthetic fracture networks that serve as explicit representations of fractures. DFN models could be generated independently for both the 2D or 3D case, or alternatively 2D DFN models could be extracted as sections from 3D models. Larger structures (e.g. faults and fractures longer than 100 m) could be defined explicitly within the synthetic DFN fabric. It is important to note that DFN models are stochastic models, therefore an infinite number
of different, but equiprobable fracture networks could be generated based on a given statistical data set. Therefore, any rock mass characterisation process centered on the use DFN models should consider multiple DFN realisations.

1.2 Research Objectives and Scope

The primary goal of this research is to study the scale effects of RQD measurements using a DFN approach, and to determine whether it is possible to define a REL, or representative elementary length (1D analogue of REV) to be used as the characterising interval length for RQD measurements. A secondary objective is to investigate the relationship between fracture intensity indicators (i.e. linear, and areal fracture frequency, and Volumetric Joint Count) and RQD.

In this work, the term fracture and discontinuity are use used interchangeably to refer to any naturally induced planar structures (i.e. joints, shears, faults, bedding, foliation and etc.) within a rock mass regardless of their origin. DFN models can serve as useful tools for evaluating fracture systems based on known parameters. As such, both conceptual and field data based models could be generated, and RQD measurements of synthetic rock mass in the DFN models can be compared to the in-situ conditions with available fracture depth data. The calculated RQD values for different measuring intervals and their resulting differences would provide useful parameters for geotechnical engineering design and help to define the discretization size to be used in large scale continuum and discontinuum based stress analysis.

The code FracMan (Dershowitz et al., 1998; Golder Associates, 2016) is the platform used in the current paper for DFN data synthesis. Whereas specific FracMan tools are applied to generate the
DFN models, the results are independent of the DFN platform being used.

1.3 Thesis Organization

An overview of the thesis chapters is given in Table 1.1 and graphically in Figure 1.3.

Table 1.1. Thesis structure

<table>
<thead>
<tr>
<th>Chapter</th>
<th>Title</th>
<th>Summary</th>
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<tr>
<td>1</td>
<td>Introduction</td>
<td>Problem statement and research objectives.</td>
</tr>
<tr>
<td>2</td>
<td>Literature Review</td>
<td>A literature review on RQD and RQD relevant rock mass characterization systems, REV and DFN methodology.</td>
</tr>
<tr>
<td>3</td>
<td>A DFN Approach to Analyze Scale Effect in RQD Measurement</td>
<td>An explanation of the methodology used in the research and descriptions of the model geometry, variables, and procedures that define models.</td>
</tr>
<tr>
<td>4</td>
<td>Result and Analysis</td>
<td>The results and analysis of the modeling (both conceptual models and the case studies) performed.</td>
</tr>
<tr>
<td>5</td>
<td>Conclusion and Recommendation</td>
<td>The thesis conclusions with recommendations for further developments for the future work.</td>
</tr>
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Figure 1.3. Thesis Structure and Main Elements.
2 Literature Review

Engineers feel more confident with numbers rather than with a qualitative or semi-quantitative description of geology. In this context, rock mass classification systems provide a unified approach to rock mass characterisation that allows for a better quantitative description of rock mass parameters. This Chapter reviews what in the author’s opinion are the most widely used rock mass classification systems; the most relevant updates, modifications, and advantages and disadvantages of those systems are also discussed.

2.1 Rock Quality Designation

The Rock Quality Designation (RQD) was introduced by Deere in 1964 to address the lack of rock mass quality descriptors for geotechnical analysis, as at that time most of the information available was in the form of geological descriptions and percent of core recovery. Note that RQD was initially introduced to assess rock mass quality in relation to tunneling projects; later the use of RQD was extended to include relationships with rock mass mechanical parameters and used as input for other rock mass classification systems (See Section 2.2).

RQD is defined as percentage of the sum of all intact core pieces greater than 10 cm across the drilled core length, Figure 2.1. Based on the value obtained, rock mass quality is then assessed according to the ranges given in Table 2.1.
Figure 2.1. Schematic representation of RQD measurement and calculation for a core run (Deere and Deere, 1998).

Table 2.1. Rock Quality Designation RQD (modified from Barton et al., 1974).

<table>
<thead>
<tr>
<th>Condition</th>
<th>RQD</th>
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<tbody>
<tr>
<td>A. Very Poor</td>
<td>0 - 25</td>
</tr>
<tr>
<td>B. Poor</td>
<td>25 - 75</td>
</tr>
<tr>
<td>C. Fair</td>
<td>50 - 75</td>
</tr>
<tr>
<td>D. Good</td>
<td>75 - 90</td>
</tr>
<tr>
<td>E. Excellent</td>
<td>90 - 100</td>
</tr>
</tbody>
</table>

Notes:

(i) Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10
(ii) RQD intervals of 5, i.e., 100, 95, 90 etc. are sufficiently accurate
RQD is strongly influenced by the relative orientation of the borehole (or scanline) with respect to the orientation of the fractures, and the orientation bias is particularly enhanced in anisotropic rock masses (e.g. Grenon and Hadjigeorgiou, 2003, Palmstrom, 1982 and 2005). For example, let assume a rock mass made of a collection of uniform bricks, Figure 2.2. In one direction, the dimension of the bricks is slightly longer than the RQD threshold value (10 cm), while a slightly shorter dimension (less than 10 cm) is defined in a direction normal to the first one. For the rock mass under consideration, the RQD would be either 0% or 100% depending on the sampling orientation.

![Figure 2.2. The “brick” model showing RQD as 0 or 100 depending on sample direction (Palmstrom, 2005).](image)

When there is a predominant joint set, an ideal borehole orientation should be set normal to it, as it would cross the joints such that the distance between them would coincide with the true spacing. Intersection at a steep angle can lead to a higher RQD by missing or only crossing the major jointing once or twice. Practically, an intersection of no greater than 45 degrees to 55 degrees is often attempted (Deere, 1989). In 2004, Choi and Park studied the RQD dependence on the orientation of measurement with site investigations for the tunneling in Korea. According to a series of practical examinations using in situ data, for tunneling sites where the designs were based
on the RQD results obtained from vertical drill core logging, the variation of RQD with sub-horizonal scanline orientation was observed to be as high as 24%.

In practice, the orientation bias could be avoided by using multiple oriented boreholes based on some prior knowledge of the in-situ jointing conditions. Note that both vertical and inclined boreholes are used in this study to account for the orientation bias when measuring RQD for the synthetic rock masses generated using DFN models.

Figure 2.3 illustrates another bias associated with the assumed threshold length for RQD (Palmstrom, 2005). Hypothetically, a rock mass with a uniform fracture spacing of 9 cm would yield a RQD of 0%, while a rock mass with a uniform spacing of 11 cm would yield a RQD of 100%. However, the difference in fracture frequency would be much less significant (11.1 vs. 9.1 for the rock mass with spacing of 9 cm and 11 cm respectively). The results clearly show the sensitivity of RQD measurements and the need to assess rock mass quality in a more comprehensive manner and relating RQD to fracture frequency and fracture spacing.

Figure 2.3. Measured RQD for different fracture intensity values (Palmstrom, 2005).
The example above (Figure 2.3) is clearly hypothetical but it shows the limitations of arbitrarily using a cut-off threshold value. In reality, fractures would be distributed in a rock mass such that enough variation would occur along a given sampling line. With the exception of specific cases (e.g. sedimentary structures), fracture spacing typically yields a negative exponential distribution, which indicates a uniform Poisson distributing of fracture centres within a given rock mass volume (e.g. Elmo, 2006 and Palleske, 2014).

RQD does not account for the length of the discontinuities, and it becomes insensitive when the rock mass is moderately or highly fractured with frequency greater than 3 m\(^{-1}\) (Palmstrom & Broch, 2006 and Priest & Hudson, 1976). There is an arbitrary threshold value of 0.1 m found by Harrison in 1999 in homogeneous and isotropic fracture network with average fracture spacing of 5 cm. Moreover, according to the same author, rock masses within a RQD in the range of 40% to 60% are seldom encountered in engineering projects, and instead rock masses tend to have either high (>85%) or low (<10%) RQD values.

The definition of RQD relates to fracture frequency and fracture spacing (even though not all fractures are considered due to the assumed 10 cm threshold). Several methods have been proposed to correlate RQD to fracture frequency and fracture spacing. For example, Priest and Hudson (1976) proposed a relationship between RQD and linear discontinuity spacing measurements made along rock cores or rock exposures:

\[
RQ = 100 e^{-t \lambda} (0.1 \lambda + 1)
\]  

[1]
Where $t$ is the threshold value and $\lambda$ is the linear fracture frequency (i.e. the number of fractures per meter length). The relation given in Equation [1] is plotted in Figure 2.4.

![Figure 2.4. Variation of RQD with mean discontinuity spacing for a range of RQD threshold values $t$ (Priest and Hudson, 1976).](image)

In order to estimate fracture frequency within a reasonable level of precision, the scanline length should be at least fifty times the average fracture spacing and at least two hundred measurement values would be required to statistically define a negative exponential distribution (Priest and Hudson, 1976). Moreover, Equation [1] is not applicable to fracture spacing less than 0.3 m, and is was developed based on data collected for sedimentary rock types.

The parameter Volumetric Joint Count, $J_v$ was initially introduced by Palmstrom in 1974 as a way of assessing the number of joints within a unit volume of rock (i.e. number of joints per $m^3$):
\[ J_v = \frac{1}{S_1} + \frac{1}{S_2} + \frac{1}{S_3} + \cdots + \frac{1}{S_n} \]  \hspace{1cm} [2] 

where, \( S_1, S_2, S_3 \) and etc., are the average spacing for the joint sets in a given rock mass.

In 1982, Palmstron added an additional parameter to incorporate random joints, therefore Equation [2] was modified to:

\[ J_v = \frac{1}{S_1} + \frac{1}{S_2} + \frac{1}{S_3} + \cdots + \frac{1}{S_n} + \frac{N_r}{5\sqrt{A}} \]  \hspace{1cm} [3] 

where \( N_r \) is the number of random joints within the mapped area (\( A, \text{ units of m}^2 \)). Palmstron then proposed a correlation between RQD and \( J_v \), shown in Equation 4 and 5, including the original definition from the 1982 paper, and the revised definition given by Palmstron in 2005.

\[ RQD = 115 - 3.3J_v \text{ for } J_v \geq 4.5m^{-1} \]  \hspace{1cm} (Palmstron, 1982)  \hspace{1cm} [4]  
\[ RQD = 110 - 2.5J_v \]  \hspace{1cm} (Palmstron, 2005)  \hspace{1cm} [5] 

An additional method to estimate RQD is based on the mapping of rock exposures. This method consists of using a simple physical measuring rod or tape held against or in front of the rock face, and similar to core-based RQD, the length of rock segments that are greater than 10 cm are then summed along the rod or tape (Hutchinson & Diederichs, 1996). Today, this approach is often carried out on high quality face photos or Lidar scans (Hoek et al., 2013).
2.2 RQD and Rock Mass Classification Systems

RQD is often incorporated into other classification systems (e.g. RMR and Q system). A variation of the GSI classification system proposed by Hoek et al. (2013) uses RQD as a quantitative parameter to determine GSI.

2.2.1 RMR and Q System

The RMR classification system was first developed for the characterization of the rock mass by Bieniawski in 1976. Table 2.2 summarizes the evolution of the ratings assigned to the various parameters considered in the definition of RMR. Table A.1 (In appendix) provides the 1989 versions of the RMR system.

Table 2.2. Evolution of RMR Ratings (modified from Milne et al., 1998).

<table>
<thead>
<tr>
<th>Year, Max Rating (Min/Max Rating in 1989)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock Strength (UCS)</td>
</tr>
<tr>
<td>RQD</td>
</tr>
<tr>
<td>Discontinuity Spacing</td>
</tr>
<tr>
<td>Separation of joints</td>
</tr>
<tr>
<td>Continuity of joints</td>
</tr>
<tr>
<td>Ground Water</td>
</tr>
<tr>
<td>Weathering</td>
</tr>
<tr>
<td>Condition of joints</td>
</tr>
</tbody>
</table>
As shown in Table 2.2, RMR values would range from 8 to 100 in the latest 1989 version of RMR. In 1989 version, the weighting factor for both water and joint condition was increased and the influence of the spacing term was reduced. This was done in order to produce a more objective assessment of joint condition, and to facilitate a comparison between RMR and the Q-system. RMR system is easy to use, but it is relatively insensitive to minor variations in rock properties and the its conservative support recommendations fail to reflect new reinforcement tools (Milne et al., 1998).

The NGI rock tunnel quality index Q was originally developed by Barton et al. in 1974. The Q index incorporates 6 parameters and the range of possible Q values is relatively large (0.001 to 1000). Q is calculated as follows:

$$Q = \frac{RQD}{J_n} \times \frac{J_c}{J_a} \times \frac{J_w}{SRF}$$ \hspace{1cm} [6]

Where RQD is the rock quality designation, $J_n$ is the joint number factor, $J_c$ is the rating for joint roughness, $J_a$ is the rating for joint alteration, $J_w$ is rating for water and SRF is the strength reduction factor. The detailed ratings for each of the factors are shown in Appendix I.

Each of the quotients can be examined individually, and they represent separately as:

- $\frac{RQD}{J_n}$, a crude estimate of block or particle size, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400.
• \( \frac{J_r}{J_a} \), a factor for fracture strength characteristics. This quotient is weighted in favour of rough, unaltered joints in direct contact which is especially favourable to tunnel stability.

• \( \frac{J_w}{SRF} \), the in-situ stress regime. This quotient consists of two stress parameters and is a complicated empirical factor describing the 'active stress'. (Barton et al., 1974; Milne et al., 1998; Hoek, 2007; Pallesk, 2014).

Compared to other rock rating system like RMR, Q classification system is relatively sensitive to minor variations in rock quality (Milne et al., 1998) and it has been modified to extend the system to many applications, and yet it is best used as a preliminary design tool. Support design and optimization should be carried out with exposed ground conditions during construction (Bieniawski, 1997; Hoek and Brown, 1980).

### 2.2.2 Geological Strength Index (GSI) & Quantification of GSI Using RQD

The geological strength index is a system that is developed for rock engineering, engineering geology and other relevant subjects that requires rock mass characterization. It provides reliable input data in numerical analysis in designing of tunnels, slopes or foundations in rocks. This approach, treating rock mass as mechanical continuum with influence of geological variables on its mechanical properties, enables a better understanding of rock mass behavior.

In order to enhance geological logic and reduce engineering uncertainty, in this system, rock mass strength and deformability are predicted through geological characterization of rock material and visual assessment of the rock mass for the quantification of manifold aspects of rock. The GSI chart published by Hoek and Marinos (2000) is reproduced in Figure 2.5.
The original GSI chart, based on the descriptive categories of rock mass structure and discontinuity surface conditions only works well when qualified and experienced geologists or engineering geologist are available. However, there are many situations where data collection is carried out by persons who are not comfortable with these qualitative descriptions, and if GSI cannot be defined correctly, the reliability of the subsequent engineering design is questionable.

Figure 2.6 illustrates the data flow for estimating the parameters required for a numerical analysis.
of rock engineering using GSI/Hoek-Brown method.

Figure 2.6. Data entry steam for estimation of rock mass parameters using the Hoek-Brown system (Hoek et al., 2013).

Hoek et al. in 2013 presented a quantification of the GSI chart based on two parameters of Joint Condition and RQD, and Figure 2.7 shows a chart where value of GSI is given in Equation 7.

\[ GSI = 1.5 \ \text{JCond}_{89} + \frac{RQD}{2}. \]  [7]
Figure 2.7. Quantification of GSI by Joint Condition and RQD (Hoek et al., 2013).

Users without a geological or an engineering background can quantitatively enter known geological conditions into the characterization table for GSI.

2.3 RQD and Fragmentation

Knowledge about the characterization of fractured rock mass is one of the crucial issues in rock engineering, as either design, operation, or safety assessments is influenced by fragmentation, but RQD can be used only in a limited range of fractured rock masses. The disadvantage of using RQD in poor as well as in good quality rocks is illustrated in Figure 2.8. RQD of both examples on the top row are 0, but they would be obviously treated differently when considering about mining and
ground support. Similarly, the two rock masses in the bottom row both have RQD=100%, but the caveability and resultant fragmentation are significantly different.

Figure 2.8. Problems with RQD and classification systems that include RQD (modified from Jakubec, 2013).

The study of fragmentation and the estimation of likely block size and shape distribution of rock mass is crucial mass mining methods such as block and panel cave mining. According to Laubscher (2000) and Brown (2003), there are many design and operating parameters influenced by fragmentation like drawpoint size and spacing, equipment selection, comminution processes and costs, and etc. In practice, nearly all increased costs and compromise on cave management protocols are caused by the oversize rocks and hang-ups resulted from poor fragmentation which requires secondary breaking of machinery or blasting in operation.

Since fragmentation determine the overall success and profitability of the mining project and fragmentation predictions are often inconsistent with those encountered during caving it is
important to develop a improved fragmentation model that provides the reliable estimation of fragmentation for use in mine planning and give a measure of the size range and distribution of the rock blocks.

The use of DFN models has emerged as a promising new tool for fragmentation assessment. There have been several programs developed with different degrees of success. Elmo et al. (2008) using DFN models established a relationship between fragmentation characterization and in 2016, Liu has demonstrated that DFN modeling clearly has application in the estimation of fragmentation in synthetic rock masses (Liu, 2016). This thesis further add to the literature of DFN modelling and fragmentation analysis by quantify the blockiness of synthetic fractured rock masses and relating that to the measured RQD at various REVs.

2.4 Discrete Fracture Network (DFN)
While less accurate empirical rock mass characterization tools failing to give adequate evaluation, constitution of powerful tools in assessment of fragmentation becomes crucial in the determination of the caving parameters. With the increasingly more sophisticated and strict requirements of geotechnical analysis and rock engineering design, the use of Discrete Fracture Network (DFN) models with the potential to improve the predictions achieved by various geomechanical simulations becomes the mainstream.

The DFN is a stochastic method of fracture simulation that extrapolate, modify and expand collected equivalent rock mass statistics (see Table 2.3) to produce a synthetic fracture network. Manipulation allows modified data to produce relatively realistic representation of the rock fabric
with the explicit generation of 3D, synthetic, probabilistically simulated joint shape, size, orientation of fracture sets and termination; however, they may not be fully representative of a particular actual rock mass of complex natural heterogeneity and anisotropy.

Table 2.3. Fracture data and derived input data for a DFN model from digitally and conventionally mapped data (Staub et al., 2002).

<table>
<thead>
<tr>
<th>“Raw” Fracture Data</th>
<th>Source</th>
<th>DFN Input Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fracture orientation</td>
<td>Boreholes, outcrops, tunnels</td>
<td>Fracture sets, orientation of fractures in each set</td>
</tr>
<tr>
<td>(strike, dip)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trace Length</td>
<td>Tunnels, outcrops, lineaments</td>
<td>Size distribution</td>
</tr>
<tr>
<td>Termination</td>
<td>Tunnels, outcrops, lineaments</td>
<td>Choice of the model hierarchy of the sets</td>
</tr>
<tr>
<td>Fracture intensity</td>
<td>Boreholes, scanlines ($P_{10}$),</td>
<td>Fracture intensity ($P_{10}$ or $P_{32}$)</td>
</tr>
<tr>
<td></td>
<td>outcrops ($P_{21}$)</td>
<td></td>
</tr>
</tbody>
</table>

Fracture size directly measured from trace map should be differentiated from fracture radius used within the framework of DFN models, as trace lengths observed on tunnel walls or bench faces are not actually diameters, but cords to larger discs. In DFN modelling, fractures are represented by circular discs or polygons with n sides in two dimensional, and the fracture radius is defined as the radius of a circle of equivalent area of the synthetic polygons (Palleske, 2014).

Since its conception, the DFN method has been used and continuously developed in applications such as civil, geological and environment engineering. (Jing, 2003). The original application of
DFN is generic studies on fracture properties (Rouleau and Gale, 1987, on water effects on underground excavations in rock) and models developed up to the late 1980’s are reviewed in detail by Dershowitz and Einstein in 1988.

DFN modelling is then used in characterization of the permeability of fractured rock by Dershowitz in 1992 (other examples include Layton et al. (1992) and Watanabe and Takahashi (1995) on hot-dry-rock reservoirs), followed by the application in the oil and gas industry for the modelling of hydrocarbon reservoirs (Dershowitz et al., 1998a) and in the nuclear industry for the simulation of nuclear waste repositories.

DFN modelling is also identified as an important tool in the subject of geomechanics in rock engineering design. Examples include stability design of tunnels by Starzec and Tsang in 2002; open stope stability study by Grenon and Hadjigeorgiou in 2003; Rogers (2007) and Elmo (2008b) analyzing of fragmentation of fractured rock; stability analysis of vertical excavations in hard rock through integration of DFN system into a PFC model by Hadjigeorgiou et al. in 2008; and Tollenaar (2009) using DFNs in the analysis of rock mass fragmentation for block caving applications using orientation distributions and fracture frequencies that could be related to varying fracture spacing, persistence and dispersions.

Representativeness of the in situ conditions and assumed joint properties is the crux of a network generation. Although recent advances in the field of data capture and synthesis can allow a more accurate derivation of 3D models of naturally jointed rock masses, numerical and empirical techniques share the same limitation inherent in an infinite ubiquitous joint approach and
challenges of field data collection. These affect the fully characterization of the rock mass, and cannot be reduced by producing a more complex model as produced models can never be better than the data based on.

Empirical approaches such as rock mass characterization and classification systems are still fundamental components in many mining and rock engineering applications. In order to provide the necessary parameters for a usual classification analysis, both qualitative and quantitative data are collected as part of the rock mass characterization process. Fundamental aspects of rock mass characterization include (Elmo et al., 2014):

- Accurate geological model definition,
- Collection of geotechnical data,
- Major geological structures assessment, and
- Determination of rock mass properties.

In this study, a DFN model with specific rock mass parameters (fracture size, fracture orientation etc.) are made to assess fracture depth on the synthetic wells in the simulated rock mass. Multiple but equi-probable realisations of fragmentation were created, and RQD calculation was performed using the depth data obtained from the defined boreholes.

2.4.1 Fracture Size

Fracture size or persistence is one of the most difficult parameters to measure and one of the critical factors to establish the formation of 3D blocks or incomplete blocks in a rock mass (Rogers et al., 2007), fracture length is a critical input in DFN models and a key parameter for sensitivity studies
as it has a significant influence on block size and fracture connectivity (Rogers et al., 2006).

Since fracture size cannot be determined without dismantling the rock mass and measuring it directly, the appropriate length of the rock discontinuities and distribution type must be assumed based on the data sampled at outcrops, rock cuts or tunnel faces.

There are nine different size distributions possible in FracMan including Uniform Distribution, Exponential Distribution, normal distribution, Lognormal Distribution, Normal of Log Distribution, Power Law Distribution, Weibull Distribution, Gamma Distribution and Poisson Distribution. In this report, fracture size distribution follows a lognormal distribution, which is noted by many researchers.

In FracMan, for the un-truncated distribution, the lognormal distribution is defined by the probability density function:

\[
    f(x) = \frac{1}{\ln 10 \sigma \sqrt{2\pi}} \exp \left\{ -\frac{1}{2} \left( \frac{\log x - \bar{y}}{\sigma} \right)^2 \right\}
\]

[8]

where \( \bar{y} \) and \( \sigma \) are the mean and standard deviation in log10 space.

There are two additional parameters (\( x_-, x_+ \)) the minimum and maximum values in the truncated lognormal distribution, and the actual mean of the truncated lognormal distribution is:
\[ \tilde{x}' = \log \bar{x} + \sqrt{\frac{2}{\pi}} \log x_\sigma \frac{e^{-y_-^2} - e^{-y_+^2}}{\text{erf}(y_+ - \text{erf}(y_-))} \]  

where \( y_- = \frac{\log x_- - \log \bar{x}}{\sqrt{2} \log x_\sigma} \), \( y_+ = \frac{\log x_+ - \log \bar{x}}{\sqrt{2} \log x_\sigma} \)

2.4.2 Fracture Orientation

Fracture orientation is generally defined from scanline or trace mapping data, which is then grouped into sets represented by dip and dip direction and a distribution around the mean orientation. Fracture orientation is determined using stereonet analysis, which is relatively straightforward, and yet fracture orientation is subject to bias due to the mapping in borehole, by scanline, or at outcrop where there are inherent sources of error. Well organized and defined data can be easily fitted to some known statistical distribution forms including Fisher, Bivariate Fisher and Bingham distributions (Dershowitz and Einstein, 1988). The most commonly used is the Fisher distribution, which is the analog for the normal distribution in fracture data (Staub et al., 2002).

Using orientation data to determine the fracture orientation can be summarized in three argumentative parts (Cortney, 2014):

- Determination of the total number of fracture sets;
- Determination of the ownership relation between individual measurements and defined sets with the consideration of the range of dip and dip direction; and
- Determination of characteristic orientation values of defined fracture set.

Grouping of fracture data into sets can be manipulated by either manual selection or automated
selection. Manual selections of visual inspection or hand selection from weighted contour plots using DIPS allow detection of complex systems and patterns and yet it may require structural geology experience; automated selection is more consistent and less subject to bias, such as iterative clustering algorithms by Shanley and Mahtab (1976) and Mahtab and Yegulalp (1982), in which weighted pole density is examined over a stereonet.

In FracMan, orientation data can also be used without selecting sets by using the bootstrapping technique, which is not a distribution, but a technique for extrapolating and/or verifying data. For the Bootstrap distribution, the analogous relationship which exists between a sub-sample of the sampled population and the total sample population should the same as between total sample population and the true population. Artificially generated data can be used in fracture generation with bootstrapping algorithm to help better fit a given rock mass.

2.4.3 Fracture Intensity

Fracture Intensity is a measure of total fracturing in a unit length (a fracture count in a borehole), area or volume which is generally combined with size of fractures. The terminology to describe the fracture intensity is $P_{xy}$, where $x$ is the dimension of sampling region, and $y$ is the dimension of the measurement. For $X$, 1 denotes measurements taken along a line, 2 denotes measurements made on a plane, and 3 denotes measurements in a volume. For $Y$, 0 denotes point measurements, 1 denotes a measurement of length, 2 denotes an areal measurement, and 3 denotes a measure of volume. For instance, the number of fractures (0 dimension) along a borehole (the linear sampling region in 1 dimension) is denoted $P_{10}$. This is shown schematically in Table 2.4.
$P_{10}$ is the most readily input, which directionally correlates to a borehole fracture count. In FracMan, a $P_{10}$ value may be associated with one or more wells (boreholes or sample lines). Different intensity values can be applied to the intervals along the length of the well object as the wells might cross multiple geotechnical domains.

Following the same convention, the $P_{32}$ value of fracture intensity is the total area (2D) of fractures within the sampling volume (3D). Unlike $P_{10}$, $P_{32}$ is non-directional and therefore is often considered as a preferred way of describing fracture intensity. $P_{32}$ value is non-unique; as many small fractures and fewer larger ones can produce the same fracture area, same $P_{32}$ value can define different sampling biases and different block sizes. The size of fractures significantly determines fracture connectedness and rock mass behavior; thus, to better describe the topology of the fracture network both the $P_{32}$ value and the fracture size are reported.

As a non-directional descriptor, $P_{32}$ value cannot be measured directly, and it must be calculated from other measured directional intensity descriptors in which sampling biases have been removed. Common practice is to estimate $P_{32}$ values based on the available measured data using Dershowitz and Herda’s (1992) relation of proportionality correlating the fracture intensity parameters:

$$P_{32} = C_{10} \times P_{10}$$  \hspace{2cm} [11]

where $C_{10}$ is an empirical constant that depends on the fracture orientation and radius size distribution. As $P_{32}$ value is derived from directionally biased descriptor, while it is a better estimate of overall fracturing it is subject to known sampling biases.
Table 2.4 FracMan fracture intensity naming convention (modified from Elmo, 2006).

<table>
<thead>
<tr>
<th>Dimension of feature</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>P₀₀</strong> Number of fractures</td>
<td></td>
<td></td>
<td></td>
<td><strong>Point measure</strong></td>
</tr>
<tr>
<td><strong>P₁₀</strong> Number of fractures per unit length of scanline (Frequency or linear intensity)</td>
<td><strong>P₁₁</strong> Length of fracture intersections per unit length of scanline</td>
<td></td>
<td><strong>Linear measure</strong></td>
<td></td>
</tr>
<tr>
<td><strong>P₂₀</strong> Number of traces per unit area of sampling plane (Areal density)</td>
<td><strong>P₂₁</strong> Length of fracture traces per unit area of sampling plane (Areal intensity)</td>
<td><strong>P₂₂</strong> Area of fractures per unit area of sampling plane</td>
<td><strong>Areal measure</strong></td>
<td></td>
</tr>
<tr>
<td><strong>P₃₀</strong> Number of fractures per unit volume of rock mass (Volumetric density)</td>
<td><strong>P₃₁</strong> Length of fracture traces per unit area of sampling plane (Areal intensity)</td>
<td><strong>P₃₂</strong> Area of fractures per unit volume of rock mass (Volumetric intensity)</td>
<td><strong>Volumetric measure</strong></td>
<td></td>
</tr>
<tr>
<td><strong>P₃₃</strong> Volume of fractures per unit volume of rock mass</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
2.4.4 Fracture Termination

In FracMan fracture, there are two types of termination style (Figure 2.9), and termination quantifies the connectivity between fractures by referring to the percentage of a fracture set that terminates on other fractures. The termination is specified by the termination probability, which is the probability that a fracture will terminate given an intersection with another fracture.

![Termination Styles](image)

Figure 2.9. Type of termination styles recognised in FracMan (Elmo, 2006).

When newly generated fracture intersects pre-existing fractures, this form of termination disturbs the distribution of fracture size, as FracMan determines if the fracture should be discarded from type X to T by checking the termination percentage. In conceptual models, all fractures were assumed to have a termination of 0%.

Joint termination is related to and can be expressed by the characteristic shape, planarity, size and to some extent by location and orientation (Dershowitz and Einstein, 1988). Fracture termination also determines the potential of block formation of given rock mass (Tollenaar, 2008).

2.5 Inherent Sampling Bias in Field Data Collections

Inherent bias in empirical field data collection techniques impact an accurate representation of
joints condition (fracture size, orientation and intensity). According to Elmo (2006), factors including phasing of mining operations, lighting, ventilation, access to mappable surfaces and supervision aspects of Health and Safety, all have an effect on the ultimate data capture. In 1998, Zhang and Einstein classified these inherent statistical sampling biases into four different types: orientation bias, size bias, truncation bias and censoring bias.

- Orientation bias: relative orientation between the outcrop and the joint determines the probability of a joint appearing in an outcrop; Baecher (1983) devised a simple correction procedure by weighting the data in inverse proportion to their probability of appearing in the sample population.

- Size bias: preferential sampling of large joints than small joints. This bias results in two ways: a) larger joint is more than a smaller one in an outcrop; and b) in a sampling area, longer trace is more likely to be recorded than a shorter one; Size bias can be fixed with lognormal distributions, as lognormals provides an acceptable fit with running test of the Kolmogorov-Smirnov (K-S) criteria at the 5% level (Baecher, 1983).

- Truncation bias: trace lengths below some known cut-off length are not recorded as very small trace lengths are difficult or sometimes impossible to measure; compared to the problem scale, if the truncation level is small, it might be safely ignored in most cases (Baecher, 1983) and according to Jimenez Rodriguez and Sitar (2006) truncation is not significant in the formation of medium to large blocks.

- Censoring bias: Discontinuities intersecting a circular sampling window in 3 ways: a) both ends censored, b) one end censored, and c) both ends observable. Long joint traces might extend beyond the visible exposure; thus, one end or both ends of the joint traces cannot be seen. (Zhang and Einstein, 1998). In 1998, Mauldon proposed a method that overcomes the
bias and obtains the true trace length distribution by using density and mean trace length estimators (Tollenaar, 2009).

### 2.6 Representative Elementary Volume (REV)

The concept of Representative Elementary Volume (REV) is the premise of the continuous-media methodology and is of great importance in understanding of fractured rock masses. The investigation of REV is inherently related to scale effects that is a fundamental area of rock mechanics research when determining rock mass properties.

In different publications, the existence of an REV can be determined based on different physical parameters according to a variety of applications. For example, Bear (2013) defined the REV using the concept of porosity in rock mass according to applicability of the theory of porous media. Figure 2.10 presents a relation between porosity and studying volume. Porosity of selected rock mass fluctuates dramatically when rock mass volume starts to increase and the critical volume where the porosity ceases to vary is defined as the REV.
Xiang and Zhou estimated a two-dimensional REV of 9m x 9m in terms of the parameter of elastic modulus through a 2D FEM rock model based on variations of the equivalent elastic modulus with the size of the rock mass under uniaxial loading. And Zhang and Xu, based on the geometric and mechanical parameters of the fractures, determine the REV using the numerically generated rock mass with defined joint networks. A REV value of approximately three to four times the maximum the trace lengths of defined joint sets is indicated in their research.

From the view of permeability tensor, Long et al. (1982), and Wang et al. (2002) discussed the existence and the size of the REV. According to Long, as each fracture that intersects other conducting fractures can contribute to the permeability of the rock mass, the possibility of existence of REV increases with increasing fracture densities, and Wang determined the REV using a 3-D stochastic fracture network model that is based on the fracture data mapped at the site. As the hydraulic conductivities in different directions cease to change with the block size when
block sizes are greater than approximately 15m, Wang suggested a REV of 15m of the studying rock mass.

Some model including all the FEM, treat rock masses as continuums, whereas others (such as DEM) treat them as isolated blocks. Normally there are many discontinuities in the rock mass and it seems plausible to be treated as an assemblage of isolated blocks; however, as the size of blocks in the rock mass might be quite small, the total rock mass can have very good integrity, and rock mass can be considered a continuum. The concept of REV is also very important in the context of “equivalent continuum” approach to modeling fractured rock masses. When to treat a rock mass as a continuum or as an assemblage of isolated blocks is a fundamental question in rock mechanics since most rock masses lie somewhere in between. Blockiness of a fractured rock mass define state or condition of being blocky, the existence of an REV and its size might be correlated to the blockiness of fractured rock masses and therefore, study of blockiness level is carried out by Lu, Zheng and Yu in 2016.

2.6.1 Estimation of the REV Size for Blockiness of Fractured Rock masses

Blockiness is defined as the volume percentage of the isolated blocks in rock mass. According to Xia et al. (2016), the size of the REV can be calculated by investigating the fluctuation of blockiness with the model domain size. To increase the efficiency, only the number of blocks was considered when determining the existence of the REV, although the quality of rock masses is determined by both the number and the size. In the analysis, 77 models were developed based on 7 classes of persistence and 11 classes of spacing suggested in the International Society for Rock Mechanics (ISRM) fracture classification. General block method was implemented to identify the
blockiness in each of the 77 types of fractured rock mass models. The effects of randomness were reduced by 9 realizations carried out for each model and coefficient of variation was defined to quantify the variability of the 9 random realizations. When the standard deviation of the 9 realizations is very small and the fluctuation in blockiness with the variation in the scale of the model region becomes smooth, the size of the REV can be calculated. According to the modelling result, in 76 of 77 models the size of the REV varies from 2 to 20 times of the fracture spacing. Only the model with a wide fracture spacing and very high persistence has a REV size that exceeds 20 times the fracture spacing.

A comparison of this theory to one actual fractured rock mass can be made using the results from the work of Xia et al. in 2015. A blockiness of 4‰ was found in the surrounding rock mass of an underground powerhouse of Three Gorges Project. The volume of isolated blocks consists of only a small proportion of the total rock volume; therefore, the rock mass shows very good integrity and can be treated as continuum. The excavation of the underground powerhouse verified these conclusions as only a few blocks were formed. The rock mass around the underground powerhouse is of medium persistence and very wide fracture spacing, and according to the calculation results of the theoretical models the blockiness of this type of rock mass should be very low, which is consistent with the number of 4‰. Therefore, the results could work as a reference for prediction of the integrity of rock masses with different fracture persistence and spacing in certain projects.

2.6.2 Study of REV to Characterise the Strength and Deformability of Fractured Rocks
Discrete Element Method can provide generic estimations on mechanical behavior of fractured rock mass. By using code UDEC, in 2014, Bidgoli studied the anisotropy of strength and
deformability, and effects of water pressure through a set of 50 two-dimensional square-shaped DFN realizations at a REV level based on realistic geometrical and mechanical data of rock mass from field mapping at Sellafield, UK. According to Bidgoli, fractured rocks behave nonlinearly, and based on a stochastic analysis, the strength of fractured rock mass represented by Young’s modulus increases significantly with increasing confining pressure; whereas, the deformability measured by the equivalent Poisson’s ratio decreases when the confining pressure increases. These ranges of values of variations are quantified using multiple realization models of the fracture system geometry. When rotate DFN models, strength and deformability varies with same the loading conditions, indicating a direction-dependence. The DFN modeling of fluid flow show that water pressure decreases the Young’s modulus of fractured rock mass, but the strength reduction may not equal to the magnitude of water pressure; whereas, the Poisson’s ratio increases slightly when a water pressure is applied.

The equivalent strength parameters of the fractured rocks, is determined by using two popular strength failure criteria, namely Mohr-Coulomb (M-C) and Hoek-Brown (H-B), which give reasonable estimates of the compressive strength of the rock mass in almost all the cases. The results, at a REV of a set of 50 two-dimensional square-shaped DFN realizations generated with a dimension of 5m × 5m, demonstrate that strength and deformation parameters of fractured rock mass can be influenced by the confining pressures, loading directions, water pressure, and maybe other mechanical and hydraulic boundary conditions.

In 2010, Esmaieli et al. quantitatively determined the REV size of massive sulphides for the underground infrastructure at Brunswick Mine in Canada. In the case study, field data collected in
the sulphides were used to generate a representative fracture system, the Fracture System Model (FSM, 40 meters’ side length cubic, Figure 2.11).

![Visualization of the generated fracture system. North coinciding with the Y-axis (Esmaieli et al., 2010).](image)

Figure 2.11. Visualization of the generated fracture system. North coinciding with the Y-axis (Esmaieli et al., 2010).

A further sampling was applied to the FSM to procure subsequent testing specimens, and generated specimens were introduced into a 3D bonded particle flow code (PFC3D) model to create synthetic rock mass samples (with a height to width ratio of 2) as illustrated in Figure 2.12.
Both geometrical and mechanical property REV sizes were analyzed. The geometrical REV was estimated based upon the number of fractures in each sampled volume ($P_{30}$) and the volumetric fracture intensity ($P_{32}$) of the samples, and the mechanical REV was determined through sequences of numerical tests of the uniaxial compressive strength (UCS) and elastic modulus (E). The estimated geometrical REV size of the rock mass was $3.5\text{m} \times 3.5\text{m} \times 7\text{m}$, while the mechanical property REV size was $7\text{m} \times 7\text{m} \times 14\text{m}$; therefore, the larger volume ($7\text{m} \times 7\text{m} \times 14\text{m}$) is considered
as the REV size for this sulphides rock mass. The results were used for the structure stability design at the mine.

The work by Elmo et al. (2011, 2012 and 2016) has discussed the applications of the integrated FEM/DEM – DFN code ELFEN to characterize rock mass strength at different scale under different loading conditions. The analysis provides different examples of the definition of REV. According to Elmo et al. (2011), it is possible to quantitatively interpret GSI value according to the varying modelling shear strengths of different sample size (see Figure 2.13.a). Similarity, Figure 2.13.b shows strength comparison for pillar models with equivalent Hoek-Brown/GSI response at different scales and for varying joint densities/blockiness; and for each pillar model, estimations of equivalent GSI/Hoek-Brown response are given at 0MPa confinement.
Figure 2.13. a) Preliminary characterization of scale effects to establish a relationship between sample size and rock mass GSI rating (Elmo et al., 2011). b) Strength curves for pillar models with width-to-height ratio of 0.5 at different scales and equivalent GSI/Hoek-Brown response at 0 MPa confinement (Elmo et al., 2012 and Elmo et al., 2016).

The results clearly show the reduction of the strength with the studying volume increasing to the REV. For the ultimate use of synthetic rock mass properties, the balance of engineering judgment, the integration of characterized field data, and numerical modeling needs to be correctly reached (Elmo et al., 2011 and Elmo et al., 2016).
3 A DFN Approach to Analyze Scale Effects in RQD measurements

This Chapter introduces the FracMan code and the built models. Both conceptual (three orthogonal fractures sets) and real data based fracture networks were generated. Collected data were extrapolated, modified and expanded to for the specific studying purpose and absolute approximation of the actual rock mass does not exist.

3.1 FracMan Introduction

FracMan is the premier software that allows analysis and modelling of fractured rock masses. In FracMan, there are four types of fracture network construction: Geometric, Geocellular, Stratigraphic, and Trace Map. Only geometric fracture generation algorithms were used in this work. In Geometric fracture generation, fractures with a given intensity are located in the defined space according to the selected statistical distribution type.

3.1.1 Geometric Fracture Networks

Although geometric model is often similar to or more appropriate for rocks of sedimentary origin, all rock type can be represented by using Geometric fracture networks. There are three statistical distribution types:

- Enhanced Baecher: fracture centers are located randomly, with negative exponential spacing distribution; fracture size is independent of fracture location.
- Levy-Lee: creates fracture centers sequentially, with fracture size related to the distance from previously generated fractures.
- Nearest Neighbour: Fracture Intensity decreases with distance from a defined zone or feature,
appropriate for cooling margins, stress relief fractures and some fault zones (Bigdoli, 2014).

Enhanced Baecher generation models were used in this work. Fracture Orientation and Fracture Intensity are crucial for the creation of an Enhanced Baecher geometric fracture network. As the simulations are conceptual models, three near orthogonal sets of fractures with Fisher distributions, representing the most basic case were first used in simulations, and then the real data collected for two room-and-pillar mines. A $P_{10}$ value is used to define the fracture intensity. Fractures are generated with the number of fracture intersections being checked against the defined well intervals when the first interval, all intervals, or the average of all intervals, as defined by the user, reaching the specified $P_{10}$ value.

### 3.2 Characteristics of the Conceptual Model

A systematic 3D numerical modeling platform based on the DFN approach was developed to create a numerical predictive tool for studying links between fracture intensity indicators and RQD measurements at different scales.

Through a Monte Carlo simulation process, FracMan allows a DFN realization of given rock mass by choosing a range of values and different models for fracture spatial distribution, including fracture orientation and orientation distribution, fracture termination percentage, fracture radius distribution, and fracture intensity. After defining a DFN stochastic model from the assumed parameters, the numerical process starts with generation of fractured rocks to represent the complex geometry system of the given rock mass.
The following assumptions were made in the development of the conceptual model:

- Fractures were modelled as planar four-sided polygons;
- The synthetic rock mass can be represented by three joint sets and for each joint set, fracture generation follows a Fisher distribution for orientation dispersion, lognormal distribution for persistence, and the enhanced Baecher model for spatial distribution; and
- Fracture generation cease with the defined rock mass boundary.

To quantitatively predict the range of measured RQD of synthetic rock masses. Several well intervals of different length were defined to represent the measuring core length in a rock mass with established REV (30 meters’ side length). Generated fractures are filtered leaving fractures that intersect defined intervals. RQD is calculated according to Deere in 1963 using the depth data of intersections. Numerical relationship can be defined between measured RQD and different sample size.

### 3.3 Boreholes and Core Run Lengths

The model used for the simulation is a cubic with height, length and width all equal to 30m. Three orthogonal boreholes passing through the center of boxes were defined to condition the fractures in the models to a given $P_{10}$ value. Another four wells (see Figure 3.1) along the diagonal line were defined to provide alternative measurements of RQD in each model.
Core run lengths of 30m (whole length of the well) 15m, 10m, 5m and 3m were individually defined in each well from 0 depth along the well to the end, which means 10 intervals for interval length of 3m (see Figure 3.2), 6 intervals for interval length of 5m and so on. For those wells along the diagonal line, 30m in the middle of the well was selected to perform the analysis. Maximum, minimum and average RQD of different intervals length were record for each well.
Because of the stochastic nature of the DFN approach, each DFN model was generated 5 times and the average RQD (for a selected core run length) for the 5 realizations was reported.

### 3.4 Grid and Volumetric Joint Count $J_v$

In the synthetic rock mass of conceptual models, grids of cubic with height, length, and width equal 1m were uniformly defined to assess $J_v$, and the results are summarized in table 3.1.

<table>
<thead>
<tr>
<th>Fracture count</th>
<th>3</th>
<th>5</th>
<th>7</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>per unit length</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>per unit grid (min)</td>
<td>0</td>
<td>2</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>per unit grid (max)</td>
<td>32</td>
<td>45</td>
<td>63</td>
<td>83</td>
</tr>
<tr>
<td>per unit grid (average)</td>
<td>12.34289</td>
<td>21.33193</td>
<td>27.93211</td>
<td>40.74796</td>
</tr>
</tbody>
</table>
3.5 Conceptual DFN Model

In this work, three near orthogonal fracture sets are used as the conceptual models (see model parameters in Table 3.2). The fracture sets have approximate orientations of (dip/dip direction): 00°/000°, 00°090° and 90°000° (the stereonet is shown in Figure 3.3). In this thesis, wells are defined along xyz axis for the measurement of RQD, fracture sets were thus defined in a pattern to avoid the brick sampling bias mentioned in Figure 2.2. The conceptual rock mass was generated to represent a homogeneous rock mass (i.e. single geotechnical domain) with no faults or shear zones. The fractures were generated within the box region defined as rock mass (Figure 3.4).

Figure 3.3. Stereonet of well logs.
Figure 3.4. Examples of FracMan generated DFN fractures with 3 joint sets.

Table 3.2. Model parameters for conceptual model

<table>
<thead>
<tr>
<th>Fracture Data</th>
<th>DFN Input Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fracture orientation</td>
<td>Three near orthogonal fracture sets (dip/dip direction of 00°/000°, 00°090° and 90°/000° respectively). Fisher dispersion value of 80 for all sets.</td>
</tr>
<tr>
<td>Fracture Length</td>
<td>Lognormal distribution, Mean 3m, standard deviation 3m. No truncation.</td>
</tr>
<tr>
<td>Fracture Terminations</td>
<td>0%</td>
</tr>
<tr>
<td>Fracture intensity</td>
<td>Linear fracture intensity $P_{10}$ intensity of 3, 5, 7 and 10 m$^{-1}$ (for each of the three sets)</td>
</tr>
</tbody>
</table>

For each conceptual model, only one variable, $P_{10}$ was changed, leaving the other parameters fixed. $P_{10}$ value of 3, 5, 7 and 10 were used in each model for the fracture generation. Fracture Aperture, Permeability and Compressibility are not specified in this work.
3.6 DFN Model of Middleton Mine

The analysis was repeated using a DFN model generated by Elmo (2006) using data collected at Middleton mine (Derbyshire, UK). The mine is a classic square room-and-pillar mining operation with drift access working mostly under a cover of about 100m. Pillars are planned for nominal 16m x 16m dimensions in plan with rooms 14m wide. However, completed pillars are usually smaller, due to over-break. Because the rock mass quality for Middleton mine is generally good to excellent, the original DFN model was modified by increasing the mapped intensity (the same increment was applied to all sets). All other parameters were unchanged. The objective was to simulate a poor to good rock mass (45 ≤ RQD ≤ 85), this time using real field data in terms of fracture orientation, fracture length and fracture terminations. The final FracMan input parameters for the DFN model in this research (see Table 3.3) were derived from the input used in Elmo’s thesis in 2006. A 2.5-fold increase in K value of orientation distribution (decrease dispersity) and 5-fold increase in volumetric intensity were used. DFN fracture realization is given in Figure 3.5.
Table 3.3. FracMan input parameters for the Middleton DFN model (modified from Elmo, 2006).

<table>
<thead>
<tr>
<th>Set</th>
<th>Orientation Distribution</th>
<th>Shape</th>
<th>Terminations</th>
<th>Radius Distribution</th>
<th>Volumetric Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>Fisher Dip/Dip direction 89/308 K=103.75</td>
<td>6 sides polygons</td>
<td>0%</td>
<td>Lognormal</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\bar{y}$=38.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\gamma=9.0$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Min. 2m</td>
<td></td>
</tr>
<tr>
<td>1b</td>
<td>Fisher Dip/Dip direction 84/323 K=20.75</td>
<td>6 sides polygons</td>
<td>23%</td>
<td>Lognormal</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\bar{y}$=3.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\gamma=0.6$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Min. 0.25m</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Max. 2.5m</td>
<td></td>
</tr>
<tr>
<td>2a</td>
<td>Fisher Dip/Dip direction 86/219 K=43</td>
<td>6 sides polygons</td>
<td>50%</td>
<td>Lognormal</td>
<td>5.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\bar{y}$=3.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\gamma=1.2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Min. 0.25m</td>
<td></td>
</tr>
<tr>
<td>2b</td>
<td>Fisher Dip/Dip direction 89/269 K=70.5</td>
<td>6 sides polygons</td>
<td>31%</td>
<td>Lognormal</td>
<td>3.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\bar{y}$=3.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\gamma=1.2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Min. 0.25m</td>
<td></td>
</tr>
<tr>
<td>3a</td>
<td>Fisher Dip/Dip direction 46/193 K=56</td>
<td>6 sides polygons</td>
<td>29%</td>
<td>Lognormal</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\bar{y}$=3.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\gamma=1.5$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Min. 0.25m</td>
<td></td>
</tr>
<tr>
<td>3b</td>
<td>Fisher Dip/Dip direction 44/016 K=31.5</td>
<td>6 sides polygons</td>
<td>0%</td>
<td>Lognormal</td>
<td>2.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\bar{y}$=3.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\gamma=1.5$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Min. 0.25m</td>
<td></td>
</tr>
</tbody>
</table>
Figure 3.5. Example of DFN realization for Middleton Mine (modified intensity) and corresponding stereonet of the fracture sets (modified from Elmo, 2006).

3.7 Fracture Depth Data of Iron Cap Deposit

The analysis was also repeated using the depth data collected at Iron Cap Deposit, which is part of the KSM property located in the Coast Mountains of northwestern British Columbia. The deposit remains open at depth and because of various environmental concerns, Seabridge government decided to change the mining option from open pit to block caving. The deposit extends approximately 1,200 m SW-NE (along strike), 700 m NW-SE, and 700 m vertically and consists of strong, moderately fractured rock. The strength of the rock mass does not change significantly within the deposit and thus fracture frequency variation determines the rock quality. According to Golder Associates (2012), the location, dimensions, and dip of the mineralized material at Iron Cap all indicated the potential for block caving.

In this case, DFN models were not generated, as fracture depth data is available from three boreholes (IC-10-014, IC-10-015 and IC-10-016 from Golder Associates, 2012) and each of three
boreholes has a length more than 200 meters. For each borehole, RQD calculation was made using depth data obtained from boreholes, using different core run length (3m, 5m, 10m, 15m, and 30m as for the conceptual model). Average of each core run length was calculated as the final calculation results.

3.8 RQD and $P_{21}$

To better study the scale effect and testify the patterns obtained from defined core run length, other frequency indicator ($P_{21}$ in the following section) was studied in the simulations, and a relationship between RQD and $P_{21}$ is given for the conceptual models using orthogonal boreholes.

Sections were taken normal to the defined boreholes at a depth along the boreholes corresponding to the half the core run length used to calculate its RQD, and the same studying core run length lengths (3m, 5m, 10m, 15m and 30m) were used for the analysis. For instance, for defined interval of 10 m, three sections were inserted at depth of 10 m, 0 m and -10m separately (see Figure 3.6) to calculate the $P_{21}$ through computation of fracture trace intersections.
Figure 3.6. Conceptual model. Example of sections and trace maps used for the calculation of $P_{21}$ at interval length of 10 m.

3.9 Block Analysis of the Synthetic Rock Masses

With varying sizes, the blocks of the synthetic rock masses were verified using an implicit block search algorithm. An example of the blockiness visualization in terms of block size is shown in Figure 3.7.

Figure 3.7. Example visualization of defined rock mass in terms of block size (model with $P_{10}$ of 10).
4 Result and Analysis

Based on 4 classes of fracture density and 2 types of wells, 8 models were generated in this work. Using method initially proposed by Deere in 1963, RQD in each well of different measuring lengths (ranging from 3 to 30 meters) was calculated. Maximum and minimum of RQD values were recorded for the study for the study of scale effect and estimation of REL.

4.1 Representativeness

In order to verify the representativeness of the centered wells used for RQD calculation, four off-center parallel wells were defined, Figure 4.1.

![Figure 4.1: Center studying well and surrounding wells defined for the representativeness test.](image)

According to Table 4.1, the difference between the RQD calculated using center well and average of the surrounding four wells is negligible; thus, the representativeness is guaranteed.
Table 4.1. RQD of center wells and surrounding wells.

<table>
<thead>
<tr>
<th>Wells</th>
<th>Calculated RQD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P_{10}</td>
</tr>
<tr>
<td>P_{10}</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>10</td>
</tr>
</tbody>
</table>

4.2 Influence of Core Run Length on RQD Determination

For each model with pre-defined fracture density, RQD data manipulations were performed separately for three orthogonal wells and four inclined wells. Take the orthogonal wells as example,
maximum and minimum of RQD values were recorded according to different core run length and an average of extremum was calculated using all three wells of the same core run length. As illustrated by Figures 4.2 to 4.5, a merging trend of the maximum and minimum measured value with increasing core run length was depicted from all four models generated using different fracture densities.

Figure 4.2. Calculated RQD vs. core run length for orthogonal borholes with $P_{10}$ of 3.
Figure 4.3. Calculated RQD vs. core run length for orthogonal boreholes with $P_{10}$ of 5.

Figure 4.4. Calculated RQD vs. core run length for orthogonal boreholes with $P_{10}$ of 7.
Figure 4.5. Calculated RQD vs. core run length for orthogonal boreholes with $P_{10}$ of 10.

For the inclined models, a similar merging trend of the maximum and minimum measured value was also depicted from all four models with different fracture densities and the trend is illustrated by the following four Figures (4.6 to 4.9).
Figure 4.6. Calculated RQD vs. core run length for inclined boreholes with $P_{10}$ of 3.

Figure 4.7. Calculated RQD vs. core run length for inclined boreholes with $P_{10}$ of 5.
Figure 4.8. Calculated RQD vs. core run length for inclined boreholes with $P_{10}$ of 7.

Figure 4.9. Calculated RQD vs. core run length for inclined boreholes with $P_{10}$ of 10.
For simplicity, the results are summarized in Figure 4.10 and 4.11 using 3D charts, to show the relationship between the range of measured RQD value and input fracture intensity $P_{10}$.

Figure 4.10. RQD relative difference versus core run length and $P_{10}$ for Orthogonal wells.

Figure 4.11. RQD relative difference versus core run length and $P_{10}$ for Inclined wells.
The following key observations and results can be highlighted:

- For every fracture intensity, the range of measured RQD values decreases with increasing core run length, and the RQD values converge to a constant value for a 30m core run length.
- Considering that the synthetic rock mass model used in the analysis represented a very homogeneous rock mass without pre-defined shear and fault zones, for relative short core run lengths (e.g. 3m), the rock mass quality ranges from fair to excellent when $P_{10} \leq 5$ and poor to fair, $P_{10} \geq 5$.
- The relative difference (max RQD minus min RQD) increases with increasing fracture intensity (for a selected core run length).

4.3 Comparison with Empirical Formulae for the Determination of RQD

RQD can also be determined from other methods without core. Comparison among the calculated RQD using depth data and results directly accessed from Linear Intensity and Volumetric Joint Count in both 1982 and 2005 modified version is given in Table 4.2.
Table 4.2. Comparison of RQD calculated with core and without core.

<table>
<thead>
<tr>
<th>Conceptual RQD</th>
<th>Orthogonal well</th>
<th>94.14</th>
<th>85.46</th>
<th>75.93</th>
<th>62.73</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inclined wells</td>
<td>91.86</td>
<td>77.60</td>
<td>66.72</td>
<td>48.88</td>
</tr>
<tr>
<td>Simulated J_v</td>
<td></td>
<td>12.34</td>
<td>21.33</td>
<td>27.93</td>
<td>40.74</td>
</tr>
<tr>
<td>Simulated λ</td>
<td>Orthogonal boreholes (OB)</td>
<td>4</td>
<td>6.45</td>
<td>9.24</td>
<td>13.30</td>
</tr>
<tr>
<td>Empirically determined RQD</td>
<td>Inclined boreholes (IB)</td>
<td>2.74</td>
<td>5.02</td>
<td>7.05</td>
<td>10.13</td>
</tr>
<tr>
<td></td>
<td>110-2.5J_v</td>
<td>79.14</td>
<td>56.67</td>
<td>40.17</td>
<td>8.13</td>
</tr>
<tr>
<td></td>
<td>115-3.3J_v</td>
<td>71.80</td>
<td>40.34</td>
<td>17.24</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>100e^{-0.1λ} (0.1λ+1) (OB)</td>
<td>93.8</td>
<td>86.2</td>
<td>76.3</td>
<td>61.6</td>
</tr>
<tr>
<td></td>
<td>100e^{-0.1λ} (0.1λ+1) (IB)</td>
<td>96.9</td>
<td>90.9</td>
<td>84.3</td>
<td>73.1</td>
</tr>
</tbody>
</table>

Note:

i) In the DFN model, \( J_v \) represents the number of fractures intersecting a defined grid of unit volume (1\(m^3\)) and \( λ \) represents the number of fractures intersecting the unit length (1\(m\)) of the borehole. For the orthogonal boreholes, the total length of the borehole is 30m, and 52m for the inclined boreholes.

ii) The simulated RQD results refer to a core length of 30m.

iii) The simulated RQD results refer to an input \( P_{10} \) of 3, 5, 7 and 10 respectively. Note that the input \( P_{10} \) differs from the measured \( λ \) as the latter represent the number of generated fractures intersecting the borehole and not just the defined input frequency along that borehole for one of the three sets.

According to Table 4.2, the overall degree of accuracy of the RQD results acquired through data from face maps generally decrease with increased fracture density. There is a good agreement between simulated RQD values and those obtained using \( λ \), but there is great difference between the result obtained with \( J_v \) and calculated RQD.
4.4 Case Study of Middleton Mine

Real data (with modification) obtained from the square room-and-pillar mine in Middleton mines were processed in the same way as previous ideal conceptual counterpart. The results are given in the Figure 4.12 and 4.13.

Figure 4.12. Middleton mines. Calculated RQD vs. core run length for orthogonal boreholes.
The results confirm the key observations made for the conceptual model, and clearly show the existence of a Representative Elementary Length (REL) above which RQD values assume a constant value for the rock mass. Similar to the previous case, the room-and-pillar DFN model represented a single homogeneous geotechnical domain without shear or fault zones. For both models the REL is approximately 15m (independent of the orientation of the core run). However, the definition of REL would also be affected by the orientation of the core run. For example, a REL of 10m could be defined for the room-and-pillar DFN model with reference to the orthogonal boreholes.

4.5 Case Study of Iron Cap Deposit

The RQD calculation results directly obtained from the depth data from Iron Cap Deposit were
given in the Figure 4.14, 4.15 and 4.16.

Figure 4.14. Iron Cap Deposits. Calculated RQD vs. core run length for boreholes IC-10-014.

Figure 4.15. Iron Cap Deposits. Calculated RQD vs. core run length for boreholes IC-10-015.
**Despite the fact that rock mass quality has little range from good to excellent, the results also confirm the key observations made for the conceptual model, and show the existence of a Representative Elementary Length (REL) of approximately 15m for all three boreholes.**

**4.6 The Relationship between Rock Mass Scale and Assumed Thresholds**

Threshold value of 10cm is universally adopted in the assessment of RQD and one of significant limitations of the RQD definition is its dependency on the arbitrarily selected threshold length (e.g., Terzaghi, 1965; Harrison, 1999; Hack, 2002; Chen et al., 2005).

Different thresholds of RQD determination were used for the study of scale effect and REL, and comparison were made among different thresholds. For conceptual models, figures of maximum and minimum RQD values and 3D charts for simplicity are shown in Figures 4.17 to 4.36.
Figure 4.17. Calculated RQD (20cm threshold) vs. core run length for orthogonal boreholes with $P_{10}$ of 3.

Figure 4.18. Calculated RQD (20cm threshold) vs. core run length for orthogonal boreholes with $P_{10}$ of 5.
Figure 4.19. Calculated RQD (20cm threshold) vs. core run length for orthogonal boreholes with P\textsubscript{10} of 7.

Figure 4.20. Calculated RQD (20cm threshold) vs. core run length for orthogonal boreholes with P\textsubscript{10} of 10.
Figure 4.21. Calculated RQD (20cm threshold) vs. core run length for inclined boreholes with $P_{10}$ of 3.

Figure 4.22. Calculated RQD (20cm threshold) vs. core run length for inclined boreholes with $P_{10}$ of 5.
Figure 4.23. Calculated RQD (20cm threshold) vs. core run length for inclined boreholes with $P_{10}$ of 7.

Figure 4.24. Calculated RQD (20cm threshold) vs. core run length for inclined boreholes with $P_{10}$ of 10.
Figure 4.25. RQD (20cm threshold) relative difference versus core run length and $P_{10}$ for Orthogonal Boreholes.

Figure 4.26. RQD (20cm threshold) relative difference versus core run length and $P_{10}$ for Inclined Boreholes.
Figure 4.27. Calculated RQD (30cm threshold) vs. core run length for orthogonal boreholes with $P_{10}$ of 3.

Figure 4.28. Calculated RQD (30cm threshold) vs. core run length for orthogonal boreholes with $P_{10}$ of 5.

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Figure 4.29. Calculated RQD (30cm threshold) vs. core run length for orthogonal boreholes with $P_{10}$ of 7.

Figure 4.30. Calculated RQD (30cm threshold) vs. core run length for orthogonal boreholes with $P_{10}$ of 10.
Figure 4.31. Calculated RQD (30cm threshold) vs. core run length for inclined boreholes with $P_{10}$ of 3.

Figure 4.32. Calculated RQD (30cm threshold) vs. core run length for inclined boreholes with $P_{10}$ of 5.
Figure 4.33. Calculated RQD (30cm threshold) vs. core run length for inclined boreholes with $P_{10}$ of 7.

Figure 4.34. Calculated RQD (30cm threshold) vs. core run length for inclined boreholes with $P_{10}$ of 10.
Figure 4.35. RQD (30cm threshold) relative difference (Max RQD - Min RQD) versus core run length and $P_{10}$ for Orthogonal Boreholes.

Figure 4.36. RQD (30cm threshold) relative difference (Max RQD - Min RQD) versus core run length and $P_{10}$ for Inclined Boreholes.
Key observations are almost the same as those of the threshold of 10cm, but the relative difference for 20cm and 30cm threshold (max RQD minus min RQD) does not increase with increasing fracture intensity (for a selected core run length).

RQD values for the same core typically vary for different threshold lengths and generally increase with increasing threshold value. The value of Max – Min RQD obtained from different fracture intensities, scales and assumed thresholds (10cm vs. 20cm vs. 30cm) are given separately in Table 4.3 for the orthogonal boreholes and Table 4.4 for the inclined boreholes.

Table 4.3. Max – Min RQD obtained from orthogonal boreholes

<table>
<thead>
<tr>
<th>Core Run Length</th>
<th>Max – Min RQD (%)</th>
<th>10cm</th>
<th>20cm</th>
<th>30cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>P₁₀</td>
<td></td>
<td>3</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>15</td>
<td>2.35</td>
<td>2.07</td>
<td>5.30</td>
<td>8.29</td>
</tr>
<tr>
<td>10</td>
<td>5.50</td>
<td>8.06</td>
<td>8.40</td>
<td>10.3</td>
</tr>
<tr>
<td>5</td>
<td>9.96</td>
<td>15.1</td>
<td>16.2</td>
<td>24.2</td>
</tr>
<tr>
<td>3</td>
<td>13.9</td>
<td>25.9</td>
<td>28.9</td>
<td>37.9</td>
</tr>
</tbody>
</table>
According to Table 4.3 and 4.4, the difference between maximum and minimum RQD measurements generally increases with increasing selected threshold value, and correspondingly, the REL of RQD for the same rock mass increases.

For the case study, RQD values of different threshold were measured for the Middleton mine along three orthogonal boreholes and four inclined boreholes. The average RQD for orthogonal and inclined boreholes are in Figures 4.37 to 4.40 respectively. The results confirm the key observations made for the conceptual model, and yet, the REL is approximately 15m (independent of the orientation of the core run).

Table 4.4. Max – Min RQD obtained from inclined boreholes

<table>
<thead>
<tr>
<th>Max – Min RQD (%)</th>
<th>10cm</th>
<th>20cm</th>
<th>30cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>P&lt;sub&gt;10&lt;/sub&gt;</td>
<td>3</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>Core Run Length</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>1.60</td>
<td>0.92</td>
<td>6.30</td>
</tr>
<tr>
<td>10</td>
<td>4.14</td>
<td>5.30</td>
<td>8.70</td>
</tr>
<tr>
<td>5</td>
<td>8.98</td>
<td>17.6</td>
<td>19.9</td>
</tr>
<tr>
<td>3</td>
<td>14.4</td>
<td>21.8</td>
<td>30.4</td>
</tr>
</tbody>
</table>
Figure 4.37. Middleton mine. Calculated RQD (20cm threshold) vs. core run length for orthogonal boreholes.

Figure 4.38. Middleton mine. Calculated RQD (20cm threshold) vs. core run length for inclined boreholes.
Figure 4.39. Middleton mine. Calculated RQD (30cm threshold) vs. core run length for orthogonal boreholes.

Figure 4.40. Middleton mine. Calculated RQD (30cm threshold) vs. core run length for inclined boreholes
4.7 Influence of Core Run Length on $P_{21}$ (along with RQD)

The results for $P_{21}$ show a very similar pattern to RQD. Difference between the maximum and minimum measured value decreases with increasing core run length for all four models (see Figures 4.41 to 4.44).

Figure 4.41. Calculated $P_{21}$ and RQD vs core run length for the orthogonal boreholes with $P_{10}$ of 3.
Figure 4.42. Calculated $P_{21}$ and RQD vs core run length for the orthogonal boreholes with $P_{10}$ of 5.

Figure 4.43. Calculated $P_{21}$ and RQD vs core run length for the orthogonal boreholes with $P_{10}$ of 7.
Figure 4.44. Calculated $P_{21}$ and RQD vs core run length for the orthogonal boreholes with $P_{10}$ of 10.

Not only the REL size of RQD, but the geometrical REL size, which is calculated as the sum length of fracture traces per unit area of sampling plane ($P_{21}$), was estimated to be around 15m.

4.8 Blockiness of Conceptual Models

Based on the defined rock mass of 30 meter’s side length, the blockiness of the rock masses is evaluated based on the accumulated block volume percentage versus increasing block size. Results obtained from the block size verification analysis of four conceptual models are given in Figure 4.45.
Figure 4.45. Block analysis of conceptual models with $P_{10}$ of 3, 5, 7 and 10.

According to the figure, an increased $P_{10}$ results in smaller blocks. In terms of the cell number defined (100 in one axis), the minimum block size is 0.027 m$^3$ and for the model with $P_{10}$ of 10, 80% of the blocks is of the smallest size of 0.027 m$^3$; whereas, in the model with $P_{10}$ of 3, there is only around 15%. For all four conceptual models, there is no block exceeding 100 m$^3$. 
5 Conclusion and Recommendation

5.1 Conclusion

Numerical technique such as the DFN modelling provides the means of simulating the discontinuity of rock mass at scales ranging from each individual rock block to the involving scale of the entire engineering structure. DFN code like FracMan allows direct two and three-dimensional modelling of the physical properties of rock mass with definable fracture length, spacing, direction and etc.; and therefore, provides a much more representative alternative than the empirical characterizations and rock mass parameter estimation approaches through core logging.

RQD has proved useful in rock engineering as it gives a rough representation of the actual quality of rock masses. Through RQD measurement, the quality of the rock masses is assessed with simple tools of low operating costs but inevitable limitations. Although these limitations have been addressed, RQD is still directly and indirectly (parameter in some rock mass rating systems) used in many geotechnical engineering applications without correction. In this thesis, the importance of data characterization process on RQD determination is studied through different measuring lengths and directions. In this way, based on the synthetic rock mass of 30 meters, the influence of measuring length and sampling direction is given. The definition of REL and REV would be very dependent on the variation in geotechnical domains. Note that the results of the current analysis apply to relatively homogeneous geotechnical domains, as both shear and fault zones should be treated separately.

For the primary goal of the study of scale effect, in this work, 8 models were developed based on 4 classes of fracture density and 2 types of wells. RQD in each well of different measuring lengths
was calculated using method initially proposed by Deere in 1963 to determine the scale effects in RQD with varying lengths, which range from 3 to 30 meters. For intervals of each defined length (i.e. 3m, 5m, 10m and 15m), a maximum, a minimum and an average value of calculated RQD were recorded. The results indicate that, with the increase of the measuring interval, the variance of calculated RQD of the synthetic fractured rock mass linearly decreases. The size of the REL of the fractured rock mass in terms of RQD was assumed to be 10 times the recommended core run length by Deere (1988). The estimated REL would represent the dimension at which the rock mass could be considered as a continuum medium, and its properties defined using an equivalent continuum approach.

When comparing all the models, the largest difference of measured RQD extremum which was calculated at interval of 3 meter exists in the rock mass of highest fracture intensity. The measured extremum difference is most sensitive to changes in core run length, followed by changes in fracture intensity (i.e. $P_{10}$ linear discontinuity frequency) and then fracture orientation by changes in well drilling direction.

Secondarily, in order to investigate the relationship between fracture intensity indicators (i.e. linear, and areal fracture frequency, and Volumetric Joint Count) and RQD. For each conceptual model, the Linear Discontinuity Frequency ($P_{10}$) and Volumetric Joint Count ($P_{30}$) were applied to alternatively assess the RQD. Difference among manually calculated RQD and determined using other data from face map questions the reliability of empirical equations, and it might be raised by the sole application of linear fracture intensity to define fracture. $P_{21}$ (only orthogonal boreholes) was calculated corresponding to its RQD at selected core run length. The results indicate that the
geometrical REL size of $P_{21}$ is also around 15m.

Different thresholds of RQD determination were used for the study of scale effect, and comparison between rock mass scale and assumed threshold (10cm vs. 20cm vs. 30cm) were made. The results show that greater REL occurs for the longer threshold.

RQD represents a limited part of rock mass quality and there are some inherent limitations resulted from its simple measuring manipulation. According to Deere in 1988, calculation of the RQD should be based on the actual drilling-run length used in the field, but laboratorial core length is no more than 1.5 m. This methodology could serve as a reference for quantification of RQD with a range and thus better predict the characteristics of the studying rock masses in engineering project.

5.2 Recommendations for the Further Work

The scale effect analysis was based on a REV concept of the representative volume of certain property in the studying rock mass. There were limitations in the observation of three sets network as in most case engineering concerned rock mass consists of more complex fracture sets of heterogeneity and anisotropy. The measurements of the RQD of the synthetic rock masses based on field data might not to be reliable as both intensity and dispersity of the fractures were changed.

This study proposed method to define fracture sets by exclusively using linear discontinuity frequency data. Due to the constant length of inserted well, higher frequency results in more fractures, whereas dispersity remained unchanged. In the future study, the spacing and persistence of fractures can be defined by constructing the models with areal density or volume density. There
is a need to extend the current work to DFN models with more complex fracture sets and to compare actual RQD measurements along boreholes to the simulated RQD values measured in the associated DFN models. Moreover, a pre-defined upper limit of the size (30m side length cube) of the rock mass when studying the existence of the REV is quite different from most of the previous studies.

A brief analysis was also carried out using Linear Discontinuity Frequency and Volumetric Joint Count to alternatively determine the RQD of the synthetic rock mass. Increase in fracture density has a negative impact on the correctness of RQD determination directly using these parameters. This was not expected, more complex models or models with in situ data should be generated to study the reliability of empirical RQD determinations without core.

Based on the work carried out in this paper, the potential of DFN models for fragmentation determination and evaluation was confirmed. DFN modeling shows great applicability to RQD assessment, and the study of the factors influencing the measured extremum. The parameters investigated here were limited to the most general cases (i.e. three orthogonal discontinuity sets) of ideally simplified rock mass. Confidence in the conclusions could be increased with assessment of the findings described above (i.e., different model generation types, different fracture size, different distribution types, etc.), and particularly in models with non-orthogonal joint sets defined based on real data without modification. A mathematic relation among fracture intensity, measured interval and range of RQD might be established with attestation of large amount RQD calculations obtained from laboratory core logging.
Reference


## Appendix I


<table>
<thead>
<tr>
<th>A. CLASSIFICATION PARAMETERS AND THEIR RATINGS</th>
<th>Range of value</th>
<th>For this low range - unconsolidated compressive test is preferred</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Strength of intact rock material</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Post/rock strength index</td>
<td>&gt;10 MPa</td>
<td>4 - 10 MPa</td>
</tr>
<tr>
<td>Uniaxial comp. strength</td>
<td>&gt;250 MPa</td>
<td>100 - 250 MPa</td>
</tr>
<tr>
<td>Rating</td>
<td>15</td>
<td>12</td>
</tr>
<tr>
<td>2. Diff. quality QD</td>
<td>90% - 100%</td>
<td>79% - 90%</td>
</tr>
<tr>
<td>Rating</td>
<td>20</td>
<td>17</td>
</tr>
<tr>
<td>Spacing of</td>
<td>&gt;2 m</td>
<td>0.6 - 2 m</td>
</tr>
<tr>
<td>Rating</td>
<td>20</td>
<td>16</td>
</tr>
<tr>
<td>3. Condition of discontinuities</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(See E)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rating</td>
<td>30</td>
<td>22</td>
</tr>
<tr>
<td>4. Groundwater</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inflow per 10 m tunnel length (lin)</td>
<td>None</td>
<td>&lt;10</td>
</tr>
<tr>
<td>General conditions</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rating</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strike and dip orientations</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ratings</td>
<td>Very favourable</td>
<td>Fair</td>
</tr>
<tr>
<td>5. Rock mass classes determined from total ratings</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class number</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Description</td>
<td>Very good rock</td>
<td>Good rock</td>
</tr>
<tr>
<td>Class number</td>
<td></td>
<td></td>
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<td>D. MEANING OF ROCK CLASSES</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class number</td>
<td>II</td>
<td>III</td>
</tr>
<tr>
<td>E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Discontinuity length (perpendicular)</td>
<td>&lt;1 m</td>
<td>1 - 3 m</td>
</tr>
<tr>
<td>Rating</td>
<td>6</td>
<td>11</td>
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<tr>
<td>Separation (spacing)</td>
<td>None</td>
<td>&lt;0.1 mm</td>
</tr>
<tr>
<td>Rating</td>
<td>8</td>
<td>3</td>
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<tr>
<td>Roughness Rating</td>
<td>Very rough</td>
<td>Rough</td>
</tr>
<tr>
<td>Rating</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>Infilling (pore)</td>
<td>None</td>
<td>Hard filling &lt; 5 mm</td>
</tr>
<tr>
<td>Rating</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>Weathering Rating</td>
<td>Unweathered</td>
<td>Slightly weathered</td>
</tr>
<tr>
<td>Rating</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strike perpendicular to tunnel axis</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strike parallel to tunnel axis</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dike with dip - Dip 45° - 90°</td>
<td>Very favourable</td>
<td>Favourable</td>
</tr>
<tr>
<td>Dike with dip - Dip 20° - 45°</td>
<td>Favourable</td>
<td>Vry favourable</td>
</tr>
<tr>
<td>Dike 45° - 90°</td>
<td>Fair</td>
<td></td>
</tr>
<tr>
<td>Dike against dip - Dip 45°-90°</td>
<td>Unfavourable</td>
<td>Fair</td>
</tr>
<tr>
<td>Dike against dip - Dip 20-45°</td>
<td>Fair</td>
<td></td>
</tr>
</tbody>
</table>

* Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

** Modified after Wickham et al. (1972).
Table A.2. Joint Number Factor ($J_n$) (from Hoek, 2007, after Barton et al., 1974).

<table>
<thead>
<tr>
<th>Massive, no or few joints</th>
<th>$J_n = 0.5 - 1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>One joint set</td>
<td>2</td>
</tr>
<tr>
<td>One joint set plus random joints</td>
<td>3</td>
</tr>
<tr>
<td>Two joint sets</td>
<td>4</td>
</tr>
<tr>
<td>Two joint sets plus random joints</td>
<td>6</td>
</tr>
<tr>
<td>Three joint sets</td>
<td>9</td>
</tr>
<tr>
<td>Three joint sets plus random joints</td>
<td>12</td>
</tr>
<tr>
<td>Four or more joint sets, heavily jointed, &quot;sugar-cube&quot;, etc.</td>
<td>15</td>
</tr>
<tr>
<td>Crushed rock, earthlike</td>
<td>20</td>
</tr>
</tbody>
</table>

**Notes:** (i) For tunnel intersections, use $(3.0 \times J_n)$; (ii) For portals, use $(2.0 \times J_n)$

Table A.3. Joint Roughness Number ($J_r$) (from Hoek, 2007, after Barton et al., 1974).

<table>
<thead>
<tr>
<th>Discontinuous joints</th>
<th>$J_r = 4$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rough or irregular, undulating</td>
<td>3</td>
</tr>
<tr>
<td>Smooth, undulating</td>
<td>2</td>
</tr>
<tr>
<td>Slickensided, undulating</td>
<td>1.5</td>
</tr>
<tr>
<td>Rough or irregular, planar</td>
<td>1.5</td>
</tr>
<tr>
<td>Smooth, planar</td>
<td>1.0</td>
</tr>
<tr>
<td>Slickensided, planar</td>
<td>0.5</td>
</tr>
</tbody>
</table>

**Notes:**
(i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
(ii) $J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided the lineations are oriented for minimum strength.

Table A.4. Joint Alteration Number ($J_a$) (from Hoek, 2007, after Barton et al., 1974).

<table>
<thead>
<tr>
<th>Joint Wall Character</th>
<th>Condition</th>
<th>Wall contact</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLEAN JOINTS</td>
<td>Healed or welded joints: filling of quartz, epidote, etc.</td>
<td>$J_a = 0.75$</td>
</tr>
<tr>
<td>Fresh jointwalls:</td>
<td>no coating or filling, except from staining (rust)</td>
<td>1</td>
</tr>
<tr>
<td>Slightly altered jointwalls</td>
<td>non-softening mineral coatings, clay-free particles, etc.</td>
<td>2</td>
</tr>
<tr>
<td>COATING OR THIN FILLING</td>
<td>Friction materials: sand, silt, calcite, etc. (non-softening)</td>
<td>3</td>
</tr>
<tr>
<td>Cohesive materials:</td>
<td>clay, chlorite, tect, etc. (softening)</td>
<td>4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Filling of:</th>
<th>Type</th>
<th>Some wall contact</th>
<th>No wall contact</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction materials</td>
<td>sand, silt calcite, etc. (non-softening)</td>
<td>$J_a = 4$</td>
<td>$J_a = 8$</td>
</tr>
<tr>
<td>Hard cohesive materials</td>
<td>compacted filling of clay, chlorite, tect, etc.</td>
<td>6</td>
<td>5 - 10</td>
</tr>
<tr>
<td>Soft cohesive materials</td>
<td>medium to low overconsolidated clay, chlorite, tect, etc.</td>
<td>8</td>
<td>12</td>
</tr>
<tr>
<td>Swelling clay materials</td>
<td>filling material exhibits swelling properties</td>
<td>8 - 12</td>
<td>13 - 20</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Condition</th>
<th>$p_w$ (kg/cm²)</th>
<th>$J_w$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry excavations or minor inflow, i.e. &lt; 5 l/min locally</td>
<td>$&lt; 1$</td>
<td>1</td>
</tr>
<tr>
<td>Medium inflow or pressure, occasional outwash of joint fillings</td>
<td>1 - 2.5</td>
<td>0.66</td>
</tr>
<tr>
<td>Large inflow or high pressure in competent rock with unfractured joints</td>
<td>2.5 - 10</td>
<td>0.5</td>
</tr>
<tr>
<td>Large inflow or high pressure, considerable outwash of joint fillings</td>
<td>2.5 - 10</td>
<td>0.3</td>
</tr>
<tr>
<td>Exceptionally high inflow or water pressure at blasting, decayed with time</td>
<td>$&gt; 10$</td>
<td>0.2 - 0.1</td>
</tr>
<tr>
<td>Exceptionally high inflow or water pressure continuing without noticeable decay</td>
<td>$&gt; 10$</td>
<td>0.1 - 0.05</td>
</tr>
</tbody>
</table>

*Note:* (i) The last four factors are crude estimates. Increase $J_w$ if drainage measures are installed. (ii) Special problems caused by ice formation are not considered.

Table A.6. Description and ratings for parameter Stress Reduction Factor (SRF) (from Hoek, 2007, after Barton et al., 1974).

<table>
<thead>
<tr>
<th>Weakness zones intersecting excavation</th>
<th>SRF = 10</th>
<th>SRF = 5</th>
<th>SRF = 2.5</th>
<th>SRF = 1</th>
<th>SRF = 0.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Multiple weakness zones with clay or chemically disintegrated rock, very loose surrounding rock (any depth)</td>
<td>SRF = 10</td>
<td>SRF = 5</td>
<td>SRF = 2.5</td>
<td>SRF = 1</td>
<td>SRF = 0.5</td>
</tr>
<tr>
<td>Single weakness zones containing clay or chemically disintegrated rock (depth of excavation &lt; 50 m)</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Single weakness zones containing clay or chemically disintegrated rock (depth of excavation &gt; 50 m)</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Single shear zones in competent rock (clay-free), loose surrounding rock (any depth)</td>
<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
</tr>
<tr>
<td>Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation &lt; 50 m)</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation &gt; 50 m)</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Loose, open joints, heavily jointed or &quot;sugar-cube&quot;, etc. (any depth)</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>

*Note:* (i) Reduce these SRF values by 25 - 50% if the relevant shear zones only influence, but do not intersect the excavation.

<table>
<thead>
<tr>
<th>Competent rock, rock stress problems</th>
<th>$\alpha_b / \alpha_c$</th>
<th>$\alpha_b / \alpha_c$</th>
<th>SRF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low stress, near surface, open joints</td>
<td>$\geq 200$</td>
<td>$&lt; 0.01$</td>
<td>2.5</td>
</tr>
<tr>
<td>Medium stress, favourable stress condition</td>
<td>200 - 10</td>
<td>0.01 - 0.3</td>
<td>1</td>
</tr>
<tr>
<td>High stress, very tight structure. Usually favourable to stability, may be except for walls</td>
<td>10 - 5</td>
<td>0.3 - 0.4</td>
<td>0.5 - 2</td>
</tr>
<tr>
<td>Moderate slaving after &gt; 1 hour in massive rock</td>
<td>5 - 3</td>
<td>0.5 - 0.65</td>
<td>5 - 50</td>
</tr>
<tr>
<td>Slabling and rock burst after a few minutes in massive rock</td>
<td>3 - 2</td>
<td>0.65 - 1</td>
<td>50 - 200</td>
</tr>
<tr>
<td>Heavy rock burst (strain burst) and immediate dynamic deformation in massive rock</td>
<td>$&lt; 2$</td>
<td>$&lt; 1$</td>
<td>200 - 400</td>
</tr>
</tbody>
</table>

*Notes:* (ii) For strongly anisotropic stress field (if measured): when $5 < \alpha_b / \alpha_c < 10$, reduce $\alpha_c$ to 0.75 $\alpha_c$. When $\alpha_b / \alpha_c > 10$, reduce $\alpha_c$ to 0.5 $\alpha_c$. (iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for low stress cases.

<table>
<thead>
<tr>
<th>Squeezing rock</th>
<th>Plastic flow of incompetent rock under the influence of high pressure</th>
<th>Mild squeezing rock pressure</th>
<th>1 - 5</th>
<th>5 - 10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Swelling rock</td>
<td>Chemical swelling activity depending on presence of water</td>
<td>Mild swelling rock pressure</td>
<td>&gt; 5</td>
<td>10 - 20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Heavy swelling rock pressure</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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