SCHOOL OF MATERIALS AND MINERAL RESOURCES ENGINEERING

UNIVERSITI SAINS MALAYSIA

DETERMINATION OF STRENGTH AND DEFORMABILITY OF FRACTURED ROCK MASS BY FINITE ELEMENTAL MODELLING

By

EMIROL AMRI BIN AMINUDDIN

Supervisor: Dr. Hareyani bt Zabidi

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DECLARATION

I hereby declare that I have conducted, completed the research work and written the dissertation entitled "**Determination of Strength and Deformability of Fractured Rock Mass by Finite Elemental Modelling**". I also declare that it has not been previously submitted for the award of any degree or diploma or other similar title of this for any other examining body or university.

Name of student : Emirol Amri bin Aminuddin Signature :

Date : 3rd July 2017

Witness by

Supervisor : Dr Hareyani bt Zabidi Signature :

Date : 3rd July 2017

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ABSTRAK

Deformasi dan kekuatan batu adalah salah satu masalah geoteknikal yang perlu diambil kira oleh jurutera apabila berurusan dengan reka bentuk apa-apa jenis struktur moden. Dalam kerja-kerja masa ini, deformability dan kekuatan jisim batuan patah ini telah dipilih dengan menggunakan terhingga Elemental Menghubungi dengan menggunakan perisisan RS2. Model ini telah memohon untuk belajar pergantungan skala kedua-dua kekuatan dan deformability jisim batuan patah. Juga, patah / tingkah laku bersama dikaji dengan menggunakan kaedah ini. analisis bersama dijalankan dengan menggunakan Dips 7.0 dengan menggunakan plot Rosette. Parameter 6 dan 11 dipilih untuk digunakan dalam permodelan untuk Analisis Finite Elemental. Arahan dip keduanya adalah 6.58 dan 326 darjah. Sifat-sifat batu yang diperolehi daripada kerja lapangan dianalisis dahulu dengan menggunakan perinsi Rocdata sebelum dimasukkan ke dalam prosedur pemodelan dalam perinsi RS2. Keputusan menunjukkan bahawa jumlah wakil diterima untuk kekuatan dan deformasi jisim batuan patah ialah 12 m dengan menggunakan pekali yang boleh diterima ubahan 1.5%.

DETERMINATION OF STRENGTH AND DEFORMABILITY OF FRACTURED ROCK MASS BY FINITE ELEMENTAL MODELLING

ABSTRACT

Rock deformation and strength of rock is one of the geotechnical problem that has to be taken into consideration by engineers when dealing with the design of any type of modern structures. In present work, the deformability and strength of fractured rock mass is determined by applying the Finite Elemental Method by using the software RS2. The model was applied to study the scale dependency of both strength and deformability of fractured rock mass. Also, the fracture/joint behaviour is studied by using this method. Joint analysis is carried out by using Dips 7.0 by using the Rosette plot. Joint parameters of 6 and 11 are chosen to be used in the model for Finite Elemental Analysis. The dip directions are 6.58 and 326 degrees respectively. The intact rock properties as acquired from the fieldwork was analysed first by using Rocdata before being inserted into the modelling procedure in RS2. The results show that the accepted representative elementary volume for the strength and deformability of fractured rock mass is 12 m by using the acceptable coefficient of variation of 1.5%.

CHAPTER 1

INTRODUCTION

1.1 Research Background

For centuries, man-made structures such as dam have been built on, in or of rock. Back in Mesopotamian times, early dam built were for controlling the water level as the weather usually affected the rivers, mainly Tigris and Euphrates. Nowadays, most modern dams are built for hydro-power generation as time progresses.

The oldest known dam recorded in the world is the Jawa Dam located in Jordan built roughly about 3000 BC. The masonry gravity dam was built with a 9-metre high and 1-m wide stone wall which is supported by a 50-metre earth rampart. However, this dam is not the oldest dam in the world. The oldest dam in the world is in fact the Kallanal Dam or which is more popularly known as Grand Anicut. It was built in 2nd century AD and as of now still serves the people of Tamilnadu, India.

As modern time progresses, more and more dams are built for different specified reasons and in different scales. As the function of the dams increase, the scales of the dams in turn need to be relative to the dam. Due to the increase in scale, more engineering challenges are faced mainly relating to two important parameters when dealing with design, operation or construction of engineering structures which are the strength and deformability of fractured rock mass. When fractures are present, the strength and deformability of a rock mass could be significantly affected. Both parameters can be acquainted with the behaviour of rocks. Hence, to precisely determine and analyse these two parameters has somewhat become an important topic mainly in geophysical studies.

In practical engineering projects, sometimes field investigations are limited due to a few factors. Such factors could be safety concerns preventing access for investigation, the cost of the investigation itself or various other reasons. Due to these limiting factors, a new approach has been taken to complete the investigation by means of geo-mechanical modelling.

For these kinds of studies, the in-situ stress is a very important aspect that needs to be identified to get a better understanding of rock mass properties surrounding the project. To precisely model the reality numerically, the knowledge of the initial state of stress in the ground and of the material properties is very important as well as to understand the structural behaviour of the materials that is being dealt with.

These types of analyses have always been a challenge due to the existing discontinuity in the rock masses such as faults, joints, beddings and foliation. A fault in geological term can be understood as a break in the rocks that make up the Earth’s crust, along which rocks on either side have been displaced past each other. What really defines a fault is the displacement of rocks on either side.

In geological term, joints can be described as a fracture that divides a rock into two sections that have not moved away from each other. The difference between a joint and another kind of fracturing such as a fault is that a joint see almost no movement compared to like in a vault whereas a gap is formed in the rock by a visible crack.

The smallest division of a geologic formation marked by a or more well defined planes separating it into layers are known as bedding planes. Foliation refers to a repeated layering in metamorphic rocks where the layers can be as thick as 1 metre or even thin as a sheet.

As both strength and deformability of rocks are important aspects of behaviour of rocks, both needs to be well understood especially in fractured rock mass. Deformability is characterised by a modulus describing the relationship between the applied load and the resulting deformation (*Bienawski, 1989)*.

By *Bienawski, 1989* the behaviour of rocks is best presented in a stress – strain curve. It will be noted that initially, deformation increases approximately proportional with increasing load. Eventually, a stress level is reached at which fracture is initiated and starts to propagate. Further increasing the stress leads to another stress level, the critical energy released. At this stage, the crack propagation is unstable and continues even when the stress increase is stopped.

Next, the maximum load bearing capacity is reached. This is in fact the strength of the rock. Hence, strength can be described as the maximum load bearing capacity of a rock.

1.2 Problem Statement

When dealing with design, operation or construction of engineering structures, strength and deformation are two of the most important parameters regarding the mechanical behaviour of rock. When fractures are present, the strength and deformability of a rock mass could be affected.

To determine these mechanical parameters of fractured rock mass, numerical methods via Finite Element Method will be used. Finite element method (FEM) is used for stability analysis purpose by parametrically varying rock joint persistence, spacing and shear strength parameters, until the condition of overbreak is reached.

1.3 Objectives

1. To study the strength characteristics and deformation properties of fractured rock mass.

2. To understand the various uses of the software RS2.

3. To analyse scale dependency of mechanical parameters of fractured rock mass.

4. To study the joint/deformation behaviour of fractured rock mass.

1.4 Project Area

The Ulu Jelai Hydroelectric Project lies within the Cameron Highlands and extends from the Pahang/Perak State borders in the west to the Telom-Bertam-Lemoi river confluences in the east. This area is wholly contained within the Main Range of Peninsular Malaysia. This a long range of hills and low mountains extending the length of the country. The highest point of the Main Range in the Cameron Highlands is Gunung Irau at 2110 m elevation, just 2 km NW of Gunung Berincang at EL 2031m, where the hilly terrain is up to 70 km across from west to east. The highest point within the project area is Bukit Bujang at 1772 m, located between the Bertam and Lemoi River valleys.

The dam site (Susu Dam) is on the Bertam River about 900 m downstream from Kg Susu. The elevation of the river bed at the centreline is 465 m. The river has a gradient of about 5%, and the channel is a series of pools and cascades flowing between accumulations of massive granite boulders to 10 m in size. The river channel is 15-20 m wide at the dam site and has banks ranging from steep to low sandy beaches, depending upon the stage of the flow. The topographic view of Susu Dam is shown in Figure 1.1.

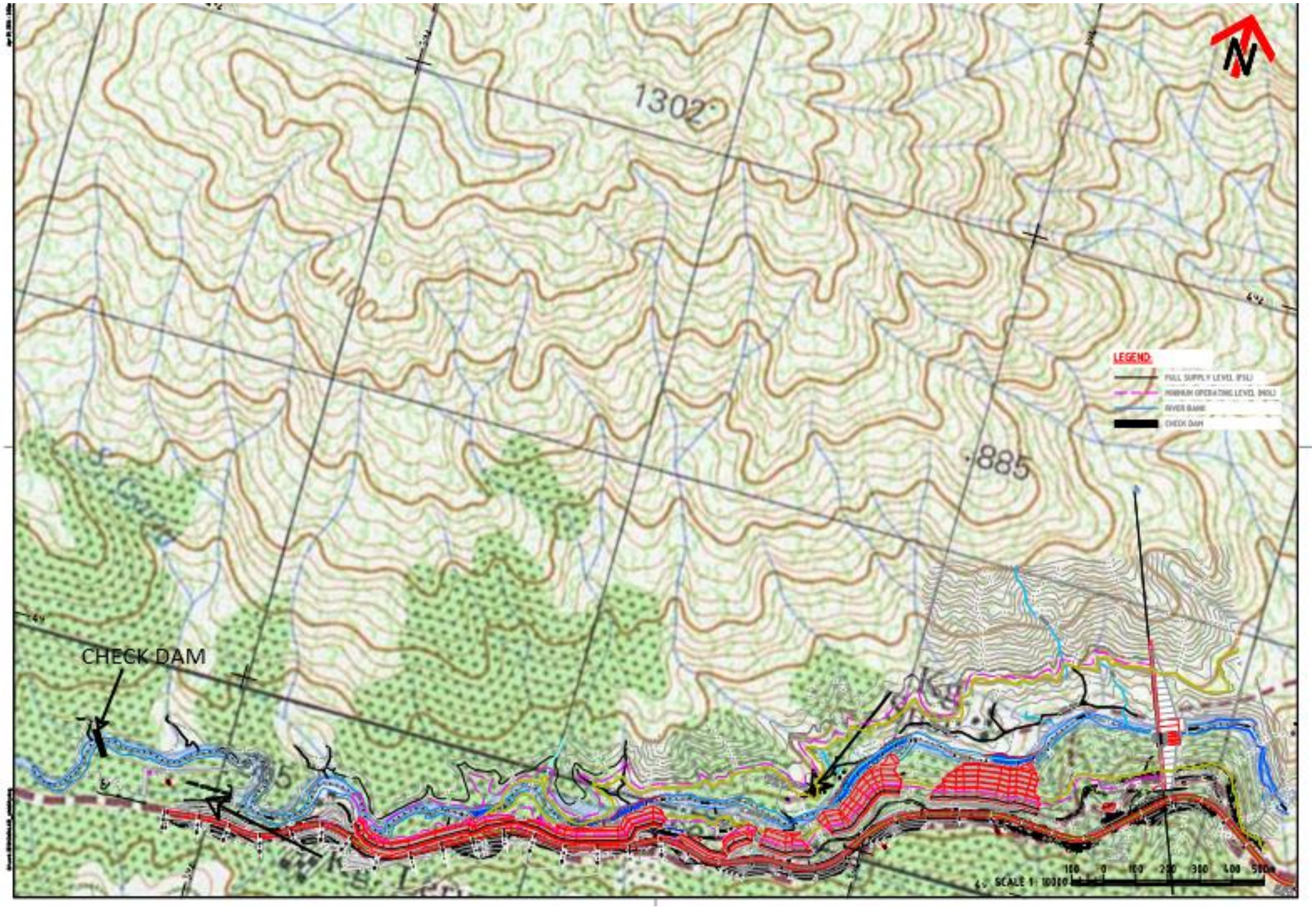

Figure 1.1 *Topographic view of Susu Dam site*

Figure 1.2 *Image of susu dam*

Figure 1.3 *Image of Susu Dam (2)*

The right abutment has an overall slope of 30 degrees, but steepens to 40 degrees near the top of the abutment slope (between EL 540 m and 565 m), and at the toe of the slope (between EL 475 m and 490 m). The right abutment is accessed by exploration tracks to the various drill holes, but many of these are now overgrown and eroded. The track at EL 540 m is accessible by 4WD vehicle from the downstream end. Above the abutment is the new National Highway at EL 565 m.

The left abutment is less even in slope profile as there is a distinct terrace between EL 490 m and 510 m, where the slope averages 12 degrees. Between this terrace and the river, the slope is quite steep at 35 degrees, and is likely an erosional bank. The higher slopes on the left abutment, from EL 510 m to 600 m have an average slope of 35 degrees, but as the slope is slightly concave it does steepen to over 40 degrees at the top. The higher slopes above the planned dam crest level at EL 545 become steeper into the head of a stream gully before a ridge line is reached at EL 650 m. The overall relief to the top of the range is over 800m. The dam site is covered in dense forest, with trees to 30 m height, due to earlier clearing there is now dense secondary growth that is almost impenetrable.

The rock type at the dam site is medium to coarse grained biotite granite that is porphyritic with phenocrysts of feldspar to 5 cm in size. There is variability in the rock mass quality due to hydrothermal alteration and brecciation, particularly beneath the left abutment. This may be related to an inferred fault that has a trace, bearing 070 degrees (ENE) beneath the previously mentioned terrace.

The main soils are residual soils derived from the weathering of granite, colluvium and alluvium. The residual soil has a deep profile, being up to 25 m in thickness. The soil profile is as follow;

Table 1.1 Soil Profile

Depth (metres)	Soil Profile
0-0.5	Topsoil, organic debris and humus, often less than 10cm
0.5-2.0	A horizon: yellow brown sandy silt
2.0-6.0	B horizon: red brown often mottled red-grey-yellow brown sandy to clayey silt
>6	Becomes more clayey silt and contains core stones of less weathered granite to 2-3 m size/

The colluvium is more variable in texture and contains spaces or voids where soil particles have moved against each other, often appearing as pin-holes in description. The soil profile may be sandier to gravelly where groundwater through-flow has removed the finer components. Where this layer is roughly less than 2 m in thickness it is termed ‘slopewash’. But if the material is much deeper and broader in extent it is likely the result of mass movement and is termed colluvium. The colluvium is mostly sandy silt to gravelly silt or sand and is found most expansively on the left abutment. This deposit is likely from a progressive failure or piecemeal failures from a gully head at about EL 650 m and occurs just upstream of the dam extending down to river level. There does not appear to be very much colluvium on the right abutment of the dam.

The alluvium is river transported and at the dam site is essentially an accumulation of boulders as described above. These are residual accumulations of boulders following erosion of the valley sides. The boulders likely have reached the valley floor by a mixture of creep and mass wasting. The finer components have been removed by the erosive power of this high-energy river. The terrace, mentioned above, is likely a mix of colluvium and alluvium, but depth to rock is quite shallow at 6-8 m which indicates that the terrace is partially due to the weathering depth on this side of the river.

The site has been drilled in 2003 and 2007. The list of relevant holes and summarised depths to CW rock and to SW-Fresh rock is presented in the following Table 5. Interpreted sections of each abutment are presented in Drawings B-10A and 10B. Briefly the depth to fresh rock ranges from zero at the river bed to over 40 m higher on both valley sides. The depths to the top of the CW rock range from nil to 18 m, while the depths to SW-Fr rock range from less than 1m to greater than 30 m. There is also quite a variation in the quality of the granite. In many places it is hydrothermally altered by sericite and chlorite replacement in the feldspars and biotite. This leaves the rock with a light green-grey bleached appearance, and is often associated with shearing. There appears to be some cataclastic fabric with quartz and calcite infillings.

The most intense alteration occurs in drill holes on the left abutment from holes UJ2/2D to UJ2/21D and this may be along the inferred fault seen as a lineament trace. This structure is expected to be steeply dipping and to contain alternating good and poor rock. The juxtaposed UJ2/3D and UJ2/21D are only 5 m apart but the drill logs indicate quite variable conditions. A second zone of alteration follows a line between holes UJ2/9D, 4D and 29D along the right abutment. This may be a parallel structure offset from the main fault zone. Higher on the right abutment hole UJ2/7D also encountered rock with extensive and strong alteration.

The drilling at another dam site in 1988, located 1 km upstream from the present Susu site, at Tawakkal encountered mostly fresh leucocratic porphyritic biotite except from drilling at CH 32, sited on the left bank. In this hole kaolinized and weathered granite was found to considerable depth. This hole is found to be along strike from the inferred fault at Susu dam site and the associated lineament trace.

At Kg Leryar site the only hole to encounter deep weathering, that may be approximated to alteration, was at CH 4 where MW rock was found to >40 m depth. This hole is on the right bank inside the large loop in the river. The lineament trace referred to above passes directly through this hole location.

This there is a strong indication of a weak hydrothermally altered and weathered zone in the granite associated with a fault zone along Sg Bertam. The strike is 070 degrees and the lineament can be traced for over 10 km.

1.5 Thesis outline

This thesis is organized into five chapters. Chapter 1 would describe a brief background about the proposed study, the problem statement and objectives of the research. Chapter 2 is a comprehensive review on previous studies conducted on strength and deformability of rocks. This chapter will also detail about the failure criterions such as Mohr – Coulomb and Hoek Brown, Finite Elemental Analysis and also on fractured rock mass and its mechanical properties. Chapter 3 details the methodology used in conducting this study which includes fieldwork, orientation analysis, determination of rock mass data and stress and deformation analysis. Next, chapter 4 presents the results obtained from the study together with a brief discussion about the results. Chapter 5 concludes the research by including the results obtained with recommendations for future works regarding the subject.

CHAPTER 2

LITERATURE REVIEW

2.1 Numerical Method (Finite Elemental Method)

There are various numerical methods that can be used for different purposes not limited to just geophysical related studies. Over time, many different authors have carried out deformation analysis by using numerical methods. By these methods, parameters can be adjusted to vary to get the specific aim of the study. Nowadays, strength and deformability of fractured rock mass can be calculated with an added flexibility by using numerical methods. Among the methods that are widely used are the Finite Elemental Method (FEM) and the Discrete Elemental Method (DEM).

In this project, the numerical method that will be used is the Finite Elemental Method. This is a numerical method that is first developed by R. Courant who used the Ritz method of numerical analysis and minimization of variational calculus to obtain approximate solutions to vibration systems. Given the improvement of computing power, FEM has now been developed to an incredible precision with its application in industries varying from mechanical, aerospace, fluid flow, automotive and for this purpose rock and soil mechanics. The FEM has developed into a powerful tool in the past decade as evidenced by the many journals or textbooks referencing this topic (Sv.vt.edu, 2015)

This method uses a complex system of points called nodes which makes a grid known as a mesh. This mesh is programmed to contain the material and structural properties which define how the structure will react to certain loading conditions. Nodes are assigned at a certain density throughout the material depending on the anticipated stress levels of an area. A higher node density will be assigned to regions which will receive larger amount of stress. Points of interest may consist of: fracture point of previously tested material, fillets, corners, complex detail and high stress areas. The mesh acts like a spider web in that from each node, there extends a mesh element to each of the adjacent nodes. This web of vectors is what carries the material properties of to the object, creating many elements (*Sv.vt.edu, 2015*).

For example, *Bidgoli et al. (2013)* evaluated the strength and deformability of fractured rocks by numerical modelling using Discrete Elemental Method (DEM). From their study, it was concluded that the strength of fractured rock masses increases with the increase of confining pressure.

Ghoureychi (2001) studied the mechanical behaviour of rock masses using Finite Elemental Method (FEM) modelling by assuming linear elasticity and Mohr-Coulomb strength criteria for both intact rock and fractures. FEM is used for stability analysis by parametrically varying rock joint, persistence, spacing and shear strength parameters until overbreak is reached.

Singh et al. (2004) observed the influence of number of joint set on the anisotropy behaviour of rocks. *Yang et al. (2014)* examined strength characteristics and deformation properties of fractured rock masses using FEM. Numerical results show that deformation modulus decreases by less than 10% as model size increases to 12m in each direction. For their studied fractured rock masses, the numerical result shows that the mechanical characteristics and parameters are consistent and constant after 12m.

Panthee et al (2016) used Finite Element Method (FEM) to model the influence of rock joint persistence, spacing and shear strength on the stability of tunnel and subsequent estimation of parameters that are responsible for creation of maximum zone of overbreak to resemble the field condition.

2.2 Fractured Rock Mass

Knowledge of the behaviours of fractured rock mass are essential especially in rock mechanics or rock engineering. Hence for studies involving fractured rock masses, it is very important to understand the structure of the rock masses. Because of discontinuities (joints, bedding planes among others) controlling their hydro-mechanical behaviours, rock masses are never isotropic *(Norian-Bidgoli, 2014).*

Hudson and Harrison (1997) states in their study that the crystalline rock masses are fractured media and complex materials in nature, consisting of intact rock matrix (block) and rock fractures (discontinuities).

Amadei and Savage (1996) pointed out that the presence of one or several fractures in a rock mass creates anisotropy in its response to loading conditions. *Amadei (1996)* also explained the importance of anisotropy of rock masses and interactions among anisotropy, stress, deformability and strength of a rock mass containing a regular fracture set. However, there is a need to study anisotropy of strength and deformability of fractured rocks more systematically, when complex fracture system geometry needs to be considered.

Bidgoli (2014) states in his study that fractured rocks behave non-linerarly, represented by their elasto-plasticity behaviour with a strain hardening trend. In is study dealing with fractured rock masses, he also stated that the fractures are usually described as assemblages, and classified into sets. This is due to the difficulty of in-situ geological surveys of fractures in the rock masses. Also, stated in his study is that fractures belonging to the same set run almost parallel to each other but could have different hydro-mechanical properties, shapes and sizes form each other.

2.3 Factors on the Strength and Deformability of Fractured Rock Mass

When dealing with engineering designs, constructions, operations and performance safety assessments of surface and subsurface structures in and on rock masses, two important parameters needed to be considered are the strength and deformability of fractured rock mass. Both parameters can be acquainted with the behaviour of rocks.

There are many factors that governs the strength and deformability of fractured rock mass such as the hydraulic and mechanical properties of rock matrices, fracture geometry system and hydro-mechanical properties of fractures. *Sridevi and Sitharam (2000)* concurred from their studies that the behaviour of fractures also depends on the basic morphological, environmental and geological factors of the rock masses being dealt with.

Based on past experimental and numerical studies on some rocks (*Guiterrez et al., 2000; Odedra et al., 2001; Tang et al., 2004; Wang, 2006; Talesnick and Shehadeh, 2007; Wang et al., 2013)*, it was found that the presence of water reduces the strength of the rocks and significantly affects the deformation behaviour of fractured rock masses.

It is also recognised that one of the more important aspects in dealing with rock masses is the sizes or scales of the model defined in representing the rock masses. This can be seen in Figure 2.1 below. *Min, Jing (2003) and Baghbanan, Jing, (2007)* numerically showed that the behaviours and properties of fractured rock masses are strongly defined by its scale. Hence, a realistic representation in terms of scale for the fracture geometry of rock masses is very important for selecting a suitable model size. Thus, the Representative Elementary Volume (REV) concept which can be defined as the minimum volume or range of a sampling size beyond which the mechanical and hydraulic properties of the sampling size remain essentially constant (*Long et al., 1982)* should be used when dealing with studies regarding the behaviours of fractured rock mass. The REV concept mentioned is as shown in Figure 2.2.

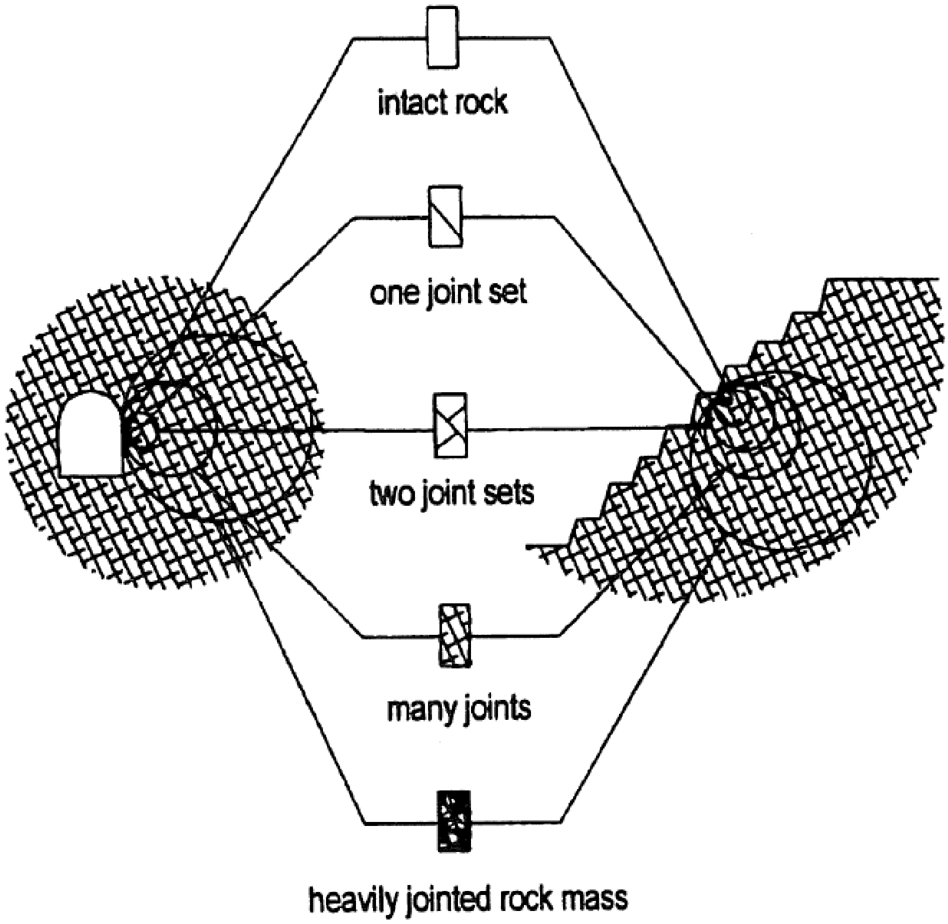

Figure 2.1 *Diagram showing transition from an isotropic intact rock to a heavily fractured rock mass with increasing sample size (Hoek, 1982)*

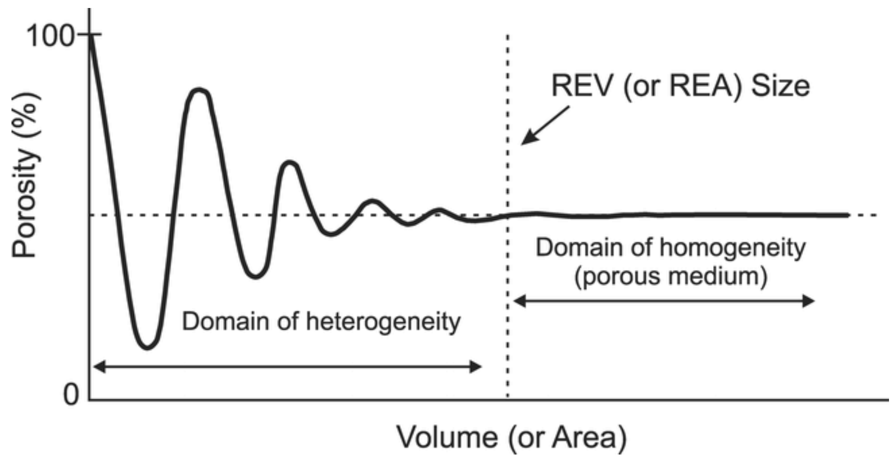

Figure 2.2 Concept of REV

By *Bienawski (1989),* the behaviour of rocks is best presented in a stress – strain curve. It will be noted that initially, deformation increases approximately proportional with increasing load. Eventually, a stress level is reached at which fracture is initiated and starts to propagate. Further increasing the stress leads to another stress level, the critical energy released. At this stage, the crack propagation is unstable and continues even when the stress increase is stopped. Further, if the stresses are sufficiently high, rocks start to behave inelastically, causing fractures in rock mass and overall reduction in the bearing capacity (*Ewy and Cook, 1990)*. Next, the bearing capacity is reached. This is in fact the strength of the rock. Therefore, strength can be described as the maximum load bearing capacity of a rock.

Bienawski (1989) states that deformability is characterised by a modulus describing the relationship between the applied load and the resulting deformation. *Yang et al. (2014)* examined strength characteristics and deformation properties of fractured rock masses using FEM. Numerical results show that deformation modulus decreases by less than 10% as model size increases to 12m in each direction. For their studied rock masses, the numerical result shows that the mechanical characteristics and parameters are consistent and constant after 12m.

Bidgoli et al. (2013) concluded in their study that the strength of fractured rocks increases with increasing confining pressures. Also, the deformation behaviour of rocks follows and elasto-plastic model with a strain hardening trend.

Bidgoli (2014) demonstrates through his studies that the strength and deformation parameters of fractured rocks are dependent on confining pressures, loading directions, water pressures and mechanical and hydraulic boundary conditions. Based on Stochastic analysis, the strength and deformation of fractured rocks are proven to have range of values, not a fixed value. Therefore, this plays an important role in consideration especially in cases where in the rock and fracture parameters exist scatter.

Rutqvist and Stephansson (2003) describes an important aspect about hydro-mechanical couplings in fractured rocks. Most past attempts considering influence of water on the deformability of rock masses did not consider the applicability of the effective stress concepts for fractured rocks under different loading conditions. They stated that loading conditions plays and important part in the deformability modulus of fractured rock mass.

2.4 Strength of Rock Mass

Hoek et all (2002) states that in some cases, it is handy to consider the overall behaviour of a rock mass instead of the process of initiating failure at a point at its propagation followed by the stabilization. As a result, the concept of rock mass strength is recognised. *Hoek et all (1998)* proposed that estimation of rock mass strength from the Mohr – Coulomb relationship.

$$\sigma'_{cm} = \frac{2c' cos\emptyset'}{1 - sin\emptyset'}$$

With c' and Φ' determined for the stress range $\sigma_t < \sigma'_3 < \sigma_{ci} / 4$ giving the equation:

$$\sigma'_{cm} = \sigma_{ci} \frac{\left(m_b + 4s - a(m_b - 8s)\right) (\frac{m_b}{4} + s)^{a-1}}{2(1 + a)(2 + a)}$$

2.5 Deformability of Rock Mass

Deformability means the capacity of rock to strain under applied loads or in response to unloads on excavation. *Goodman (1980)* states that due to the capability of locally large rock displacements to raise stress within structures, the strains in rock should be taken into consideration. *Hoek et al (2002)* defined the rock mass modulus of deformation as follow:

$$E^{m}(GPa) = \left[1 - \frac{D}{2}\right]\sqrt{\frac{\sigma_{ci}}{100}}\,.\ 10^{\left(\frac{GSI-10}{40}\right)}$$

The deformability of rock masses plays an important role in the design of a few types of structures, because their behaviour is mostly dependent on the displacements undergone by the rock mass. *Bruno et all (2010)* states that for the design of these kind of structures built in, on or of rock masses, it is simply not enough to just characterize the rock mass deformability by just using laboratory tests and inferring the results by using rating systems such as the RMR, Q or GSI. Hence, in situ tests are very important and deformation analysis can be done numerically by using RS2.

2.6 Rocscience2 (RS2)

Phase2 is a two-dimensional finite element programme for modelling of soil and rock. Its applications are mainly in the fields of geotechnical, geomechanics and in civil and mining engineering (Rocscience.com, 2015). It is a programme initially developed for underground excavation simulation but subsequently, new features have been added for other means as well. Such new feature is it now provides material models such as Mohr-Coulomb, Hoek-Brown failure criteria that can be used to represent the rock systems (*Cai et al. 2001 & 2007)*.

Kutalike (1985); Zertsalov, Sakaniya (1997); Pouya, Ghoreychi (2001); Yan et al (2014) have all used this method to study the strength of fractured rock masses. *Bienawski (1978)* applied this method to study the deformability of rocks conducted on three major projects in South Africa: The Orange River Water Project, the Drakensberg Pumped Storage Scheme and the Sandsberg Pumped Storage

Scheme. *Schubert and Schubert (1993)* studied the effect of geological structure on the deformation of rock mass using this software.

2.7 Failure criterion

2.7.1 Hoek Brown Failure Criterion

Hoek and Brown (1980) developed their failure criterion to provide input data for the analyses required for the design of underground excavations in hard rock. *Hoek (1968) and Brown (1970)* derived the criterion from the results of research into the brittle failure of intact rock on model studies of jointed rock mass behaviour respectively. The criterion started from the properties of intact rock and then introduced factors to reduce these properties based on the characteristics of joints in a rock mass. Hoek and Brown chose one of the available rock mass classification schemes which is the Rock Mass Rating (RMR) proposed by Bienawski (1976) to link the empirical criterion to geological observations. *Hoek and Brown (1988)* then introduced the idea of 'disturbed' and 'undisturbed' rock masses as they found it necessary to re-examine these relationships and to introduce new elements from time to time to account for the wide range of the practical problems to which the criterion was being applied. *Hoek et all (1992)* came out with a modified criterion to force the rock mass tensile strength to zero for very poor quality rock masses to enhance its applicability. The original equation of Hoek-Brown criterion is;

$$\sigma_1' = \sigma_3' + \sigma_{ci} \left[m \frac{\sigma_3'}{\sigma_{ci}} + s \right]^{0.5}$$

Where,

σ_1' *and* σ_3' = the major and minor effective principal stresses at failure

σ_{ci} = the uniaxial compressive strength of the intact rock material m and s = material constants, where s = 1 for intact rock.

One of the main difficulties with this particular equation was that it does not deal with shear and normal stresses which are more conveniently used in facing geotechnical problems. It was also noted that the Rock Mass Rating system of Bienawski is no longer capable as a system to realte the failure criterion to the geological observations in the field, especially when it involves very weak rock masses. Hence, *Hoek et all, 1992* introduced the Geological Strength Index (GSI)

Hoek et al (2002) proposed and application of the Generalized Hoek Brown criterion. It is an empirical failure criterion which establishes the strength of rocks in terms of major and minor principal stresses and predicts strength envelopes that agree well with values determined from laboratory tests and from observed failures in jointed rock masses. (Rocscience.com 2017)

The Generalized Hoek-Brown criterion is non -linear and relates the major and minor effective principal stresses according to;

$$\sigma_1' = \sigma_3' + \sigma_{ci} \left[m_b \frac{\sigma_3'}{\sigma_{ci}} + s \right]^a$$

Where m_b is a reduced value of the material constant m_i and is given by:

$$m_b = m_i \exp\left[\frac{GSI - 100}{28 - 14D}\right]$$

s and a are constants for the rock mass given by the following:

$$s = \exp\left[\frac{GSI - 100}{9 - 3D}\right]$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right)$$

D is the factor which depends on the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses. Guideline for estimating the disturbance factor can be seen in Table 2.1. GSI relates the geological observations in the field to the failure criterion. This will be further explained in 2.8.

Table 2.1: Guidelines for estimating disturbance factor D (*Hoek et all, 2002)*

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	$D = 0$
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	$D = 0$ $D = 0.5$ No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	$D = 0.8$
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	$D = 0.7$ Good blasting $D = 1.0$ Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	$D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation

2.7.2 Mohr-Coulomb Criterion

The Mohr-Coulomb criterion explains a linear relationship between normal and shear stresses (or maximum and minimum principal stresses) at failure. This failure criterion is the most common as it is widely encountered in geotechnical engineering. Many geotechnical analysis methods and programs require use of this strength model (*Rocscience.com 2017).*

By using the concept of cohesion (i.e. the shear strength of the rock when no normal stress is applied) and the angle of internal friction (equivalent to the angle of inclination of a surface sufficient to cause sliding of super incumbent block of similar material down the surface), the Mohr envelope is generated (*Harrison and Hudson, 2000*).

Hoek et al (2002) stated that the Mohr-Coulomb shear strength, for a given normal stress σ, is found by substitution of these values of c' and Φ' into the equation:

$$\tau = c' + \sigma \tan \emptyset'$$

The equivalent plot in terms of the major and minor principal stresses is defined by

$$\sigma_1' = \frac{2c' \cos \emptyset'}{1 - \sin \emptyset'} + \frac{1 + \sin \emptyset'}{1 - \sin \emptyset'} \sigma_3'$$

Hoek et all (2002) found it necessary to determine equivalent angles of friction and cohesive strengths for each rock mass and stress range since most geotechnical software is still written in terms of the Mohr-Coulomb failure criterion and they fit an average linear relationships to the curve generated by solving

Hoek-Brown Criterion fo r a range of minor principal stresses defined by $\sigma_t < \sigma_3 < \sigma_{3max}$' , as shown in Figure 2.2.

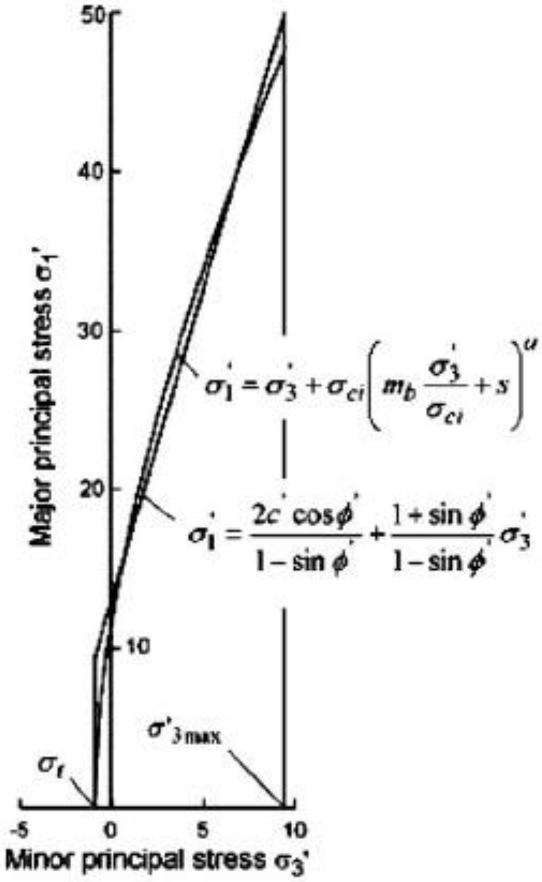

Figure 2.3 *Relationships between major and minor principal stresses for Hoek-Brown and equivalent Mohr-Coulomb criteria (Hoek et al, 2002)*

1.6 Rock Quality Designation

Rock-quality designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mass, measured as a percentage of the drill core in lengths of 10 cm or more. High-quality rock has an RQD of more than 75%, low quality of less than 50%. Rock quality designation (RQD) has several definitions. The most widely used definition was developed in 1964 by D. U. Deere. It is the borehole core recovery percentage incorporating only pieces of solid core that are longer