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Evaluation of the Bekhme Dam Site – NE Iraq using the Proposed Reduction System of the Rock Mass Strength

Azealdeen S. Al-Jawadi [1], Ibrahim S. Al-Jumaily [2], Thanoon H. Al-Dabbagh [3], Colin Davie [4]

[1,2] *College of Petroleum and Mining Engineering-University of Mosul, Iraq.*
[3] *College of Science- University of Mosul, Iraq.*
[4] *College of Civil Engineering and Geosciences-Newcastle University, UK.*

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Correspondence:
Name: Azealdeen S. Al-Jawadi

Email:
azealdeenaljawadi@uomosul.edu.iq

ABSTRACT

The proposed Reduction System of the Rock Mass Strength RSRMS depends on the effects of discontinuities to reduce the strength of rocks. The reduction of rock mass strength happened when the properties of discontinuities are inferior. Most classification systems worldwide used are employing engineering parameters separately. The association of the related parameters and subsequent correlations are the base of the RSRMS, which forms part of the output of the present work. The RSRMS would be applied systematically in multiple stages to arrive at the final view of the site. The system can be applied at any engineering site that has rock mass varied from soil properties to intact mass. The application of RSRMS at the Bekhme Dam Site – NE Iraq has clarified the separation between the zones of different rock mass quality along the Bekhme Gorge, Spillway, and Access tunnels. The evaluations of rock mass matched the common worldwide used rock mass classification systems. The proposed dam site is classified as high quality by RSRMS, which is classified between 4-10 according to Q-System, high according to RMR, and 10-1 according to RMi. At the Spillway Tunnel, three small zones have very low grades at the distance from the SW entrance, two zones have a low grade, and two zones have a medium grade. The best qualities extend to the eight long zones and five zones are having very good quality. At the Access Tunnel, there are two very low-grade zones at the distance from the SW entrance, six low-grade zones, and eight medium-grade zones. Approximately half the length of the tunnel has good and very good quality for five zones and four zones have very good.

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تقييم موقع سد بخمة – شمال شرق العراق باستخدام نظام التخفيض المقترح لقوة كتلة الصخور

عزالدين صالح الجوادي[1] ، إبراهيم سعد الجميلي[2] ، ذنون حامد الدباغ[3]، كولين ديفي[4]
[1,2] كلية هندسة النفط والتعدين، جامعة الموصل، العراق
[3] كلية العلوم، جامعة الموصل، العراق
[4] كلية الهندسة المدنية وعلوم الأرض ، جامعة نيوكاسل ، المملكة المتحدة.

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المراسلة:
الاسم: عزالدين صالح الجوادي
Email: azealdeenaljawadi@uomosul.edu.iq

الملخص

يعتمد نظام التخفيض المقترح لـ RSRMS لقوة الكتلة الصخرية على تأثيرات الانقطاعات لتقليل قوة الصخور . يحدث انخفاض قوة كتلة الصخور عندما تكون خصائص الانقطاعات أقل شأنا. تستخدم معظم أنظمة التصنيف المستخدمة في جميع أنحاء العالم العوامل الهندسية بشكل منفصل. يعتبر ارتباط العوامل ذات الصلة والارتباطات اللاحقة أساس نظام RSRMS، والذي يشكل جزءًا من ناتج العمل الحالي. سيتم تطبيق RSRMS بشكل منهجي في مراحل متعددة للوصول إلى العرض النهائي للموقع. يمكن تطبيق النظام في أي موقع هندسي تتنوع فيه كتلة الصخور من خصائص التربة إلى الكتلة السليمة. أوضح تطبيق RSRMS في موقع سد بخمة – شمال شرق العراق الفصل بين مناطق جودة كتلة الصخور المختلفة على طول مضيق بخمة، ونفقي المرور والمسيل المائي. تطابقت تقييمات كتلة الصخور مع أنظمة تصنيف كتلة الصخور الشائعة في جميع أنحاء العالم. تم تصنيف موقع السد المقترح على أنه عالي الجودة من قبل RSRMS، والذي تم تصنيفه بين 4-10 وفقًا لنظام Q-System، مرتفع وفقًا لنظام RMR، و 1-10 وفقًا لنظام RMi. شخصت ثلاث مناطق صغيرة لها درجات منخفضة للغاية في نفق المسيل المائي على مسافة من مدخل الجنوب الغربي، ومنطقتان لهما درجة منخفضة، ومنطقتان بهما درجة متوسطة. تمتد أفضل الصفات إلى ثماني مناطق طويلة وخمس مناطق تتمتع بنوعية جيدة جدًا. في نفق الوصول، توجد منطقتان منخفضتان للغاية على مسافة من مدخل جنوب غرب، وست مناطق منخفضة الدرجة، وثماني مناطق متوسطة الدرجة. ما يقرب من نصف طول النفق يتمتع بجودة جيدة وجيدة جدًا لخمس مناطق ولأربع مناطق جيدة جدًا.

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Introduction

The rock mass strength is reduced due to undermining the discontinuity properties, e.g., roughness, openness, persistence, healing, spacing as well as orientation, and normal stress. Initially, the principles of the classification system depend on dividing the site into zones having different characteristics at the field according to the overall look. Rock masses in different zones are classified as continuous if they are rarely fractured (intact) or highly fractured (crushed), while it classifies as discontinuous if the density of fractures is normal. Each zone should be studied in the field by the syllogism of discontinuity characteristics and sampled for laboratory tests. The discontinuous zones were investigated for rock mass classification to obtain the reduction factors that weaken the rock mass strength. The Rock Mass Strength Reduction System RSRMS mainly branches into two paths depending on the type of laboratory and field tests. The field tests are divided into two kinds that are measurable and portrait or complementary tests. Measurable parameters are the backbone of this system that related together to deduce the reduction factors. There are five measurable parameters for discontinuities; they are continuity, attitude (dip and strike), spacing, openness, and roughness. The reduction factors are scale, morphology, persistence, and orientation obtained from the measurable parameters and the reduction factors ranged from zero to one. The laboratory tests are physical, mechanical, static, dynamic, petrography, and mineralogy. The three types of rock mass strength i.e., compression, tensile, and shear that are appreciated or measured would be reduced by multiplying the strength by the reduction factors. The scale and orientation factors are illustrating the type of failure, which may be, typify compression, tensile, or shear.

Shear strength of discontinuities for example depends on several parameters at the same time, e.g., roughness, openness, persistence, healing as well as orientation, and normal stress. The reliability of the site investigation depends primarily upon the extent of the proposed works and the nature of the site, i.e., scale. The spacing of discontinuities is a parameter of most classification systems, while the relation between spacing and engineering structure size is of even more significance. The quality of discontinuity surfaces such as roughness, openness, type, and degree of filling materials, moisture condition, weathering, and wall strength are complementary characteristics. Density and frequency of discontinuities can be included along with the relationship between scale and spacing. The persistence of discontinuities is the factor controlled by the expected daylight of discontinuities on the free surface of the engineering body.

Site investigation for rock mass classification depends on the stage of the engineering project (Brand, 2000). Each stage of the engineering project requires some parameters that may be different from those of other stages. The stages of an engineering project are four, preplanning, planning, construction, and post-construction. Basic data collected from the field and that obtained from the laboratory for all four stages were approximately the same. Formulated parameters for classification systems may vary according to the application. The application of well-known rock mass classification systems to prepare different kinds of engineering geological maps is to take place in the preplanning stage. Stages of planning and construction require some parameters that differ from what is ordinary. The post-construction stage deals with the treatment of the problems that appear or that may be expected.

Objective

Parameters obtained from the RSRMS classification surpass simplicity and are easy to use for providing quantitative data. Simple and easy calculations are useful for the treatment of a huge amount of data in a short time. The strength of rock masses depends on the strength of the intact pieces and on their discontinuities, which in turn, depends on the number, orientation, spacing, and strength of the discontinuities. The understanding of the problem for estimating the strength of jointed rock masses depends on the strength of the intact pieces and their freedom of movement along discontinuities (Hoek, 1983). For engineering purposes, there is no single parameter or index that can fully and quantitatively describe a rock mass (Bieniawski, 1989). The critical analysis of the experimental methods in the classification of rock blocks emphasizes the importance of applying these classifications in designing engineering structures and calculating their safety coefficient (Yang, and Elmo, 2022). This proposed classification is very close to the possibility of being used in classification and safety coefficient calculation. The idea of this classification is close to the idea of the rock mass deformation coefficient (Hussain, et al., 2022) which can be used in its assessment of the sustainable design of engineering structures.

Methodology

Zonation

At the beginning of any study to evaluate the rock mass, the site must first be divided into zones according to the dissimilarity of rock mass in the field. Differences in geologic structure, lithology, weathering, morphology, hydrogeology and other field conditions delineate the boundaries between zones (Fig. 1). The reconnoitering survey helps the investigator to make the primary decision for which path the site investigation will take. All structural geology and engineering geology data must be obtained from the field according to the field form (Table. 1). Tests on rock samples are also performed in the laboratory (Fig.1) to extract the parameters of any classification system. Differences in geologic structure, lithology, weathering, morphology and other field conditions delineate the boundaries between zones (Fig.1).

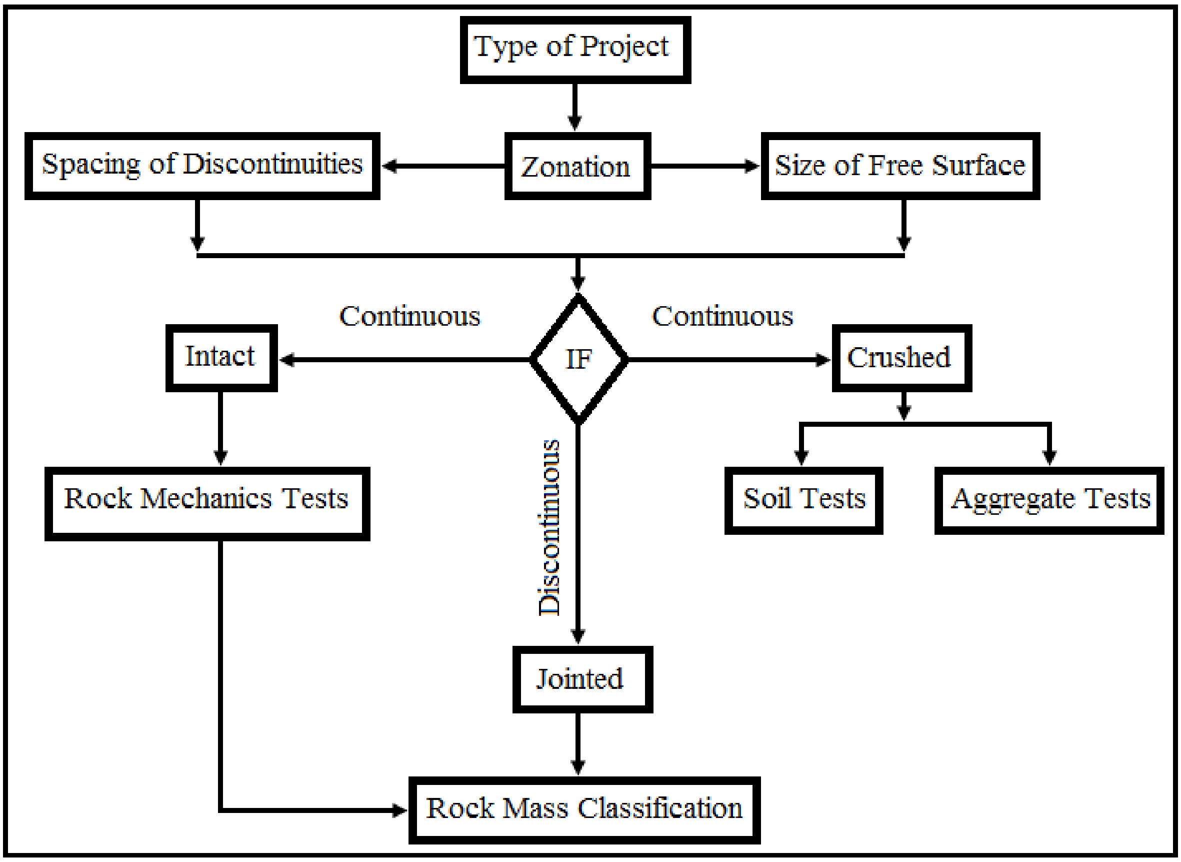


Fig.1. The first step for entrance to the rock mass classification.

Prediction of failure type

The relationship between shape, size, and attitude of the restricted blocks between discontinuities and the free surface of engineering structure predicts the type of failure (Goodman and Shi, 1985) (Fig. 2). Visualization of blocks and free surface relation in the field inflicts some difficulties. From the complimentary desk study, workers can usually predict the type of failure.

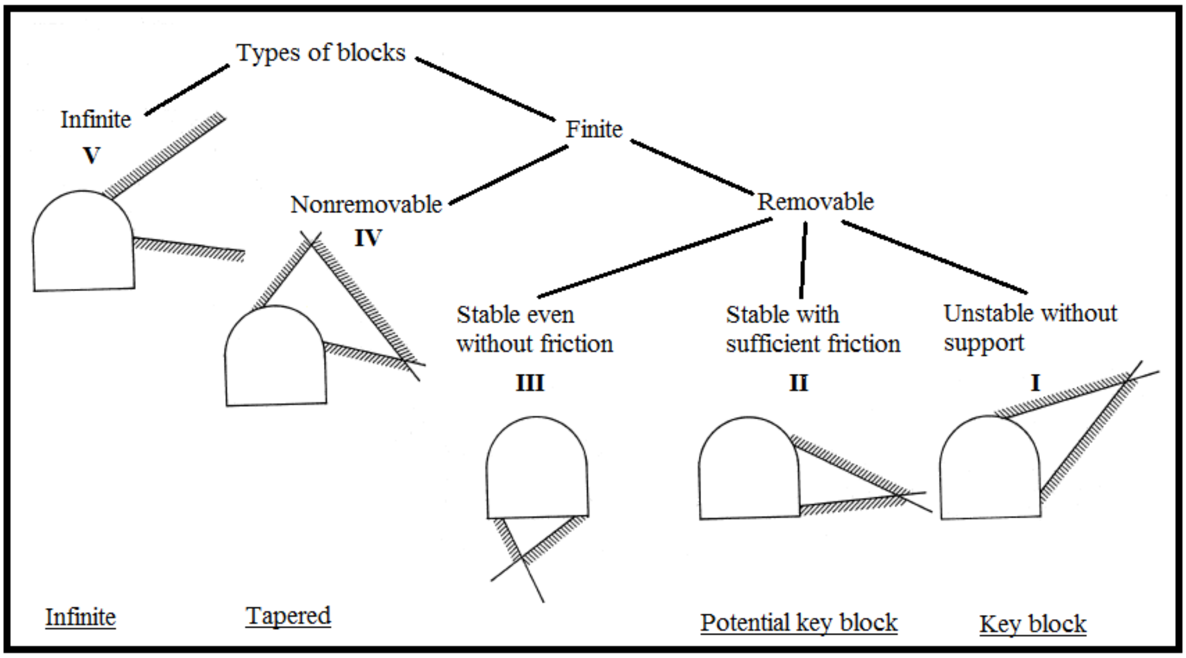


Fig.2. Classification of blocks that are based on block theory (Goodman and Shi, 1985).

Plotting of geological structures and free surfaces of engineering structures on orthographic projection or stereographic lower hemisphere projection are useful methods to assess the shape and direction of block failure. Hammett and Hoek, 1981 explain the potential failure mechanisms in a vertical wall and horizontal roof in the orthographic and stereographic diagrams. Block posture refers to the stability, flexure, tensile, compressive, and shear failure related to gravity and internal forces (Hoek and Bray, 1989). The most probable failures of rock blocks at the free surface of engineering structures are shear, fall, and burst. Shear failure mostly depends on discontinuity characteristics, while the others depend on intact rock and/or discontinuity characteristics.

Reduction factors

From the combination of different related parameters, reduction factors of rock mass strength can be obtained. The reduction factor is the average of all reduction factors that are the percent of one time the intact rock strength. Singh, 1979 defined the modulus reduction factor MRF (Fig. 3) as a ratio of the deformation modulus of a rock mass E_d to the elastic modulus of the rock material E_r obtained from the core. Thus, the deformation modulus of a rock mass can be determined as a product of the modulus reduction factor corresponding to a given rock mass rating and the elastic modulus of the rock material from the equation in Fig.3 (Singh, 1979). Other researchers suggested different equations for the reduction factor (Bieniawski, 1978; Nicholson and Bieniawski, 1990; Hoek and Brown, 1997 and others). Reduction factors were formulated from measurable parameters (free surface extension, discontinuity attitude, spacing, openness, roughness, and continuity). Other field and laboratory rock mass classification parameters are complementally parameters (Fig.4).

Table 1. Field form that is used to record the discontinuity parameters.

Project		*Location*		*Recorded by*			*Date*	
Traverse length		*Northing*		*Easting*			*Trend*	
Unit designation	*Mineralogy*	*Rock type*	*Degree of weathering*	*Hardness*	*Texture*	*Color*	*Alteration*	*Primary porosity*

Fracture type	Dip	Strike	C	E	R	M	H	W	O	T	HL

Parameter	Code	Description
Continuity	C1	<1m.
	C2	1-3m.
	C3	3-10m
	C4	10-30m.
	C5	>30m.
Ends	E0	No ends visible
	E1	One end visible
	E2	Both ends visible
Roughness	R1	stepped
	R2	Rough
	R3	M. rough
	R4	S. rough
	R5	Smooth
	R6	Polished
Moisture	M1	Dry, not possible
	M2	Dry, no evidence
	M3	Dry, some evidence
	M4	Damp, no free water
	M5	Wet, some drops
	M6	Cont. flow, low pres.
	M7	Cont. flow, high pres.
Wall hardness	H1	Extremely hard
	H2	Very hard
	H3	Hard
	H4	Moderately hard
	H5	Moderately soft
	H6	Soft
	H7	Very soft
Wall weathering	W1	Fresh
	W2	Slightly weathered to fresh
	W3	Slightly weathered
	W4	Moderately to slightly weathered
	W5	Moderately weathered
	W6	Intensely to moderately weathered
	W7	Intensely weathered
	W8	Very Intensely weathered
	W9	Decomposed
Openness	O1	Tight
	O2	<1mm.
	O3	1-3mm.
	O4	3-10mm.
	O5	10-30mm.
	O6	>30mm.
Thickness	T0	None
	T1	<1mm.
	T2	1-3mm.
	T3	3-10mm.
	T4	10-30mm.
	T5	>30mm.
Healing	Hl0	Completely, to strength of wall rock
	Hl2	>50% or weaker than wall rock
	Hl3	<50%
	Hl5	No indurations

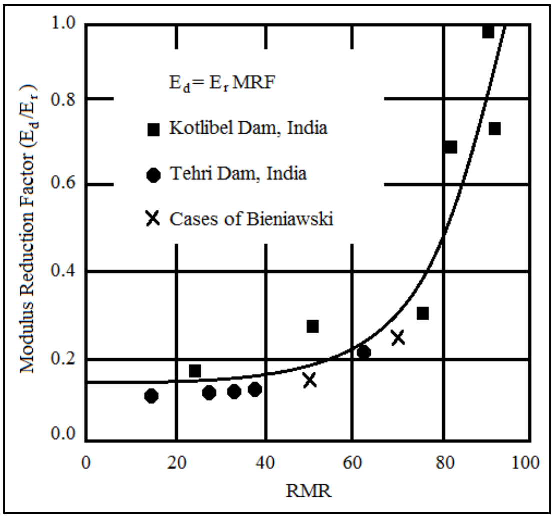


Fig.3. Relationship between rock mass rating RMR and modulus reduction factor MRF (Singh, 1979).

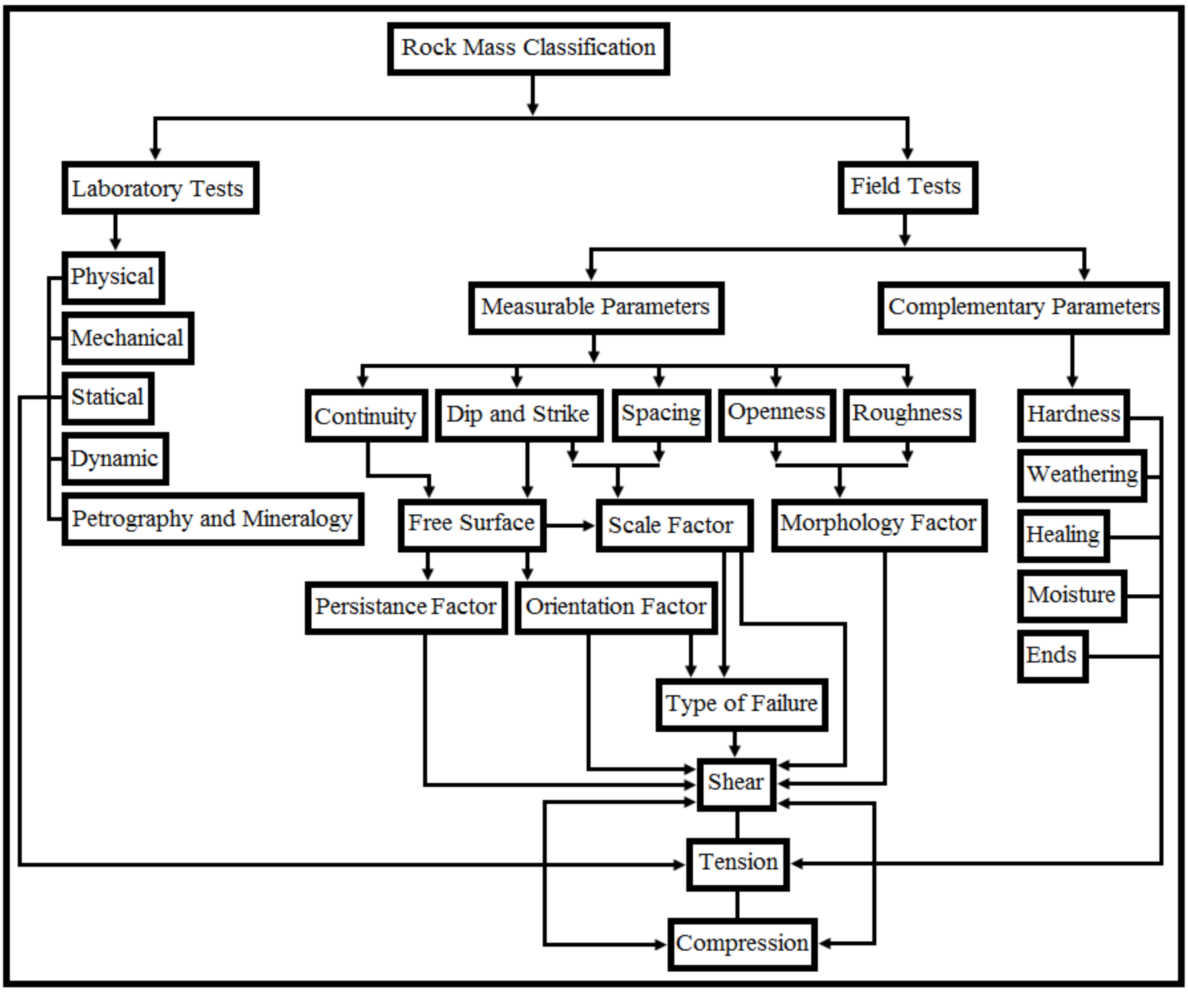


Fig.4. Flowchart of the RSRMS rock mass classification system.

Scale effect

Many researchers have studied the scale effect on the evaluation of rock masses previously. Scale effect depends on the size of the engineering project (extent of free surfaces) and spacing of discontinuities. Discontinuity set spacing is the distance between individual discontinuities within a set. The smallest and biggest spacing give continuous crushed and intact rock mass respectively. Convergence between size and spacing gives a discontinuous rock mass (Fig.5). The shear strength also decreases on rock discontinuities rather than on small-scale rocks (Borri-Brunetto, 2004).

Block size restricted between sets of discontinuities is an extremely important indicator of a rock mass. Large blocks tend to be less deformable and develop favorable arching and interlocking in underground openings. In the case of slopes, the small block size may cause rotational slides instead of structurally controlled modes of failure (Sonmez and Ulusay, 1999). Kovari, 1979 presents the influence of the ratio between the span of the tunnel and the average spacing of discontinuities that is decisive, in many cases, for stability considerations (Fig.6). The extent of the free surface of any engineering structure represents the value of the span D in Kovari explanation. With increasing span, or the ratio between span and spacing D/d respectively, the influence of the jointing becomes more marked and the probability of an unfavourable joint combination could give rise to increased rock mass failure (Kovari, 1979).

The Scale Reduction Factor SRF derived as the ratio between discontinuity spacing and free surface extent, takes into account the angle between them.

$$SRF = d \sin^{-1} \alpha / D \qquad 1$$

Where:

d is the spacing of discontinuities,

D is the free surface extent, and

α is the angle between discontinuity and free surface planes.

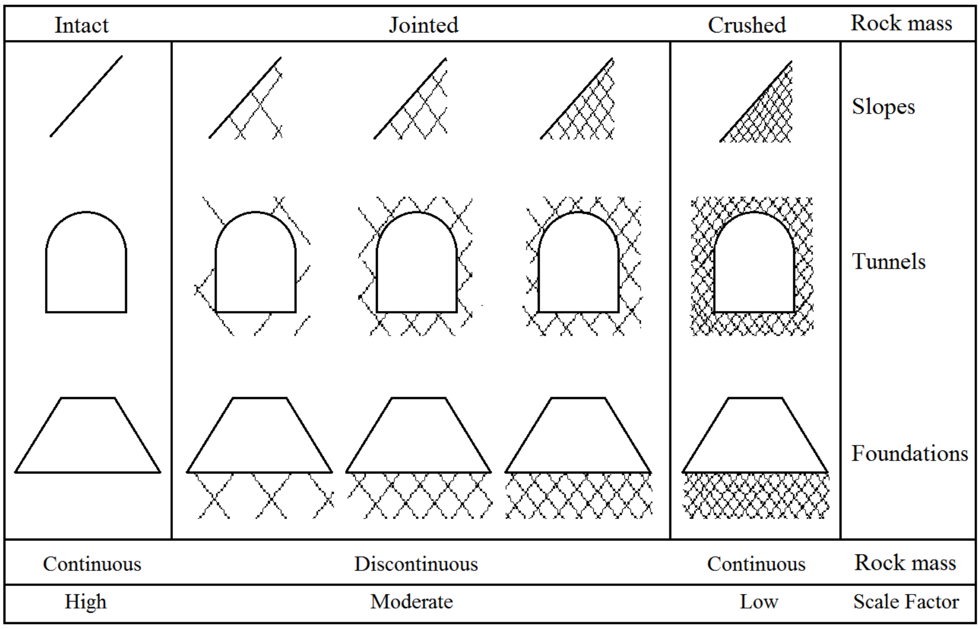


Fig.5. Various spacing of discontinuities of the same engineering project size reproduce various amounts of scale reduction factor SRF.

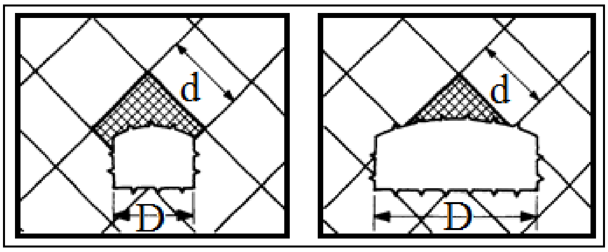


Fig.6. Influence of the span D on the stability in jointed rock (Kovari, 1979).

Equation 1 shows a linear relation between scale factors plotted as ordinate and a sequence value of spacing values for the constant value of free surface extent as abscissa (Fig.7). In the case of application of the scale factor, the rock quality designation RQD, block volume V_b, volumetric joint count J_v and joint number J_n conjoined with it. In addition, block shape, joint set, the orientation of the main joint set, and thickness of weak zones are sometimes conjoining with the scale factor.

A New Engineering Classification System (Capigian and Al-Khateeb, 2008) for rock according to the number of fractures per meter can also be conjoining with the scale effect. The difficulty of scanning discontinuities in three dimensions leading to assuming the rock mass is homogeneous for J_v calculation (Sonmez and Ulusay, 1999).

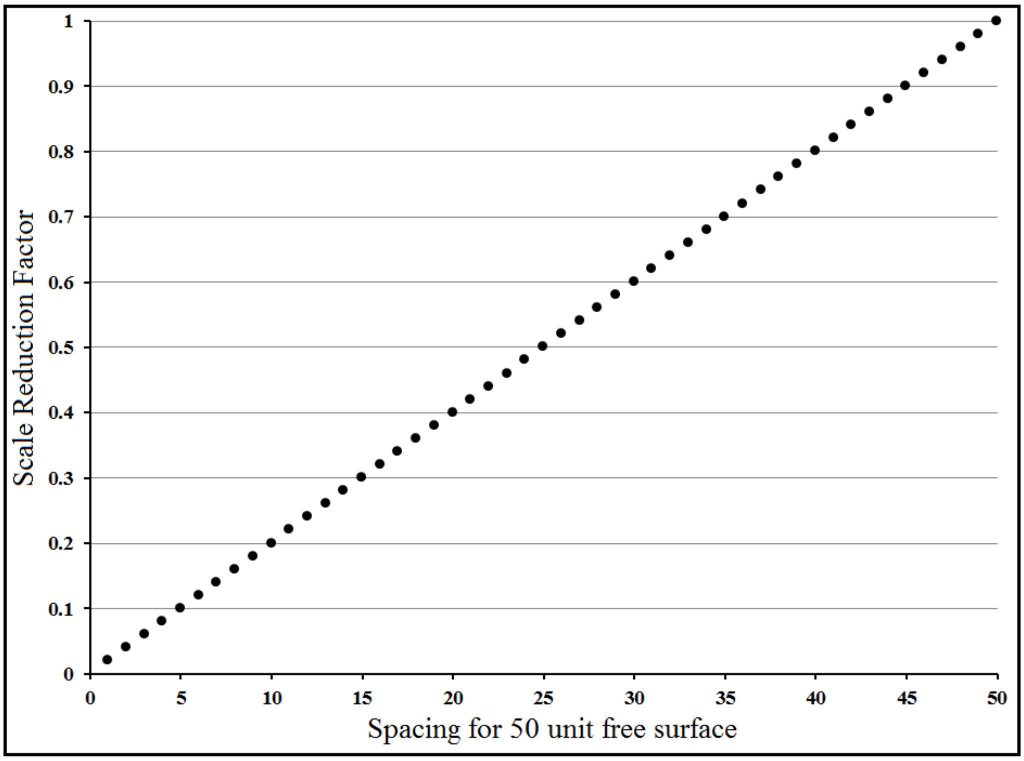


Fig.7. Scale reduction factor for 50-unit free surface.

Openness and roughness effect

The relationship between openness and shear strength of discontinuities is inversive. Contrarily, the relation between asperities amplitude with shear strength is extrusive. Increasing openness or decreasing asperities amplitude serves to facilitate easy movement on discontinuity planes. The shear strength drops as a hyperbolic function of the ratio between infilling thickness (openness) and asperity height (Indraratna, et al., 2005). The rating of discontinuity description for the RMR_{76} classification system drop from 25 to 0 when the openness increases from 0 to 5mm. or more (Bieniawski, 1976). The relation between openness and roughness was formulated as the Morphology Reduction Factor MRF:

$$MRF = (r - o)/r \qquad 2$$

Where:

r is the asperities amplitude, and

o is openness.

Equation 2 represents the morphology effect for one unit of openness, while if openness equals asperities amplitude the value of the morphology reduction factor will be zero, so the shear strength is not related to this factor. Shear strength in this case is dependent on the adhesion of the filling grains. This equation gives the logarithmic relation between a series value of asperities amplitude and morphology effect with the constant value of openness (Fig.8).

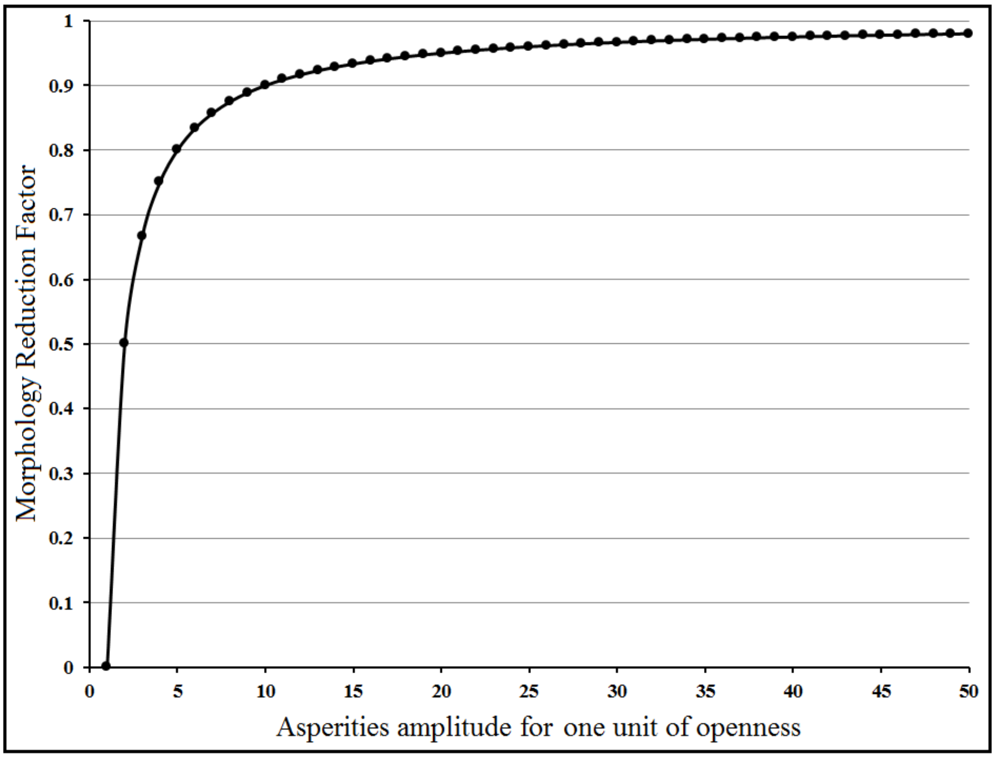


Fig.8. Morphology reduction factor for one unit of openness.

The profile that was published by Barton and Choubey (1977) and its origin (Barton, 1976) omits the effect of openness. In the case where asperities amplitudes equal the openness or are less than it is, the morphology factor will be zero (Fig.9).

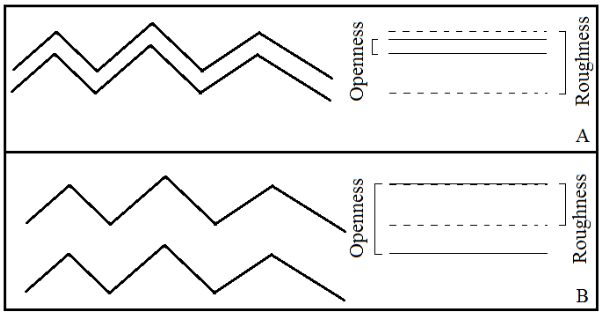


Fig.9. Relationship between roughness and openness A: roughness greater than openness refers to high shear strength, B: roughness smaller than openness refers to no effect of roughness to shear.

Orientation effect

The relationship between the discontinuity attitude and the free surface of the engineering structure is more decisive. This relation relates to the type of expected failure (Fig.2). Discontinuities orientation can be critical to the deformation or stability of engineering structures concerning applied loads (USDIBR, 2001). The existence of one set or two sets of discontinuities in the rock mass is infrequent as there are usually three sets. Most sedimentary rocks contain bedding planes and at least two sets of joints that present three sets of discontinuities (Van der Pluijm and Marshak, 2004). Deformed igneous and metamorphic rocks also contain many sets of joints, while if the rock mass has rare discontinuities, it will treat as intact. The case of shear failure most probably occurs when the angle between the free surface and discontinuity plane exceeds zero, approximately between 20°-70° (Ramsay and Huber, 1987). The lowest value of major principal stress at fracture to uniaxial tensile strength for uniaxial and triaxial loads is 30°, while it exceeds on both sides of the angle increases or decreases (Hoek, 1964). The angle of internal friction of most rocks will vary from about (75°-80°) down to (20°-25°) (Barton, 1973). The values out-of-range of these angles point to tension or compression failure. Equation 3 explains the Orientation Reduction Factor ORF of shear strength reduction according to the angle between discontinuity and free surface (Fig.10).

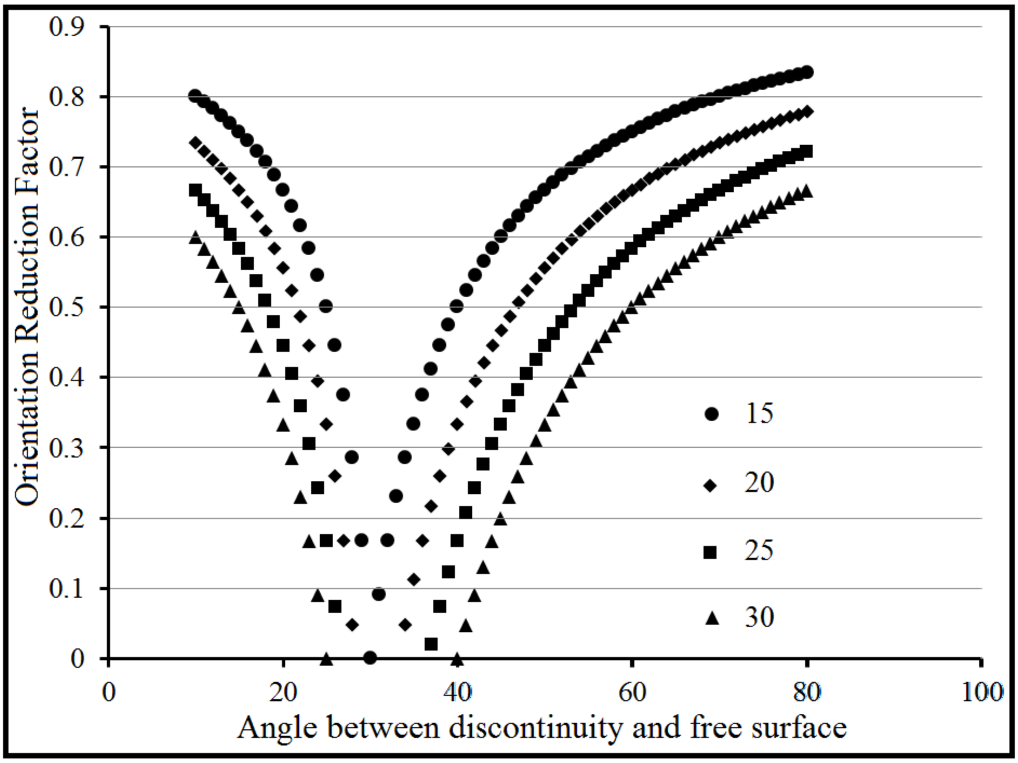


Fig.10. Orientation reduction factor for angles 0-90 between discontinuity and free surface.

$$ORF = (\alpha - 15)/\alpha \qquad \text{3a}$$

$$ORF = (\alpha - 20)/\alpha \qquad \text{3b}$$

$$ORF = (\alpha - 25)/\alpha \qquad \text{3c}$$

$$ORF = (\alpha - 30)/\alpha \qquad \text{3d}$$

Where:

α is the angle between discontinuity and free surface planes.

Ramamurthy, 1994 published the inclination effect as a joint inclination parameter that can be obtained from Fig.11 shown in the graph included with its table. The parameters of this figure somewhat are the same as that can be obtained from Fig.10.

Inclination of joint in degrees	Joint inclination parameter
0	0.82
10	0.46
20	0.11
30	0.05
40	0.07
50	0.31
60	0.46
70	0.63
80	0.82
90	1.00

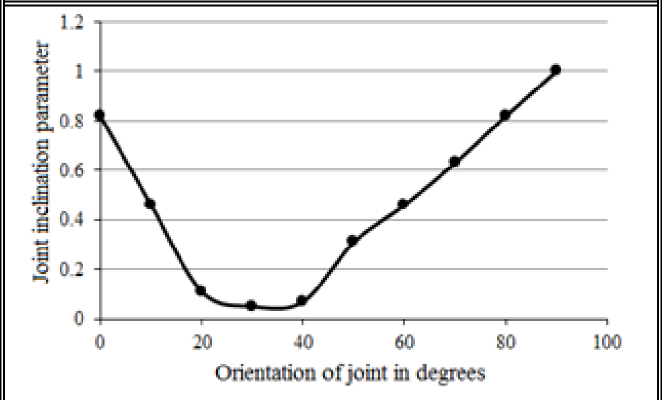


Fig.11. Joint inclination parameter (after Ramamurthy, 1994).

Persistence effect

A continuous discontinuity is weaker and more deformable than disjunctive short discontinuities that are bridged by intact rock. Recording trace lengths to describe persistence is useful in large exposures because persistence is a difficult parameter to measure (Einstein, *et al*., 1983). Identification of the more continuous fractures is an important aspect of formulating rock stability input data, especially for high-cut slopes and large underground openings (USDIBR, 2001). The persistence Reduction Factor PRF is the ratio of the difference between free surface extent and persistence to the free surface extent (Equation 4).

$$PRF = (fs - p)/fs \qquad 4$$

Where:

fs is the free surface extension, and

p is persistence.

The presence of a rock bridge between discontinuities may change the failure type from shear to tension or compression. The relation between persistence reduction factors with persistence values is represented in Fig.12. The relation is for the persistence of discontinuities from zero to 50 units on a free surface that also extends to 50 units. This relation shows the acceleration of the persistence reduction factor when the discontinuity extension reaches near the extension of the free surface. This may be due to the weakness of the rock bridge through discontinuity propagation.

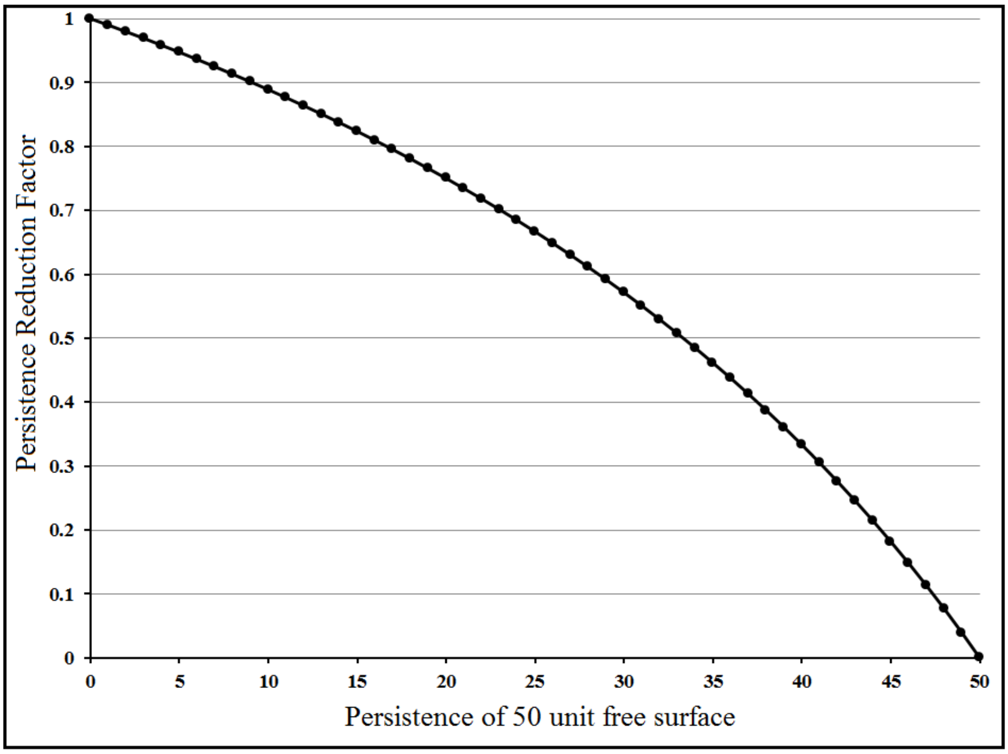


Fig.12. Persistence reduction factor for 50 units of the free surface.

Complementary parameters

The engineering characteristics of rocks are complex due to the varied physical, chemical, and tectonic processes associated with the formation of rock mass in time and space. After formation, many processes act on the rock mass due to changes in environmental elements. In addition to reduction factors that affect rock mass strength, complementary parameters are considered for shear, tensile and compressive failure. The field complementary parameters are hardness, weathering, healing, moisture, and ends of discontinuities are concluded in Table 2.

Table 2. Reduction factors of complementary parameters.

Parameters	**Reduction Factors**				
	>0-0.2	0.2-0.4	0.4-0.6	0.6-0.8	0.8-<1
Hardness	Very Low	Low	Medium	High	Very High
Weathering	Very High	High	Medium	Low	Very Low
Healing	Not Healed	Partially 50% Soft	Completely Soft	Partially 50% Hard	Completely Hard
Moisture	High Flow	Low Flow	Damp	Wet	Dry
Ends	No End Visible		One End Visible		Both Ends Visible

The complementary parameters are measured on or near the adjacent walls of discontinuities that affect the failure of the rock mass. Suggested methods for estimating the compressive strength of rock surface were published by the Williamson, 1984 (Fig.13). In cases where MRF is near zero and the normal stresses are low, hardness and weathering are not important. The alteration of the discontinuity wall nearly always will be accompanied by infill material, which will, generally, have lower shear strength than the altered wall material (Hack and Price, 1995). Weathering negatively influences the engineering properties of rock (Farah, 2011), which decreases the strength of discontinuity wall and filling materials. The presences of water through discontinuities further in pores decrease the rock mass strength and the intact rock strength respectively.

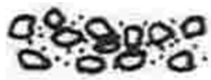
Code	Abbreviation	Meaning and estimated strength	Illustration
A	RQ	Rebound reaction to hammer blow (>103 MPa)	
B	PQ	Pits with hammer blow (55-103 MPa)	
C	DQ	Dents with hammer blow (55-21 MPa)	
D	CQ	Craters with hammer blow (21-7 MPa)	
E	MQ	Can be remolded with finger pressure (<7 MPa)	

Fig.13. Unified Rock Classification System estimating for rock strength (Williamson, 1984).

Physical parameters

Laboratory tests are used to determine the net worth of intact rock as new parameters that can be used to evaluate the rock mass. These parameters are classified into five grades for reduction factors and are added to the measurable and complementary factors. For the classification of bulk density, one can use the Unified Rock Classification System (Williamson, 1984) for soft rocks or the stiffer rocks (NBG, 1985). In the case of crushed rock mass, soil or aggregate classifications can be used.

Strength of intact rock and rock mass

The intact material strength is shear, tensile, and compressive, though the tensile, rather than the compressive, plays a major role in predicting the shear strength (Grasselli, 2001). The shear failure of intact rocks results from kinematic constraints and external compressive or tensile forces. In the case of open discontinuities, in the rock mass, when the normal stress is low, the shear strength is due to sliding along the inclined surfaces of asperities. At high normal stresses, the shear strength is due to the breaking of the intact material (Barton, 1976).

Shear strength

The shear strength for filled discontinuities that have a thickness more than the amplitude of asperities depends on the strength of this material. The shear strength of materials in the Coulomb Equation is written as:

$$\tau = c + \sigma_n \tan\varphi \qquad 5$$

Where:

τ is the shear stress along the shear plane at failure

c is the cohesion

σ_n is the normal stress acting on the shear plane, and

φ is the friction angle of the shear plane

Equation 5, often called the Mohr-Coulomb criterion, is applied in rock mechanics for shear failure in intact rock, discontinuities, and rock masses (Edelbro, 2003).

Barton, 1976, Barton, and Choubey, 1977 have studied the behavior of natural rock discontinuities in detail and have proposed Equation 6 for shear strength of discontinuities:

$$\tau = \sigma_n \tan[\varphi_b + JRC \log_{10}(JCS/\sigma_n)] \qquad 6$$

Where:

σ_n is the normal stress

φ_b is the angle of internal friction

JRC is the joint roughness coefficient, and

JCS is the discontinuity wall compressive strength.

The direct and alternative methods for estimating JRC are presented in Fig.14 (Barton and Choubey, 1977).

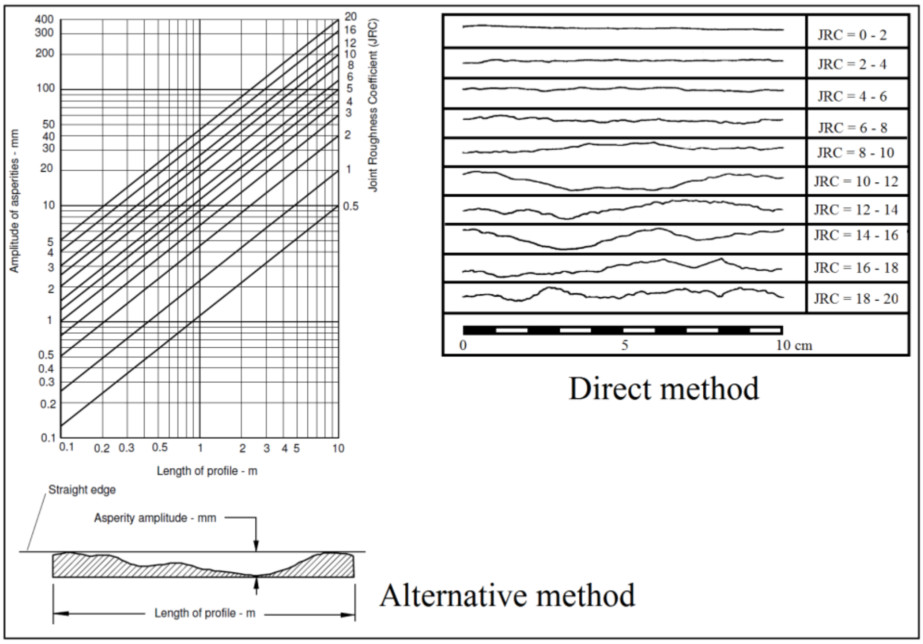


Fig.14. Direct and indirect methods for estimating JRC (Barton, 1976 and Barton and Choubey, 1977).

Tensile strength

Tensile strength is very low or it is equal to zero on discontinuity surfaces if it is not healed (USDIBR, 2001). Direct tensile strength tests of rocks are not easy because of the difficulty in specimen preparation. Indirect methods, such as bending and Brazilian tests were used. The estimation of the tensile strength of rocks depends on tensile crack initiation stress that is identical to the tensile crack propagation stress and the peak tensile strength (Cai, 2010). Griffith, 1924 proposed that the failure of brittle materials is governed by the initial presence of micro-cracks. Under uniaxial tension, the tensile strength predicted by Griffith's theory is:

$$\sigma_t = \sqrt{\frac{\lambda E' \gamma}{c}} \qquad 7$$

Where:

$E' = E$ for plane stress problems and $= E / (1-\upsilon)$ for plane strain problems,

E is Young's Modulus,

υ is the Poisson's ratio,

γ is the specific surface energy,

c is the half-crack length, and

λ is a numerical constant = $2/\pi$

Usually, the tensile strength is less than the compressive strength of intact rocks. Many researchers suggest a relationship between uniaxial compressive strength and tensile strength (Cai, 2010). UCS is between 8 to 20 times for tensile strength.

The relation between tensile strength and shear strength of rock mass depends on friction angle and cohesion. The tensile strength of the rock mass can be estimated from the GSI classification system by using the RocLab Software (Hoek, *et. al.,* 2002). Tensile strength is more affected by weathering in crystalline rocks in which micro-fractures are more important (Gupta and Rao, 1998). The tensile strength of rock masses is often the critical mechanical parameter in the engineering practice involving rocks. Surprisingly, on the contrary, some authors have even suggested that tensile strength should not be considered a material property (Coviello, *et al*., 2005).

Compressive strength

The unconfined compressive strength UCS and tensile strength of rocks are widely used in the design stage of engineering structures. Although there are several classical approaches in the literature for strength prediction and there are soft computing techniques such as artificial intelligence (Baykasoglu, *et al.*, 2008), the field estimation of rock strength is useful for preliminary stages of engineering projects. Testing procedures for direct determination of unconfined compressive strength are standardized by the ISRM, 2007. Discontinuities, at any scale influence UCS according to its orientation to the direction of maximum load, which gives mastery over. The relationship obtained between UCS for intact rock and rock mass is logarithmic. The low influence of discontinuities on UCS of the rock mass is for medium to low intact compressive strength (Figure 13). Each one of the different classifications for unconfined compressive strength (Fig.15) can be used to obtain the reduction factor.

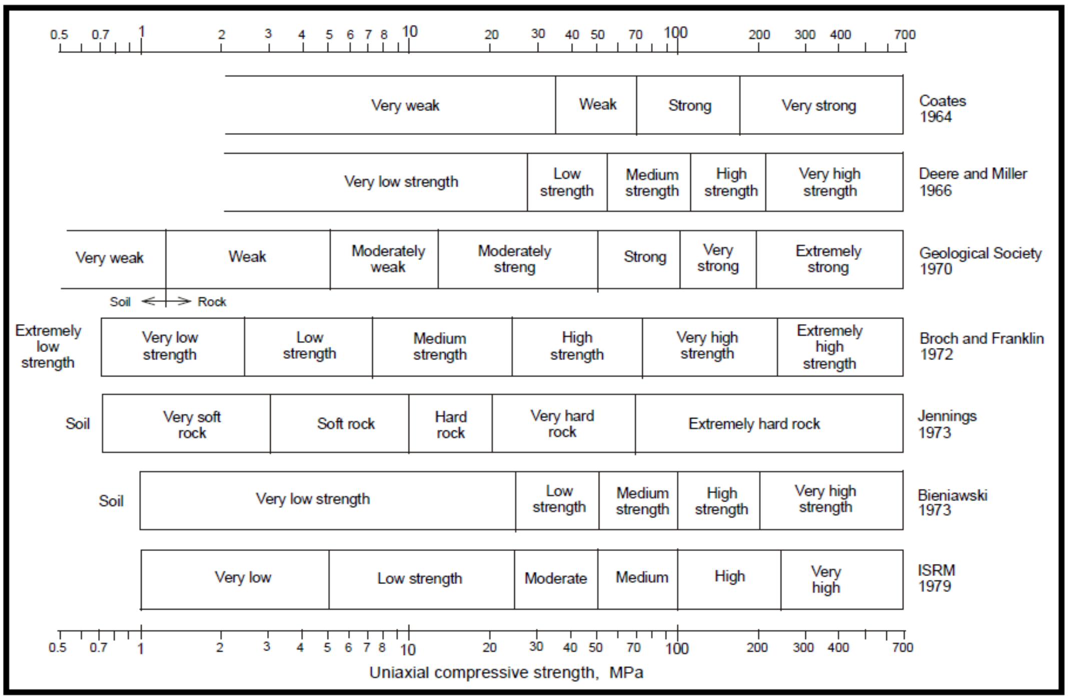


Fig.15. Different classifications of unconfined compressive strength.

Weigh of parameters

Reduction factors of measurable parameters are extracted from the relations that are explained in equations 1 to 4. Each parameter has the same weigh, from zero to one.

Complementary parameters and laboratory parameters are also classified as a percent of one. The whole classification parameter is the percent of one, the average of used parameters. This value is multiplied by the shear, tensile or compressive strength of intact rock to establish the rock mass strength. The classification grades depend on the value of reduction factors and can be divided into five categories, very low, low, medium, high, and very high having the values (>0-0.2, 0.2-0.4, 0.4-0.6, 0.6-0.8 and 0. 8-<1) respectively.

Systematic application

Systematic application is explained in the flow chart of Fig. 1 and Fig.4. Seven steps are stated as follows to explain the systematic application:

1. Zonation: is the first stage of any engineering project, from images, maps, literature, and site reconnoitering.
2. Continuity examination: identification of project continuity, is known from stage one to specify the path of future investigations.
3. Data acquisition: for continuous paths (Fig.1), the rock mechanics tests are used on an intact rock branch, the soil mechanics tests for clastic and weathered rocks, or the aggregate tests for highly crushed and fragmented rock mass.
4. The discontinuous path led to rock mass classification and data amassment from the field and laboratory.
5. Reduction factors: the four measurable reduction factors are calculated from equations 1 to 4. The complementary and laboratory factors are classified as the percent of one and added to the measurable factors.
6. Type of failure: is determined from the scale and orientation effects of the rock mass. The discontinuities control the development of the sliding surfaces, which are sub-parallel to the topographic slope (Ganerod, 2008).
7. Rock mass evaluations: are divided into two parts. The first one is if the value of expecting failure strength is not known, the evaluation is classified into five categories. In the second part when the value of strength is known, the reduction factor is multiplied by the strength and gives the expected rock mass strength.

Results and Discussion

At the Bekhme Dam Site, the proposed system is applied to evaluate the rock mass. The system was applied along the spillway tunnel, the access tunnel (Fig.16), and at the Bekhme Gorge (Fig.17). At the Spillway Tunnel (Fig.16), three small zones having very low grade at the distance from the SW entrance (38-40, 95-102, 303.3-305.3 m), two zones having low grade are (130.7-142.3, 154.4-163 m) and two zones having medium grade are (102-130.7, 305.3-384.5 m). The best qualities extend to long zones that are five (47.5-77, 142.3-154.4, 163-259.6, 384.5-433, 449-720 m) and five zones are having very good quality (0-38, 40-47.5, 77-95, 259.6-303.3, 433-449 m). At the Access Tunnel (Fig.16) there are two very low-grade zones at the distance from the SW entrance (182-200, 1033-1054 m), six low-grade zones (148-162, 200-223, 298-363, 683-711, 974-1033, 1127-1148 m) and eight medium grade zones (0-41, 53-78, 100-123, 123-148, 162-182, 506-683, 1100-1127, 1148-1261 m). Approximately half the length of the tunnel has good and very good quality for five zones (41-53, 78-100, 438-486, 1054-1100, 1261-1300 m) and four zones very good (223-298, 363-438, 486-506, 711-974 m).

The surface section (Fig.17) shows that the carbonate formation from the Chia Gara Formation to the Bekhme Formation has good to very good quality with only some zones that show moderate quality (Fig.17). The Shiranish Formation appears to have moderate quality at the bottom and top, while low to very low quality at the middle part. The Khurmala Formation displays moderate quality over the whole zone. Clastic formations i.e. the Kolosh-Tanjero formations, the Gercus Formation, and the marly part of the Shiranish formation appear to have low to very low-quality rock mass.

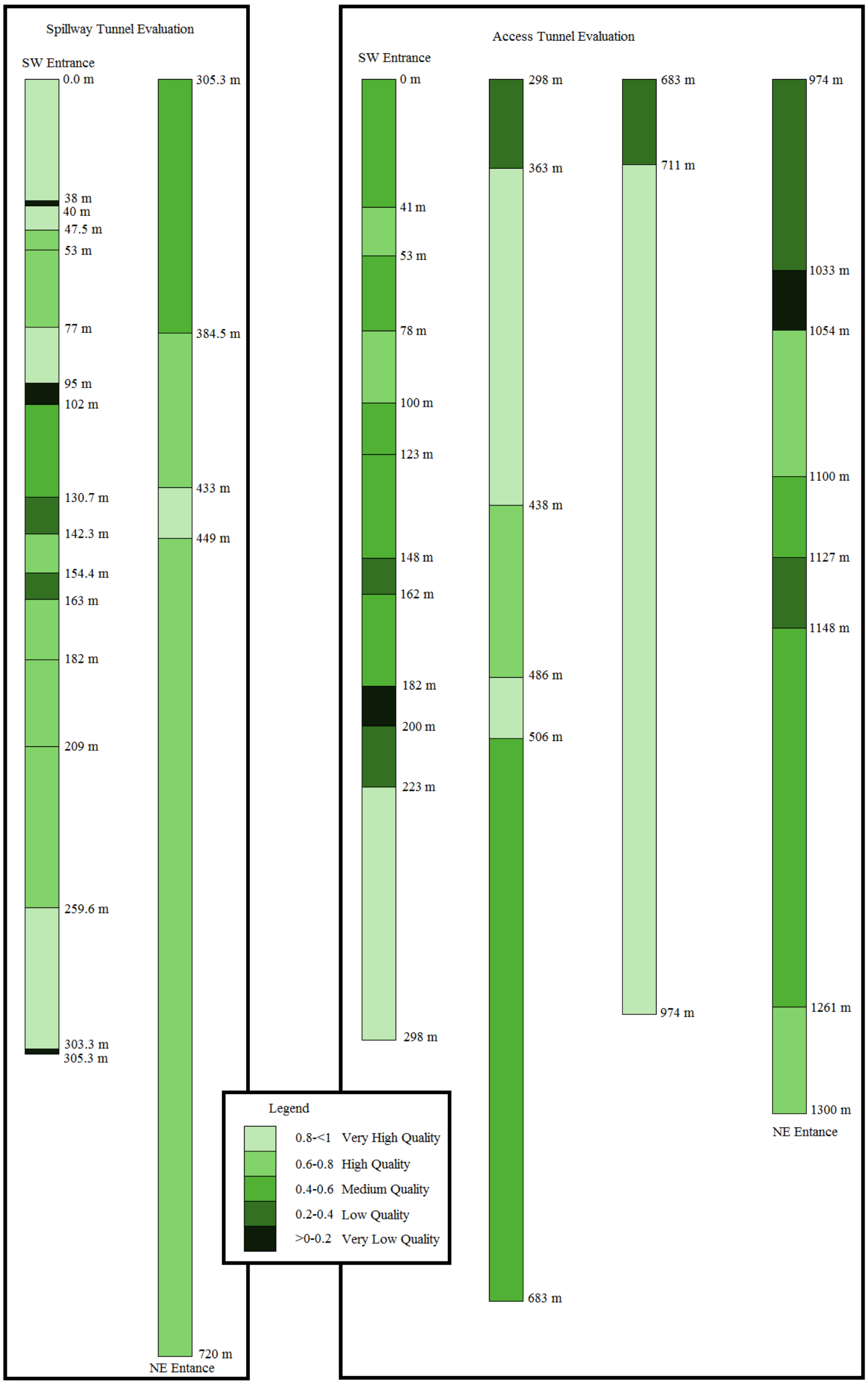


Fig.16. Evaluation of the spillway tunnel at the left and access tunnel at the right according to RSRMS.

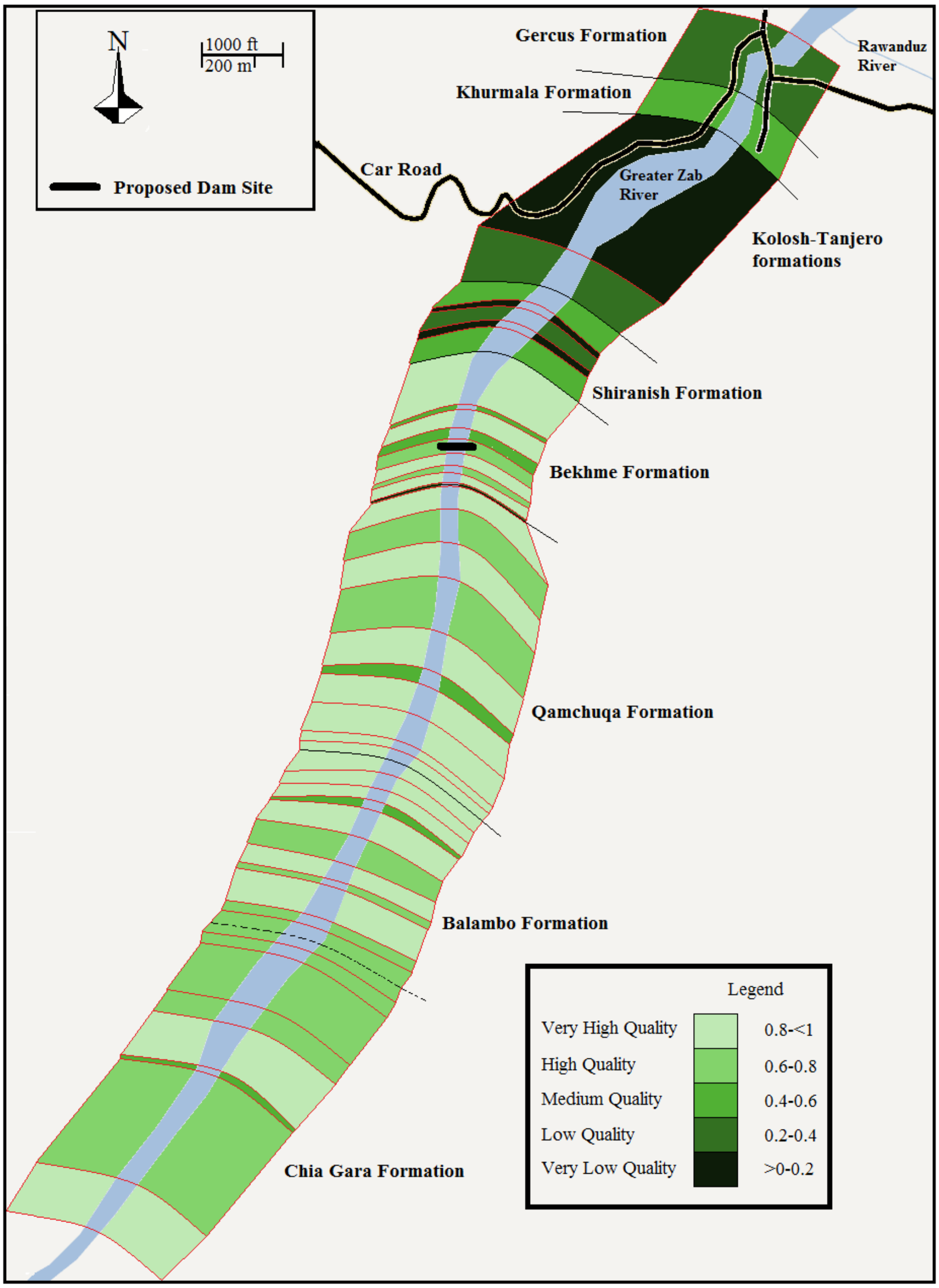


Fig.17. Evaluation of The Bekhme Gorge according to RSRMS.

Conclusion

Wide-spreading types of rock types with a wide range of weathering degrees and different properties can be classified by the RSRMS. The conjugation of parameters led to more fidelity to evaluate the rock mass. The process of linking each of the two parameters in influence is of great importance in evaluating the rock masses more than the effect of each parameter separately. The surface data assemblages by simple means without test boring are sufficient for the evaluation of rock mass. Many worldwide systems are used RQD found from the boreholes or sometimes estimated from the spacing of discontinuities. The weighing of parameters in this classification is the same to reduce the strength of the rock mass as for intact rock. This study proved that the site of the previously chosen Bekhme Dam must be changed and pushed to the north by a distance of no more than 50 meters or to the south by about 20 meters to settle on a very high-quality rock mass.

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References

Barton, N. and Choubey, V., 1977. The Shear Strength of Rock Joints in Theory and Practice, Rock Mechanics, 1/2 Vienna: Springer, Also NGI Publ. 119, 1978, pp. 1-54. https://doi.org/10.1007/BF01261801

Barton, N., 1973. Review of a New Shear-Strength Criterion for Rock Joints. Engineering Geology, Vol. 7, pp. 287-332. https://doi.org/10.1016/0013-7952(73)90013-6

Barton, N., 1976. Rock Mechanics Review: The Shear Strength of Rock and Rock Joints, International Journal of Rock Mechanics and Mining Science and Geomechanics Abstract, Vol. 13, pp. 255-279. https://doi.org/10.1016/0148-9062(76)90003-6

Baykasoglu, A., Guellue, H., Canakci, H. and Oebakir, L., 2008. Prediction of Compressive and Tensile Strength of Limestone via Genetic Programming. Expert Systems with Applications, Vol. 35, No. 1-2, pp.111-123. https://doi.org/10.1016/j.eswa.2007.06.006

Bieniawski, Z.T., 1978. Determining Rock Mass Deformability. Experience from Case Histories. International Journal of Rock Mechanics and Mining Science and Geomechanics Abstract, Pergamon, Vol. 15, pp. 237-247. https://doi.org/10.1016/0148-9062(78)90956-7

Bieniawski, Z.T., 1976. Rock Mass Classification of Jointed Rock Masses, Exploration for Rock Engineering, Z.T. Bieniawski Ed. Balkema, Johannesburg, pp. 97-106.

Bieniawski, Z.T., 1989. Engineering Rock Mass Classification: A Complete Manual for Engineers and Geologists in Mining, Civil and Petroleum Engineering, John. Wiley and Sons, Inc., ISBN 0-471-60172-1, 250 P.

Borri-Brunetto, M., Carpinteri, A., and Chiaia, B., 2004. The Effect of Scale and Criticality in Rock Slope Stability, Rock Mech. Rock Engng., Vol. 37, No. 2, pp. 117–126. https://doi.org/10.1007/s00603-003-0004-1

Brand, E.W., 2000. Guide to Site Investigation, Geotechnical Engineering Office, Civil Engineering Department, Hong Kong, 360 P.

Cai, M., 2010. Practical Estimates of Tensile Strength and Hoek–Brown Strength Parameter mi of Brittle Rocks, Rock Mechanics and Rock Engineering, Vol. 43, pp. 167–184.

Capigian, A.O. and Al-Khateeb, A.A.G., 2008. A New Geotechnical Classification System for Rock According to Number of Fractures per Meter, GEO India Expo. XXI, Noida, New Delhi, India, AAPG Search and Discovery, 14 P.

Coviello, A., Lagioia, R., and Nova, R., 2005. On the Measurement of the Tensile Strength of Soft Rocks, Rock Mechanics Rock Engineering, Vol. 38, No. 4, pp. 251-273.

Edelbro, C., 2003. Rock Mass Strength a Review, Technical Report, Department of Civil Engineering, Division of Rock Mechanics, Lulea University of Technology, Sweden, 160 P.

Einstein, H.H., Veneziano, D., Baecher, G.B. and Oreilly, K.J., 1983. The Effect of Discontinuity Persistence on Rock Slope Stability, International Journal of Rock Mechanics and Mining Science and Geotechnical Abstracts, Vol. 20., No.5, pp. 227-236.

Farah, R., 2011. Correlations Between Index Properties and Unconfined Compressive Strength of Weathered Ocala Limestone, MSc Thesis, University of North Florida, School of Engineering, 72 P.

Ganerod, G.V., 2008. Applied Structural Geology – Case Studies of Underground Constructions and Rockslides, Ph.D. thesis, Department of Earth Science, University of Bergen, Norway, pp. 210.

Goodman, R. and Shi, G.H., 1985. Block Theory and Its Application to Rock Engineering, 1st ed., London, Prentice-Hall, 338 P.

Grasselli, G., 2001. Shear Strength of Rock Joints Based on Quantified Surface Description, Ph.D. Thesis n.2404, EPF-Lausanne, Switzerland.

Griffith, A.A., 1924. The Theory of Rupture, Proceedings of the First International Congress on Applied Mechanics, p. 55-63.

Gupta, A.S. and Rao, K.S., 1998. Index Properties of Weathered Rocks: Inter-Relationships and Applicability, Bulletin of Engineering Geology and Environment, Vol. 57, pp. 161-172.

Hack, H.R.G.K. and Price, D.G., 1995. Determination of Discontinuity Friction by Rock Mass Classification, Reprint: Proceeding of the 8th International Society for Rock Mechanics (ISRM) congress, Tokyo, Japan, pp. 23-27.

Hammett, R.D. and Hoek, E., 1981. Design of Large Underground Caverns for Hydroelectric Projects with Reference to Structurally Controlled Failure Mechanisms. Proceeding of the American Society of Civil Engineers, International Conference on Recent Developments in Geotechnical Engineering for Hydro Projects, pp. 190-206. New York, ASCE, USA.

Hoek, E, and Brown, E.T., 1997. Practical Estimates of Rock Mass Strength, International Journal of Rock Mechanics and Mining Sciences, Vol. 34, No. 8, pp. 1165-1186.

Hoek, E. and Bray, J., 1989. Rock slopes: Design, Excavation, Stabilization, U.S. Department of Transportation, Federal Highway Administration, FHWA-TS-89-045, 548 P.

Hoek, E., 1964. Fracture of Anisotropic Rock, Journal of the South African Institute of Mining and Metallurgy, Vol. 64, No. 10, pp. 501-518.

Hoek, E., 1983. Strength of Jointed Rock Masses, Geotechnique, Vol. 23, No. 3, pp. 187-223.

Hoek, E., Carranza-Torres, C.T., and Corkum, B., 2002. Hoek-Brown Failure Criterion – 2002 Edition. Proc. North American Rock Mechanics Society meeting in Toronto in July 2002.

Hussain, S., Muhammad Khan, N., Emad, M.Z., Naji, A.M., Cao, K.; Gao, Q., Ur Rehman, Z., Raza, S., Cui, R., Salman, M., et al., 2022. An Appropriate Model for the Prediction of Rock Mass Deformation Modulus among Various Artificial Intelligence Models. Sustainability, 14, 15225. https://doi.org/10.3390/su142215225.

Indraratna, B., Welideniya, H. S. and Brown, E. T., 2005. A Shear Strength Model for Idealised Infilled Joints Under Constant Normal Stiffness, Geotechnique, Vol. 55, No. 3, pp. 215–226. https://doi.org/10.1680/geot.2005.55.3.215

ISRM, 2007. The Complete ISRM Suggested Methods for Rock Characterisation, Testing and Monitoring: 1974 – 2006, edited by Ulusay, R. and Hudson, J.A., ISRM Turkish National Group, Ankara, Turkey, 628 p. https://doi.org/10.2113/gseegeosci.15.1.47

Kovari, K., 1979. Decision Marking and Field Measurement in Tunneling, 25th OYO Anniversary Lecture Meeting, Tokyo, Japan, 43 P.

NBG, 1985. Norwegian Rock Mechanics Group: Handbook in Engineering Geology-Rock. (in Norwegian), Tapir, Trondheim, Norway, 140 pp.

Nicholson, G.A., and Bieniawski, Z. T., 1990. A Non-Linear Deformation Modulus Based on Rock Mass Classification, International Journal of Minning and Geological Engineering, Vol. 8, pp. 181-202. https://doi.org/10.1007/BF01554041

Ramamurthy, T., 1994. Strength and Modulus Responses of Anisotropic Rocks, Rock Engineering, edited by Hudson, J.A., Vol. 1., No.13, pp. 313-329.

Ramsay, J.G., and Huber, M. I., 1987. The Techniques of Modern Structural Geology: V.2, Folds and Fractures, Academic Press, London, UK, 700 P.

Singh, B., 1979. Geological and Geophysical Investigation in Rocks for Engineering Projects, International Symposium Institute Testing of Soils and Performance of Structures, Vol. 1, India, pp. 486-492.

Sonmez, H. and Ulusay, R., 1999. Modifications to the Geological Strength Index (GSI) and their Applicability to Stability of Slopes, International Journal of Rock Mechanics and Mining Sciences, Vol. 36, pp. 743-760.

USDIBR, 2001. Engineering Geology Field Manual, Vol. I, 535 p., and II, 488 p., Stop SSOP, Washington, DC 20402-0001, USA.

Van der Pluijm, B.A. and Marshak, S., 2004. Earth Structure: An Introduction to Structural Geology and Tectonics, W. W. Norton and Company, Inc., USA, 656 P.

Williamson, D.A., 1984. Unified Rock Classification System, Bulletin Association of Engineering Geologists, Vol. 21, No. 3, pp. 345-354.

Yang, B. and Elmo, D., 2022. Why Engineers Should not Attempt to Quantify GSI. Geosciences, 12, 417. https://doi.org/10.3390/geosciences12110417.