Rock Mass Classification Systems

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1 Rock Mass Classification Systems

1.1 Introduction

Rock Mass Classification is the process of placing a rock mass into groups or classes on defined relationships (Bieniawski, 1989) and assigning a unique description (or number) to it on the basis of similar properties/characteristics such that the behavior of the rock mass can be predicted. Rock mass is referred to an assemblage of rock material separated by rock discontinuities, mostly by joints, bedding planes, dyke intrusions and faults etc. Bedding planes, dyke intrusions, and faults are not so common as compared to joints and are dealt individually (Bieniawski, 1993). Rock mass classification systems allow the user to follow a guideline and place the object in an appropriate class.

The rock mass characterization and classification is a mean to properly communicate the estimated rock mass characteristics and should not be taken as an alternative to detailed engineering design procedures. According to Bieniawski (1989), the classification systems are not suitable for use in the elaborated and final design, particularly for complex underground openings. Such use of classification needs further development of these systems. The rock mass classification systems were designed to act as an engineering design aid and were not intended to substitute field observations, analytical considerations, measurements, and engineering judgment (Bieniawski, 1993).

These systems form an essential part of foremost design approaches (the empirical and the numerical design methods) and are increasingly used in both design approaches as computing power improves. It should be used in conjunction with other design schemes to devise an overall rationale compatible with the design objectives and site geology. In practice, rock mass classification systems have provided a valuable systematic design aid on many engineering projects especially on underground constructions, tunneling and mining projects (Hoek, 2007).

1.2 Functions of classification systems

These systems provide a basis for understanding the characteristic behavior and relate to experiences gained in rock conditions at one site to another. In the feasibility and preliminary design stages of a project, comprehensive information related to the rock mass parameters, its stress, and hydrologic characteristics is mostly unavailable. Thus, rock mass classification proves helpful at this stage for assessing rock mass behavior. It not only gives information about the composition, strength, deformation properties and characteristics of a rock mass required for estimating the support requirements but also shows which information is relevant and required (Bieniawski, 1989).

According to Bieniawski (1993), the objectives of rock mass characterization and classification are:

i) to identify the most significant parameters influencing the behavior of a rock mass;
ii) to divide a particular rock mass formation into a number of rock mass classes of varying quality;
iii) to provide a basis for understanding the characteristics of each rock mass class;
iv) to derive quantitative data for engineering design;
v) to recommend support guidelines for tunnels and mines;

vi) to provide a common basis for communication between engineers and geologists;
vii) to relate the experience on rock conditions at one site to the conditions encountered and experience gained at other.

Nowadays, rock mass classification schemes are also used in conjunction with numerical simulations, especially in early stages of geotechnical projects, where data are often rare. Based on rock mass classifications, strength (e.g. Bieniawski 1993) and deformation (e.g. Hoek and Diederichs, 2006) parameters according to specific constitutive laws or the rock mass (e.g. Mohr-Coulomb or Hoek-Brown material models) can be deduced and applied in numerical simulations to consider stability, failure pattern, Factor-of-safety, deformations etc. Examples for application of rock mass classification schemes in engineering praxis in respect to underground mining and slope stability are given for example by Walter & Konietzky (2012), Chakraborti et al. (2012) or Herbst & Konietzky (2012).

1.3 Advantages of rock mass classification

Classification of rock mass improves the quality of site investigations by calling for a systematic identification and quantification of input data. A rational, quantified assessment is more valuable than a personal (non-agreed) assessment. Classification provides a checklist of key parameters for each rock mass type (domain) i.e. it guides the rock mass characterization process. Classification results in quantitative information for design purposes and enables better engineering judgment and more effective communication on a project (Bieniawski, 1993). A quantified classification assists proper and effective communication as a foundation for sound engineering judgment on a given project (Hoek, 2007).

Correlations between rock mass quality and mechanical properties of the rock mass have been established and are used to determine and estimate its mechanical properties and its squeezing or swelling behavior.

1.4 Disadvantages of rock mass classification

According to Bieniawski (1993), the major pitfalls of rock mass classification systems arise when:

i) using rock mass classifications as the ultimate empirical 'cook book', i.e. ignoring analytical and observational design methods;
ii) using one rock mass classification system only, i.e. without cross-checking the results with at least one other system;
iii) using rock mass classifications without enough input data;
iv) using rock mass classifications without full realization of their conservative nature and their limits arising from the database on which they were developed.

Some people are of the opinion that

i) natural materials cannot be described by a single number,
ii) other important (often dominating) factors are not considered,
iii) results of rock mass classification are prone to misuse (e.g., claims for changed conditions) (Bieniawski, 1989).

1.5 Parameters for Rock Mass Classification

The behavior of intact rock material or blocks is continuous while that of the highly fractured rock mass is discontinuous in nature. For any engineering design in the rock mass, the engineering properties of rock material and discontinuities should be taken into consideration. Various parameters of greatest and different significance have to be considered in order to describe a rock mass satisfactorily for assuring rock mass stability.

The various important parameters used for description and classification of rock mass (Bieniawski, 1993) are:

i) the strength of the intact rock material (compressive strength, modulus of elasticity);
ii) the rock quality designation (RQD) which is a measure of drill core quality or intensity of fracturing;
iii) parameters of rock joints such as orientation, spacing, and condition (aperture, surface roughness, infilling and weathering);
iv) groundwater pressure and flow;
v) in situ stress
vi) major geological structures (folds and faults).

1.6 Types of classification systems

On the basis of mode of characterization, these systems can be grouped as qualitative and quantitative. Qualitative i.e. descriptive systems include GSI (Geological Strength Index), Rock Load and SIA 199 (Schweizerischer Ingenieur- und Architekten-Verein) while Q, RMR, RSR and RQD systems are quantitative.

Classification systems can also be classified on the basis of the aim of the rating systems: for stability assessment, Q and RMR systems are used; Q gives no support limit while RMR system is meant to calculate stand-up time. To calculate the ground support design (liner thickness, bolt spacing etc.) Q system is used (to a minor extent also RMR System). To identify and to determine the excavation class and support classes, SIA 199 system is used, and to determine the engineering design parameters only, GSI is used.

1.7 Commonly used classification systems

Rock mass classification schemes owe its origin to 1879 when Ritter (1879) devised an empirical approach to tunnel design for finding out support requirements (Hoek, 2007). Since then, these systems have been developing. Most of the multi-parameter classification schemes (Barton et al., 1974; Bieniawski, 1968; Bieniawski, 1973, 1989; Wickham, 1972) were developed from civil engineering case histories (Hoek, 2007). The rock mass classification schemes that are often used in rock engineering for assisting in designing underground structures are RMR, Q and GSI systems. Some well-known systems are listed in Table 1.

Table 1. Major rock mass classification systems (Cosar, 2004)

Rock Mass Classification System	Originator	Country of Origin	Application Areas
Rock Load	Terzaghi, 1946	USA	Tunnels with steel Support
Stand-up time	Lauffer, 1958	Australia	Tunneling
New Austrian Tunneling Method (NATM)	Pacher et al., 1964	Austria	Tunneling
Rock Quality Designation (RQD)	Deere et al., 1967	USA	Core logging, tunneling
Rock Structure Rating (RSR)	Wickham et al., 1972	USA	Tunneling
Rock Mass Rating (RMR)	Bieniawski, 1973 (last modification 1989-USA)	South Africa	Tunnels, mines, (slopes, foundations)
Modified Rock Mass Rating (M-RMR)	Ünal and Özkan, 1990	Turkey	Mining
Rock Mass Quality (Q)	Barton et al., 1974 (last modification 2002)	Norway	Tunnels, mines, foundations
Strength-Block size	Franklin, 1975	Canada	Tunneling
Basic Geotechnical Classification	ISRM, 1981	International	General
Rock Mass Strength (RMS)	Stille et al., 1982	Sweden	Metal mining
Unified Rock Mass Classification System (URCS)	Williamson, 1984	USA	General
Communication Weakening Coefficient System (WCS)	Singh, 1986	India	Coal mining
Rock Mass Index (RMi)	Palmström, 1996	Sweden	Tunneling
Geological Strength Index (GSI)	Hoek and Brown, 1997	Canada	All underground excavations

1.7.1 Rock Load Classification

Terzaghi (1946) introduced this semi-quantitative but comprehensive classification system in cooperation with the Procter and White Steel Company. In this classification, the influence of geology on designing steel supported tunnels was discussed and rock loads carried by steel sets were estimated based on the descriptive classification of rock classes (Hoek, 2007). The objective of this system is to estimate the rock load to be carried by the steel arches installed to support a tunnel. As discussed earlier, it was not the first classification system but it was the first one in the English language that integrated geology into the design of tunnel support. This system forms the foundation for the development of three most common rock mass classification schemes i.e. Q, RMR, and GSI.

This conservative method has been modified and improved over time and is still used today to aid in the design of tunnels. It does not include the basic geological rock types, though it considers some important characteristics that control rock mass behavior such as the distinction between foliated and non-foliated rocks, block size, discontinuities, swelling and squeezing. The rock mass was divided into nine categories, each with a description of the characteristic discontinuities, block size, as well as swelling or squeezing potential (Singh and Geol, 1999).

Rock Load Factor was defined for each rock class and accordingly the appropriate support intensity was recommended. Recommendations and comments were given related to characteristic observations from different tunnels.

Terzaghi devised the equation $p = H_p \gamma H$ to obtain support pressure (p) from the rock load factor (H_p), where γ is the unit weight of the rock mass, H is the tunnel depth or thickness of the overburden (Terzaghi, 1946).

According to Deere et al. (1970), Class I of Rock Load Classification corresponds to RQD 95–100%, Class II to RQD 90–99%, Class III to RQD 85–95%, and Class IV to RQD 75–85%.

1.7.1.1 Limitations of Rock Load Classification

Singh and Geol (1999) are of the opinion that Rock Load Factor classification provides reasonable support pressure estimates for small tunnels with diameter up to 6 meters but gives over-estimates for tunnels having diameter more than 6 meters, and that the estimated support pressure range for squeezing and swelling rock conditions is wide enough to be meaningfully applied.

Brekke (1968) is of the opinion that water table has little effect on the rock load. Therefore, Rose (1982) proposed that Terzaghi's rock conditions 4-6 should be reduced by 50% from their original rock load values.

Cording and Deere (1972) suggest that Terzaghi's rock load system should be limited to tunnels supported by steel sets because it does not apply to openings supported by rock bolts.

According to Cecil (1970), this classification system does not provide any quantitative information regarding the rock mass properties.

Contrary to Terzaghi (1946), Singh et al. (1995) consider that the support pressure in rock tunnels and caverns does not increase directly with the excavation size.

Table 2. Rock class and rock load factor classification by Terzaghi for steel arch supported tunnels Terzaghi (1946)

Rock Class	Definition	Rock Load Factor Hp (in feet, B and Ht in feet)	Remark
I. Hard and intact	Hard and intact rock contains no joints and fractures. After excavation, the rock may have popping and spalling at excavated face.	0	Light lining required only if spalling or popping occurs.
II. Hard stratified and schistose	Hard rock consists of thick strata and layers. The interface between strata is cemented. Popping and spalling at the excavated face is common.	0 to 0.5 B	Light support for protection against spalling. Load may change between layers.
III. Massive, moderately jointed	Massive rock contains widely spaced joints and fractures. Block size is large. Joints are interlocked. Vertical walls do not require support. Spalling may occur.	0 to 0.25 B	Light support for protection against spalling.
IV. Moderately blocky and seamy	Rock contains moderately spaced joints. Rock is not chemically weathered and altered. Joints are not well interlocked and have small apertures. Vertical walls do not require support. Spalling may occur.	0.25 B to 0.35 $(B + H_t)$	No side pressure.
V. Very blocky and seamy	Rock is not chemically weathered and contains closely spaced joints. Joints have large apertures and appear separated. Vertical walls need support.	(0.35 to 1.1) $(B + H_t)$	Little or no side pressure.
VI. Completely crushed but chemically intact	Rock is not chemically weathered and highly fractured with small fragments. The fragments are loose and not interlocked. Excavation face in this material needs considerable support.	1.1 $(B + H_t)$	Considerable side pressure. Softening effects by water at tunnel base. Use circular ribs or support rib lower end.
VII. Squeezing rock at moderate depth	Rock slowly advances into the tunnel without a perceptible increase in volume. Moderate depth is considered as 150 ~ 1000 m.	(1.1 to 2.1) $(B + H_t)$	Heavy side pressure. Invert struts required. Circular ribs recommended.
VIII. Squeezing rock at great depth	Rock slowly advances into the tunnel without a perceptible increase in volume. Great depth is considered as more than 1000 m.	(2.1 to 4.5) $(B + H_t)$	
IX. Swelling rock	Rock volume expands (and advances into the tunnel) due to swelling of clay minerals in the rock at the presence of moisture.	up to 250 feet, irrespective of B and H_t	Circular ribs required. In extreme cases use yielding support.

Notes: The tunnel is assumed to be below the ground water table. For tunnel above water tunnel, Hp for Classes IV to VI reduces 50 %.

The tunnel is assumed excavated by blasting. For tunnel boring machine and road header excavated tunnel, Hp for Classes II to VI reduces 20 - 25 %.

Notations: B = tunnel span in meters, H_t = Height of the opening in meters, and H_p = Height of the loosened rock mass above tunnel crown developing load.

1.7.2 Stand-up Time Classification

Lauffer, (1958) established a relationship between the stand-up time for an unsupported span to the quality of the rock mass in which the span is excavated (Hoek, 2007). The unsupported span/active span is defined as the unsupported tunnel section or the distance between the face of the tunnel and the nearest installed support if this distance is greater than the tunnel span. Stand-up time is referred to as the time span which an excavated active span can stand without any form of support or reinforcement (Hoek, 2007).

Rock mass is classified into classes ranging from A to G on the basis of the relationship of stand-up time and unsupported span; such that Class A represents very good rock and Class G signifies very poor (Figure 1). RMR system was applied to correlate with excavated active span and stand-up time. This classification does not cover the spalling, slabbing, rock bursts or wedge failure in a tunnel. Many authors notably (Pacher et al., 1974) have modified Lauffer's original classification and now forms part of the general tunneling approach called the NATM (New Austrian Tunneling Method) (Hoek, 2007).

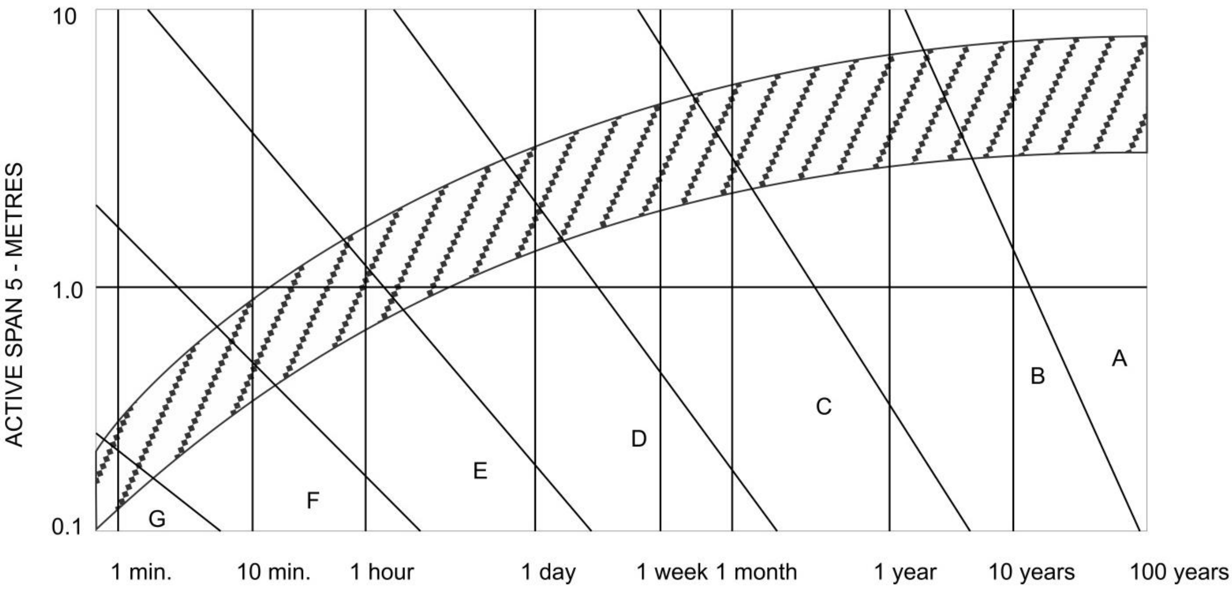
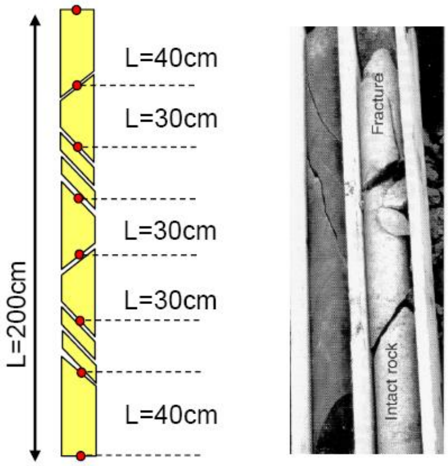


Figure 1. Relationship between active span and stand-up time and rock mass classes (Lauffer, 1958)

1.7.3 Rock Quality Designation (RQD)

In order to quantify the quality of the rock from drill cores, Deere et al. (1967) developed the concept of the RQD. RQD is defined as the percentage of intact core pieces longer than 100 mm (4 inches) in the total length of a core having a core diameter of 54.7 mm or 2.15 inches, as shown in Figure 2 (Hoek, 2007).



$$RQD=\frac{\sum \text{length of core pieces}>10\text{cm}}{\text{total length of the core}}\times 100\%$$

$$RQD=\frac{40+30+30+30+40}{200}\times 100=85\%$$

Figure 2. Procedure for measurement and calculation of RQD (After Deere, 1967)

Palmström (1982) demonstrated that the *RQD* may be estimated from the number of discontinuities per unit volume, which are exposed on the outcrops or exploration adits, using the following relationship for clay-free rock masses:

$$RQD = 115 - 3.3J_v$$

Where J_v, known as the volumetric joint count, is the sum of the number of joints per unit length for all joint sets. *RQD* is dependent on the orientation of the borehole. The use of the volumetric joint count can be quite useful in reducing this directional dependence.

RQD is a measure of the degree of fracturing of the rock mass and is aimed to represent the in situ rock mass quality. As shown in Table 3, the greater the RQD value the better the rock mass quality.

Table 3. Rock mass quality classification according to *RQD* (Deere et al. 1967)

RQD	Rock Mass Quality
< 25	Very poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
99 – 100	Excellent

RQD is used as an input parameter in RMR and Q systems. Cording and Deere (1972), Merritt (1972) and Deere and Deere (1988) related RQD to Terzaghi's rock load factors and to rock bolt requirements in tunnels.

1.7.3.1 Limitations of RQD

RQD does not reflect fully the rock mass quality as it only considers the extent of fracturing of the rock mass and does not account for the strength of the rock or mechanical and other geometrical properties of the joints. As *RQD* depends on the sampling line orientation relative to preferential orientation distribution of discontinuities, it does not give a reliable estimate of the degree of jointing of the rock mass. Furthermore, it cannot account for the length of the considered joints. Another limitation is that it is insensitive when the total frequency is greater than $3m^{-1}$ or when the rock mass is moderately fractured (Palmstrom and Broch, 2006).

1.7.4 Rock Structure Rating (RSR)

Wickham et al. (1972) developed a quantitative method for describing the quality of a rock mass and for selecting appropriate support (Bieniawski, 1989), based on case histories of relatively small tunnels supported by steel sets. In spite of its limitation of being based on relatively small tunnels supported by steel sets, this quantitative, multi-parameter rating system, and a ground-support prediction model, was the first complete rock mass classification and was the first to make reference to shotcrete support (Bieniawski, 1989).

RSR is a rating system for rock mass. In RSR system, two kinds of factors influencing the rock mass behavior in tunneling are considered; geological parameters and construction parameters (Hoek, 2007). Among the below-mentioned parameters, size of the tunnel, the direction of drive and method of excavation are the construction parameters (Bieniawski, 1989). The weighted values of each of the individual components (parameters) listed below (Wickham, 1972) are summed together to get a numerical value of RSR i.e. $RSR = A + B + C$.

1. Parameter A, Geology: General appraisal of geological structure on the basis of:
    a. Rock type origin (igneous, metamorphic, and sedimentary).
    b. Rock hardness (hard, medium, soft, and decomposed).
    c. Geologic structure (massive, slightly faulted/folded, moderately faulted/folded, intensely faulted/folded).

2. Parameter B, Geometry: Effect of discontinuity pattern with respect to the direction of the tunnel drive on the basis of:
    a. Joint spacing.
    b. Joint orientation (strike and dip).
    c. Direction of tunnel drive.

3. Parameter C: Effect of groundwater inflow and joint condition on the basis of:
    a. Overall rock mass quality on the basis of A and B combined.
    b. Joint condition (good, fair, poor).
    c. Amount of water inflow (in gallons per minute per 1000 feet of the tunnel).

Note that the RSR classification uses Imperial units.

Three tables from Wickham et al.'s 1972 paper are reproduced in Tables 4a, 4b, and 4c. These tables can be used to evaluate the rating of each of these parameters to arrive at the *RSR* value (maximum *RSR* = 100).In order to determine the typical ground-support system based on RSR prediction, support requirement charts have been prepared for 3m, 6m, 7m and 10m diameter tunnels (Bieniawski, 1989) (Figure 3). The support for a tunnel of specific diameter includes the shotcrete thickness, rock bolts spacing and steel ribs spacing of typical sizes used for the tunnel of specified diameter (Hoek, 2007). Based on sufficient and reliable data, it can also be used to evaluate which support system (either rock bolts and shotcrete or steel set solution) is cheaper and more effective. Although this system is not widely used today, it played a significant role in the development of other advanced classification schemes (Hoek, 2007).

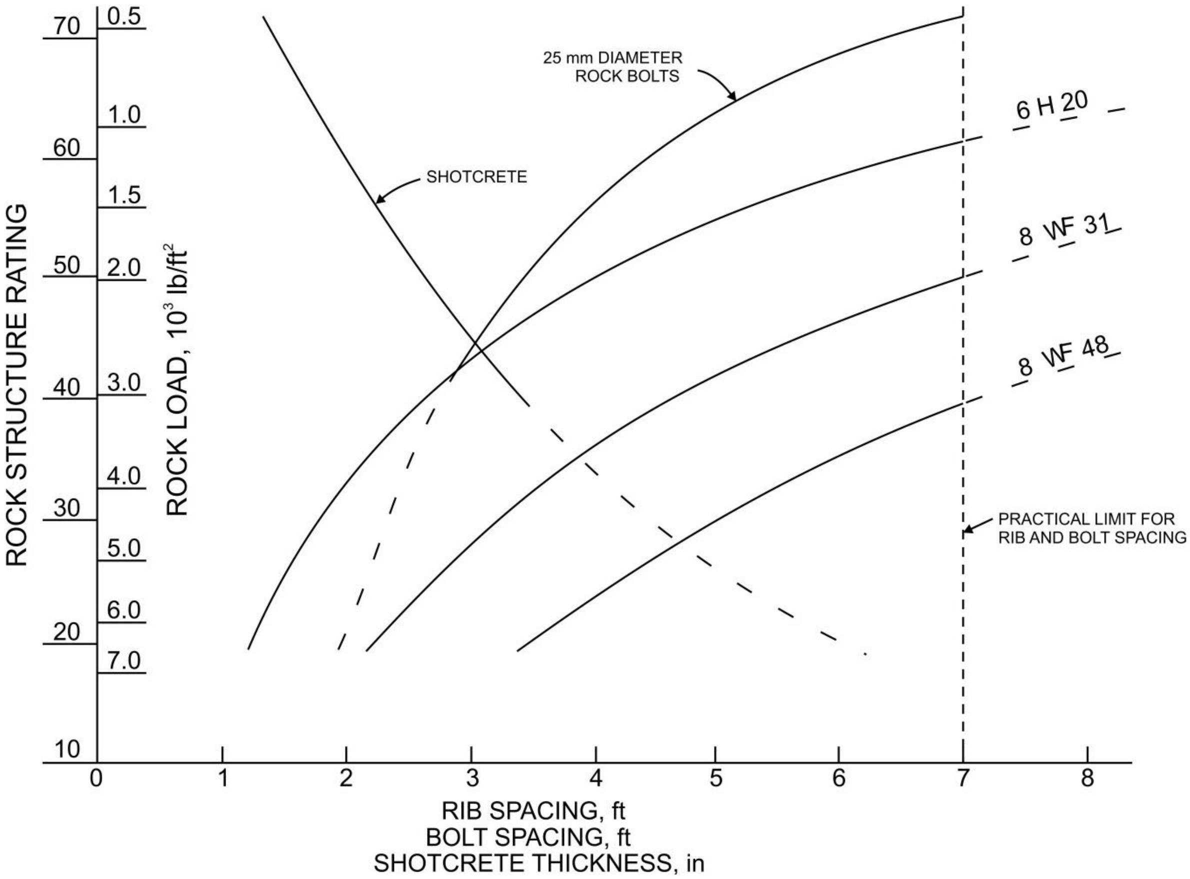


Figure 3. RSR support chart for a 24 ft. (7.3 m) diameter circular tunnel (after Wickham et al., 1972)

Table 4a. Rock Structure Rating: Parameter A: General area geology (Bieniawski, 1989)

	Basic Rock Type: Hard	Medium	Soft	Decomposed	Geological Structure: Massive	Slightly folded or faulted	Moderately folded or faulted	Intensively folded or faulted
Igneous	1	2	3	4				
Metamorphic	1	2	3	4				
Sedimentary	2	3	4	4				
Type 1					30	22	15	9
Type 2					27	20	13	8
Type 3					24	18	12	7
Type 4					19	15	10	6

Table 4b. Rock Structure Rating: Parameter B: Joint pattern, direction of drive (Bieniawski, 1989)

Average joint spacing	Strike perpendicular to Axis					Strike parallel to Axis		
	Direction of Drive					Direction of Drive		
	Both	With Dip		Against Dip		Either Direction		
	Dip of Prominent Joints [a]					Dip of Prominent Joints		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
1. Very closely jointed, < 2 in	9	11	13	10	12	9	9	7
2. Closely jointed, 2 – 6 in	13	16	19	15	17	14	14	11
3. Moderately jointed, 6 – 12 in	23	24	28	19	22	23	23	19
4. Moderate to blocky, 1 – 2 ft	30	32	36	25	28	30	28	24
5. Blocky to massive, 2 – 4 ft	36	38	40	33	35	36	24	28
6. Massive, > 4 ft	40	43	45	37	40	40	38	34

[a] Dip: flat: 0 - 20°, dipping: 20 – 50°, and vertical: 50 - 90°

Table 4c. Rock Structure Rating: Parameter C: Groundwater, joint condition (Bieniawski, 1989)

Anticipated water inflow gpm/1000ft of tunnel	Sum of Parameters A+B: 13-44			45-75		
	Joint condition [b]					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight, < 200 gpm	19	15	9	23	19	14
Moderate, 200 - 1000 gpm	15	22	7	21	16	12
Heavy, > 1000 gpm	10	8	6	18	14	10

[b] Joint condition: good = tight or cemented; fair = slightly weathered or altered; poor = severely weathered, altered or open

1.7.5 Rock Mass Rating (RMR) System

The RMR system or the Geomechanics Classification was developed by Bieniawski during 1972-1973 in South Africa to assess the stability and support requirements of tunnels (Bieniawski, 1973b). Since then it has been successively refined and improved as more case histories have been examined. The advantage of this system is that only a few basic parameters relating to the geometry and mechanical conditions of the rock mass are used. To classify a rock mass, the RMR system incorporates the following six basic parameters (Bieniawski, 1989).

• The uniaxial compressive strength of the intact rock (σ_c): for rocks of moderate to high strength, point load index is also acceptable (Bieniawski, 1989).
• Rock Quality Designation (RQD)
• Discontinuity spacing
• Condition of discontinuity surfaces
• Groundwater conditions
• Orientation of discontinuities relative to the engineered structure

It does not include in-situ stress conditions. In applying this classification system, the rock mass is divided into a number of structural regions separated from other regions by faults. A structural region has same rock type or same discontinuities characteristics. Each region is classified and characterized separately (Hoek, 2007).

Section A of Table 5 includes the first five classification parameters. Since various parameters have different significance for the overall classification of a rock mass, different value ranges of the parameters have been assigned based on their importance; a higher value represents better rock mass conditions (Bieniawski, 1989).

Section B represents ratings for discontinuity characteristics. Sections C and D reflect the effect of discontinuity angles with respect to excavation direction and subsequent adjustment of ratings for different engineering applications (Bieniawski, 1989).

Sections E and F, describes rock mass classes based on RMR values, show estimates of tunnel stand-up time and maximum stable rock span, and the Mohr-Coulomb rock mass strength parameters (equivalent rock mass cohesion c and friction angle Φ) for the rock mass classes (Bieniawski, 1989).

1.7.5.1 Applications of RMR System

1. RMR system provides a set of guidelines for the selection of rock reinforcement for tunnels as shown in Table 6 (Bieniawski, 1989). These guidelines depend on factors such as depth below the surface (in-situ stress), tunnel size and shape, and method of excavation. It is recommended in many mining and civil engineering applications to consider steel fibre reinforced shotcrete instead of wire mesh and shotcrete (Hoek, 2007).
2. RMR is also applied to correlate with excavated active span and stand-up time, as shown in Figure 4 (after Lauffer, 1988).
3. RMR can be used to obtain properties of rock mass as shown in Table 7.

Table 5. Rock Mass Classification RMR system ratings (Bieniawski, 1989)

(a) Five basic rock mass classification parameters and their ratings

	Parameter		Range of Vaules						
1	Strength of intact rock material	Point load strength index (MPa)	>10	4–10	2–4	1–2	for this low range uniaxial compressive strength is preferred		
		Uniaxial Compressive Strength (MPa)	>250	100–250	50–100	25–50	5–25	1–5	<1
	Rating		15	12	7	4	2	1	0
2	Drill Core Quality RQD (%)		90–100	75–90	50–75	25–50	<25		
	Rating		20	17	13	8	3		
3	Joint Spacing (m)		>2	0.6-2	0.2–0.6	0.06–0.2	<0.06		
	Rating		20	15	10	8	5		
4	Condition of Joints		Not continuous, very rough surfaces, unweathered, no separation	Slightly weathered surfaces, slightly weathered, separation < 1 mm	Slightly rough surfaces, highly weathered, separation < 1 mm	Continuous, slickensided surfaces, or gouge < 5mm thick, or separation 1–5 mm	Continuous joints, soft gouge > 5mm thick, or separation > 5 mm		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (1/min)	none	<10	10–25	25–125	>125		
		Joint water pressure / major in situ stress	0	0–0.1	0.1–0.2	0.2–0.5	>0.5		
		General conditions	dry	damp	wet	dripping	flowing		
	Rating		15	10	7	4	0		

(b) Guidelines for classification of discontinuity conditions

Parameter	Ratings				
Discontinuity length (persistence)	<1 m	1–3 m	3–10 m	10–20 m	>20 m
	6	4	2	1	0
Separation (aperture)	None	<0.1 mm	0.1–1.0 mm	1–5 mm	>5 mm
	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
	6	5	3	1	0
Infilling (gouge)	Hard filling			Soft filling	
	None	<5 mm	>5 mm	<5 mm	>5 mm
	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
	6	5	3	1	0

(c) Effects of joint orientation in tunneling

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Dip 0° – 20°
Drive with dip		Drive against dip				
Dip 45° – 90°	Dip 20° – 45°	Dip 45° – 90°	Dip 20° – 45°	Dip 45° – 90°	Dip 20° – 45°	Irrespective of strike
Very favorable	favorable	fair	unfavorable	Very unfavorable	fair	fair

(d) Rating adjustment for joint orientations

Strike and dip orientation of joints		Very favorable	favorable	fair	unfavorable	Very unfavorable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

(e) Rock mass classes determined from total ratings

Rating	100-81	80-61	60-41	40-21	<20
Class no.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

(f) Meaning of rock mass classes

Class no.	I	II	III	IV	V
Average stand-up time	20 years for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of rock mass (kPa)	>400	300–400	200–300	100–200	<100
Friction angle of rock mass (degrees)	>45	35–45	25–35	15–25	<15

RMR = Σ (classification parameters) + discontinuity orientation adjustment

Table 6. Guidelines for excavation and support of 10 m span horseshoe shaped rock tunnels constructed using drill and blast method at a depth of < 900 m, in accordance with the RMR system (after Bieniawski, 1989)

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I- Very good rock RMR: 81–100	Full face, 3 m advance	Generally, no support required except spot bolting		
II- Good rock RMR: 61–80	Full face, 1–1.5 m advance complete support 20 m from the face.	Locally, bolts in crown 3m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None
III- fair rock RMR: 41–60	Top heading and bench 1.5–3 m advance in top heading. Commence support after each blast. Complete support 10 m from the face.	Systematic bolts 4 m long, spaced 1.5–2 m in crown and walls with wire mesh in the crown.	50–100 mm in crown and 30 mm in sides	None
IV- Poor rock RMR: 21–40	Top heading and bench 1.0–1.5 m advance in top heading Install support currently with excavation, 10 m from the face.	Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and wall with wire mesh	100–150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required.
V- Very poor rock RMR: <20	Multiple drifts 0.5–1.5 m advance in top heading. Install support currently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5–6 m long, spaced 1–1.5 m in crown and walls with wire mesh, Bolt invert	150–200 m in the crown, 150 mm in sides, and 50 mm on the face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

Table 7. Direct relations between rock mass classification and properties of rock mass (Aydan, Ulusay, & Tokashiki, 2014)

Property	Empirical relation	Proposed by
Deformation modulus, E_m	$E_m=2RMR-100$ (GPa)(for RMR>50)	Bieniawski (1978)
	$E_m=10^{((RMR-10)/40)}$(GPa)	Serafim and Pereira (1983)
	$E_m= e^{(4.407+0.081+RMR}$(GPa)	Jasarevic and Kovacevic (1996)
	$E_m=0.0097RMR^{3.54}$ (MPa)	Aydan et al. (1997)
	$E_m=25\log Q$ (GPa)	Grimstad and Barton (1993)
	$E_m=(1-\frac{D}{2})\sqrt{\frac{\sigma_{ci}}{100}}10^{((GSI-10)/40)}$ (GPa) (for σ_{ci} < 100 MPa)	Heok et al. (2002)
	$E_m=100\ \frac{(1-0.5D)}{1+e^{(\frac{25+250-GSI}{11})}}$ (GPa)	Hoek and Diederichs (2006)
	$E_m=0.135\ \left[E_i 1+\frac{1}{WD}-\frac{RQD}{100}\right]^{1.1811}$ (GPa)	Kayabasi et al. (2003)
	$E_m=5.6RMi^{0.3}$ (GPa)(for RMi>0.1)	Palmstrom (1996)
	$E_m=0.1(\frac{RMR}{10})^3$	Mitri et al. (1994)
	$E_m=7(\pm 3)\sqrt{10^{(RMR-44)/21}}$ (GPa)	Diederichs and Kaiser (1999)
	$E_m=10Q^{1/3}$ (GPa)	Barton (1995)
	$E_m=10\left(Q\frac{\sigma_{ci}}{100}\right)^{\frac{1}{3}}$(GPa)	Barton (2002)
	$E_m=10^{(\frac{GSI-10}{40})}\sqrt{\frac{\sigma_{ci}}{100}}$ (GPa)	Hoek and Brown (1997)
	$E_m=0.0876RMR$ (GPa)(for RMR>50)	Galera et al. (2005)
	$E_m=0.0876RMR+1.056(RMR-50)+0.015\ (RMR-50)^2$ (GPa)(for RMR≤50)	Galera et al. (2005)
Uniaxial compressive strength, σ_{cm} (MPa)	$\sigma_{cm}=0.0016RMR^{25}$	Aydan et al. (1997)
	$\sigma_{cm}=5\gamma\left(Q\frac{\sigma_{ci}}{100}\right)^{1/3}$	Barton (2002)
Friction angle, ϕ_m (°)	$\phi_m= 20 + 0.5RMR$	*Aydan and Kawamoto (2001)*

…

	$\phi_m = 20\,\sigma_{cm}^{0.25}$	*Aydan et al. (1993)*
	$\phi_m = \tan^{-1}\left(\frac{J_r}{J_a} \times \frac{J_w}{1}\right)$	Barton (2002)
Cohesion, C_m (MPa)	$c_m = \frac{\sigma_{cm}}{2}\frac{1-\sin\phi_m}{\cos\phi_m}$	Aydan and Kawamato (2001)
	$c_m = \left(\frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_{ci}}{100}\right)$	Barton (2002)
Poisson's ratio, v_m	$v_m = 0.25(1+e^{-\sigma_{cm}/4})$	Aydan et al. (1993)
	$v_m = 0.5-0.2\frac{RMR}{RMR+0.2(100-RMR)}$	Tokashiki and Aydan (2010)

E_m deformation modulus of rock mass, E_i Young's modulus of intact rock, RMR rock mass rating, Q rock mass quality, GSI Geological Strength Index, D Disturbance factor, σ_{ci} uniaxial compressive strength of intact rock, σ_{cm} uniaxial compressive strength of rock mass, RQD Rock Quality Designation, RMi Rock Mass Index, WD weathering degree, ϕ_m friction angle of rock mass, c_m cohesion of rock mass, v_m Poisson's ratio of rock mass, J_n joint set rating, J_r joint roughness rating, J_w joint water rating, J_a joint alteration rating, SRF stress reduction factor, γ rock density (t/m^3)

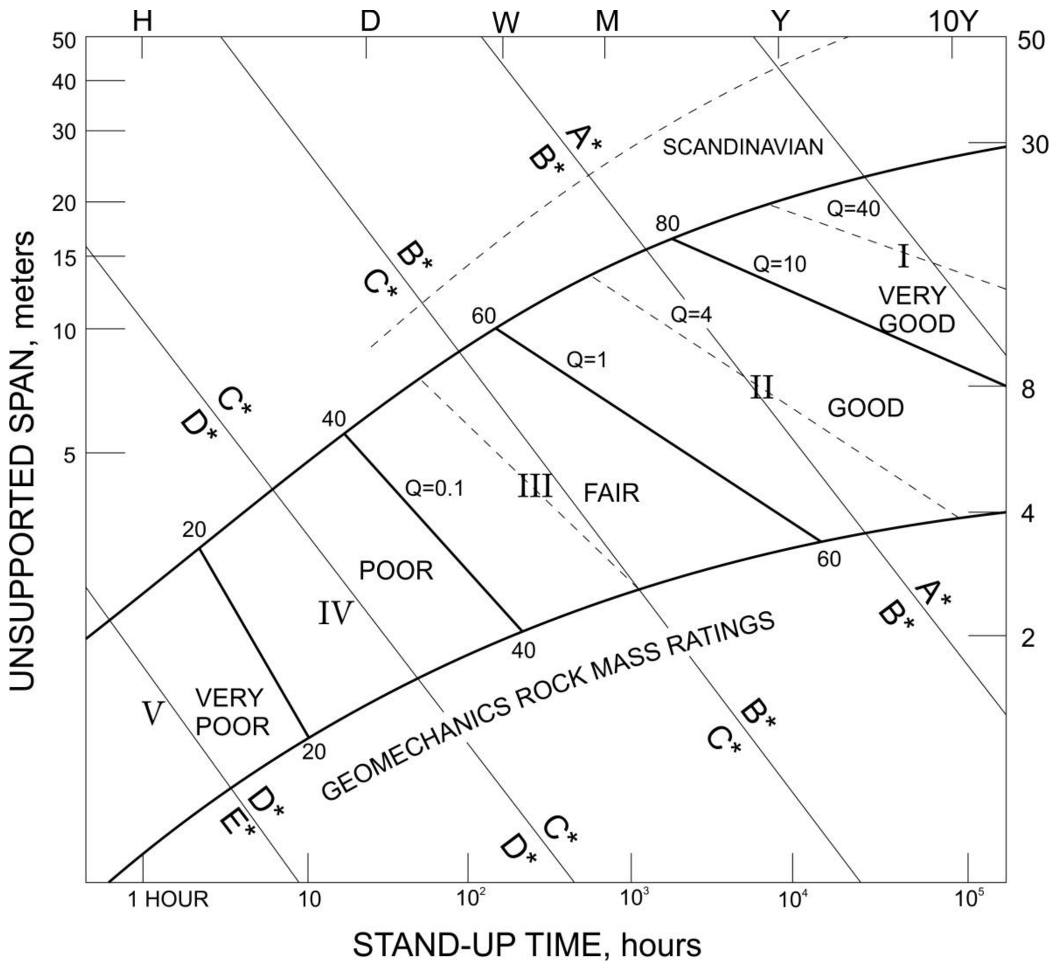


Figure 4. Relationship between the stand-up time and span for various rock mass classes according to the RMR system (after Lauffer, 1988, modified after Bieniawski, 1979).

1.7.5.2 Modifications to RMR for mining

Several modifications (Laubscher, 1977, 1984), Laubscher and Taylor (1976), and Laubscher and Page (1990) have been made to Bieniawski's Rock Mass Rating (RMR) system to be effectively used in mining applications as the original RMR system was based on civil engineering case histories (Bieniawski, 1989).

The modified RMR system (MRMR) adjusts the basic *RMR* value by considering the in-situ and induced stresses, stress changes and the effects of blasting and weathering, and support recommendations are proposed for the new value accordingly. Laubscher's *MRMR* system is based on case histories of caving operations (Hoek, 2007).

Another modification of RMR for block cave mining is made by Cummings et al. (1982) and Kendorski et al. (1983) resulting in the *MBR* (modified basic *RMR*) system. It implies different ratings for the original RMR parameters and the resulting *MBR* value adjustments for blast damage, induced stresses, structural features, distance from the cave front and size of the caving block. It presents support recommendations for isolated or development drifts as well as for the final support of intersections and drifts (Hoek, 2007).

1.7.5.3 Extension of RMR – Slope Mass Rating (SMR)

Romana (1985) developed an extension of the RMR system called slope mass rating (SMR) for use in rock slope engineering. It includes new adjustment factors for joint orientation and blasting/excavation to RMR system for slopes as shown below (Romana et al., 2003):

$$SMR = RMR + (F_1 * F_2 * F_3) + F_4$$

where $F_1 = (1 - \sin A)^2$

and *A = angle between the strikes of the slope and the joint =* $(\alpha_j - \alpha_s)$.

$F_2 = (\tan \beta_j)^2$

β_j - joint dip angle

For toppling, $F_2 = 1.0$

F_1 relates parallelism between joints and slope face strike, F_2 refers to joint dip angle in the planar mode of failure, F_3 reflects the relationship between slope and joints and F_4 is the adjustment factor for the method of excavation (Romana et al., 2003). Values of F_1, F_2, F_3, and F_4, and the classification categories of rock mass slope are shown in Table 8a and Table 8b respectively.

Table 8a. Adjustment rating of F_1, F_2, F_3, and F_4 for joints (Romana et al., 2003)

Joint Orientation	Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
P $\|\alpha_j - \alpha_s\|$	> 30	30 – 20	20 – 10	10 – 5	< 5
T $\|(\alpha_j - \alpha_s) - 180\|$	> 30	30 – 20	20 – 10	10 – 5	< 5
F_1 (for P & T)	0.15	0.40	0.70	0.85	1.00
P $\|\beta_j\|$	< 20	20 – 30	30 – 35	35 – 45	> 45
F_2 (for P)	0.15	0.40	0.70	0.85	1.00
F_2 (for T)	1.00	1.00	1.00	1.00	1.00
P $\beta_j - \beta_s$	> 10	10 – 0	0	0 – (-10)	<-10
T $\beta_j + \beta_s$	< 110	110 – 120	> 120	–	–
F_3 (for P & T)	0	-6	-25	-50	-60
Method	Natural slope	Presplitting	Smooth blasting	Blasting/Ripping	Deficient blasting
F_4	+15	+10	+8	0	-8

P, Plane failure; T, Toppling failure, α_j, joint dip direction; α_s, slope dip direction; β_j, joint dip; β_s, slope dip

Table 8b. Classification of Rock Slope according to SMT (Hoek, 2007)

SMR	Class	Description	Stability	Failure	Support
81 - 100	I	Very good	Completely stable	None	None
61 - 80	II	Good	Stable	Some blocks	Spot
41 - 60	III	Fair	Partially stable	Some joints or many wedges	Systematic
21 - 40	IV	Poor	Unstable	Planar or large wedges	Important/ Corrective
0 - 20	V	Very poor	Completely unstable	Large wedges or circular failure	Re-excavation

1.7.5.4 Limitations of RMR system

The output of RMR system can lead to overdesign of support systems because it is conservative (Bieniawski, 1989). For example, the no-support limit is too conservative and to adjust RMR at the no-support limit for opening size effects, Kaiser et al. (1986) suggested the following relation.

$$RMR(NS) = 22\ln(ED + 25)$$

where NS stands for No Support and ED is the equivalent dimension.
RMR system cannot be used reliably in weak rock masses because it is mostly based on case histories of competent rocks (Singh and Geol, 1999). This system is not useful for deciding excavation method.

1.7.6 Rock Tunneling Quality Index Q-System

The Q-system was developed in 1974 by Barton, Lien, and Lunde at the Norwegian Geotechnical Institute, Norway for the determination of rock mass characteristics and tunnel support requirements (Barton et al., 1974). This quantitative engineering system was proposed on the basis of an analysis of 212 hard rock tunnel case histories from Scandinavia (Bieniawski, 1989).

RMR and Q-Systems use essentially the same approach but different log-scale ratings, as Q-value is the product of the ratio of parameters while RMR is the sum of parameters (Hoek, 2007). The Q-rating is developed by assigning values to six parameters that are grouped into three quotients (Singh and Geol, 1999). The numerical value of the index Q ranges from 0.001 to a maximum of 1,000 on a logarithmic scale (Bieniawski, 1989). Value of Q is defined and is calculated as:

$$Q = \frac{RQD}{J_n}\frac{J_r}{J_a}\frac{J_w}{SWR}$$

where:

- *RQD* (Rock quality designation) $\geq$ 10 (measuring the degree of fracturing)
- J_n, Joint set number (number of discontinuity sets)
- J_r, Joint roughness number for critically oriented joint set (roughness of discontinuity surfaces)
- J_a, Joint alteration number for critically oriented joint set (degree of alteration or weathering and filling of discontinuity surfaces)
- J_w, Joint water reduction number (pressure and inflow rates of water within discontinuities)
- *SRF*, Stress reduction factor (presence of shear zones, stress concentrations, squeezing or swelling rocks)

The first quotient (RQD/J_n) represents the rock mass geometry and is a measure of block/wedge size. Since RQD generally increases with decreasing number of discontinuity sets, the numerator and denominator of the quotient mutually reinforce one another (Hoek, 2007).

Table 9a. Rock Quality Designation, RQD (Barton et al., 1974)

1. Rock Quality Designation	*RQD*
A Very Poor	0 – 25
B Poor	25 – 50
C Fair	50 – 75
D Good	75 – 90
E Excellent	90 – 100

Note: (i) Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q. (ii) RQD interval of 5, i.e., 100, 95, 90, etc., are sufficiently accurate.

Table 9b. Joint Set number, J_n (Barton et al., 1974)

2. Joint Set Number	J_n
A Massive, no or few joints	0.5 – 1
B One joint set	2
C One joint set plus random joints	3
D Two joint set	4
E Two joint set plus random joints	6
F Three joint set	9
G Three joint set plus random joints	12
H Four or more joint sets, heavily jointed	15
J Crushed rock, earthlike	20

Note: (i) For intersections, use ($3.0 \times J_n$). (ii) For portals, use ($2.0 \times J_n$).

The second quotient (J_r/J_a) relates to inter-block shear strength i. e. it represents the roughness and frictional characteristics of the joint walls or filling materials (Singh and Geol, 1999). This quotient is weighted in favor of rough, unaltered joints in direct contact. High values of this quotient represent better 'mechanical quality' of the rock mass.

Table 9c. Joint roughness number, J_r (Barton et al., 1974)

3. Joint Roughness Number		J_r
	(a) Rock-wall contact, and (b) Rock wall contact before 10 cm shear	
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
(c) No rock-wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock-wall contact	1.0
J	Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact	1.0

Note: (i) Descriptions refer to small and intermediate scale features, in that order. (ii) Add 1.0 if the mean spacing of the relevant joint set ≥ 3 m. (iii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented for minimum strength.

Table 9d. Joint alteration number J_a (Barton et al., 1974)

	4. Joint Alteration Number	φ_r approx.	J_a
	(a) Rock-wall contact (no mineral fillings, only coatings)		
A	Tight healed, hard, non-softening, impermeable filling, i.e., quartz or epidote	–	0.75
B	Unaltered joint walls, surface staining only	25 – 35°	1.0
C	Slightly altered joint walls. Non-softening mineral coating, sandy particles, clay-free disintegrated rock, etc.	25 – 30°	2.0
D	Silty- or sandy-clay coatings, small clay fraction (non-softening)	20 – 25°	3.0
E	Softening or low friction mineral coatings, i.e., kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays	8 – 16°	4.0
	(b) Rock wall contact before 10 cm shear (thin mineral fillings)		
	Sandy particles, clay-free disintegrated rock, etc.	25 – 30°	4.0
G	Strongly over-consolidated non-softening clay mineral fillings (continuous, but < 5 mm thickness)	16 – 24°	6.0
H	Medium or low over-consolidated softening clay mineral fillings (continuous, but < 5 mm thickness)	12 – 16°	8.0

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J	Swelling-clay fillings, i.e., montmorillonite (continuous, but < 5 mm thickness). Value of J_a depends on the percent of swelling clay size particles, and access to water, etc.	6 – 12°	8 – 12

(c) No rock-wall contact when sheared (thick mineral fillings)			
K, L, M	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6 – 24°	6, 8, or 8 – 12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	–	5
O, P, R	Thick, continuous zones or bands of clay (see G, H, J for clay condition description)	6 – 24°	10, 13, or 13 – 20

The third quotient (J_W/SRF) is an empirical factor representing active stress incorporating water pressures and flows, the presence of shear zones and clay bearing rocks, squeezing and swelling rocks and in situ stress state (Hoek, 2007). According to Singh and Geol (1999), SRF is a measure of 1) loosening load in the case of an excavation through shear zones and clay bearing rock, 2) rock stress in competent rock, and 3) squeezing loads in plastic incompetent rocks. The quotient increases with decreasing water pressure and favorable in situ stress ratios.

Table 9e. Joint water reduction factor, J_w (Barton et al., 1974)

5. Joint Water Reduction Factor		Water pressure	J_w
A	Dry excavation or minor inflow, i.e., < 5 l/min locally	< 1 (kg/cm^2)	1.0
B	Medium inflow or pressure, occasional outwash of joint fillings	1 – 2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	2.5 – 10	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	2.5 – 10	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2 – 0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	>10 (kg/cm^2)	0.1 – 0.05

Note: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
(ii) Special problems caused by ice formation are not considered

Table 9f. Stress reduction factor, SRF (Barton et al., 1974)

6. Stress Reduction Factor	SRF

(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated		
A	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B	Single weakness zone containing clay or chemically disintegrated rock (depth of excavation ≤ 50 m)	5
C	Single weakness zone containing clay or chemically disintegrated rock (depth of excavation > 50 m)	2.5
D	Multiple shear zones in competent rock (clay-free) (excavation depth ≤ 50 m)	7.5
E	Single shear zone in competent rock (clay-free) (excavation depth ≤ 50 m)	5
F	Single shear zone in competent rock (clay-free) (excavation depth > 50 m)	2.5
G	Loose, open joint, heavily jointed (any depth)	5

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

(b) Competent rock, rock stress problems		σ_c / σ_1	σ_θ / σ_c	SRF
H	Low stress, near surface, open joints	200	< 0.01	2.5
J	Medium stress, favorable stress condition	200 – 10	0.01 – 0.03	1
K	High stress, very tight structure. Usually favorable to stability, may be unfavorable to wall stability	10 – 5	0.3 – 0.4	0.5 – 2
L	Moderate slabbing after > 1 hour in massive rock	5 – 3	0.5 – 0.65	5 - 50
M	Slabbing and rock burst after a few minutes in massive rock	3 – 2	0.65 – 1	50 – 200
N	Heavy rock burst (strain-burst) and immediate dynamic deformation in massive rock	< 2	> 1	200 – 400

Note: (ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1 / \sigma_3 \leq 10$, reduce σ_c to 0.75 σ_c; when $\sigma_1 / \sigma_3 > 10$, reduce σ_c to 0.5 σ_c; where σ_c is unconfined compressive strength, σ_1 and σ_3 are major and minor principal stresses, and σ_θ is maximum tangential stress (estimated from elastic theory). (iii) Few case records are available where the depth of crown below the surface is less than span width. Suggest increase in SRF from 2.5 to 5 for such cases (see H).

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(c) Squeezing rock: plastic flow in incompetent rock under the influence of high rock pressure		σ_θ / σ_c	SRF
O	Mild squeezing rock pressure	1 – 5	5 – 10
P	Heavy squeezing rock pressure	5	10 – 20

Note: (vi) Cases of squeezing rock may occur for depth H > 350 Q1/3. Rock mass compressive strength can be estimated from Q = 7 γ Q1/3 (MPa), where γ = rock density in g/cm^3.

(d) Swelling rock: chemical swelling activity depending on presence of water		SRF
R	Mile swelling rock pressure	5 – 10
S	Heavy swell rock pressure	10 – 15

Note: J_r and J_a classification is applied to the joint set or discontinuity that is least favorable for stability both from the point of view of orientation and shear resistance.

1.7.6.1 Applications of Q-System

Q value is applied to estimate the support measure for a tunnel of a given dimension, and the usage of excavation by defining the Equivalent Dimension (D_e) of the excavation (Barton et al., 1974):

$$D_e = \frac{\text{Excavation span(s), diameter (d) or height (m)}}{\text{Excavation Support Ratio (ESR)}}$$

Span/diameter is used for analyzing the roof support, and height of the wall is used in case of wall support.
The value of ESR (Table 10) depends upon the intended use of the excavation and the degree of its safety demanded (Singh and Geol, 1999).

Based on the relationship between the index Q and the equivalent dimension of the excavation, 38 different support categories have been suggested (Figure 5), and permanent support has been recommended for each category in the support tables (Barton et al., 1974). To supplement these recommendations, Barton et al. (1980) proposed to determine the rock bolt length (L) and the maximum support spans (S_{max}) from the following equations respectively.

$$L = 2 + (0.15 \cdot B / ESR)$$

where B is the excavation width.

$$S_{max} = 2 \cdot ESR \cdot Q^{0.4}$$

Since the early 1980s, due to the increased use of wet mix steel fiber reinforced shotcrete (SFRS) together with rock bolts, Grimstad and Barton (1993) suggested a different support design chart using SFRS, as shown in Figure 6. This chart is recommended for tunneling in poor rock conditions (Singh and Geol, 1999).

Grimstad and Barton (1993) suggested that the relationship between Q and the permanent roof support pressure (P_{roof}) is estimated from:

$$P_{roof} = \frac{2\sqrt{J_n}Q^3}{3J_r}$$

Q-value in relation with overburden thickness (H) can also be used to identify squeezing in underground structures using the following equation (Singh, Jethwa, Dube, & Singh, 1992).

$$H = 350Q^{1/3}$$

where H is in meters.
Overburden thickness (H) greater than 350 $Q^{1/3}$ indicates squeezing conditions and value of H less than 350 $Q^{1/3}$ generally represents non-squeezing conditions. Another application of Q-system is that it can be used to estimate deformation modulus of the rock mass (Em) of good quality by using equations below (Grimstad & Barton, 1993).

$$Em = 25\log Q$$
$$\text{for} \quad Q > 1$$

$$Em = 10\left(\frac{Q\sigma_c}{100}\right)^{1/3}$$

$$Em = 10^{(15\log Q + 40)/40}$$

Table 10. Values of Excavation Support Ratio, ESR (Barton et al. 1974)

Excavation category		**ESR**
A	Temporary mine openings	3-5
B	Permanent mine openings, water tunnels for hydro power (excluding high-pressure penstocks), pilot tunnels, drifts and headings for large excavation	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	1.3
D	Power stations, major road, and railway tunnels, civil defense chambers, portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories	0.8

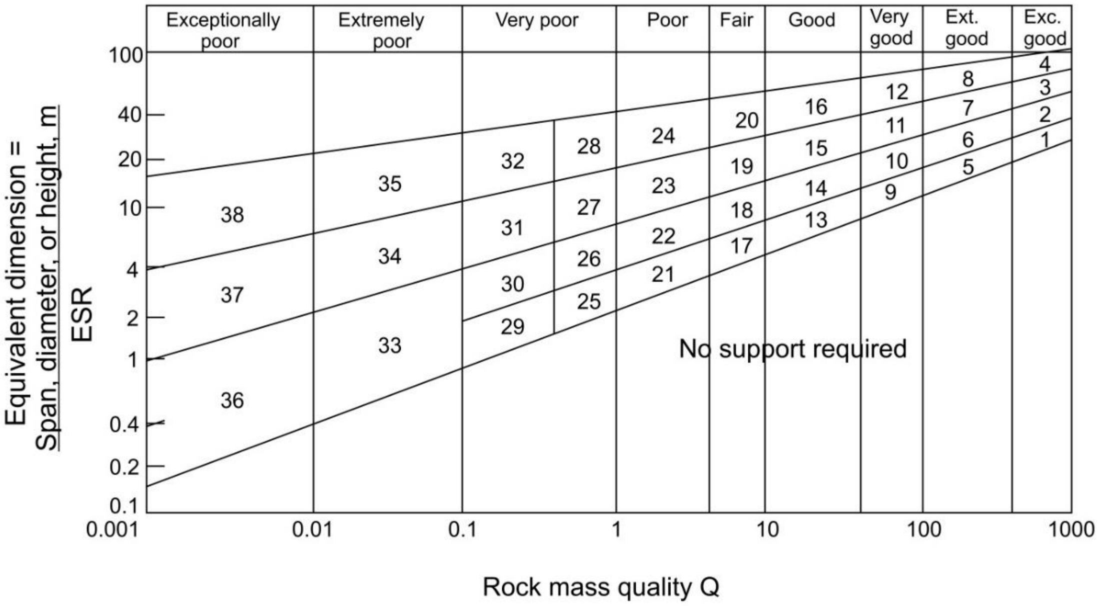
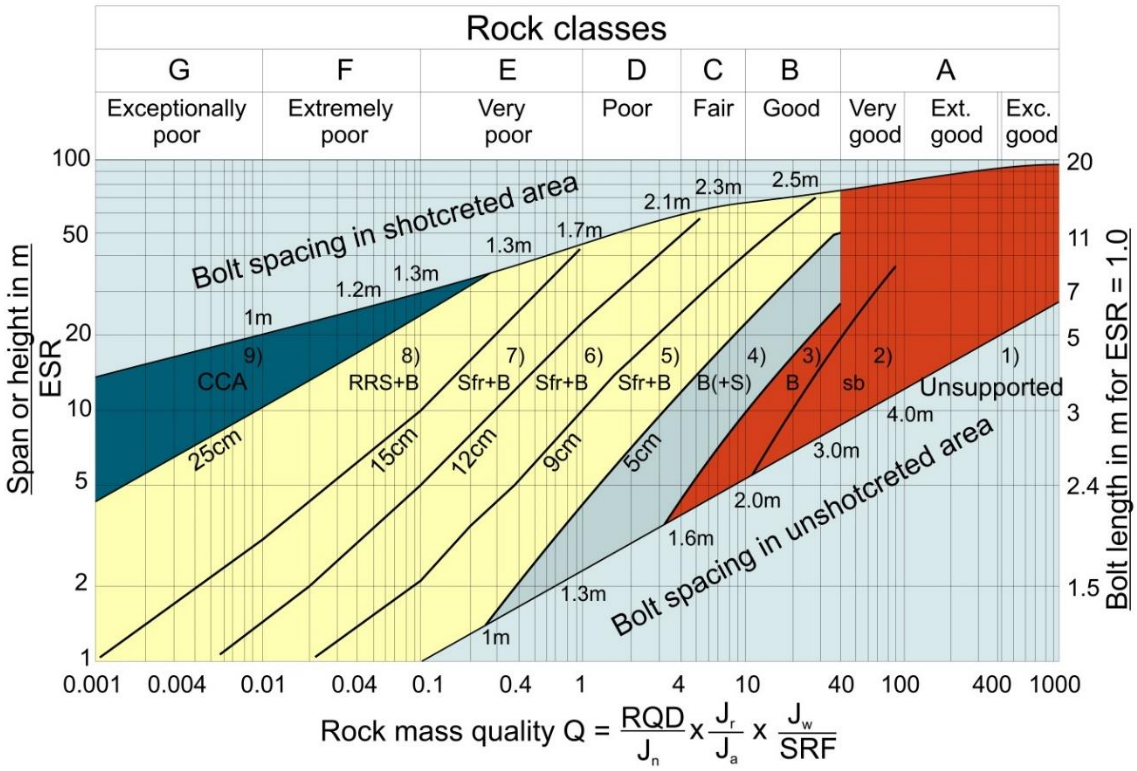


Figure 5. Tunnel support chart showing 38 support categories (Barton et al., 1974)



REINFORCEMENT CATEGORIES:

1) Unsupported
2) Spot bolting, sb
3) Systematic bolting, B
4) Systematic bolting. (and unreinforced shotcrete, 4-10cm), B(+S)
5) Fibre reinforced shotcrete and bolting, 5-9cm, Sfr+B
6) Fibre reinforced shotcrete and bolting, 9-12cm, Sfr+B
7) Fibre reinforced shotcrete and bolting, 12-15cm, Sfr+B
8) Fibre reinforced shotcrete >15cm, reinforced ribs of shotcrete and bolting, Sfr, RRS+B
9) Cast concrete lining, CCA

Figure 6. Different Support Categories (type of support) for different rock mass classes defined by the Q or Q_c relationships and the support width or height (Grimstad and Barton, 1993)

1.7.6.2 Q-System modified for UCS

Since 1974, the number of quoted case histories evaluated has increased to over 1260. Due to the incorporation of new data and improvements in excavation support methods and technologies, Q-System has been modified many times and has led to new relationships and support modifications (Barton, 2002). After realizing that the engineering properties gets affected by the uniaxial compressive strength σ_c of the intact rock between discontinuities, a normalization factor was applied to the original Q-value for hard rocks resulting in a new value Q_c as shown below (Barton, 2002):

$$Q_c = \left[\frac{RQD}{J_n}\frac{J_r}{J_a}\frac{J_w}{SRF}\right]\frac{\sigma_c}{100}$$

The relationship between the modified Q value i.e. Q_c and the Seismic Velocity V_p, depth (H), Rock Mass Modulus, required support pressures (P_r), porosity, and Uniaxial Compressive Strength σ_c has been established and presented in the form of a chart as shown in Figure 7 (Barton, 2002).

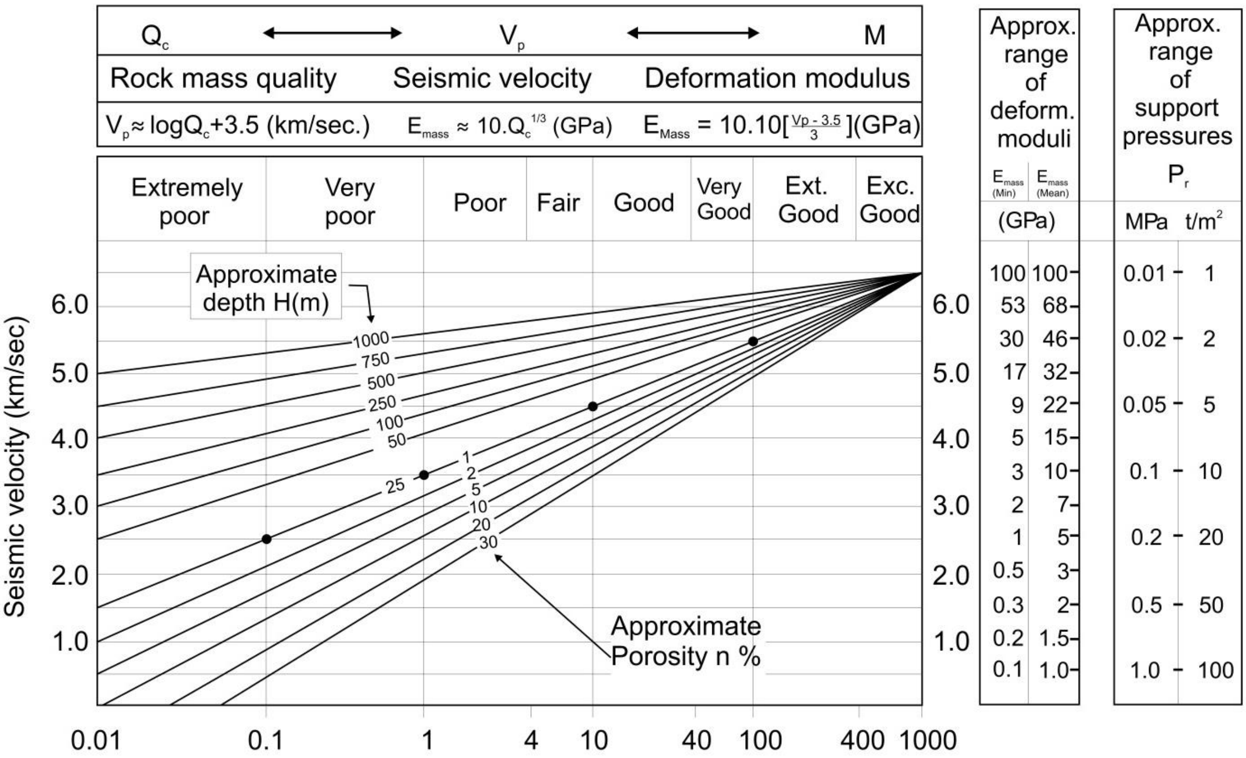


Figure 7. An integration of Seismic Velocity V_p, Q_c index, depth (H), Rock Mass Modulus, required support pressures (P_r), porosity, and Uniaxial Compressive Strength σ_c (Barton, 2002)

1.7.6.3 Correlation between the RMR and Q-System

Bieniawski (1976) has developed the following correlation between the Q-index and the RMR in the form of a semi-log equation.

$$RMR = 9\log Q + A$$

where A varies between 26 and 62, and the average of A is 44 (derived from 111 case histories in tunneling).

A similar relation was derived by Abad et al. (1983) on the basis of 187 case histories in coal mining:

$$RMR = 10.5 \log Q + 42$$

Further comparisons between Q and RMR systems are given by Barton (1988). It is advised to relate Q and RMR with caution (Bieniawski, 1989).

1.7.6.4 Limitations of Q system

It is difficult to obtain the Stress Reduction Factor SRF in the Q-system and any of its value covers a wide range of in-situ stress for rocks of a certain strength. As the importance of in situ stress on the stability of underground excavation is insufficiently represented in the Q-system, hence it cannot be used effectively in rock engineering design (Kaiser et al., 1986).
Use of open logarithmic scale of Q varying from 0.001 to 1000 as compared to the linear scale of up to 100 induces difficulty in using the Q-system (Bieniawski 1989). According to Palmstrom and Broch (2006), the ration RQD/J_n does not provide a meaningful measure of relative block size and the ratio J_w/SRF is not a meaningful measure of the stresses acting on the rock mass to be supported.
Q-system is not suitable for soft rocks; their best application is with drill and blast tunnels (mining origins) (Palmstrom & Broch, 2006).

1.7.7 Geological Strength Index (GSI)

Hoek in 1994 introduced the Geological Strength Index (GSI) as a way to facilitate the determination of rock mass properties of both hard and weak rock masses for use in rock engineering (Hoek, 1994). GSI resulted from combining observations of the rock mass conditions (Terzaghi's descriptions) with the relationships developed from the experience gained using the RMR-system (Singh and Geol, 1999). The relationship between rock mass structure (conditions) and rock discontinuity surface conditions is used to estimate an average GSI value represented in the form of diagonal contours (Figure 8). It is recommended to use a range of values of GSI in preference to a single value (Hoek, 1998). This simple, fast and reliable system represents nonlinear relationship for weak rock mass, can be tuned to computer simulation of rock structures (Singh and Geol, 1999) and can provide means to quantify both the strength and deformation properties of a rock mass.

In its primitive form, GSI related between four basic rock mass fracture intensities and the respective quality of those discontinuity surfaces. The rock mass structure ranged from blocky (cubical blocks formed by 3 orthogonal joint sets) to a crushed rock mass with poorly interlocked angular and rounded blocks. The surface conditions ranged from very rough, un-weathered and interlocked to slickensided with clayey coatings or thicker clay filling.

Since 1994, it has been modified by many authors (Cai et al., 2004; Hoek and Marinos, 2000; Hoek et al., 1998; Marinos and Hoek, 2000; Sonmez and Ulusay, 1999) and improved from a purely qualitative (in relation to assigning a value) to a quantitative relationship (Cai et al., 2004; Hoek and Marinos, 2000; Hoek et al., 1998; Marinos and Hoek, 2000; Sonmez and Ulusay, 1999, Marinos et al. 2005). To cover more complex geological features, such as shear zones and heterogeneous rocks, an additional category was added to the original chart to help characterize a highly sheared and folded flysch series known as the Athens Schist (Hoek et al., 1998) (Figure 9).

A group for massive rock has been included in which brittle Hoek-Brown parameters have been shown to be useful in predicting the breakout depth in deep hard rock excavations (Kaiser et al., 2000; Martin et al., 1999). Besides that, both axes for block size and joint conditions are quantified (Cai et al., 2004) (Figure 10). The joint spacing is the first indication of block size and is shown as varying from over 150 cm to less than 1 cm. The strength of a joint or block surface is quantified and represented by a factor called Joint Condition Factor (J_C) following (Barton and Bandis, 1990; Palmstrom, 1995b), and is defined as:

$$J_c = \frac{J_w \cdot J_s}{J_a}$$

where J_w is the Joint Waviness, J_s is the Joint Smoothness, and J_a represents Joint Alteration. These parameters are described in tables 11a, 11b, and 11c.
For persistent joints block volume V_0 is given by the equation (Cai et al., 2004):

$$V_0 = \frac{s_1 s_2 s_3}{\sin\gamma_1 \sin\gamma_2 \sin\gamma_3}$$

where s_i = spacing between joints in each set; γ_i = angles between the joints sets (Cai et al., 2004).

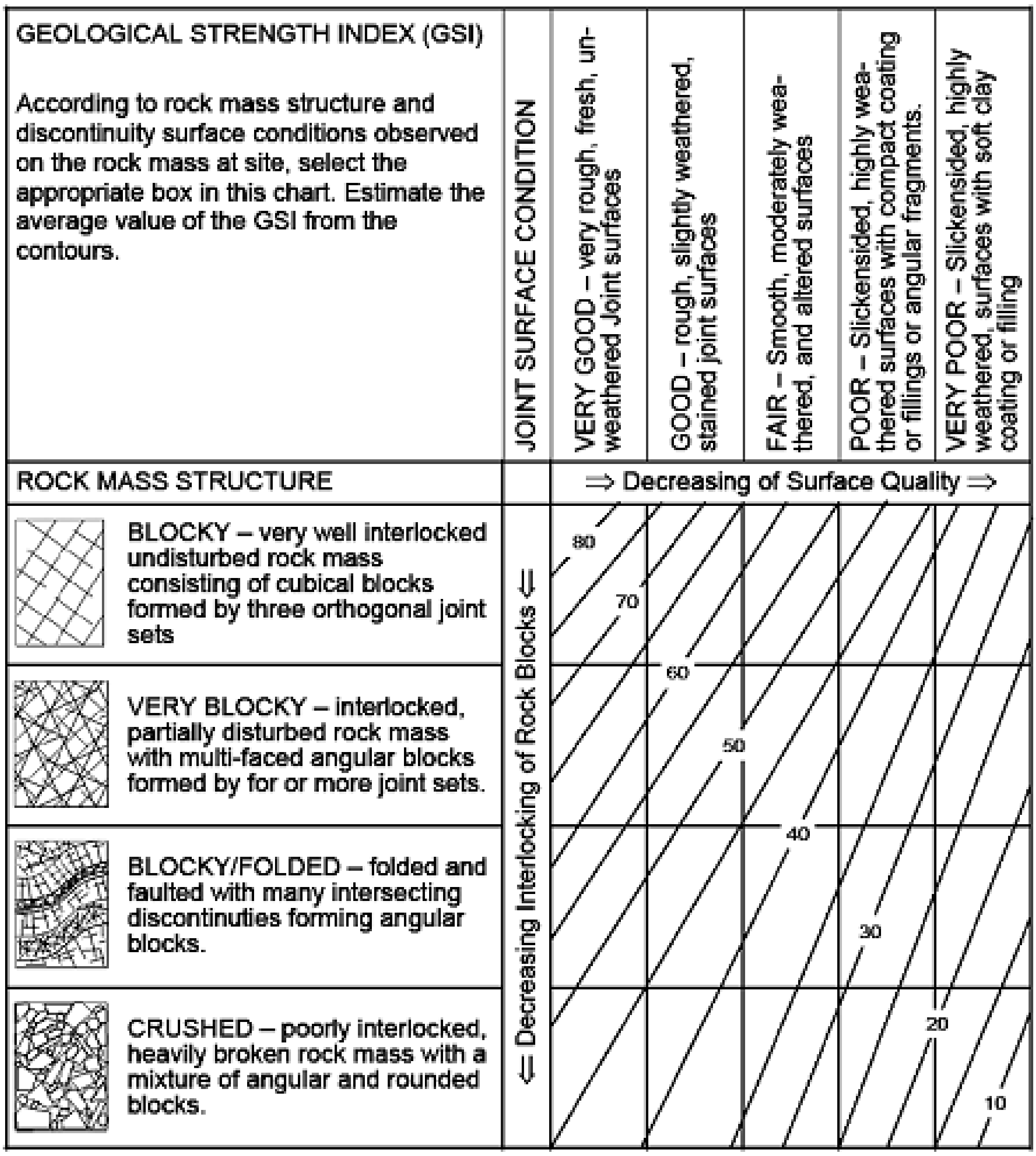


Figure 8. Estimate of Geological Strength Index (GSI) based on visual inspection of geological conditions (Hoek and Brown, 1997)

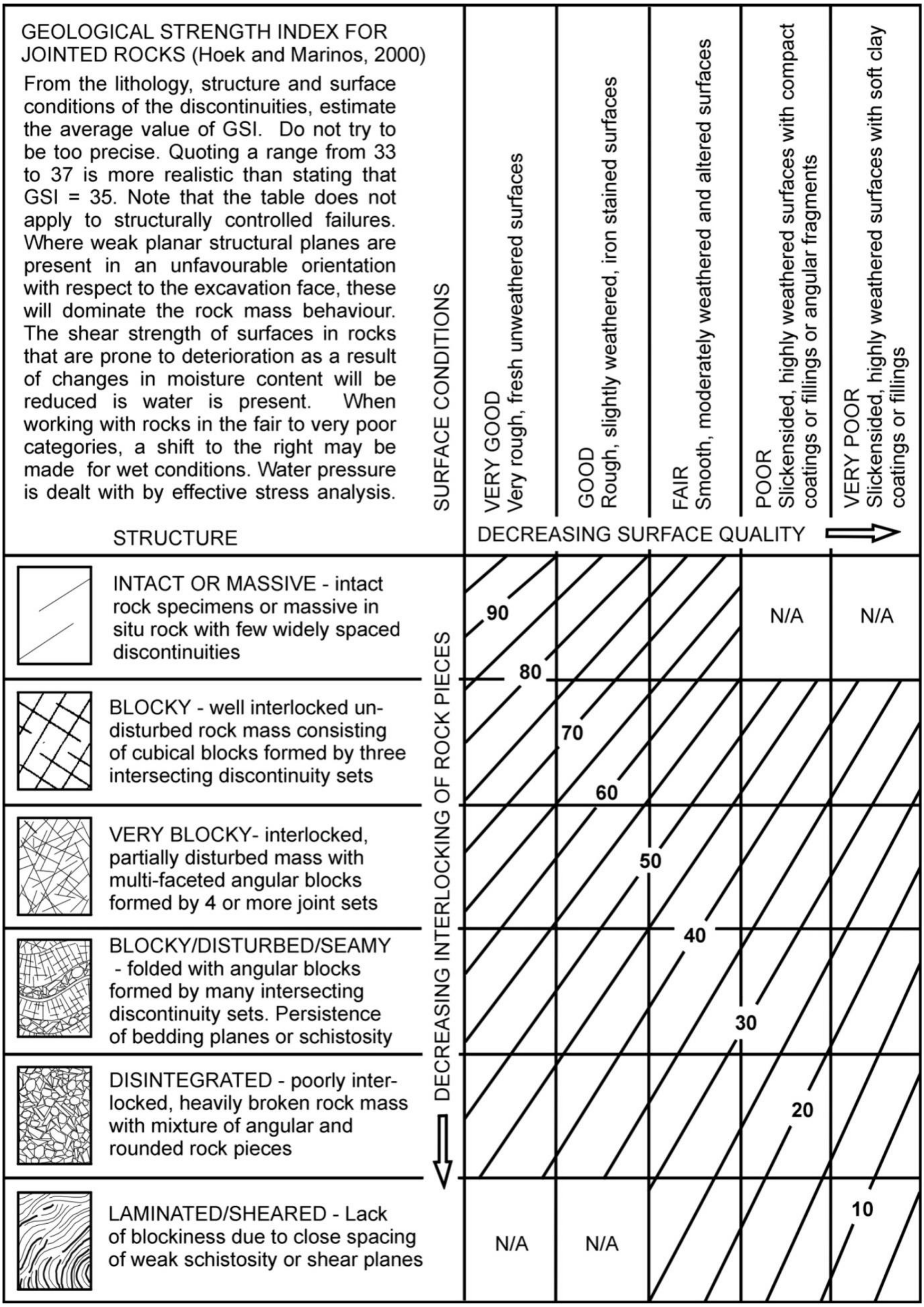


Figure 9. Modified table for estimating the Geological Strength Index (Hoek et al., 1998)

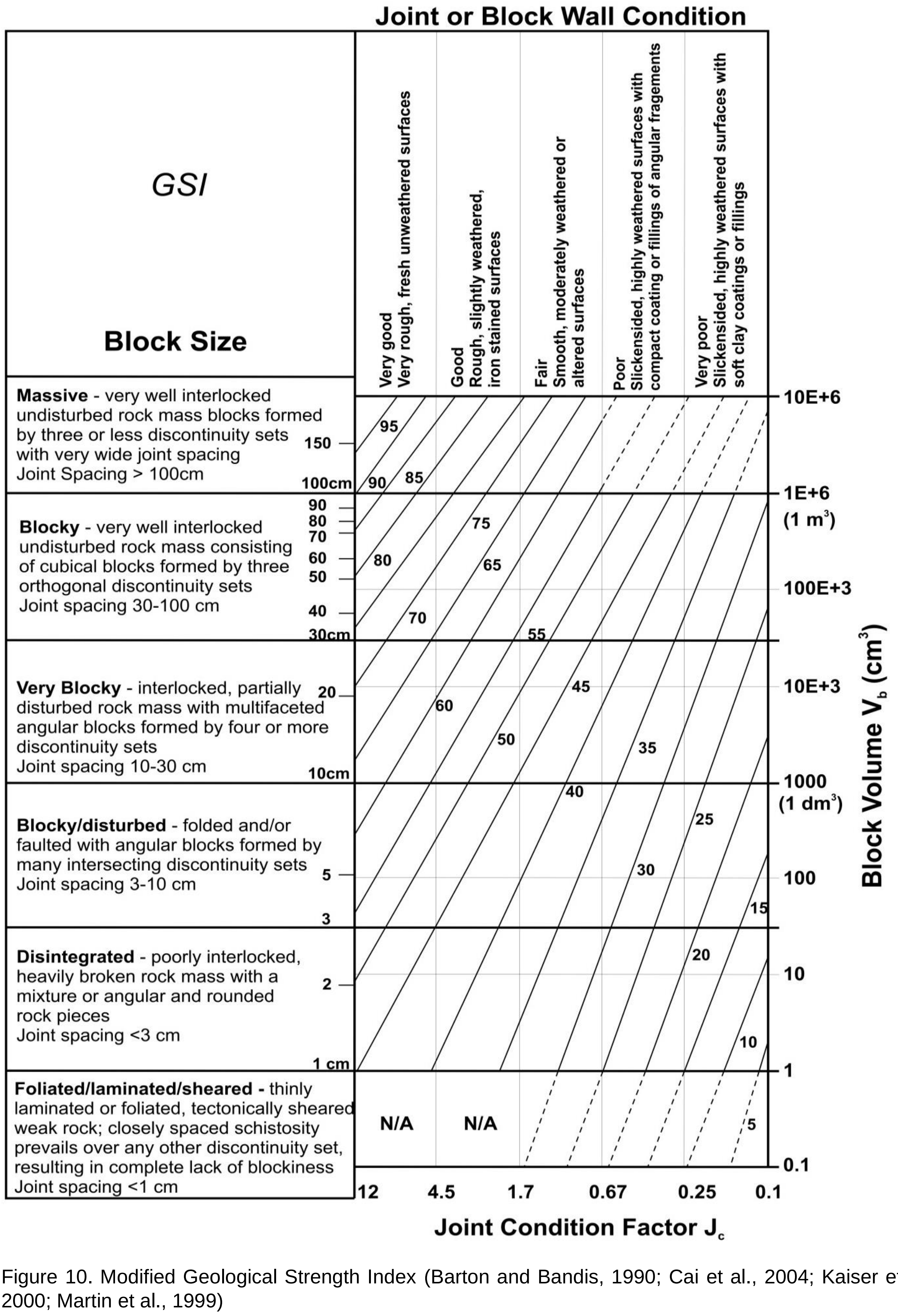


Figure 10. Modified Geological Strength Index (Barton and Bandis, 1990; Cai et al., 2004; Kaiser et al., 2000; Martin et al., 1999)

Table 11a. Terms to describe large-scale waviness (Palmstrom, 1995b)

Waviness terms	Undulation	Rating for waviness J_w
Interlocking (large scale)		3
Stepped		2.5
Large undulation	> 3 %	2
Small to moderate undulation	0.3 – 3 %	1.5
Planar	< 0.3 %	1

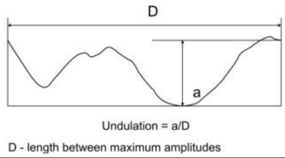


Table 11b. Terms to describe small-scale smoothness (Palmstrom, 1995b)

Smoothness terms	Description	Rating for smoothness J_s
Very rough	Near vertical steps and ridges occur with interlocking effect on the joint surface	3
Rough	Some ridge and side-angle are evident; asperities are clearly visible; discontinuity surface feels very abrasive (rougher than sandpaper grade 30)	2
Slightly rough	Asperities on the discontinuity surfaces are distinguishable and can be felt (like sandpaper grade 30 - 300)	1.5
Smooth	Surface appear smooth and feels so to touch (smoother than sandpaper grade 300)	1
Polished	Visual evidence of polishing exists. This is often seen in coating of chlorite and especially talc	0.75
Slickensided	Polished and striated surface that results from sliding along a fault surface or other movement surface	0.6 - 1.5

Block volume of non-persistent joints (Vb) can be calculated by the formula (Cai et al., 2004):

$$V_b = \frac{s_1 s_2 s_3}{\sin\gamma_1 \sin\gamma_2 \sin\gamma_3 \sqrt[3]{p_1 p_2 p_3}}$$

where s_i = spacing between joints in each set; γ_i = angles between the joints sets and p_i is the persistence factor (Cai et al., 2004).

Table 11c. Rating for the joint alteration factor jA (Palmstrom, 1995b)

	Term	Description	jA
Rock wall contact	*Clear joints*		
	Healed or "welded" joints (un-weathered)	Softening, impermeable filling (quartz, epidote, etc.)	0.75
	Fresh rock wall (unweathered)	No coating or filling on joint surface, except for staining	1
	Alteration of joint wall: slightly to moderately weathered	The joint surface exhibits one class higher alteration than the rock	2
	Alteration of joint wall: highly weathered	The joint surface exhibits two classes higher alteration than the rock	4
	Coating or thin filling		
	Sand, silt, calcite, etc.	Coating of frictional material without clay	3
	Clay, chlorite, talc, etc.	Coating of softening and cohesive minerals	4
Filled joints with partial or no contact between the rock wall surfaces	Sand, silt, calcite, etc.	Filling of frictional material without clay	4
	Compacted clay materials	"Hard" filling of softening and cohesive materials	6
	Soft clay materials	Medium to low over-consolidation of filling	8
	Swelling clay materials	Filling material exhibits swelling properties	8 - 12

1.7.7.1 Applications of GSI

The GSI was designed primarily to be used as a tool to estimate the parameters in the Hoek-Brown strength criterion for rock masses, and deformability and strength of rock mass using relationship modified from other classification systems (Hoek et al., 2002). Since the uniaxial strength of rock material is used as a basic parameter in Hoek-Brown strength criterion, therefore this parameter of rock strength is not included in GSI. GSI value is related to parameters of Hoek-Brown strength criterion as follows (Hoek, 1994; Hoek and Brown, 1997; Hoek et al., 2002):

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

$$m_b = 0.135 \cdot m_i \cdot Q^{1/3}$$

(Singh and Geol, 1999)

Where, m_i = material constant for intact rock in the Hoek-Brown failure criterion (to be found from triaxial test on rock cores or simply by table values corresponding to rock type)
m_b = material constant for broken rock in the Hoek-Brown failure criterion

GSI is related to s and a as follows (Hoek et al., 2002):

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

Also

$$s = 0.002 \cdot Q = JP^{\ln}$$

(Singh and Geol, 1999)

And

$$a = \frac{1}{2} + \frac{1}{6}\ e^{GSI/15} - e^{-20/3}$$

where, JP = jointing parameter (Palmstrom, 1995a)
- s = material constant in the Hoek-Brown failure criterion
- a = material constant for broken rock in the Hoek-Brown failure criterion
- D = Disturbance factor; the degree of disturbance caused by blast damage and stress relaxation

To predict the deformability and strength of rock mass, the relationship between the Rock Mass Modulus (Young‘s Modulus) and the GSI index, for poor rocks (σ_{ci} < 100 MPa) is defined as (Hoek et al., 2002):

$$E_{rm} = \left(1 - \frac{D}{2}\right) \cdot \sqrt{\frac{\sigma_c}{100}} \cdot 10^{\ GSI-10\ /40}$$

The rock mass modulus is expressed in GPa. D ranges from 0 for no damage to 1 for highly damaged (poor blasting), typical ranges for good blasting are reported to be around 0.7 to 0.8 (Hoek et al., 2002).

Hoek and Diedrichs (2006) improved the equation for estimating rock mass modulus E_{rm} and represented E_{rm} as a function of the disturbance due to blasting D, the GSI and the deformation modulus of the intact rock (E_i) by developing the following empirical relationship:

$$E_{rm} = E_i\left(0.02 + \frac{1 - D/2}{1 + e^{\ 60+15D-GSI\ /11}}\right)$$

To avoid the uncertainty of the intact deformation modulus caused by sample disturbance, E_i can be estimated using the modulus ratio MR and the uniaxial compressive strength σ_c of the intact rock defined by Deere (1968):

$$MR = \frac{E_i}{\sigma_c}$$

$$\Leftrightarrow E_i = MR\sigma_c$$

Values of Modulus Ratio (MR) for different rock types are presented by Hoek and Diedrichs (2006) as shown in Table 12.

1.7.7.2 Correlation between RMR, Q and GSI values

According to Hoek and Brown (1997), for competent rock masses (GSI > 25, RMR > 23), the value of GSI can be estimated from Rock Mass Rating RMR value as,

$$GSI = RMR_{89} - 5$$

RMR_{89} is the basic RMR value (1989 version of Bieniawski (1989), having the Groundwater rating set to 15 (dry), and the adjustment for joint orientation set to 0 (very favorable). For very poor quality rock masses (GSI < 25), the correlation between RMR and GSI is no longer reliable hence RMR classification should not be used for estimating the GSI values of such rock masses (Hoek and Brown, 1997).

For poor quality rock masses, GSI can be estimated from Q values (Barton et al., 1974) using the following relation (Singh and Geol, 1999).

$$GSI = 9 \ln Q' + 44$$

where Q' = modified tunneling quality index $(RQD/J_n)\cdot(J_r/J_a)$.

It is worth noting that each classification uses a set of parameters that are different from other classifications. That is why; estimating the value of one classification from another is not advisable (Eberhardt, 2010).

Table 12. Guidelines for the selection of modulus ration (MR) values in equation $E_i = MR. \sigma_{ci}$ based on Deere (1968) and Palmstrom and Singh (2001)

Rock Type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
Sedimentary	Clastic		Conglomerates 300-400 Breccias 230-350	Sandstone 200-350	Siltstones 350-400 Greywackes 350	Claystones 200-300 Shales 150-250 [a] Marls 150-200
Sedimentary	Non Clastic	Carbonates	Crystalline Limestones 400-600	Sparitic Limes-tones 600-800	Micritic Li-mestones 800-1000	Dolomites 350-500
Sedimentary	Non Clastic	Evaporites		Gypsum (350) [b]	Anhydrite (350) [b]	
Sedimentary	Non Clastic	Organic				Chalk 1000+
Metamorphic	Non-foliated		Marble 700-1000	Hornfels 400-700 Metasandstone 200-300	Quartzites 300-400	
Metamorphic	Slightly foliated		Migmatite 350-400	Amphibolites 400-500	Gneiss 300-750 [a]	
Metamorphic	Foliated			Schists 250-1100 [a]	Phyllites/Mica Schist 300-800 [a]	Slates 400-600 [a]
Igneous	Plutonic	Light	Granite [c] 300-550	Diorite [c] 300-400		
Igneous	Plutonic	Light		Granodiorite [c] 400-450		
Igneous	Plutonic	Dark	Gabbro 400-500	Dolorite 300-400		
Igneous	Plutonic	Dark		Norite 350-400		
Igneous	Hypabasal			Porphyries (400) [b]	Diabase 300-350	Peridotite 250-300
Igneous	Volcanic	Lava		Rhyolite 300-500 Andesite 300-500	Dacite 350-450 Basalt 250-450	
Igneous	Volcanic	Pyroclastic	Agglomerate 400-600	Volcanic breccia (500) [b]	Tuff 200-400	

[a] Highly anisotrophic rocks: the value of MR will be significantly different if normal strain and/or loading occurs parallel (high MR) or perpendicular (low MR) to a weakness plane. Uniaxial test loading direction should be equivalent to field application.

[b] No data available: estimated on the basis of geological logic.

[c] Felsic Granitoid: coarse grained or altered (high MR), fine grained (low MR).

1.7.7.3 Limitations of GSI System

GSI assumes the rock mass to be isotropic (Singh and Geol, 1999).

1.8 Other Classification Systems

As discussed earlier that, several other classification systems have been developed, some of them are listed in Table 1. Two of them are briefly discussed for their unique application in a certain aspect.

1.8.1 Rock Mass Number (N)

Rock Mass Number is the rock mass quality Q value when SRF is set at 1 (i.e., normal condition, stress reduction is not considered) (Geol et al., 1995). N can be computed as,

$$N = (RQD / J_n)(J_r / J_a)J_w$$

$$N = Q(SRF = 1)$$

The difficulty in obtaining Stress Reduction Factor SRF in the Q-system favors the use of this system. SRF in the Q-system is not sensitive in rock engineering design because SRF value covers a wide range. For instance, the value of SRF is 1 for the ratio of σ_c/σ_1 ranging from 10 ~ 200, i.e., for a rock with σ_c = 50 MPa, in situ stresses of 0.25 to 5 MPa yield the same SRF value. It shows that the significance of in situ stress for the stability of underground excavation is inadequately characterized in the Q-system (Geol et al., 1995).

Unlike Q-system, N system separates in situ stress effects from rock mass quality. In situ stress is the external cause of squeezing and is related to overburden depth (H). Rock Mass Number can be used effectively for predicting squeezing and its intensity in the underground excavation. The following equation presents the squeezing ground condition (Geol et al., 1995).

$$H = (275N^{1/3})B^{-0.1}$$

where H is the tunnel depth or overburden in meters and B is the tunnel span or diameter in meters.

The degree of squeezing can be characterized from the following equations (Geol et al., 1995). Mild squeezing occurs when $(275\ N^{1/3})\ B^{-0.1} < H < (450\ N^{1/3})\ B^{-0.1}$, moderate squeezing occurs when $(450\ N^{1/3})\ B^{-0.1} < H < (630\ N^{1/3})\ B^{-0.1}$ and high squeezing occurs at $H > (630\ N^{1/3})\ B^{-0.1}$.

1.8.2 Rock Mass Index, RMi

Rock Mass Index was proposed by Palmström (1995a) to characterize rock mass strength as a construction material. It demonstrates the reduction in inherent strength of rock mass due to different adverse effects of joints (Singh and Geol, 1999). In other words, it denotes uniaxial compressive strength of the rock mass in MPa and is expressed as

$$RMi = \sigma_c \cdot JP$$

where σ_c is the uniaxial compressive strength of the intact rock material in MPa.
JP is the jointing parameter; composed of 4 joint characteristics, namely, block volume or joint density, joint roughness jR (Table 13a), joint size jL (Table 13b) and joint alteration jA (Table 13c). JP is reduction factor representing the effects of jointing on the strength of rock mass. JP is 1 for intact rock and is 0 for crushed rock masses (Singh and Geol, 1999). The four jointing parameters can be used to calculate jointing parameter as (Singh and Geol, 1999)

$$JP = 0.2(jC)^{0.5} \cdot (Vb)^{D}$$

where Vb is given in m^3, and $D = 0.37 \cdot jC^{0.2}$

Joint condition factor jC is correlated with *jR*, *jA* and *jL* as follows (Singh and Geol, 1999):

$$jC = jL(\frac{jR}{jA})$$

The overall rating of RMi and the classification is presented in Table 14.

Table 13a. The Joint Roughness J_R factor (Palmstrom, 1996)

Small Scale Smoothness of Joint Surface*	Large Scale Waviness of Joint Plane Planar	Slightly undulating	Strongly undulating	Stepped	Interlocking
Very rough	3	4	6	7.5	9
Rough	2	3	4	5	6
Slightly rough	1.5	2	3	4	4.5
Smooth	1	1.5	2	2.5	3
Polished	0.75	1	1.5	2	2.5
Slickensided**	0.6 – 1.5	1 – 2	1.5 – 3	2 – 4	2.5 – 5
	For irregular joints, a rating of jR = 5 is suggested				

*for filled joints: jR = 1; ** for slickensided joints the value of R depends on the presence and outlook of the striations; the highest value is used for marked striations

Table 13b. The Joint Length and Continuity Factor jL (Palmstrom, 1996)

Joint Length (m)	Term	Type	jL Continuous joints	Discontinuous joints**
< 0.5	Very short	Bedding/foliation parting	3	6
0.1 – 1.0	Short/small	Joint	2	4
1 – 10	Medium	Joint	1	2
		…		
10 – 30	Long/Large	Joint	0.75	1.5
>30	Very long/large	Filled joint seam* or shear*	0.5	1

*often a singularity, and should in these cases be treated separately
**Discontinuous joints end in massive rock mass

Table 13c. Characterization and rating of the Joint Alteration Factor *jA* (Palmstrom, 1996)

Term	Description		*jA*
A. Contact between rock wall surfaces			
Clean joints			
Healed or welded joints	Softening, impermeable filling (quartz, epidote, etc.)		0.75
Fresh rock walls	No coating or filling on joint surface, except staining		1
Alteration of joint wall			
i. 1 grade more altered	The joint surface exhibits one class higher alteration than the rock		2
ii. 2 grade more altered	The joint surface shows two classes higher alteration than the rock		4
Coating or thin filling			
Sand, silt, calcite, etc.	Coating of friction materials without clay		3
Clay, chlorite, talc, etc.	Coating of softening and cohesive minerals		4
B. Filled joints with partial or no contact between the rock wall surfaces			
Type of filling material	Description	Partly Wall Contact (thin filling <5mm*)	No Wall Contact (thick filling or gouge)
Sand, silt, calcite, etc.	Filling of friction material without clay	4	8
Compacted clay materials	"Hard" filling of softening and cohesive materials	6	10
Soft clay materials	Medium to low over consolidation of filling	8	12
Swelling clay materials	Filling material exhibits clear swelling properties	8 – 12	12 – 20

*Based on joint thickness division in the RMR system (Bieniawski, 1973).

Table 14. Classification based on RMi (Palmstrom, 1996)

TERM		RMi Value
For RMi	Related to Rock Mass strength	
Extremely low	Extremely weak	< 0.001
Very low	Very weak	0.001 – 0.01
Low	Weak	0.01 – 0.1
Moderate	Medium	0.1 – 1.0
High	Strong	1.0 – 10
Very high	Very strong	10 – 100
Extremely high	Extremely strong	>100

1.8.2.1 Applications of RMi

According to (Palmstrom, 1996), RMi can easily be used for rough estimates in the early stages of a feasibility design of a project. This system offers a stepwise system suitable for engineering judgment. Using RMi, values of the parameter (s) of Hoek-Brown Criterion can be easily and more accurately determined by the relation $s = JP^2$. Thus, use of parameters in RMi can improve inputs in other classification systems. RMi system covers a wide range of rock mass variations; hence it has wider applications than other classification systems.

1.9 Conclusions

From the above discussion about the different classification systems, it can be concluded that classification systems are meant to assist the engineer and engineering geologist in estimating the conditions of the rock mass in areas where samples or observations cannot be made. These systems allow estimating the rock mass strength and deformability through homogenizing the influence of discontinuities and the intact rock into a pseudo-continuum. Therefore they do not consider how discontinuities or local changes in the rock mass conditions influence the failure characteristics (modes and mechanisms) of the rock mass. They are a tool that can be misused when applied in situations where they are not applicable. Therefore, they have limited applicability in regions where distinct structures dominate.

Although these systems give a rational and quantified assessment, they guide the rock mass characterization process and assist in communication but still, there is room for improvement in these systems.

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