Chapter 3

Rock Mass Description

3.1. Introduction

Rock mass classifications constitute the foundation of experimental geotechnical design and are widely used in rock engineering. It is proven that the classification of rock masses can be used as a powerful tool in rock engineering if the parameter estimations are done accurately. In fact, in most projects, it forms the basis of complicated underground designs, for example an underground Ice-hockey stadium in Gjovik of Norway with 60 m diameter was designed based on rock mass classification. The minimum and maximum ratings in these classifications are designated to the weakest and strongest rock mass respectively and each classification parameter significantly governs the final rating of rock mass quality (Singh and Goel, 1999). However, classification is frequently used in the initial phase of a project to foresee the rock mass quality and the probable support required. The consequence is an assessment of the stability quantified in subjective terms such as bad, fair, good and excellent conditions.

Several rock mass quantitative classification systems are established in South Africa, United States, Europe and India which present satisfactory outcomes mainly due to:

- Better relationship between geologists, designers, contractors and engineers;
- Coincidence with observations, experiences and engineering judgments and
- Introducing quantitative results for engineers.

The classification systems were updated with new developments in the rock support technology over the past 50 years. These improvements started with steel arches and progressed over time to more innovative methods such as rockbolts and reinforced shotcrete (with steel fibre) as well as instrumentation and monitoring devices for geotechnical control purpose (Singh and Goel, 1999).

The early 1960s were very important in the general development of rock engineering throughout the world, due to several disastrous failures that happened which obviously established that, in rock and soil, ‘we were over-stepping the limits of our ability to predict the consequences of our actions’ (Terzaghi and Voight, 1979).

The failure of the Malpasset concrete dam in France in 1959 and the Vajont dam in Italy had a major influence on rock mechanics in geotechnical engineering and a large number of articles were introduced on the possible reasons of the failures (Jaeger, 1972). These incidents were responsible for the commencement of several research programmes that resulted in major progress in the techniques used in rock engineering.

The four most commonly used rock mass classification systems today are the geomechanics classification or rock mass rating (RMR, Bieniawski, 1974- South Africa) the Norwegian Geotechnical Institute (NGI) index (Q system, Barton et al., 1974), rock quality designation (RQD), which was introduced by Deer in 1963 as an index for assessing rock quality quantitatively, and also recently a classification introduced by Hoek et al. (1995) named geological strength index (GSI). In this research, the Asmari Formation succession rock mass has been classified by these four methods. Since different classification/characterisation systems pay attention to different parameters, it is often recommended that at least two methods should be used when classifying a rock mass (Hoek, 2000).

3.2. Engineering Rock Mass Classification

3.2.1. Rock Quality Designation (RQD)

Deere (1963) introduced an index to assess rock quality quantitatively, called the rock quality designation (RQD). The RQD (Table 3.1) is a core recovery percentage that is indirectly based on the number of fractures and the amount of softening in the rock mass that is observed from drill cores. Only the intact pieces with a length longer than 100 mm are summed and divided by the total length of the core run (Deere, 1968).

It is used as a standard parameter in drill core logging and its greatest value is perhaps its simplicity and quick determination, and also that it is inexpensive. RQD is to be seen as an index of rock quality where problematic rock that is highly weathered, soft, fractured, sheared and jointed is encountered in rock mass. This means that the RQD is simply a measurement of the percentage of good rock recovered from an interval of a borehole (Hoek, 2000).

The International Society of Rock Mechanics (ISRM) Commission on Standardization of Laboratory and Field Tests recommends RQD calculations using variable run lengths to separate individual beds, structural domains and weakness zones so as to indicate any inherent variability and provide a more accurate picture of the location and width of zones with low RQD values (Hoek, 2000). The relationship between the numerical value of RQD and the engineering quality of the rock mass as proposed by Deere (1968) is given in Table 3.1. RQD can also be found from the number of joints/ discontinuities per unit volume (Jv) on the rock surface. Palmstrom (1982) presented a relationship for a clay free rock mass along a tunnel:

$$RQD = 115 - 3.3\,Jv \qquad (3.1)$$

where Jv is known as the volumetric joint count and is the sum of the number of joints per unit length for all joint sets in a clay free rock mass. For $Jv < 4.5$, RQD = 100.

Palmstrom (1996) suggested a method to achieve better information from the surface instead of drill cores, though RQD depends on the borehole orientation. In principle, it is based on the measurement of the angle between each joint and the surface or the drill hole. The weighted joint density (wJd) is for measurements on rock surfaces and given by:

$$wJd = \frac{1}{\sqrt{A}} \times \sum \frac{1}{Sin\delta_1} \qquad (3.2)$$

And for measurements along a drill core or scan line:

$$wJd = \frac{1}{\sqrt{L}} \times \sum \frac{1}{Sin\delta_1} \qquad (3.3)$$

where δ_1 is the intersection angle, i.e., the angle between the observed plane or drill hole and the individual joint, A is the size of the observed area in m^2 and L is the length of the measured section along the core or scan line.

Table 3.1. Correlation between RQD and rock mass quality (after Deere, 1968).

RQD (%)	Rock Quality
< 25	Very Poor
25- 50	Poor
50- 75	Fair
75- 90	Good
90- 100	Excellent

The major rock mass classifications and parameters included in some of the classification systems are presented in Table 3.2.

3.2.1.1. Disadvantages of RQD

According to Merritt (1972), the RQD system has limitations in areas where the joints contain clay fillings. The clay fillings would reduce the joint friction and the RQD would be high despite the fact that the rock is unstable. It is unlikely as mentioned by Douglas et al. (1999) that all defects found in the boreholes would be of significance to the rock mass stability.

The RQD is not a good parameter in the case of a rock mass with joint distances near 100 mm. If the distance between continuous joints is 105 mm (core length), the RQD value will be 100%. If the distance between continuous joints is 95 mm, the RQD value will be 0%. If the parameter J_v (Palmstrom, 1982) should be used, its value would be close to 10 joints/ metre for both of the cases described above (Helgstedt, 1997). As mentioned by Milne et al. (1991), a rock mass with a calculated RQD of 100% could have 3 joint sets with an average spacing of 0.4 m or 1 joint set with spacing of several metres.

The RQD value may change significantly depending on the borehole orientation relative to the geological structure and according to Hoek et al. (1993), the use of the volumetric joint count is useful in reducing this dependence.

3.2.2. Rock Mass Rating (RMR)

Bieniawski (1973) introduced the Geomechanics Classification also named the Rock Mass Rating (RMR), at the South African Council of Scientific and Industrial Research (CSIR). The rating system was based on Bieniawski's (1984) experiences in shallow tunnels in sedimentary rocks. Over the years, this system has been successively refined as more case records have been examined and Bieniawski has made significant changes in the ratings assigned to different parameters. The following six parameters are used to classify a rock mass using the RMR system (Table 3.3):

1. Uniaxial compressive strength of rock material;
2. Rock Quality Designation (RQD);
3. Spacing of discontinuities;
4. Condition of discontinuities;
5. Groundwater conditions; and
6. Orientation of discontinuities.

In applying this classification system, the rock mass is divided into a number of structural regions and each region is classified separately. The boundaries of the structural regions usually coincide with a major structural feature such as a fault or with a change in rock type. In some cases, significant changes in discontinuity spacing or characteristics, within the same rock type, may necessitate the division of the rock mass into a number of small structural regions. The Rock Mass Rating system is presented in Table 3.3 giving the ratings for each of the six parameters listed above. These ratings are summed to give the RMR value.

Bieniawski (1989) published a set of guidelines for the selection of support in tunnels in rock for which the value of RMR has been determined (Table 3.4). Note that these guidelines have been published for a 10 m span horseshoe shaped tunnel, constructed using drill and blast-

Table 3.2. Major rock mass classification/characterisation systems (modified after Palmstrom 1995).

Name of Classification	Author and First version	Country of origin	Applications	Form and Type	Remarks
Rock load Theory	Terzhagi, 1946	USA	Tunnels with steel supports	Descriptive F Behaviour F Functional	Unsuitable for modern tunneling
Stand up time	Lauffer, 1958	Austria	Tunnelling	Descriptive F, General T	Conservative
NATM	Rebcewicz 1964/65 and 1975	Austria	Tunnelling in incompetent (overstressed) ground	Descriptive F Behaviouristic F, Tunnelling concept	Utilized in squeezing ground conditions
RQD	Deere et al, 1966	USA	Core logging, tunnelling	Numerical F, General T	Sensitive to orientation effects
A recommended rock classification for rock mechanical purpose	Patching and Coates, 1968		For input in rock mechanics	Descriptive F, General T	
The unified classification of soils and rocks	Deere et al, 1969	USA	Based on particles and blocks for communication	Descriptive F, General	
i)RSR concept	Wickham et al, 1972	USA	Tunnel with steel support	Numerical F, Functional T	Not useful with steel fibre shotcrete
RMR-system (CSIR)	Bieniawski 1974	South Africa	Tunnels, mines, foundations etc.	Numerical F, Functional T	Unpublished based case records
Q- system	Barton et al, 1974	Norway	Tunnels, large chambers	Numerical F, Functional T	
Mining RMR	Laubscher, 1975		Mining	Numerical F, Functional T	
The typological classification	Matula and Holzer, 1978		For use in communication	Descriptive F, General	
ii) The Unified Rock Classification System (URCS)	Williamson, 1980	USA	For use in communication	Descriptive F, General	
Basic geotechnical description (BGD)	ISRM, 1981	---	For general use	Descriptive F, General	
Rock mass strength (RMS)	Stille et al, 1982	Sweden		Numerical F, Functional T	Modified RMR
Modified basic RMR (MBR)	Cummings et al, 1982		mining	Numerical F, Functional T	
Simplified rock mass rating	Brook and Dharmaratne, 1985		Mines and tunnels	Numerical F, Functional T	Modified RMR and MRMR
Slope mass rating	Romana, 1985	Spain	Slopes	Numerical F, Functional T	
Ramamurthy Arora	Ramamurthy and Arora, 1993	India	For intact and jointed rocks	Numerical F, Functional T	Modified Deere and Miller approach
Geological Strength Index- GSI	Hoek et al, 1995	---	Mines and tunnels	Numerical F, Functional T	
Rock mass Number- N	Geol et al, 1995	India		Numerical F, Functional T	Stress- free Q-system
Rock mass index- RMi	Arild Palmstrom, 1995	Norway	Rock engineering, communication, characterisation	Numerical F, Functional T	

methods, in a rock mass subjected to a vertical stress < 25 MPa (equivalent to a depth below surface of < 900 m). The relationship between stand-up time, span and RMR classification in tunnels introduced by *Bieniawski* is shown in Figure 3.1.

Table 3.3. Rock Mass Rating (RMR) System (after Bieniawski 1989).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	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	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

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

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.
** Modified after Wickham et al (1972).

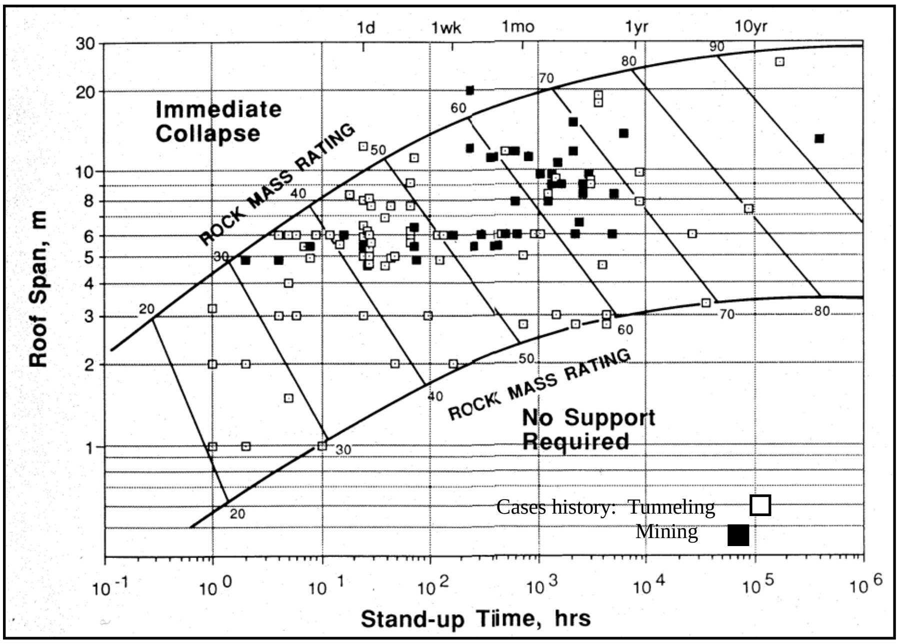

Figure 3.1. Relationship between Stand-up time, span and RMR classification (after Bieniawski (1989).

Table 3.4. Guidelines for excavation and support of 10 m span rock tunnels 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 face.	Locally, bolts in crown 3 m 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 face.	Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in 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 concurrently with excavation, 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls 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 concurrently 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 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

2.1.3. Rock Tunneling Quality Index, Q

On the basis of an evaluation of a large number of case histories of underground excavations, Barton et al (1974) of the Norwegian Geotechnical Institute proposed a

Tunneling Quality Index (*Q*) for the determination of rock mass characteristics and tunnel support requirements. The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1 000 and is defined by:

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF} \qquad (3.4)$$

Where;

RQD is the Rock Quality Designation
Jn is the joint set number
Jr is the joint roughness number
Ja is the joint alteration number
Jw is the joint water reduction factor
SRF is the stress reduction factor

In explaining the meaning of the parameters used to determine the Q value, Barton et al (1974) offer the following comments:
The first quotient (*RQD*/*Jn*), representing the structure of the rock mass, is a crude measure of the block or particle size, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400.
The second quotient (*Jr/Ja*) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favor of rough, unaltered joints in direct contact.
The third quotient (*Jw/SRF*) consists of two stress parameters. *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.

It can be regarded as a total stress parameter. The parameter *Jw* is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. Water may, in addition, cause softening and possible out-wash in the case of clay-filled joints. It has proved impossible to combine these two parameters in terms of inter-block effective stress, because paradoxically a high value of effective normal stress may sometimes signify less stable conditions than a low value, despite the higher shear strength (Hoek, 2000).
It appears that the rock tunneling quality Q, can now be considered to be a function of only three parameters which are crude measurments of:

1. Block size (*RQD/Jn*),
2. Inter-block shear strength (*Jr/ Ja),* and
3. Active stress (*Jw/SRF*)

Undoubtedly, there are several other parameters which could be added to improve the accuracy of the classification system. One of these would be the joint orientation.
If joint orientations had been included the classification would have been less general, and its essential simplicity lost. Table. 3.5 gives the classification of individual parameters used to obtain the Tunneling Quality Index Q for a rock mass (Hoek, 2000).
In relating the value of the index Q to the stability and support requirements of underground excavations, Barton et al. (1974) defined an additional parameter which they called the Equivalent Dimension, *De*, of the excavation. This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the *Excavation Support Ratio, ESR*.

The value of *ESR* is related to the intended use of the excavation and to the degree of security which is demanded of the support system installed to maintain the stability of the excavation. Barton et al (1974) suggest the following values shown in Table 3.6.
The equivalent dimension, De, plotted against the value of Q, is used to define a number of support categories in a chart introduced by Barton et al (1974). This chart has been updated by Grimstad and Barton (1993) to reflect the increasing use of steel fibre reinforced shotcrete in underground excavation support (Figure 3.2).
Barton et al (1980) provide additional information on rockbolt length, maximum unsupported spans and roof support pressure to supplement the support recommendations published in 1974.
The length, *L*, of rockbolts can be estimated from the excavation width, *B*, and the *ESR* by:

$$L = \frac{2 + 0.15B}{ESR} \qquad (3.5)$$

The maximum unsupported span can be estimated from:

$$\text{Maximum span (unsupported)} = 2\ ESR.\ Q^{0.4} \qquad (3.6)$$

Based upon analyses of case records, Grimstad and Barton (1993) suggest that the relationship between the value of Q and the permanent roof support pressure P_{roof} is estimated as:

$$P_{roof} = \frac{2\sqrt{Jn}\, Q^{-\frac{1}{3}}}{3Jr} \qquad (3.7)$$

Other correlations are:

$$P_{wall} = 0.7\ P_{roof} \qquad (3.8)$$

$$RMR = 9\ \ln Q + 44 \text{ (Bieniawski, 1989)} \qquad (3.9)$$

$$RMR = 5.9\ \ln Q + 43 \text{ (Rutledge and Preston, 1978)} \qquad (3.10)$$

Table 3.5. Classification of individual parameters used in the Tunneling Quality Index Q (after Barton et al., 1974).

DESCRIPTION	VALUE	NOTES	
1. ROCK QUALITY DESIGNATION	***RQD***		
A. Very poor	0 - 25	1. Where *RQD* is reported or measured as ≤ 10 (including 0),	
B. Poor	25 - 50	a nominal value of 10 is used to evaluate *Q*.	
C. Fair	50 - 75		
D. Good	75 - 90	2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently	
E. Excellent	90 - 100	accurate.	
2. JOINT SET NUMBER	J_n		
A. Massive, no or few joints	0.5 - 1.0		
B. One joint set	2		
C. One joint set plus random	3		
D. Two joint sets	4		
E. Two joint sets plus random	6		
F. Three joint sets	9	1. For intersections use ($3.0 \times J_n$)	
G. Three joint sets plus random	12		
H. Four or more joint sets, random,	15	2. For portals use ($2.0 \times J_n$)	
heavily jointed, 'sugar cube', etc.			
J. Crushed rock, earthlike	20		
3. JOINT ROUGHNESS NUMBER	J_r		
a. Rock wall contact			
b. Rock wall contact before 10 cm shear			
A. Discontinuous joints	4		
B. Rough and irregular, undulating	3		
C. Smooth undulating	2		
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is	
E. Rough or irregular, planar	1.5	greater than 3 m.	
F. Smooth, planar	1.0		
G. Slickensided, planar	0.5	2. J_r = 0.5 can be used for planar, slickensided joints having	
c. No rock wall contact when sheared		lineations, provided that the lineations are oriented for	
H. Zones containing clay minerals thick	1.0	minimum strength.	
enough to prevent rock wall contact	(nominal)		
J. Sandy, gravely or crushed zone thick	1.0		
enough to prevent rock wall contact	(nominal)		
4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
a. Rock wall contact			
A. Tightly healed, hard, non-softening,	0.75		1. Values of ϕr, the residual friction angle,
impermeable filling			are intended as an approximate guide
B. Unaltered joint walls, surface staining only	1.0	25 - 35	to the mineralogical properties of the
C. Slightly altered joint walls, non-softening	2.0	25 - 30	alteration products, if present.
mineral coatings, sandy particles, clay-free			
disintegrated rock, etc.			
D. Silty-, or sandy-clay coatings, small clay-	3.0	20 - 25	
fraction (non-softening)			
E. Softening or low-friction clay mineral coatings,	4.0	8 - 16	
i.e. kaolinite, mica. Also chlorite, talc, gypsum			
and graphite etc., and small quantities of swelling			
clays. (Discontinuous coatings, 1 - 2 mm or less)			

Table 3.5. continued.

DESCRIPTION	VALUE	NOTES	
4, JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
b. Rock wall contact before 10 cm shear			
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24	
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16	
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0 - 12.0	6 - 12	
c. No rock wall contact when sheared			
K. Zones or bands of disintegrated or crushed	6.0		
L. rock and clay (see G, H and J for clay	8.0		
M. conditions)	8.0 - 12.0	6 - 24	
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0		
O. Thick continuous zones or bands of clay	10.0 - 13.0		
P. & R. (see G.H and J for clay conditions)	6.0 - 24.0		
5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	
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)		10.0	1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth < 50 m)		5.0	
C. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth > 50 m)		2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)		7.5	
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)		5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)		2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)		5.0	

Table 3.5. continued.

DESCRIPTION	VALUE			NOTES
6. STRESS REDUCTION FACTOR			***SRF***	
b. Competent rock, rock stress problems				
	σ_c/σ_1	$\sigma_t\sigma_1$		2. For strongly anisotropic virgin stress field (if measured): when $5\leq\sigma_1/\sigma_3\leq10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When σ_1/σ_3 > 10, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
H. Low stress, near surface	> 200	> 13	2.5	
J. Medium stress	200 - 10	13 - 0.66	1.0	
K. High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10 - 5	0.66 - 0.33	0.5 - 2	
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10 - 20	3. Few case records available where depth of crown below surface is less than span width. Suggest *SRF* increase from 2.5 to 5 for such cases (see H).
c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				
N. Mild squeezing rock pressure			5 - 10	
O. Heavy squeezing rock pressure			10 - 20	
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5 - 10	
R. Heavy swelling rock pressure			10 - 15	

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass Quality (Q), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, *RQD* can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to *RQD* for the case of clay free rock masses: $RQD = 115 - 3.3 J_v$ (approx.), where J_v = total number of joints per m^3 ($0 < RQD < 100$ for $35 > J_v > 4.5$).
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of J_r/J_a is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating *Q*. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor *SRF* appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

Table 3.6. Excavation support ratio – ESR for various excavation categories (Barton et al., 1974).

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

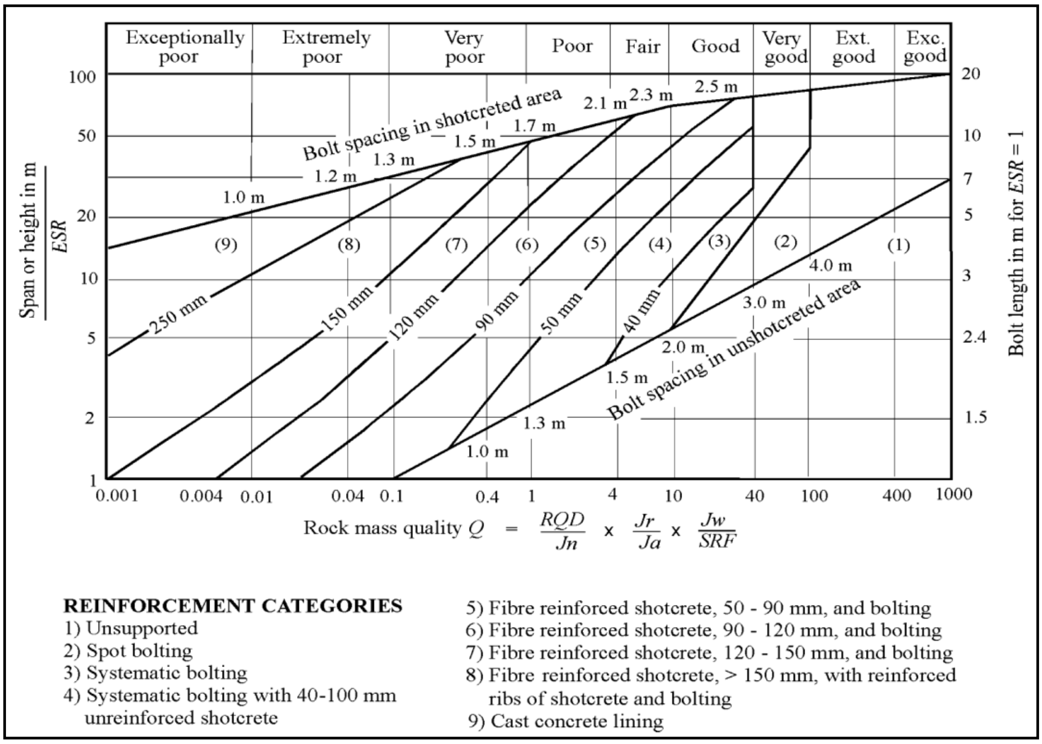

Figure 3.2. Estimated support categories based on the tunnelling quality index Q (after Grimstad and Barton, 1993).

2.1.4. Geological Strength Index (GSI)

The geological strength index (*GSI*) is a system of rock-mass characterization that has been developed in engineering rock mechanics to meet the need for reliable input data, particularly those related to rock-mass properties required as inputs into numerical analysis or closed form solutions for designing tunnels, slopes or foundations in rocks (Hoek, 2000). The geological character of rock material, together with the visual assessment of the mass it forms, is used as a direct input to the selection of parameters relevant for the prediction of rock-mass strength and deformability. This approach enables a rock mass to be considered as a mechanical continuum without losing the influence geology has on its mechanical properties. It also provides a field method for characterizing difficult-to-describe rock masses (Hoek, 2000).

The heart of the GSI classification is the careful engineering geological description of the rock mass which is essentially qualitative, because it was felt that the numbers associated with RMR and Q-systems were largely meaningless for weak and heterogeneous rock masses (Hoek, 2000). Note that the GSI system was never intended as a replacement for RMR or Q as it has no rock-mass reinforcement or support design capability as its only function is the estimation of rock-mass properties.

This index is based upon an assessment of the lithology, structure and condition of discontinuity surfaces in the rock mass and it is estimated from visual examination of the rock mass exposed in outcrops, in surface excavations such as road cuts and in tunnel face and borehole cores. The GSI, by combining the two fundamental parameters of the geological process, the blockiness of the mass and the conditions of discontinuities, respects the main geological constraints that govern a formation and is thus a geologically sound index that is simple to apply in the field (Hoek, 2000).

Once a GSI ‘‘number’’ has been decided upon, this number is entered into a set of empirically developed equations to estimate the rock-mass properties which can then be used as input into some form of numerical analysis or closed-form solution. The index is used in conjunction with appropriate values for the unconfined compressive strength of the intact rock σ_{ci} and the petrographic constant m_i, to calculate the mechanical properties of a rock mass, in particular the compressive strength of the rock mass (σ_{cm}) and its deformation modulus (E). Updated values of *mi* can be found in Marinos and Hoek (2000) or in the RocLab program (Hoek, 2000).

The geological strength index (***GSI***) based on two simple equations which were introduced by Hoek and Brown (1997) can be calculated indirectly as follows:

For $GSI \geq 18$ $RMR \geq 23$ $GSI = RMR - 5$ (3.11)

For $GSI < 18$ $GSI = 9\ln Q' + 44$ (3.12)

(Q′: Tunnelling Quality Index $Q' = [RQD/Jn].[Jr/Ja])$ (3.13)

Basic procedures are explained in Hoek and Brown (1997) but a more recent refinement of the empirical equations and the relation between the Hoek–Brown and the Mohr–Coulomb criteria have been addressed by Hoek et al. (2002) for appropriate ranges of stress encountered in tunnels and slopes. Attempts to ‘‘quantify’’ the GSI classification to satisfy the perception that ‘‘engineers are happier with numbers’’ (Sonmez and Ulusay, 1999; Cai et al. 2004) are interesting but have to be applied with caution.

The quantification processes used are related to the frequency and orientation of discontinuities and are limited to rock masses in which these numbers can easily be measured. The quantifications do not work well in tectonically disturbed rock masses in which the structural fabric has been destroyed. In such rock masses, it is recommended that the original qualitative approach based on careful visual observations is used (Hoek, 2000).

3.2.4.1. When not to Use GSI

The GSI classification system is based upon the assumption that the rock mass contains a sufficient number of ‘‘randomly’’ oriented discontinuities such that it behaves as an isotropic mass. In other words, the behavior of the rock mass is independent of the direction of the applied loads. Therefore, it is clear that the GSI system should not be applied to those rock masses in which there is a clearly defined dominant structural orientation. Undisturbed slate is an example of a rock mass in which the mechanical behavior is highly anisotropic and which should not be assigned a GSI value based upon the charts presented in Figure 3.3 (Hoek, 2000). However, the Hoek–Brown criterion and the GSI chart can be applied with caution if the failure of such rock masses is not controlled by their anisotropy (e.g. in the case of a slope when the dominant structural discontinuity set dips into the slope and failure may occur through the rock mass). For rock masses with a structure such as that shown in the sixth (last) row of the GSI chart (Figure 3.3), anisotropy is not a major issue as the difference in the strength of the rock and that of the discontinuities within it is small (Hoek, 2000).. It is also inappropriate to assign GSI values to excavated faces in strong hard rock with a few discontinuities spaced at distances of similar magnitude to the dimensions of the tunnel or slope under consideration. In such cases the stability of the tunnel or slope will be controlled by the three dimensional geometry of the intersecting discontinuities and the free faces created by the excavation. Obviously, the GSI classification does not apply to such cases. Geological description in the GSI should not only be limited to the visual similarity with the sketches of the structure of the rock mass as they appear in the charts, but the associated descriptions must also be read carefully, so that the most suitable structure is chosen. The most appropriate case may well lie at some intermediate point between the limited number of sketches or descriptions included in the charts (Hoek, 2000).

3.2.4.2. Projection of GSI values into the Ground

Outcrops, excavated slopes, tunnel faces, and borehole cores are the most common sources of information for the estimation of the GSI value of a rock mass. How should the numbers estimated from these sources be projected or extrapolated into the rock mass behind a slope or ahead of a tunnel?

Outcrops are an extremely valuable source of data in the initial stages of a project but they suffer from the disadvantage that surface relaxation, weathering and/or alteration may have significantly influenced the appearance of the rock-mass components. This disadvantage can be overcome (where permissible) by trial trenches but, unless these are machine excavated to considerable depth, there is no guarantee that the effects of deep weathering will have been eliminated. Judgment is therefore required in order to allow for these weathering and alteration effects in assessing the most probable GSI value at the depth of the proposed excavation (Hoek, 2000).

Excavated slope and tunnel faces are probably the most reliable source of information for GSI estimates provided that these faces are reasonably close to and in the same rock mass as the structure under investigation. In hard strong rock masses it is important that an appropriate allowance be made for damage due to mechanical excavation or blasting. As the purpose of estimating GSI is to assign properties to the undisturbed rock mass in which a tunnel or slope is to be excavated, failure to allow for the effects of blast damage when assessing GSI will result in the assignment of values that are too conservative. Therefore, if borehole data are absent, it is important that the engineering geologist or geologist attempts to ‘‘look behind’’ the surface damage and try to assign the GSI value on the basis of the inherent structures in the rock mass. This problem becomes less significant in weak and tectonically disturbed rock masses as excavation is generally carried out by ‘‘gentle’’ mechanical means and the amount of surface damage is negligible compared to that which already exists in the rock mass. Borehole cores are the best source of data at depth, but it has to be recognized that it is necessary to extrapolate the one-dimensional information provided by the core to the three-dimensional *in situ* rock mass (Hoek, 2000).

However, this is a problem common to all borehole investigations, and most experienced engineering geologists are comfortable with this extrapolation process. Multiple boreholes and inclined boreholes can be of great help in the interpretation of rock-mass characteristics at depth (Hoek, 2000).

For stability analysis of a slope, the evaluation is based on the rock mass through which it is anticipated that a potential failure plane could pass. The estimation of GSI values in these cases requires considerable judgment, particularly when the failure plane could pass through several zones of different quality. Mean values may not be appropriate in this case. For tunnels, the index should be assessed for the volume of rock involved in carrying loads, e.g. for about one diameter around the tunnel in the case of tunnel behavior or more locally in the case of a structure such as an elephant foot. For particularly sensitive or critical structures, such as underground powerhouse caverns, the information obtained from the sources discussed above may not be considered adequate, particularly as the design advances beyond the preliminary stages. In these cases, the use of small exploration tunnels can be considered and this method of data gathering will often be found to be highly cost effective (Hoek, 2000).

Figure 3.3 provides a visual summary of some of the adjustments discussed in the previous paragraphs. When direct assessment of depth conditions is not available, upward adjustment of the GSI value to allow for the effects of surface disturbance, weathering and alteration are indicated in the upper (white) part of the GSI chart. Obviously, the magnitude of the shift will vary from case to case and will depend upon the judgment and experience of the observer. In the lower (shaded) part of the chart, adjustments are not normally required as the rock mass is already disintegrated or sheared and this damage persists with depth (Hoek, 2000).

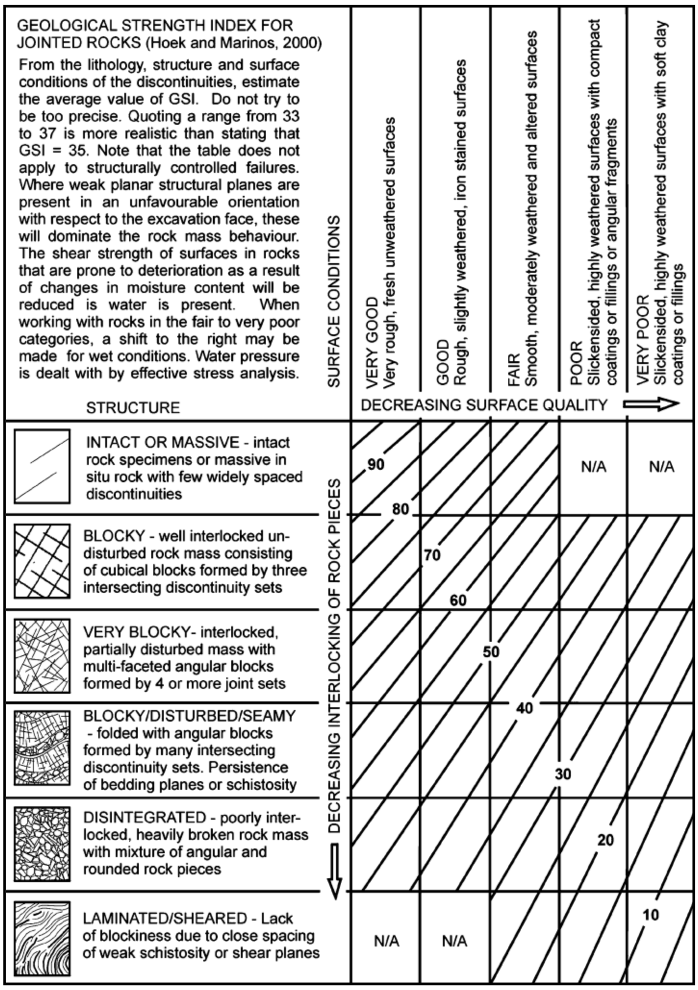

Figure 3.3. The General Geological Strength Index (GSI) chart for jointed rock masses estimates from the geological observations (after Hoek and Brown 1997, Hoek and Karzulovic, 2000).

3.2.5. Slope Stability

3.2.5.1. Slope Mass Rating (SMR)

The Asmari Formation limestones were classified using the *SMR* method (Romana, 1985). This classification is based on the RMR-system, by using an adjustment factor depending on the relation between the slope and joints and also a factor depending on the excavation method. The relationship between RMR and SMR is as follows:

$$\boldsymbol{SMR = RMR_{Basic} + (F_1.F_2.F_3) + F_4} \qquad (3.14)$$

Where F_1 depend on the parallelism between joints and the strike of the slope face as:

$$F_1 = (1 - \sin A)^2 \qquad (3.15)$$

A is the angle between the strike of the slope face and strike of the joint. The value of F_1 varies from 1.0 (nearly parallel) to 0.15 (when the angle is more than 30°) and the probability of failure is very low.

Table 3.7. Adjustments rating for joints (Romana, 1993).

Failure Type		Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable
P *T* *W*	***αj- αs*** ***αj-αs-180°*** ***αj- αs***	>30°	30°- 20°	20°- 10°	10°- 5°	<5°
P/T/W	***F1***	0.15	0.4	0.7	0.85	1.0
P *W*	***ßj*** ***ßj***	< 20°	20°- 30°	30°- 35°	35°- 45°	> 45°
P/W	***F2***	0.15	0.4	0.7	0.85	1.0
T	***F2***	1.0	1.0	1.0	1.0	1.0
P *W*	***ßj- ßs*** ***ßj- ßs***	> 10°	10°- 0°	0°	0°- (-10°)	< 10°
T	***ßj+ ßs***	< 110°	110- 120	> 120	-	-
P/W/T	***F3***	0	-6	-25	-50	-60

P*,** plane Failure; ***T, Toppling Failure; ***αj*,** joint dip direction; ***αs*,** Slope dip direction; ***ßj***, joint dip; ***ßs*,** slope dip

F_2 depends on the joint dip angle in planar failure mode, and its value varies from 1.0 (for joints dipping more than 45°) to 0.15 (for joints dipping less than 20°) and F_3 refers to the relationship between the slope face and joint dips. The value of F_3 is based on Bieniawski's (1976) figures and the conditions are fair when the slope face and joints are parallel. Unfavourable conditions occur when the slope dips 10° more than joints.

F_4 is the adjustment factor depending on the excavation method (Tables 3.7. and 3.8). As with most other classification systems, the SMR suggests need for and typt of support and describes five different classes. The tentative descriptions of SMR classes are shown in Table 3.9.

Table 3.8. Adjustment factor due to method of excavation of slopes (Romana, 1993).

Method	Natural slope	Presplitting	Smooth Blasting	Blasting or mechanical	Deficient blasting
F4	+15	+10	+8	0	-8

Table 3.9. The SMR classes (Romana, 1993).

Class	SMR	Description	Stability	Failures	Support
I	81- 100	Very Good	Completely stable	No failures	None
II	61- 80	Good	Stable	Some blocks	Occasional
III	41- 60	Normal	Partially stable	Planar failure in some joints and many wedge failures	Systematic
IV	21- 40	Bad	Unstable	Planar failure in many joints or big wedge failures	Important/ corrective
V	0- 20	Very Bad	Completely unstable	Big planar or soil- like	Re- excavation

3.2.5.2. Falling Rock Hazard Index (FRHI)

Falling Rock Hazard Index (FRHI) was developed based on work done earlier at the Oregon and Washington Department of Transportation of United States (Singh, 2004). FRHI has been developed for excavations that seem apparently stable, to determine the degree of dangerous situation to workers and installations in the immediate vicinity of the rock slope excavation at site. This method described herein considers rock slope parameters (Table 3.10).

Explanation of FRHI parameters:

- Face height
  The greater the face height, the greater the potential energy of falling rock, and consequently, the greater the danger to workers in the immediate vicinity.
- Face inclination
  rocks from a vertical slope free-fall while rocks from a slope angle of 30° to 60° bounce and roll, rendering greater hazard to workers. Vertical slopes and slopes <30° are safest in this regard. Slopes of 60° to 75° are worst (Ritchie 1963).
- Face irregularities
  Pfeiffer and Higgins (1990) claim that interaction of face irregularities with the falling rock is the most important factor in predicting rock fall behavior. The irregularities, or launching features. Determine the character of the bounce and the subsequent volatility of danger.
- Rock condition
  Rock mass conditions, such as, fractures, dip, dip direction, and discontinuities are a crucial indication of falling rock hazards. A highly fractured rock face exhibits more potential for rockfall hazard than a hard, intact rock face. The rock quality designation (RQD) developed by Deere et al. (1967) can be used for this.
- Spacing of discontinuity
  The spacing of the discontinuities is an indicator of how the planes of weakness affect the mechanical properties of the rock mass discontinuity under external force. Hence, closely linked discontinuities have more effect than isolated discontinuities, allowing smaller blocks to be easily detached on disturbance.
- Block size
  The larger the block size of falling rocks the greater the danger to the workers below.
- Volume of rockfall
  This is a highly visible indicator of the seriousness of falling rock. The more the amount and weight of falling rock, the worse the hazard. It is worthwhile noting that falling rock situations where the weight of rocks is above 45.36 kg should be closely evaluated for loss of structural integrity.
- Excavation method
  Several rock mass properties are compromised as a result of the excavation method. These excavation methods open existing discontinuities in the rock face and break the joint asperities. Excavation methods that cause less damage to the rock face are preferred over other types.
- Duration without remedy
  Long periods of exposure of a rock face to the natural elements allow increased weathering effects to take place. Weathering: can weaken the exposed face and lead to possible dislodging of rock pieces.

Table 3.10. FRHI worksheet (after Singh, 2004).

Falling Rock Hazard Index (FRHI)				
Face height Scoring breakdown	< 1.5 m < 1.5 m = 1	1.5m- 4.5 m 1.5 m 2 m = 2 2 m- 3 m = 3 3 m – 4 m = 5 4 m – 4.5 m = 6	4.5 m – 7.5 m 4.5 m 5 m = 7 5 m – 6 m = 8 6 m – 7.5 m = 9	> 7.5 m 7.5 m – 9 m = 10 > 9 m = 12
Face inclination Scoring Breakdown	< 30° or 90° 1	90° - 75° or 30° – 35° 90° – 80° = 2 80° – 75° = 3 30° – 35° = 4	35° – 60° 35° – 40° = 5 40° – 50° = 6 50° – 60° = 7	60° – 75° 60° – 65° = 8 65° – 70° = 9 70° – 75° = 10
Face irregularity Scoring breakdown	Few Clear cut = -1	Occasional Occasional Irregularities = 3	Many Many Irregularities = 8	Major Major launching Features = 11
Rock condition Scoring breakdown	Hard and intact No joints or cracks = -1	Massive,moderately jointed and blocky Few joints and cracks; Firm interlock of blocks between joints = 3	Very blocky, many fractures Imperfect interlock of intact rock fragments; Many fractures = 7	Highly fractured Completely Crushed = 10
Equivalent RQD, %	100 - 90	90 – 50	50 – 25	< 25
Spacing of discontinuity Scoring	Very wide > 0.9 m > 1.2 m = 0 1.2 m – 0.9 m = 1	Wide 0.9 m – 0.2 m 0.9 m – 0.6 m = 2 0.6 m – 0.3 m = 3 0.3 m – 0.2 m = 4	Close 0.2 m – 0.05 m 0.2 m – 0.15 m = 6 0.15 m – 0.10 m = 7 0.10 m – 0.05 m = 8	Very close < 0.05 m < 0.05 m = 9
Block size of falling rocks Scoring breakdown	< 0.05 m < 0.025 m = 0 0.025 m – 0.05 m =1	0.05 m – 0.1 m 0.05 – 0.076 m = 2 0.076 – 0.1 m = 3	0.1m – 0.2 m 0.1 m – 0. 127 m = 4 0.127m – 0.15m = 5 0.15m – 0.17 m = 6 0.17m – 0.20 m = 7	0.2m – 0.3 m 0.20m – 0.23 m = 8 0.23m– 0.25 m = 10 0.25 m – 0.3 m = 12
Volume of rockfall Scoring breakdown	< 4.54 kg < 4.54 kg = 1	4.54 kg – 13.6 kg 4.54 kg – 6.8 kg = 3 6.8 kg – 9.1 kg = 5 9.1 kg – 13.6 kg = 7	13.6 kg – 22.7 kg 13.6 kg–15.88 kg = 9 15.88kg –18.14 kg =10 18.14kg – 22.7 kg = 11	> 22.7 kg > 22.7 kg = 12
Excavation method Scoring breakdown	Control blasting None to few fractures = 1	Mechanical excavation Smooth exca. = – 1 Regular cut; some fractures = 3 Manual cut = 4	Regular blasting Fractures; some irregularities = 5	Poor blasting Highly fractured; Very irregular Rock face = 8
Time factor w/o remedy Scoring breakdown	< 1 day Remedied rock face = 0	1 day – 1 month < 1 day = 1 1 day – 5 days = 2 5 days – 10 days = 3 10 days– 1 month = 4	> 4 years or 1 month - 4 months (> 4 year = 5) 1 month – 2months = 5 2 months- 4 months = 6	> 4 months Maintained rock face = 7; Not maintained rock face = 8
Rockfall frequency Scoring breakdown	No rockfall No rockfal l= 0	Rare rockfall No rockfall in natural condition; rockfalls when disturbed = 3	Occasional rockfall Rockfall in natural condition; Much falls with disturbance = 6	Frequent rockfall Rockfalls without disturbance; high requency = 8 Total Score

- Rockfall frequency

  A falling rock problem is evident when site workers observe rocks falling under natural conditions. The greater the frequency, the more serious the falling rock hazard.

Table 3.11. Rock fall hazard classification (after Singh, 2004)

Rockfall hazard classification				
Class	**I**	**II**	**III**	**IV**
Score range **Fall hazard** **Mitigation measure**	0- 20 Minimal risk Scaling only; No netting	21- 40 Low risk Type I netting	41- 70 Moderate risk Type II netting	71- 100 High risk Type III netting

3.3. Using Rock Mass Classification Systems

The two most widely used rock mass classifications are Bieniawski's RMR (1976, 1989) and the Q system by Barton et al's (1974). Both methods incorporate geological, geometric and design/engineering parameters in arriving at a quantitative value of the rock mass quality. The similarities between RMR and Q stem from the use of identical, or very similar, parameters in calculating the final rock mass quality rating. The differences between the systems lie in the different weightings given to similar parameters and in the use of distinct parameters in one or the other scheme. RMR uses compressive strength directly while Q only considers strength as it relates to in situ stress in competent rock. Both schemes deal with the geology and geometry of the rock mass, but in different ways. Both consider groundwater, and both include some component of rock material strength. Some estimate of orientation can be incorporated into the Q system using a guideline presented by Barton et al. (1974). The main difference between the two systems is the lack of a stress parameter in the RMR system. When using either of these methods, two approaches can be followed. One is to evaluate the rock mass specifically for the parameters included in the classification methods; the other is to accurately characterize the rock mass and then attribute parameter ratings at a later stage (Hoek, 2000). The latter method is recommended since it gives a full and complete description of the rock mass which can easily be translated into either classification index. If rating values alone had been recorded during mapping, it would be almost impossible to carry out verification studies. In many cases, it is appropriate to give a range of values to each parameter in a rock mass classification and to evaluate the significance of the final result. The average value of Q can be used in choosing a basic support system while the range gives an indication of the possible adjustments which will be required to meet different conditions encountered during construction (Hoek, 2000).