Chapter 3
Shear Strength Behaviour of Jointed Rock Masses

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Abstract Rocks encountered in civil and mining engineering structures are generally jointed in nature. The presence of joints renders anisotropy in rock and makes them weaker in their engineering response. Assessment of shear strength response of such jointed rocks, subject to given stress state, is a challenging task. Large size field tests are very expensive and time consuming and hence not feasible for majority projects. The best alternative available is to use indirect methods to describe the shear strength behaviour of jointed rocks.

The present articles presents some of the most widely used techniques developed during last few decades, using which the shear strength response of jointed rock can be assessed with reasonable accuracy. Relatively simple tests and observations are required for applying these techniques and hence input data can be procured without much difficulty. The shear strength response is divided into two broad categories i.e. strength behaviour of joints and strength behaviour of jointed rock mass. Shear strength models described in this article cover linear as well as non-linear strength response. Classification systems are widely used to characterize the rock masses in the field. It has been explained, how, these classification systems could be used to assess the shear strength response of the rock masses.

Keywords Jointed rock · Shear strength · Strength criteria · Classification systems

3.1 General

While analysing structures like tunnels, underground caverns, landslides, road cuts and foundations of heavily loaded structures like dams, bridges situated in or on rocks, the engineers and geologists are often required to assess the shear strength of

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jointed rock masses. The discontinuities e.g. joints, foliations and bedding planes are invariably present in rock masses and induce planes of weakness in the mass. While shearing, failure may occur due to a complex combination of sliding on pre-existing discontinuities, shearing of rock substance, translation and/or rotation of intact rock blocks. Consequently, the jointed rock is quite incompetent and anisotropic in strength and deformational behaviour. In addition, the strength behaviour of jointed rock is non-linear with increase in confining pressure. The assessment of shear strength response is therefore, an extremely difficult task. The present article discusses in brief, how the shear strength of the rocks can be assessed in the field.

From application point of view, two broad categories may occur: In the first category, a single or a set of planar persistent discontinuities exists in the rock. Sliding may occur along the discontinuities depending on the kinematics of the problem. These types of the conditions are commonly encountered in case of slopes. The second category pertains to the failure of the rock mass as a whole. The potential failure surface lies partly along discontinuity surfaces, and partly through the intact rock. Sliding, rotation, translation, splitting or shearing of intact rock blocks occur at the time of failure. The rock mass may behave isotropically or anisotropically depending upon the number, orientation and spacing of discontinuities and level of confining stress. These types of failure are common in tunnels and other underground structures. These two broad categories are discussed in the following sections.

3.2 Discontinuity Shear Strength

In case of shear strength along the surface of discontinuity, the shear strength is represented as a function of normal stress across the failure surface and the shear strength parameters. The most widely used shear strength model is linear Coulomb's model. To account for non-linearity in shear strength response, Patton, Ladanyi-Archambault and Barton's models are commonly used.

3.2.1 Coulomb's Model

Coulomb's shear strength parameters cohesion, c_j and friction angle, ϕ_j are used to estimate the shear strength for given normal stress. The shear strength parameters may be obtained by performing direct shear tests on the discontinuity surfaces. Direct shear tests are conducted for various normal loads and shear stress vs. shear displacement plots are obtained. From these plots, the peak and residual shear strength of the joints are obtained. The failure envelopes of peak and residual

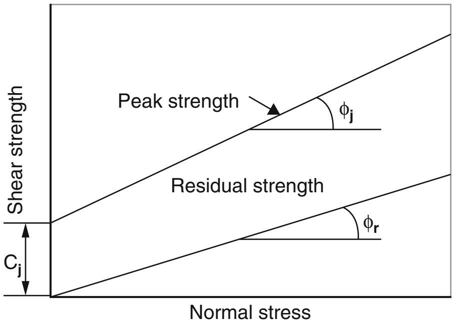

Fig. 3.1 Failure envelopes of shear strength for rough rock joints

shear strength are plotted (Fig. 3.1). The shear strength of the discontinuity is defined as:

$$\tau_f = c_j + \sigma_n \tan \phi_d \quad \text{(Peak strength)} \tag{3.1}$$

$$\tau_f = \sigma_n \tan \phi_r \quad \text{(Residual strength)} \tag{3.2}$$

Where τ_f is the shear strength along the discontinuity; σ_n is the effective normal stress over the discontinuity; ϕ_j is the peak friction angle of the discontinuity surface; c_j is the peak cohesion of the discontinuity surface, and ϕ_r is residual friction angle for discontinuity surface.

3.2.2 *Patton's Bi-Linear Shear Strength Model*

Patton [1] recognised the importance of failure modes and suggested a bi-linear shear strength model. It was postulated that at low normal stress level, sliding occurs along the asperities of the joint surfaces, and at high normal stress, the shearing of the asperities takes place. The model is expressed as:

$$\tau_f = \sigma_n \tan\left(\phi_\mu + \mathrm{i}\right) \quad \text{for low } \sigma_n \quad (\textit{Sliding}) \tag{3.3}$$

$$\tau_f = c_j + \sigma_n \tan(\phi_r) \quad \text{for high } \sigma_n \quad (\textit{Shearing}) \tag{3.4}$$

Where i defines the roughness angle, ϕ_μ is basic friction angle and ϕ_r is the residual friction angle.

In the field, it is very difficult to assess the normal stress level at which transition from sliding to shearing takes place. In reality, there is gradual transition from sliding to shearing and as such, there is no distinct and clear-cut normal stress level, which defines the boundary between the two failure modes.

3.2.3 Ladanyi and Archambault Criterion

Based on the principle of strain energy, Ladanyi and Archambault [2] equated the external work done in shearing a jointed rock to the internal energy stored in the rock and expressed the shear strength of a joint or rock mass as:

$$\tau_f = \frac{\sigma_n(1 - a_s)\left(\dot{v} + \tan\phi_\mu\right) + a_s(c_i + \sigma_n \tan\phi_i)}{1 - (1 - a_s)\,\dot{v}\,\tan\phi_\mu} \tag{3.5}$$

where τ_f is the shear strength; a_s is sheared area ratio equal to A_s/A; A is the total area of joint surface; A_s is the sheared area of joint surface; σ_n is the mean applied normal stress; $\dot{v}$ is the rate of dilation at failure; ϕ_μ is the basic friction angle of joint surface, and c_i, ϕ_i are Mohr-Coulomb parameters for intact rock.

The sheared area ratio, a_s and dilation rate, $\dot{v}$ are estimated as:

$$a_s = 1 - \left(1 - \frac{\sigma_n}{\sigma_{trn}}\right)^{K_1} \tag{3.6}$$

$$\dot{v} \approx \left(1 - \frac{\sigma_n}{\sigma_{trn}}\right)^{K_2} \tan i \tag{3.7}$$

Where K_1 and K_2 are equal to 1.5 and 4 respectively; σ_{trn} is the brittle – ductile transition stress, which may be taken equal to the UCS of intact rock; and i is the initial roughness of the joints.

3.2.4 Barton's JRC-JCS Model

Barton's model, also known as JRC-JCS model is very simple and is the most widely used strength criterion for assessing shear strength along discontinuity surfaces. The shear strength of a joint is expressed as:

$$\tau_f = \sigma_n \tan\left[\phi_r + \mathrm{JRC}\log_{10}\frac{\mathrm{JCS}}{\sigma_n}\right] \tag{3.8}$$

Where JRC is the joint roughness coefficient, which is a measure of the initial roughness (in degrees) of the discontinuity surface. JRC is assigned a value in the range of 0–20, by matching the field joint surface profile with the standard surface profiles on a laboratory scale of 10 cm [3] as shown in Fig. 3.2. JCS is the joint wall compressive strength of the discontinuity surface, and σ_n is the effective normal stress acting across the discontinuity surface.

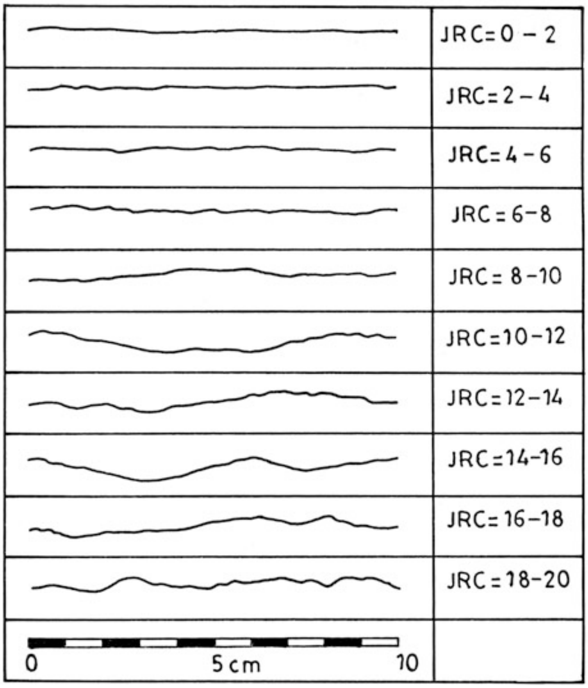

Fig. 3.2 Roughness profiles to estimate JRC [3]

3.3 Shear Strength of Rock Mass

Depending on the scale of structure, geometry of discontinuities and interlocking conditions, the failure may occur due to complex interaction of sliding, rotation, splitting, shearing and translation of rock blocks. In such cases, the jointed rock mass may be replaced with an equivalent continuum for mechanical analysis. Several strength criteria are used to express the strength behaviour of such rock masses. Some of the strength criteria are discussed in the followings in two sub-headings i.e. linear and non-linear strength criteria.

3.3.1 Linear Mohr-Coulomb Criterion

The rock mass is treated as an isotropic continuum and the shear strength along the failure surface is expressed as follows:

$$\tau_f = c_m + \sigma_n \tan \phi_m \tag{3.9}$$

Where c_m and ϕ_m are Mohr-Coulomb shear strength parameters for jointed rock or rock mass. The values of c_m and ϕ_m may be obtained from field shear tests on rock mass. Alternatively, classification approaches provide a rough estimate of the shear strength and some of these approaches are given below.

3.3.1.1 Rock Mass Rating (RMR)

The RMR classification system for rock masses was suggested by Bieniawski [4–6] to characterise the quality of the rock mass. The following parameters of the rock mass are used to classify the mass:

(a) UCS of intact rock material,
(b) Rock Quality Designation (RQD),
(c) Spacing of discontinuities,
(d) Condition of discontinuities, and
(e) Groundwater condition.

Basic RMR is obtained based on above five parameters and then rating is adjusted based and orientation of discontinuities with respect to the structure. The values of shear strength parameters c_m, ϕ_m are presented in Table 3.1 [5].

Mehrotra [7], based on experience from Indian project sites, observed that the shear strength is under-predicted by expressions suggested by Bieniawski [5]. Figure 3.3 may be used for assessing the shear strength parameters of rock masses especially for slopes.

3.3.1.2 Q System

The classification system Q [8, 9] is very popular for characterisation of rock mass. The rock mass quality index Q, is defined as:

$$Q = \left(\frac{RQD}{J_n}\right)\left(\frac{J_r}{J_a}\right)\left(\frac{J_w}{SRF}\right) \tag{3.10}$$

Where, RQD is the rock quality designation [10]; J_n is the joint set number; J_r is the joint roughness number; J_a is the joint alteration number; J_w is the joint water reduction factor, and SRF is stress reduction factor.

The shear strength parameters are obtained as:

$$c_m = \left(\frac{RQD}{J_n}\right)\left(\frac{1}{SRF}\right)\left(\frac{\sigma_{ci}}{100}\right) \quad \text{MPa} \tag{3.11}$$

$$\phi_m = \tan^{-1}\left(\frac{J_r}{J_a}J_w\right) \tag{3.12}$$

where c_m is the cohesion of the undisturbed rock mass; ϕ_m, the friction angle of the mass; and σ_{ci} is the uniaxial compressive strength of intact rock material.

Table 3.1 Mohr-Coulomb parameters from RMR [5]

Class number	I	II	III	IV	V
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

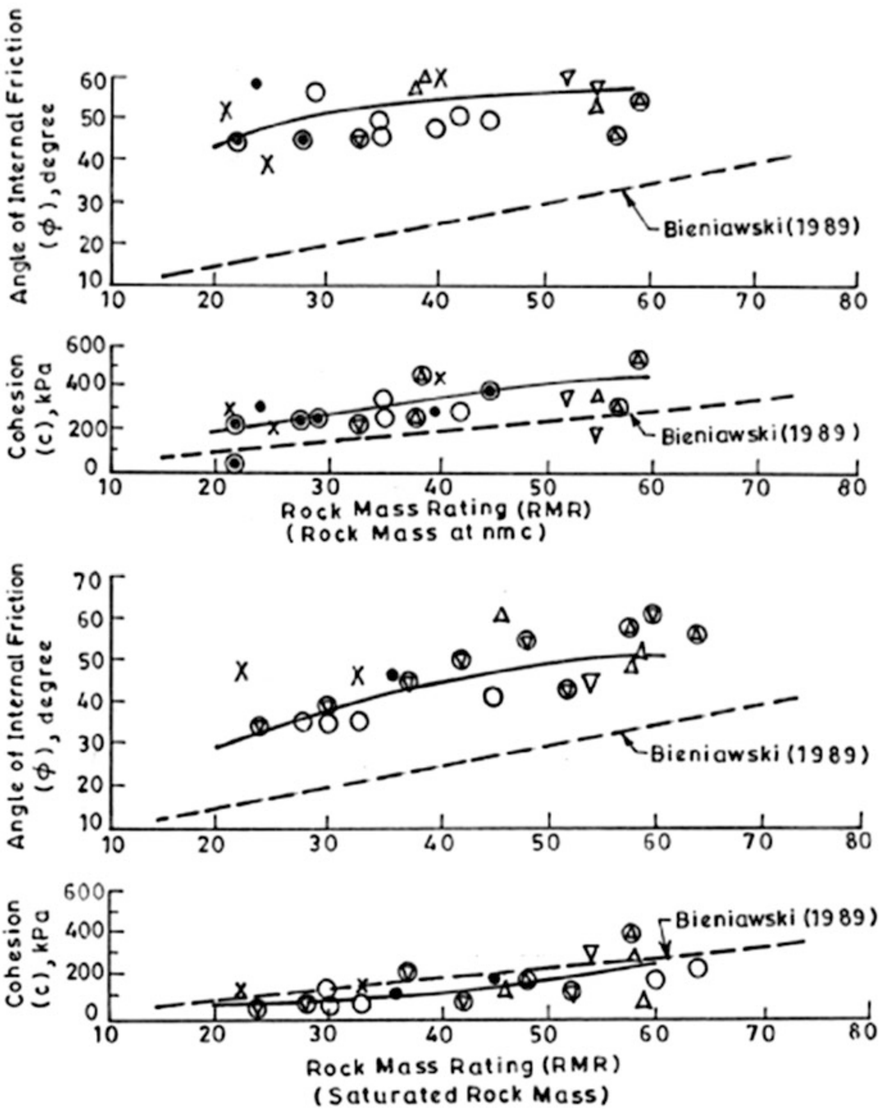

Fig. 3.3 Estimation of friction angle of rock mass from RMR [7]

If used for slopes, an overestimation in the strength may be expected. For slopes, this author personally feels that the relationship between shear strength parameters and RMR as suggested by Mehrotra [7] will be more appropriate for the Himalayan rock masses. The relationships were developed based on extensive in-situ direct shear tests on saturated rock masses in Himalayas.

3.3.2 Non-linear Strength Criteria

The Mohr-Coulomb strength criterion considers the rock mass shear strength as a linear function of normal stress σ_n. In reality, the shear strength response is highly non-linear and the parameters c_m, ϕ_m change with the range of confining pressure used in estimating these parameters. To resolve this issue, several non-linear strength criteria have been proposed for jointed rocks and rock masses. Some of them are presented in the following section.

3.3.2.1 Empirical Criteria Based on RMR and Q

Mehrotra [7] utilised results of large number of in-situ direct shear tests on rock masses in the Himalayas, and suggested the non-linear variation of shear strength as:

$$\frac{\tau_f}{\sigma_{ci}} = A\left(\frac{\sigma_n}{\sigma_{ci}} + B\right)^C \tag{3.13}$$

where A, B and C are empirical constants and depend on RMR or Q. Their values for different moisture contents, RMR and Q index are presented in Table 3.2.

3.3.2.2 Hoek-Brown Strength Criterion

Hoek-Brown [11] proposed a non-linear strength criterion for intact rocks as follows:

$$\sigma_1 = \sigma_3 + \sqrt{m_i \sigma_{ci} \sigma_3 + \sigma_{ci}^2} \tag{3.14}$$

where σ_1 is the effective major principal stress at failure; σ_3 is the effective minor principal stress at failure; m_i is a criterion parameter; and σ_{ci} is the UCS of the intact rock, which is also treated as a criterion parameter. The parameters m and σ_{ci} are obtained by fitting the criterion into the laboratory triaxial test data. If triaxial test data is not available the approximate values of parameters m_i can also be obtained from Table 3.3 [12].

The criterion was extended to heavily jointed isotropic rock masses [11]. The latest form of the criterion [13] is expressed as:

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_j \frac{\sigma_3}{\sigma_{ci}} + s_j\right)^a \tag{3.15}$$

Where m_j is an empirical constant, which depends upon the rock type; and s_j is an empirical constant, which varies between zero (for crushed rock) to one (for intact

Table 3.2 Shear strength parameters for jointed rock masses [7]

Rock type quality	Limestone	Slate, xenolith, phyllite	Sandstone, quartzite	Trap, metabasic
Good rock mass	NMC	NMC	NMC	NMC (S_{av} = 0.30)
RMR = 61–80	A = 0.38, B = 0.005, C = 0.669	A = 0.42, B = 0.004, C = 0.683	A = 0.44, B = 0.003, C = 0.695	A = 0.50, B = 0.003, C = 0.698
Q > 10	Saturated (S = 1)	Saturated (S = 1)	Saturated (S = 1)	Saturated (S = 1)
	A = 0.35, B = 0.004, C = 0.669	A = 0.38, B = 0.003, C = 0.683	A = 0.43, B = 0.002, C = 0.695	A = 0.49, B = 0.002, C = 0.698
Fair rock mass	NMC	NMC	NMC (S_{av} = 0.15)	NMC (S_{av} = 0.35)
RMR = 41–60	A = 2.60, B = 1.25 , C = 0.662	A = 2.75, B = 1.15 , C = 0.675	A = 2.85, B = 1.10 , C = 0.685	A = 3.05, B = 1.00 , C = 0.691
Q = 2–10	Saturated (S = 1)	Saturated (S = 1)	Saturated (S = 1)	Saturated (S=1)
	A = 1.95, B = 1.20, C = 0.662	A = 2.15, B = 1.10, C = 0.675	A = 2.25, B = 1.05, C = 0.688	A = 2.45, B = 0.95, C = 0.691
Poor rock mass	NMC (S_{av} = 0.25)	NMC (S_{av} = 0.40)	NMC (S_{av} = 0.25)	NMC (S_{av} = 0.15)
RMR = 21–40	A = 2.50, B = 0.80, C = 0.646	A = 2.65, B = 0.75, C = 0.655	A = 2.85, B = 0.70, C = 0.672	A = 3.00, B = 0.65, C = 0.676
Q = 0.5 – 2	Saturated (S = 1)	Saturated (S = 1)	Saturated (S=1)	Saturated (S=1)
	A = 1.50, B = 0.75, C = 0.646	A = 1.75, B = 0.70, C = 0.655	A = 2.00, B = 0.65, C = 0.672	A = 2.25, B = 0.50, C = 0.676
Very poor rock mass	NMC	NMC	NMC	NMC
RMR < 21	A = 2.25; B = 0.65, C = 0.534	A = 2.45; B = 0.60, C = 0.539	A = 2.65; B = 0.55, C = 0.546	A = 2.90; B = 0.50, C = 0.548
Q < 0.5	Saturated (S = 1)	Saturated (S = 1)	Saturated (S = 1)	Saturated (S = 1)
	A = 0.80, B = 0.0, C = 0.534	A = 0.95, B = 0.0, C = 0.539	A = 1.05, B = 0.0, C = 0.546	A = 1.25, B = 0.0, C = 0.548

S Degree of saturation, *NMC* Natural moisture content, S_{av} Average value of degree of saturation

rock) depending upon the degree of fracturing. The expressions for m_j and s_j are given as:

$$m_j = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) \tag{3.16}$$

$$s_j = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \tag{3.17}$$

Table 3.3 Approximate estimation of parameter m_i [12]

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
Sedimentary	Clastic		Conglomerate	Sandstone	Siltstone	Claystone
			(22)	19	9	4
				Greywacke		
				(18)		
	Non-clastic	Organic		Chalk		
				7		
				Coal		
				(8–21)		
		Carbonate	Breccia	Sparitic	Micritic	
			(20)	Limestone	Limestone	
				(10)	8	
		Chemical		Gypstone	Anhydrite	
				16	13	
Metamorphic	Non Foliated		Marble	Hornfels	Quartzite	
			9	(19)	(24)	
	Slightly Foliated		Migmatite	Amphibolite	Mylonites	
			(30)	(25–31)	(6)	
	Folaited[a]		Gneiss	Schists	Phyllites	Slate
			33	4–8	(10)	9
Igneous	Light		Granite		Rhyolyte	Obsidian
			33		(16)	(19)
			Granodiorite		Dacite	
			(30)		(17)	
	Dark		Diorite		Andesite	
			(28)		19	
			Gabbro	Dolerite	Basalt	
			27	(19)	(17)	
			Norite			
			22			
	Extrusive		Agglomerate	Breccia	Tuff	
	Pyroclastic type		(20)	(18)	(15)	

Note: Values in parenthesis are estimates

[a]These values are for intact rock specimens tested normal to bedding or foliation. The value will be significantly different if failure occurs along a weakness plane

where D is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from zero for undisturbed in situ rock masses to one for very disturbed rock masses. For blasted rock slopes, D is taken in the range 0.7–1.0.

GSI is the Geological Strength Index [13, 14] which depends on the structure of mass and surface characteristics of the discontinuities (Fig. 3.4).

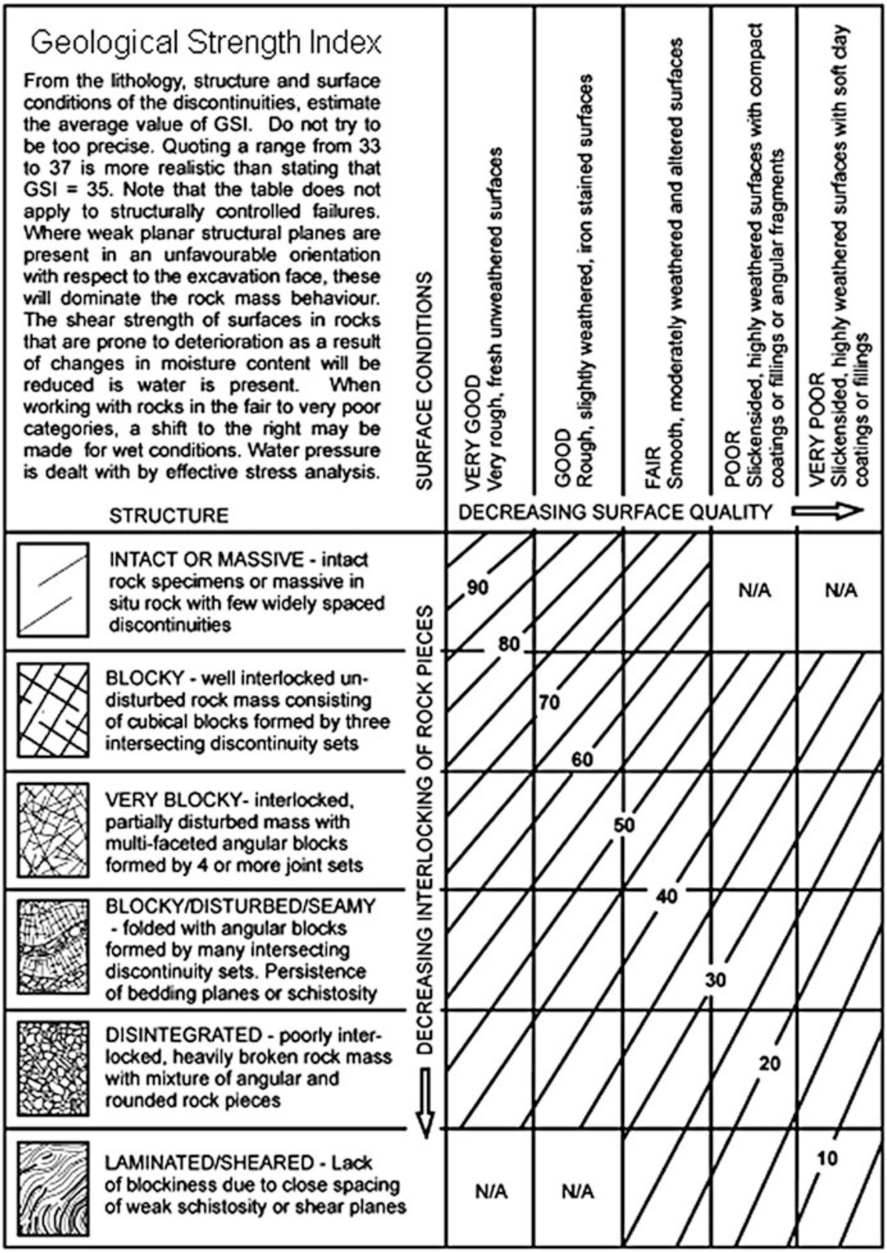

Fig. 3.4 Estimation of geological strength index [15]

The index a is obtained as:

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \quad (3.18)$$

The limitation of the GSI approach is that the GSI is estimated only from geological features and disturbance to the mass, and no measurements e.g. joint mapping are done in the field.

3.3.2.3 Ramamurthy Criterion

Based on extensive triaxial tests conducted on rocks and model materials, Ramamurthy and co-workers [16–19] suggested the following non-linear strength criterion for intact isotropic rocks:

$$\left(\frac{\sigma_1 - \sigma_3}{\sigma_3 + \sigma_t}\right) = B_i\left(\frac{\sigma_{ci}}{\sigma_3 + \sigma_t}\right)^{\alpha_i} \quad (3.19)$$

where σ_3 and σ_1 are the minor and major principal stresses at failure; σ_t is the tensile strength of intact rock; σ_{ci} is the UCS of the intact rock; and α_i, B_i are the criterion parameters.

Parameters α_i and B_i are criterion parameters and are suggested to be obtained by fitting the criterion into the laboratory triaxial test data for intact rock. Alternatively, the following approximate correlations may be used:

$$\alpha_i = 2/3; \text{ and } B_i = 1.1\left(\frac{\sigma_{ci}}{\sigma_t}\right)^{1/3} \text{ to } 1.3\left(\frac{\sigma_{ci}}{\sigma_t}\right)^{1/3} \quad (3.20)$$

The criterion was extended to jointed rocks and rock masses as:

$$\left(\frac{\sigma_1 - \sigma_3}{\sigma_3}\right) = B_j\left(\frac{\sigma_{cj}}{\sigma_3}\right)^{\alpha_j} \quad (3.21)$$

where α_j and B_j are the criterion parameters for jointed rock; and σ_{cj} is the UCS of the jointed rock.

The criterion parameters α_j and B_j are suggested to be obtained from the following correlations:

$$\frac{\alpha_j}{\alpha_i} = \left(\frac{\sigma_{cj}}{\sigma_{ci}}\right)^{0.5} \quad (3.22)$$

$$\frac{B_i}{B_j} = 0.13\exp\left[2.037\left(\frac{\sigma_{cj}}{\sigma_{ci}}\right)^{0.5}\right] \quad (3.23)$$

Where σ_{cj} is the UCS of the rock mass.

3.3.2.4 Modified Mohr Coulomb Criterion

The Mohr-Coulomb criterion, though most widely used criterion, has the limitation in that the non-linear strength response of rocks is not captured by this criterion. Singh and Singh [20] used critical state concept for rocks [21] and suggested the Modified Mohr Coulomb (MMC) criterion to incorporate non-linearity in strength behaviour. The advantage of MMC is that the parameters c_m and ϕ_m are retained as such. The criterion is expressed as:

$$(\sigma_1 - \sigma_3) = \sigma_{cj} + \frac{2\ \sin\ \phi_{m0}}{1\text{-}\sin\ \phi_{m0}}\sigma_3 - \frac{1}{\sigma_{ci}}\frac{\sin\ \phi_{m0}}{(1\text{-}\sin\phi_{m0})}\sigma_3^2 \text{ for } 0 \le \sigma_3 \le \sigma_{ci} \tag{3.24}$$

Where σ_{ci} and σ_{cj} are the UCS of the intact rock and rock mass respectively; ϕ_{m0} is the friction angle of the rock mass corresponding to very low confining pressure range ($\sigma_3 \rightarrow 0$) and can be obtained as:

$$\sin\phi_{m0} = \frac{(1 - \text{SRF}) + \frac{\sin\phi_{i0}}{1\text{-}\sin\phi_{i0}}}{(2\text{-SRF}) + \frac{\sin\phi_{i0}}{1\text{-}\sin\phi_{i0}}} \tag{3.25}$$

Where SRF = Strength Reduction Factor = σ_{cj}/σ_{ci}; ϕ_{i0} is friction angle for the intact rock and is obtained by conducting triaxial strength tests on intact rock specimens at low confining pressures ($\sigma_3 \rightarrow 0$).

If triaxial test data on intact rock is not available, the following non-linear form of the criterion may be used [22, 23]:

$$\sigma_1 = A(\sigma_3)^2 + (1 - 2A\sigma_{ci})\sigma_3 + \sigma_{cj}; \qquad \sigma_3 \le \sigma_{ci} \tag{3.26}$$

Where A is criterion parameter and may be estimated from the experimental value of σ_{ci}, using the following expressions:

$$\text{For average } \sigma_1 \qquad A = -1.23(\sigma_{ci})^{-0.77} \tag{3.27}$$

$$\text{For lower bound } \sigma_1 \quad A = -0.43(\sigma_{ci})^{-0.72} \tag{3.28}$$

For design purposes, the lower bound values of σ_1 are recommended to be used.

3.3.3 Rock Mass Strength (σ_{cj})

An important input to the strength criteria for rock masses is the UCS of rock mass σ_{cj}. The following approaches may be used to determine the UCS of the rock mass:

(i) Joint Factor concept, J_f
(ii) Rock quality designations, RQD
(iii) Rock mass quality, Q
(iv) Rock mass rating, RMR
(v) Modulus ratio concept (Strength reduction factor)

3.3.3.1 Joint Factor Concept

Ramamurthy and co-workers have defined a weakness coefficient that characterises the effect of fracturing on rocks and termed it Joint Factor [16, 18, 24–26]. The Joint Factor considers the combined effect of frequency, orientation, shear strength of joints, and is defined as:

$$J_f = \frac{J_n}{n\,r} \tag{3.29}$$

where, J_n = joint frequency, i.e., number of joints/metre; n is inclination parameter, depends on the inclination of sliding plane with respect to the major principal stress direction (Table 3.4); r is a parameter for joint strength; it is obtained from direct shear tests conducted along the joint surface at low normal stress levels and is given by:

$$r = \frac{\tau_j}{\sigma_{nj}} = \tan \phi_j \tag{3.30}$$

Where τ_j is the shear strength along the joint; σ_{nj} is the normal stress across the joint surface; and ϕ_j is the equivalent value of friction angle incorporating the effect of asperities [27]. The tests should be conducted at very low normal stress levels so that the initial roughness is reflected through this parameter. For cemented joints, the value of ϕ_j includes the effect of cohesion intercept also. In case the direct shear tests are not possible and the joint is tight, a rough estimate of ϕ_j may be obtained from Table 3.5 [27].

Table 3.4 Joint inclination parameter n [16]

Orientation of joint θ°	Inclination parameter n	Orientation of joint θ°	Inclination parameter n
0	1.00	50	0.071
10	0.814	60	0.046
20	0.634	70	0.105
30	0.465	80	0.460
40	0.306	90	0.810

θ = Angle between the normal to the joint plane and major principal stress direction

Table 3.5 Values of joint strength parameter, r for different values of σ_{ci} (After Ramamurthy [16, 27])

Uniaxial compressive strength of intact rock, σ_{ci} (MPa)	Joint strength parameter, r	Remarks
2.5	0.30	Fine grained micaceous to coarse grained
5.0	0.45	
15.0	0.60	
25.0	0.70	
45.0	0.80	
65.0	0.90	
100.0	1.0	

Table 3.6 Joint strength parameter, r for filled-up joints at residual stage (After Ramamurthy [16, 27])

Gouge material	Friction angle (ϕ_j)	$r = \tan \phi_j$
Gravelly sand	45°	1.00
Coarse sand	40°	0.84
Fine sand	35°	0.70
Silty sand	32°	0.62
Clayey sand	30°	0.58
Clay silt		
Clay – 25%	25°	0.47
Clay – 50%	15°	0.27
Clay – 75%	10°	0.18

Table 3.7 Empirical constant 'a' for estimating σ_{cj}

Failure mode	Coefficient a
Splitting/shearing	−0.0123
Sliding	−0.0180
Rotation	−0.0250

If the joints are filled with gouge material and have reached the residual shear strength, the value of r may be assigned from Table 3.6 [27].

The UCS of the rock mass is obtained as:

$$\sigma_{cj} = \sigma_{ci} \exp(a\, J_f) \tag{3.31}$$

where a is an empirical coefficient equal to −0.008.

Singh [25] and Singh et al. [26] suggested that the failure of the rock mass under uniaxial stress condition may occur due to various failure modes i.e. splitting, shearing, sliding and rotation. The values for different modes of failure are presented in Table 3.7. The failure mode may be decided as per guideline given below [25, 26]. If it is not possible to assess the failure mode, an average value of the empirical constant, 'a' may be taken equal to −0.017.

Let θ be the angle between the normal to the joint plane and the major principal stress direction:

(i) For $\theta = 0$–$10°$, the failure is likely to occur due to *splitting* of the intact material of blocks.
(ii) For $\theta = 10°$ to $\approx 0.8\ \phi_j$, the mode of failure shifts from splitting (at $\theta = 10°$) to sliding (at $\theta \approx 0.8\ \phi_j$).
(iii) For $\theta = 0.8\phi_j$ to 65°, the mode of failure is expected to be sliding only.
(iv) For $\theta = 65$–$75°$, the mode of failure shifts from sliding (at $\theta = 65°$) to rotation of blocks (at $\theta = 75°$).
(v) For $\theta = 75$–$85°$, the mass fails due to rotation of blocks only.
(vi) For $\theta = 85$–$90°$, the failure mode shifts from *rotation* at $\theta = 85°$ to *shearing* at $\theta = 90°$.

3.3.3.2 Rock Quality Designation, RQD

Zhang [28] has proposed the following correlation for obtaining UCS of rock mass as a function of RQD. It may be noted that joint attributes like frequency and surface roughness have not been given any consideration in this approach.

$$\frac{\sigma_{cj}}{\sigma_{ci}} = 10^{(0.013\,RQD-1.34)} \tag{3.32}$$

3.3.3.3 Rock Mass Quality, Q

Based on extensive database, Singh et al. [29] have proposed correlations of rock mass strength, σ_{cj} with Q by analysing block shear tests in the field.

$$\sigma_{cj} = 0.38\gamma Q^{1/3}\text{MPa} \quad \text{for slopes} \tag{3.33}$$

$$\sigma_{cj} = 7\gamma Q^{1/3}\,\text{MPa} \quad \text{for tunnels} \tag{3.34}$$

Barton [9] modified the above equation for tunnels and suggested the expression:

$$\sigma_{cj} = 5\gamma\left(\frac{Q\,\sigma_{ci}}{100}\right)^{1/3}\text{MPa} \quad \text{for tunnels} \tag{3.35}$$

Where γ is the unit weight of rock mass in gm/cm^3; and Q is the Barton's rock mass quality index.

3.3.3.4 Rock Mass Rating, RMR

Rock Mass Rating (RMR) may be used to get the shear strength parameters c_m and ϕ_m from RMR [4–6] and the rock mass strength σ_{cj} may be obtained as:

$$\sigma_{cj} = \frac{2c_m\ \cos\phi_m}{1\text{-}\sin\phi_m} \tag{3.36}$$

Ramamurthy [19] has suggested that the shear strength parameters recommended by Bieniawski [4–6] appear to be on lower side resulting in very low values of σ_{cj}.

The other commonly used correlations with RMR are as follows:

(i) Kalamaras and Bieniawski [30]

$$\frac{\sigma_{cj}}{\sigma_{ci}} = \exp\left(\frac{\text{RMR} - 100}{24}\right) \tag{3.37}$$

(ii) Sheorey [31]

$$\frac{\sigma_{cj}}{\sigma_{ci}} = \exp\left(\frac{\mathrm{RMR} - 100}{20}\right) \tag{3.38}$$

3.3.3.5 Strength Reduction Factor

In the opinion of this author the best estimates of rock mass strength, σ_{cj} can only be made in the field through large size field-testing in which the mass may be loaded upto failure to determine rock mass strength. It is, however, rarely feasible. An alternative will be to get the deformability properties of rock mass by stressing a limited area of the mass upto a certain stress level, and then relate the ultimate strength of the mass to the laboratory UCS of the rock material through a strength reduction factor, SRF. Singh and Rao [32] have shown that the Modulus Reduction Factor, MRF and Strength Reduction Factor, SRF are correlated with each other by the following expression approximately:

$$\mathrm{SRF} = (\mathrm{MRF})^{0.63} \tag{3.39}$$

$$\Rightarrow \frac{\sigma_{cj}}{\sigma_{ci}} = \left(\frac{E_j}{E_i}\right)^{0.63} \tag{3.40}$$

Where SRF is the ratio of rock mass strength to the intact rock strength; MRF is the ratio of rock mass modulus to the intact rock modulus; σ_{cj} is the rock mass strength; σ_{ci} is the intact rock strength; E_j is the elastic modulus of rock mass; and E_i is the intact rock modulus available from laboratory tests and taken equal to the tangent modulus at stress level equal to 50% of the intact rock strength.

It is recommended that field deformability tests should invariably be conducted on project sites. The elastic modulus of rock mass, E_j may be obtained in the field by conducting uniaxial jacking tests [33] in drift excavated for the purpose. The test consists of stressing two parallel flat rock faces (usually the roof and invert) of a drift by means of a hydraulic jack [7]. The stress is generally applied in two or more cycles as shown in Fig. 3.5. The second cycle of the stress deformation curve is used for computing the field modulus as:

$$E_j = \frac{m(1 - \upsilon^2)P}{\sqrt{A}\ \delta_e} \tag{3.41}$$

where E_j is the elastic modulus of the rock mass in kg/cm^2; υ is the Poisson's ratio of the rock mass (= 0.3); P is the load in kg; δ_e is the elastic settlement in cm; A is the area of plate in cm^2; and m is an empirical constant (=0.96 for circular plate of 25 mm thickness).

The size of the drift should be sufficiently large as compared to the plate size so that there is little effect of confinement. The confinement may result in over prediction of the modulus values.

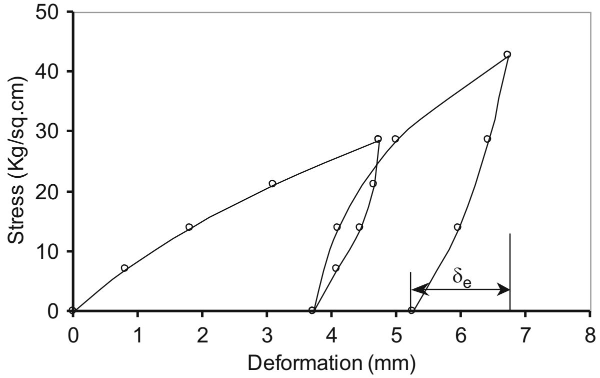

Fig. 3.5 Field modulus of elasticity [33]

A number of methods for assessing the rock mass strength, σ_{cj} have been discussed above. It is desirable that more than one method be used for assessing the rock mass strength and generating the failure envelopes. A range of values will thus be obtained and design values may be taken according to experience and confidence of the designer.

3.4 Concluding Remarks

Assessment of shear strength behaviour of jointed rocks and rock masses is a difficult task. At the time of failure, the strength may be mobilised along a dominating persistent discontinuity or through blocks of the rock mass. Accordingly, discontinuity shear strength or rock mass strength will govern the design. Some of the approaches available for obtaining the shear strength of an individual discontinuity or of a mass as a whole have been discussed in the present article. The strength behaviour is known to vary non-linearly with confinement and hence special emphasis has been given to non-linear strength criteria. In real life problems, heterogeneity and uncertainty are very common. It may be expected that shear strength obtained from different approaches will vary over a range. It is advisable parametric analysis be done to examine the behaviour of the structure for the range of values to gain more confidence in the design.

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