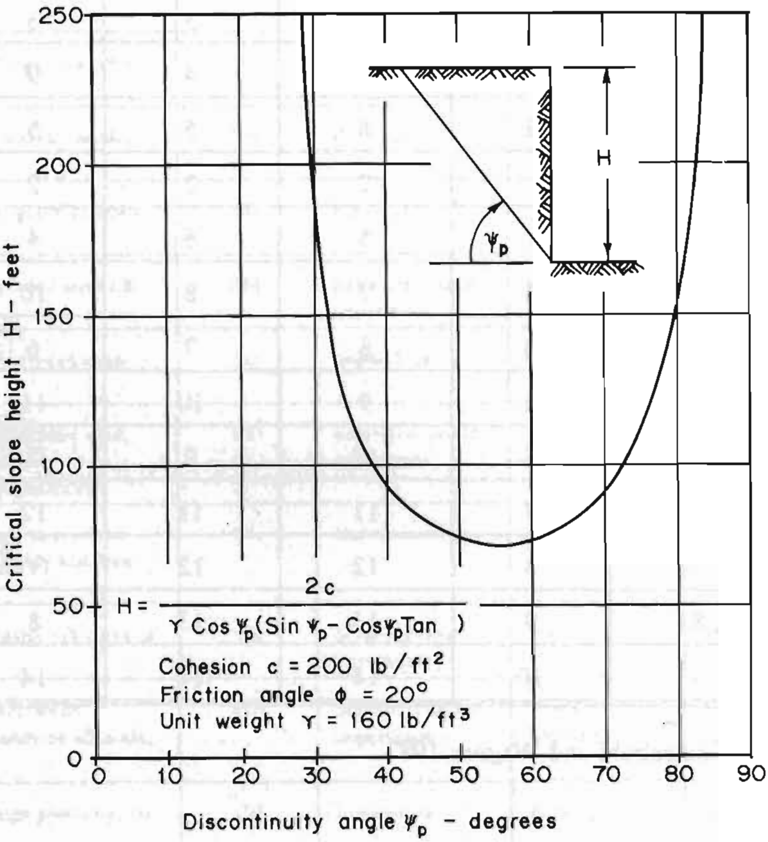
Chapter 10*

ROCK MECHANICS

10.1 SHEAR STRENGTH OF ROCKS

The planning and design of structures in mountainous areas require understanding of the geological and mechanical behavior of rock masses.



Source: United States Department of Transportation (USDT) 1981

Fig. 10.1 Critical height of a drained vertical slope containing a planar discontinuity dipping at an angle ψ_p

---

Unless otherwise stated all Figures and Tables in this chapter are based on "Rock Slopes" published by the United States Department of Transportation (USDT) Federal Highway Agency (FHA) in 1981.

Analysis of rock slope stability has been approached by a number of investigators based on the assumption that rock mass behaves as an elastic continuum. The most analysis of practical rock slope problems is currently based on a **discontinuum approach**. The discontinuum approach emphasizes that the behavior of rock mass is dominated by discontinuities such as faults, joints, and bedding planes.

The stability analysis of rock slopes requires an understanding of the discontinuities, effect of the discontinuities on a failure plane, shear strength properties of rock masses, and the mechanics of stability. Rock mechanics is a subject that includes all these areas concerned with the engineering of structures in a rocky terrain.

The shear strength of rocks along a single discontinuity surface is influenced by friction angle and roughness of discontinuity, uniaxial compressive strength of joint surface, and type of infilling and water pressure in the joint. Figure 10.1 illustrates the influence of a discontinuity on critical slope height.

Shear strength of rock mass with a number of closely spaced joint sets is influenced by confining pressure, uniaxial compressive strength, water pressure, and constants defining the Mohr Failure Envelope.

10.1.1 *Peak and Residual Shear Strength*

Peak Shear Strength

This is the maximum shear strength or shear stress at yield point given by a curve obtained by plotting shear displacement against shear stress at constant normal stress (Fig. 10.2a).

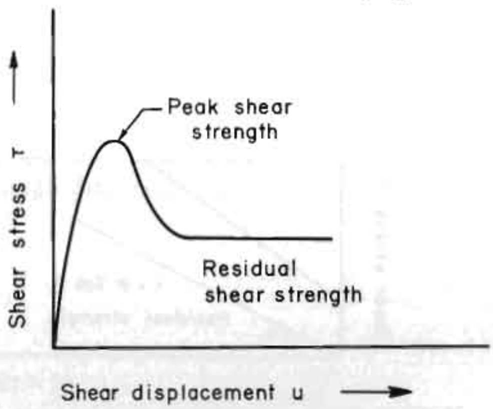


Fig. 10.2 (a) Peak shear strength

Peak Friction Angle

This is the friction angle given by the slope of a straight line representing the relationship between normal stress and peak shear strength from shear strength tests carried out at varying normal stresses (Fig. 10.2b).

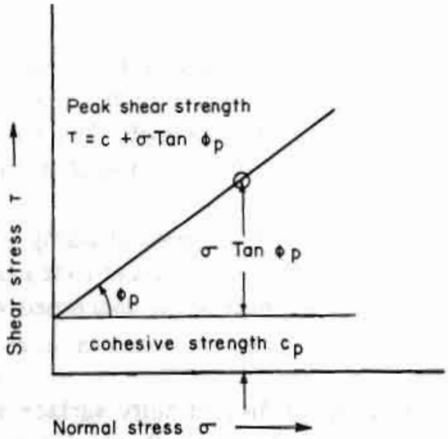


Fig 10.2(b) The relationship between normal stress and shear stresses

Residual Shear Strength

This is the shear stress that levels out at a constant value with increasing shear displacement in a shear test at constant normal stress (Fig. 10.2a)

Residual Friction Angle

This is the friction angle represented by the slope of a straight line obtained by a plot of normal stress against residual shear strength from shear tests at different normal stresses (Fig. 10.2c).

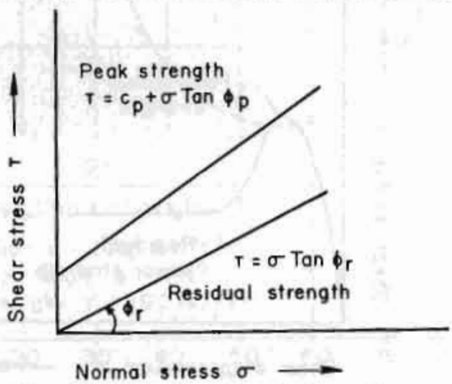


Fig. 10.2(c) Residual strength

10.1.2 ***Shear Strength of Rocks with Single Discontinuity - Plane Surface***

The shear strength of rocks with even bedding planes having no surface undulation or roughness, can be expressed in a simple linear relationship. There are two cases, and these are given below.

In dry conditions, the peak shear strength $\tau = C_p + \sigma \tan\phi_p$,

and the residual shear strength $\tau = \sigma \tan \phi_\gamma$.

In conditions of water-filled discontinuity, peak shear strength $\tau = c_p + (\sigma - u) \tan \phi_p$,

and residual shear strength $\tau = (\sigma - u) \tan \phi_\gamma$

where,

u = water pressure in the discontinuity.

10.1.3 *Shear Strength of Single Discontinuity*

Barton's Equation

Barton (Barton and Chaubey 1977) proposed the following equations for predicting the shear strength of rough joints

$$\tau = \sigma \tan (\phi_\gamma + JRC \log_{10} [\frac{JCS}{\sigma}])$$

$$\phi = \phi_\gamma + i$$

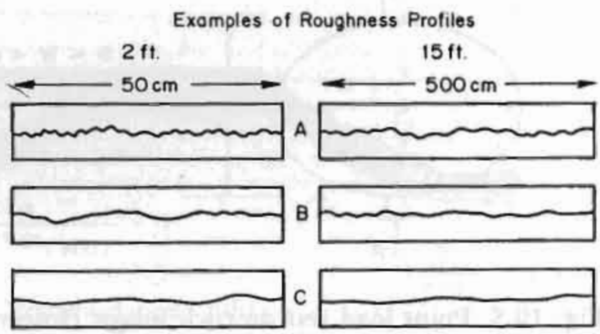
where, JRC = joint roughness coefficient,
JCS = joint-wall compressive strength,
ϕ_γ = basic friction angle of a smooth diamond saw-cut surface,
σ = normal stress across a joint surface,
i = angle of dilation or primary as parities, and
ϕ = friction angle of rough joints.

Barton's equation is for low values of normal stress and is probably most applicable in the range of $0.01 < \sigma/\text{JCS} < 0.3$. Figures 10.3 and 10.4 illustrate Barton's definition of joint roughness, JRC, and the prediction of shear strength of rough discontinuities.

A. Rough undulating - tension joints, rough sheeting, rough bedding. JRC = 20

B. Smooth undulating - smooth sheeting, non-planar foliation, undulating JRC = 10

C. Smooth nearly planar - planar shear joints, planar foliation, planar bedding JRC = 5



Source: Barton and Chaubey 1977

Fig. 10.3 Barton's definition of joint roughness coefficient (JRC)

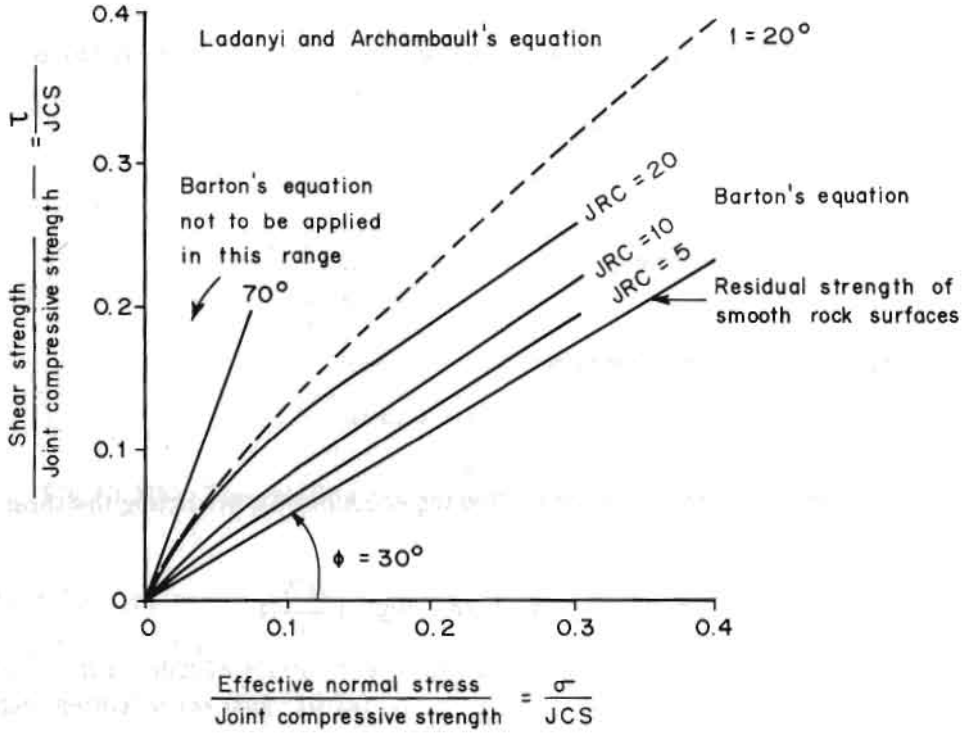


Fig. 10.4 Barton's prediction for the shear strength of rough discontinuities

The uniaxial compressive strength of the joint wall material can be obtained in a simpler manner by point load testing of a lump specimen by the following relationship (Fig. 10.5):

$$\sigma_c = 15\ I_L$$

where,

σ_c = uniaxial compressive strength, and
I_L = point load lump strength index.

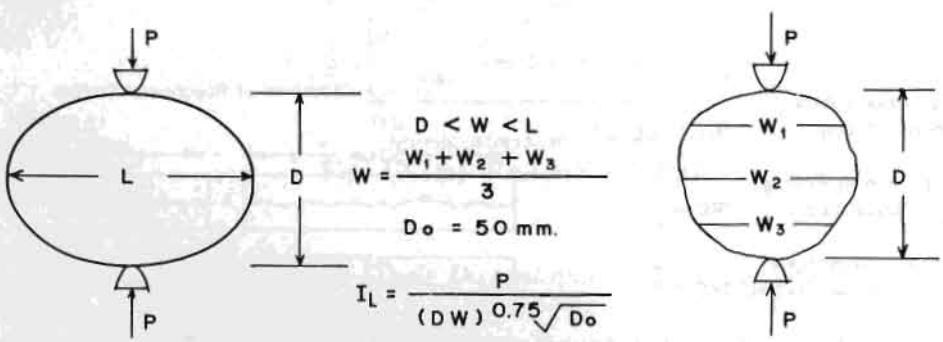


Fig. 10.5 Point load test on rock lumps (International Society for Rock Mechanics [ISRM])

Note that Joint Compressive Strength (JCS) is the compressive strength of the rock material adjacent to the joint surface and may be lower than σ_c as a result of weathering of the surface.

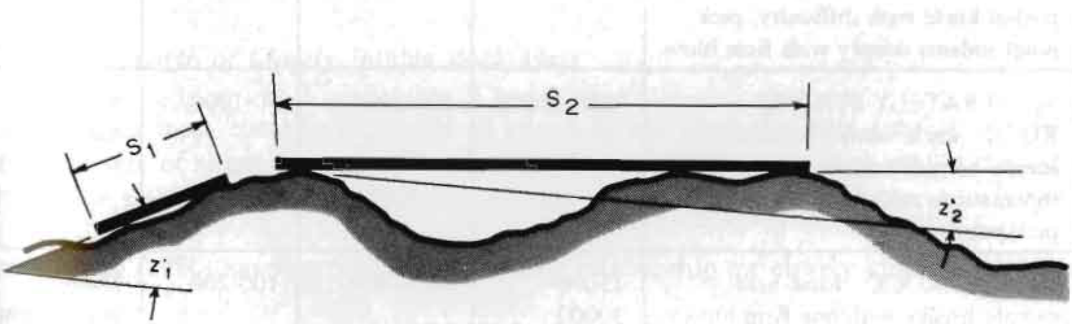
Table 10.1 gives the approximate basic friction angle for different rocks. Table 10.2 gives the approximate values of uniaxial compressive strength for cohesive soils and rocks. Surface roughness *i* can be measured as shown in Figure 10.6.

Table 10.1 Approximate values for the basic friction angle for different rocks

Rock	Degrees
Amphibolite	32
Basalt	31-38
Conglomerate	35
Chalk	30
Dolomite	27-31
Gneiss (schistose)	23-29
Granite (fine grain)	29-35
Granite (coarse grain)	31-35
Limestone	33-40
Porphyry	31
Sandstone	25-35
Shale	27
Siltstone	27-31
Slate	25-30

Source: Barton and Chaubey 1977

Lower value is generally given by tests on wet rock surfaces



Source: adapted from Rock Slopes, U.S. Dept. of Transportation, 1981

Fig. 10.6 Measurement of surface roughness with different lengths

Short base length give high values for the effective roughness angle, while long bases give smaller angles.

Table 10.2 Approximate classification of cohesive soil and rock

No.	Description	Uniaxial	Compressive strength		Examples
		lb/in²	*kg/cm²*	*MP^a^*	
S1	VERY SOFT SOIL - easily moulded with fingers, shows distinct heel marks	< 5	< 0.4	< 0.04	
S2	SOFT SOIL - moulds with strong pressure from fingers, shows faint heel marks	5-10	0.4-0.8	0.04-0.08	
S3	FIRM SOIL - very difficult to mould with fingers, indented with finger nail, difficult to cut with hand spade	10-20	0.8-1.5	0.08-0.15	
S4	STIFF SOIL - cannot be moulded with fingers, cannot be cut with hand spade, requires hand picking for excavation	20-80	1.5-6.0	0.15-0.60	
S5	VERY STIFF SOIL - very tough, difficult to move with hand pick, requires a pneumatic spade for excavation	80-150	6-10	0.6-1.0	
R1	VERY WEAK ROCK - crumbles under sharp blows with geological pick point, can be cut with pocket knife	150-3500	10-250	1-25	Chalk, rocksalt
R2	MODERATELY WEAK ROCK - shallow cuts or scraping with pocket knife with difficulty, pick point indents deeply with firm blow	3500-7500	250-500	25-50	Coal, schist, siltstone
R3	MODERATELY STRONG ROCK - knife cannot be used to scrape or peel surface, shallow indentations under firm blow from pick point	7500-15000	500-1000	50-1000	Sandstone, slate, shale
R4	STRONG ROCK - hand-held sample breaks with one firm blow from the hammer head of a geological pick	15000-30000	1000-2000	100-200	Marble, granite, gneiss
R5	VERY STRONG ROCK - requires many blows from geological pick to break intact sample	> 30000	> 2000	>200	Quartzite, dolerite, gabbro, basalt

10.1.4 *Shear Strength of Filled Discontinuities*

Often there are no rock-to-rock contacts in discontinuities. They can be filled with detrital material or gouge from previous shear movements, or material deposited by the movement of water through the rock mass.

Shear strength decreases with the increase in thickness of infilling; once the thickness exceeds the amplitude of surface projections, the shear strength of the joint is controlled only by the strength of the filling material (Fig. 10.7).

Filled joints influence the permeability of the rock mass. The permeability of clay gouge and similar joint filling material may be three or four orders of magnitude lower than that of the surrounding rock mass, and this can give rise to the damming of groundwater into compartments within the rock mass. The building of water pressure and also the very low shear strength of filling materials drastically weakens the stability of slopes. Table 10.3. gives shear strength of filled discontinuities from tests carried out by various persons.

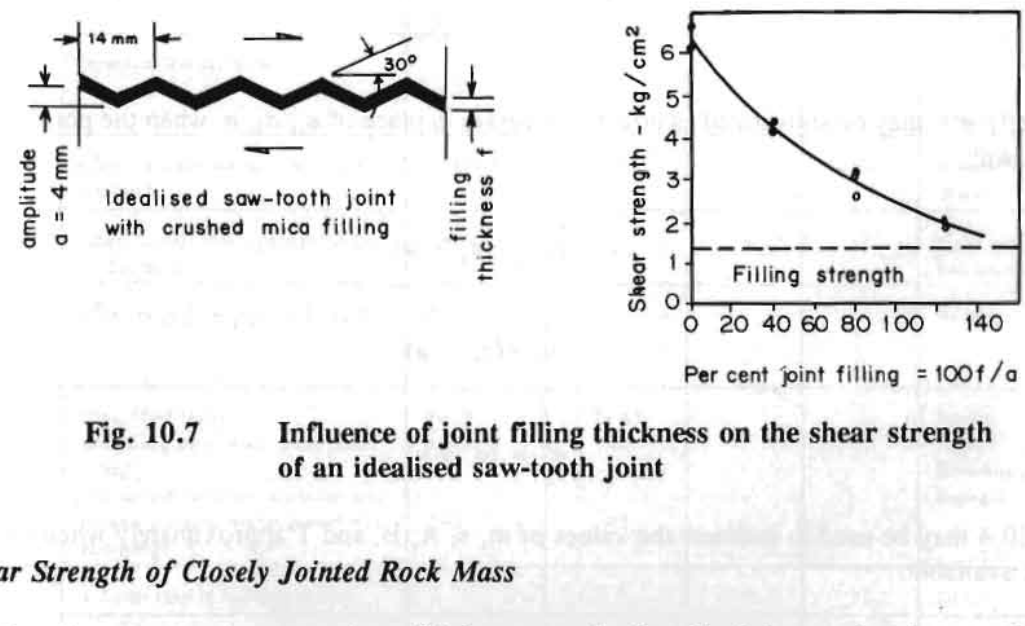


Fig. 10.7 **Influence of joint filling thickness on the shear strength of an idealised saw-tooth joint**

10.1.5 *Shear Strength of Closely Jointed Rock Mass*

When a hard rock mass contains a number of joint sets and when the joint spacing is very close, in relation to the size of slope being considered, the behaviour of the rock mass may differ significantly from that of the single discontinuity discussed earlier.

Hoek and Brown (1980) have proposed the following relationship for closely jointed rock masses:

$$\tau = A\,\sigma_c\,(\sigma/\sigma_c - T)^B$$

$$T = \frac{1}{2}\left(m - \sqrt{m^2 + 4s}\right)$$

where,

A,B = constants defining the shape of the Mohr Failure Envelope,

or,

$$\sigma_1 = \sigma_3 + \sqrt{m\ \sigma_c\ \sigma_3 + s\ \sigma_c^2}$$

where,

σ_1	=	axial failure stress,
σ_3	=	confining pressure,
σ_c	=	uniaxial compressive strength of the intact rock pieces, and
m and s	=	dimensionless constants which depend upon the shape and degree of interlocking of the individual pieces of rock within the mass.

σ_1', σ_3', σ' may be substituted as effective stresses in place of σ_1, σ_3, σ when the pore-water pressure is known:

$$\sigma_1' = (\sigma_1 - u)$$

$$\sigma_3' = (\sigma_3 - u)$$

$$\sigma' = (\sigma - u).$$

Table 10.4 may be used to estimate the values of m, s, A, B, and T approximately when *in situ* test data are not available.

10.2 DETERMINATION OF SHEAR STRENGTH

Shear strength of rock masses may be determined for minor structures without major tests by the use of the tables and equations in the preceding sections. Major structures require large-sale field tests and laboratory tests to accurately determine the shear strength.

An alternative method to determine shear strength is to back-analyze existing slope failure, to determine the shear strength parameters that must have been mobilized in the full-scale rock mass at the time of failure. It may be noted that back analysis cannot determine both c and ϕ. It is, therefore, necessary to either determine one of these, usually ϕ, from direct shear tests or run back analyses for several failures in the same material.

Table 10.3 Shear strength of filled discontinuities

Rock	Description	Peak strength		Residual strength		Tested by
		c' kg/cm^2	ϕ'	$c'kg/cm^2$	ϕ'	
Basalt	Clayey basaltic breccia, wide variation from clay to basalt content	2.4	42			Ruiz, Camargo, Midea, and Nieble
Bentonite	Bentonite seam in chalk Thin layers Triaxial tests	0.15 0.9-1.2 0.6-1.0	7.5 12-17 9-13			Link Sinclair and Brooker
Bentonitic shale	Triaxial tests Direct shear tests	0-2.7	8.5-29	0.3	8.5	Sinclair and Brooker
Clays	Over-consolidated slips, joints, and minor shears	0-1.8	12-18.5	0-0.03	10.5-16	Skempton and Petley
Clay shale	Triaxial tests	0.6	32			Sinclair and Brooker
Clay shale	Stratification surfaces			0	19-25	Leussink and Muller-Kirchenbauer
Coal measure rocks	Clay mylonite seams, 1.0 to 2.5 cm thick	0.11-0.13	16	0	11-11.5	Stimpson and Walton
Dolomite	Altered shale bed, approximately 15 cm thick	0.41	14.5	0.22	17	Pigot and Mackenzie
Diorite, granodiorite and porphyry	Clay gouge (2% clay, PI = 17%)	0	26.5			Brawner
Granite	Clay-filled faults Weakened with sandy-loam fault filling Tectonic shear zone, schistose and broken granites, disintegrated rock and gouge	0-1.0 0.5 2.42	24-45 40 42			Rocha Nose Evdokimov and Sapegin
Greywacke	1-2 mm clay in bedding planes			0	21	Drozd
Limestone	6 mm clay layer 1-2 cm clay fillings <1 mm clay fillings	 1.0 0.5-2.0	 13-14 17-21	0	13	Krsmanovic et al. Krsmanovic & Popovic
Limestone, marl and lignites	Interbedded lignite layers Lignite/marl contact	0.8 1.0	38 10			Salas and Uriel
Limestone	Marlaceous joints, 2 cm thick	0	25	0	15-24	Bernaix
Lignite	Layer between lignite and underlying clay	0.14-0.3	15-17.5			Schultze
Montmorillonite clay	8 cm seams of bentonite (montmorillonite) clay in chalk	3.6 0.16-0.27	14 7.5-11.5	0.8	11	Eurenius Underwood
Schists, quartzites, and siliceous schists	10-15 cm thick clay filling Stratification with thin clay Stratification with thick clay	0.3-0.8 6.1-7.4 3.8	32 41 31			Serafim and Guerreiro
Slates	Finely laminated and altered	0.5	33			Coates, McRorie and Stubbins
Quartz/kaolin/pyrolusite	Remoulded triaxial tests	0.42-0.9	36-38			

Care must be taken in applying the results obtained from back analysis of a particular slope to the design of a slope of different dimensions in which the normal stress levels may be different. This is because many rough discontinuity surfaces or shear zones in a closely jointed rock mass exhibit strongly non-linear Mohr envelopes.

Figure 10.8 presents the relationship between the friction angles and cohesive strengths mobilized at failure from the results of back analysis of the slope failures. This will be useful as a starting point for stability analysis or as a check on the reasonableness of assumed shear strength data.

10.3 ROCK MASS CLASSIFICATION

Rock mass classification systems have been developed in order to relate the performance of excavations made in different rock masses. These empirical systems quantify those factors that affect the performance of rock and they are then added to produce a rating number. The relationship between this rating number and the strength of rock mass is given in Tables 10.4 and 10.5 (see Section D). Unlike in soils, the friction angle of rock mass tends to increase with cohesion.

Almost all these classification systems are applicable to tunnels and not to slopes. Bieniawski's Rock Mass Rating (RMR)(1979) and Romana's Slope Mass Rating (SMR)(1988) classification systems are perhaps the ones most applicable to slopes.

Rock Quality Designation (RQD) is the ratio of the sum of lengths of cores longer than 10 cm and the total length of the drill-run. 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:

$$RQD = 115 - 3.3\,Jv \text{ (approx.)}$$

where,

Jv = total number of joints per m^3 (volumetric joint count), and
RQD = 100 for $Jv < 4.5$.

Table 10.5 presents the Council for Scientific and Industrial Research in South Africa (CSIR), Bieniawski (1979), or Rock Mass Rating (RMR) System. Table 10.6 (a,b, and c) extends it for slopes and gives slope mass ratings.

Table 10.4 Approximate relationship between rock mass quality and empirical constants

Empirical failure criterion $\sigma_1 = \sigma_3 + \sqrt{m\sigma_c\sigma_3 + s\sigma_c^2}$ $\tau = A\sigma_c\,(\frac{\sigma}{\sigma_c} - T)^B$ *Where* $T = \frac{1}{2}\,(m - \sqrt{m^2 + 4s})$	CARBONATE ROCKS WITH WELL-DEVELOPED CRYSTAL CLEAVAGE dolomite, limestone, and marble	LITHIFIED ARGILLACEOUS ROCKS mudstone, siltstone, shale, and slate (normal to cleavage)	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE sandstone and quartzite	FINE-GRAINED POLYMINERAL, IGNEOUS CRYSTALLINE ROCKS andesite, dolerite, diabase, and rhyolite	COARSE-GRAINED POLYMINERAL, IGNEOUS AND METAMORPHIC CRYSTALLINE ROCKS amphibolite, gabbro, gneiss, granite, norite, and quartz-diorite
INTACT ROCK SAMPLES *Laboratory size specimens free from joints* CSIR rating 100 NGI rating 500	m = 7.0 s = 1.0 A = 0.816 B = 0.658 T = -0.140	m = 10.0 s = 1.0 A = 0.918 B = 0.677 T = -0.099	m = 15.0 s = 1.0 A = 1.044 B = 0.692 T = -0.067	m = 17.0 s = 1.0 A = 1.086 B = 0.696 T = -0.059	m = 25.0 s = 1.0 A = 1.220 B = 0.705 T = -0.040
VERY GOOD QUALITY ROCK MASS *Tightly interlocking undisturbed rock with unweathered joints at ± 3m* CSIR rating 85 NGI rating 100	m = 3.5 s = 0.1 A = 0.651 B = 0.679 T = -0.28	m = 5.0 s = 0.1 A = 0.739 B = 0.692 T = -0.020	m = 7.5 s = 0.1 A = 0.848 B = 0.702 T = -0.013	m = 8.5 s = 0.1 A = 0.883 B = 0.705 T = -0.012	m = 12.5 s = 0.1 A = 0.998 B = 0.712 T = -0.008
GOOD QUALITY ROCK MASS *Fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3m* CSIR rating 65 NGI rating 10	m = 0.7 s = 0.004 A = 0.369 B = 0.669 T = -0.006	m = 1.0 s = 0.004 A = 0.427 B = 0.683 T = -0.004	m = 1.5 s = 0.004 A = 0.501 B = 0.695 T = -0.003	m = 1.7 s = 0.004 A = 0.525 B = 0.698 T = -0.002	m = 2.5 s = 0.004 A = 0.603 B = 0.707 T = -0.002
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 0.3 to 1m CSIR rating 44 NGI rating 1.0	m = 0.14 s = 0.0001 A = 0.198 B = 0.662 T = -0.0007	m = 0.20 s = 0.0001 A = 0.234 B = 0.675 T = -0.0005	m = 0.30 s = 0.0001 A = 0.280 B = 0.688 T = -0.0003	m = 0.34 s = 0.0001 A = 0.295 B = 0.691 T = -0.0003	m = 0.50 s = 0.0001 A = 0.346 B = 0.700 T = -0.0002
POOR QUALITY ROCK MASS *Numerous weathered joints at 30 to 500 mm with some gouge - clean waste rock.* CSIR rating 23 NGI rating 0.1	m = 0.04 s = 0.00001 A = 0.115 B = 0.646 T = -0.0002	m = 0.05 s = 0.00001 A = 0.129 B = 0.655 T = -0.0002	m = 0.08 s = 0.00001 A = 0.162 B = 0.672 T = -0.0001	m = 0.09 s = 0.00001 A = 0.172 B = 0.676 T = -0.0001	m = 0.13 s = 0.00001 A = 0.203 B = 0.686 T = -0.0001
VERY POOR QUALITY ROCK MASS *Numerous heavily weathered joints spaced < 50 mm with gouge - waste with fines.* CSIR rating 3 NGI rating 0.01	m = 0.007 s = 0 A = 0.042 B = 0.534 T = 0	m = 0.010 s = 0 A = 0.050 B = 0.539 T = 0	m = 0.015 s = 0 A = 0.061 B = 0.546 T = 0	m = 0.017 s = 0 A = 0.065 B = 0.548 T = 0	m = 0.025 s = 0 A = 0.078 B = 0.556 T = 0

Note: The CSIR method of classifying rock masses is described in Section 10.3

CSIR = Council for Scientific and Industrial Research in South Africa
NGI = Norwegian Geotechnical Institute

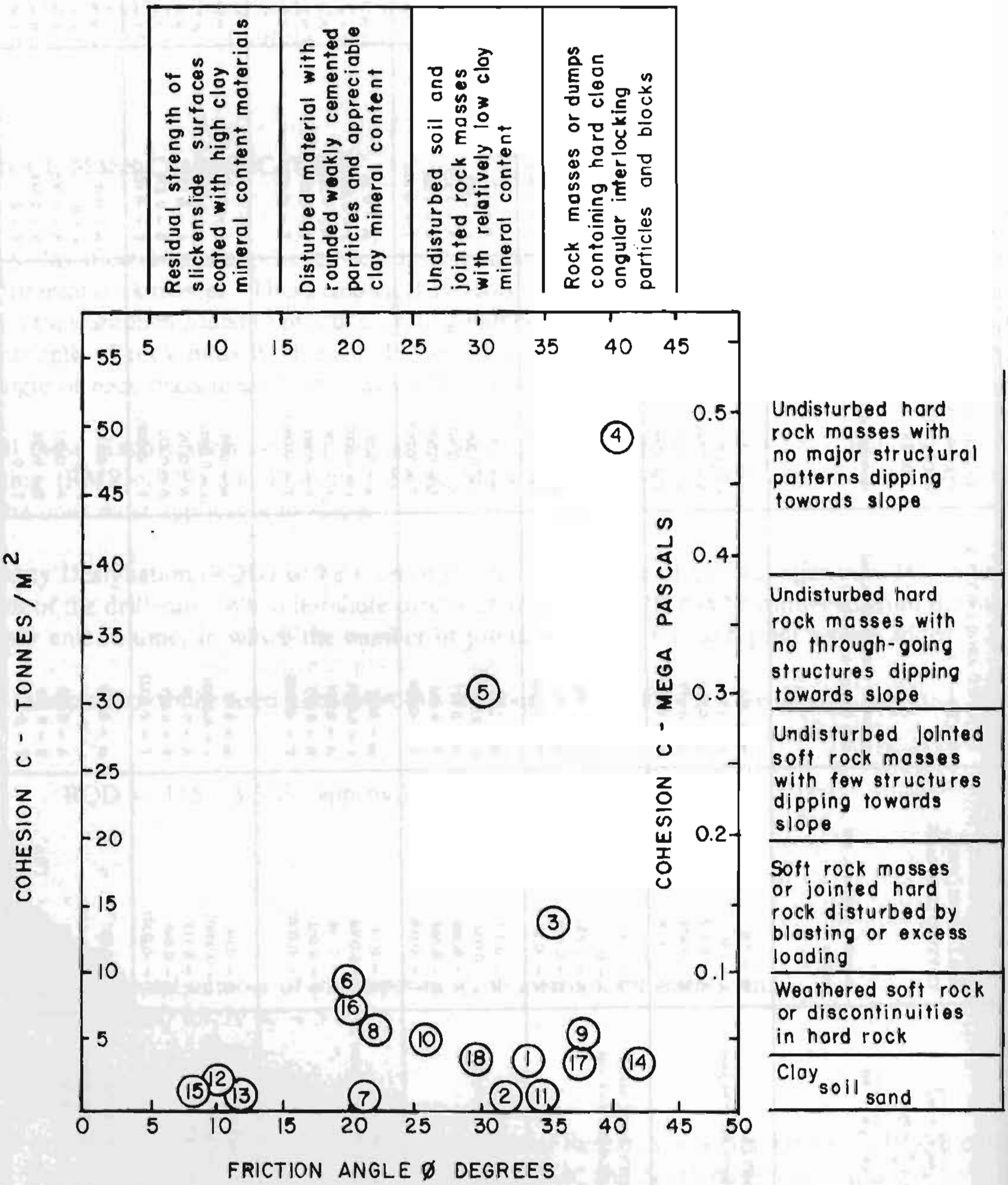


Fig. 10.8 **Relationship between the friction angles and cohesive strengths from back analysis of failed slopes**

First of all RMR (basic) is estimated by adding ratings for the first five parameters from Part A of Table 10.5. Then SMR is obtained from RMR (basic) by adding a negative frictional adjustment factor as follows:

$$SMR = RMR\ (basic) + (F1.F2.F3) + F4.$$

The adjustment ratings for factors F1,F2,F3, and F4 are given in Tables 10.6a and b. Table 10.6c gives a description of the stability of cut slopes/natural slopes.

It may be mentioned here that wedge failures may also be taken into account by substituting the dip of planes by the dip of the intersection of joint planes in Table 10.6a.

The following remedial measures are recommended as the basis of SMR (Romana 1988):

Ia.	91-100	:	None
Ib.	81-90	:	None. Scaling
IIa.	71-80	:	None. Toe ditch or fence. Spot bolting.
IIb.	61-70	:	Toe ditch or fence. Nets. Spot or systematic bolting.
IIIa.	51-60	:	Toe ditch and/or nets Spot systematic bolting Spot shotcrete
IIIb.	41-50	:	(Toe ditch and/or nets) Systematic bolting. Anchors Systematic shotcrete. Toe wall and/or dental concrete.
IVa.	31-40	:	Anchors. Systematic shotcrete. Toe wall and/or concrete. Re-excavation. Drainage.
IVb.	21-30	:	Systematic reinforced shotcrete Toe wall and/or concrete Re-excavation. Deep drainage.
Va.	11-20	:	Gravity or anchored wall. Re-excavation.

Table 10.5 Geomechanics' classification of jointed rock masses

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	PARAMETER		RANGE OF VALUES					
1.	Strength of intact rock material	Pointload strength index	> 8 MPa	4-8 MPa	2-4 MPa	1-2 MPa	For this low range, uniaxial compressive test is preferred	
		Uniaxial compressure strength	> 250 MPa	100-200 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa
	Rating		15	12	7	4	2	1
2.	Drill core quality RQD (%)		90%-100%	75%-90%	50%-75%	25%-50%	< 25%	
	Rating		20	17	13	8	3	
3.	Spacing of joints		> 2 m	0.6-2 m	0.2-0.6 m	60-200 mm	< 60 mm	
	Rating		20	15	10	8	5	
4.	Condition of joints		Very rough surface. Not continuous. No separation. Unweathered.	Slightly rough surfaces. Separation < 1 mm. Slightly weathered.	Slightly rough surfaces. Separation <1 mm. Highly weathered.	Slickenside surface 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.	Groundwater	Inflow per 10m tunnel length	None	1-10 l/min	10-25 l/min	25-125 l/min	> 125 l/min	
			OR	OR	OR	OR	OR	
		Joint water pressure ratio. Major principal stress	0	0-0.1	0.1-0.2	0.2-0.5	> 0.5	
			OR	OR	OR	OR	OR	
		General conditions	Dry	Damp	Wet or water under moderate pressure	Dripping or severe water problem	Flowing	
	Rating		15	10	7	4	0	

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	Use slope mass rating				

Note: RMR is sum of all ratings for parameters 1-6.

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rock Mass Rating (RMR)	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

D. MEANING OF ROCK MASS CLASSES

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 5m span	6 months for 4m span	1 week for 3m span	5 hours for 1.5m span	10 min for 0.5m span
Cohesion of rock mass	> 400 kPa	300-400 kPa	200-300 kPa	100-200 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	35-45°	15-25°	15-25°	< 15°
Allowable bearing pressure (t/m^2) (Indian Code)	440-600	280-440	55-145	145-280	40-55

Source: Bieniaswski 1979

Table 10.6a Adjustment rating for joints for slope mass rating

CASE	--	Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable
P T P/T	$[\alpha_j-\alpha_s]$ $[\alpha_j-\alpha_s-180°]$ F_1	> 30° 0.15	30°-20° 0.40	20°-10° 0.70	10°-5° 0.85	<5° 1.00
P P T	β_j F_2 F_2	< 20° 0.15 1	20°-30° 0.40 1	30°-35° 0.70 1	35°-45° 0.85 1	> 45° 1.00 1
P T P/T	$\beta_j-\beta_s$ $\beta_j+\beta_s$ F_3	> 10° < 110° 0	10°-0° 110°-120° -6	0° > 120° -25	0°-(-10°) -- -50	<-10° -- -60

P Plane failure
T Toppling failure
α_s Slope dip direction
β_s Slope dip
α_j Joint dip direction
β_j Joint dip

Table 10.6b Adjustment rating for methods of excavation of slopes for SMR

Method	Natural slope	Presplitting	Smooth blasting	Blasting or Mechanical	Deficient blasting
F_4	+ 15	+ 10	+ 8	0	-8

Table 10.6c Tentative description of SMR classes

Class No	V	IV	III	II	I
SMR	0-20	21-40	41-60	61-80	81-100
Description	Very bad	Bad	Normal	Good	Very good
Stability	Completely Unstable	Unstable	Partially stable	Stable	Completely stable
Failures	Big planar or soil-like	Planar or Big wedges	Some joints or Many wedges	Some blocks	None
Support	Reexcavation	Important/ Corrective	Systematic	Occasional	None
Probability of failure	0.9	0.6	0.4	0.2	0

Source: Romana 1988