Rock Mass Characterization for Engineering Design

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Abstract

Soil and rock are geological materials formed through very complex natural processes. One may be the outcome of the other. All engineering construction activities on ground and underground involve them to various extent and in different form. Understanding their engineering responses is essential to evolve economical and rational designs and adopt appropriate construction methodologies. This article very briefly brings out characterising rock as an engineering material for adoption in practice. A number of correlations established recently are presented for solving rock engineering problems.

Introduction

Rock masses have been supporting/ accommodating large structures such as foundations of buildings and dams, tunnels, shafts, underground power houses and deep excavations for mining. In any construction activity/exploration for resources, understanding of rock response is of the prime concern of engineering geologists, geophysicists, civil, mining and petroleum engineers

A rock may exist in an intact form in a limited volume but is often found as a discontinuous mass due to jointing and , its anisotropy controls the engineering behaviour. These joints could be tight or open; they may have gouge material formed as a result of shear along the joints as in the case of faults or soil may be deposited in the open joints by flowing water. The number of joints/joint sets (number of joints per meter is joint frequency), the inclination of the joints and the strength along the joints control the rock mass strength, modulus and deformational responses, i.e. the stability/failure of the rock mass. Test results on a small specimen of rock cannot be directly applied to solve engineering problems as is the case in soil. The rock mass to be treated as a discontinuum, anisotropic and inhomogeneous naturally occurring pre-stressed medium. But depending upon the ratio of extent of mass considered to the spacing of joints, it is sometimes treated as an equivalent continuum for the overall rock mass behavior.

Scenario up to 1995

The estimation of compressive strength and modulus of rock mass based on rock mass classifications was often questioned for applying to underground and open excavations, foundations of gravity dams and rock slopes. Rock core recovery, rock quality designation (RQD) and intact rock response could not establish relations, which could be verified, to estimate the response of jointed rock mass. Uniaxial compressive strength and modulus (Eti) of intact rocks were used to classify them (Deere and Miller 1966) by defining modulus ratio ranges (Eti /) as more than 500, 500-200 and less than 200. The soil-rock boundary was fixed at a compressive strength of 1MPa, based on intact rock strength value alone.

RMR Approach

With the introduction of Rock Mass Rating (RMR) by Bieniawski, (1973), the shear strength parameters cj and for rock mass

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were suggested for adoption. RMR values, varying from 100 to 0 corresponding to intact rock to heavily fractured rock, have been used to classify rock mass. RMR was adopted and linked to deformation modulus from field tested by Serafim and Pereira (1983) as

$E_{tj} = 10^{(RMR-10)/40}$, GPa. ...(1)

The compressive strength (σ_{cj}) using c_j and ϕ_j for RMR = 80, will be just 1.97 MPa, interpreting a good rock to be a very poor rock. For RMR less than 60, the compressive strength is less than 1 MPa and the rock is to be treated as soil. By considering the ratio, Etj/σ_{cj}, one observes very high unacceptable values more than a thousand even for poor rock mass ; when these values are supposed to be much less than 200 (Ramamurthy 2004).

Hoek and Brown (1980) suggested a failure criterion for rocks and the material parameters m and s in the criterion were related later on in the modified form to RMR for undisturbed rocks as (Hoek and Brown 1988),

$m_j = m_i \exp[(RMR - 100) / 28]$ ...(2)

$s_j = \exp[(RMR - 100)/9]$ ...(3)

where, subscripts i and j refer to intact and jointed rocks respectively. The uniaxial compressive strength from their criterion is given by $\sigma_{cj}/\sigma_{ci} = \sqrt{s_j}$ ...(4)

The compressive strength from Eq. (4) will be 17 times more at RMR = 80 and 5 times more at RMR = 20 for intact rock compressive strength of 100 MPa compared to Bieniawski's values; an unacceptable prediction. The modulus ratio with Eqs. (1) and (4) will result high values varying between 1500 to 1700.

GSI Approach

The Geological Strength Index (GSI) by Hoek (1994), suggests modifications to RMR and Q-system and recommends for undisturbed rock

$s_j = \exp[(GSI - 100)/9]$ ...(5)

for GSI > 25. The ratios of Etj/σ_{cj} in this case for various values of GSI are again high i.e. more than 1500 and do not decrease with the decrease in the quality of rock mass (Ramamurthy 2004).

Q-system Approach

In the Q-system, Barton et al. (1974) did not consider inclusion of σ_{ci} in their classification. Barton (2002) suggests modification to earlier Q values by a factor (σ_{ci} /100) to account for the influence of σ_{ci}. Based on the modified / corrected Q (called Q_c), Barton now suggests estimation of compressive strength and modulus from Eqs. (6) and (7),

$\sigma_{cj} / = 5\gamma\, Q_c^{1/3}$, MPa ...(6)

$E_{tj} = 10\, Q_c^{1/3}$, GPa ...(7)

For γ = 2.5 g/cc, the ratio Etj/σ_{cj} is a constant value of 800 irrespective of the Q_c values varying from 0.001 to 1000 i.e. extremely weak rock to extremely strong rock. This is unacceptable. For the compressive strength from Eq. (6), say $\sigma_{cj\,1}$, and the compressive strength ($\sigma_{cj\,2}$) calculated from c_j and φ_j suggested by Barton (2002), the ratio $\sigma_{cj\,1}$, / $\sigma_{cj\,2}$ varies from 1:7 to 54:1 depending upon the Q_c value, Ramamurthy, 2004. All the three classifications indicated above fail when tested with the modulus ratio over the entire range of rock mass quality.

A New Approach

With the scenario presented in the foregoing

Table 1: Values of n for different joint orientation angles, ?0 ,Ramamurthy (1993)

Joint orientation angle, β^0		0	10	20	30	40	50	60	70	80	90
Joint inclination parameter, n	U- shaped	0.82	0.46	0.11	0.05	0.09	0.30	0.46	0.64	0.82	0.95
	Shoulder shaped	0.85	0.60	0.20	0.06	0.12	0.45	0.80	0.90	0.95	0.98

paragraphs in estimating the compressive strength and modulus of rock mass, an extensive and well planned experiments on jointed and intact rocks, isotropic and anisotropic rocks has been of considerable importance in many ways. Most of the parameters or ratings used in the estimation of RMR, GSI and Q have not been verified experimentally. They are based on intution, experience and some back analyses! At IIT Delhi extensive testing was carried out on sandstones, dolomites, phyllites, schists, granites and number of rock-like materials in uniaxial and triaxial conditions. The number of joints varied from 13 to 92 joints per metre, friction on joints varied from 20° to 45° and sizes of specimen involved were from 38 mm diameter and 76 mm ht. to 150 x 150 x 150 mm^3 sizes. The test data of various sizes of specimens up to 620 x 620 x 1200 mm^3 from elsewhere was utilized. The joints were developed by cutting, breaking in the desired directions, stepped and berm with or without gouge material. The uniaxial compressive strength of intact rock varied from 9.5 to 123 MPa. These findings are summaries briefly in the following.

Table 2a: Suggested values of r for different values of ci (Ramamurthy 1993)

Uniaxial compressive strength, σ_{ci}, (MPa)	Joint strength parameter, r	Remarks
2.5	0.30	Fine
5.0	0.45	grained
15.0	0.60	
25.0	0.70	to
45.0	0.80	
65.0	0.90	Coarse
100.0	1.00	grained

Table 2b: Suggested values of r for different values of ci (Ramamurthy 1993)

Uniaxial compressive strength, σ_{ci}, (MPa)	Joint strength parameter, r	Remarks
2.5	0.30	Fine grained
5.0	0.45	
15.0	0.60	
25.0	0.70	to
45.0	0.80	
65.0	0.90	Coarse
100.0	1.00	grained

Strength and Modulus in Uniaxial Compression

Intact Rocks

The uniaxial compressive strength (σ_{ci}) of intact rocks (weakest to the strongest) varies from 1 MPa to more than 200 MPa. The anisotropy (inherent) influences the uniaxial compressive strength. The variation of σ_{ci} with the inclination of the plane of weakness (β°) with the specimen axis is either presented by a U-shaped, shoulder - shaped or wavy - shaped response. The expression defining shouldery response is given by (Ramamurthy 1993)

$$\sigma_{ci} = A - D\,[\cos 2(\beta_{min} - \beta)]^k \qquad ...(8)$$

where A and D are constant obtained by conducting tests at β = 0°, 30° and 90°, β_{min} is the inclination of the joint when σ_{ci} is minimum β is the inclination of the joint at which σ_{ci} is to be calculated.

The exponent k will vary from 1 to 6 depending upon whether the response is U - shaped (k = 1) or wide shouldered shaped (k = 6). More often k = 3 is used to represent the case of strongly bedded sedimentary rock formations showing shouldering effect. By conducting

Table 3: Suggested values of coefficient, k_s

M_{rj}	500	200	100	50
k_s	100	5	1/5	1/100

compression tests in three orientations $\beta = 0^o, 30^o$ and 90^o the constants are evaluated and full response is predicted.

Jointed Rocks

In jointed rocks, the uniaxial compressive strength and modulus are primarily influenced by joint frequency (J_n), joint inclination (β^o) and the strength along the critical / sliding joint. The joint which is closer to (45 - ϕ/2) with the vertical will be the critical one. The combined effect of these three factors is called the joint factor (J_f). It reflects the weakness in a jointed mass as compared to that of the intact rock. The value J_f = 0 per meter is for intact rock and could be more than 500 per meter for heavily fractured rock. The relationships for uniaxial strength (σ_{cj}) and the modulus (E_{tj}, value at 50% of the failure stress) for jointed rock are linked to the corresponding values of intact rock through the joint factor as given below (Ramamurthy 1993, 2001, Ramamurthy & Arora 1994),

$$\sigma_{cj} / \sigma_{ci} = \exp\,[-0.008 J_f] \qquad ...(9)$$

$$E_{tj} = E_{ti} \exp\,[-1.15 \times 10^{-2}\, J_f] \qquad ...(10)$$

$J_f = J_n / n.r$

where J_n = joint frequency per meter depth of rock, n = joint inclination coefficient from Table 1, r = joint strength parameter = tan ϕ_j, Table 2a and 2b, i, j = subscripts represent intact and jointed rocks respectively.

If cohesion / cementation exists on the critical joint, its effect is converted as equivalent friction. The Eqs. 9 to 11 are based on large experimental data of jointed specimens of rocks and rock-like materials (Ramamurthy 2001).

Strength and Modulus in Triaxial Compression

Intact Rocks

The compressive strength and modulus of intact rocks increases nonlinearly with the confining pressure. The failure strength of intact rocks, which have tensile strength, is represented by Eq.12 (Ramamurthy 1993, 2001),

$$\frac{\sigma'1 - \sigma'3}{\sigma'3 + \sigma t} = B_i \left(\frac{\sigma_{ci}}{\sigma'3 + \sigma_t}\right) \alpha_i$$

(12) where σ_1' = major principal stress, σ_3' = minor principal stress, σ_{ci} = uniaxial compressive strength of intact rock, σ_t = tensile strength of rock. The value of β_i and α_i are material / strength parameters. For most intact rock the values of α_i and β_i are $\alpha_i \approx$ 2/3 and $B_i \approx 1.3\ (\sigma_{ci} / \sigma_t)^{1\backslash 3}$ respectively. The modulus under any confining pressure (E_{ti}3 is linked to that of the modulus in the uniaxial (unconfined) case (E_{ti}0) by the following Eq.13,

$$E_{ti}0 / E_{ti}\,3 = 1 - \exp\,[= 0.1\sigma_{ci} / \sigma_3']$$

Jointed Rocks

Triaxial strength of jointed rocks is also given by Eq. 12 in which case, σ_t = 0 for jointed rocks and B and α become B_j and α_j. Therefore, the strength criterion for jointed mass is

$$\frac{\sigma'1 - \sigma'3}{\sigma'3} = B_j \left(\frac{\sigma_{cj}}{\sigma'3}\right)\alpha_j \qquad ...(14)$$

where σ_{ci} is obtained from Eq. 9.

The values of α_j and β_j are given by the following Eqs. 15 & 16,

$$\alpha_j = \alpha_i\ (\sigma_{cj} / \sigma_{ci})^{0.5} \qquad (15)$$

$B_i = B_j\,[0.13 \exp(2.04\,\alpha_j / \alpha_j)]$. ...(16)

The modulus is influenced by the confining pressure even in the case of jointed rocks and is given by Eq. 17,

$E_{tj}0 / E_{tj}3 = 1\text{-}\exp\,[-0.1\sigma_{cj} / \sigma'_3]$ ...(17)

Brittle - Ductile Boundary

In the case of soil, the brittle-ductile boundary is not ascertained and considered in the analysis of soil mass whereas in the case of rocks it has to be considered and is given by the following Eq.18 for intact and jointed rocks (Ramamurthy 1993, 2001).

$$\frac{\sigma' 1}{\sigma' 3} = B_{i,}\ j + 1 \qquad ...(18)$$

For the values of $\sigma' 1 / \sigma' 3$ greater than (B_i+1) or (B_j+1), brittle behaviour will be exhibited by the rock. Based on this check an analysis is conducted to represent the brittle or ductile response of the rock.

Soil-rock Boundary for Intact and Jointed Mass

The usual soil-rock boundary based on the uniaxial compressive strength of 1 MPa has been suggested for intact rocks. For jointed rocks Eq. 9 will enable to estimate the uniaxial compressive strength of jointed mass and classify it first whether it is to be treated as soil or rock. In addition to compressive strength of rock mass, Joint factor and modulus ratio have been evolved based on experimental data to catagorise the mass as rock (Ramamurthy 2001) and not as soil. For rocks

σ_{ci} > 1 MPa, J_f < 200 per meter,

$E_{tj} / \sigma_{cj} > 50$ ...(19)

If any one of the above mentioned criteria suggests that the mass is likely to behave as soil, it should be considered as such.

STAND-UP TIME IN TUNNELS

Lauffer (1958) was the first to propose the concept of stand-up time for maximum unsupported span of a tunnel and related it to seven categories of classification of rock mass. This concept was further pursued and modified to the requirements of Q-values and RMR by Barton et al (1975) and Bieniawski (1976) respectively. Bieniawski (1993) updated his earlier version by including Lauffer (1988) data..

It is very essential to estimate the stand-up time which is a function not only of the length of the unsupported tunnel but also of the modulus ratio of the rock mass in this reach, the maximum in situ stress and the seepage pressure. Therefore, the stand up time is given by (Ramamurthy 2007)

$$t_f = \frac{k\,M_{rj}}{S_u\,(p_o + u)} \qquad ...(20)$$

where, t_s = the stand-up time in years, M_{rj} = modulus ratio of rock mass estimated as given in Eq. 23, S_u = un-supported length of tunnel, m, p_o = maximum in situ stress, t/m^2 or MPa, u = seepage pressure, t/m^2 or MPa, k_s = a coefficient linked to M_{rj} as per Table 3.

Penetration Rate of Tunnel Boring Machine (TBM)

Even though prevailing rock mass classifications, RSR, RMR and Q which have been developed primarily for the tunnel stability, have been directly applied to predict the performance of TBM, none of the approaches has been found applicable to most cases. The correlations with penetration rate (P_R) have been very much case specific. Various penetration predictive models consided only a few parameters of rock. All the empirical expressions suggested are also not dimensionally in order.

Barton (2000) suggested prediction of P_R in terms of Q_{TBM} as

$P_R = 5\,(Q_{TBM})^{-0.2}$ m/hr ...(21)

Table 4: Pieve tunnel, Micaschist, Rock mass density - 2.6 T / m³ Average overburden - 500m, Cutterhead rotation – 678 rph

RMR	Joint factor J_f per m	P_R, m / hr from equation 22			Actual range P_R, m/hr
		Min. M_{rl}	Avrg. M_{rl}	Max. M_{rl}	
83	85	1.7	1.7	1.5	1.2 – 2.2
75	125	2.0	1.9	1.8	1.6 – 2.5
68	160	2.4	2.2	2.0	2.0 – 2.8
57	215	2.7	2.7	2.4	2.1 - 3.3
50	250	3.0	3.1	2.8	2.3 – 3.4
35	325	3.9	4.0	3.5	2.5 – 3.5

where Q_{TBM} is a modification of Q_c -values of Barton et al. (1974) and not Q_c-value of Barton (2002) and involves 21 parameters. Q-values as such do not give reliable estimation of compressive strength of rock mass, Ramamurthy (2004).

Present Approach

Whether it is in Q_{TBM}, RMR or any other rock mass classification linked to P_R, the modulus of rock has been ignored. For producing indentation by crushing under the tip of the cutter, compressive and tensile strengths are important. In doing so, whatever deformation/ penetration is produced will depend on the modulus response of rock mass. It is therefore very essential that the modulus of rock mass be considered. More precisely, the modulus ratio to account for the combined influence of compressive strength σ_{ci} and modulus (E_{tj}) of the rock mass, i.e. $M_{rj} = E_{tj} / \sigma_{ci}$ Basically under each cycle of boring by TBM, the various other major factors which control P_R are included in the following Eq. (22). This equation is dimensionally correct and predicts P_R value per meter of advance of boring as indicated below,

$$P_R = \frac{(T?A)\,(\sigma_{ci}/\sigma_t)\,R.N.\,(DR1/100).s}{P_o\,.\,M_{rj}}$$

...(22)

where T= net thrust, tons, A = area of the cutter head, m², σ_{ci} = compressive strength of intact rock, MPa, σ_t = tensile strength of intact rock, MPa, R=number of rotations of cutterhead, per hour, N = number of cutters, per m², DRI = drilling rate index based on compressive strength of intact rock, s=Unit length of drilling, m , p_o = mean biaxial stress on the cutting face, T/m² (or taken as density of rock mass times over burden height).

M_{rj} = modulus ratio of rock mass, (= Etj/ σ_{cj}), obtained from Eq. (23),

$$\frac{M_{rj}}{M_{ri}} = \exp\,(-00035\,J_f),\ \text{(Ramamurthy 2004)}$$

(23)

M_{ri} = modulus ratio of intact rock, E_i / σ_{ci} in unconfined condition.

In the above Eq. (23), the influence of seepage pressure is not considered, since most of the seepage pressure is dissipated at the cutting face due to the presence of fractures, joints, etc. The seepage pressure acting through the intact rock will be negligible anyway on the cutting face. The rock parameters are to be obtained under saturated condition, if seepage exists.

The ratio (σ_{ci}/σ_t) takes care of inherent anisotropy in the intact rock and also its brittleness.

Case Study

Excellent data was collected by Sapigni et al. (2003) from NW Alps. This data is applied to verify Eq (22). The data reported by

Sapigni et al. for metabasite in Maen tunnel and for micaschist and metadiorite in Pieve tunnel for various values of RMR is used. These RMR values have been converted to J_f, Joint factor, as per Eq. (24) based on matching compressive strengths from these approaches, Ramamurthy (2004).

$$J_f = 5\,(100 - RMR)$$

(24) The values of compressive strength, tensile strength and modulus values for the three rocks and with the basic tunnel equipment data of Maen tunnel and that of Pieve tunnel, estimation of P_R is made. Table 4 presents actual range of P_R versus J_f, RMR and the values of P_R estimated from Eq. (22) for Pieve tunnel only. A comparison of the calculated and field measured mean P_R values for 16 rock types clearly suggested a good agreement.

Particularly for RMR < 60 (or J_f > 200) decrease of P_R is generally observed. This is mainly due to the dislodging of rock blocks hindering P_R values. Such decrease in P_R is indicated from the data of Sapigni et al. In such situations the operators usually reduce the rotations of TBM, which also results lower .

The special advantage of adopting Eq. (22) for predicting P_R is that all the input data is factual and from test conducted on the rocks as per approved practice. It is dimensionally correct compared to other prevailing expressions. The P_R may be calculated per meter of boring in a specified length having similar formation. Assessment of P_R per meter length of tunnel is specified because the J_f value is estimated per meter length. On the basis of this one could estimate average P_R in each zone and then an overall estimation of the P_R or for the entire length of the tunnel would result. Since an excellent site investigation of a tunnel alignment is essential for its successful execution with TBM, Eq (22) will certainly be very handy in predicting P_R.

Conclusions

The rock mass being a naturally occurring pre-stressed discontinuum, its engineering response is only partly understood / estimated, even though the methods of analysis, design and excavation have significantly well advanced. A failure criteria for rocks and rock masses has been developed. The concept of Joint has been used to estimate, in unconfined state, the compressive strength and modulus of the rock mass for engineering application. It also enables to estimate strength and modulus under any confining pressure and predict stand up time and penetration rate of TBMs. These research findings have been applied to predict the response of rock mass around underground chambers, tunnels, mine excavations and stability of rock foundations,

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