Estimating Hoek-Brown Rock Mass Strength Parameters from Rock Mass Classifications

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The use of rock mass classifications for designing support of underground excavations in rock has gained acceptance over the past 15 years to the extent that most geotechnical data collection programs now focus on the input parameters to the Norwegian Geotechnical Institute tunneling quality index (Q), the geomechanics classification rock mass rating (RMR), or both. In developing their empirical failure criterion for intact and heavily jointed rock masses, Hoek and Brown turned to rock mass classification schemes for the prediction of rock mass strength. The backgrounds of the two classifications used most frequently are reviewed, and ways in which they may be adapted to derive the Hoek-Brown rock mass strength parameters m, s, and σ_c are suggested. To incorporate the results of practical applications of the failure criterion under real engineering conditions, Hoek and Brown proposed equations to estimate rock mass strength parameters from classifications. These equations relate Bieniawski's RMR to m/m_i and s (where m_i is the Hoek-Brown parameter m for intact rock). The Barton et al. Q-index can also be used according to Bieniawski through a relationship between RMR and Q. The use of the complete quantitative rating or index from either classification is not recommended, and it is suggested that some components of the classification schemes are more appropriate than others in estimating the Hoek-Brown parameters. The proposed adaptations of Bieniawski's and Barton's work partially overcome the concern that classifications derived specifically for the estimation of tunnel support may not be appropriate for estimating rock mass strength.

The requirements of a characterization method developed for the design of tunnel support may be quite different from those needed for the estimation of rock mass strength parameters. Bieniawski (*1*) proposed that, in a tunneling application, a rock mass classification scheme has four purposes:

1. To divide a particular rock mass into groups of similar behaviour;
2. To provide a basis for understanding the characteristics of each group;
3. To yield quantitative data for the design of tunnel support; and
4. To provide a common basis for communication.

These principles led Bieniawski in his development of the geomechanics classification rock mass rating (RMR) (*1–6*).

The "quality" of the ground as an engineering medium is an intrinsic property that is spatially variable. It is a function of the strength of the intact material, the geometry of the rock mass fabric, and the character of the discontinuities that divide the intact rock into discrete blocks. Because the rock mass is used in engineering for civil or mining excavations, more variables are added to the behavioral character of the ground associated with excavation-induced effects. However, although properties such as induced stresses, excavation size, or water pressures are justifiably included in some classifications for designing tunnel support, rock mass strength is not a function of engineering use, and such parameters should not be considered in estimating strength parameters from a classification.

From field observations and discussions with practicing rock mechanics engineers, it appears that the behavior of better-quality rock masses is dominated by the geometry of the rock mass fabric, specifically block size and block shape; that of fair- to poor-quality rock masses, by the interblock shear strength and deformational characteristics; and that of worse-quality rock masses, by the low strength of the intact material. It is within this very broad generalization that developing Hoek-Brown rock mass strength parameters from rock mass classifications is considered.

BACKGROUND

The background of the Hoek-Brown failure criterion, Bieniawski RMR and the Norwegian Geotechnical Institute (NGI) tunneling quality index (Q) will be reviewed as it applies to this paper.

Hoek-Brown Failure Criterion

The most detailed description of the Hoek-Brown failure criterion is contained in the Rankine lecture by Hoek (*7*) that discusses the trial-and-error process of experimentally fitting triaxial test data with distorted parabolic curves to arrive at the following relationship:

$$\sigma_1' = \sigma_3' + \sqrt{m\sigma_c\sigma_3' + s\sigma_c^2} \quad (1)$$

where

σ_1' = major principal effective stress at failure,
σ_3' = minor principal effective stress or confining pressure,
m, s = material constants, and
σ_c = uniaxial compressive strength of the intact rock.

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The Hoek-Brown empirical failure criterion contains three constants: m, s, and σ_c. In Hoek and Brown's words, m and s are "constants which depend on the properties of the rock and upon the extent to which it has been broken before being subjected to the [failure] stresses. . ." (*8*). All three constants are intrinsic or generic parameters and not related to any condition imposed by engineering.

"The manner in which fracture initiates and [failure] propagates . . . is reflected in the value of m. . ." (*8*). Hoek and Brown clearly indicate the way in which m is dependent on material properties, crystalline matrix, geological history, and so on. In the 1983 Rankine lecture, Hoek commented that m was "very approximately analogous to the angle of friction, ϕ', of the conventional Mohr-Coulomb criterion" (*7*). The same paper describes s as being very approximately analogous to the cohesive strength (c') of the Mohr-Coulomb criterion and goes on to discuss its bounds. Intact rock specimens with finite tensile strength have a maximum value of s equal to 1. Heavily jointed or broken rock in which the tensile strength, cohesive strength, and effective normal stress are zero is characterized by a minimum s-value of zero.

The main requirement of a classification to estimate rock mass strength parameters, then, is a close correspondence between the parameters included in the classification and the factors that affect the constants in the Hoek-Brown criterion. Parameters related to the geology and mineralogy of the rock mass, the degree to which the rock mass is broken, and the intact material strength should therefore be considered in deriving a relationship between rock mass strength and a classification.

Hoek and Brown (*9*) showed a plot of the parameter s and the ratio m/m_i against the NGI and adjusted RMR classification ratings (*4*) estimated for intact, undisturbed jointed and recompacted andesites in the initial publication on the empirical strength criterion for rock masses. The two classification schemes were scaled on the graph using Bieniawski's (*4*) correlation

$$\mathrm{RMR} = 9 \log_e \mathrm{Q} + 44 \tag{2}$$

Equations were derived from these plots by Priest and Brown (*10*), who related m/m_i and s directly to Bieniawski's RMR. These equations were modified by Hoek and Brown and published in the 1983 Rankine lecture (*7*). They were derived empirically from relatively few data points generated by extensive work on the Panguna andesites in Bougainville, Papua New Guinea. As the Hoek-Brown failure criterion has gained acceptance and has been used by the engineering community, it has been found that the values of m and s listed by Hoek (*7*) were somewhat conservative for practical engineering design. The values of the constants were then increased to model the behavior of "undisturbed or interlocked" rock masses by an arbitrary amount based on the experience of the authors.

Present correlations between the geomechanics classification and the Hoek-Brown failure criterion constants are given by Hoek and Brown (*11*) as

Disturbed rock masses:

$$\frac{m}{m_i} = \exp\left(\frac{\mathrm{RMR} - 100}{14}\right) \tag{3}$$

$$s = \exp\left(\frac{\mathrm{RMR} - 100}{6}\right) \tag{4}$$

Undisturbed or interlocked rock masses:

$$\frac{m}{m_i} = \exp\left(\frac{\mathrm{RMR} - 100}{28}\right) \tag{5}$$

$$s = \exp\left(\frac{\mathrm{RMR} - 100}{9}\right) \tag{6}$$

These equations were used to generate the values of m and s given in Table 1, which has been used extensively by the engineering community with a reasonable amount of success. However, experience in evaluating the behavior of underground excavations in civil and mining engineering projects shows that the values in Table 1 still underestimate the strength of rock masses at low confining stresses, that is, close to the boundary of an excavation. This is not too surprising in light of the meager data from which the relationships were derived and the difficulty in obtaining a complete suite of test results to "prove" the proposed criterion under a wide range of broken rock conditions. It must be remembered that the Hoek-Brown failure criterion was developed by curve-fitting the results of many triaxial compressive strength tests of intact rock and extended empirically to cover isotropic broken rock masses with little substantive correlation.

In the remainder of this paper, disturbed rock mass strength parameters will be discussed because they most closely represent the situation found in rock slope engineering. Increased rock mass strength parameters would be required if the engineering application were an underground excavation.

Geomechanics Classification

The geomechanics classification has been developed over the past 15 years by Bieniawski, who first proposed the RMR in 1973 and revised the scheme subsequently in 1974, 1975, 1976, 1979, and 1989. The RMR is the sum of a number of weighted parameters, and it is the number of parameters, the parameters themselves, and the weightings that have changed over the years. Table 2 shows the changes to the parameter ratings that have been suggested during the development of the geomechanics classification. The current recommendations use a basic RMR found by summing individual partial ratings [after Bieniawski (*5,6*)]:

Characteristic	*Rating*
Strength of intact rock (point load or compressive)	0–15
Drill core quality, Deere's RQD (*12*)	3–20
Spacing of discontinuities	5–20
Condition of discontinuities	0–30
Groundwater	0–15

In applying his classification to the estimation of support in tunnels, Bieniawski includes an adjustment for the orientation of predominant discontinuity sets relative to the orientation of the tunnel drive. This adjustment is inapplicable in the estimation of rock mass strength and will not be considered further in this paper.

TABLE 1 APPROXIMATE RELATIONSHIP BETWEEN ROCK MASS QUALITY AND MATERIAL CONSTANTS

		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 POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS & METAMORPHIC CRYSTAL-LINE ROCKS - *amphibolite, gabbro gneiss, granite, norite, quartz-diorite*
INTACT ROCK SAMPLES						
Laboratory size specimens free	m	7.00	10.00	15.00	17.00	25.00
from discontinuities	s	1.00	1.00	1.00	1.00	1.00
CSIR rating: RMR = 100	*m*	*7.00*	*10.00*	*15.00*	*17.00*	*25.00*
NGI rating: Q = 500	*s*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
VERY GOOD QUALITY ROCK MASS						
Tightly interlocking undisturbed rock	m	2.40	3.43	5.14	5.82	8.56
with unweathered joints at 1 to 3m.	s	0.082	0.082	0.082	0.082	0.082
CSIR rating: RMR = 85	*m*	*4.10*	*5.85*	*8.78*	*9.95*	*14.63*
NGI rating: Q = 100	*s*	*0.189*	*0.189*	*0.189*	*0.189*	*0.189*
GOOD QUALITY ROCK MASS						
Fresh to slightly weathered rock, slightly	m	0.575	0.821	1.231	1.395	2.052
disturbed with joints at 1 to 3m.	s	0.00293	0.00293	0.00293	0.00293	0.00293
CSIR rating: RMR = 65	*m*	*2.006*	*2.865*	*4.298*	*4.871*	*7.163*
NGI rating: Q = 10	*s*	*0.0205*	*0.0205*	*0.0205*	*0.0205*	*0.0205*
FAIR QUALITY ROCK MASS						
Several sets of moderately weathered	m	0.128	0.183	0.275	0.311	0.458
joints spaced at 0.3 to 1m.	s	0.00009	0.00009	0.00009	0.00009	0.00009
CSIR rating: RMR = 44	*m*	*0.947*	*1.353*	*2.030*	*2.301*	*3.383*
NGI rating: Q = 1	*s*	*0.00198*	*0.00198*	*0.00198*	*0.00198*	*0.00198*
POOR QUALITY ROCK MASS						
Numerous weathered joints at 30-500mm,	m	0.029	0.041	0.061	0.069	0.102
some gouge. Clean compacted waste rock	s	0.000003	0.000003	0.000003	0.000003	0.000003
CSIR rating: RMR = 23	*m*	*0.447*	*0.639*	*0.959*	*1.087*	*1.598*
NGI rating: Q = 0.1	*s*	*0.00019*	*0.00019*	*0.00019*	*0.00019*	*0.00019*
VERY POOR QUALITY ROCK MASS						
Numerous heavily weathered joints spaced	m	0.007	0.010	0.015	0.017	0.025
<50mm with gouge. Waste rock with fines.	s	0.0000001	0.0000001	0.0000001	0.0000001	0.0000001
CSIR rating: RMR = 3	*m*	*0.219*	*0.313*	*0.469*	*0.532*	*0.782*
NGI rating: Q = 0.01	*s*	*0.00002*	*0.00002*	*0.00002*	*0.00002*	*0.00002*

NOTE: m and s are values for disturbed rock mass; *m* and *s* are values for undisturbed rock mass.

TABLE 2 CHANGES TO BIENIAWSKI'S RATINGS SINCE FIRST PUBLICATION

Year	Strength	RQD	Spacing	Condition	Groundwater	Comment
1973	0–10	3–16	5–30	2–19	2–10	Orient. +ve
1974	0–10	3–20	5–30	0–15	2–10	Orient. +ve
1975	0–15	3–20	5–30	0–25	0–10	Orient. –ve
1976	0–15	3–20	5–30	0–25	0–10	Interpretn.
1979	0–15	3–20	5–20	0–30	0–15	Interpretn.
1989	0–15	3–20	5–20	0–30	0–15	Interpretn.

Note: Other modifications have been made in the interpretations of RMR values, including new class ranges, alterations in stand-up time, Mohr–Coulomb rock mass strength parameters.

Although the geomechanics classification can yield RMR values anywhere between 0 and 100, Bieniawski recommends consideration of only five rock mass classes in order to design support. However, he suggests that the exact basic RMR be used to estimate m/m_i and s parameters from Equations 3–6 (*6*). Thus, the classification required for estimating support need not be as sensitive or accurate as that required for estimating rock mass strength parameters.

The characteristics that affect the behavior of an excavation in rock are a combination of generic parameters and engineering-induced effects. The geomechanics classification combines both, and this may be justified in the design of support. In contrast, a classification for rock mass strength should contain only generic parameters. These two observations are the main reasons that a refinement of the classification—into RMRm, RMRs, and intact uniaxial compressive strength—

is proposed for the estimation of Hoek-Brown parameters. Because the Hoek-Brown criterion is stated in effective stress terms, the influence of groundwater pore pressure is also explicitly considered.

The remainder of this section is devoted to determining rock mass strength parameters from the partial ratings RMRm and RMRs.

Geomechanics Classification m *Parameter: RMRm*

Bieniawski's basic RMR incorporates strength of intact rock material, drill core quality, spacing of discontinuities, condition of discontinuities, and groundwater. Underground excavation experience suggests that the way in which failure would propagate through a rock mass would be very sensitive to the condition of discontinuities. It is proposed that the partial rating for Bieniawski's discontinuity condition term be referred to as "RMRm" and that it be related to the Hoek-Brown parameter m. In developing a relationship between RMRm and m, reference will be made to the parameter m either as m_b for broken rock or as m_i for intact rock.

Plotting the values of m_b/m_i against Bieniawski's discontinuity condition (RMRm) originally calculated by Hoek and Brown (*8*) for the Panguna andesites gives curves with a poor visual fit to the data. One reason for this is that the early assessment used the incremental rating values given by Bieniawski (*4*), which proceed from 0 to 10, 20, 25, and 30 and incorporate three earlier terms used by Bieniawski (*2*): state of weathering, separation of joints, and continuity of joints.

Bieniawski eliminated the weathering term in 1974 because it was considered to be included in uniaxial compressive strength and discontinuity condition. In the current assessment, however, the Hoek-Brown constant σ_c refers to the uniaxial compressive strength of the intact rock material and does not, therefore, include an allowance for weathering. The author considers that weathering is one of the important factors in rock mass strength, because interblock shear is dominated by the presence or absence of weathering products caused by the passage of groundwater through discontinuities. Bieniawski (*6*) reintroduced weathering, along with roughness and infilling, in an amplified classification chart given in Table 3. It is this chart, extended to include intact rock, that is used to derive RMRm.

Intact rock, without discontinuities, relates to the initiation of fracture and has been evaluated by extrapolating discontinuity length, separation, and roughness in Table 3. It is proposed that Bieniawski's rating table be extended to include intact rock with a rating of 40. Table 3 has been used to refine the Panguna andesite data given by Hoek and Brown (*8*) and Jaeger (*13*). The rock mass strength values from triaxial testing and the interpreted RMRm ratings are shown in Table 4.

TABLE 3 RMRm = $\sum$ (DISCONTINUITY CONDITION RATINGS) (*6*)

Parameter	Ranges of Values				
Trace Length	>1 m	1–3 m	3–10 m	10–20 m	<20 m
Rating	6	4	2	1	0
Separation	None	<0.1 mm	0.1–1 mm	1–5 mm	>5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Smooth	Polished	Slickensided
Rating	6	5	3	1	0
		Hard filling		*Soft filling*	
Infilling	None	<5 mm	>5 mm	<5 mm	>5 mm
Rating	6	4	2	2	0
Weathering	Fresh	Slight	Moderate	High	Complete
Rating	6	5	3	1	0
Intact	Rating enhanced by 10				

Figure 1 shows a plot of RMRm against m_b/m_i for the revised data. This figure may be used as a design chart to estimate m_i/m_b from RMRm; alternatively, the following correlation may be used:

$$\frac{m_b}{m_i} = \exp\left(\frac{\text{RMRm} - 40}{5}\right) \tag{7}$$

Geomechanics Classification s *Parameter: RMRs*

Although there may be some overlap in the two parameters, Bieniawski's drill core quality and the spacing of discontinuities together make up the geometry of the rock mass. It is proposed that the sum of the partial ratings for drill core quality and spacing of discontinuities be referred to as "RMRs" and that it be related to the Hoek-Brown parameter s (see Table 5).

The maximum ratings for rock quality designation (RQD) and discontinuity spacing are 20 each (*6*). It is proposed that Bieniawski's spacing table be extended to include a rating of 25 for unjointed rock without discontinuities. This would give intact rock a combined partial sum of RMRs = 45. The minimum value of RMRs is 8 (minimum RQD rating of 3 plus minimum spacing rating of 5), which Hoek and Brown applied

TABLE 4 RMRs = RQD RATING AND SPACING RATING (*6*)

Intact Rock	$m_i = 18.9$		$s = 1$		$\sigma_c = 265MPa$	
Ratio m_b/m_i	1.0	0.0147	0.0061	0.0021	0.0016	0.0006
s	1.0	0.0002	0	0	0	0
Rock Mass	Intact	Undist	Recomp	Fresh	ModWeath	HiWeath
RMRm from Table 3, after Bieniawski [6]						
Length	6	1	0	0	0	0
Separation	6	4	1	1	1	1
Roughness	6	5	5	3	3	1
Infilling	6	4	4	2	2	1
Weathering	6	6	6	5	3	1
Intact	10					
Total RMRm	40	20	16	11	9	4

Note: Rock mass terms used by Hoek and Brown are: intact, undisturbed, recompacted, fresh, moderately weathered and highly weathered.

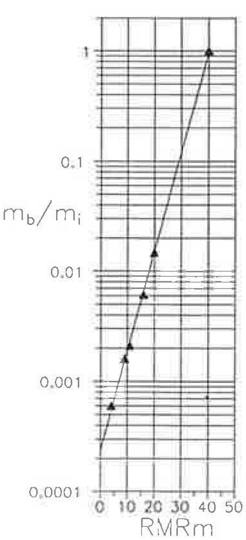

FIGURE 1 Correlation of RMRm with Hoek-Brown parameters.

TABLE 5 ROCK MASS STRENGTHS AND RMRm VALUES FOR PANGUNA ANDESITES (9)

Parameter	Ranges of Values				
Drill quality (RQD)	90–100%	75–90%	50–75%	25–50%	<25%
Rating	20	17	13	8	5
Joint spacing	>2 m	0.6–2 m	0.2–0.6 m	60–200 mm	<60 mm
Rating	20	15	10	8	5
Intact rock	Rating enhanced by 5				

to the rock mass conditions for undisturbed core samples of the Panguna andesites. This limits the ability to predict s from RMRs, although allowing RMRs to tend to zero as the rock mass becomes more broken may be warranted.

In their evaluation of the rock mass strength envelopes for the Panguna andesties, Hoek and Brown assumed that a value of $s = 0$ applied to the recompacted and weathered specimens. The only data points that can be derived in a plot of RMRs against s are for intact rock, $s = 1$, and undisturbed rock, $s = 0.0002$. The relationship between these s values and their respective RMRs values (45 and 8) is shown in Figure 2. Because there are only two points, a straight line relationship on the semilog plot has been inferred. This design envelope is obviously more tenuous than the one drawn for m_b/m_i, although it is considered as valid as the original presented by Hoek and Brown (9). The equation of the line is given by

$$s = \exp\left(\frac{\text{RMRs} - 45}{4.5}\right) \tag{8}$$

Other Geomechanics Classification Parameters

The strength of intact rock is obviously identical to the Hoek-Brown parameter σ_c and should be used directly rather than by ascribing a rating value. The ranges of strength values currently used in the geomechanics classification follow International Society for Rock Mechanics (ISRM) recommendations (14), and each rock strength group is assigned a rating value. It should be noted that the other two Hoek-Brown rock mass strength parameters, m and s, are dimensionless. The introduction of a dimensioned parameter (σ_c) becomes critical in establishing the input parameters for design analyses. It is therefore suggested that considerable care be taken in evaluating material strength of the intact rock.

Groundwater conditions are directly associated with the engineering structure to be created or the engineering role that the rock mass is required to play. Although an assessment of water conditions is undoubtedly important in the design of rock mass support, it should not be included in an evaluation of rock mass strength parameters that are generic in origin and not a function of the engineering project in question. It is again noted that the Hoek-Brown failure criterion is expressed in effective stress terms, and groundwater pore pressure is therefore explicitly considered.

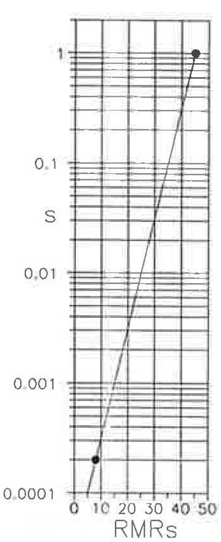

FIGURE 2 Correlation of RMRs with Hoek-Brown parameters.

NGI Tunneling Quality Index

Barton et al. (15) proposed a guide for estimating tunnel support requirements using a classification index. The original document, first published as an internal NGI report, contains a wealth of background information that forms the basis of the present discussion. It should be noted that the rating system selected by Barton et al. has not changed since the first publication. As with Bieniawski's RMR, a relationship between the classification index and rock mass strength parameters was not proposed, although various components of rock mechanics behavior are mentioned—for example, support pressure, approximate joint residual friction angles, and the effective shear strength of the rock mass. A review of the classification parameters follows.

The rock mass Q-index is derived from six parameters (15):

- Degree of jointing of the rock, in terms of RQD,
- Number of joint sets (J_n),
- Roughness and degree of planarity of the joints (J_r),
- Alteration of filling along the joints (J_a),
- Water inflow (J_w), and
- Rock load (SRF).

The complete index is found by multiplying the three quotients shown in Equation 9.

$$Q = \frac{\text{RQD}}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\text{SRF}} \tag{9}$$

The numerical value of the index varies from 0.001 for exceptionally poor quality rock conditions to 1,000 for exceptionally good quality, intact rock. Each quotient represents a different rock mass characteristic [Barton et al. (15)]. The first quotient, RQD/J_n, represents the structure of the rock mass and is a crude measure of the block or particle size. This would suggest a possible relationship with the Hoek-Brown parameter s. The second quotient, J_r/J_a, represents the frictional characteristics of the joint walls or infillings. Barton et al. report that $\tan^{-1}(J_r/J_a)$ approximates rock mass shear strength, and this suggests a possible relationship with the Hoek-Brown parameter m. The third quotient, J_w/SRF, comprises two stress parameters associated with water pressure and rock load. It is therefore design-dependent and will not be considered further.

The rock mass strength data reported for the Panguna andesites by Hoek and Brown (*8*), are shown in Table 6 along with the proposed NGI parameters. Figure 3 shows a plot of the natural logarithms of the quotient J_r/J_a against m_b/m_i and RQD/J_n against s for these data. The triangular data points give a good visual fit to the correlation:

$$\log_e\left(\frac{m_b}{m_i}\right) = 2 \log_e\left(\frac{J_r}{J_a}\right) - 3.35 \qquad (10)$$

Again, the relationship between RQD/J_n and s is tenuous. Not only is it based on only two data points, but selection of the minimum NGI value of RQD = 10 percent constrains the location of one of the data points with an uncertain error margin. A suggested correlation is

$$\log_e s = 2 \log_e\left(\frac{\text{RQD}}{J_n}\right) - 9.2 \qquad (11)$$

It is interesting to note that in Table 6, "block size" quotients RQD/$J_n < 1$ are all associated with interpreted s values from triaxial testing of zero. The significance of this may be seen from a comment by Barton et al. (*15*): "If the quotient is interpreted in units of centimetres, the . . . particle sizes . . . are seen to be crude but fairly realistic approximations." A rock mass with individual particles or blocks only 10 mm (1 cm) across represents a condition in which the tensile strength and cohesive strength would be zero. Hoek and Brown (*11*) suggest that such a rock mass be characterized by a minimum value of s = 0. The triaxial work on the Panguna andesites was clearly in this range; however, extrapolation of block sizes for low values of RQD/J_n to the scale of a real rock mass may not be realistic.

TABLE 6 NGI PARAMETERS FOR PANGUNA ANDESITES (*9*)

Rock Mass	Intact	Undist	Recomp	Fresh	ModWeath	HiWeath
Ratio m/m_i	1.0	0.0147	0.0061	0.0021	0.0016	0.0006
s	1.0	0.0002	0	0	0	0
RQD	100	10	10	10	10	10
J_n	1	9	12	15	18	20
RQD/J_n	100	1.11	0.833	0.667	0.555	0.5
J_r	4	2*	1.5	1	1	1
J_a	0.75	3*	4	4	6	8
J_r/J_a	5.33	0.63	0.375	0.25	0.167	0.125

All values of NGI parameters taken from Hoek and Brown [9] except those marked *, which have been re–assessed after reviewing the original publication of this data in Jaeger [13].

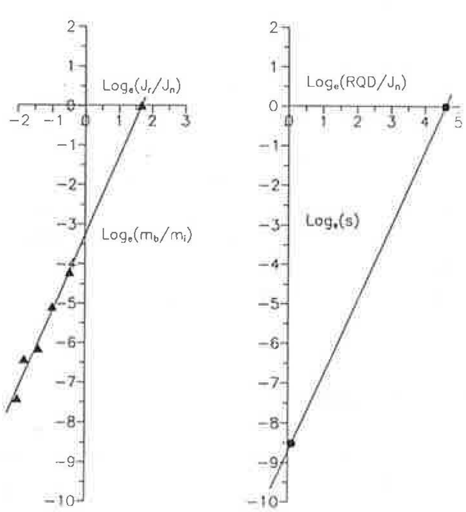

FIGURE 3 Correlations between NGI quotients and Hoek-Brown parameters.

ESTIMATING HOEK-BROWN PARAMETERS

The process of deriving the complete Hoek-Brown parameter set is described and followed by a worked example.

The relationships established between rock mass strength and classifications are all in terms of the ratio m_b/m_i and s. It follows that a value of m_i is required in order to calculate m_b. The complete parameter set thus includes m_i, for intact rock; m_b/m_i, from one or more classification; s, from a classification; and σ_c, preferably from laboratory testing.

On certain projects it may be justifiable to set out a complete rock mechanics testing program and generate a full suite of triaxial results for the prototype rocks on a particular site. In general, however, only a limited amount of testing is likely, probably restricted to uniaxial compressive strength and point-load index.

In the absence of site-specific data on intact rock material strength, a field approximation can be used, such as that presented in Table 7, based on the proposals of ISRM (*14*). Under these circumstances, tabulated values for m_i must be used. Table 8 shows values of the constant m_i taken from published results of triaxial testing by Hoek and Brown (*8*), Jaeger (*13*), and Jackson et al. (*16*). Other published works have not been directly concerned with rock mass strength, and interpretations of m_i have not been made. The various rock types tested were grouped according to mineralogy and grain size within the geological classification of sedimentary, metamorphic, and igneous rocks. It was found that values of m_i decrease with grain size for any particular group, with up to a 50 percent reduction from coarse to very fine grained.

TABLE 7 APPROXIMATION OF UNIAXIAL COMPRESSIVE STRENGTH

Uniaxial Compressive Strength, σ_c (MPa)	Point Load Index, I_s (MPa)	Term	Field Estimate of Strength	Examples*
> 250	> 10	Very Strong	Requires many blows of geological hammer to break intact rock specimen	Basalt, chert, diabase, quartzite
100 – 250	4 – 10	Strong	Hand held specimen broken by single blow of geological hammer	Amphibolite, basalt, gneiss dolomite, gabbro, granite, limestone, marble, tuff
50 – 100	2 – 4	Mod. Strong	Knife cannot scrape surface, shallow indentations under firm blows of pick	Andesite, limestone, marble phyllite, sandstone, schist, shale, slate
25 – 50	1 – 2	Mod. Weak	Firm blow with geological pick indents rock to 5mm, knife just scrapes surface	Claystone, coal, concrete, schist, shale, siltstone
5 – 25	**	Weak	Knife cuts material, but too hard to shape into triaxial specimen	Chalk, rocksalt, potash
1 – 5	**	Very Weak	Material crumbles under firm blows of geological pick, can be shaped with a knife	Highly weathered or altered rock, fault zone

* All rock types exhibit a broad range of uniaxial compressive strengths which reflects heterogeneity in composition and anisotropy in structure. Stronger rocks are characterized by well interlocked crystal fabric and few voids.

** Rocks with a uniaxial compressive strength below 25MPa are likely to yield highly ambiguous results under point load testing.

This table developed after ISRM, [14].

TABLE 8 PARAMETER m_i BY ROCK GROUP

Grain size	Sedimentary Calcic	Sedimentary Silica	Metamorphic Calcic	Metamorphic Acidic	Igneous Acid	Igneous Basic	Igneous Basic
Coarse	Dolomite 6.8	(Conglom.)	Marble 10.6	Gneiss 24.5	Granite 29.2	Gabbro 23.9	Norite 23.2
Medium	Limest. 5.4	Sandstone 14.3		Amphibolite 25.1		Dolerite 15.2	
Fine	(Micrite)	(Siltstone)		Quartzite 16.8	(Rhyolite)	Andesite 18.9	(Basalt)
V.Fine	(Chalk)	Mudstone 7.3		Slate 12.5	(Obsidian)		

Values shown were derived from curve fitting routines to triaxial data for each rock type. Rock names in parentheses have not yet been assessed for m_i.

Rocks with a high calcite content have lower m_i-values than corresponding rocks with a high silica content, and coarse-grained polymineral rocks (including foliated metamorphic gneisses) have similar values of m_i regardless of exact mineralogy.

The use of partial classification parameters RMRm or J_r/J_a is recommended in establishing a value for m_b/m_i in accordance with design charts such as those presented in Figures 1–3 or Equations 7 and 10. A design value for the broken rock parameter m_b can then be found by multiplying m_i from Table 8 by m_b/m_i.

A design value for s can be derived in a similar way using RMRs or RQD/J_n and Figures 1–3 or Equations 8 and 11.

The following illustrates the determination of rock mass strength parameters for a blocky sandstone rock mass. It is described in engineering geological terms as slightly weathered, moderately widely bedded, pale gray, fine to medium grained, moderately strong sandstone with two orthogonal sets of joints creating tight blocks 0.1 to 0.2 m across; surfaces are planar and rough. No laboratory tests have been carried out, so Tables 7 and 8 are used to determine $\sigma_c = 75$ MPa and $m_i = 14.3$.

Using the geomechanics classification, RMRm and RMRs can be found by reference to Tables 3 and 5.

Characteristic	*Value*	*Rating*
Discontinuity length	1–3 m	4
Separation	None	6
Roughness	Rough	5
Infilling	None	6
Weathering	Slight	5
Total RMRm		26

Equation 7 gives $m_b/m_i = \exp(\text{RMRm} - 40/5)$, from which, $m_b/m_i = 0.061$, or $m_b = 0.87$.

RQD, as defined by Deere (*12*), is a measure of jointing in rock core. To estimate a value of RQD from surface mapping, a relationship first proposed by Palmström in the paper by Barton et al. (*15*) is often used.

$$\text{RQD} = 115 - 3.3J_v \tag{12}$$

where J_v is the joint volume and is the sum of the number of discontinuities per cubic meter of rock. In this case, with bedding at 60 to 200 mm, and two sets of jointing at 100 to 200 mm, it may be expected that there would be about 23 discontinuities/m^3, giving an RQD of 39 percent. Table 5 shows an RQD rating of 8 and a spacing rating of 8, giving an RMRs value of 16.

Equation 8 gives $s = \exp[(\text{RMRs} - 45)/4.5]$, from which $s = 0.0016$.

Using the NGI classification, the first two quotients will be used to derive values of the Hoek-Brown parameters:

Parameter	*Value*	*Quotient*
RQD	39%	
J_n	Three sets—9	RQD/J_n = 4.33
J_r	Rough, planar—1.5	
J_a	Surface staining—1.0	$J_r/J_a = 1.5$

Equation 10 gives $\log_e(m_b/m_i) = 2\log_e(J_r/J_a) - 3.35$, from which $m_b/m_i = 0.079$, or $m_b = 1.13$. Equation 11 gives $\log_e s = 2\log_e(\text{RQD}/J_n) - 9.2$, from which $s = 0.0019$.

It can be seen that the two methods give comparable results. However, one of the classification methods may be easier to derive on a particular project and more confidence may be obtained in the output. It is suggested that both methods be attempted and that the final Hoek-Brown parameter set be selected depending on the confidence level of the data set. Although this example used Equations 7, 8, 10, and 11, the design charts given in Figures 1–3 may also be used. It should be remembered that these rock mass strength values are appropriate for design in rock slope engineering. Further modifications may be required to extrapolate these values to underground excavations in rock.

CONCLUSIONS AND RECOMMENDATIONS

The rock mass classifications proposed by Barton et al. (*15*) and Bieniawski (*1–6*) can be used to estimate the rock mass strength parameters proposed by Hoek and Brown (*7–9*). However, it is recommended that only partial ratings be used because the complete index or rating comprises characteristics associated with engineering design in addition to the generic rock mass features on which rock mass strength is dependent. The possibilities of introducing errors in the use of empirical relationships should be borne in mind, especially in attempts to relate different parameters derived for different purposes. In the context of this paper, it appears that both Barton et al.'s tunneling quality index quotient J_r/J_a, and Bieniawski's RMRm can be used to estimate a value of the rock mass strength parameter m_b, although most published work has concentrated on a relationship between RMR and the Hoek-Brown parameters. The data base is too small to compare the correlations between the two partial classifications and s.

The limitations that exist in classification methods for tunnel support design should be considered in attempts to estimate rock mass strength parameters, as should the limitations in rock mass conditions under which the Hoek-Brown failure criterion itself is considered valid. Figure 4 shows different scales of rock mass geometries relative to the size of a design excavation for which the Hoek-Brown criterion is considered valid. Scale factors associated with size of the prototype design excavation to the host rock mass geometry or block size should be considered to ensure that the Hoek-Brown parameters estimated are to be used in a valid constitutive model in which truly jointed rock mass conditions prevail.

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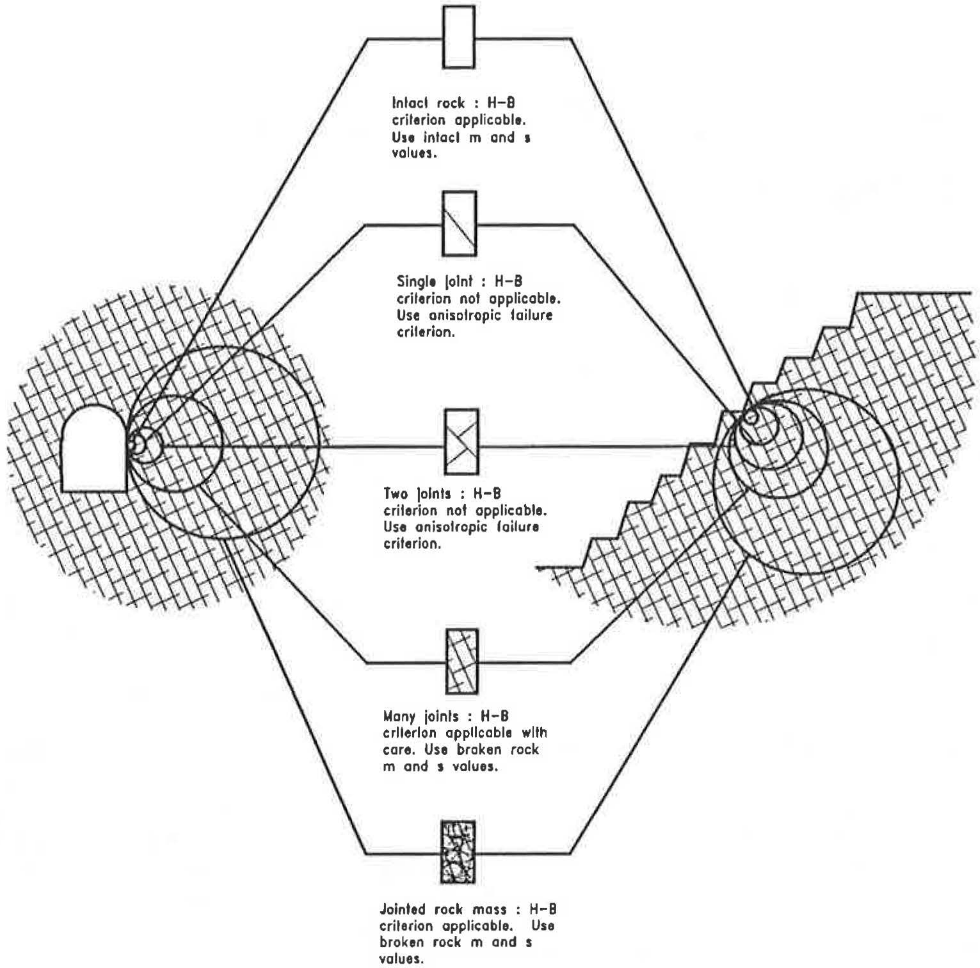

FIGURE 4 Applicability of Hoek-Brown failure criterion.

fundamental research activities supported in this program are concerned with the development of methods for estimating the strength and deformation characteristics of rock masses. The stimulating environment created by the research group involved is highly appreciated.

REFERENCES

1. Z. T. Bieniawski. Estimating the Strength of Rock Materials. *Journal of South African Institute Mining and Metallurgy*, Vol. 74, No. 8, 1974, pp. 312–320.
2. Z. T. Bieniawski. Engineering Classification of Jointed Rock Masses. *Transactions of the South African Institution of Civil Engineers*, Vol. 15, 1973, pp. 335–344.
3. Z. T. Bieniawski. Case Studies: Prediction of Rock Mass Behavior by the Geomechanics Classification. *Proc., 2nd Australia–New Zealand Conference on Geomechanics*, Brisbane, Australia, 1975, pp. 36–41.
4. Z. T. Bieniawski. Rock Mass Classifications in Rock Engineering. In *Exploration for Rock Engineering*. Z. T. Bieniawski, ed., A. A. Balkema, Johannesburg, South Africa, 1976, pp. 97–106.
5. Z. T. Bieniawski. The Geomechanics Classification in Rock Engineering Applications. *Proc., 4th International Congress on Rock Mechanics*, International Society for Rock Mechanics, Montreaux, Switzerland, Vol. 2, 1979, pp. 41–48.
6. Z. T. Bieniawski. *Engineering Rock Mass Classifications*. John Wiley & Sons, New York, N.Y., 1989.
7. E. Hoek. Strength of Jointed Rock Masses. *Geotechnique*, Vol. 33, No. 3, 1983, pp. 187–223.
8. E. Hoek and E. T. Brown. *Underground Excavations in Rock*. Institute of Mining and Metallurgy, London, England.
9. E. Hoek and E. T. Brown. Empirical Strength Criterion for Rock Masses. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 106, No. GT9, 1980, pp. 1,013–1,035.
10. S. D. Priest and E. T. Brown. Probabilistic Stability Analysis of Variable Rock Slopes. *Transactions of the Institution of Mining and Metallurgy, Section A*, Vol. 92, 1983, pp. A1–A12.
11. E. Hoek and E. T. Brown. The Hoek-Brown Failure Criterion—A 1988 Update. *Proc., 15th Canadian Rock Mechanics Symposium*, University of Toronto, 1988, pp. 31–38.
12. D. U. Deere. Technical Description of Rock Cores for Engineering Purposes. *Rock Mechanics and Engineering Geology*, Vol. 1, No. 1, 1963, pp. 16–22.
13. J. C. Jaeger. Behavior of Closely Jointed Rock. *Proc., 11th Symposium on Rock Mechanics*, Berkeley, Calif., 1970, pp. 57–68.
14. International Society for Rock Mechanics. *Rock Characterization, Testing and Monitoring—ISRM Suggested Methods*. Pergamon, London, England, 1981.
15. N. R. Barton, J. Lunde, and R. Lien. Engineering Classification of Rock Masses for the Design of Tunnel Support. *Rock Mechanics*, Vol. 6, No. 4, 1974, pp. 189–236.
16. R. Jackson, J. S. O. Lau, and A. Annor. Mechanical, Thermo-Mechanical and Joint Properties of Rock Samples from the Site of AECL's Underground Research Laboratory, Lac du Bonnet, Manitoba. *Proc., 42nd Canadian Geotechnics Conference*, Winnipeg, Manitoba, 1989, pp. 41–49.

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