From Palmström A.: RMi – a rock mass characterization system for rock engineering purposes. PhD thesis, Oslo University, Norway, 1995, 400 p.

Chapter 8

POSSIBLE OTHER APPLICATIONS OF THE RMi IN ROCK MECHANICS AND ROCK ENGINEERING

> *"The responsibility of the design engineer is not to compute accurately but to judge soundly."*
> Evert Hoek and Pierre Londe (1974)

The Rock Mass index, RMi, is different from earlier *general* classifications of rock masses as it is more numerical. This is a prerequisite for applications in rock mechanics, rock engineering and design.

RMi can either be applied directly in the engineering as the main input, or only as part of the input of the ground composition. In other cases it is more appropriate to apply some of the parameters used in RMi, for example the block volume (Vb), the joint condition factor (jC), or the jointing parameter (JP).

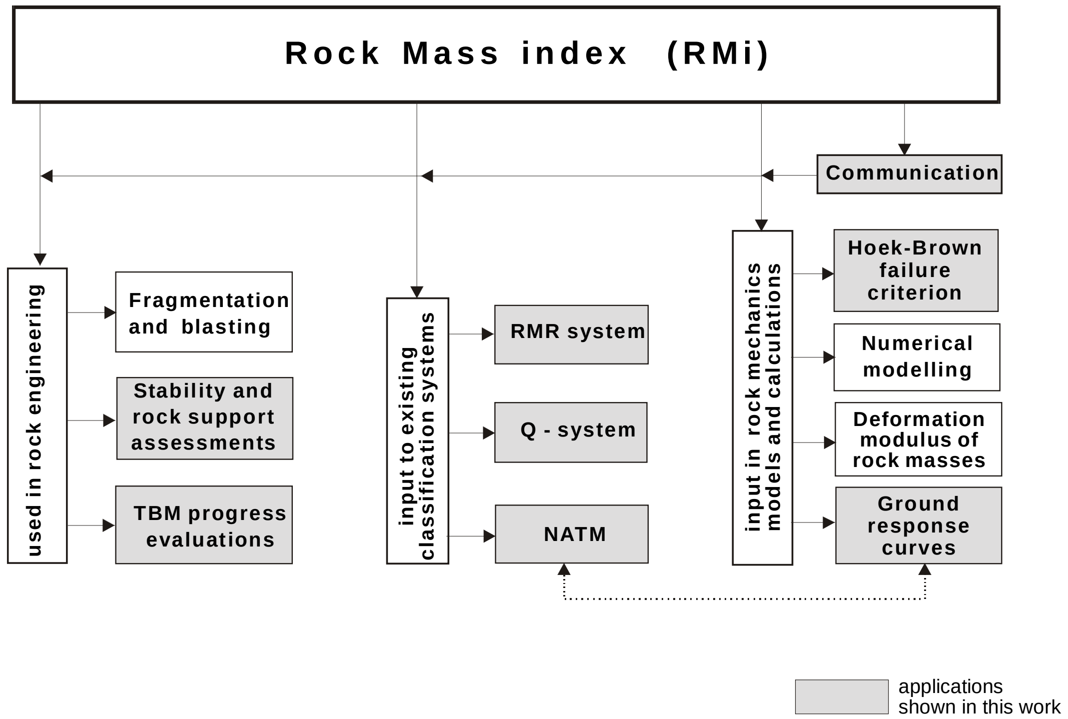


Fig. 8-1 Various applications of RMi or of the parameters included in the RMi

Some practical applications of RMi are shown in Chapters 6 and 7 for rock support assessment in underground excavations and TBM boring penetration estimates. This chapter outlines some of the possibilities in applying RMi in various types of calculations applied in rock mechanics and rock engineering.

8.1 APPLYING RMi TO DETERMINE THE CONSTANTS IN THE HOEK-BROWN FAILURE CRITERION

The Hoek-Brown failure criterion provides engineers and geologists with a means of estimating the strength of jointed rock masses. After the criterion was presented in 1980, the ratings of its constants have been adjusted in 1988, 1991 and 1992. A modified failure criterion was published by Hoek et al. (1992) as is outlined later in this section.

8.1.1 The original Hoek-Brown failure criterion

In its original form the Hoek-Brown criterion is expressed in terms of the major and the minor principal stresses at failure as

$$\sigma_1' = \sigma_3' + (m \times \sigma_c \times \sigma_3' + s \times \sigma_c^2)^{½} \qquad \text{eq. (8-1)}$$

where σ_1' is the major principal effective stress at failure.
σ_3' is the minor principal effective stress.
σ_c is the uniaxial compressive strength of the intact rock material from which the rock mass is composed.
s and m are empirical constants representing inherent properties of jointing conditions and rock characteristics.

For $\sigma_3' = 0$, eq. (8-1) expresses the unconfined *compressive strength* of a rock mass:

$$\sigma_{cm} = \sigma_c \times s^{½} \qquad \text{eq. (8-2)}$$

According to Hoek and Brown (1980) the constants m and s depend on the properties of the rock and the extent to which it has been broken before being subjected to the [failure] stresses. Both constants are dimensionless. Hoek (1983) explains that they are *"very approximately analogous to the angle of friction, Φ_i', and the cohesive strength, c', of the conventional Mohr-Coulomb failure criterion"*.

To determine m and s Hoek and Brown (1980) adapted the classifications of Bieniawski (1973) and of Barton et al. (1974). This is shown in Table 8-1. As the structure of RMi is similar to eq. (8-2) which expresses the uniaxial compressive strength for rock masses, RMi offers a method to determine the constants m and especially s, as described in the following.

8.1.1.1 The constant *s*

As described in Section 4.5.2 in Chapter 4, the jointing parameter (JP) is similar to s, though the understanding is somewhat different regarding the features in a rock mass each of them represents. From eq. (8-2) and the expression RMi = $\sigma_c \times$ JP, it is found that

$$\text{JP} = \sqrt{s} \qquad \text{eq. (8-3)}$$

JP can be found directly from the registration of block size (Vb) and joint condition factor (jC), while s is determined via values found by Q or RMR in Table 8-1. As these classification systems also include external features such as ground water and stresses, they do not in the best way characterize the mechanical properties of a rock mass. Another drawback is that they both apply

RQD, which in Appendix 4, Section 6 has been shown to often poorly represent the variation in jointing.

TABLE 8-1 THE CONSTANTS *s* AND *m* FOR UNDISTURBED AND DISTURBED ROCK MASSES VARYING WITH THE ROCK TYPE AND THE COMPOSITION OF THE ROCK MASS (from Hoek and Brown, 1988).

Approximate relationship between rock mass quality and material constants

Disturbed rock mass m and s values — *undisturbed rock mass m and s*

EMPIRICAL FAILURE CRITERION $\sigma'_1 = \sigma'_3 + \sqrt{m\sigma_c\sigma'_3 + s\sigma_c^2}$ σ'_1 = major principal effective stress σ'_3 = minor principal effective stress σ_c = uniaxial compressive strength of intact rock, and m and s are empirical 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 CRYSTALLINE ROCKS – amphibolite, gabbro, gneiss, granite, norite, quartz-diorite
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities CSIR rating: RMR = 100 NGI rating: Q = 500	m s *m* *s*	7.00 1.00 *7.00* *1.00*	10.00 1.00 *10.00* *1.00*	15.00 1.00 *15.00* *1.00*	17.00 1.00 *17.00* *1.00*	25.00 1.00 *25.00* *1.00*
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 1 to 3 m CSIR rating: RMR = 85 NGI rating: Q = 100	m s *m* *s*	2.40 0.082 *4.10* *0.189*	3.43 0.082 *5.85* *0.189*	5.14 0.082 *8.78* *0.189*	5.82 0.082 *9.95* *0.189*	8.56 0.082 *14.63* *0.189*
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3 m CSIR rating: RMR = 65 NGI rating: Q = 10	m s *m* *s*	0.575 0.00293 *2.006* *0.0205*	0.821 0.00293 *2.865* *0.0205*	1.231 0.00293 *4.298* *0.0205*	1.395 0.00293 *4.871* *0.0205*	2.052 0.00293 *7.163* *0.0205*
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 0.3 to 1 m CSIR rating: RMR = 44 NGI rating: Q = 1	m s *m* *s*	0.128 0.00009 *0.947* *0.00198*	0.183 0.00009 *1.353* *0.00198*	0.275 0.00009 *2.030* *0.00198*	0.311 0.00009 *2.301* *0.00198*	0.458 0.00009 *3.383* *0.00198*
POOR QUALITY ROCK MASS Numerous weathered joints at 30-500 mm, some gouge Clean compacted waste rock CSIR rating: RMR = 23 NGI rating: Q = 0.1	m s *m* *s*	0.029 0.000003 *0.447* *0.00019*	0.041 0.000003 *0.639* *0.00019*	0.061 0.000003 *0.959* *0.00019*	0.069 0.000003 *1.087* *0.00019*	0.102 0.000003 *1.598* *0.00019*
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced >50 mm with gouge. Waste rock with fines CSIR rating: RMR = 3 NGI rating: Q = 0.01	m s *m* *s*	0.007 0.0000001 *0.219* *0.00002*	0.010 0.0000001 *0.313* *0.00002*	0.015 0.0000001 *0.469* *0.00002*	0.017 0.0000001 *0.532* *0.00002*	0.025 0.0000001 *0.782* *0.00002*

Hoek and Brown worked out their failure criterion mainly from triaxial test data on intact rock specimens. For jointed rock masses they had very few triaxial test data, in fact only those made on the Panguna andesite. Therefore, the values of *s* given by Hoek and Brown for the various jointed rock masses, are very approximate. The jointing parameter (JP) is based on measured strength in 8 "samples" of rock masses. By applying the defined parameters block volume (Vb) and jointing parameter (JP) in RMi, the accuracy of the parameter *s* in Hoek Brown failure criterion can be considerably improved.

8.1.1.2 The constant *m*

In addition to adjustments in the ratings of the constant *m*, Wood (1991) and Hoek et al. (1992) have introduced the ratio m_b / m_i , where m_i represents intact rock as given in Table A3-8 (in Appendix 3). The constant m_b is the same as *m* in the original criterion. It varies with the jointing Based on the variation of *m* in Table 8-1 and of its ratings for disturbed and undisturbed values of *m* in Wood (1991), Fig. 8-2 has been worked out.

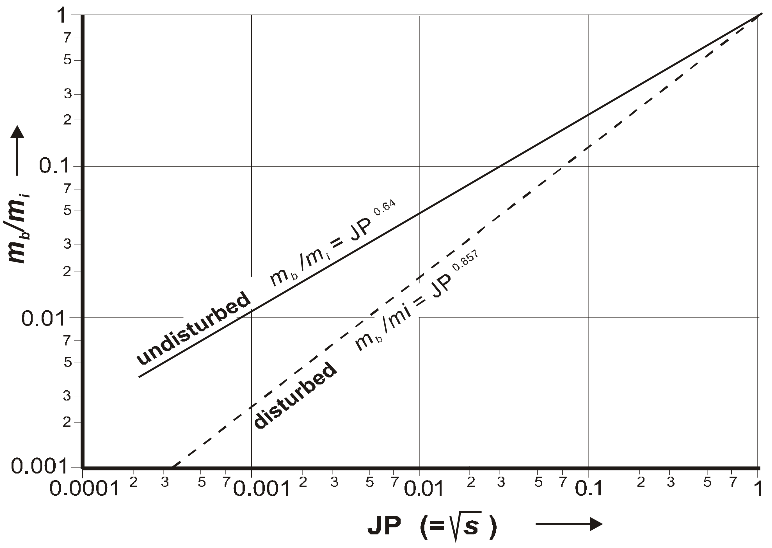


Fig. 8-2 The variation of m_b/m_i with the jointing parameter (JP), based on the data in Table 8-1 and Wood (1991).

As shown in Fig. 8-1, the variation of m_b can be mathematically expressed as:

a) for undisturbed rock masses

$$m_b = m_i \times \mathrm{JP}^{0.64} \qquad \text{eq. (8-4)}$$

b) for disturbed rock masses

$$m_b = m_i \times \mathrm{JP}^{0.857} \qquad \text{eq. (8-5)}$$

Applying eqs. (8 - 3) and (8-4) in eq. (8-1), the failure criterion can be written as

$$\sigma_1' = \sigma_3' + [\sigma_c \times \mathrm{JP}^{0.64} (m_i \times \sigma_3' + \sigma_c \times \mathrm{JP}^{1.36})]^{½} \qquad \text{eq. (8-6)}$$

where *s* and *m* are replaced by JP and m_i

8.1.2 The modified Hoek-Brown failure criterion

From more than 10 years of experience in using the Hoek-Brown criterion, Hoek et al. (1992) found a need to modify the criterion to the following form:

$$\sigma_1' = \sigma_3' + \sigma_c (\mathrm{m_b} \frac{\sigma_3'}{\sigma_c})^{a} \qquad \text{eq. (8-7)}$$

where $\mathrm{m_b}$ and *a* are constants which depend on the composition, structure and surface of the jointed rock mass.

m_b is found from the ratio m_b/m_i in Table 8-3. m_b/m_i varies between 0.001 in crushed rock masses with highly weathered, very smooth or filled joints to 0.7 in blocky rock masses with rough joints. In massive rock $m_b/m_i = 1$. The value of *a* varies between 0.3 and 0.65. It has its highest value for the crushed rock masses with altered, smooth joints and lowest for massive rock masses.

The value of *a* varies between 0.3 and 0.65. It has its highest value for the crushed rock masses with altered, smooth joints and lowest for massive rock masses, as shown by Hoek et al. (1992) in Fig. 8-3.

MODIFIED HOEK-BROWN FAILURE CRITERION

$$\sigma'_1 = \sigma'_3 + \sigma_c \left(m_b \frac{\sigma'_3}{\sigma_c} \right)^a$$

σ'_1 = major principal effective stress at failure
σ'_3 = minor principal effective stress at failure
σ_c = uniaxial compressive strength of *intact* pieces in the rock mass

m_b and *a* are constants which depend on the condition of the rock mass

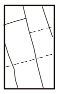
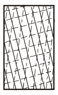
STRUCTURE		SURFACE CONDITION	VERY GOOD Unweathered, discontinuous, very tight aperture, very rough surface, no infilling	GOOD Slightly weathered, continuous, tight aperture, rough surface, iron staining to no infilling	FAIR Moderatelyweathered, continuous, extremely narrow, smooth surfaces, hard infilling	POOR Highly weathered, continuous, very narrow, polished / slickensided surfaces, hard infilling	VERY POOR Highly weathered, continuous, narrow polished / slickensided surfaces, soft infilling
	BLOCKY - well interlocked, undisturbed rock mass; large to very large block size	m_b/m_i a	0.7 0.3	0.5 0.35	0.3 0.4	0.1 0.45	
	VERY BLOCKY - interlocked, partially disturbed rock mass; medium block size	m_b/m_i a	0.3 0.4	0.2 0.45	0.1 0.5	0.04 0.5	
	BLOCKY / SEAMY - folded and faulted, many intersecting joints; small blocks	m_b/m_i a		0.08 0.5	0.04 0.5	0.01 0.55	0.04 0.6
	CRUSHED - poorly interlocked, highly broken rock mass; very small blocks	m_b/m_i a		0.03 0.5	0.015 0.55	0.003 0.6	0.001 0.65

Fig. 8-3 Estimation of m_b/m_i and *a* based on the degree of jointing (block size) and joint characteristics (from Hoek et al., 1992).

The exponent *a* may partly be compared with the factor D in the expression for RMi in eq. (4-4) which varies between 0.2 and 0.6. D has its highest values for smooth, or altered joints large joints, and lowest values for rough, small joints, see Section 4.2 in Chapter 4. The exponent D does not, however, include the block volume (Vb) as is the case for *a*, as Vb has been included directly in eq. (4-4) to determine the jointing parameter (JP).

8.2 RMi USED TO EVALUATE THE SHEAR STRENGTH OF ROCK MASSES

In the paper by Hoek (1983) the empirical failure criterion has been derived by Dr. J. Bray into the failure envelope given by

$$\tau = (\mathrm{Cot}\Phi_i' - \mathrm{Cos}\Phi_i')\,(m \times \sigma_c / 8) \qquad \text{eq.(8-8)}$$

where τ is the shear stress at failure, and
Φ_i' is the instantaneous friction angle.

The value of the instantaneous friction angle is given by

$$\Phi_i' = \text{Arctan}\,[4h\text{Cos}^2(\pi/6 + \text{Arcsin}\,h^{-3/2}) - 1]^{-1/2} \qquad \text{eq. (8-9)}$$

where σ' = the effective stress

$$h = 1 + 16(m \times \sigma' + s \times \sigma_c)\,/3m^2\sigma_c \qquad \text{eq. (8-10)}$$

$$m = m_i \times \text{JP}^{0.64} \quad \text{(for undisturbed rock masses)} \qquad \text{eq. (8-11)}$$

$$s = \text{JP}^2 \qquad \text{eq. (8-12)}$$

The instantaneous cohesive strength is found as

$$c_i' = \tau - \sigma'\,\text{Tan}\Phi_i' \qquad \text{eq. (8-13)}$$

TABLE 8-2 COMPUTER SPREADSHEAT USED TO CALCULATE THE CONSTANTS s AND m, THE SHEAR STRESS (τ), THE INSTANTANEOUS FRICTION ANGLE (Φ_i'), AND THE COHESIVE STRENGTH (c_i') FROM INPUT OF RMi PARAMETERS.

INPUT DATA				example 1	example 2	example 3
ROCK CHARACTERISTICS		**Type of rock**	**=**	limestone	granite	gneiss
Rock compressive strength		(MPa)	σ_c	50,00	160,0	130,0
H & B's m - factor for intact rock	Table A3-8		m_i	8,40	32,7	29,2
JOINT CHARACTERISTICS						
Joint smoothness factor	Table 4-2		jS	2,00	3,0	1,0
Joint waviness factor	Table 4-3		jw	3,00	1,0	2,0
Joint alteration factor	Table 4-5		jA	2,00	2,0	3,0
Joint length and continuity factor	Table 4-7		jL	3,00	1,0	2,0
JOINTING DENSITY MEASUREMENTS						
Alt. 1: measured joint spacings						
Main joint set (min. spacing)		(m)	S1	0,30		
Joint set 2		(m)	S2	0,50		
Joint set 3 (max. spacing)		(m)	S3	0,50		
Alt. 2: measured block volume		(m^3)	Vb			0,01
Assumed block shape factor	(Fig. A3-31)		β			
Alt. 3: RQD measurement			RQD		45	
STRESSES						
Effective normal stress		(MPa)	σ_n'	0,10	1,0	10,0
CALCULATIONS						
RMi PARAMETERS						
Volumetric joint count	eq. (A3-22)		Jv	7,33		
Joint condition factor	eq. (4-2)		jC	9,00	1,50	1,33
Block volume, eq. (A3-19) or	eq. (A3-27)	(m^3)	Vb	0,0750	0,0036	0,0100
Block shape factor	eq. (A3-28)		β	29,58		
Jointing parameter	eq. (4-4)		JP	0,3235	0,0359	0,0462
Rock Mass index	eq. (4-1)		RMi	16,18	5,74	6,01
HOEK - BROWN PARAMETERS						
s - value (= JP^2)	eq. (8-11)		s	0,1047	0,0013	0,0021
m - value	eq. (8-10)		m	4,08	3,89	4,08
Calculation factor	eq. (8-9)		h	1,0362	1,0090	1,1012
SHEAR STRENGTH PARAMETERS						
Instantaneous friction angle	eq. (8-8)	(degree)	ϕ	56,32	65,61	47,81
Shear stress	eq. (8-7)	(MPa)	τ	2,85	3,15	15,58
Instantaneous cohesion	eq. (8-12)	(MPa)	c	2,70	0,94	4,55

Though the expression in eq. (8-8) seems complex, it can easily be applied using a spreadsheet on a desk computer. Table 8-2 shows an example where eqs. (8-8) to (8-13) have been applied in an Excel spreadsheet.

It should be born in mind that the Hoek-Brown failure criterion is only valid for continuous rock masses (Hoek and Brown, 1980), i.e. massive rock or highly jointed and crushed rock masses, as is outlined in Chapter 5, Section 5.1 and in Chapter 6, Section 6.4.

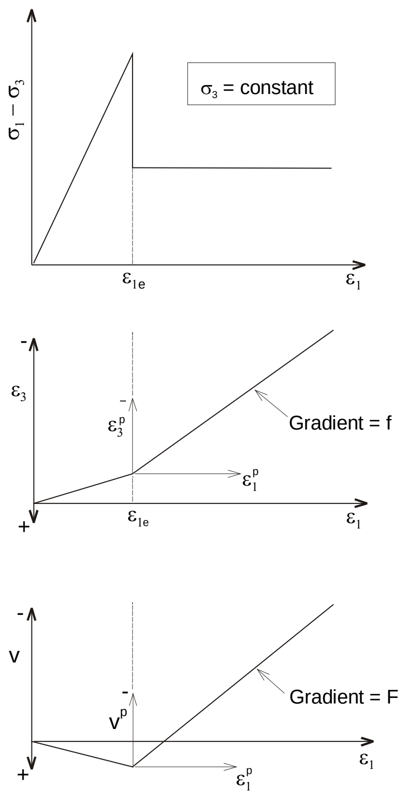
8.3 RMi USED IN THE INPUT TO GROUND RESPONSE CURVES

> *"However, in our field, theoretical reasoning alone does not suffice to solve the problems which we are called upon to tackle. As a matter of fact it can even be misleading unless every drop of it is diluted by a pint of intelligently digested experience."*
> Karl Terzaghi (1953)

Ground-response interaction diagrams are well established aids to the understanding of rock mass behaviour and tunnel support mechanics. They are limited to continuous materials, i.e. massive rock or highly jointed and crushed (particulate) rock masses (see Chapter 5, Section 5.1). According to several authors (Rabcewicz, 1964; Ward, 1978; Muir Wood, 1979; Hoek and Brown, 1980; Brown et al. 1983) they may also be used quantitatively in designing tunnel support. For this use it is essential to be able, from the field observations and assessment of the stresses and moduli, to predict the ground response curve for a particular rock mass, stress regime, and tunnel geometry.

Many approaches to the calculation of ground response curves have been reported in the literature. Most use closed-form solutions to problems involving simple tunnel geometry and hydrostatic in-situ stresses, but some use numerical methods for more complex excavation geometries and stress fields. However, with improved knowledge of the engineering behaviour of rock masses and the use of desk computers it is now possible to incorporate more complex and realistic models of rock mass behaviour into the solutions.

Two solutions of the ground-support interaction diagrams using a simple axisymmetric tunnel problem were presented by Brown et al. (1983). Both analyses incorporate the Hoek- Brown failure criterion for rock masses. The material behaviour applied in the closed-form solution is shown in Fig. 8-4.



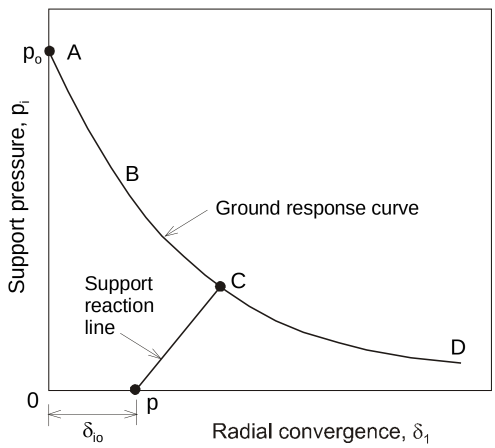


Fig. 8-4 Left: The material behaviour used by Brown et. al.(1983) in the closed-form solution.
Right: The ground response curve.

The input data used in the closed-form solution are:

- r_i the internal tunnel radius
- σ_c the compressive strength of intact rock
- p_o the in situ hydrostatic rock stress
- f the gradient of line in the $-\varepsilon_3^p$, ε_1^p diagram (Fig. 8-4)

Data for *original non-disturbed* rock mass:

- m and s are material constants in the Hoek-Brown failure criterion
- E and ν are Young's modulus and Poisson's ratio

Data for *broken* rock mass in the 'plastic zone':

- m_r and s_r are material constants in the Hoek-Brown failure criterion.

The following calculation sequence is given by Brown et al. (1983):

1. $M = ½ [(m/4)^2 + (mp_o/\sigma_c) + s]^{½} - m/8.$
2. $G = E/[2(1 + \nu)].$
3. For $p_i \geq p_o - M\sigma_c$, deformation around the tunnel is elastic: $\delta_i/r_i = (p_o - p_i /2G.$
4. For $p_i < p_o - M\sigma_c$, plastic deformation occurs around the tunnel: $u_e /r_e = M\sigma_c /2G.$
5. $N = 2\{[(p_o - M\sigma_c)/m_r \sigma_c] + s_r/m_r^2)\}^{½}.$
6. $r_e /r_i = exp\{N - 2[pi/m_r \sigma_c) + (s_r/m_r^2)]^{½}\}.$
7. $\delta_i/r_i = M\sigma_c /[G(f + 1)]\{[(f - 1)/2] + (r_e /r_i)^{f+1}\}.$

Brown et al. (1983) indicate that where appropriate for a given rock mass, the constant f can, in place of an experimentally determined or back-calculated value, be calculated from

$$f = 1 + F \qquad \text{eq. (8-13)}$$

$$\text{where } F = \frac{m}{2(m\frac{\sigma_{re}}{\sigma_c} + s)^{1/2}} \qquad \text{eq. (8-14)}$$

$$\text{and } \sigma_{re} = p_o - M \times \sigma_c \qquad \text{eq. (8-15)}$$

$s = JP^2$ can be found from eq. 4-4 or from Fig. 4-4 based on field characterization of the block size (Vb) and joint condition (jC) as described in Chapter 4, while m can be found from Table A3-8 and Fig. 8-2.

For the broken, (plastic) zone the corresponding s_r and m_r values have to be estimated from experience. It is known that the rock mass breaks up during the deformation (squeezing) process, which is gradually reduced towards the boundary between the plastic and elastic zone. Applying the 'common' joint condition (joint condition factor jC = 1.75) for the new breaks, eq. 4-5 ($JP = 0.25\ Vb^{1/3}$) can be applied to find

$$s_r = JP^2 = 0.06\ Vb^{2/3} \qquad \text{eq. (8-16)}$$

The calculations can be readily carried out using a desk computer. If the actual case is not axisymmetric, because the tunnel cross section is not circular or the in situ stress field is not hydrostatic, it will be necessary to use numerical method to calculate the stresses, strains and displacements in the rock masses surrounding the tunnel.

Another method of finding the ground response curve has been shown by Hoek and Brown (1980), where also data to determine reaction from the support is given.

8.4 RMi USED FOR NUMERICAL GROUND CHARACTERIZATION IN THE NATM

The principles of the new Austrian tunnelling method (NATM) is outlined in Fig. 8-5.

–Key features of the New Austrian Tunnelling Method –

Professor Rabcevicz, of Salzburg, Austria, has been one of the chief developers of the *New Austrian Tunnelling Method. Goal of NATM*: to provide safe and economic support in tunnels excavated in materials incapable of supporting themselves – e.g. crushed rock, debris, even soil. Support is achieved by mobilizing whatever humble strength the rock or earth possesses.

The *New Austrian Tunnelling Method* has several features:

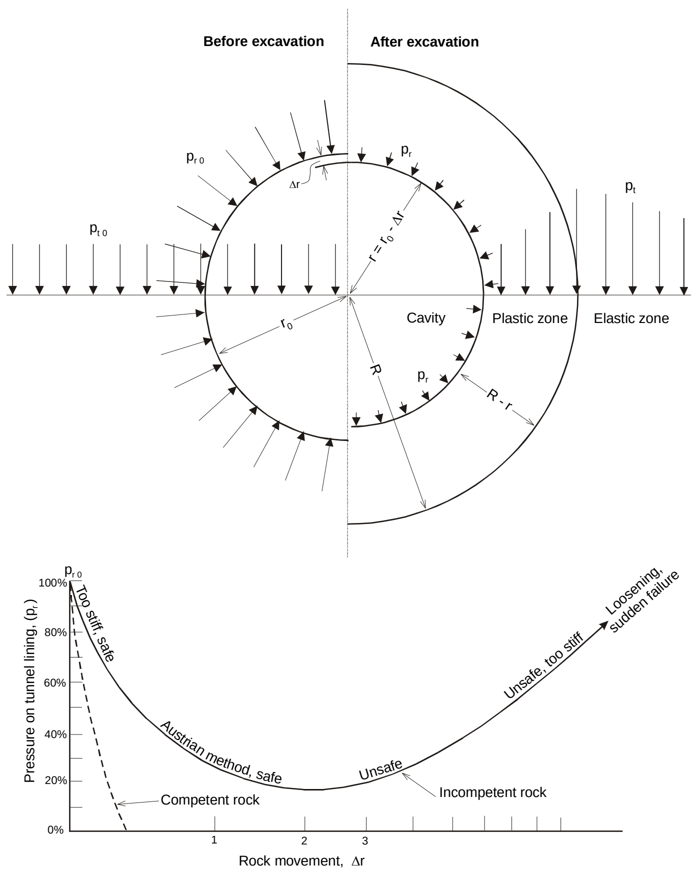
- It relies on strength of surrounding rock to provide tunnel support. This is done by inhibiting rock deterioration, joint opening, and loosening due to excessive rock movements.
- It uses protective measures like lining tunnel walls with shotcrete and driving anchors into unstable rock. In many cases, a second, inner linings not needed – e.g. for water conduits, short highway tunnels.
- It involves installation of sophisticated instrumentation at the time initial shotcrete lining is placed, to provide info for designing a second inner lining.
- It completely eliminates costly interior supports for tunnel walls, such as heavy steel arches.

Tunneling into hard or inferior rock disturbs the existing equilibrium of forces. A rearrangement of stresses within the rock surrounding the cavity follows (See Fig.). As time progresses, the freshly excavated tunnel radius r_o decreases to (r_o – Δr). If tunneling in competent rock, further deformation is followed by final collapse – unless resisted in time by a lining.

The tunnel lining must be neither too stiff nor too flexible. If stiff, p_r will remain unnecessarily large – the lining will be uneconomic. With increased Δr, however, the pressure p_r decreases. With a lining allowing too much yield, p_r will be big and the lining uneconomic and unsafe.

NATM aims *at a temporary semi-rigid lining* stressed by a moderate rock load p_r that will be just above the theoretical minimum value.

(Our thanks to Mr. Herbert Nussbaum, an engineering consultant and expert on tunneling from Weat Vancouver, B.C., Canada. Based on phone interviews with him, we were able to write this box and captions for all figures – GD).



When surrounding rock moves into tunnel cavity, stresses pr at cavity-rock interface decrease dramatically, making possible the use of tunnel linings that are much less thick, much less costly. But since total weight of mountain is constant, stresses in rock must increase elsewhere to carry this constant load. Note stresses p_t increase to max at a short distance from cavity.

Fig. 8-5 The main ideas and principles of NATM (from Rabcewicz, 1975).

Brosch (1986) recommends that *"informative geological parameters lending themselves to quantification be used for describing rock mass in future tunnel projects in Austria. This calls for* *characterization based on verifiable parameters to provide numerical geo-data for rock engineering and design to be used in rock construction".*

From this statement it is obvious that RMi offers an excellent opportunity to improve the input parameters used in design works of NATM projects.

NATM has its own classification, mainly based on the behaviour in the excavated tunnel. The various classes can also be assessed from field observations of the rock mass composition and assessment of the rock stresses. There does not seem to exist any numerical system for classifying the important parameters of the rock mass. The ground is mainly characterized on an individual basis, based on personal experience (Kleeberger, 1992).

TABLE 8-3 THE CLASSIFICATION OF GROUND BEHAVIOUR APPLIED IN ÖNORM B 2203

NATM class	ROCK MASS BEHAVIOUR
1 Stable	Elastic behaviour. Small, quick declining deformations. No relief features after scaling. The rock masses are long-term stable.
2 Slightly ravelling	Elastic behaviour, with small deformations which quickly decline. Some few small structural relief surfaces from gravity occur in the roof.
3 Ravelling	Far-reaching elastic behaviour. Small deformations that quickly decrease. Jointing causes reduced rock mass strength, as well as limited stand-up time and active span*) . This results in relief and loosening along joints and weakness planes, mainly in the roof and upper part of walls.
4 Strongly ravelling	Deep, non-elastic zone of rock mass. The deformations will be small and quickly reduced when the rock support is quickly installed. Low strength of rock mass results in possible loosening effects to considerable depth followed by gravity loads. Stand-up time and active span are small with increasing danger for quick and deep loosing from roof and working face.
5 Squeezing or swelling	"Plastic" zone of considerable size with detrimental structural defects such as joints, seams, shears. Plastic squeezing as well as rock spalling (rock burst) phenomena. Moderate, but clear time-dependent squeezing with only slow reduction of deformations (except for rock burst). The total and rate of displacements of the opening surface is moderate. The rock support can sometimes be overloaded.
6 Strongly squeezing or swelling	Development of a deep squeezing zone with severe inwards movement and slow decrease of the large deformations. Rock support can often be overloaded.

*) Active span is the width of the tunnel or the distance from support to face in case this is less than the width of the tunnel

The NATM uses the Fenner-Pacher diagram, which is similar to the ground reaction curve, for calculation of the ground behaviour and rock support determination. A comparison between terms applied in NATM and by Terzaghi is presented in Table 6-1 in Chapter 6.

8.4.1 The use of RMi in NATM classification

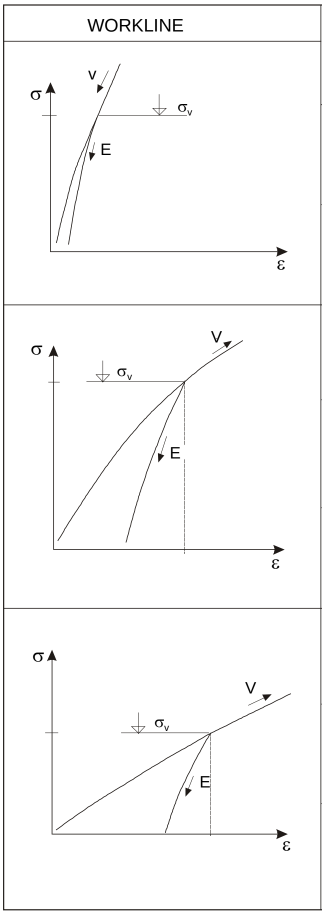
Seeber et al. (1978) have made an interesting contribution to quantify the behaviouristic classification in the NATM by dividing the ground into two main groups:

1. The¨'Gebirgsfestigkeitsklassen' ('rock mass strength classes') based on the shear strength properties of the rock mass.

2. The 'Gebirgsgüteklassen' ('rock mass quality classes') determined from the 'rock mass strength classes' and the rock stresses acting. These are the same classes as applied in the NATM classification in Table 8-3 (see also Table 6-1 in Chapter 6).

The first group can be compared to RMi, but the input parameters are different. Fig 8-6 shows that it is possible to use the shear strength parameters found in Section 2 to determine these data, as they consist of rock mechanics data characterized by one of the following parameters:

- friction angle of rock mass (Φ), found from eq. (8-8) using very low normal stress,
- cohesion of rock mass (c), which can be found by applying eq. (8-12), and/or
- modulus of elasticity (E) and modulus of deformation (V).



ROCK STRENGTH CLASS	E_{el} V_{el} V_{pl} (N/cm²)	ϕ (Grad)	c_{el} c_{pl} (N/cm²)
1	10,00 000 10,00 000 5,00 000	55 50 45	1000 10
2	800 000 800 000 400 000	50 45 40	800 10
3	1 000 000 500 000 250 000	45 40 35	500 10
4	800 000 400 000 200 000	40 35 30	400 10
5	600 000 300 000 150 000	35 30 25	300 10
6	450 000 150 000 75 000	35 30 25	150 10
7	300 000 100 000 50 000	30 25 20	100 10
8	150 000 50 000 25 000	25 20 15	50 10

Fig. 8-6 Rock mass strength classes ('Gebirgsfestigkeitsklassen') applied by Seeber et al. (1978)

The value of the shear strength parameters can be determined from the defined parameters in RMi as shown in Section 8.2. In this way, the NATM classes can be defined and determined also from numerical rock mass characterizations. NATM may effectively benefit from this contribution, especially when it is applied in the planning stage of tunnelling projects.

Suggested RMi parameters to characterize the various NATM classes are shown in Table 8-4. The competency factor is further described in Chapter 6, Section 4.1.

TABLE 8-4 SUGGESTED NUMERICAL DIVISION OF GROUND ACCORDING TO NATM CLASSIFICATION

NATM class	**Rock mass properties** (JP = jointing parameter)	**Competency factor** (Cg = RMi/σ_θ)
1 Stable	Massive ground (JP > approx. 0.5)	Cg > 2
2 Slightly ravelling	0.2 < JP < 0.6	Cg > 1
3 Ravelling	0.05 < JP < 0.2	Cg > 1
4 Strongly ravelling	JP < 0.05	0.7 < Cg < 2

5 Squeezing	Continuous ground *)	0.35 < Cg < 0.7
6 Strongly squeezing	Continuous ground *)	Cg < 0.35

*) Continuous ground is where CF < approx. 5 or CF > approx. 100 (CF = tunnel diam./block diam.)

8.4.2 RMi used for input to Fenner-Pacher ground response diagrams

The Fenner-Pacher curves are, as mentioned, similar to the ground response curves described in Section 8.1. These curves can, therefore, be applied also for NATM support evaluations.

The benefit in applying RMi to characterize the ground is that the curves can then be based on appropriate numerical strength parameters. As RMi can be estimated from simple pre-investigations, the curves can be worked out at an early phase of the project.

By combining the rock mass strength classes ('Gebirgsfestigkeitsklassen') in Fig. 8-6 with rock stresses from overburden Seeber et al. (1978) have worked out characteristic ground response curves for the 8 typical rock behaviour classes in the NATM, as shown for class 3 - 7 in Fig. 8-7. These curves can be applied for the purpose of dimensioning or controlling rock support. They enable, theoretically in a simple manner, to assess the effect of bolt length and also to find the connection between deformation and load on rock support.

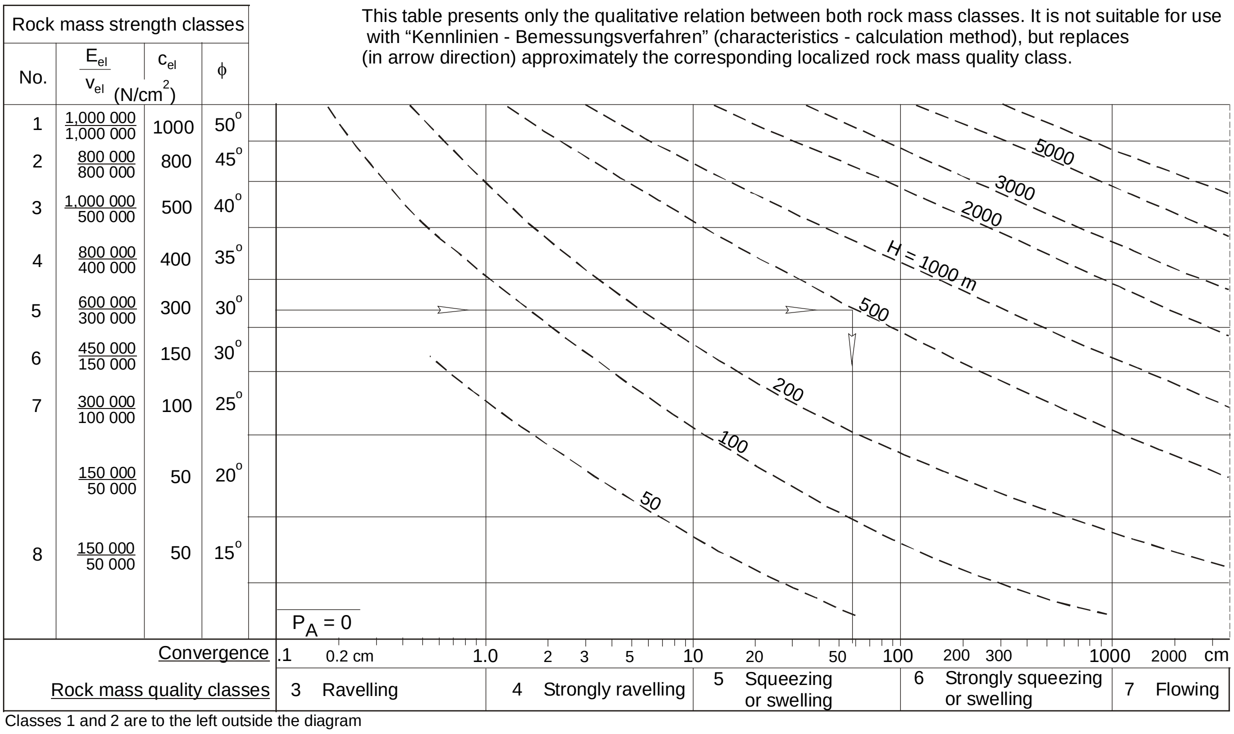


Fig. 8-7 The characterization of the ground into NATM classes as applied by Seeber et al. (1978). Classes 1 and 2 are not covered.

The practical use of the 'standard characteristic (ground response) curves' is shown in Figs. 8-8 and 8-9. Both figures are for the same type of NATM class 5 ('Gebirgsfestigkeitsklasse' 8, ϕ = 15 - 25°, and overburden 400 m), i.e. 'sehr grebräch' or 'squeezing or swelling'. Lines for bolt lengths and concrete lining are shown in both figures. Fig. 8-9 shows how the curves in Fig. 8-8 can be used to determine the support pressure and the corresponding displacement, which depends on the type of rock support.

The characteristic ground response curves are for circular tunnels with 6 m radius. As the displacements are approximately proportional to the excavation radius (Seeber et al., 1978), they can easily be estimated also for other tunnel sizes. Fig. 8-10 shows the displacements in circular tunnels of various sizes located in the same NATM class.

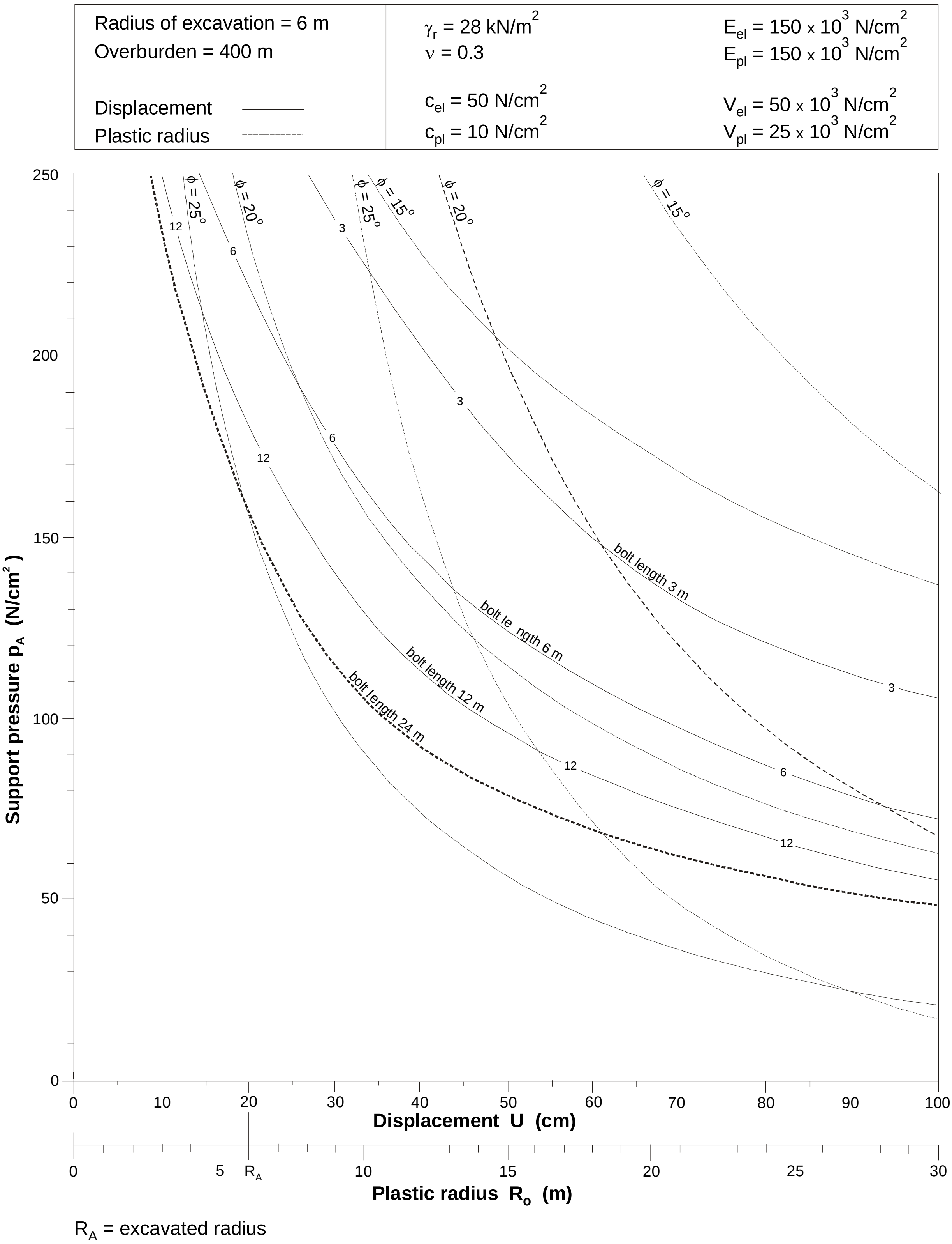
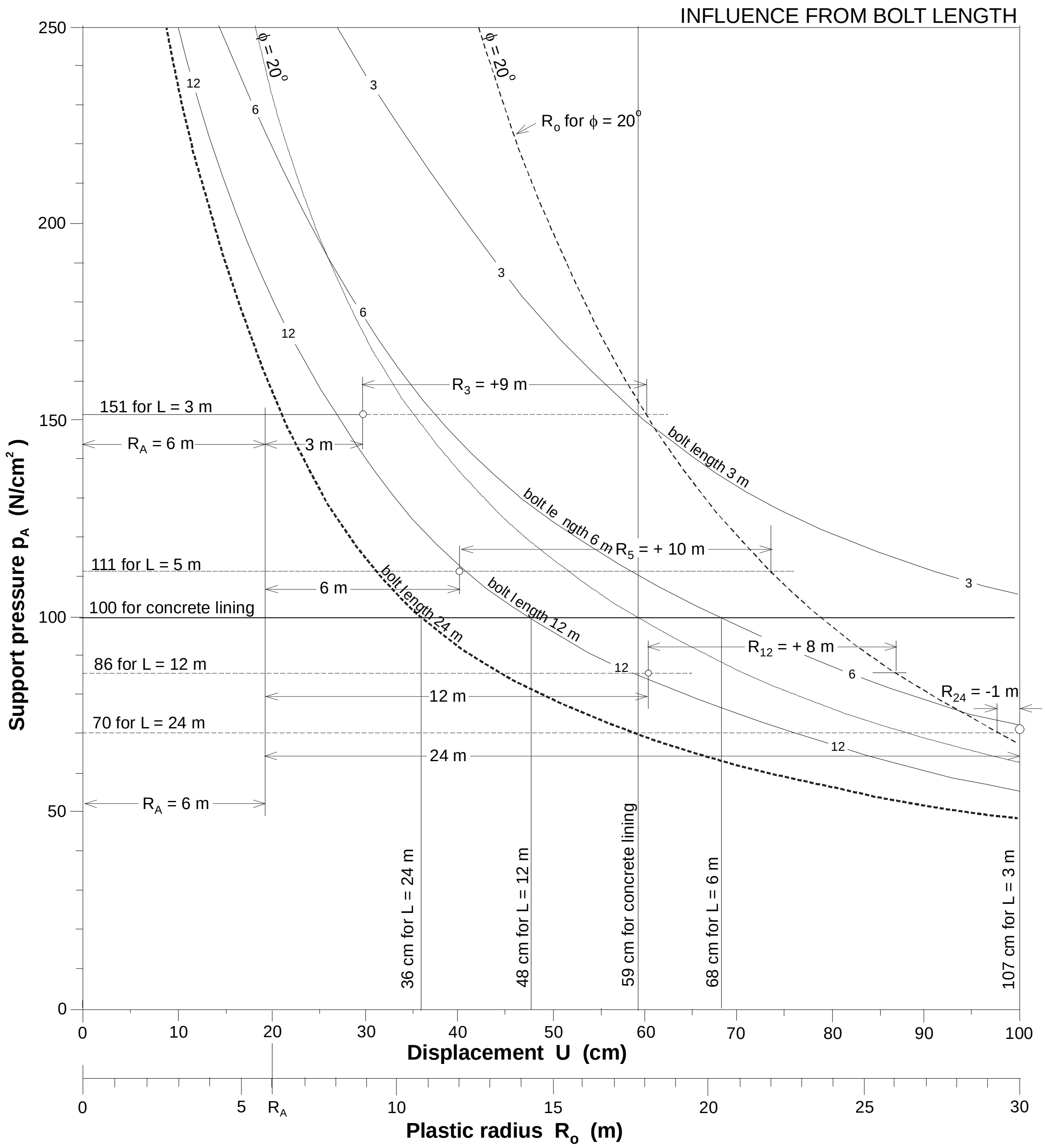


Fig. 8-8 One example of the 96 standard ground response curves worked out by Seeber et al. (1978) for circular tunnels with 12 m diameter.

Radius of excavation = 6 m Overburden = 400 m Displacement ——— Plastic radius - - - - -	γ_r = 28 kN/m^2 ν = 0.3 c_{el} = 50 N/cm^2 c_{pl} = 10 N/cm^2	E_{el} = 150 x 10^3 N/cm^2 E_{pl} = 150 x 10^3 N/cm^2 V_{el} = 50 x 10^3 N/cm^2 V_{pl} = 25 x 10^3 N/cm^2

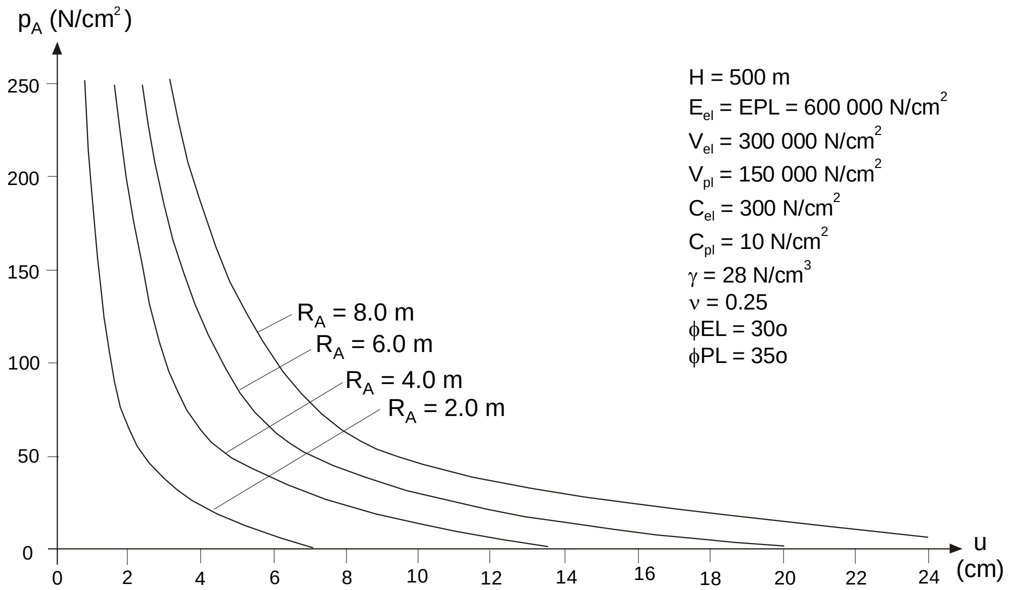


Bolt length and required support pressure for constant displacement U = 59 cm					
Rock support	Rock bolts L_{AN} = 3 m	6 m	12 m	24 m	Concrete lining
p_A (N/cm^2)	151	111	86	70	100

Displacement and bolt lengths required for constant support pressure p_A = 100 N/cm^2					
Rock support	Rock bolts L_{AN} = 3 m	6 m	12 m	24 m	Concrete lining
p_A (N/cm^2)	107	68	48	36	59

L_{AN} = length of rock bolts p_A = support pressure U = displacement

Fig. 8-9 The influence of bolt length on the support pressure and displacement in the tunnel. The response curve is the same as shown in Fig. 8-8 (revised from Seeber et al., 1978).



U = displacement *p_A = support pressure* *R_A = excavation radius*

Fig. 8-10 The displacements varying with the size of the tunnel within the same ground class (from Seeber et al., 1978).

It is obvious that the accuracy of the procedure depends in particular on the accuracy of the input parameters. As they, according to Seeber et al. (1978), generally present a scatter of approx. 100%, a computation, which bases itself on these data cannot possibly results in a better accuracy. If, however, convergence measurements are available at a somewhat later date, the results can then be used to improve the accuracy of the input parameters considerably.

8.5 THE USE OF RMi PARAMETERS IN CLASSIFICATION SYSTEMS

> *"A fundamental requirement for any classification is the need for established criteria in order to arrange the rock being classified systematically into significant groups and categories."*
> Williamson and Kuhn (1988)

RMi is not directly applicable in the main classification systems, as they often are completed systems of 'their own'. Some of the parameters involved in RMi may, however, be used, which can be of interest where they are considered more accurate or if they are easier measured.

The existing two main classification systems are the RMR (or Geomechanics) system developed by Bieniawski (1973) and the NGI Q-system by Barton et al. (1974). The systems partly apply different parameters in different modes; consequently, the established mathematical connections between them are generally empirical and approximate.

TABLE 8-5 THE RMR CLASSIFICATION SYSTEM OF ROCK MASSES. THE RATINGS FOR EACH PARAMETER ARE SUMMED UP TO ARRIVE AT THE RMR VALUE FOR THE ACTUAL ROCK MASS (from Bieniawski, 1984).

A. Classification parameters and their ratings

PARAMETER			Range of values // **RATINGS**						
1	Strength of intact rock material	Point-load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range: uniaxial compr. strength is preferred		
		Uniaxial com-pressive strength	> 250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	RATING		**15**	**12**	**7**	**4**	**2**	**1**	**0**
2	Drill core quality RQD		90 - 100%	75 - 90%	50 - 75%	25 - 50%	< 25%		
	RATING		**20**	**17**	**13**	**8**	**5**		
3	Spacing of discontinuities		> 2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm		
	RATING		**20**	**15**	**10**	**8**	**5**		
4	Condition of discon-tinuities	Length, persistence	< 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	slightly rough	smooth	slickensided		
		Rating	**6**	**5**	**3**	**1**	**0**		
		Infilling (gouge)	none	Hard filling		Soft filling			
			-	< 5 mm	> 5 mm	< 5 mm	> 5 mm		
		Rating	**6**	**4**	**2**	**2**	**0**		
		Weathering	unweathered	slightly w.	moderately w.	highly w.	decomposed		
		Rating	**6**	**5**	**3**	**1**	**0**		
5	Ground water	Inflow per 10 m tunnel length	none	< 10 litres/min	10 - 25 litres/min	25 - 125 litres/min	> 125 litres /min		
		p_w / σ1	0	0 - 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
		General conditions	completely dry	damp	wet	dripping	flowing		
	RATING		**15**	**10**	**7**	**4**	**0**		

p_w = joint water pressure; σ1 = major principal stress

B. Rating adjustment for discontinuity orientations

Strike and dip orientation of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
RATINGS	Tunnels	**0**	**-2**	**-5**	**-10**	**-12**
	Foundations	**0**	**-2**	**-7**	**-15**	**-25**
	Slopes	**0**	**-5**	**-25**	**-50**	**-60**

C. Rock mass classes determined from total ratings

Rating	100 - 81	80 - 61	60 - 41	40 - 21	< 20
Class No.	I	II	III	IV	V
Description	VERY GOOD	GOOD	FAIR	POOR	VERY POOR

D. Meaning of rock mass classes

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	< 45°	35 - 45°	25 - 35°	15 - 25°	< 15°

8.5.1 Input to the RMR (Geomechanics) system

As the RMR is based on the sum of several parameters, while RMi and partly also the parameters involved in are expressed exponentially, it is difficult to directly apply RMi in RMR. An exception is the compressive strength, σ_c , which is the same in both systems. Also the joint condition factor (jC) has similarities with the joint condition applied in RMR.

In the RMR system the jointing is characterized by the RQD and by the spacing of joints. As shown in Appendix 4, RQD generally is an inaccurate measure of the block size or discontinuity intensity. Also discontinuity spacing - though Bieniawski (1989) has made attempts to define it better - is often insufficiently defined (refer to Section 3.7 in Appendix 3). These two parameters for block size in RMR may be considerably better characterized by the block size (Vb). This would in addition make RMR simpler to use.

The characterization of the parameter for 'condition of discontinuities' has been significantly improved in the 1989 version of RMR as can be seen by comparing Tables 8-5 and 8-6. Still, the joint condition factor (jC) in RMi may be considered as an improvement compared to the corresponding RMR.

TABLE 8-6 THE IMPROVED DIVISION AND RATING OF DISCONTINUITY CONDITIONS (from Bieniawski, 1989).

Guidelines for Classification of Discontinuity Conditions *)

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

*) Note: Some conditions are mutually exclusive: For example, if infilling is present, it is irrelevant what the roughness may be, since its effect will be overshadowed by the influence of the gouge. In such cases, use the main classification table directly.

8.5.2 Input to the Q-system

The Q-system for classification of rock masses is defined as

$$Q = (RQD/Jn) \times (Jr/Ja) \times (Jw/SRF) \qquad \text{eq. (8-17)}$$

where RQD is the rock quality designation (Deere, 1966)
Jn is the joint set number,
Jr is the joint roughness number,
Ja is the joint alteration number,
Jw is the joint water reduction number, and
SRF is the stress reduction factor.

The Q system and the RMi system have a similar structure and also some parameters are similar. It is probably the classification system in which the parameters in RMi best can be utilized.

As the Q-system includes external features (stresses and water pressure) acting, only the first four parameters (RQD/Jn)·(Jr/Ja) can be compared with RMi. The Q-system does not directly include a parameter for the rock material; therefore these four parameters express the jointing in the rock mass similar to the jointing parameter (JP) in RMi. A comparison has been discussed in Chapter 9.

The ratio (RQD/Jn) in the Q-system is an expression for the block size (Barton et al., 1974) and can be compared to the block volume (Vb) in RMi. Appendix 4 concludes, however, that this ratio very poorly represents the block size. Using the block volume (Vb) instead of (RQD/Jn) would improve the quality of the Q-system.

The values of the factors for joint roughness (Jr with jR) and joint alteration (Ja with jA) are similar in both systems. As the RMi also includes a factor for the joint size (jL) in its joint condition factor (jC), a better characterization of the shear strength of joints may be achieved. Also here the Q-system may benefit from applying this RMi parameter.

8.5.3 Input to other classification systems

From the simple structure of RMi it is easy to determine the effect of the various parameters, and consequently how RMi can be developed for other purposes.

In the literature 'new' classification systems are often developed for a new project, based on the requirement to 'tailor' the classification to the actual rock masses found in the specific area. With its simple structure, RMi is suited for such developments adapting it to local ground conditions.

8.6 A CONTRIBUTION TO IMPROVED COMMUNICATION

In engineering geology and civil engineering, as in other areas, there is need for clear and effective communication between individuals involved. The geologist and the engineering geologist provide the basic data of the ground on which the engineering calculations are based. Generally, interpretations and correlations between geological and geotechnical data have been made by individuals, based on their personal experience rather than on any collective basis. For successful results, close association must exist between geologist and engineer, with full appreciation and understanding of the parts played by each. *"The accuracy of the final answer can only be as accurate as the geological data at hand."* (Piteau, 1970).

Communication problems are compounded by the fact that the engineering geologist is dealing with a material that is difficult to define due to its complex nature. Williamson and Kuhn (1988) are of the opinion that *"The use of subjective geologic terminology has proven to be less than helpful in resolving this problem with such terms as 'slightly weathered', 'moderately hard' and 'highly fractured'. These terms do not communicate the true picture even from one geologist to another, because each has a different perception of the meaning"*. Thus, there is still a demand for improving the applied terms in engineering geology and rock mechanics.

8.6.1 Identification chart for geological materials

As a part of the contribution for improved communication a general identification chart for geological materials has been developed. It is a further development of the chart presented by Palmström (1986). The chart is in part similar to the 'unified classification chart' developed by Deere et. al (1969) as shown in Fig. 8-11.

Deere et al. used a combination of the following geological features:

- the particle or block size; and
- the continuity of the geologic materials.

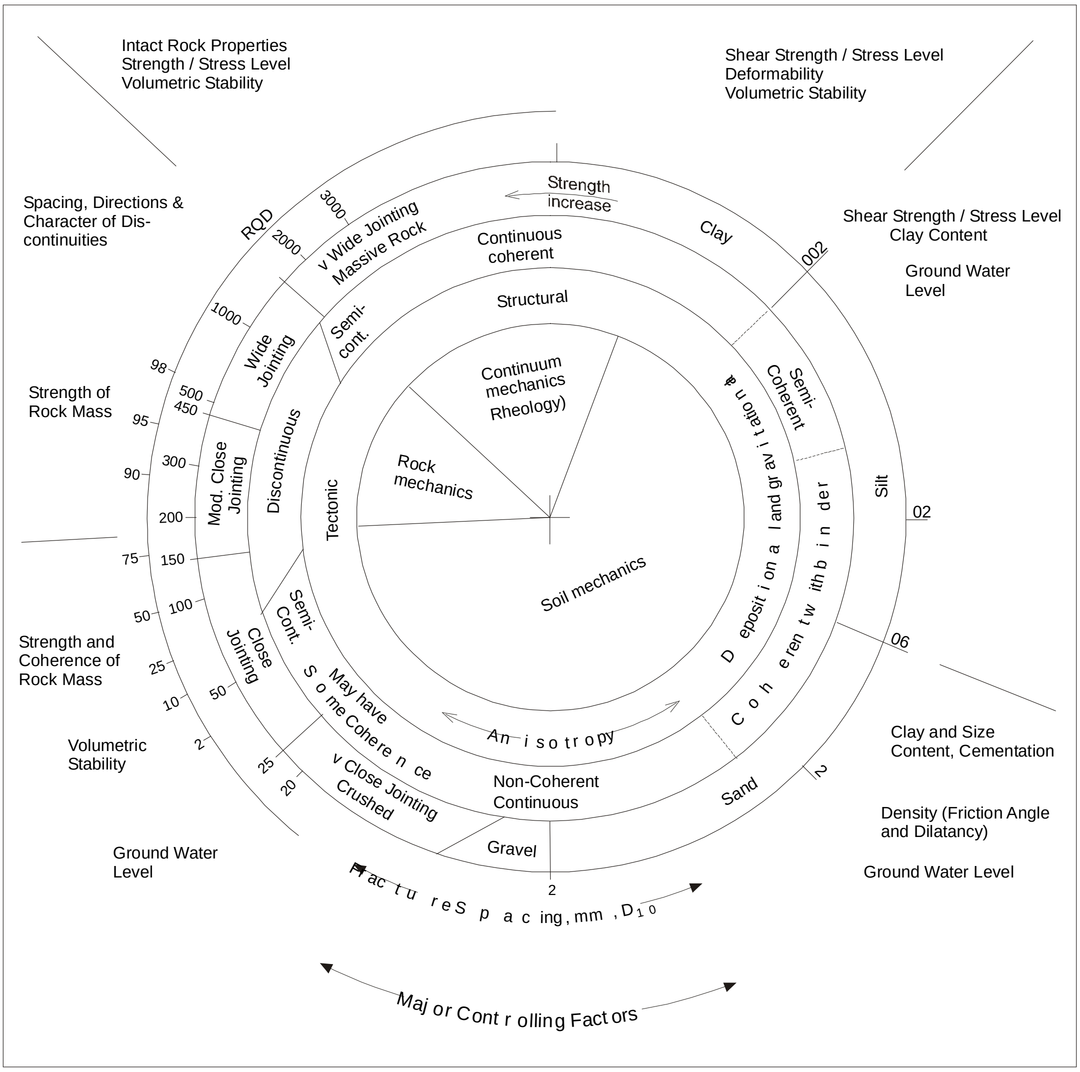


Fig. 8-11 The unified classification chart (from Deere et al., 1969)

There are many overlapping characteristics of soils and rocks in rock engineering; stiff clays extend naturally into shales and slates; varying degrees of alteration or cementation can form any intermediate characteristic between solid rock and indurated soil. It is significant that major problems in rock tunnelling are frequently associated with weakness zones where the rock approaches the character of soil. Often, the most important characteristic separating soil from rock is the relative importance of the discontinuities in rocks. Other differences are:

* A *soil* mass consists of an assemblage of uncemented angular to round particles randomly located. The voids in between the particles may or may not be filled with water (or more fine-grained materials in the case of moraines and coarse-grained materials (scree)). It is essentially a continuous material.
* A *rock* mass, on the other hand, can sometimes be considered as a continuous, sometimes as a discontinuous material made up of an aggregate of blocks or particles properly organized or piled like the bricks in a wall, more or less separated by planes of weakness. These blocks generally fit tightly. The spaces between the blocks may or may not contain water and soft and/or hard infilling materials.

In soils there will be a tendency for failure to occur arbitrarily, but in rock masses the tendency is for the failure to follow pre-existing planes of weakness. A second important difference between soil and rock behaviour is, that in rock masses, the shear strength will be determined largely by the shear strength of the discontinuities and not by the rock strength.

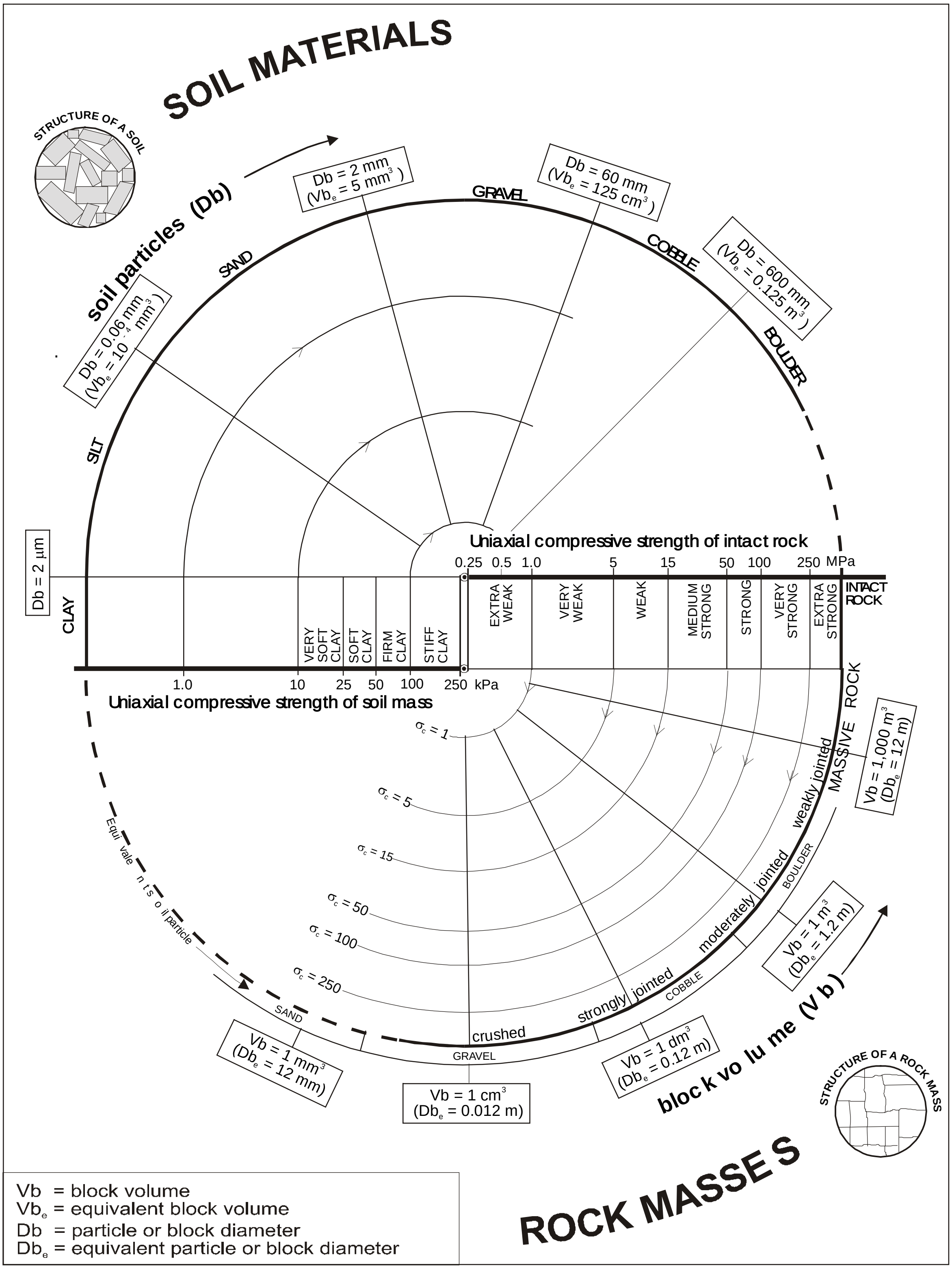


Fig. 8-12 A general identification chart for rock masses and soil materials

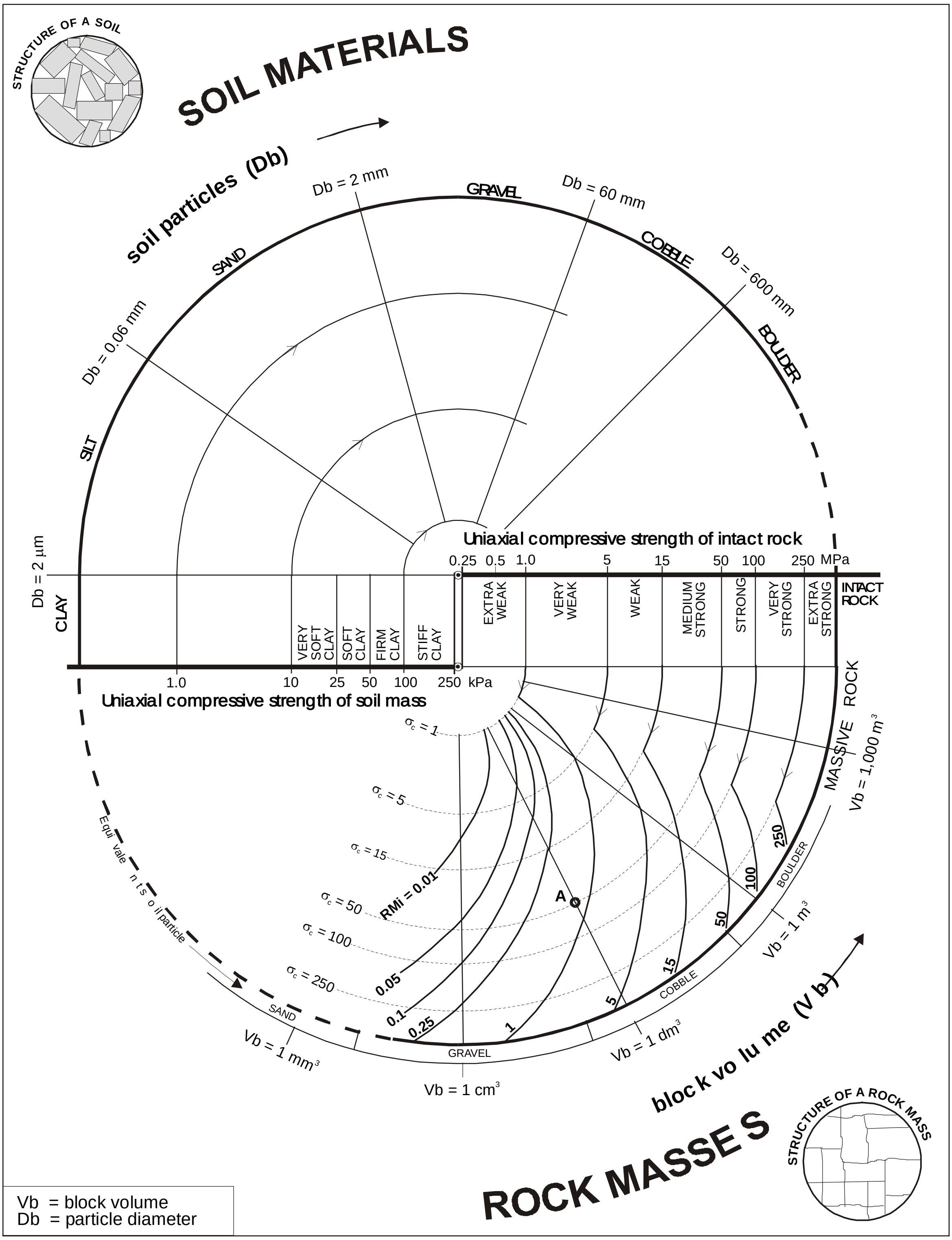


Fig. 8-13 The RMi values for 'normal joint condition' (jC = 1) plotted on the identification chart. The RMi value is found from the intersection between uniaxial strength (following the circles) and block size.
Example: uniaxial compressive strength of rock $\sigma_c = 50$ *MPa and block volume* $Vb = 1\ dm^3$ *gives a location in 'A'. Here RMi* ≈ 0.7

The identification chart can be used to show the position of and the relation between actual materials. It may help

- to identify the difference/similarities between various geologic materials,
- to identify and compare the different types of behaviour for various materials, and
- to improve communication.

Thus, it is hoped that the identification chart presented in Fig. 8-12 for rock masses will shorten the gap between various rock classification systems and will contribute to a better communication between soil and rock mechanics people. In Fig. 8-13 the RMi values may be found roughly for 'normal joint condition' i.e. the joint condition factor jC = 1 - 2.

8.7 POSSIBLE USE OF RMi IN NUMERICAL MODELS

Numerous authors have demonstrated the use of numerical models in tunnel design. They have produced a wealth of information, much of which is of considerable general interest, concerning the two-dimensional stress and deformation patterns around tunnels. In using these powerful numerical tools, it is necessary to be constantly aware of the fact that the answers produced are only as good as the input information. In view of this limitation, the potential of numerical models can today rarely be fully utilized in the practice of engineering design.

However, sensitivity studies made possible by computers can explore the influence of variations in the value of each input parameter making a contribution to engineering judgement of the accuracy in the calculation.

No attempts have been made in this work to apply RMi or its input parameters in numerical models. Block size, block type and shape in the RMi system can, however, be valuable as input to numerical models. Due to the fact that the input parameters to RMi are well defined, their possible use in numerical models may consequently result in improved numerical predictions.