The Rock Mass index (RMi) applied in rock mechanics and rock engineering

Arild Palmström
Berdal Strömme a.s., partner of Norconsult International
(formerly at Norwegian Geotechnical Institute)
Vestfjordgaten 4, N-1300 Sandvika, Norway
Tel.: +47 67 57 11 00, Fax: +47 67 54 45 76

ABSTRACT

The RMi system is based on defined inherent rock mass parameters. Basically, it combines the compressive strength (σ_c) of intact rock and a jointing parameter (JP) in the expression RMi = $\sigma_c \cdot$ JP. JP represents the main jointing features, namely block volume (or density of joints), plus roughness, alteration, and size of the joints. This paper shows how RMi can be applied to a) determine the constants s and *m* in the Hoek-Brown failure criterion for rock masses to assess the shear strength parameters of continuous rock masses; b) work out ground response curves using the same s and *m* constants; c) quantify the descriptive NATM classification; d) estimate stability and rock support in underground openings. Rock support charts are presented for the three main groups of rock masses: discontinuous (jointed) rock masses, continous (massive and highly jointed) rock masses, and weakness zones. The applications of RMi in rock engineering probably include a wider range of rock masses than any other numerical characterization or classification system.

1 INTRODUCTION

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

This is the second of two papers presenting results from the Ph.D. thesis "RMi - a rock mass characterization system for rock engineering purposes" (Palmström, 1995a) worked out 1991 - 1995. The main goals of the RMi (Rock Mass index) system have been to improve the input data and their use in rock engineering. RMi makes use of selected inherent parameters in the rock mass which are combined of to express the following relative rock mass strength index:

$$\mathbf{RMi} = \sigma_c \cdot \mathbf{JP} \qquad \text{eq. (1)}$$

where σ_c = the uniaxial compressive strength of intact rock
JP = the jointing parameter; it is composed of the block volume and three joint characteristics (roughness, alteration, and size)

The development of RMi and how it is practically determined have been given in the first paper (Palmström, 1995d) of which a summary is given in Appendix I. The present paper shows the application of RMi and/or its parameters in in rock mechanics and rock engineering.

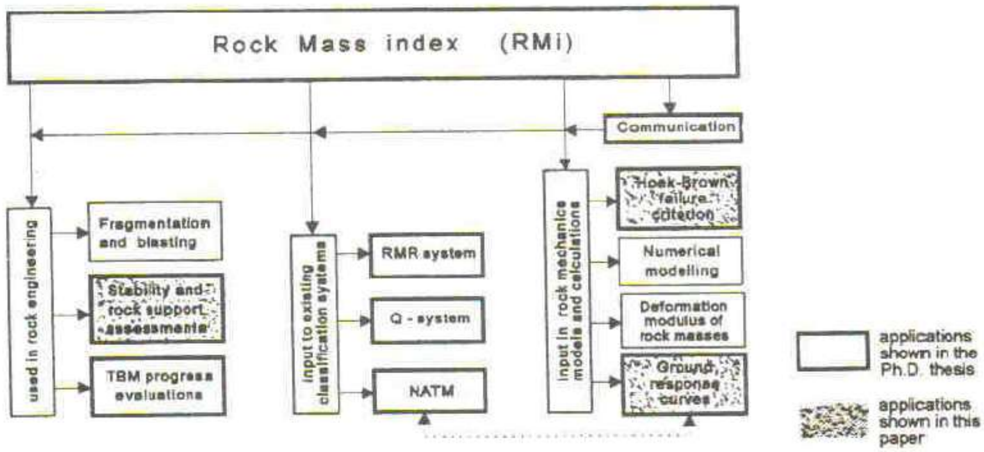


Fig. 1 Various applications of RMi or of the parameters included in RMi. (from Palmström, 1995a).

The Rock Mass index (RMi) is different from earlier *general* classifications of rock masses, which mainly are descriptive or qualitative, as RMi is numerical. This is a prerequisite being applicable in rock mechanics and rock engineering calculations. Fig. 1 shows the main applications of RMi.

2 RMi APPLIED TO DETERMINE THE CONSTANTS IN THE HOEK-BROWN FAILURE CRITERION FOR ROCK MASSES

"Provision of reliable input data for engineering design of structures in rock is one of the most difficult tasks facing engineering geologists and design engineers."
Z. T. Bieniawski, 1984

The Hoek-Brown failure criterion provides engineers and geologists with a means of estimating the strength of jointed rock masses. After presentation 1980, the ratings of the criterion's constants (s and m) have been adjusted 1988, 1991 and 1992. A modified criterion was published by Hoek et al. (1992) as is shown in Paragraph 2.2.

2.1 The original criterion

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

$$\sigma_1' = \sigma_3' + (m \cdot \sigma_c \cdot \sigma_3' + s \cdot \sigma_c^2)^{1/2} \qquad \text{eq. (2)}$$

here σ_1' = the major principal effective stress at failure ,
σ_3' = the minor principal effective stress (for triaxial tests, the confining pressure)
σ_c = the uniaxial compressive strength of the intact rock material
s and m are empirical constants representing inherent properties of joints and rocks

For $\sigma_3' = 0$ eq. (2) expresses the unconfined *compressive strength* of a rock mass

$$\sigma_{cm} = \sigma_c \sqrt{s} \quad (= \sigma_c \cdot \mathrm{JP}) \qquad \text{eq. (3)}$$

TABLE 1 THE VARIATION OF s AND m WITH THE COMPOSITION OF ROCK MASSES AND THE ROCK TYPES (from Wood, 1991).

		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	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.

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 main classification systems; the CSIR system of Bieniawski (1973) and the NGI system of Barton et al. (1974). This is is shown in Table 1. As these systems 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 only approximately represents the variation in jointing (Palmström, 1995d).

As both RMi and eq. (3) expresses the unconfined compressive strength of a rock mass, RMi can preferably be applied to determine the constants s and m.

The constant s

From eqs. (1) and (3) the constant s can be found from the jointing parameter (JP)

$$s = \mathbf{JP}^2 \qquad \text{eq. (4)}$$

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 described and tested by Jaeger (1969). Therefore, Hoek (1983) mentions that the values of s may be approximate.

As shown by Palmström (1995d) the value of JP is found from input of block size (Vb) and joint condition factor (jC) (see Appendix I), i.e. only the inherent features in the rock mass are used. It is based on measured strength in 8 different "samples" of rock masses, and therefore offers a better accuracy of the constant s in Table 1.

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 2. The constant m_b is the same as m in the the original criterion. It varies with the jointing. As shown in Fig. 2 it can be expressed as:

a) For undisturbed rock masses $\quad m_b = m_i \cdot \mathbf{JP}^{0.64}$ $\qquad$ eq. (5)

b) For disturbed rock masses $\quad m_b = m_i \cdot \mathbf{JP}^{0.857}$ $\qquad$ eq. (6)

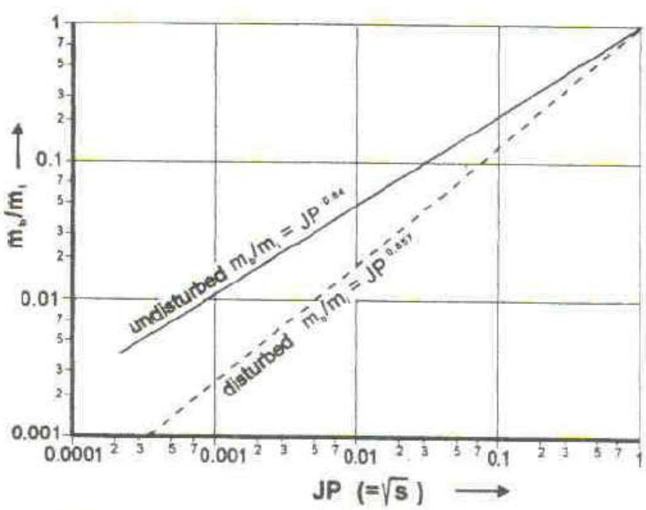


Fig. 2 The variation of m_b/m_i with the jointing parameter (JP), based on Wood (1991) and the data in Table 1.

Applying eqs. (4) and (5) in eq. (2), the failure criterion for undisturbed rock masses can be written as

$$\sigma_1' = \sigma_3' + [\sigma_c \cdot \mathbf{JP}^{0.64} (m_i \cdot \sigma_3' + \sigma_c \cdot \mathbf{JP}^{1.36})]^{½} \qquad \text{eq. (7)}$$

where s and m have been replaced by JP and m_i .

TABLE 2 THE VALUE OF m_i FOR SOME COMMON ROCKS
(based on Wood, 1991, and Hoek 1994)

Amphibolite	31.2	Gabbro	25.8	Mica schist	15 ?
Anhydrite	13.2	Gneiss	29.2	Monzonite	30 ?
Andesite	18.9	Gneiss granite	30 ?	Norite	21.7
Augen gneiss	30 ?	Granite	32.7	Phyllite	13 ?
Basalt	(17)	Granite gneiss	30 ?	Quartzite	23.7
Claystone	3.4	Granodiorite	20 ?	Rhyolite	(20)
Conglomerate	(20)	Greenstone	20 ?	Sandstone	18.8
Coral chalk	7.2	Limestone	8.4	Siltstone	9.6
Diabase (dolerite)	15.2	Marble	9.3	Slate	11.4
Diorite	27 ?	Mica gneiss	30 ?	Syenite	30 ?
Dolomite	10.1	Mica quartzite	25 ?	Talc schist	10 ?

Values in parenthesis have been estimated by Hoek (1994); the ones with question mark by Palmström (1995a).

2.2 The modified Hoek-Brown failure criterion

From more than 10 years of experience in using the Hoek-Brown criterion, Hoek et al. (1992) presented a modified version in the following, simplified form:

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

where 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 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.

To a certain extent a can be compared with the factor D in the expression for RMi (see Appendix I) 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; D does not, however, include the degree of jointing (i.e. block volume (Vb) since Vb has been included directly in the jointing parameter (JP)).

TABLE 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).

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 composition, structure and surface conditions 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 Moderately weathered, 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 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 sizes	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	0.004 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

SHEAR STRENGTH PARAMETERS - examples

INPUT DATA			example 1	example 2	example 3
ROCK CHARACTERISTICS *	Type of rock =		limestone	granite	granite
Rock compressive strength (MPa)		σ_c	50,00	160,0	160,0
H-B's m - factor for intact rock	Table 2	m_i	8,40	32,7	32,7
JOINT CHARACTERISTICS					
Joint roughness factor	Table AI-1	jR	2,00	3,0	3,0
Joint alteration factor	Table AI-2	jA	1,00	2,0	2,0
Joint length and continuity factor	Table AI-3	jL	2,00	1,0	1,0
BLOCK VOLUME (m^3)		Vb	0,001	1	1
EFFECTIVE NORMAL STRESS (MPa)		σ'	1	0,1	10
CALCULATIONS					
RMi PARAMETERS					
Scale factor compressive strength	eq. (23)	f_s	0,87		
Joint condition factor	eq. (AI-2)	jC	4,00	1,50	1,50
Jointing parameter	eq. (AI-3)	JP	0,0577	0,2449	0,2449
Rock Mass index	eq. (1)	RMi	2,88	39,19	39,19
CONSTANTS IN HOEK-BROWN FAILURE CRITERION					
s $(= JP^2)$	eq. (4)	s	0,0033	0,0600	0,0600
m_b (undisturbed)	eq. (5)	m_b	1,35	13,29	13,29
Calculation factor	eq. (11)	h	1,0885	1,0021	1,0269
SHEAR STRENGTH PARAMETERS					
Instantaneous friction angle (degree)	eq. (10)	ϕ_i	48,98	72,93	58,51
Shear stress (MPa)	eq. (9)	τ_i	1,81	3,60	23,98
Instantaneous cohesion (MPa)	eq. (12)	c_i	0,66	3,27	7,65

Fig. 3 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 (from Palmström, 1995a).

2.4 RMi used to evaluate the shear strength of rock masses

Hoek (1983) presented the following failure envelope derived from the Hoek-Brown failure criterion:

$$\tau = (\mathrm{Cot}\Phi_i' - \mathrm{Cos}\Phi_i')\ (m\sigma_c / 8) \qquad \text{eq.(9)}$$

where τ = the shear stress at failure
Φ_i' is the instantaneous friction angle which is expressed as

$$\Phi_i' = \mathrm{Arctan}\ [4h\mathrm{Cos}^2(\pi/6 + ⅓\ \mathrm{Arcsin}\ h^{-3/2}) - 1]^{-1/2} \qquad \text{eq. (10)}$$

Here $h = 1 + 16(m\sigma' + s\sigma_c)\ /3m^2\sigma_c$ eq. (11)
in which σ' = the effective normal stress, $m_b = m_i \cdot \mathrm{JP}^{0.64}$ and $s = \mathrm{JP}^2$

The instantaneous cohesive strength is found as

$$c_i' = \tau - \sigma' \cdot \mathrm{Tan}\Phi_i' \qquad \text{eq. (12)}$$

Though eq. (10) seems complex, Φ_i' can easily be found using a spreadsheet on a desk computer. Fig. 3 shows an example where the shear stress, friction angle and the cohesion for a rock mass has been calculated from eqs. (9) to (12).

It should be borne in mind that the Hoek-Brown failure criterion for rock masses is only valid for *continuous rock masses* (Hoek and Brown, 1980). The occurence and structure of continuous rock masses are further outlined in Paragraph 4.2.

3 RMi USED IN THE INPUT TO GROUND RESPONSE CURVES

"The basic aim of any underground excavation design should be to utilize the rock itself as the principal structural material, creating as little disturbance as possible during the excavation process and adding as little as possible in the way of concrete or steel support. In their intact state and when subjected to compressive stresses, most hard rocks are far stronger than concrete and many are of the same order of strength as steel. Consequently, it does not make economic sense to replace a material which may be perfectly adequate with one which may be no better." Evert Hoek and Edwin T. Brown (1980)

Ground-response interaction diagrams are well established aids to the understanding of rock mass behaviour and tunnel support involvement. They are limited to continuous materials, i.e. massive rock or highly jointed and crushed (particulate) rock masses (see Paragraph 4.2). 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.

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 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 a closed-form solution as shown in Fig. 4, in which the input data used 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 $-\epsilon_3^p$, ϵ_1^p diagram (Fig. 4)

Data for *original non-disturbed* rock mass:

- m and s are the 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 the constants in the Hoek-Brown failure criterion.

These input data are applied in the following calculation sequence given by Brown et al. (1983):

1. $M = 1/2\,[(m/4)^2 + (mp_o/\sigma c) + s]^{1/2} - m/8$.
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: $u_r/r_e = M\sigma_c/2G$.
5. $N = 2\{[(p_o - M\sigma_c)/m_r\sigma_c] + (s_r/m_r^2)\}^{1/2}$.
6. $r_e/r_i = \exp\{N - 2[(pi/m_r\sigma_c) + (s_r/m_r^2)]^{1/2}\}$.
7. $\delta_i/r_i = M\sigma_c/[G(f + 1)]\{[(f - 1)/2] + (r_e/r_i)^{f+1}\}$.

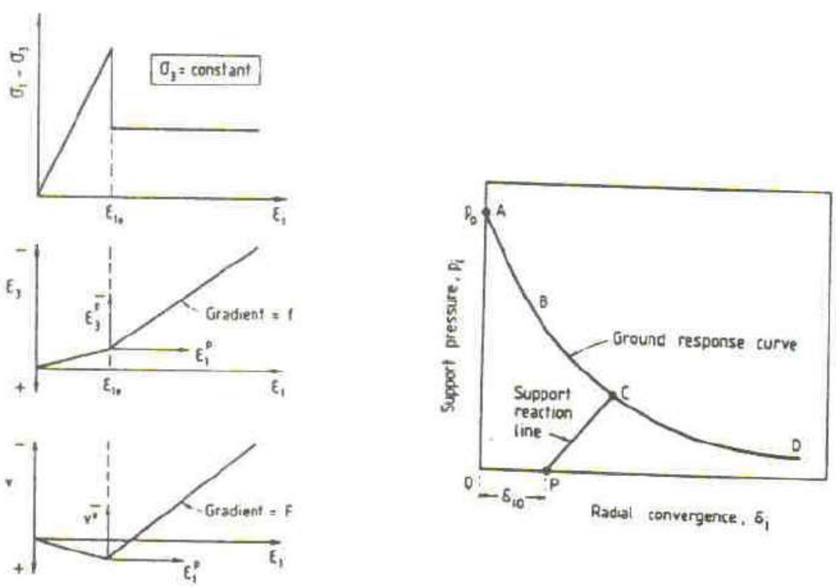


Fig. 4 Left: The material behaviour used by Brown et. al.(1983) in the closed-form solution Right: the ground response curve.

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 found from

$$\mathbf{f = 1 + F} \qquad \text{eq. (14)}$$

where $$F = \frac{m}{2\left(m\frac{\sigma_{re}}{\sigma_c} + s\right)^{1/2}} \qquad \text{eq. (15)}$$

in which $$\sigma_{re} = \mathbf{p_o - M \cdot \sigma_c} \qquad \text{eq. (16)}$$

The constant s $(= JP^2)$ can be found from the description and tables in Appendix I or from Palmström (1995d). It is based on field characterization of the block size (Vb) and the joint characteristics in the joint condition factor (jC). The constant m $(= m_b)$ can be found from Table 2 and eq. (5).

For the broken, (plastic) zone the appropriate jointing parameter (JP) in eqs. (4) and (6) must be estimated to find the corresponding s_r and m_r values. 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 'common' joint conditions (joint condition factor jC = 1.75) for the ground containing new breaks, the expression $JP = 0.25\ Vb^{1/3}$ can be applied (refer to Palmström, 1995d). Thus the value of the constants can be found from the block volume in the following equations:

$$\boldsymbol{s_r = JP^2 = 0.06\ Vb_r^{2/3}} \qquad \text{eq. (17)}$$

and

$$\boldsymbol{m_r = m_i \cdot JP^{0.857} = 0.3\ Vb_r^{0.29}} \qquad \text{eq. (18)}$$

where Vb_r = the resulting block volume from the breaking up during the squeezing process.

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 methods 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. Also Seeber et al. (1978) have presented a method where ground response curves are applied to estimate the rock support. This is briefly outlined in Section 5.

4 THE USE OF RMi IN THE EVALUATION OF ROCK SUPPORT

> *"It is essential to know whether the problem is that of maintaining stability with the pre-existing jointing pattern or whether it is the very different problem of a yielding rock mass. The stress situation is therefore one of the main parameters in stability and rock support evaluations."* Sir A.M. Muir Wood (1979)

There are no standard analyses for determining rock support, because each design is specific to the circumstances (scale, depth, presence of water, etc.) at the actual site and the national regulations and experience. Support design for a tunnel in rock often involve problems that are of relatively little or no concern in most other branches of solid mechanics. *"The material and the underground opening forms an extremely complex structure. It is seldom possible, neither to acquire the accurate mechanical data of the ground and forces acting, nor to theoretically determine the exact interaction of these"* (Hoek and Brown, 1980).

Therefore, the rock engineer is generally faced with the need to arrive at a number of design decisions and simplifications in which judgement and practical experience must play an important part. Prediction and/or evaluation of support requirements for tunnels is largely based on observations, experience and personal judgement of those involved in tunnel construction (Brekke and Howard, 1972). Often, the estimates of rock support are backed by theoretical approaches in support design of which three main groups have been practised in recent years, namely

- the classification systems,
- the ground-support interaction analysis (or the Fenner-Pacher curves in NATM),
- the key block analysis.

The design of excavation and support systems for rock, although based on some scientific principles, has to meet practical requirements. In order to select and combine the parameters of importance for stability in an underground opening the main features determining the stability have been reviewed in the following paragraph.

4.1 Instability and failure modes in underground excavations

Basically, the instability of rock masses surrounding an underground opening may be divided into two main groups (Hudson, 1989):

1. One is block failure, where preexisting blocks in the roof and side walls become free to move because the excavation is made. These are called *'structurally controlled failures'* by Hoek and Brown (1980) and involve a great variety of failure modes as loosening, ravelling, and block falls.
2. The other is where failures are induced from *overstressing*, i.e. the stresses developed in the ground exceed the local strength of the rock mass, which may occur in two main forms, namely:
    a. Overstressing of massive or intact rock (which takes place in the mode of spalling, popping, rock burst etc.).
    b. Overstressing of particulate materials, i.e. soils and heavy jointed rocks (where squeezing and creep may take place).

A third group is instability in *faults and weakness zones*. They often require special attention in underground constructions, because their structure, composition and properties may be quite different from the surrounding rock masses. Zones of significant size can have a major impact upon the stability as well as on the excavation process of an underground opening. These and several other possible difficulties connected with such zones require that special investigations often are necessary to predict and avoid such events. Bieniawski (1984, 1989) therefore recommends that faults and othe weakness zones are mapped and treated as regions of their own.

Many faults and weakness zones contain materials quite different from the 'host' rock as a result from hydrothermal activity and other geologic processes. Thus, the instability of weakness zones may depend on other features than the surrounding rock. They all interplay in the final failure behaviour. An important inherent property in this connection is the character of the *gouge or filling material* in the zone.

Experience and knowledge of the behaviour of various types of rock masses in underground openings are important in stability analyses and rock support design. Further, the understanding of the possible failure modes in the actual ground conditions is a prerequisite in the estimates of rock support.

Failures in an underground excavation may be the result of numerous variables in the ground. Wood (1991) and several other authors find they mainly depend on:

- the generic or internal features of the rock mass;
- the external forces acting, (the ground water and stresses); and
- the activity of man in creating the opening and its use.

It is not possible to include all the factors which effect the stability of an underground excavation in one practical system to assesses the stability and evaluates rock support. Therefore, only the dominant factors have been selected as shown in Table 4.

In the author's opinion it is very difficult to work out a general method to express the *stand-up time* accurately as it is a result of many variables - among others the geometric constellations. Such variables are generally difficult to characterize by a simple number or value.
Regarding other factors which influence the stability in underground openings, the following comments are made:

- The effect from *swelling* of some rocks, and some gouge or filling material in seams and faults has not been included.[1] The swelling effect highly depends on local conditions and should preferably be linked to a specific design carried out for the actual site conditions.
- The *long-term* effects must be evaluated in each case from the actual site conditions. These effects may be creep effects, durability (slaking etc.), and access to and/or influence of water.

There are features linked to the specific case, which should be evaluated separately. They are the safety requirements, and the vibrations from earthquakes or from nearby blasting or other disturbances from the activity of man.

[1] The influence from weakening and loss of friction in swelling clays is, however, included in the joint alteration factor (jA) as input to the joint condition factor (jC) in RMi

TABLE 4 THE GROUND PARAMETERS OF MAIN INFLUENCE ON STABILITY IN UNDERGROUND OPENINGS (from Palmström, 1995a)

GROUND CONDITIONS	CHARACTERIZED BY
The inherent properties of the rock mass:	
- The intact rock strength	* The uniaxial compressive strength (included in RMi)
- The jointing properties	* The joint characteristics and the block volume (represented in the jointing parameter (JP))
- The structural arrangement of the discontinuities	(*) 1) Block shape and size (joint spacings) * 2) The intersection angle between discontinuity and tunnel surface
- The special properties of weakness zones	* 1) Width, orientation and gouge material in the zone 2) The condition of the adjacent rock masses
The external forces acting:	
- The stresses acting	* The magnitude of the tangential stresses around the opening, determined by virgin rock stresses and the shape of the opening
- The ground water	(*) Although ground water tends to reduce the effective stresses acting in the rock mass the influence of water is generally of little importance where the tunnel tends to drain the joints. Exceptions are in weak ground and where large inflows disturbs the excavation and where high ground water pressures can be built up close to the tunnel
The excavation features:	
- The shape and size of the opening	* The influence from span, wall height, and shape of the tunnel
- The excavation method	(*) The breaking up of the blocks surrounding the opening from blasting
- Ratio tunnel dimension/block size	* Determines the amount of blocks and hence the continuity of the ground surrounding the underground opening

* Applied in the system for stability and rock support (*) Partly applied

4.2 Combination of the ground characteristics for support evaluations

The behaviour of the rock mass surrounding an underground opening is mainly the combined result of the parameters mentioned in Table 4. Their importance will vary with the shape and size of the opening and with the composition of the rock mass and stresses at the specific site. In the selection of these parameters it has been found beneficial to combine those parameters which have a similar effect on the stability, into two main groups. These are *continuity* and the *condition* of the ground:

- The ***continuity of the ground*** expresses whether the volume of rock masses involved in the excavation can be considered discontinuous or not, see Fig. 5. This is important not only as a parameter in the characterization of the ground, but also to determine the appropriate method of analysis.

  The volume required for a 'sample' of a rock mass to be considered *continuous* is a matter of judgement. It depends on the size and range of blocks making up the 'sample' volume. This matter has been discussed by several authors:

- John (1969) suggests that a sample of about 10 times the average (linear) size of the single units (i.e. blocks) may be considered a uniform continuum. It is clear that this will depend to a great extent on the uniformity of the unit sizes in the material or the uniformity of the spacings of the discontinuities.
- Another approximate assumption is based on the experience from large sample testing at the University of Karlsruhe, Germany, where a volume containing at least $5 \cdot 5 \cdot 5 = 125$ blocks is considered continuous (Mutschler, 1993).
- Deere et al. (1969) have tied the 'sample' size to the size of the tunnel expressed by the ratio 'block size/tunnel size' to characterize the continuity of the ground. They found that a 'sample' should be considered discontinuous *"when the ratio of fracture spacing to a tunnel diameter is between the approximate limits of 1/5 and 1/100. For a range outside these limits, the rock may be considered continuous, though possibly anisotropic."* .

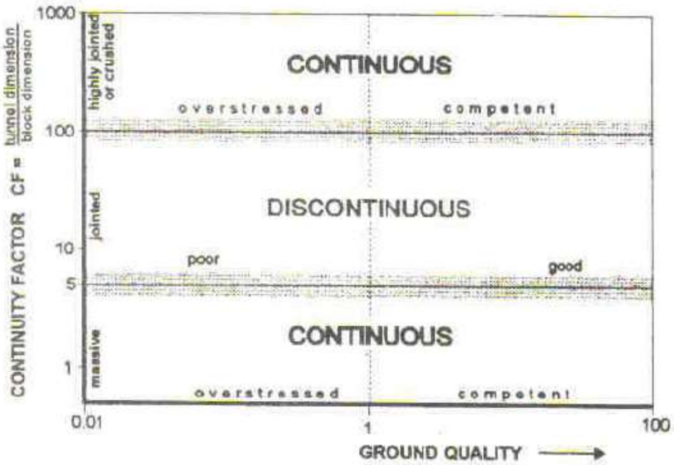


Fig. 5 The division of the ground into continuous and discontinuous rock masses (from Palmström, 1995a).

For the application of RMi in rock engineering, the division into continuous and discontinuous materials is based on Deere et al. (1969). It has, however, been chosen to express a continuity factor as the ratio

$$\mathbf{CF = tunnel\ diameter/block\ diameter = Dt/Db} \qquad \text{eq. (19)}$$

Continuous rock masses occur as

1. Slightly jointed (massive) rocks with continuity factor CF $\leq$ approx. 5
2. Highly jointed and crushed (particulate) rocks, where CF $>$ approx. 100

Else the ground is discontinuous.

- The ***condition of the ground*** is composed of selected, inherent rock mass parameters and the type of stresses having the strongest influence on stability in the actual type of ground. A competency factor has been applied in *continuous ground* as described in Paragraph 4.3. In *discontinuous ground* and for *weakness zones* a ground condition factor is introduced, see Paragraphs 4.4 and 4.5.

4.3 Stability and rock support in continuous ground

The principle of a method for evaluating rock support in continuous ground is shown in Fig. 6. Instability in this group of ground can, as mentioned, be both stress-controlled and structurally influenced. The structurally released failures occur in the highly jointed and crushed rock masses. According to Hoek and Brown (1980) they are generally overruled by the stresses where *overstressing* (incompetent ground) occurs. In *competent* ground the failures and rock support will be similar as described for discontinuous materials in Paragraph 4.4.

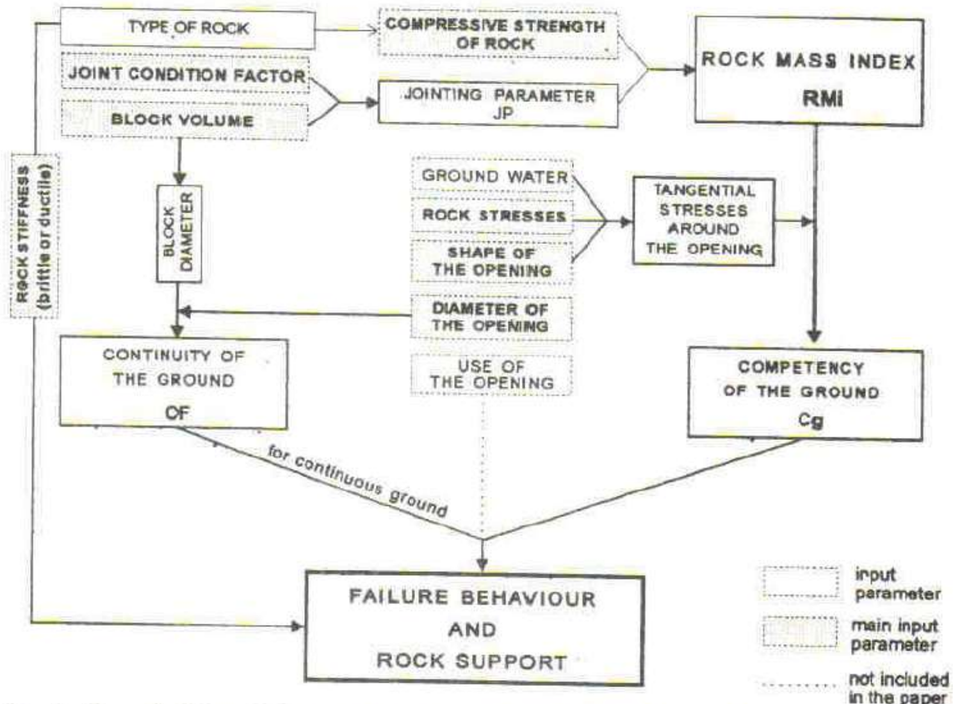


Fig. 6 The principle and the parameters involved in assessment of stability and rock support in continuous ground.

Whether overstressing will take place, is determined by the ratio between the stresses set up in the ground surrounding the opening and the strength of the rock mass. As the rock mass index (RMi) is valid in continuous ground, and expresses the (relative) compressive strength of the rock mass (Palmström, 1995a, 1995b, 1995d), it can be used in assessing the *competency factor* given as

$$\mathbf{Cg = RMi/\sigma_\theta} \qquad \text{eq. (20)}$$

where σ_θ = the tangential stresses set up around the underground opening. It can be found from input of the vertical and horizontal rock stresses, the ground water pressure, and the shape of the opening as outlined in Appendix II.

The term competency factor has earlier been proposed by Muir Wood (1979) as the ratio of uniaxial strength of rock to overburden stress. This parameter has also been used by Nakano (1979) to recognize the squeezing potential of soft rock tunnels in Japan.

In massive rock the rock mass index according to Palmström (1995d) is

$$\mathbf{RMi} = \mathbf{f}_\sigma \cdot \sigma_c \qquad \text{eq. (21)}$$

and

$$\mathbf{Cg} = \mathbf{RMi}/\sigma_\theta = \mathbf{f}_\sigma \cdot \sigma_c / \sigma_\theta \qquad \text{eq. (22)}$$

Here f_σ = the scale effect for the uniaxial compressive strength given as

$$\mathbf{f}_\sigma = (0.05/\mathbf{Db})^{0.2} \qquad \text{eq. (23)}$$

(Db is the block diameter measured in metre).

In highly jointed and crushed rock masses

$$\mathbf{Cg} = \sigma_c \cdot \mathbf{JP}/\sigma_\theta = \mathbf{RMi}\ /\sigma_\theta \qquad \text{eq. (24)}$$

Overstressed (incompetent) ground leads to failure if not confinement by rock support is established. The following main types of instability may take place:

- If the deformations take place instantaneously (often in connection with sound), the phenomenon is called *rock burst*. It occurs as breaking up into fragments or slabs in hard, strong *brittle* rocks such as quartzites and granites.
- If the deformations occur more slowly, *squeezing* takes place. It acts as slow inward movements of the tunnel surface in *crushed or highly jointed* rocks or in *deformable, flexible or ductile* rocks such as soapstone, evaporites, clayey rocks (mudstones, clay schist, etc.) or weak schists.

Thus, in overstressed, massive rocks the deformation properties and/or the stiffness of the rock material often determines whether bursting or squeezing will take place.

4.3.1 Rock burst and spalling in brittle rocks

Rock burst is also known as *spalling*[2)] or *popping*, but also a variety of other names are in use, among them 'splitting' and 'slabbing'. It often takes place at depths in excess of 1,000 m below surface, but can also be induced at shallow depth where high horizontal stresses are acting. Selmer-Olsen (1964) and Muir Wood (1979) mention that great differences between horizontal and vertical stresses will increase rock burst activity. Selmer-Olsen (1964, 1988) has experienced that in the hard rocks in Scandinavia such anisotropic stresses might cause spalling or rock burst in tunnels located inside valley sides steeper than 20° and with the top of the valley reaching higher than 400 m above the level of the tunnel.

Rock burst failures can consist of small rock fragments or slabs of many cubic metres. The latter may involve the movement of the whole roof, floor or both walls. These failures do not involve progressive failures, except for very heavy rock burst. They cause, however, often significant problems and reduced safety for the tunnel crew during excavation.

Hoek and Brown (1980) have made studies of the stability of tunnels in various types of massive quartzites in South Africa. In this region the ratio between horizontal and vertical stress is $k = p_h/p_v = 0.5$. The tangential stresses in the walls of the squared

---

[2] Terzaghi (1946), Proctor (1971) and several other authors use the term 'spalling' fo *"any drop off of spalls or slabs of rock from tunnel surface several hours or weeks after blasting"*.

tunnels where the main stability problems occurred, will be $\sigma_\theta \approx 1.4\ p_z$ as outlined in Appendix II. Thus, the rock burst activity can be classified as:

$\sigma_c/\sigma_\theta > 7$	Stable
$\sigma_c/\sigma_\theta = 3.5$	Minor (sidewall) spalling
$\sigma_c/\sigma_\theta = 2$	Severe spalling
$\sigma_c/\sigma_\theta = 1.7$	Heavy support required
$\sigma_c/\sigma_\theta < 1.4$	Severe (sidewall) rock burst problems.

Similarly, based on Russenes (1974), who used measured point load strength (Is) of intact rock, the following classification has been found for horseshoe shaped tunnels: [3]

$\sigma_c/\sigma_\theta > 4$	No rock spalling activity
$\sigma_c/\sigma_\theta = 4 - 3$	Low rock spalling activity
$\sigma_c/\sigma_\theta = 3 - 1.5$	Moderate rock spalling activity
$\sigma_c/\sigma_\theta < 1.5$	High rock spalling/rock burst activity

As seen, these results fit relatively well with the results of Hoek and Brown.

Later, Grimstad and Barton (1993) made a compilation of rock stress measurements and laboratory strength tests and arrived at the following relation, which supports the findings of Hoek and Brown as well as Russenes:

$\sigma_c/\sigma_\theta > 100$	Low stress, near surface, open joints
$\sigma_c/\sigma_\theta = 3 - 100$	Medium stress, favourable stress condition
$\sigma_c/\sigma_\theta = 2 - 3$	High stress, very tight structure. Usually favourable to stability, maybe unfavourable to wall stability
$\sigma_c/\sigma_\theta = 1.5 - 2$	Moderate slabbing after > 1 hour
$\sigma_c/\sigma_\theta = 1 - 1.5$	Slabbing and rockburst after minutes in massive rock
$\sigma_c/\sigma_\theta < 1$	Heavy rockburst (strain-burst) and immediate dynamic deformations in massive rock

The value for σ_c referred to above is related to the compressive strength of 50 mm thick samples. In massive rock masses the block size is significantly larger - in the range 1 - 15 m³ for which the factor for scale effect of compressive in eq. (23) is in the range $f_\sigma = 0.45 - 0.55$. From this eq. (22) is roughly $\text{RMi} \approx 0.5\sigma_c$. Thus, the values of RMi/σ_θ in Table 5 are half of the values given for σ_c/σ_θ above.

TABLE 5 CHARACTERIZATION OF FAILURE MODES IN BRITTLE, MASSIVE ROCK (from Palmström 1995a)

Competency factor $Cg = f_\sigma \cdot \sigma_c/\sigma_\theta = \text{RMi}/\sigma_\theta$	**FAILURE MODES** **in massive, brittle rocks**
> 2.5	No rock stress induced instability
2.5 - 1	High stress, slightly loosening
1 - 0.5	Light rock burst or spalling
< 0.5	Heavy rock burst

Ideally, the strength of the rock should be measured in the same direction as the tangential stress is acting. Strength anisotropy in the rock may, however, cause that the values of the competency factor in Table 5 may not always be representative.

[3] The uniaxial compressive strength (σ_c) has been calculated from the point load strength (Is) using the correlation $\sigma_c = 20$ Is.

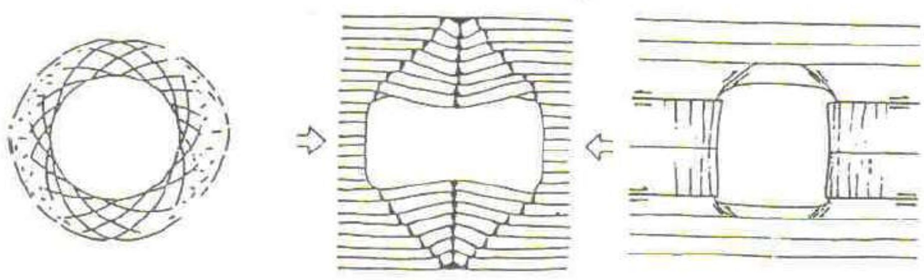
In Scandinavia, tunnels with spalling and rock burst problems are mostly supported by shotcrete (often fibre reinforced) and rock bolts, as this has practically been found to be most appropriate means of confinement. This general trend in support design is shown in Table 6. Earlier, wire mesh and rock bolts in addition to scaling, were used as reinforcement in this type of ground. This is only occasionally applied today in Norwegian tunnels.

TABLE 6 ROCK SUPPORT APPLIED IN NORWEGIAN TUNNELS UP TO APPROXIMATELY 15 m SPAN SUBJECTED TO ROCK BURST AND SPALLING (from Palmström 1995a)

Stress problem	Characteristic behaviour	Rock support
High stresses	May cause loosening of a few fragments	Some scaling and occasional spot bolting
Light rock burst	Spalling and falls of thin rock fragments	Scaling plus rock bolts spaced 1.5 - 3 m
Heavy rock burst	Loosening and falls, often as violent detachment of fragments and platy blocks	Scaling + rock bolt spaced 0.5 - 2 m, plus 50 -100 mm thick fibre reinforced shotcrete

4.3.2 Squeezing in continuous ground

Squeezing in tunnels can be very large; according to Bhawani Singh et al. (1992) deformations as large as 17% of the tunnel diameter have been measured in India. The squeezing process can occur not only in the roof and walls, but also in the floor of the tunnel. A general opinion is that squeezing is associated with volumetric expansion (dilation), as the radial inward displacement of the tunnel surface develops. Einstein (1993) writes, however, that squeezing may also be associated with swelling. Examples of squeezing behaviour are shown in Fig. 7.



a Complete shear failure, b buckling failure, c tensile splitting shearing and sliding

Fig. 7 Main types of failure modes in squeezing ground (from Aydan et al., 1993).

Fig. 8 shows the experience gained from practical studies made by Aydan et al. (1993) from studies of squeezing in 21 Japanese tunnels located in mudstones, tuffs, shales, serpentinites and other 'ductile' rocks with compressive strength $\sigma_c < 20$ MPa. No description of the rocks is presented in their paper. In the following it is assumed that the rocks contain relatively few joints as the presence of joints is not mentioned.

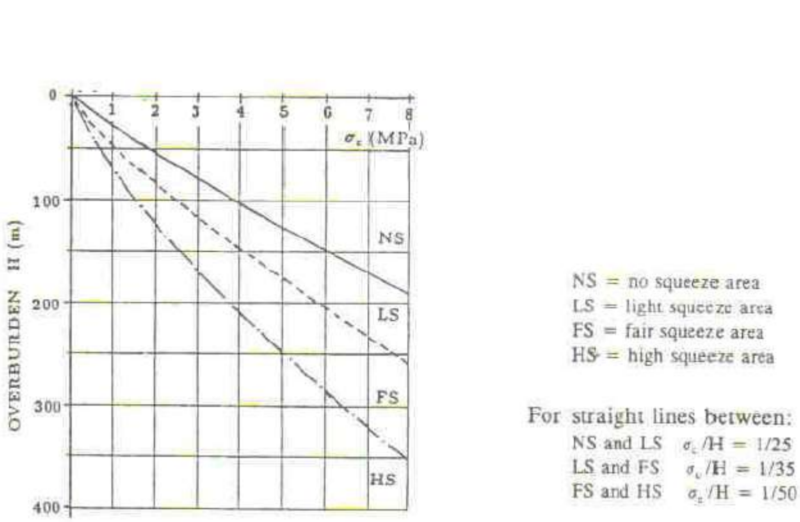


Fig. 8 A chart for estimating the possibility for squeezing (from Aydan et al., 1993)

TABLE 7 CHARACTERIZATION OF GROUND AND SQUEEZING ACTIVTY (based on Aydan et al., 1993)

Squeezing class	The tunnel behaviour according to Aydan et al. (1993)
No squeezing RMi $/\sigma_\theta > 1$	The rock behaves elastically and the tunnel will be stable as the face effect ceases.
Light squeezing RMi $/\sigma_\theta = 0.7 - 1$	The rock exhibits a strain-hardening behaviour. As a result, the tunnel will be stable and the displacement will converge as the face effect ceases.
Fair squeezing RMi $/\sigma_\theta = 0.5 - 0.7$	The rock exhibits a strain-softening behaviour, and the displacement will be larger. However, it will converge as the face effect ceases.
Heavy squeezing RMi $/\sigma_\theta = 0.35^{*)} - 0.5$	The rock exhibits a strain-softening behaviour at much higher rate. Subsequently, displacement will be larger and will not tend to converge as the face effect ceases.
Very heavy squeezing RMi $/\sigma_\theta < 0.35^{*)}$	The rock flows which will result in the collapse of the medium and the displacement will be very large and it will be necessary to re-excavate the tunnel and install heavy supports.

*) This value has been assumed

Applying a simplification with straight lines instead of the slightly curved ones in Fig. 8 the division given in Table 7 has been found. In this evaluation the following assumptions have been made:

- $k = p_h/p_v = 1$ and $p_v = \gamma \cdot z = 0.02z$ (Aydan et al. found $\gamma = 18 - 23\ MN/m^3$)
- Circular tunnels for which $\sigma_\theta/p_v \approx 2.0$ in roof (see Appendix II)
- The two expressions above are combined into $\sigma_c/z = (2 \cdot 0.02)\sigma_c/\sigma_\theta$. It is probable that the scale effect of compressive strength has been included in Fig. 8; therefore σ_c has been replaced by RMi, and the values for the ratio RMi $/\sigma_\theta$ in Table 7 have been found. This table is based on a limited amount of results from massive rocks and should, therefore, be updated when more data from practical experience in squeezing ground - especially in highly jointed ground - can be made available.

Based on the ground response curves presented by Seeber et al. (1978) the deformations and rock support in squeezing ground may roughly be as shown in Table 8.

TABLE 8 CONVERGENCE AND ROCK SUPPORT IN SQUEEZING GROUND (based on Seeber et al., 1978)

NATM ÖNORM B 2203 (1983)	English term	Approx. convergence and rock support according to Seeber et al. (1978) for tunnels with diameter 12 m			
		Without support	With support installed		
		Convergence range	Convergence range	Support pressure	Possible rock support
Stark gebräch oder druckhaft	Squeezing or swelling	min. 2 · 5 cm = 10 cm - - - - - max. 2 · 30 cm = 60 cm	2 · 3 cm = 6 cm - - - - - 2 · 5 cm = 10 cm	0.2 MPa - - - - - 0.7 MPa	bolts [1] spaced 1.5 m - - - - - bolts [1] spaced 1.5 m shotcrete 10 cm
Stark druckhaft	Heavy squeezing or swelling	min. 2 · 40 cm = 80 cm - - - - - max. > 2 m	2 · 10 cm = 20 cm - - - - - 2 · 20 cm = 40 cm	0.8 MPa - - - - - 1.5 MPa	bolts [1] spaced 1 m shotcrete 10 cm - - - - - - bolts [2] spaced 1 m shotcrete 20 cm

[1] bolt length 3 m [2] bolt length 6 m

4.4 Stability and rock support in discontinuous (jointed) materials

The principle of the method for evaluating rock support in discontinuous ground is shown in Fig. 9. The failures in this type of ground (jointed rock masses) occur when wedges or blocks, limited by joints, fall or slide from the roof or sidewalls. They develop as local sliding, rotating, and loosening of blocks and may occur in excavations at most depths. The properties of the intact rock are of relatively little importance as these failures, in general, do not involve development of fracture(s) through the rock (Hoek, 1981). The strength of the rock influences, however, often the wall strength of the joint and may in this way contribute to the stability.

The stability in jointed rock masses may be divided between instability of an individual block and cases in which failure involve two or more blocks. [4]

As the condition, orientation, frequency and location of the joints in the rock mass relative to the tunnel are the main controlling factors, the stability can generally not be predicted by equations derived from theoretical considerations (Deere et al., 1969). A common solution is to apply charts or tables in which the experienced amount and types of support are found from combination of rock mass and excavation parameters. This principle has been applied among others in the Q and the RMR systems.

[4] The key block method may be used as analysis in this group as it applies knowledge of orientation and condition of significant, joints and weakness planes in the rock mass; refer to Goodman (1989) and to Hoek and Brown (1980).

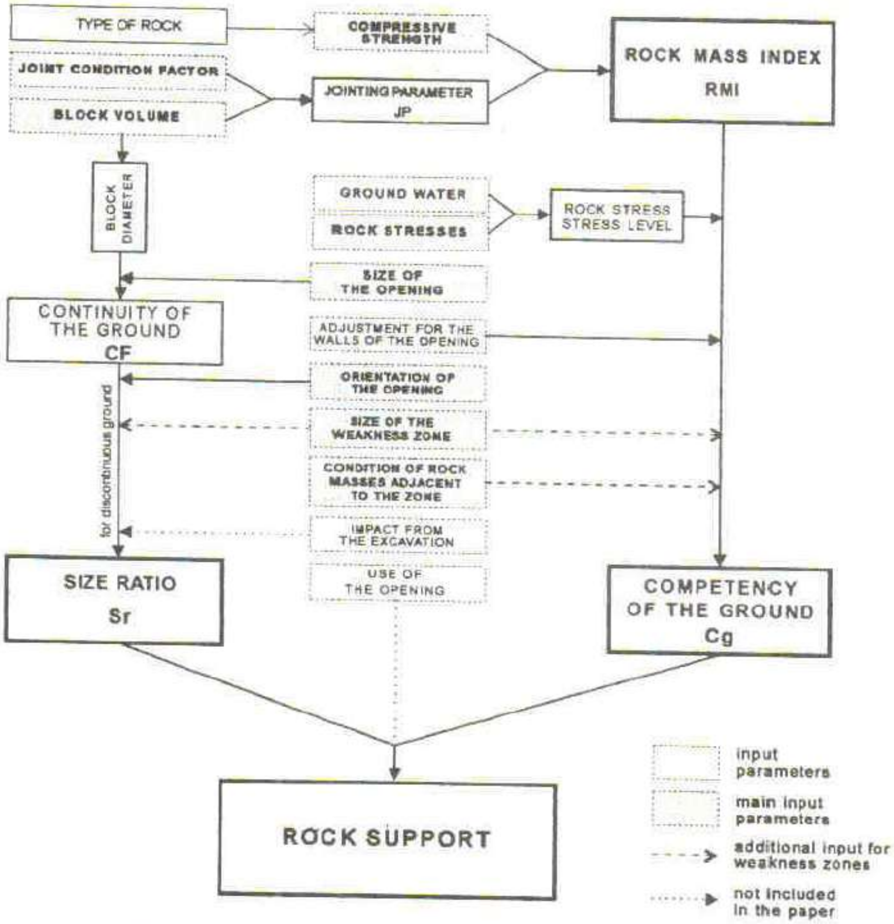


Fig. 9 The parameters involved in stability and rock support assessment in discontinuous ground. For weakness zones the size ratio and the ground condition factor are adjusted for parameters of the zone as indicated (from Palmström, 1995a).

4.4.1 *The ground condition factor (Gc) in discontinuous ground*

The ground condition factor in discontinuous ground includes the *inherent* rock mass features of main influence on stability and the *external* stresses acting. It is expressed as

$$\mathbf{Gc = RMi \cdot SL \cdot C} \qquad \text{eq. (25)}$$

RMi represents the inherent features in the rock mass (see Appendix I)

SL is the stress level factor, expresses the contribution from the external forces acting. In addition to the inherent properties the stability is influenced by the stresses acting across the joints in the rock masses surrounding the tunnel. A relatively high stress level will contribute to a 'tight structure' with increased shear strength along joints and, hence, increased stability. This has often been observed in deep tunnels. For the same reason a low stress level is unfavourable to stability. This effect is frequently seen in portals and tunnels near the surface where the low stress level often is 'responsible' for loosening and falls of blocks.

TABLE 9 THE RATINGS OF THE STRESS LEVEL FACTOR (SL)
(from Palmström, 1995a)

Term	Maximum stress σ_1	Approximate overburden (for $k \approx 1$)	Stress level factor (SL)*)	
				average
Very low stress level (in portals etc.)	< 0.25 MPa	< 10 m	0 - 0.25	0.1
Low stresses level	0.25 - 1 MPa	10 - 35 m	0.25 - 0.75	0.5
Moderate stress level	1 - 10 MPa	35 - 350 m	0.75 - 1.25	1.0
High stress level	> 10 MPa	> 350 m	1.25**) - 2.0	1.5**)

*) In cases where ground water pressure is of importance for stability, it is suggested to:
- divide SL by 2.5 for moderate influence
- divide SL by 5 for significant influence

**) A high stress level may be unfavourable for stability of high walls, SL = 0.5 - 0.75 is suggested

However, in a jointed rock mass containing variable amount of joints with different orientations, it is not possible in a simple way to calculate and incorporate the detailed effect from the stresses. The Q-system uses a 'stress reduction factor' (SRF) for this effect. Similarly for RMi, a general stress level factor (SL) has been chosen as a very simple contribution of the stresses on the shear strength. As increased stress level has a positive influence on stability in discontinuous ground the stress level factor (SL) forms a multiplication factor. Its ratings in Table 9 are roughly based on SL = 1/SRF.

The influence of joint water pressure is generally difficult to incorporate in the stress level. Often, the joints around the tunnel will drain the ground water in the volumes nearest to the tunnel, hence the influence from ground water pressure on the effective stresses is limited. The *total stresses* have, therefore, been selected. In some cases, however, where unfavourable orientation of joints combined by high ground water pressure will tend to reduce the stability by extra loading on key blocks, the stress level factor should be reduced as shown in Table 9.

C is a factor adjusting for the obvious greater stability of a vertical wall compared to a horizontal roof. Milne et al. (1992) have introduced a gravity adjustment factor to compensate for this.[5] Based on Milne et al. (1992) the ratings are found from

$$C = 5 - 4 \cos\beta \qquad \text{eq. (26)}$$

where β = angle (dip) of the surface from horizontal.
(C = 1 for horizontal roofs, C = 5 for vertical walls.)

[5] Similarly, Barton (1975) has applied a wall/roof adjustment factor of the Q-value. This factor depends, however, on the quality of the ground, having a value of 5 for good quality (Q > 10), 2.5 for medium (Q = 0.1 - 10), and 1.0 for poor quality ground (Q < 0.1).

Possible instability induced from high ground stresses.

As earlier mentioned, the experience is that rock bursting is less developed in jointed rock than in massive rock under the same stress level. At depths where the stresses developed around the excavation may exceed the strength of the rock mass, both stress induced and structurally controlled failures may occur simultaneously. According to Hoek (1981) one of these two forms, tends to dominate at a particular site.

Terzaghi (1946) describes this type of stress controlled failures in jointed rock as *"If the rock masses around the tunnel is in a state of intense elastic deformation, the connections or interlocks between blocks such as A and B in Fig. 10 and their neighbours, may suddenly snap, whereupon the block is violently thrown into the tunnel. If such an incident occurs, it is necessary to provide the tunnel with the support prescribed for popping."*

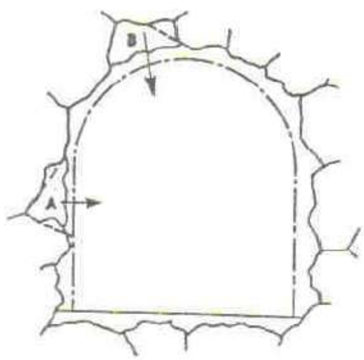


Fig. 10 Possible instability in jointed rock masses exposed to a high rock stress level (from Terzaghi, 1946).

Little information has, however, been found in the literature on this effect. Barton (1990) has experienced that *"if jointing is present in highly stressed rock, extensional strain and shear strain can be accommodated more readily and are partially dissipated."* The result is that stress problems under high stress levels are less in jointed than in massive rock. This has also been clearly shown in tunnels where destress blasting is carried out in the tunnel periphery with the purpose to develop additional cracking and in this way reducing the amount of rock bursting.

In moderately to slightly jointed rock masses subjected to high stress level compared to the strength of intact rock, cracks may develop in the blocks and cause reduced stability from the loosening of fragments. This phenomenon has been observed by the author in the Thingbæk chalk mine in Denmark with σ_c = 1 - 3 MPa.

4.4.2 The size ratio

The size ratio includes the dimension of the blocks and the underground opening. It is meant to represent the geometrical conditions at the actual location. The size ratio for discontinuous (jointed) rock masses is expressed as:

$$Sr = (Dt / Db)(Co / Nj) \qquad \text{eq. (27)}$$

Dt is the diameter (span or wall height) of the tunnel.

Db is the block diameter represented by the smallest dimension of the block which often turns out to be the spacing of the main joint set. Often the *equivalent block diameter* is applied where joints do not delimit separate blocks (for instance where less than 3 joint sets occur). In these cases Db may be found from the following expression which involves the block volume (Vb) and the block shape factor (β) as shown by Palmström (1995a, 1995d): [6]

$$Db = (27/\beta)\sqrt[3]{Vb} \qquad \text{eq. (28)}$$

N_j is a factor representing the number of joint sets as an adjustment to Db in eq. (24) where more or less than three joint sets are present. As decribed by Barton et al. (1974) the degree of freedom determined by the *number of joint sets* significantly contributes to stability. The value of N_j is found from the expression

$$Nj = 3/n_j \qquad \text{eq. (29)}$$

where nj = the number of joint sets (n_j = 1 for one set; nj = 1.5 for two sets plus random joints; nj = 2 for two sets, etc.)

Co is an orientation factor factor representing the influence from the *orientation* of the joints on the block diameter encountered in the underground opening. Joints across the opening will have significantly less influence on the behaviour than parallel joints. The ratings of Co shown in Table 10 are based on Bieniawski (1984) and Milne et al. (1992). The strike and dip are measured relative to the tunnel axis. As the jointing is three-dimensional, the effect of joint orientation is often a matter of judgement, often the orientation of the main joint set is has the main influence and is applied to determine Co.

TABLE 10 THE ORIENTATION FACTOR FOR JOINTS AND ZONES (from Palmström, 1995a, based on Bieniawski, 1984).

IN WALL		IN ROOF	TERM	**Rating of orientation factor (Co)**
for strike > 30°	for strike < 30°	for all strikes		
dip < 20°	dip < 20°	dip > 45°	favourable	1
dip = 20 - 45°	dip = 20 - 45°	dip = 20 - 45°	fair	1.5
dip > 45°	-	dip < 20°	unfavourable	2
-	dip > 45°		very unfavourable	3

[6] The block shape factor (β) has been described by Palmström (1995a, 1995d). The ratio $27/\beta$ has been chosen as a simple expression to find the smallest block diameter Eq. (28) is most appropriate for $\beta < 150$. For higher values of β a dominating joint set will normally be present for which the average joint spacing may be used.

4.5 Stability and rock support of faults and weakness zones

Weakness zones consist of rock masses having properties significantly poorer than those of the surrounding ground. Included in the term weakness zones are faults, zones or bands of weak rocks in strong rocks, etc. Weakness zones occur both geometrically and structurally as special types of rock masses in the ground. The following features in the zones are of main importance for stability:

1. *The geometry and dimensions of the zone.*
   The instability and problems in weakness zones will generally increase with the width of the zone. However, this feature should always be assessed in relation to the quality and structure of the adjacent rock mass, and the existence of adjacent seams or faults (if any).
   The orientation of the zone relative to the tunnel can have a considerable influence on the stability of the opening. As for joints, the problems in general increase as the strike becomes more parallel to the opening and when the zone is low-dipping. This comes also from the fact that for such orientation the zone affects the tunnel over a longer distance.
2. *The reduced stresses in the zone compared to the overall ground stresses.*
   An important effect in weakness zones is the fact that the stresses in and near the zone will be other than in the surrounding rock masses. Selmer-Olsen (1988) has experienced that faults and weakness zones may cause large local variations in the rock stresses. Although the overall stresses in an area may indicate that a weakness zone should be overstressed and behave as incompetent (squeezing) ground when encountered in an excavation, this will often not be the case. The reason is the greater deformability in the zone and transfer of stresses onto the adjacent rock masses. Failures in weakness zones will, therefore, seldom be squeezing, but gravity induced. Very wide zones, however, are expected to have stresses and behaviour equal to those of the surrounding ground.
3. *The arching effect from the ground surrounding the weakness zone.*
   Terzaghi (1946) explained that the the rock load on the roof support, even in sand and in completely crushed rock, is only a small fraction of the weight of rock located above the tunnel because of the *arch action* or silo effect. Where the width of the zone is smaller than the tunnel diameter, additional arch action from the stronger, adjacent rock masses leads to reducing of rock load on the support compared to that of a rock mass volume with the same composition.
4. *Possible occurence of swelling, sloughing, or permeable materials in the zone.*
   As mentioned in Paragraph 4.1, these features often depend on the geometry and the site conditions. They have, therefore, not been included in this general layout.

The composition of weakness zones and faults can be characterized by RMi and/or by its parameters. The mataterial in many weakness zones occur as continuous materials in relation to the the size of the tunnel, and may be considered as such in calculations. However, the system presented for discontinuous (jointed) rock masses in Paragraph 4.3, has been found to also cover many types of zones where the size ratio and the ground condition factor are adjusted for zone parameters as shown in Fig. 9.

4.5.1 The ground condition factor for zones

As mentioned, stability is influenced by the interplay between the properties of the zone and the properties of the adjacent rock masses, especially for small and medium sized zones. The inherent features of both can be characterized by their respective qualities. Löset (1990) has presented a method to combine the conditions in the zone and in the adjacent rock masses in the following expression:

$$\log Q_m = (Tz \cdot \log Q_z + \log Q_a)/(Tz + 1) \qquad \text{eq. (30)}$$

where Tz = the width of the zone in metres, Q_z = the quality of the zone, and Q_a = the quality of the adjacent rock masses.

In this expression Q can be replaced by RMi. As an alternative to the complicated eq. (26) Palmström (1995a) has presented a simplified expression

$$RMi_m = (10Tz^2 \cdot RMi_z + RMi_a)/(10Tz^2 + 1) \qquad \text{eq. (31)}$$

For larger zones the effect of stress reduction from arching is limited; the ground condition for such zones should therefore be that of the zone ($RMi_m \approx RMi_z$). This is assumed to take place for zones smaller than $Tz = 20$ m as is found from eq. (31). Applying eq. (31) a ground condition factor for zones can be found for weakness zones similar to that for discontinuous (jointed) rock masses.

$$Gc_z = SL \cdot RMi_m \cdot C \qquad \text{eq. (32)}$$

Palmström (1995a) discusses whether the stress level factor (SL) should be included in the ground condition factor (Gc_z) for zones, since in zones the stresses are often lower than those in the adjacent rock masses. A rating of SL = 1 may include most cases. However, sometimes SL may influence the shear strength (and hence the stability) along the joints in zones. Another argument for including SL is to maintain simplicity by applying similar expressions for Gc and Gc_z .

4.5.2 The size ratio for zones

As mentioned in the beginning of this paragraph there is increased arching effect in weakness zones compared to the overall rock mass for zones with thickness less than approximately the diameter (span) of the tunnel. For such zones the size ratio Sr = Co (Dt/Db) is adjusted for the zone ratio Tz/Dt to form the size ratio for zones [7]

$$Sr_z = \frac{Tz}{Db_z} \cdot Co_z \cdot Nj_z \qquad \text{eq. (33)}$$

Here Co_z = factor for the orientation of the zone with ratings as shown in Table 11
Db_z = the diameter of the representative blocks in the zone
Nj_z = the adjustment factor for joint sets in the zone found from eq. (29)

Eq. (33) is valid where Tz is smaller than the diameter (span or height) of the tunnel. For thicker zones eq. (27) should be applied [Sr = (Co/Db)Dt · Nj].

[7] This ratio is applied provided $Tz/Db_{zone} < Dt/Db_{adjacent}$

4.6 Support chart

The support chart in Fig 11 covers most types of rock masses. Is is worked out from the author's experience backed by description of 24 cases from Norwegian and Danish tunnels. The compressive strength of the rocks in these cases varies from 2 to 200 MPa and the degree of jointing from crushed to massive. From use during two years the application of RMi in stability and support calculations seems very promising. In squeezing ground the chart is based on Table 9. Work still remains, however, to develop more adequate support chart for this type of ground.

The support chart is based on the condition that loosening and falls which may involve blocks or large fragments should be avoided. Appropriate timing of installation of rock support is a prerequisite for applying the charts. In this connection it should again be pointed out that, as the loosening or failures in jointed rock is mainly geometrically related - i.e. influenced by the orientation and location of each joint - it is impossible to develop a precise support chart.

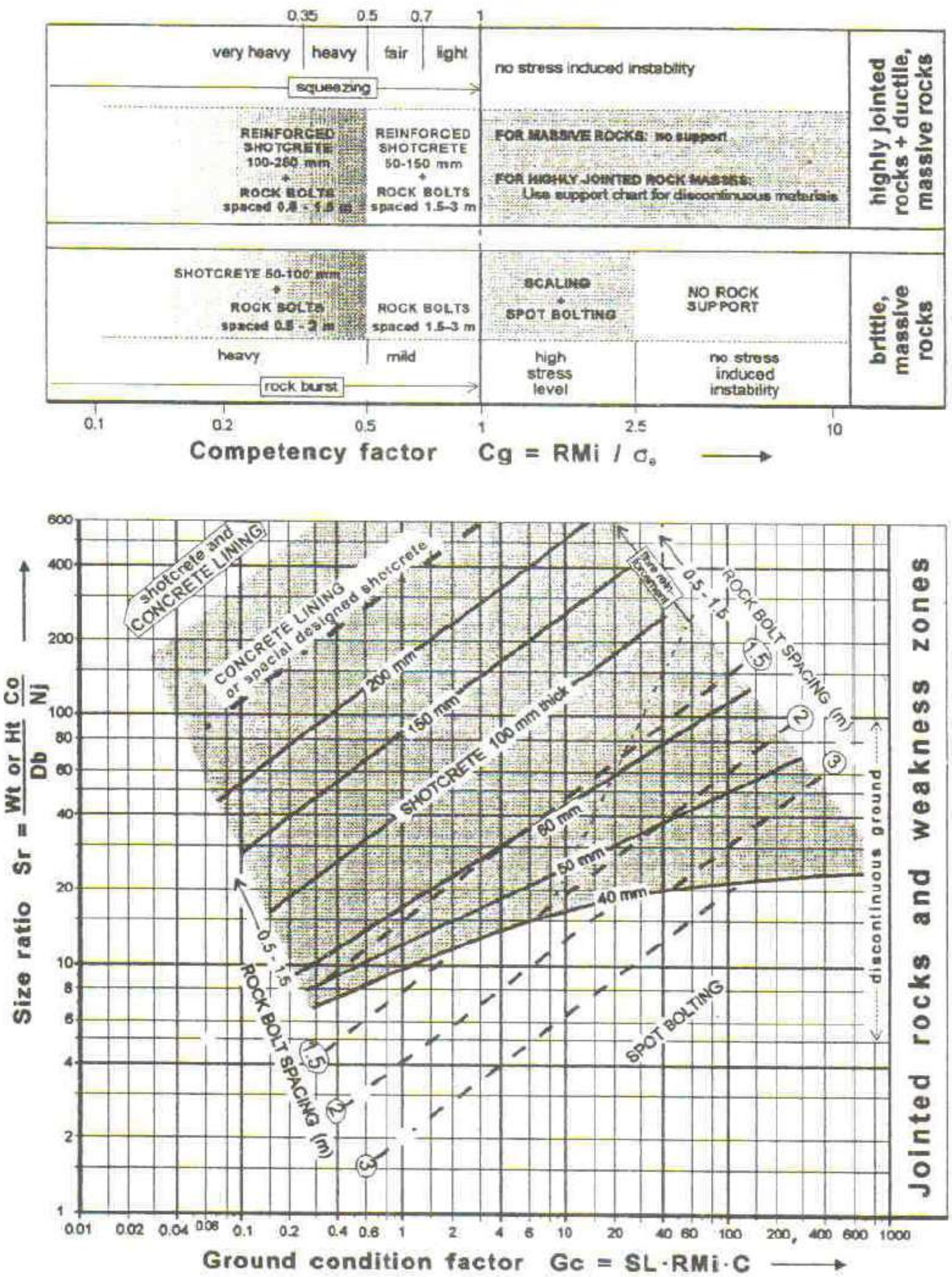
A support chart can generally only indicate the average amount of rock support. It may, therefore, be considered as an expression for the 'statistical average' of appropriate rock support. Further, a support chart can only give the amount and methods for support based on the support regulations and experience in the region. In other regions where other methods and applications have been developed, other support charts should be worked out based on the current practice and the principles applied for rock support.
The required stability level and amount of rock support is determined from the utility of the underground opening. The Q-system uses the ESR (excavation support ratio) as an adjustment of the span to include this feature. From the current practice in underground excavations, however, the author is of the opinion that it is difficult to include various requirements for stability and rock support in a single factor. For example, the roof in an underground power houses will probably never be left unsupported even for a Q-value higher than 100. Also, in large underground storage caverns in rock the roof is generally shotcreted before benching, because, in the 30 m high caverns, falls of even small fragments may be harmful to the workers. As a result of this, a chart should preferably be worked out for each main category of excavation.

To simplify and limit the size of the support diagram $Vb = 10^{-6}$ m^3 $(= 1\ cm^3)$ has been chosen as the minimum block (or fragment) size. This means that where smaller particles than this (being og medium gravel size) occur, $Vb = 1\ cm^3$ or block diameter $Db \approx 0.01$ m is used.

Roughly, for *'common' hard rock mass conditions*, i.e. $RMi = 40\ \sqrt[3]{Vb}$ (for $jC = 1.75$) three joint sets ($Nj = 3/n_j = 1$), block shape factor $\beta = 40$, fair joint orientation ($Co = 1.5$), and moderate stress level ($SL = 1$), the following expressions can be applied

The ground condition factor $\mathbf{Gc = RMi \cdot SL \cdot C = 0.25\ \sigma_c\ C\ \sqrt[3]{Vb}}$
The size ratio $\mathbf{Sr = Wt \cdot Nj \cdot Co/Db = Wt\ / \sqrt[3]{Vb}}$ **or** $\mathbf{Sr = Ht\ / \sqrt[3]{Vb}}$

where $C = 1$ for horizontal roofs, $C = 5$ for vertical walls,
Wt = width (span) and Ht = (wall) height of tunnel



For weakness zones:

$Sr_z = Tz \cdot Co_z / Db_z$ for zones where $Sr_z < Sr$ and $Tz < Wt$ or $Tz < Ht$ else $Sr_z = Sr$

$Gc_z = SL \cdot RMi_m \cdot C$ where $RMi_m = (10\,Tz^2 \cdot RMi_z + RMi_s) / (10\,Tz^2 + 1)$

Fig. 11 Rock support charts for continous and discontinuous ground. The support in continuous ground is for tunnels with diameter Dt < 15 m. (The diagram for squeezing in particulate materials is based on limited amount of data.) (from Palmström, 1995a) ground condition factor (Gc). Knowing or estimating the change in block size from excavation it is, therefore, easy to calculate the adjusted values for (Sr) and (Gc) and thus include the impact from excavation in the assessments of rock support.

Mathematical expression have been developed for all the parameters characterizing the ground as well as the other input features included in the stability and the rock support assessment. This makes the use of computers favourable to calculate the factors used in the support chart. This is shown in Table 11.

5 RMi USED TO QUANTIFY THE DESCRIPTIVE GROUND CLASSIFICATION IN THE NATM

"Paradoxically, the excavation of an underground opening in a highly stressed environment is likely to be less hazardous when the rock is jointed than when it is intact." Nick Barton (1990)

The principles of the new Austrian tunnelling method (NATM) are shown in Fig. 12.

—Key features of the New Austrian Tunneling Method—

Professor Rabcewicz, of Salzburg, Austria, has been one of the chief developers of the New Austrian Tunneling 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 Tunneling 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 lining is not needed—e.g. for water conduits, short highway tunnels.

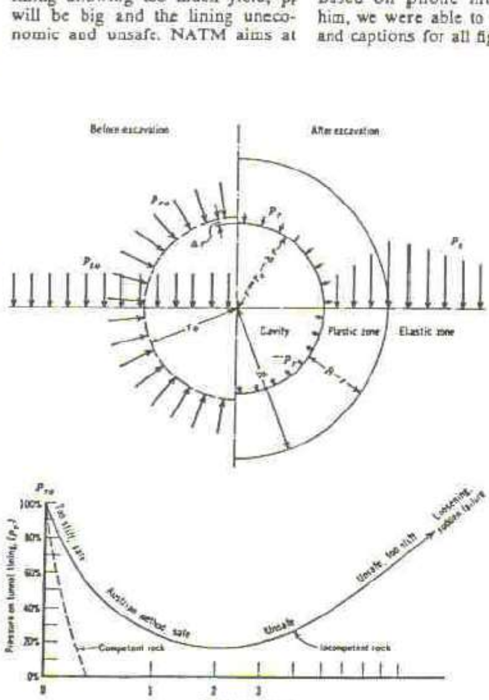
• It involves installation of sophisticated instrumentation at the time the initial shotcrete lining is placed, to provide info for designing a second inner lining.

• 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 $(r_o - \Delta r)$. If tunneling is in competent hard rock, a stable equilibrium is reached. For less 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 its theoretical minimum value.

(Our thanks to Mr. Herbert Nussbaum, an engineering consultant and expert on tunneling from West 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 p_r 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 increases to max a short distance from cavity.

Fig. 12 The main ideas and principles of NATM (from Rabcewicz, 1975).

Table 12 shows the descriptive classification used in the NATM. Class 1 refers to massive and lightly jointed competent rock masses, class 2 and 3 to moderately and strongly jointed rock masses, while class 5 and 6 are related to squeezing from overstressing as described in Table 7 and/or swelling of rocks. The NATM classification is mainly based on the behaviour of the ground observed in the excavated tunnel. The various classes can also be assessed from field observations of the rock mass composition and estimates of the rock stresses. The ground is mainly characterized on an individual basis, based on personal experience (Kleeberger, 1992).

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 possibility to improve the input parameters used in design works of NATM projects.

The NATM uses the Fenner-Pacher diagram, which is similar to the ground reaction curve outlined in Section 3, for calculation of the ground behaviour and rock support determination.

TABLE 12 THE CLASSIFICATION OF GROUND BEHAVIOUR APPLIED IN ÖNORM B 2203 (1993)

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)

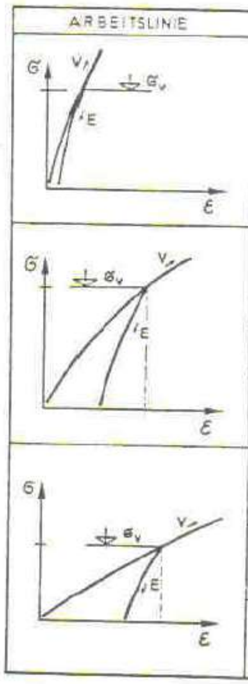
5.1 The use of RMi in NATM classification

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

1. The 'Gebirgsfestigkeitsklassen' ('rock mass strength classes') based on the shear strength properties of the rock mass. This group can be compared with RMi, but the input parameters are different. Fig. 13 shows that it is possible to use the shear strength parameters found in Paragraph 2.4 to determine these data, as they consist of rock mechanics data characterized by two of the following parameters:
   - the friction angle of the rock mass (Φ), found from eq. (10);
   - the cohesion of rock mass (c), which can be foundfrom eq. (12); and/or
   - the modulus of elasticity (E) and the modulus of deformation (V).

   Preliminary, from an ongoing work is to estimate the modulus of deformation from the RMi value, the following expression has been found: [8]

$$E = 5.6\ \mathbf{RMi}^{0.375} \qquad \text{eq. (38)}$$



GEBIRGS-FESTIG-KEITS-KLASSE	E_{el} / V_{el} / V_{pl} (N/cm²)	φ (Grad)	c_{el} / c_{pl} (N/cm²)
1	1000 000 1000 000 500 000	55 50 45	1000 10
2	800 000 800 000 400 000	50 45 40	800 10
3	1000 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

1 N/cm2 = 10 kPa

Fig. 13 Rock mass strength classes ('Gebirgsfestigkeitsklassen') as applied by Seeber et al. (1978)

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 shown in Table 12.

---

[8] This equation has been found from the correlation $RMi = 10^{(RMR - 40)/15}$ between RMR and RMi (Palmström, 1995a) and $E = 10^{(RMR - 10)/40}$ (Serafim and Pereira, 983)

By combining the rock mass strength classes ('Gebirgsfestigkeitsklassen') in Fig. 13 with rock stresses from overburden the actual NATM class is found from Fig. 14. Using the RMi characterization directly, Table 13 may be applied. More work remains, however, to check the suggested values in this table.

TABLE 13 SUGGESTED NUMERICAL DIVISION OF GROUND ACCORDING TO NATM CLASSIFICATION

NATM class		Rock mass properties (JP = jointing parameter)	Competency factor ($Cg = RMi/\sigma_\theta$)
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.)

In this way, the NATM classes can be determined from numerical rock mass characterizations. NATM may effectively benefit from this contribution, especially in the planning stage of tunnelling projects before the behaviour of the rock masses can be studied in the excavation.

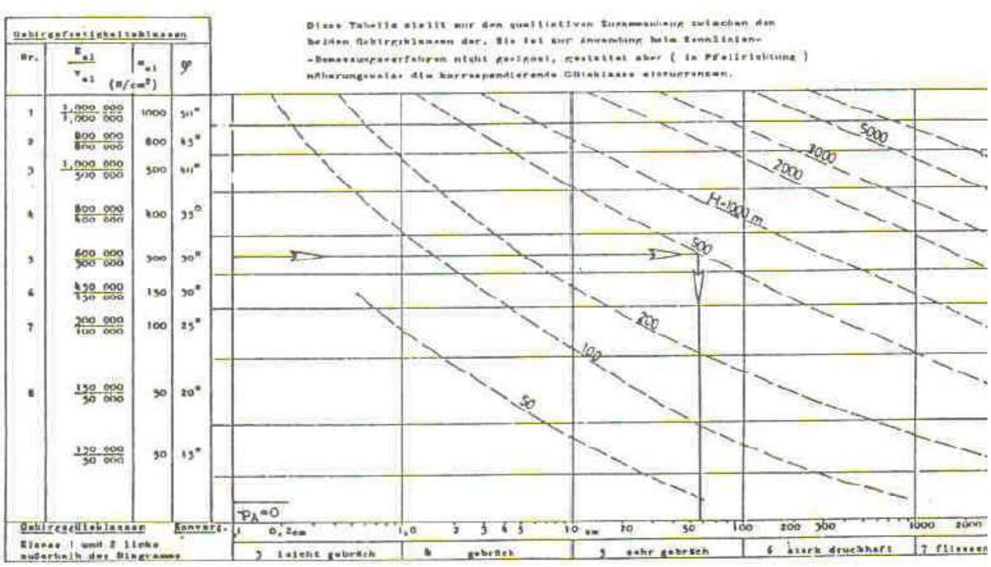


Fig. 14 The numerical characterization of the ground into NATM classes used by Seeber et al. (1978). NATM classes 1 and 2 are not included.

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 present a better accuracy. If, however, convergence measurements are available at a somewhat later date, the results from these can be used to improve the accuracy of the input parameters considerably.

6 DISCUSSION

> *"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)

As mentioned by Palmström (1995d) the RMi offers several benefits and possibilities in rock engineering and rock mechanics, as it expresses a general strength characterization of the rock mass. Therefore, it includes only the inherent parameters of the rock mass. Hence RMi and its parameters will introduce advantages and improved quality when used:

- for input to Hoek-Brown failure criterion for rock masses;
- for input to ground response curves; or
- to quantify the rock mass classification applied in the NATM;
- in stability and rock support assessments.

When applied in practical rock engineering the RMi is adjusted for the local features of main importance for the actual use, work or utility. Thus, a flexible system is applicable for many different purposes connected to rock construction as indicated in Fig. 1.

As for the Hoek-Brown failure criterion, RMi is - when applied directly in calculations - restricted to *continuous* rock masses. In discontinuous rock masses the use RMi must be adjusted for the local conditions. This has been shown in the application of RMi in design of rock support (Section 4) where discontinuous and continuous rock masses have been treated separately in the assessments.

6.1 The application of RMi in stability and rock support

The behaviour of continuous and discontinuous ground in underground openings is completely different which is reflected in the two approaches to assess the rock support. Common for both is the use of RMi to characterize the composition and inherent properties of the structural material (i.e. rock mass). The influence from stresses is, however, different for the two types. For continuous ground the magnitude of the tangential stresses (σ_θ) set up in the ground surrounding the opening is applied, while for discontinous ground a stress level factor has been selected.

In *continuous* ground the effect of ground water can be included in the effective stresses applied to calculate the tangential stresses set up in the rock masses surrounding the underground opening. In *discontinuous* ground the direct effect of ground water is often small, hence this feature has not been selected. It is, however, possible to adjust the stress level factor where water pressure has a marked influence on stability.

The block volume (Vb) is the most important parameter applied in the support charts, as it is included in the ground condition factor as well as in the size ratio. Great care should, therefore, be taken when this parameter is determined. Where less than three joint sets occur defined block are not formed. In these cases, methods have been shown by Palmström (1995a, 1995d) how to assess an equivalent block volume. An additional problem is to indicate methods for characterizing the variations in block size. Therefore, engineering calculations should, be based on a variation range.

The uniaxial compressive strength (σ_c) of the rock can, especially for support assessments of discontinuous (jointed) rock masses, often be found with sufficient accuracy from simple field tests, or from the name of the rock using standard strength tables in textbooks.

What is new in the RMi support method?

The method using RMi to determine rock support differs from the the existing classification systems for support. While these combine all the selected parameters to directly arrive at a quality or rating for the ground conditions, the RMi method applies an index to characterize the material, i.e. the rock mass. This index is then applied as input to determine the ground quality. The way the ground is divided into continuous and discontinuous materials and the intruduction of the size ratio (tunnel size/block size) are also new features in the RMi support method.

The application of the RMi in rock support involves a more systemized collection and application of the input data. RMi makes also use of a clearer definition of the different types of ground. It probably covers a wider range of ground conditions and includes more variables than the two main support classification systems, the RMR and the Q-system.

The structure of RMi and its use in rock support engineering allows for accurate calculations where high quality data are available. As shown in eqs. (30) to (33) it is also possible to apply simplified expressions for the ground conditions and size ratio when only rough support estimates are required. As this only requires input from the block volume, the support estimates can quickly be carried out.

Using the RMi in assessment of rock support may seem complicated at first glance. Possible beginner problems using the support chart should be relatively quickly overcome. Descriptions and collection of input data require, however, involvement of experienced persons, as is the case for most rock engineering projects.

Acknowledgement

This paper is part of the Ph.D thesis titled "RMi - A rock mass characterization system for rock engineering purposes" which has been worked out at the University of Oslo, Norway. The funding by the Norway Research Council (NFR) has made this work possible. I am most grateful for all support from the Norwegian Geotechnical Institute during my studies. Thanks also to mr. Rajinder Bhasin for valuable help.

7 REFERENCES

Aydan Ö., Akagi T. and Kawamoto T. (1993): "The squeezing potential of rocks around tunnels; theory and prediction." Rock Mech. Rock Engn., No. 26, pp. 137-163.

Barton, N., Lien, R. and Lunde, J. (1974): "Engineering classification of rock masses for the design of rock support." Rock Mechanics 6, 1974, pp. 189-236.

Barton N., Lien R. and Lunde J. (1975): "Estimation of support requirements for underground excavations. Proc. Sixteenth Symp. on Rock Mechanics, Minneapolis, pp. 163-177.

Barton N. (1990b): "Scale effects or sampling bias?" Proc. Int. Workshop Scale Effects in Rock Masses, Balkema Publ., Rotterdam, pp. 31-55.

Bhawani Singh, Jethwa J.L., Dube A.K. and Singh B. (1992): "Correlation between observed support pressure and rock mass quality." Tunnelling and Underground Space Technology, Vol. 7, No. 1, pp. 59-74.

Bieniawski, Z.T. (1973): "Engineering classification of jointed rock masses." Trans. S. African Instn. Civ. Engrs., Vol 15, No 12, Dec. 1973, pp 335 - 344.

Bieniawski Z.T. (1984): "Rock mechanics design in mining and tunneling." A.A. Balkema, Rotterdam, 272 pp.

Bieniawski Z.T. (1989): "Engineering rock mass classifications." John Wiley & Sons, New York, 251 pp.

Brekke T.L. and Howard T.R. (1972): "Stability problems caused by seams and faults." Rapid Tunneling & Excavation Conference, 1972, pp. 25-41.

Brosch F.J. (1986): "Geology and the classification of rock masses - examples from Austrian tunnels." Bull. IAEG no 33, 1986, pp 31 - 37.

Brown E.T., Bray J.W., Ladanyi B. and Hoek E. (1983): "Ground response curves for rock tunnels." J. Geot. Engn., Vol. 109, No. 1, pp. 15 - 39.

Deere D.U., Peck R.B., Monsees J.E. and Schmidt B. (1969): "Design of tunnel liners and support system." Office of high speed ground transportation, U.S. Department of transportation. PB 183799.

Einstein H.H. (1993): "Swelling rock." ISRM News, No. 2, pp. 57-60.

Grimstad E. and Barton N. (1993): "Updating the Q-system for NMT." Proc. Int. Symp. on Sprayed Concrete, Fagernes, Norway 1993, Norwegian Concrete Association, Oslo, 20 pp.

Hoek E. and Londe P. (1974): "General report, surface workings in rock." Proc. Third Int. Congr. on Rock Mech., Denver.

Hoek E. and Brown E.T. (1980): "Underground excavations in rock." Institution of Mining and Metallurgy, London 1980, 527 pp.

Hoek E.: (1981): "Geotechnical design of large openings at depth." Rapid Exc. & Tunn. Conf. AIME 1981.

Hoek, E. (1983): "Strength of jointed rock masses." The Rankine Lecture 1983, Geotechnique 33, no 3 pp 187-223.

Hoek E., Wood D. and Shah S. (1992): "A modified Hoek-Brown failure criterion for jointed rock masses." Proc. Int. Conf. Eurock '92, Chester, England, pp. 209-214.

Hoek E. (1994): "Strength of rock masses." News Journal of ISRM, Vol. 2, No. 2, pp. 4-16.

Hudson J.A. (1989): "Rock mechanics principles in engineering practice." CIRIA Ground Engineering report, 72 pp.

Jaeger J.C. (1969): "Behavior of closely jointed rock." Proc. 11th Symp. Rock Mech., pp. 57 - 68.

John K.W.: (1969): "Civil engineering approach to evaluate strength and deformability of regularly jointed rock." 11th int. symp. on rock mech. pp. 69-80

Kleeberger J. (1992): "Private communication."

Löset F. (1990): "Use of the Q-method for securing small weakness zones and temporary support." (in Norwegian) Norwegian Geotechnical Institute, internal report No. 548140-1, 40 pp.

Milne D., Germain P. and Potvin Y. (1992): "Measurement of rock mass properties for mine design." Proc. Int. Conf. Eurock '92, Thomas Telford, London, pp. 245-250.

Muir Wood A.M. (1979): "Ground behaviour and support for mining and tunnelling." Tunnels and Tunnelling, Part 1 in May 1979 pp. 43-48, and Part 2 in June 1979, pp. 47-51.

Mutschler T. (1993): "Private communication."

Nakano R. (1979): "Geotechnical properties of mudstone of Neogene Tertiary in Japan." Proc. Int. Symp. Soil Mechanics, Oaxaca, pp. 75 - 92.

Palmström A. (1995a): "RMi - A for rock mass characterization system for rock engineering purposes." Ph.D. thesis, University of Oslo, Norway, 400 pp.

Palmström A. (1995b): "Characterizing the strength of rock masses for use in design of underground structures." Int. Conf. on Design and Construction of Underground Structures; New Delhi, 1995, 10 pp.

Palmström A. (1995c): "Characterizing rock burst and squeezing by the rock mass index." Int. conf. on Design and Construction of Underground Structures; New Delhi, 1995, 10 pp.

Palmström A. (1995d): "RMi - a system for characterizing rock mass strength for use in rock engineering". J. Rock Mech. & Tunnelling Tech., Vol. 1, No. 2, 40 pp.

Rabcewicz L.v. (1964/65): "The new Austrian tunnelling method." Water Power, part 1 November 1964 pp. 511-515, Part 2 January 1965 pp. 19-24.

Rabcewicz L.v. (1975): "Tunnel under Alps uses new, cost-saving lining method." Civil Engineering-ASCE, October 1975, pp.66-68.

Seeber G., Keller S., Enzenberg A., Tagwerker J., Schletter R., Schreyer F. and Coleselli A. (1978): "Methods of measurements for rock support and installations in road tunnels using the new Austrian tunnelling method." (in Germain) Bundesministerium f. Bauten u. Technik, Strassenforschung Heft 133, 200 pp.

Serafim J.L. and Pereira J.P. (1983): "Considerations of the Geomechanics Classification of Bieniawski." Int. Symp. on Engn. Geol. and Underground Constr., Lisbon.

Terzaghi K. (1946): "Introduction to tunnel geology." In Rock tunnelling with steel supports, by Proctor and White, pp 5 - 153.

Terzaghi K. (1953): "Address". Proc. Int. Conf. on Soil Mech. and Foundation Engn., Vol. 3, p. 76.

Ward W.H. (1978): "Ground supports for tunnels in weak rocks." The Rankine Lecture. Geotechnique 28, No. 2, pp. 133-171.

Wood D. (1991): "Estimating Hoek-Brown rock mass strength parameters from rock mass classifications." Transportation Research Record 1330, pp. 22-29.

APPENDIX I

DETERMINATION OF THE RMi VALUE

Basically, the rock mass index expresses the effect various defects (discontinuities) in a rock mass have to reduce the strength of the inherent rock. This is given as

$$\mathbf{RMi} = \sigma_c \cdot \mathbf{JP} \qquad \text{eq. (AI-1)}$$

σ_c = the uniaxial compressive strength of the intact rock material. It can be determined from laboratory tests, or estimated from standard strength tables. For anisotropic rocks, *the lowest compressive strength should be applied* (which generally will be at a test direction 25 - 45° to the schistocity or layering).

JP = the *jointing parameter*. It is a reduction coefficient representing the properties of the joints in a rock mass. The value of JP varies from almost 0 for crushed rocks to 1 for intact rock. JP is a combination of the following features:

- The *block volume* (Vb) is a measure of the degree of jointing or the density (amount) of joints. It can be determined from various jointing measurements as described by Palmström (1995b).
- The *joint condition factor* (jC) includes important joint characteristcs in the following expression **jC = jL · jR/jA** eq. (AI-2)

jL = the joint size and continuity factor.

jR = the joint roughness factor of the joint wall surface and its planarity. (It is similar to Jr in the Q-system.)

jA = the joint alteration factor, representing the character of the joint wall, i.e. the presence of coating or weathering and possible filling. (It is similar to Ja in the Q system.)

Ratings have been given for each of these three factors in Tables AI-1 to AI-3

The jointing parameter (JP) can be found from Fig. AI-1, either using the block volume or the volumetric joint count (Jv), the joint spacing (where one joint set occurs), or the RQD. JP can also be determined by the following expression:

$$\mathrm{JP} = 0.2\sqrt{\mathrm{jC}} \cdot \mathrm{Vb}^{D} \qquad (\mathrm{JP}_{max} = 1) \qquad \text{eq. (AI-3)}$$

where Vb = the block volume, given in m^3, and $D = 0.37\ \mathrm{jC}^{-0.2}$

D has the following values:

jC = 0.1	0.25	0.5	0.75	1	1.5	2	3	4	6	9	12	16	20
D = 0.586	0.488	0.425	0.392	0.37	0.341	0.322	0.297	0.28	0.259	0.238	0.225	0.213	0.203

For most conditions where jC = 1 - 2, the JP will vary between $\mathrm{JP} = 0.2\ \mathrm{Vb}^{0.37}$ and $\mathrm{JP} = 0.28\ \mathrm{Vb}^{0.32}$. For jC = 1.75 where the jointing parameter is

$$\mathrm{JP} = 0.25\sqrt[3]{\mathrm{Vb}}, \text{ the rock mass index } \mathrm{RMi} = 0.25\,\sigma_c\sqrt[3]{\mathrm{Vb}} \qquad \text{eq. (AI-4)}$$

Jaeger J.C. (1969): "Behavior of closely jointed rock." Proc. 11th Symp. Rock Mech., pp. 57 - 68.

John K.W.: (1969): "Civil engineering approach to evaluate strength and deformability of regularly jointed rock." 11th int. symp. on rock mech. pp. 69-80

Kleeberger J. (1992): "Private communication."

Löset F. (1990): "Use of the Q-method for securing small weakness zones and temporary support." (in Norwegian) Norwegian Geotechnical Institute, internal report No. 548140-1, 40 pp.

Milne D., Germain P. and Potvin Y. (1992): "Measurement of rock mass properties for mine design." Proc. Int. Conf. Eurock '92, Thomas Telford, London, pp. 245-250.

Muir Wood A.M. (1979): "Ground behaviour and support for mining and tunnelling." Tunnels and Tunnelling, Part 1 in May 1979 pp. 43-48, and Part 2 in June 1979, pp. 47-51.

Mutschler T. (1993): "Private communication."

Nakano R. (1979): "Geotechnical properties of mudstone of Neogene Tertiary in Japan." Proc. Int. Symp. Soil Mechanics, Oaxaca, pp. 75 - 92.

Palmström A. (1995a): "RMi - A for rock mass characterization system for rock engineering purposes." Ph.D. thesis, University of Oslo, Norway, 400 pp.

Palmström A. (1995b): "Characterizing the strength of rock masses for use in design of underground structures." Int. Conf. on Design and Construction of Underground Structures; New Delhi, 1995, 10 pp.

Palmström A. (1995c): "Characterizing rock burst and squeezing by the rock mass index." Int. conf. on Design and Construction of Underground Structures; New Delhi, 1995, 10 pp.

Palmström A. (1995d): "RMi - a system for characterizing rock mass strength for use in rock engineering". J. Rock Mech. & Tunnelling Tech., Vol. 1, No. 2, 40 pp.

Rabcewicz L.v. (1964/65): "The new Austrian tunnelling method." Water Power, part 1 November 1964 pp. 511-515, Part 2 January 1965 pp. 19-24.

Rabcewicz L.v. (1975): "Tunnel under Alps uses new, cost-saving lining method." Civil Engineering-ASCE, October 1975, pp.66-68.

Seeber G., Keller S., Enzenberg A., Tagwerker J., Schletter R., Schreyer F. and Coleselli A. (1978): "Methods of measurements for rock support and installations in road tunnels using the new Austrian tunnelling method." (in Germain) Bundesministerium f. Bauten u. Technik, Strassenforschung Heft 133, 200 pp.

Serafim J.L. and Pereira J.P. (1983): "Considerations of the Geomechanics Classification of Bieniawski." Int. Symp. on Engn. Geol. and Underground Constr., Lisbon.

Terzaghi K. (1946): "Introduction to tunnel geology." In Rock tunnelling with steel supports, by Proctor and White, pp 5 - 153.

Terzaghi K. (1953): "Address". Proc. Int. Conf. on Soil Mech. and Foundation Engn., Vol. 3, p. 76.

Ward W.H. (1978): "Ground supports for tunnels in weak rocks." The Rankine Lecture. Geotechnique 28, No. 2, pp. 133-171.

Wood D. (1991): "Estimating Hoek-Brown rock mass strength parameters from rock mass classifications." Transportation Research Record 1330, pp. 22-29.

APPENDIX I

DETERMINATION OF THE RMi VALUE

Basically, the rock mass index expresses the effect various defects (discontinuities) in a rock mass have to reduce the strength of the inherent rock. This is given as

$$\mathrm{RMi} = \sigma_c \cdot \mathbf{JP} \qquad \text{eq. (AI-1)}$$

σ_c = the uniaxial compressive strength of the intact rock material. It can be determined from laboratory tests, or estimated from standard strength tables. For anisotropic rocks, *the lowest compressive strength should be applied* (which generally will be at a test direction 25 - 45° to the schistocity or layering).

JP = the *jointing parameter*. It is a reduction coefficient representing the properties of the joints in a rock mass. The value of JP varies from almost 0 for crushed rocks to 1 for intact rock. JP is a combination of the following features:

- The *block volume* (Vb) is a measure of the degree of jointing or the density (amount) of joints. It can be determined from various jointing measurements as described by Palmström (1995b).
- The *joint condition factor* (jC) includes important joint characteristcs in the following expression $\mathbf{jC = jL \cdot jR/jA}$ eq. (AI-2)

jL = the joint size and continuity factor.

jR = the joint roughness factor of the joint wall surface and its planarity. (It is similar to Jr in the Q-system.)

jA = the joint alteration factor, representing the character of the joint wall, i.e. the presence of coating or weathering and possible filling. (It is similar to Ja in the Q system.)

Ratings have been given for each of these three factors in Tables AI-1 to AI-3

The jointing parameter (JP) can be found from Fig. AI-1, either using the block volume or the volumetric joint count (Jv), the joint spacing (where one joint set occurs), or the RQD. JP can also be determined by the following expression:

$$\mathrm{JP} = 0.2\sqrt{\mathrm{jC}} \cdot \mathrm{Vb}^{D} \qquad (\mathrm{JP}_{max} = 1) \qquad \text{eq. (AI-3)}$$

where Vb = the block volume, given in m^3, and $D = 0.37\ \mathrm{jC}^{-0.2}$

D has the following values:

jC = 0.1	0.25	0.5	0.75	1	1.5	2	3	4	6	9	12	16	20
D = 0.586	0.488	0.425	0.392	0.37	0.341	0.322	0.297	0.28	0.259	0.238	0.225	0.213	0.203

For most conditions where jC = 1 - 2, the JP will vary between $\mathrm{JP} = 0.2\ \mathrm{Vb}^{0.37}$ and $\mathrm{JP} = 0.28\ \mathrm{Vb}^{0.32}$. For jC = 1.75 where the jointing parameter is

$$\mathrm{JP} = 0.25\sqrt[3]{\mathrm{Vb}}, \text{ the rock mass index } \mathrm{RMi} = 0.25\sigma_c\sqrt[3]{\mathrm{Vb}} \qquad \text{eq. (AI-4)}$$

In massive rock where the jointing parameters JP = 0.5 - 1, the rock mass index is

$$\mathbf{RMi} = \mathbf{f}_\sigma \cdot \sigma_c \qquad \text{eq. (AI-5)}$$

where f_σ = the scale effect for the compressive strength given as $f_\sigma = (0.05/Db)^{0.2}$ (Db is the block diameter measured in m).

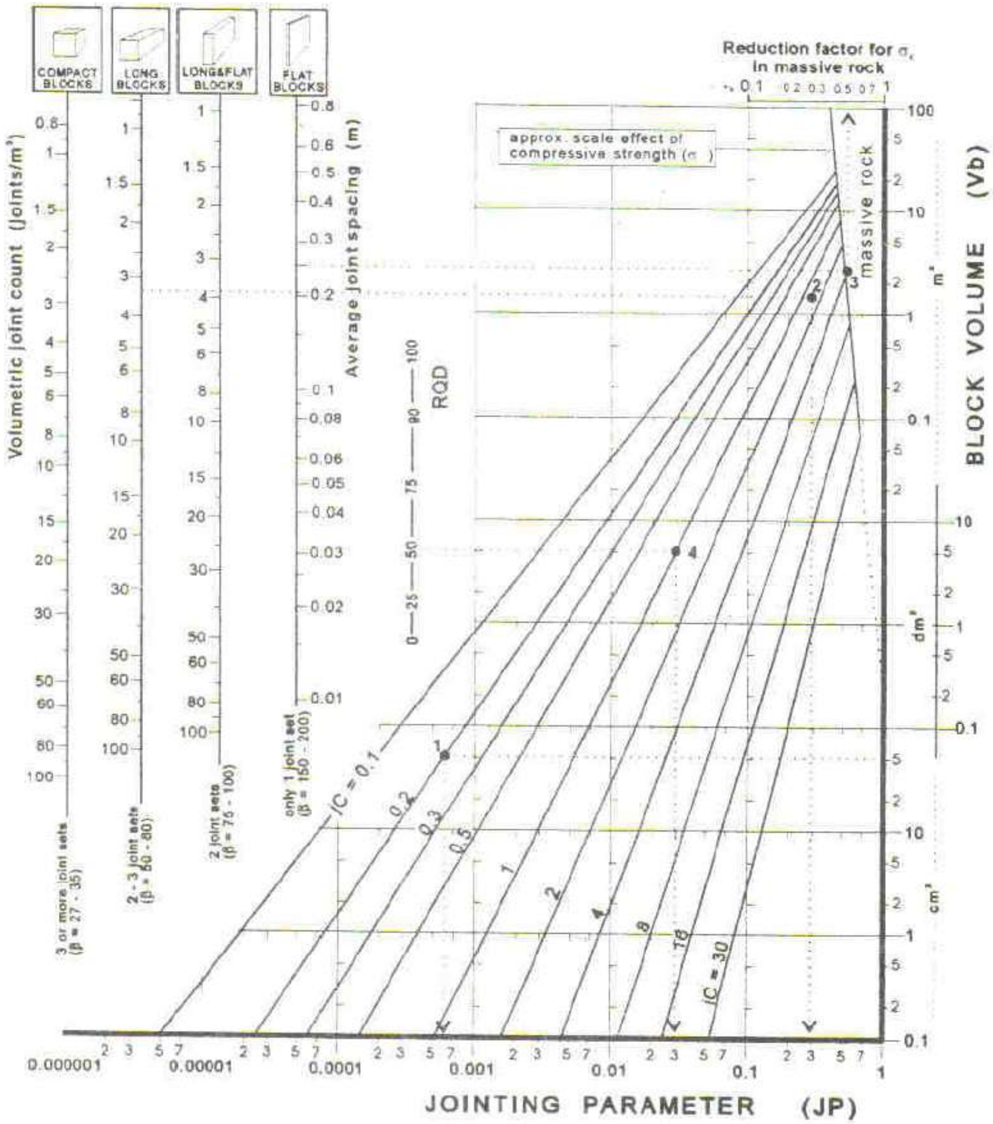


Fig. AI-1 Diagram for finding the value of the jointing parameter (JP) from the joint condition factor (jC) and the block volume (Vb). Also other joint density measurements (Jv and RQD) can be used instead of Vb. (revised from Palmström, 1995a)

Examples shown in Fig. AI-1:

1: For Vb = 0.00005 m³ (50 cm³) and jC = 0.2, JP = 0.0006;
2: For Jv = 3.2 (long blocks) and jC = 1.5, JP = 0.3;
3: For joint spacing S = 0.2 (one joint set) and jC = 4, JP = 0.5 (from scale effect);
4: For RQD = 50 and jC = 1, JP = 0.03.

TABLE AI-1 RATINGS OF THE JOINT ROUGHNESS FACTOR (jR)

small scale smoothness *) of joint surface	large scale waviness *) of joint plane: planar	slightly undulating	strongly undulating	stepped	interlocking (large scale)
very rough	3	4	6	7.5	9
rough	2	3	4	5	6
slightly rough	1.5	2	3	4	4.5
smooth	1	1.5	2	2.5	3
polished	0.75	1	1.5	2	2.5
slickensided**)	0.6 - 1.5	1 - 2	1.5 - 3	2 - 4	2.5 - 5
	For irregular joints a rating of jR = 5 is suggested				

*) For filled joints: jR = 1 **) For slickensided joints the highest value is used for marked striations.

TABLE AI-2 RATINGS OF THE JOINT ALTERATION FACTOR (jA).

A. CONTACT BETWEEN THE TWO ROCK WALL SURFACES

TERM	DESCRIPTION	jA
Clean joints		
-Healed or welded joints	Softening, impermeable filling (quartz, epidote etc.)	0.75
-Fresh rock walls	No coating or filling on joint surface, except of staining	1
-Alteration of joint wall:		
1 grade more altered	One class higher alteration than the intact rock	2
2 grades more altered	Two classes higher alteration than the intact rock	4
Coating or thin filling		
-Sand, silt, calcite, etc.	Coating of friction materials without clay	3
-Clay, chlorite, talc, etc.	Coating of softening and cohesive minerals	4

B. FILLED JOINTWITH PARTIAL OR NO JOINT WALL CONTACT

TYPE OF FILLING MATERIAL	DESCRIPTION OF FILLING MATERIAL	Partly wall contact thin filling (< 5 mm*)) jA	No wall contact thick filling jA
-Sand, silt, calcite, etc.	Friction materials without clay	4	8
-Compacted clay	"Hard" clayey material	6	10
-Soft clay	Medium to low over-consolidation of filling	8	12
-Swelling clay	The material shows clear swelling properties	8 - 12	12 - 20

*) Based on joint thickness division in the RMR system (Bieniawski, 1973)

TABLE AI-3 RATINGS OF THE JOINT SIZE AND CONTINUITY FACTOR (jL).

JOINT LENGTH	TERM	TYPE	jL: continuous joints	discontinuous joints**)
< 0.5 m	very short	bedding/foliation partings	3	6
0.1 - 1.0 m	short/small	joint	2	4
1 - 10 m	medium	joint	1	2
10 - 30 m	long/large	joint	0.75	1.5
> 30 m	very long/large	(filled) joint, seam*) or shear*)	0.5	1

*) Often occurs as a single discontinuity, and should in these cases be treated separately.

**) Discontinuous joi..s end in massive rock.

APPENDIX II

A METHOD TO ESTIMATE THE TANGENTIAL STRESSES AROUND UNDERGROUND OPENINGS

The stresses developed in the ground surrounding an underground opening are mainly a result of the original, in situ (virgin) stresses, the impact from the excavation works, and the dimensions and shape of the opening. Their distribution may, however, be influenced by joints occurring around the opening.

Assessment of the in situ stresses

Hoek (1981) and several others have found that the approximate increase of the vertical stress in excess of 1000 m depth can be reasonably well predicted by:

$$\mathbf{p_v = 0.027\ z} \qquad \text{eq. (AII-1)}$$

where p_v = the vertical stress (in MPa), and
z = the depth below surface (in m).

For the horizontal stresses (p_h) there is not a similar general increase with depth. Especially in the upper 500 meters, the horizontal stresses can vary locally. They are generally higher the vertical stresses. No simple method exists, however, for estimating the horizontal stresses which often vary in magnitude and direction. Where the stresses cannot be measured they may be evaluated from theory and/or the stress conditions experienced at other nearby locations.

A practical method to find the tangential stress (σ_θ)

From a large number of detailed stress analyses by means of the boundary element technique, Hoek and Brown (1980) presented the following correlations:

- The tangential stress in roof $\sigma_{\theta r} = (A \cdot k - 1)\, p_v$ eq. (AII-2)
- The tangential stress in wall $\sigma_{\theta w} = (B - k)\, p_v$ eq. (AII-3)

here A and B = roof and wall factors for various excavation shapes in Table AII-1;
k = the ratio horizontal/vertical stress; and
p_v = the vertical virgin stress

Applying eq. (AII-2) and (AII-3) approximate estimates of the tangential stresses acting in the rock masses surrounding a tunnel can be found. The method requires input of the magnitudes of the vertical stresses and the ratio

$$\mathbf{k = p_h / p_v} \qquad \text{eq. (AII-4)}$$

TABLE AII-1 VALUES OF THE FACTORS 'A' AND 'B' FOR VARIOUS SHAPES OF UNDERGROUND OPENINGS (from Hoek and Brown, 1980).

A	5.0	4.0	3.9	3.2	3.1	3.0	2.0	1.9	1.8
B	2.0	1.5	1.8	2.3	2.7	3.0	5.0	1.9	3.9

<-- tunnel shape