Published in the journal of Tunnelling and Underground Space Technology, Vol. 11, No. 3, pp. 287-303, 1996

Characterizing Rock Masses by the RMi for Use in Practical Rock Engineering

Part 2: Some practical applications of the Rock Mass index (RMi)

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Abstract *- The RMi system is based on defined inherent parameters of the rock mass and is obtained by combining the compressive strength of intact rock and a jointing parameter. The jointing parameter represents the main jointing features, namely block volume (or density of joints), joint roughness, joint alteration, and joint size. This paper discusses the following applications of RMi:*
a) an improved method to determine the constants s and m in the Hoek-Brown failure criterion for rock masses; b) quantification of the descriptive classification in the new Austrian tunnelling method (NATM), and c) estimation of stability and rock support in underground openings.
Rock support charts are presented for the three main groups of rock masses: discontinuous (jointed) rock masses, continuos (massive rock and highly jointed) rock masses, and weakness zones. Mathematical expressions have been developed for all applications, which allow the use of computers in the calculations. The applications of RMi in rock engineering arguably include a wider range of rock masses than any of the classification systems currently in use.

> *"The geotechnical engineer should apply theory and experimentation but temper them by putting them into the context of the uncertainty of nature. Judgement enters through engineering geology."*
>
> Karl Terzaghi, 1961

1 Introduction

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). The main goals of the RMi (Rock Mass index) system have been to improve the geological input data and their use in rock engineering. RMi makes use of selected inherent parameters of the rock mass which are combined ~~of~~ to express the following relative rock mass strength index:

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

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 determined has been given in the first paper of this series (Palmström, 1996a). The Rock Mass index (RMi) is numerical and differs therefore from earlier *general* classifications of rock masses, which are mainly descriptive or qualitative. A numerical system is a prerequisite for application in rock mechanics and rock engineering calculations.
This paper shows the following application of RMi and/or its parameters in rock mechanics and rock engineering:

- Input to the Hoek-Brown failure criterion for rock masses.
- Assessments of stability and rock support in underground excavations.
- Quantification of the classification applied in the New Austrian Tunnelling Method (NATM).

A general use of RMi is in communication between people involved in rock engineering and construction, for example in description of ground conditions and in exchange of information. Other applications of RMi are:

- input to ground response curves,
- assessment of penetration rates of full-face tunnel boring machines (TBM),
- assessment of rock blasting and fragmentation, and
- input to numerical models.

2 Application of RMi in Determining Constants in the Hoek-Brown Failure Criterion for Rock Masses

The Hoek-Brown failure criterion provides engineers and geologists with a means of estimating the strength of jointed rock masses.[1)] Following presentation of the criterion in 1980, the ratings of the criterion's constants (*s* and *m*) have been adjusted in 1988, 1991 and 1992. A modified failure criterion was published by Hoek et al. (1992).

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 (Hoek and Brown, 1980; Hoek, 1983)

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

where 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 the empirical constants which represent inherent properties of joints and rocks

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

$$\sigma_1' = \sigma_{cm} = \sigma_c \sqrt{s} \qquad \text{eq. (3)}$$

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. To determine *m* and *s* Hoek and Brown adapted the main classification systems; the RMR system of Bieniawski (1973) and the Q system of Barton et al. (1974). As these systems include external factors 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, 1995a, 1995b, 1995d, 1996a).

As both RMi and eq. (3) express the unconfined compressive strength of a rock mass, RMi can (with advantage) 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):

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

As shown by Palmström (1995a, 1996a) the value of JP is found from the block size (Vb) and the joint condition factor (jC), i.e. only the inherent features of the rock mass.

The constant m

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1) When applying the Hoek-Brown failure criterion for rock masses in calculations, it should be borne in mind that it is only valid for *continuous* rock masses.

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 1[2] **.** Palmström (1995a, 1996b) has shown that m_b , which varies with the jointing, can be expressed as:

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

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

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

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

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

Table 1. Values for the m_i factor in the Hoek-Brown failure criterion (after Palmström, 1995a, based on Wood, 1990, and Hoek et al., 1992).

Sedimentary rocks	Rating of the factor m_i 1)	**Igneous rocks**	Rating of the factor m_i 1)	**Metamorphic rocks**	Rating of the factor m_i 1)
Anhydrite	13.2	Andesite	18.9	Amphibolite	31.2
Claystone	3.4	Basalt	(17)	Amphibolitic gneiss	31 ?
Conglomerate	(20)	Diabase (dolerite)	15.2	Augen gneiss	30 ?
Coral chalk	7.2	Diorite	27 ?	Granite gneiss	30 ?
Dolomite	10.1	Gabbro	25.8	Gneiss	29.2
Limestone	8.4	Granite	32.7	Gneiss granite	30 ?
Sandstone	18.8	Granodiorite	20 ?	Greenstone	20 ?
Siltstone	9.6	Monzonite	30 ?	Marble	9.3
		Norite	21.7	Mica gneiss	30 ?
		Rhyolite	(20)	Mica quartzite	25 ?
		Syenite	30?	Mica schist	15 ?
				Phyllite	13 ?
				Quartzite	23.7
				Slate	11.4
				Talc schist	10 ?

1) Values in parenthesis have been estimated by Hoek et al (1992); some others with question mark have been assumed by Palmström (1995a)

3 The Use of RMi in Evaluating Rock Support

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 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 evaluation of support requirements for tunnels are largely based on observations, experience and the personal judgement of those involved in tunnel construction (Brekke and Howard, 1972).

The design of excavation and support systems for rock, although based on scientific principles, has to meet practical requirements. In order to select and combine the parameters of importance for stability of an

2) The constant m_b is the same as m in the the original criterion shown in eq. (2)

underground opening, the main features determining the stability have been reviewed in the following section.

3.1 Instability and Failure Modes in Underground Excavations

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

1. **Block failure**, where pre-existing 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 such as loosening, ravelling, and block falls.
2. **Failures 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.

   In addition, squeezing may take place in over-stressed ductile rocks.

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, commonly require special investigations to predict and avoid such events. Bieniawski (1984, 1989) therefore recommends that faults and other 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 of hydrothermal activity and other geologic processes. Thus, the instability of weakness zones may depend on factors other than the properties of the surrounding rock. They all interplay in the final failure behaviour. An important factor in this connection is the character of the gouge or filling material in the zone.

It is not possible to include all the factors, which may affect the stability of an underground excavation in a single practical method, which assesses the stability and evaluates rock support. Therefore, only the dominant factors have been selected in the RMi method for rock support, see Table 2.

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 geometrical factors. Such variables can generally not be combined in a simple number or value. On other factors which influence the stability in underground openings, the following comments are made:

- The effect from *swelling* of some rocks, and gouge or filling material in seams and faults has not been included.[3)] The swelling effect is dominated by 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 influence of water.

There are aspects of specific cases which should be evaluated separately. They include safety requirements, vibrations from earthquakes or from nearby blasting and other disturbances from the activity of man.

[3)] 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 2. 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 properties specific to 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 by blasting
- The ratio tunnel dimension/block size	* Determines the amount of blocks and hence the continuity of the ground surrounding the underground opening.

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

3.2 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 2. The importance of the parameters will vary with the shape and size of the opening and with the composition of the rock mass and the stresses at the specific site. In selecting the parameters, it has been found beneficial to combine those parameters that have a similar effect on the stability, into two main groups.

1. Parameters that affect the continuity of the ground, and
2. Parameters that affect the condition (quality) of the ground.

Both groups of parameters are discussed below.

1. The continuity of the ground refers to whether the volume of rock masses involved in the excavation can be considered discontinuous or not (see Fig. 1). 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 characteristic size and size range of blocks compared to the 'sample' volume, i.e. the tunnel size.
For the application of RMi in rock engineering, the division into continuous and discontinuous materials is based on Deere et al. (1969) to express a continuity factor as the ratio:

CF = tunnel diameter/block diameter = Dt/Db   eq. (8)

Continuous rock masses occur as:

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

Discontinuous rock masses have CF-factors between the above values.

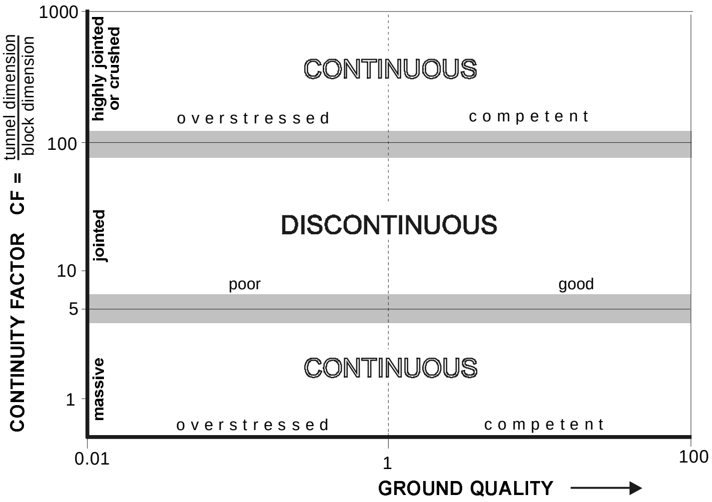

Figure 1 The division of the ground into continuous and discontinuous rock masses. The various groups of ground behaviour are indicated. (from Palmström, 1995a).

2. The condition (quality) of the ground factor comprises selected, inherent rock mass parameters and the type of stress having the strongest influence on the stability of the ground. A competency factor has been applied in *continuous ground* as described in Section 3.3. In *discontinuous ground* and for *weakness zones* a ground condition factor is introduced, see Sections 3.4 and 3.5.

The principles in the RMi method for evaluation of stability and rock are shown in Fig. 2.

3.3 Stability and Rock Support in Continuous Ground

As indicated above, instability in this group of ground can be both stress-controlled and structurally influenced. The structurally related failures in the highly jointed and crushed rock masses are, according to Hoek and Brown (1980**),** generally overruled by the stresses where *overstressing* (incompetent ground) occurs. In *competent* ground the failures and rock support will be similar to those described for discontinuous materials in Section 3.4.

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 RMi is valid in continuous ground, and expresses the (relative) compressive strength of the rock mass (see part 1 of this paper), it can be used in assessing the *competency factor* given as:

$$Cg = RMi / \sigma_\theta \qquad \text{eq. (9)}$$

where σ_θ = the tangential stresses set up around the underground opening. This stress can be found from the vertical and horizontal rock stresses and the shape of the opening as outlined in the Appendix.

The term **"competency factor"** has earlier been used by Nakano (1979) to recognise the squeezing potential of soft rock in tunnels in Japan.

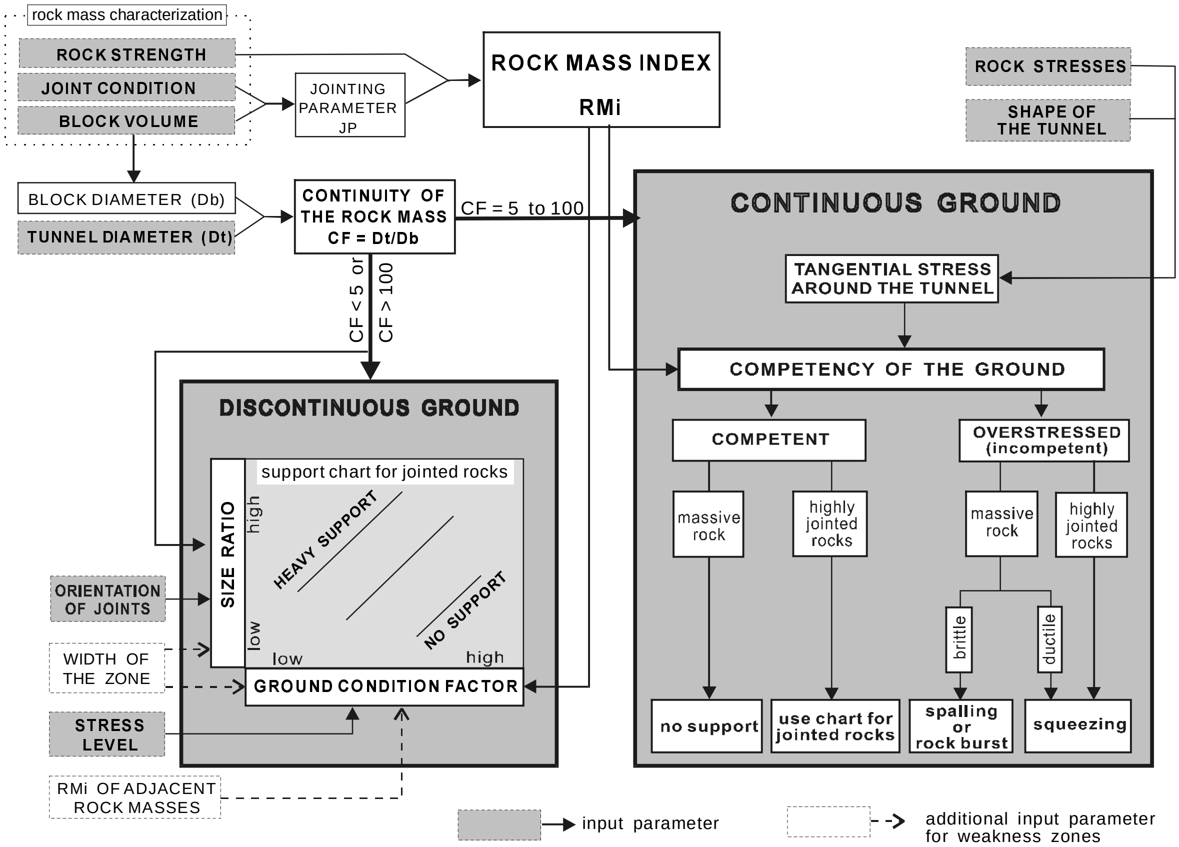

Figure 2 The parameters involved in the RMi method for stability and rock support. For weakness zones the size ratio and the ground condition factor are adjusted for parameters of the zone as indicated. (revised from Palmström, 1995a).

In *massive* rock the competency factor is:

$$Cg = RMi/\sigma_\theta = f_\sigma \times \sigma_c / \sigma_\theta \qquad \text{eq. (10)}$$

where f_σ = the scale effect for the uniaxial compressive strength given by:

$$f_\sigma = (0.05/Db)^{0.2} \qquad \text{eq. (11)}$$

(Db is the block diameter measured in metres; see part 1 of this paper).

In *highly jointed* and crushed rock masses the competency factor is

$$Cg = RMi / \sigma_\theta = JP \times \sigma_c / \sigma_\theta \qquad \text{eq. (12)}$$

Over-stressed (incompetent) ground leads to failure if not confined by rock support. The following main types of instability may take place:

- If the deformations take place instantaneously (often accompanied by sound), the phenomenon is called *rock burst*. This occurs as fragmentation or slabbing in massive, hard, *brittle rocks,* such as quartzite and granites.
- If the deformations occur more slowly, *squeezing* takes place. This acts as slow inward movements of the tunnel surface in *crushed or highly jointed* rocks or in massive *deformable, flexible or ductile* rocks such as soapstone, evaporites, clayey rocks (mudstones, clay schist, etc.) or weak schists.

Thus, in massive rocks the failure behaviour, i.e. whether bursting or squeezing will take place, is determined by the deformation properties of the rock material.

3.3.1 Rock burst and spalling in brittle rocks

Rock burst is also known as *spalling* [4] or *popping*, but also a variety of other names are in use, among them 'splitting' and 'slabbing'. 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 within valley sides steeper than 20^o and with the top of the valley reaching higher than 400 m above the level of the tunnel.

Hoek and Brown (1980) have made studies of the stability of tunnels in various types of massive quartzites in South Africa. Similarly, Russenes (1974) used the point load strength (Is) [5] of intact rock and rock stresses measured in several Scandinavian tunnels. Later, Grimstad and Barton (1993) made a compilation of rock stress measurements and laboratory strength tests and arrived at a relation for spalling conditions similar to Hoek and Brown, and Russenes. This is shown in Table 3.

The values for σ_c in Table 3 refer to the compressive strength of 50 mm diameter samples. In the massive rocks where rock spalling and rock burst occur, RMi = $f_\sigma \times \sigma_c$ for which f_σ (the factor for scale effect of compressive strength) is in the range $f_\sigma = 0.45$ to 0.55. Thus, RMi $\approx 0.5\ \sigma_c$ and hence the competency factor in Table 4 is Cg = RMi/ $\sigma_\theta = f_\sigma \times \sigma_c / \sigma_\theta \approx 0.5\ \sigma_c / \sigma_\theta$, i.e. half the values given for the ratio σ_c / σ_θ in Table 3.

Table 3. Rock burst activity related to the ratio σ_c / σ_θ, the data are based on results presented by Hoek and Brown (1980), Russenes (1974), and Grimstad and Barton (1993)

Value of the ratio σ_c / σ_θ			**Description of the stability by the three authors respectively**
Hoek and Brown (1980)	Russenes (1974)	Grimstad and Barton (1993)	
		> 100	Low stress, near surface, open joints
> 7	> 4	100 - 3	Stable / No rock spalling activity /Medium stress, favourable stress condition
7 - 3	4 – 3	3 -2	Minor spalling / Low rock spalling activity / High stress, very tight structure
3 - 1.7	3 - 1.5	2 - 1.5	Severe spalling / Moderate rock spalling / Moderate slabbing after > 1 hour
1.7 - 1.4	< 1.5	1.5 - 1	Heavy support required / High rock spalling activity / Slabbing and rock burst
< 1.4		< 1	Severe (side wall) rock burst problems / Heavy rock burst.

Table 4. Characterization of failure modes in brittle, massive rock (from Palmström 1995a)

Competency factor Cg = RMi/ $\sigma_\theta = f_\sigma \cdot \sigma_c / \sigma_\theta$	**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

Strength anisotropy in the rock may cause the values of the competency factor in Table 4 not always to be representative.

In Scandinavia, tunnels with spalling and rock burst problems are mostly supported by shotcrete (often fibre reinforced) and rock bolts, as these have been found to be the most appropriate practical means of confinement. This general trend in support design is reflected in Table 5. In addition to scaling, wire mesh

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

5) The uniaxial compressive strength (σ_c) in Table 3 has been calculated from the point load strength (Is) using the correlation σ_c = 20 Is.

and rock bolts were used earlier as reinforcement in this type of ground. This is now only occasionally applied in Norwegian tunnels.

Table 5. 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 and rock bolt spaced 0.5 - 2 m, plus fibre reinforced shotcrete, 50 -100 mm thick

3.3.2 Squeezing in continuous ground

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.

The application of RMi in squeezing rock masses, as presented in Table 6, is mainly based on studies made by Aydan et al. (1993) of 21 Japanese tunnels located in mudstones, tuffs, shales, serpentinites and other ’ductile’ rocks with compressive strength $\sigma_c < 20$ MPa**.** As the presence of joints is not mentioned in their paper, it is assumed that the rocks contain relatively few joints. This is also evident from the photographs presented.

Table 6 is based on a limited number of results from massive rocks and should, therefore, be revised 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 be approximately as shown in Table 7, see also Section 4.1.

Table 6. Characterization of ground and squeezing activity (from Palmström, 1995a and 1995c, based on Aydan et al., 1993)

Squeezing class	**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 / σ_θ 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.
Moderate squeezing RMi / σ_θ = 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 large 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 be necessary to re-excavate the tunnel and install heavy support.

*) This value has been assumed

Table 7. Convergence and rock support in squeezing ground (from Palmström, 1995a, based on Seeber et al., 1978)

NATM	Approximate convergence and rock support according to Seeber et al. (1978) for tunnels with diameter 12 m

class	Without support	With support installed		
	convergence range	convergence range	support pressure	possible rock support
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
Heavy squeezing or swelling	min. 2 · 40 cm = 80 cm ----- max. > 2 m	2 · 10 cm = 20 cm ----- 2 · 20 cm = 40 cm	0.2 MPa ---- 0.7 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

3.4 Stability and Rock Support in Discontinuous (jointed) Materials

The principles in the method for evaluating rock support in this type of ground are shown in Figure 2. The failures occur when wedges or blocks, limited by joints, fall or slide from the roof or sidewalls. They develop as local sliding, rotation, 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, do not commonly involve development of fracture(s) through the rock (Hoek, 1981). However, the strength of the rock often influences the wall strength of the joint and may in this way contribute to the stability.

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 average amount and types of support are found from combination of rock mass and excavation parameters. This principle has been applied in the Q and the RMR classification systems, among others.

3.4.1 The ground condition factor (Gc) in discontinuous ground

The ground condition factor for discontinuous ground includes the *inherent* rock mass characteristics which have a significant influence on stability as well as the *external* stresses acting. It is expressed as:

$$Gc = RMi \times SL \times C \qquad \text{eq. (13)}$$

RMi = represents inherent features in the rock mass, see part 1 of this paper (Palmström, 1996a)

SL = the stress level factor, expresses the contribution from the external forces 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. Conversely, 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 an important cause of loosening and falls of blocks.

However, in a jointed rock mass containing a variable number of joints with different orientations, it is not possible to calculate and incorporate in a simple way the exact effect of 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 an increased stress level has a positive influence on the stability in discontinuous ground the stress level factor (SL) forms a multiplication factor. The ratings of SL in Table 8 are based approximately on SL = 1/SRF.
The influence of joint water pressure is generally difficult to incorporate in a stress level factor. Often, the joints around the tunnel will drain the water in the rock volume 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 Table 8. In some cases, however, where unfavourable joint

orientations, combined with high ground water pressure, will reduce the stability by extra loading on key blocks, the stress level factor should be reduced as shown in Table 8.

Table 8. The ratings of the stress level factor (SL) (from Palmström, 1995a).

Term	**Maximum stress** σ_1	**Approximate overburden** (valid for k =1)	**Stress level factor (SL)**[*)]	
				average
Very low stress level (in portals etc.)	< 0.25 MPa	< 10 m	0 - 0.25	0.1
Low stress 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 major influence

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

C = 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.[6)] Based on Milne et al. (1992) this factor is found from:

$$C = 5 - 4 \cos \theta \qquad \text{eq. (14)}$$

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

Possible instability induced from high ground stresses.

As stated above, the experience shows that rock bursting is less developed in jointed rock than in massive rock at 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.

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 rock than in massive rock. This has also been clearly shown in tunnels where de-stress blasting is carried out in the tunnel periphery with the purpose of developing additional cracking and in this way reducing the amount of rock bursting.

In moderately to slightly jointed rock masses subjected to high stress levels 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 at σ_c = 1 to 3 MPa.

3.4.2 The size ratio

The size ratio includes the dimension of the blocks and the underground opening and is a representation of the geometrical conditions at the particular site. The size ratio for discontinuous (jointed) rock masses is expressed as:

$$Sr = (Dt / Db) (Co / Nj) \qquad \text{eq. (15)}$$

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

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6) Similarly, Barton (1975) has applied a wall/roof adjustment factor of the Q-value. This factor depends, however, on the quality of the ground. It has 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).

Db = is the block diameter represented by the smallest dimension of the block which often corresponds 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):[7)] $Db = (27/\beta)\sqrt[3]{Vb}$ eq. (16)

Co = is an orientation factor representing the influence of 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 9 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 9. 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 $> 30^o$	for strike $< 30^o$	for all strikes		
dip $< 20^o$	dip $< 20^o$	dip $> 45^o$	favourable	1
dip = 20 - 45^o	dip = 20 - 45^o	dip = 20 - 45^o	fair	1.5
dip $> 45^o$	-	dip $< 20^o$	unfavourable	2
-	dip $> 45^o$	-	very unfavourable	3

Nj = 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 described by Barton et al. (1974**),** the degree of freedom determined by the *number of joint sets* significantly contributes to stability. The value of Nj is found from the expression:

$$Nj = 3/n_j \qquad \text{eq. (17)}$$

where n_j = the number of joint sets (n_j = 1 for one set; n_j = 1.5 for two sets plus random joints; n_j = 2 for two sets, nj = 2.5 for two sets plus random; etc.)

3.5 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 within strong rocks, etc. Weakness zones occur both geometrically and structurally as special types of rock masses. The following features of the zones are of main importance for stability:

1. The orientation and dimensions (width) of the zone.
2. Reduced stresses in the zone compared to the stresses in the surrounding rock masses.
3. The arching (or silo) effect from the ground surrounding the weakness zone.
4. The possible occurrence and effect of swelling, sloughing, or permeable materials in the zone.

As mentioned earlier, these aspects often depend on the geometry and the site conditions. They have, therefore, not been included in this general support evaluation method.

The composition of weakness zones and faults can be characterized by the RMi or by its parameters. The material in many weakness zones may be considered as a continuum when related to the size of the tunnel.

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7) The block shape factor (β) has been described by Palmström (1995a, 1995d, 1996a). The ratio 27/β has been chosen as a simple expression to find the smallest block diameter. Eq. (16) is most appropriate for β < 150. For higher values of β a dominating joint set will normally be present for which the average joint spacing may be used.

However, the system presented for discontinuous (jointed) rock masses in Section 3.3 has been found to cover also many types of zones where the size ratio and the ground condition factor are adjusted for the zone parameters.

3.5.1 The ground condition factor for zones

As mentioned above, stability is influenced by the interaction of the properties of the zone and the properties of the adjacent rock mass, especially for small and medium sized zones. Palmström (1995a) has presented a method of combining the conditions in the zone and in the adjacent rock masses in the following simplified expression, based on Löset (1990):

$$RMi_m = (10Tz^2 \times RMi_z + RMi_a)/(10Tz^2 + 1) \qquad \text{eq. (18)}$$

where Tz = the thickness of the zone
RMi_a refers to the weakness zone
RMi_z refers to the surrounding rock

For larger zones the effect of stress reduction from arching is limited; the ground condition factor for such zones should therefore be that of the zone ($RMi_m \approx RMi_a$). This is assumed to take place for zones where $Tz > 20$ m as is found from eq. (18). Applying eq. (18), a ground condition factor for weakness zones can be found similarly to that for discontinuous (jointed) rock masses:

$$Gc_z = SL \times RMi_m \times C \qquad \text{eq. (19)}$$

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 apply in 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 .

3.5.2 The size ratio for zones

As mentioned in the beginning of this section there is an arching effect in weakness zones with thickness less than approximately the diameter (span) of the tunnel. For such zones the size ratio in eq. (15) [Sr = (Dt/Db)(Co/Nj)] is adjusted for the zone ratio Tz/Dt to form the following size ratio for zones: [8)]

$$Sr_z = Co_z \times Nj_z\,(Tz / Db_z) \qquad \text{eq. (20)}$$

where Co_z = factor for the orientation of the zone with ratings as shown in Table 9
Db_z = the diameter of the representative blocks in the zone
Nj_z = the adjustment factor for joint sets in the zone similar to Nj in eq. (17)

Eq. (20) is valid where Tz is smaller than the diameter (span or height) of the tunnel. For thicker zones eq. (15) should be applied.

3.6 Comments on the Support Chart

The support chart for *discontinuous* rock masses in Fig 3 covers most types of rock masses. It 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. Application of RMi in stability and support calculations over a two-year period suggests that the method works in practice.
A support chart for discontinuous ground 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,

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[8)] This ratio is applied provided Tz / Dbzone < Dt / Dbadjacent the support chart in Fig. 3 may be revised based on the current practice and the principles applied for rock support.

For continuous ground, the chart is based on Tables 5 and 7. Work still remains, however, to develop improved support chart for this type of ground.

The support charts are 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. As loosening or failures in jointed rock is mainly geometrically related, i.e. influenced by the orientation and location of each individual joint, it is impossible to develop a support chart which covers such detail.

The required stability level and amount of rock support is determined from the use of the underground opening. The Q-system uses the ESR (excavation support ratio) as an adjustment of the span to include this aspect. From current practice in underground excavation, 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. Alternatively, universal charts may be used to give the minimum rock support, subject to review of safety and other factors which may dictate enhanced support.

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 of medium gravel size) occur, Vb = 1 cm^3 or block diameter $Db \approx 0.01$ m is used.

Assuming the following characteristics for *'common' hard rock mass conditions*:

- $RMi = 40\sqrt[3]{Vb}$ (for $\sigma_c = 160$ MPa),
- planar, slightly rough joints of medium length (joint condition factor jC = 1.75),
- three joint sets ($Nj = 3/n_j = 1$),
- the block shape factor $\beta = 40$,
- fair joint orientation (Co = 1.5) and
- moderate stress level (SL = 1),

the following expressions are found:

- The ground condition factor: $Gc = RMi \times SL \times C = 0.25\ \sigma_c \times C\sqrt[3]{Vb}$ eq. (21)
- The size ratio: $Sr = Wt \times Nj \times Co/Db = Wt/\sqrt[3]{Vb}$ or $Sr = Ht/\sqrt[3]{Vb}$ eq.(22) and eq. (23)

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

The various excavation techniques used may disturb and to some degree change the rock mass conditions. Especially, excavation by blasting tends to develop new cracks around the opening. This will cause that the size of the original blocks to be reduced, which will cause an increase of the size ratio (Sr) and a reduction of the ground condition factor (Gc). Knowing or estimating the change in block size from excavation, the adjusted values for (Sr) and (Gc) can be calculated readily and thus include the impact from excavation in the assessments of rock support.

Mathematical expression have been developed for all the parameters characterising 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 10.

Example 1

Information on the tunnel and the ground conditions:

A horseshoe shaped tunnel with 5 m span is located 200 m below the surface in a gneiss with average compressive strength σ_c = 150 MPa. It is cut by three joint sets with average spacings S1 = 0.2 m, S2 = 0.5 m and S3 = 0.6 m, i.e. the average block volume is Vb = 0.06 m^3.
The average joint characteristics are: slightly undulating, rough joints with fresh walls.
The 1 to 10 m long continuous joints cut the tunnel roof at a moderate (fair) angle.

Input values:

From Tables 1 - 3 in part 1 of this paper the following ratings are found: jR = 3, jA= 1, and jL = 1
The joint orientation factor is Co = 1.5 as seen in Table 9.
The stress level factor (for discontinuous ground) for this overburden is SL = 1 as seen in Table 8.
With 3 joint sets n_j = 3 the factor for the number of joint sets is Nj = 3/3 = 1.

Calculations:

As shown in part 1, the joint condition factor is jC = jL × jR/jA = 3
The jointing parameter is JP = 0.15 giving the rock mass index RMi = 22.5 (as found from Fig. 3 (or eq. 2) and eq. 1 shown in part 1)
The block shape factor is β = 39 (using eq. A-8 or Fig. A4 in part 1). Applying β = 40 in eq. 16 the block diameter is Db = 0.26 m.
The continuity factor: CF = tunnel diam. / block diam. = 18.9, hence the ground is discontinuous with the following parameters:

- the ground condition factor for the roof	Gc = RMi × SL × C = 22.5	(eq. 13)
- the size ratio for the roof	Sr = (Dt/Db)(Co/Nj) = 28.4	(eq. 15)

Estimated rock support:

The rock support according to Fig. 3 is: shotcrete 40 - 50 mm thick and rock bolts spaced 2 m.

Example 2

A vertical *weakness zone* is encountered in the same tunnel. The zone crossing at 60° (Co_z = 1 for the roof as given in Table 9). The zone is 2 m thick and consists of crushed rock. The fresh rock pieces of gneiss (σ_c = 150 MPa) in the zone have an average volume of Vb_z = 0.01 dm^3 = 0.00001 m^3
The smooth, short, continuous joints in the zone have coating of clay , i.e. jC_z = 1· 2/4 = 0.5
With 3 joint sets and some random joints in the zone (n_j = 3.5) the factor for the number of joint sets is Nj_z = 3/3.5 = 0.86.

Calculations for the weakness zone:

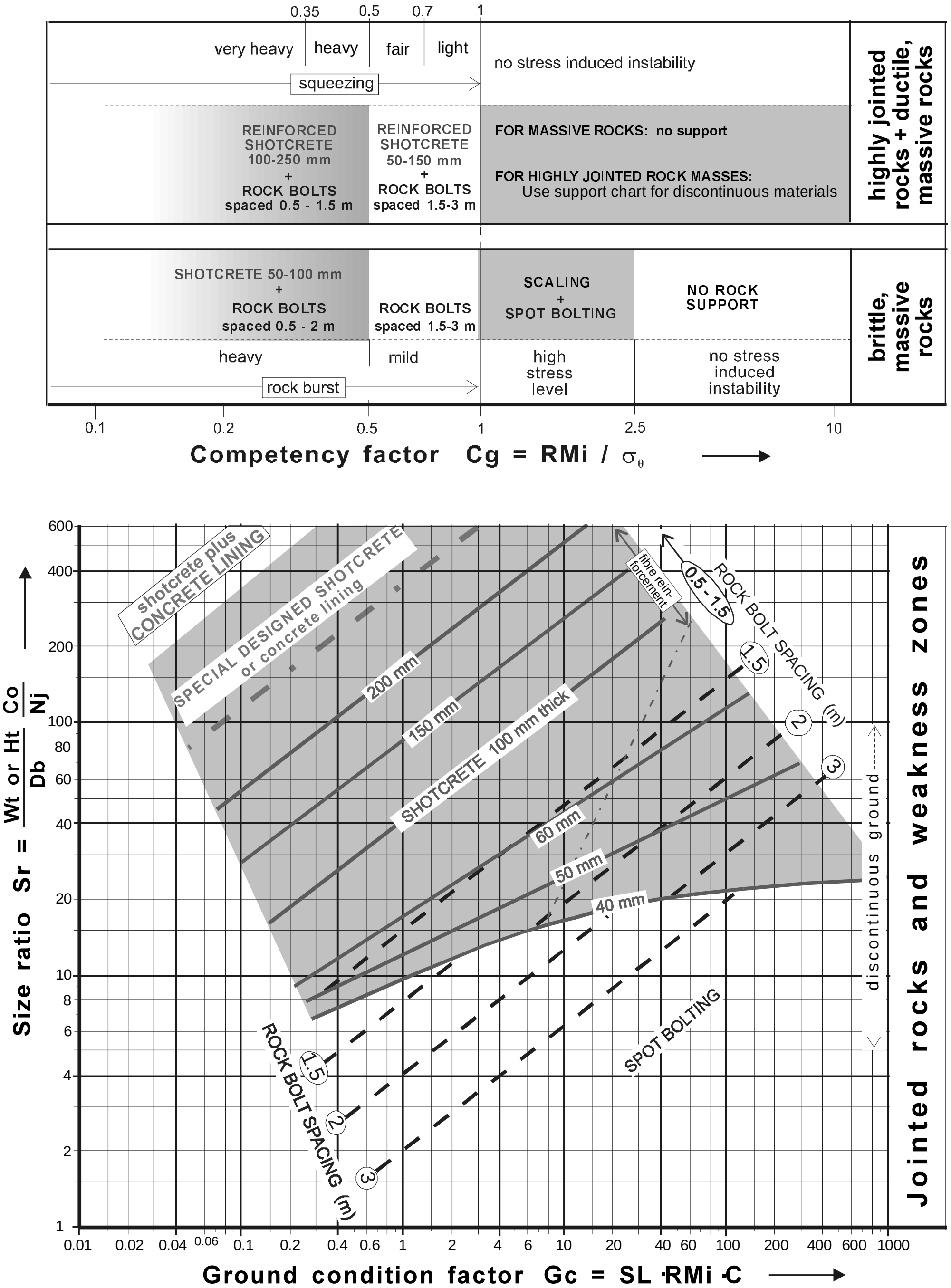
The jointing parameter is	JP_z = 0.001	(eq. 2 in part 1)
Rock Mass index in the zone is	RMi_z = 0.16	(eq. 1)
The combined Rock Mass index is	RMi_m = 0.7	(eq. 18)

With assumed block shape factor β = 40 the equivalent block diameter is Db_z = 0.015 m (eq. A-8 or Fig. A4 in part 1)
From the data above the following parameters are found for the zone:

- the ground condition factor for the roof	Gc_z = 0.7	(eq. 19)
- the size ratio for the roof	Sr_z = 160.4	(eq. 20)

Estimated rock support in the weakness zone:

The rock support according to Fig. 3 is: 200 mm thick fibre reinforced shotcrete and rock bolts spaced 0.5 - 1.5 m.



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 $RMi_m = (10\,Tz^2 \cdot RMi_z + RMi_a) / (10\,Tz^2 + 1)$

Figure 3. Rock support charts for continuos and discontinuous ground. The support in continuous ground is for tunnels with diameter Dt < 15 m. Note that the diagram for squeezing in particulate materials is based on limited amount of data. (from Palmström, 1995a)

Table 10. Application of a computer spreadsheet to calculate the factors used in Figure 3 to determine appropriate rock support. (The input values used in location 1 are the same as used in Examples 1 and 2)

20 cm

4. RMi applied to improve the NATM classification

The goal of the New Austrian Tunnelling Method (NATM) is to provide safe and economic support in tunnels excavated in materials incapable of supporting themselves, i.e. crushed rock, debris, even soil (Rabcewicz, 1964/1965). Support is achieved by mobilising whatever limited strength the rock mass or earth possesses. The main features of NATM are (Rabcewicz 1975):

- It relies on strength of the rock masses surrounding the tunnel to reduce the loads on the support.
- It uses flexible rock supporting methods tailored to the actual ground conditions, such as shotcrete and rock bolts.
- It involves installation of sophisticated instrumentation at the tunnel face to provide information for designing the support.
- It eliminates costly rock supports, such as heavy steel arches and stiff, thick concrete linings.

The classification of the ground applied in the NATM is shown in Table 11. It is qualitative, based mainly 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 condition and estimates of the rock stresses mainly made 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.

Table 11. The classification of ground behaviour

NATM class	Description of rock mass and behaviour
1 Stable	The rock masses are long-term stable.
2 Slightly ravelling	Some few small structural relief surfaces from gravity occur in the roof.
3 Ravelling	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	Low strength of rock mass results in possible loosening effects to considerable depth, resulting in heavy support load. 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	Moderate squeezing as well as rock spalling (rock burst) phenomena, often caused by structural defect such as closely jointing, seams and/or shears. The rock support can sometimes be overloaded.
6 Strongly squeezing or swelling	Development of a deep zone with inward 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)

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 swelling of rocks.

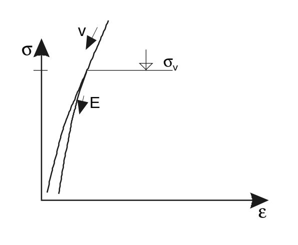
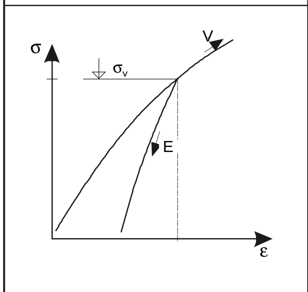
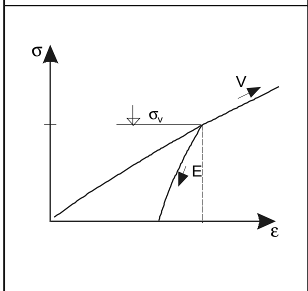
4.1 The Use of RMi to Quantify the NATM Classification

Seeber et al. (1978) have made an interesting contribution towards quantifying 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, although the input parameters are different. Fig. 4 shows that it is possible to apply 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 found from eq. (12); and/or
   - the modulus of elasticity (E) and the modulus of deformation (V).

These shear strength parameters can for example be found using the Hoek-Brown failure criterion for rock masses as described in Chapter 2. The modulus of elasticity can be estimated from the following preliminary expression: [9)]

$$E = 5.6\ \mathrm{RMi}^{0.375} \qquad \text{eq. (23)}$$

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

(1 NM/m² = 10kPa)

Figure 4. Rock mass strength classes as applied by Seeber et al. (1978)

2. The **"Gebirgsgüteklassen"** ("rock mass quality classes") which is 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 11.

By combining the 'rock mass strength classes' in Fig. 4 with rock stresses from overburden the actual NATM class is found from Fig. 5. Using the RMi characterization directly, Table 12 may be applied. More work remains, however, to check the suggested values in this table.

[9)] This equation has been found from the correlation $\mathrm{RMi} = 10^{(\mathrm{RMR} - 40)/15}$ between RMR and RMi (Palmström, 1995a) and $E = 10^{(\mathrm{RMR} - 10)/40}$ (Serafim and Pereira, 1983)

Table 12. Suggested numerical classification the NATM (from Palmström, 1996b)

NATM class	**Rock mass properties** (JP = jointing parameter)	**Competency factor** (Cg = RMi / σ_θ)
1 Stable	Massive ground (JP > approx. 0.5)	Cg > 2
2 Slightly ravelling	JP = 0.2 - 0.6	Cg > 1
3 Ravelling	JP = 0.05 - 0.2	Cg > 1
4 Strongly ravelling	JP < 0.05	Cg = 0.7 - 2
5 Squeezing	Occurs in continuous ground *)	Cg = 0.35 - 0.7
6 Strongly squeezing	Occurs in 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 characterisations. 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.

It is obvious that the accuracy of this 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.

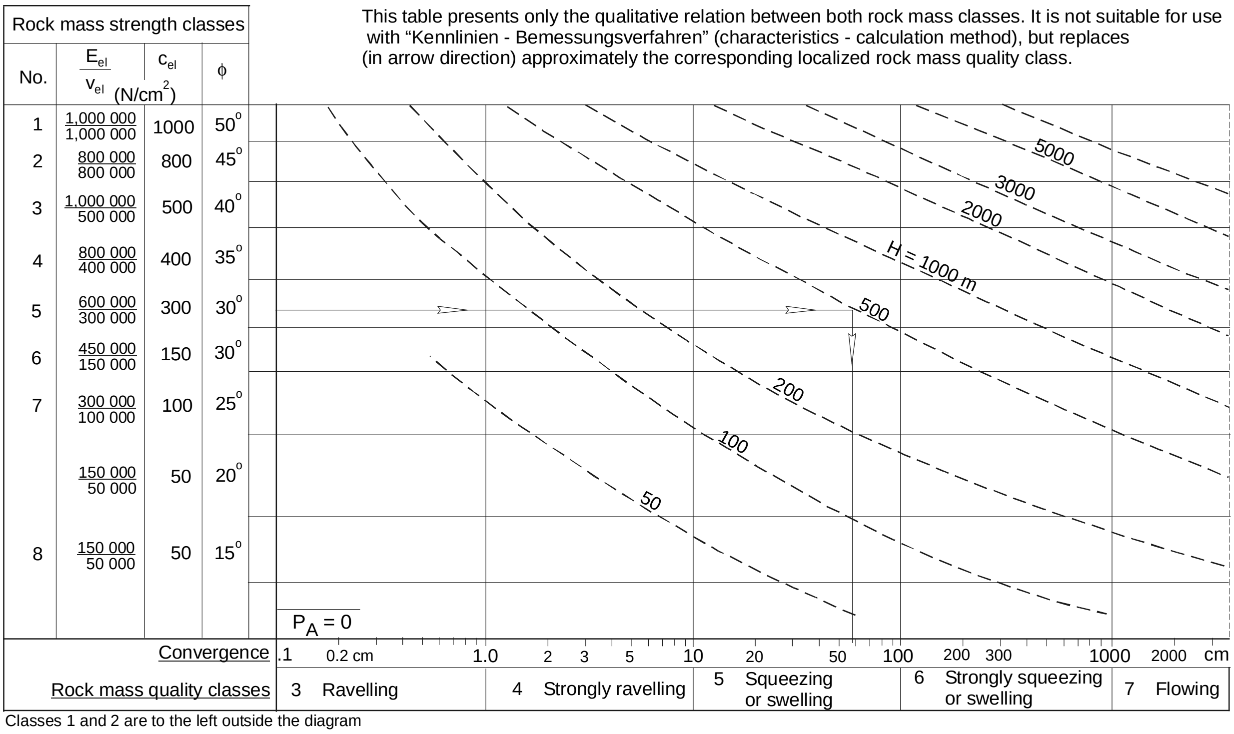


Figure 5 Connection between rock mass strength classes, rock mass quality classes and overburden (from Seeber et al. (1978). Note: Seeber et al. applied an earlier NATM classification of the rock mass quality.

5. Discussion

The RMi offers several benefits and possibilities in rock engineering and rock mechanics, as it expresses a general strength characterization and involves the main inherent characteristics of the rock mass. Being adjusted for the local features of main importance for the actual use, work or utility, the RMi offers a flexible system applicable to many different purposes connected with rock construction, such as:

- input to Hoek-Brown failure criterion for rock masses, as shown in Chapter 2;
- in stability and rock support assessments, described in Chapter 3;
- quantification of the rock mass classification applied in the NATM, as outlined in Chapter 4;
- input to ground response curves;
- in assessments of penetration rate of full-face tunnel boring machines (TBM);
- in assessments of rock blasting and fragmentation; or
- input to numerical models.

When applied directly in calculations, RMi is restricted to *continuous* rock masses, as is the case for the Hoek-Brown failure criterion. To apply RMi in discontinuous rock masses, it is adjusted for or combined with the local conditions. This is the reason why RMi in evaluation of rock support in Section 3, is applied differently in discontinuous and continuous rock masses. As this use of the RMi may have the main interest, it is discussed in the following section.

5.1 Comments on 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, however, the use of RMi to characterize the composition and inherent properties of the ground. The influence from stresses is 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 discontinuous ground a stress level factor (SL) 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 generally included. However, the stress level factor may be adjusted 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 determines the continuity of the ground, i.e. whether it is continuous or not. In discontinuous ground Vb is included both in the ground condition factor and in the size ratio. Great care should, therefore, be taken when this parameter is determined. Where less than three joint sets occur, defined blocks are not formed. In these cases, methods have been given by Palmström (1995a, 1995d, 1996a) to assess an equivalent block volume. An additional problem is to indicate methods for characterising the variations in block size. Therefore, engineering calculations should generally 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 rock type using standard strength tables in textbooks.

What is new about the RMi support method?

The method using RMi to determine rock support differs from the existing classification systems for support. While previous methods combine all the selected parameters to directly arrive at a quality or rating for the ground conditions, the RMi method applies an index (RMi) 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 introduction 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 systematic collection and application of the geological input data. RMi also makes 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. (21) to (23) it is also possible to apply simplified expressions for the ground conditions (Gc) and size ratio (Sr) when only rough support estimates are required. As this only requires input from the block volume, the support estimates can quickly be carried out.

The support method has a flexible structure and can be tailored to the actual ground by selecting the appropriate parameters. In this way, the method for evaluation of support can be simplified for the actual case. As mathematical expressions have been given for all parameters and factors, the method can be worked into a spreadsheet in which all calculation are made. 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 studies and to Ole Berthelsen for valuable comments.

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Appendix. A Method to Estimate the Tangential Stress 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 the joints occurring around the opening.

Assessment of the in situ stresses

Several authors have contributed to the understanding and knowledge of ground stresses in the earth's crust from in situ measurements. Many of the results from these have been summarized and linear regression analyses performed to find the distribution by depth. Fig. A-1 shows a summary of some results.

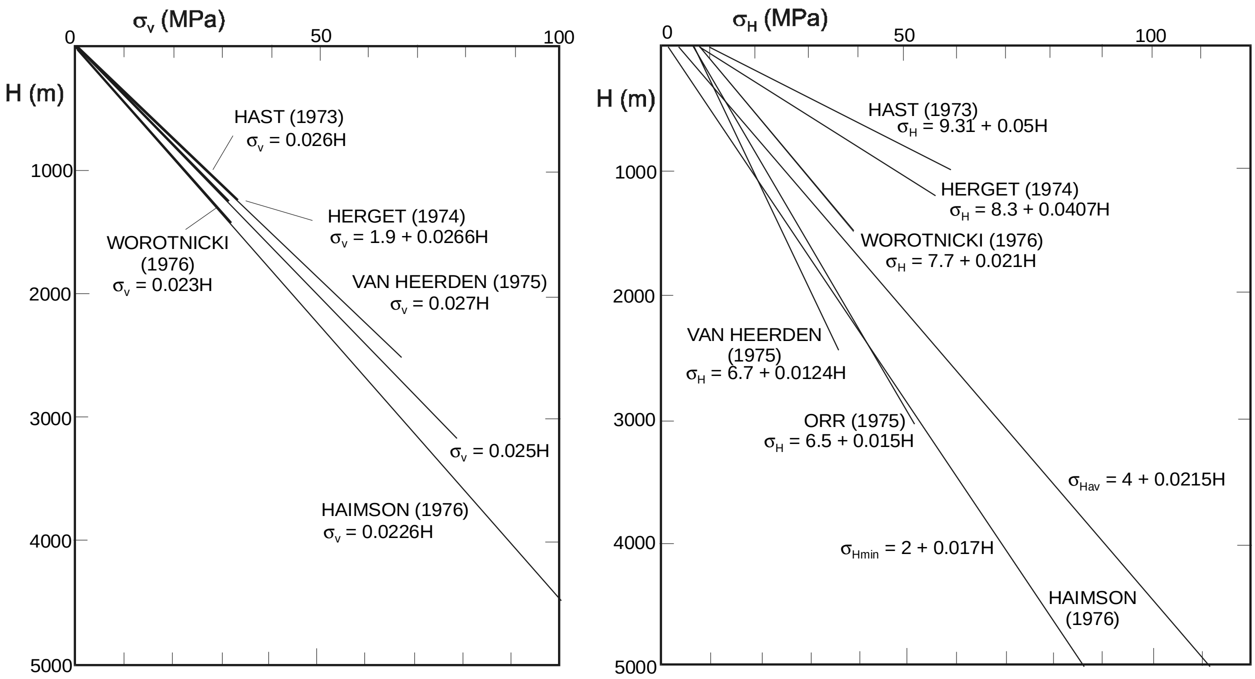


Figure A-1. Vertical and horizontal stresses versus depth below surface according to various authors. Left: Vertical stresses. Right: Horizontal stresses (from Bieniawski, 1984)

As shown the approximate increase of the vertical stress can be reasonably well predicted by:

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

where z = the depth below surface (in metres)

As is evident from Fig. A-1, there is not a similar general increase with depth for horizontal stresses. Especially in the upper 500 metres, the horizontal stresses can vary locally. They are generally higher than the vertical stress. The following trends of the horizontal stresses were formulated by Hoek (1981):

- With the exception of deep level South African gold mines, average horizontal stresses are generally higher than vertical stress for depths of less than 1,000 m below surface.
- At a depth of 500 m below surface, the average horizontal stress is approximately 1.5 times the vertical stress with higher ratios being evident at shallower depths.
- For depths in excess of 1,000 m below surface, the horizontal and vertical stresses tend to equalize, except in South African mines in quartzites where the ratio of average horizontal to vertical stress is k = 0.75.
- In the Scandinavian Precambrian and Palaeozoic and in the Canadian crystalline rocks the horizontal stresses are significantly higher than the vertical stress down to a few hundred meters.

However, 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.

For the method of estimating rock support in *discontinuous* (jointed) rock masses, described in Section 3.4, only a rough estimate of the stresses is required to arrive at a factor for the overall stress level. For *continuous* rock masses in Section 3.3, however, the effect of tangential stresses around the opening may be important where they result in overstressed (incompetent) ground.

A practical method to estimate the tangential stress (σ_θ)

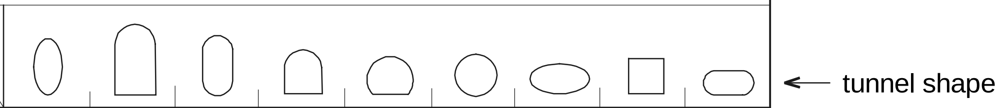
From a large number of detailed stress analyses by means of the boundary element technique, Hoek and Brown (1980) presented the following correlations:

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

Here A and B = roof and wall factors for various tunnel shapes given in Table A-1;
$k = p_h / p_v$, the ratio horizontal/vertical stress eq. (A-4)

Eqs. (A-2) and (A-3) can be applied in approximate estimates of the tangential stresses acting in the rock masses surrounding a tunnel. The method requires input of the magnitudes of the vertical stresses and assumption of the ratio $k = p_h / p_v$

Table A-1. Values of the factors "A" and "B" for various shapes of underground openings (from Hoek and Brown, 1980).



VALUES OF CONSTANTS A & B									
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