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A NEW METHOD TO CHARACTERIZE ROCK MASSES FOR APPLICATIONS IN ROCK ENGINEERING

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SUMMARY

The Rock Mass index, RMi, has been developed to satisfy a need for a strength characterization of rock masses. The method gives a measure of the reduction of intact rock strength caused by joints expressed as RMi = $\sigma_c \cdot$ JP. Here σ_c is the uniaxial compressive strength of the intact rock measured on 50 mm diameter samples, and JP is the jointing parameter which is a combined measure of block size and joint characteristics as measured by joint roughness, alteration and size. In massive rock JP is expressed as a scale factor for σ_c .

RMi can be applied for several applications in rock mechanics and rock engineering, such as TBM penetration assessment, determination of the input factor in the Hoek-Brown failure criterion for rock masses, and the E - modulus for rock masses. The paper shows how RMi can be used to evaluate stability and rock support. Rock support charts are presented for the three main groups of rock masses: discontinuous (jointed) rock masses, continuos (massive rock or 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.

SAMMENDRAG

En ny metode for karakterisering av bergmasser for bruk i bergtekniske beregninger

Bergmasser involver så store volum at mekaniske tester av representative prøver ikke lar seg praktisk/økonomisk gjennomføre. Derfor må bestemmelse av parametre som kan benyttes i bergtekniske beregninger for bergrom, tunneler og skjæringer i stort monn baseres på observasjoner.

Bergmasseindeksen RMi er utviklet for å kunne gi en bedre karakterisering av bergmasser basert på slike observasjoner. Den finnes ved å koble sammen sprekkeparameteren (JP) (som representerer oppsprekningsgrad og sprekkekarakteristika) og bergartens enaksete trykkfasthet (σ_c) til uttrykket RMi = $\sigma_c \cdot$ JP. RMi er et materialteknisk uttrykk for en bergmasses enaksete fasthet. Dette gjør at RMi med fordel kan benyttes for ulike formål, hvorav kan nevnes beregning av: stabilitet / sikring, E-modul for bergmasser, inngangsparametre i Hoek-Brown bruddkriterium for bergmasser, inndrifter i TBM drevne tunneler.

I artikkelen er det vist en metode for å beregne stabilitet og sikring av tunneler og bergrom. I tillegg til RMi, benytter metoden spenningsforhold i berget, data for bergrommets dimensjoner samt orientering av sprekkene i forhold til bergrommet. For svakhetssoner inngår i tillegg sonens tykkelse og orientering, samt sidefjellets beskaffenhet. Det er presentert tabeller og matematiske uttrykk for de ulike parametre som benyttes. Dette muligjør anvendelsene av i computere i beregningene.

1 INTRODUCTION

"I see almost no research effort being devoted to the generation of the basic input data which we need for our faster and better models and our improved design techniques. These tools are rapidly reaching the point of being severely data limited."
Evert Hoek, 1994

As there is great diversity both in the composition of the intact rock and in the nature and extent of its discontinuities, rock masses exhibit a wider range in structure, composition and mechanical properties than most other construction materials. Reliable tests of the strength of such complex materials are impossible or so difficult to carry out with today's technology, that rock engineering pertaining to rock masses is currently based mainly on qualitative observational data. These qualitative observational data have to be expressed as numerical values to make calculations in rock engineering possible.

Construction materials commonly used in civil engineering and mining are mostly characterized by their strength properties.. This basic property of the material is used in the engineering and design. In rock engineering, no such specific strength characterization of the rock mass is in common use. Most engineering is carried out using various descriptions, classifications and unquantified experience. Hoek and Brown (1980), Bieniawski (1984), Nieto (1983) and several other authors have, therefore, indicated the need for a *strength characterization* of rock masses.

2 THE ROCK MASS INDEX, RMi

The Rock Mass index, RMi, has been developed to characterize the strength of the rock mass for construction purposes. An important issue has been to use well defined geological parameters in the RMi which have the greatest significance in engineering. This is discussed in detail by Palmström (1995).

The RMi can be applied in various types of rock engineering with adjustment for features related to the particular project or utilisation of the rock. These applications are described in Section 2.2.

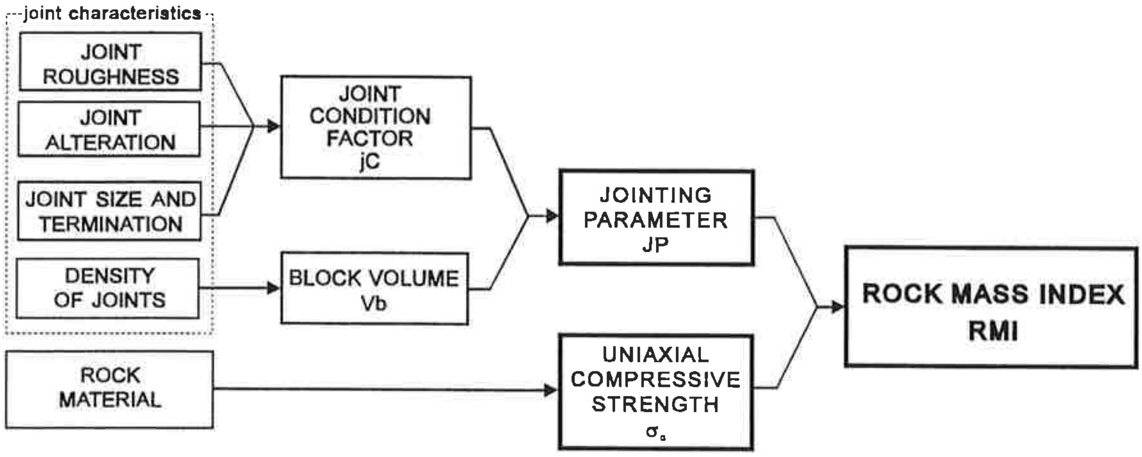

Figure 1 The main inherent parameters in the rock mass are applied in the RMi (from Palmström, 1995).

RMi applies only *intrinsic parameters* of the rock mass, see Figure 1. The importance to use such parameters in characterizing rock masses has earlier been stressed by Patching and Coates (1968).
As the RMi is principally based on the reduction in strength of a rock caused by jointing,[1] it is expressed as:

[1] The term 'joint' has been used for most natural discontinuities which have thickness smaller than approx. 0.1 m. Thus, joints cover fissures, partings, fractures, natural cracks, as well as many shears and seams.

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

where σ_c = the uniaxial compressive strength of intact rock measured on 50 mm samples;
JP = the jointing parameter which is a reduction factor representing the degree of jointing (i.e. block size) and the characteristics of the joints (roughness, alteration and size).

The influence of JP has been found using calibrations from test results. Because of problems of obtaining compression test results on rock masses at a scale similar to that of typical rock works, it was possible to find appropriate data from only eight large scale tests and one back analysis. These have been used to arrive at the following mathematical expression:

$$\text{JP} = 0.2\sqrt{\text{jC}} \cdot \text{Vb}^{\text{D}} \qquad \text{eq. (2)}$$

where jC is the joint condition factor expressing the characteristics of the joints,
Vb is the block volume is given in m^3, and
$D = 0.37\,\text{jC}^{-0.2}$ has the following values:

for jC =	0.1	0.25	0.5	0.75	1	1.5	2	2.5	3	4	6	9	12	16
D =	0.586	0.488	0.425	0.392	0.37	0.341	0.322	0.308	0.297	0.28	0.259	0.238	0.225	0.213

The joint condition factor in eq. (2) is expressed as jC = jL (jR/jA) where jL, jR and jA are factors for respectively, joint length and continuity, joint wall roughness, and joint surface alteration. Their ratings are shown in Tables 1 to 3. The factors jR and jA are similar to the joint roughness number (Jr) and the joint alteration number (Ja) in the Q-system.[2] The joint size and continuity factor (jL) has been introduced in the RMi system to represent the scale effect of the joints.

The value of JP varies from near 0 for crushed rocks to 1 for intact rock. The exponential form of eq. (2) fits well with the general experience that joint spacings have an exponential statistical distribution as shown by Merritt and Baecher (1981).

Most commonly, the joint condition factor jC = 1 to 2; for jC = 1.75 the jointing parameter is simply expressed as:

$$\text{JP} = 0.25\,\sqrt[3]{\text{Vb}} \qquad \text{eq. (2a)}$$

Eqs. (1) and (2) are related to *jointed* rock masses. For massive rock masses, i.e. rock masses with few joints where JP is close to 1, RMi depends mainly of the strength of the rock material. For this case the uniaxial compressive strength of the rock material (σ_c) found from tests on 50 mm samples cannot be applied directly in RMi, which generally involves volumes of several m^3, because of *scale effects.*[3] Barton (1990) suggests from data presented by Hoek and Brown (1980) and Wagner (1987), that the actual compressive strength for large 'field samples' may be determined from:

$$\sigma_c f = \sigma_c\,(0.05/\text{Db})^{0.2} = \sigma_c \cdot f_\sigma \qquad \text{eq. (3)}$$

where Db = block diameter measured in m, and
$f_\sigma = (0.05/\text{Db})^{0.2}$ is the scale factor for compressive strength.

From this

$$\text{RMi} = \sigma_c \cdot f_\sigma \qquad \text{eq. (3a)}$$

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[2] The symbols Jr and Ja have been changed into jR and jA because some minor modifications have been made in their definitions.

[3] During the calibration of JP the scale effect has been included in eq. (2).

Eq. (3) is valid for sample diameters up to some metres, and may, therefore, be applied for massive rock masses. The block diameter (Db) may be found from $Db = \sqrt[3]{Vb}$ or, in cases where a pronounced joint set occurs, from Db = S, where S is the spacing of this set.

If the block shape factor (β) is known (see Appendix, Section A4) the block diameter is:

$$Db = \frac{\beta_o}{\beta}\sqrt[3]{Vb} = \frac{27}{\beta}\sqrt[3]{Vb} \qquad \text{eq. (4)}$$

TABLE 1 THE JOINT ROUGHNESS FACTOR (jR) (the ratings of jR are similar to Jr in the Q-system)

Small scale smoothness of joint surface	Large scale wavyness of joint plane				
	Planar	Slighty 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 filled joints: jR = 1 For irregular joints a rating of jR = 5 is suggested					

*) For slickensided joints the rating of jR depends on the presence and appearance of striations; the highest value is used for marked striations

TABLE 2 THE JOINT ALTERATION FACTOR (jA) (the ratings of jA are similar to Ja in the Q-system)

A. CONTACT BETWEEN THE TWO JOINT WALLS

Joint wall character	Description	Rating of *jA*
CLEAN JOINTS:		
Healed or welded joints	Non-softening, impermeable filling (quartz, epidote, etc.)	0,75
Fresh joint walls	No coating or filling in joint, except from staining (rust)	1
Altered joint walls		
1 grade higher	One grade higher alteration than the rock in the block	2
2 grades higher	Two grades higher alteration than the rock in the block	4
COATINGS OR THIN FILLING OF:		
Friction materials	Materials of sand, silt calcite, etc. without content of clay	3
Cohesive materials	Materials of clay, chlorite, talc, etc.	4

B. FILLED JOINTS WITH PARTLY OR NO JOINT WALL CONTACT

Type of filling	Description	*Partly wall contact* Thin filling (approx. < 5 mm) Rating of *jA*	*No wall contact* Thick filling or gouge Rating of *jA*
Friction materials	Sand, silt calcite, etc. without content of clay	4	8
Hard cohesive materials	Compacted filling of clay, chlorite, talc, etc.	6	10
Soft cohesive materials	Medium to low overconsolidated clay, chlorite, talc, etc.	8	12
Swelling clay materials	Filling material exhibits swelling properties	8 - 12	12 - 20

TABLE 3 THE JOINT SIZE FACTOR (jL)

Joint length	Term	Type	Continuous joints*) Rating of *jL*	Discontinuous joints Rating of *jL*
< 0.5 m	Very short	Bedding or foliation partings	3	6
0.1 - 1 m	Short or small	Joint	2	4
1 - 10 m	Medium	Joint	1	2
10 - 30 m	Long or large	Joint	0,75	1,5
> 30 m	Very long or large	(Filled) joint, seam or shear **)	0,5	1

**) Often a singlularity and should in these cases be treated separately

*) Discontinuous joints end i massive rock

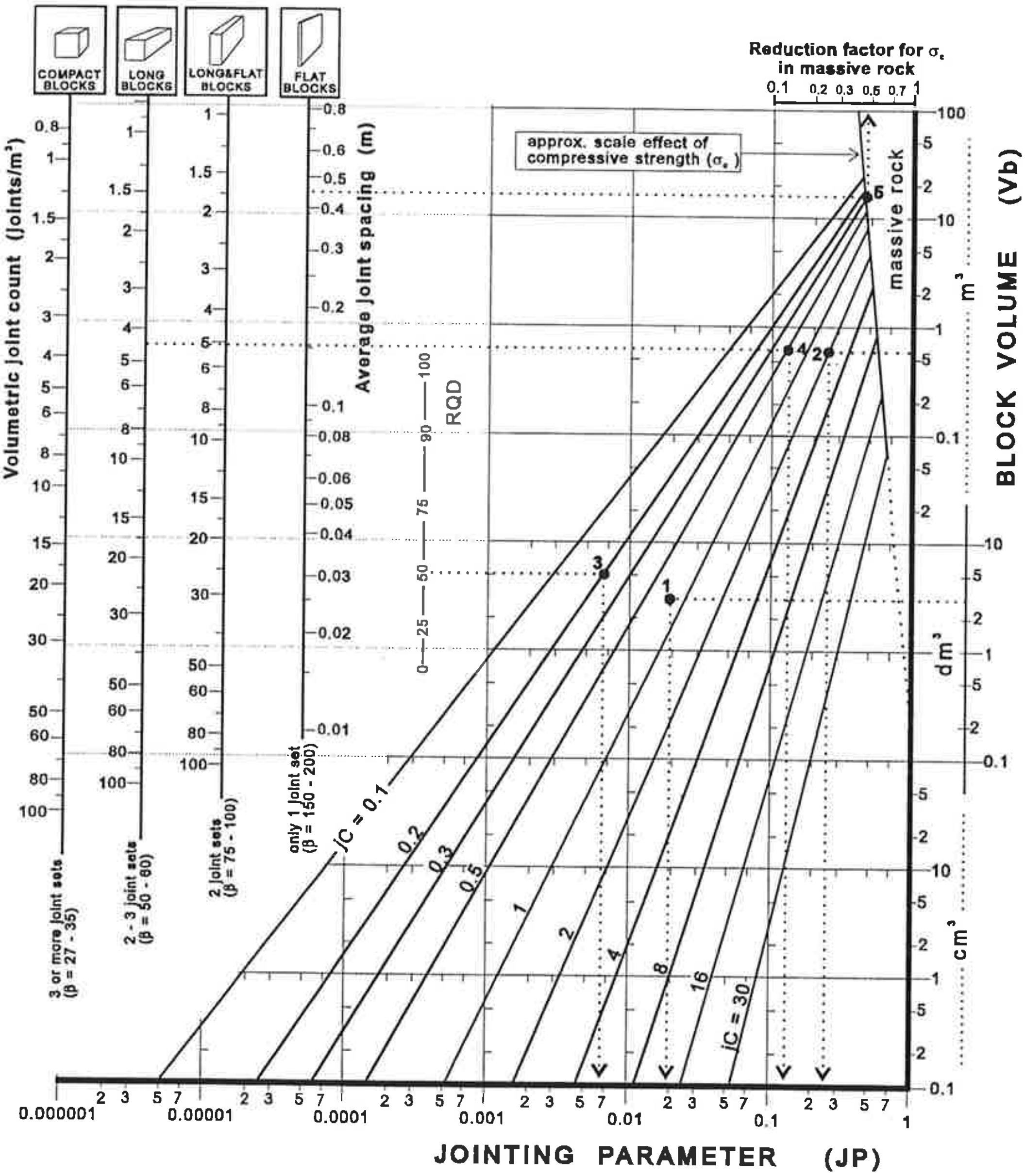

Figure 2 *The jointing parameter (JP) found from the joint condition factor (jC) and various measurements of jointing intensity (Vb, Jv, RQD). The determination of JP from Vb (or RQD or Jv) in the examples are indicated (from Palmström, 1996a).*

TABLE 4 CLASSIFICATION OF RMi. From Palmström (1995)

TERM		
For RMi	**Related to rock mass strength**	**RMi Value**
Extremely low	Extremely weak	< 0.001
Very low	Very weak	0.001 - 0.01
Low	Weak	0.01 - 0.1
Moderate	Medium	0.1 - 1
High	Strong	1 - 10
Very high	Very strong	10 - 100
Extremely high	Extremely strong	> 100

Figure 2 shows how the jointing parameter (JP) can be found from the block volume (Vb) and the joint condition factor (jC). As shown in the upper left part of the diagram, the volumetric joint count (Jv) for various joint sets (and/or block shapes) can be used instead of the block volume. Also, the RQD can be used, but its inability to characterise massive rock or highly jointed rock masses leads to a reduced quality of JP as shown by Palmstrom (1995).

2.1 Examples

The values of the jointing parameter (JP) found in the following examples are also shown in Figure 2.

Example 1
The block volume has been measured as Vb = 0.003 m^3 (= 3 dm^3). As given in Tables 1 to 3, the joint condition factor jC = 0.75 is determined from:
- the rough joint surfaces and small undulations of the joint wall which give jR = 3;
- the clay coated joints, i.e. jA= 4; and
- the 3 - 10 m long, continuous joints, which give jL = 1.

Applying the values for Vb and jC in Figure 2, a value of JP = 0.02 is found.[4] With a compressive strength of the rock σ_c = 150 MPa, the value of RMi = 0.02 · 150 = 3 (high).

Example 2
The block volume Vb = 0.6 m^3. The joint condition factor jC = 2 is determined from Tables 1 to 3, based on:
- smooth joint surfaces and planar joint walls which give jR = 4;
- fresh joints, jA= 1; and 1 - 3 m long discontinuous joints, i.e. jL = 3.

From Figure 2 the value JP = 0.25 is found.[5] With a compressive strength σ_c = 50 MPa of the rock, the value of RMi = 12.5 (very strong).

Example 3
Values of RQD = 50 and jC = 0.2 give JP = 0.007 using Figure 2.

Example 4
Two joint sets spaced 0.3 m and 1 m, and some random joints have been measured. The volumetric joint count is [6] Jv = 1/0.3 + 1/1 + 0.5 = 4.5
With a joint condition factor jC = 0.5 the jointing parameter JP = 0.12 (by using the column for 2 to 3 joint sets in Figure 2)

Example 5
The following jointing features are measured: one joint set with spacing S = 0.45 m, and a joint condition factor jC = 8. For this massive rock it is seen in Figure 2 that the value of JP is determined from the scale factor for compressive strength f_σ = 0.45. For a rock with σ_c = 130 MPa the value of RMi = 59.6 (very strong).

2.2 Possible applications of the RMi

The main purpose during development of the RMi has been to work out a practical system to characterize rock masses which is applicable to rock engineering and design. Figure 3 shows the main areas for application of RMi.

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[4] Using eq. (2) a value of JP = 0.018 is found
[5] JP = 0.24 is found using eq. (2)
[6] A value of 0.5 is assumed for the random joints

The RMi-value can seldom be used directly in classification systems as many of them are systems made for a particular purpose. Some of the input parameters in RMi are sometimes similar to those used in the classifications and may then be applied more or less directly.

The system for characterizing block geometry (volume, shape factor, angles) and the fact that RMi expresses the strength of a rock mass, may be of use in numerical models.

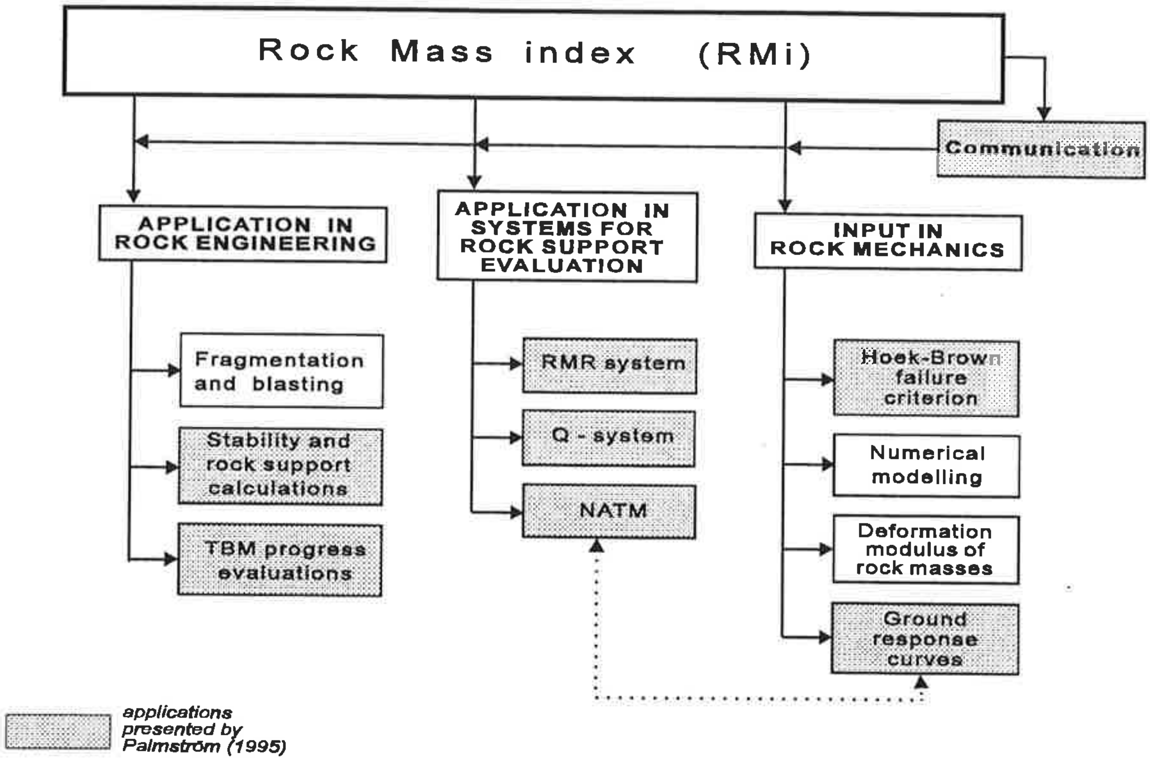

Figure 3 The main applications of RMi in rock mechanics and rock engineering (from Palmström, 1995).

3 RMi USED TO EVALUATE 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 as well as national regulations and experience. Support design for a tunnel in rock often involves problems that are of relatively little or no concern in most other branches of solid mechanics. 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.

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 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 the following main groups:

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, brittle rock, which takes place in the mode of spalling, popping, rock burst etc.
   b. Overstressing of massive or intact ductile rock, which takes place in the mode of squeezing.
   c. Overstressing of particulate materials, i.e. soils and heavy jointed rocks, where squeezing and creep may take place.

3. Instability in *faults and weakness zones*. Such features 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. Bieniawski (1984, 1989) therefore recommends that faults and other weakness zones are mapped and treated as regions of their own.

3.2 Combination of the ground characteristics for support evaluations

The stability and behaviour of the rock mass surrounding an underground opening is mainly the combined result of the strength and structure of the rock mass and the stresses acting. Their importance will vary with the shape and size of the opening. In the selection of these parameters it has been found beneficial to combine those parameters in the ground which have a similar effect on the stability, into the following two groups:

1. The continuity of the ground

This expresses whether the volume of rock masses involved in the excavation can be considered discontinuous or not, see Figure 4. 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 range of the rock blocks compared to the 'sample' volume, i.e. the tunnel size. The continuity factor based on Deere et al. (1969) is expressed a as the ratio:

CF = tunnel diameter/block diameter = Dt/Db   eq. (5)

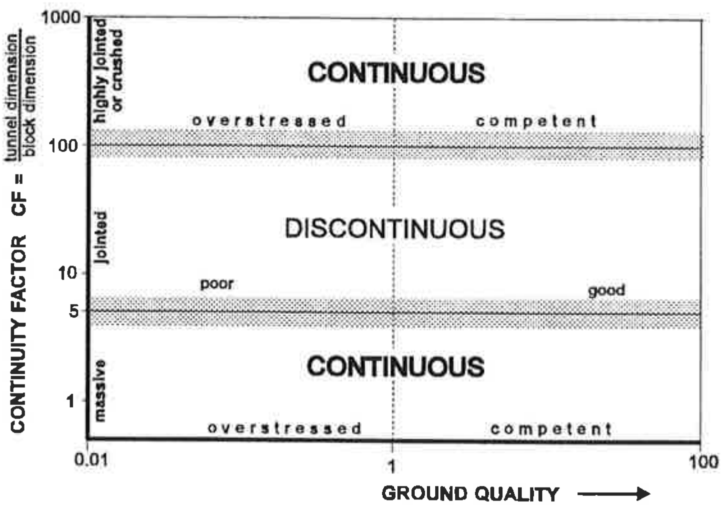

Figure 4 The division of the ground into continuous and discontinuous rock masses. The various groups of ground behaviour are indicated (from Palmström, 1995).

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.

2. *The condition (quality) of the ground*
   This factor is composed of 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 support are shown in Figure 5.

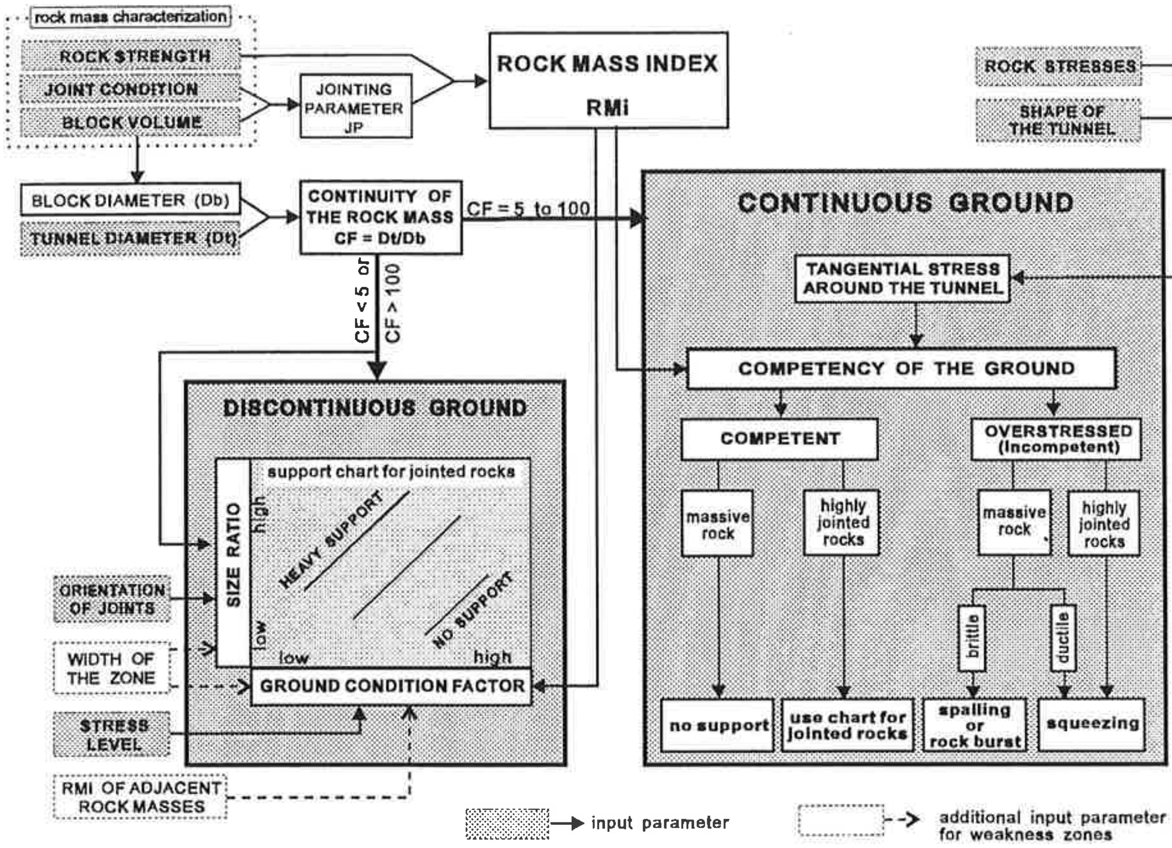

Fig. 5 The parameters involved in the RMi method for stability and rock support. For weakness zones the size ratio and the ground condition factor are adjusted for parameters of the zone as indicated (from Palmström, 1996b).

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 compressive strength of the rock mass, it can be used in assessing the *competency factor* given as:

$$Cg = RMi / \sigma_\theta \qquad \text{eq. (6)}$$

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, refer to Hoek and Brown (1980).

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

In massive rock the competency factor is:

$$Cg = RMi / \sigma_\theta = f_\sigma \cdot \sigma_c / \sigma_\theta \qquad \text{eq. (7)}$$

where f_σ is the scale effect for the uniaxial compressive strength is given in eq. (3)

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

$$Cg = RMi / \sigma_\theta = JP \cdot \sigma_c / \sigma_\theta \qquad \text{eq. (8)}$$

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, see Section 3.3.1.
- 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 *flexible or ductile* rocks such as soapstone, evaporites, clayey rocks (mudstones, clay schist, etc.) or weak schists. This type of instability is not further dealt with in this paper. For more information refer to Palmstrom (1995, 1996b).

3.3.1 Rock burst and spalling in brittle rocks

Rock burst is also known as *spalling* [7] 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° 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) [8] 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. The results of these works are shown in Table 5.

In massive rocks eq. (3a) can be applied. As the factor for the scale effect of compressive strength has values in the range f_σ = 0.45 to 0.55, the value of RMi $\approx$ 0.5 σ_c. Hence the competency factor in

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[7] 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"*.

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

Table 6 is $Cg = RMi/\sigma_\theta = f_\sigma \cdot \sigma_c / \sigma_\theta \approx 0.5\ \sigma_c / \sigma_\theta$, i.e. approximate half the values given for the ratio σ_c / σ_θ in Table 5.

TABLE 5 ROCK BURST ACTIVITY RELATED TO THE RATIO σ_c / σ_θ BASED ON HOEK AND BROWN (1980), RUSSENES (1974), AND GRIMSTAD AND BARTON (1993) (from Palmström, 1996b)

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 rockburst
< 1.4		< 1	Severe (sidewall) rock burst problems / Heavy rockburst.

TABLE 6 CHARACTERIZATION OF FAILURE MODES IN BRITTLE, MASSIVE ROCK (from Palmström 1995)

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 ground may cause the values of the competency factor in Table 6 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 7. In addition to scaling, wire mesh and rock bolts were used earlier as reinforcement in this type of ground. This is now only occasionally applied in Norwegian tunnels.

TABLE 7 ROCK SUPPORT APPLIED IN NORWEGIAN TUNNELS UP TO APPROXIMATELY 15 m SPAN SUBJECTED TO ROCK BURST AND SPALLING. THIS INFORMATION IS APPLIED IN THE SUPPORT CHART IN FIGURE 6 (from Palmström 1995)

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

3.4 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 5. The failures occur when wedges or blocks, limited by joints, fall or slide from the roof or sidewalls. The properties of the intact rock are of relatively little importance in this type of ground 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 joints 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:

$$Gc = RMi \cdot SL \cdot C \qquad \text{eq. (9)}$$

SL = the stress level factor, which 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 partly based 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, 1995)

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

$$C = 5 - 4 \cos \theta \qquad \text{eq. (10)}$$

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:

$$Sr = (Dt / Db) (Co / Nj) \qquad \text{eq. (11)}$$

Dt = the diameter (span or wall height) of the tunnel, measured in m.

Db = the block diameter (in m) represented by the smallest dimension of the block which often corresponds to be the spacing of the main joint set. Db may roughly be found from eq. (4) or more roughly from Db = $\sqrt[3]{Vb}$. Often an *equivalent block diameter* is applied where joints do not delimit separate blocks (where less than 3 joint sets occur, see Appendix Section A7).

Nj = a factor representing the number of joint sets as an adjustment to Db in eq. (11) 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 adjustment by Nj is found from the expression:

$$Nj = 3/n_j \qquad \text{eq. (12)}$$

where n_j = the number of joint sets, see Also Appendix, Section A7.
(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.)

[9] 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 quality (Q = 0.1 - 10) and 1.0 for poor quality ground (Q < 0.1).

Co = an orientation factor representing the influence of the *orientation* of the joints encountered in the underground opening. Joints across the opening will have significantly less influence on the stability than parallel joints. The strike and dip in Table 9 are measured relative to the tunnel axis. Often, the orientation of the main joint set has the main influence and is applied.

TABLE 9 THE ORIENTATION FACTOR FOR JOINTS AND ZONES
(from Palmström, 1995, based on Bieniawski, 1984 and Milne et al., 1992).

IN WALL		IN ROOF	TERM	
for strike > 30°	for strike < 30°	for all strikes		**Rating of Co**
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

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. These aspects often depend on the geometry and the site conditions. They have, therefore, not been included in this general support 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. However, the system presented for discontinuous (jointed) rock masses in Section 3.4 has been found to cover also many types of weakness zones for which 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 (1995) 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):

$$RMi_m = (10Tz^2 \cdot RMi_z + RMi_a)/(10Tz^2 + 1) \qquad \text{eq. (13)}$$

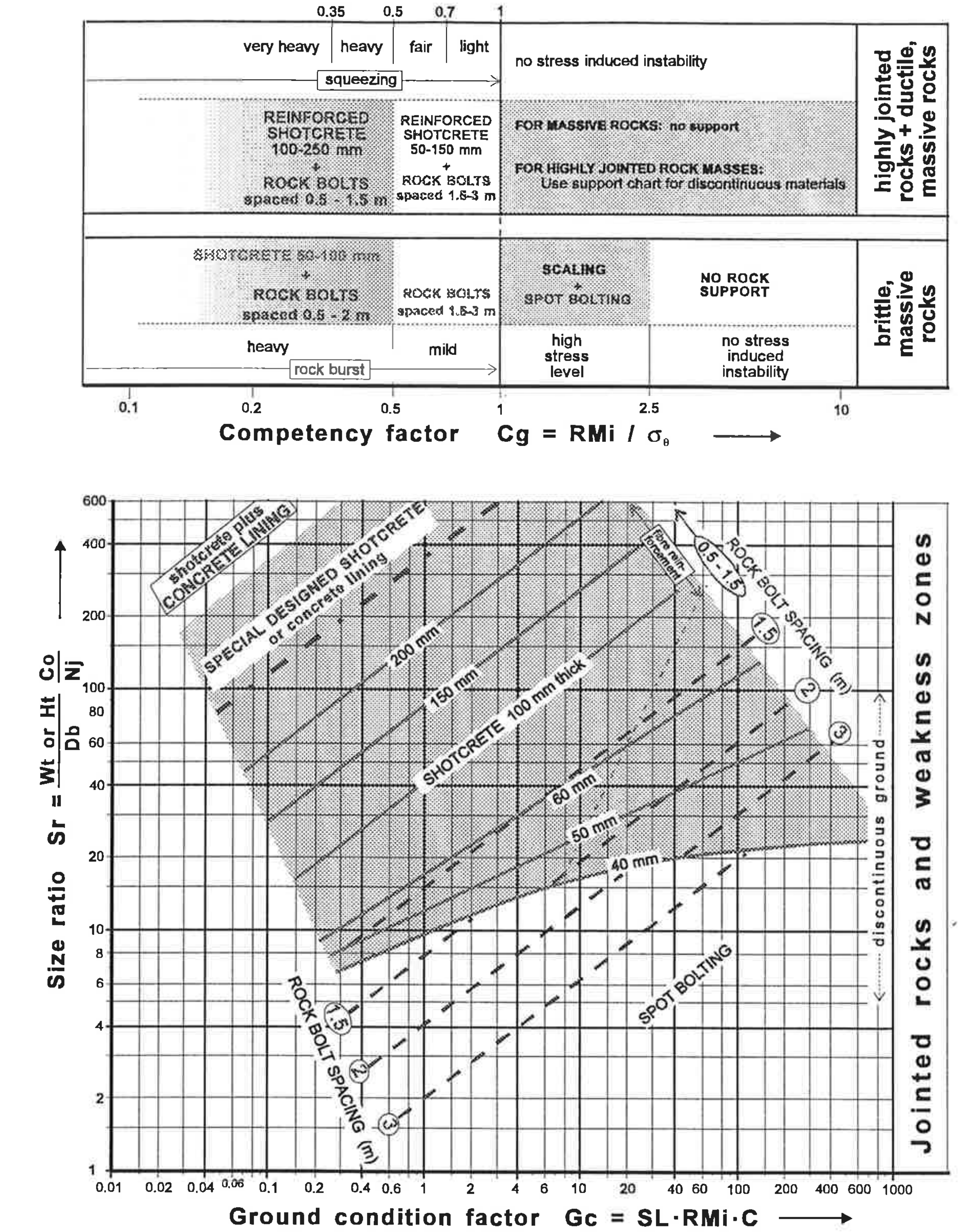
where Tz = the thickness of the zone in m; RMi_a refers to the weakness zone; RMi_z to the surrounding rock

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. (13). Applying eq. (13) in eq. (9), a ground condition factor for weakness zones can be found similarly to that for discontinuous (jointed) rock masses:

$$Gc_z = SL \cdot RMi_m \cdot C \qquad \text{eq. (14)}$$

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



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 6 *Rock support chart. The chart for 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, 1995)* eq. (11) is adjusted for the zone ratio Tz/Dt to form the following size ratio for zones: [10]

$$Sr_z = (Co_z / Nj_z)(Tz / Db_z) \qquad \text{eq. (15)}$$

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. (11)

Eq. (15) is valid is smaller than the diameter (span or height) of the tunnel. For thicker zones where $Tz > Dt$, eq. (11) should be applied.

3.6 Comments on the support chart

The support chart for *discontinuous* rock masses in Figure 8 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.

Work still remains, however, to develop better support chart for continuous rock masses.

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 in competent massive ground. 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 (medium gravel size, see Table A1 in Appendix) occur, $Vb = 1$ cm^3 or block diameter $Db \approx 0.01$ m shall be used.

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 $\sigma_c = 150$ 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.

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

Calculations:
From the joint condition factor $jC = jL \cdot jR/jA = 3$ the jointing parameter is $JP = 0.15$ (Figure 2 or eq. 2) which gives the rock mass index $RMi = 22.5$ (eq. 1)
As the block shape factor is $\beta = 39$ (eq. A5 or Figure A2), the block diameter is $Db = 0.26$ m (using eq. 4)
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	$Gc = RMi \cdot SL \cdot C = 22.5$	(eq. 9)
- the size ratio for the roof	$Sr = (Dt/Db)(Co/Nj) = 28.4$	(eq. 11)

Estimated rock support in roof:
The rock support according to Figure 6 is: shotcrete 40 - 50 mm thick and rock bolts spaced 2 m .

Example 2
A vertical <u>weakness zone</u> is encountered in the same tunnel, crossing at 60° ($Co_z = 1$ for the roof as given in Table 9). The 2 m thick zone 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, and continuous joints in the zone have coating of clay, i.e. $jC_z = jR \cdot jL / jA = 1 \cdot 2 / 4 = 0.5$
With 3 joint sets and some random joints in the zone ($n_j = 3.5$) the factor for the number of joint sets is $Nj_z = 3/3.5 = 0.86$ (eq. 12).

Calculations of the RMi parameters for weakness zone:

The jointing parameter in the zone is	$JP_z = 0.001$	(eq. 2)
The Rock Mass index in the zone is	$RMi_z = 0.16$	(eq. 1)
The combined Rock Mass index is	$RMi_m = 0.7$	(eq. 13)

With assumed block shape factor $\beta = 40$ the equivalent block diameter is $Db_z = 0.015$ m (eq. 4)
From the data above the following parameters are found for the zone:

- the ground condition factor for the roof	$Gc_z = 0.7$	(eq. 14)
- the size ratio for the roof	$Sr_z = 160.4$	(eq. 15)

Estimated rock support in the weakness zone:
The rock support according to Figure 6 is: 200 mm thick fibre reinforced shotcrete and rock bolts spaced 0.5 - 1.5 m.

4 DISCUSSION

The RMi system has been worked out to cover as many type of ground as possible. Consequently, the many expressions presented may at first seem complicated. Normally, at site only a few of the parameters or factors vary. Thus the work required to use the system in practice is limited after the main principles have been learned. By using a computer spreadsheet in the calculations much work can be saved.

4.1 Comments on the application of RMi in stability and rock support

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. On other factors which influence the stability in underground openings, the following comments are made:

- The effect from *swelling* of some rocks and of some gouge or filling material in seams and faults, has not been included.[11] 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 also other aspects which should be evaluated separately. They include safety requirements, vibrations from earthquakes or from nearby blasting and other disturbances from the activity of man.

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 in the appendix to assess an equivalent block volume. An additional problem is to indicate methods to characterize the variations in block size. Therefore, engineering calculations should generally be based on a variation range.
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 volume caused by the excavation, the adjusted values for (Sr) and (Gc) can be calculated readily and thus the impact from excavation in the assessments of rock support can be included.

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.

The structure of RMi and its use in rock support engineering allows for accurate calculations where high quality data are available. For the following 'common' conditions for the joint characteristics and a 'normal' hard rock, simplified expressions for the ground condition factor (Gc) and the size ratio (Sr) can be applied when rough support estimates are sufficient. Such ground features may be:

- $\mathrm{RMi} = 40\sqrt[3]{\mathrm{Vb}}$ (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 ($SL = 1$).

Applying these data in eqs. (9) and (11), the following expressions may be used to find the amount of support in Figure 6:

11) 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 the RMi.

• The ground condition factor:	for roof	$Gc = 0.25\ \sigma_c \cdot \sqrt[3]{Vb}$	eq. (16)
	for walls	$Gc = 1.25\ \sigma_c \cdot \sqrt[3]{Vb}$	eq. (17)
• The size ratio:	for roof	$Sr = Wt / \sqrt[3]{Vb}$	eq. (18)
	for walls	$Sr = Ht / \sqrt[3]{Vb}$	eq. (19)

where Wt = width (span) and Ht = (wall) height of the tunnel

As this only requires input from the block volume, 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 preferably be worked into a computer spreadsheet.

Descriptions and collection of input data require, however, involvement of experienced persons, as is the case for most rock engineering projects.

What is new in 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 properties of 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 is new in the RMi support method. The different influence from the rock stresses is, however, reflected for the two types of ground. 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 an average stress level factor (SL) has been selected.

The introduction of the size ratio (tunnel size/block size for discontinuous ground) is also a new feature.

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.

4.2 Benefits and limitations of the RMi

Some of the benefits of the RMi system are:

- It *will give significant improvements in the use of geological input data*
  This is mainly achieved by its systematic use of well defined parameters in which the three-dimensional character of rock masses is represented by the block volume.
- *It can easily be used for rough estimates when limited information is available on the ground conditions.*
  For example, in early stages of a project where rough estimates are sufficient, eq. (2a) can be applied.
- *It is well suited for comparisons and exchange of knowledge between different locations.*
  In this way it may contribute to improved communication between people involved in rock engineering and design.
- *It covers a wide spectrum of rock mass variation.*
  It therefore has possibilities for wider applications than other rock mass classification and characterization systems of today.

Any attempt to mathematically express the variable structure and properties of jointed rock masses in a general failure criterion, may result in complex expressions. By restricting the RMi to uniaxial compressive strength only, it has been possible to arrive at the relatively simple expressions in eqs. (1) and (2). Because simplicity has been preferred in the structure as well as in the selection of parameters in RMi, it is clear that such an index may result in inaccuracy and limitations.

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7 REFERENCES

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. (1990):
Scale effects or sampling bias? Proc. Int. Workshop Scale Effects in Rock Masses, Balkema Publ., Rotterdam, pp. 31-55.
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.
Brekke T.L. and Howard T.R. (1972):
Stability problems caused by seams and faults. Rapid Tunneling & Excavation Conference, 1972, pp. 25-41.
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.
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 Brown E.T. (1980):
Underground excavations in rock. Institution of Mining and Metallurgy, London 1980, 527 pp.
Hoek, E. (1983):
Strength of jointed rock masses. The Rankine Lecture 1983, Geotechnique 33, no 3 pp 187-223.
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.
Matula M. and Holzer R. (1978):
Engineering topology of rock masses. Proc. of Felsmekanik Kolloquium, Grundlagen und Andwendung der Felsmekanik, Karlsruhe, Germany, 1978, pp. 107-121.
Merritt A.H. and Baecher G.B. (1981):
Site characterization in rock engineering. 22nd U.S. Symp. on Rock Mechanics, pp. 49-66.
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.
Nieto A.S. (1983):
Some geologic factors in the location, design and construction of large underground chambers in rock. Proc. Rapid Excavation & Tunneling Conf. AIME 1983, pp. 569-596.

Patching T.H. and Coates D.F. (1968):
A recommended rock classification for rock mechanics purposes. CIM Bull., Oct. 1968, pp. 1195-1197.

Piteau D.R. (1970):
Geological factors significant to the stability of slopes cut in rock. Proc. Symp. on Planning Open Pit Mines, Johannesburg, South Africa, 1970, pp. 33-53.

Palmström, A. (1982):
The volumetric joint count - a useful and simple measure of the degree of jointing. Proc. IV Int. Congr. IAEG, New Delhi, 1982, pp V.221-V.228.

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

Palmström A. (1996a):
Characterizing rock masses by the RMi for use in practical rock engineering. Part 1: The development of the Rock Mass index (RMi). Tunnelling and Underground Space Technology, Vol. 11, No. 2, pp. 175 - 188.

Palmström A. (1996b):
Characterizing rock masses by the RMi for use in practical rock engineering. Part 2: Some practical applications of the Rock Mass index (RMi). Tunnelling and Underground Space Technology, Vol. 11, No. 3, pp. 287 - 303.

Russenes B.F. (1974):
Analysis of rock spalling for tunnels in steep valley sides (in Norwegian). M.Sc. thesis, Norwegian Institute of Technology, Dept. of Geology, 247 pp.

Selmer-Olsen R. (1964):
Geology and engineering geology. (in Norwegian) Tapir publishers, Trondheim, Norway, 409 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.

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

Wagner H. (1987):
Design and support of underground excavations in highly stressed rock. Proc. 6th ISRM Congr., Montreal; Keynote paper Vol. 3.

APPENDIX
METHODS AND CORRELATIONS TO DETERMINE THE BLOCK VOLUME

A1 Introduction

The block size is usually the most important factor in the RMi. Consequently, the accuracy of this measure has a significant impact on the quality of the RMi. This appendix presents methods to determine the block volume from various types of jointing observations and measurements. A summary of these methods is shown in Figure A1. A classification of the block size for rock massess and the particle size for soils is presented in Table A1.

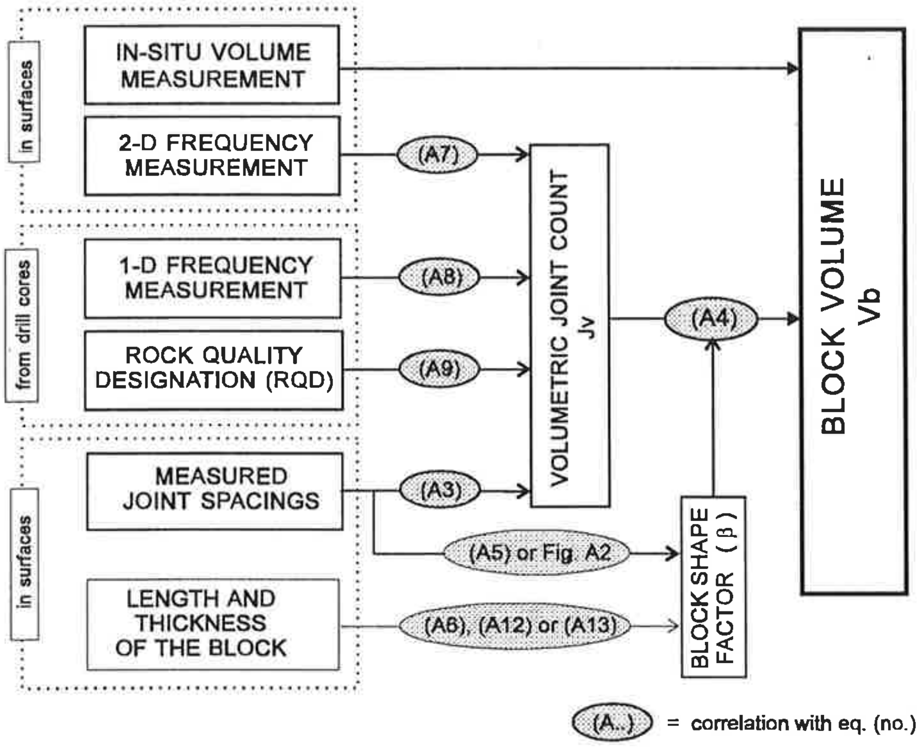

Figure A1 Summary of correlations presented in this Appendix to estimate the block volume from various types of joint density measurements (based on Palmström, 1995)

TABLE A1 CLASSIFICATION OF BLOCK VOLUME RELATED TO PARTICLE SIZE FOR SOILS From Palmström (1995)

TERM FOR DENSITY OF JOINTS	**TERM FOR BLOCK SIZE**	**Block volume*) (Vb)**	**TERM FOR SOIL PARTICLE**	**Approximate particle volume*)**
			Coarse sand	0.1 - 5 mm^3
			Fine gravel	5 - 100 mm^3
Extremely high	Extremely small	< 10 cm^3	Medium gravel	0.1 - 5 cm^3
Very high	Very small	10 - 200 cm^3	Coarse gravel	5 - 100 cm^3
High	Small	0.2 - 10 dm^3	Cobbles	0.1 - 5 dm^3
Moderate	Moderate	10 - 200 dm^3	Boulders	5 - 100 dm^3
Low	Large	0.2 - 10 m^3	Blocks	> 0.1 m^3
Very low	Very large	10 - 200 m^3		
Extremely low	Extremely large	> 200 m^3		

*) $Vb = 0.58\ Db^3$ has been applied in the correlation between particle diameter and particle or block volume.

If less than 3 joint sets occur, defined blocks may not be found. However, in many cases the presence of random joints or other weakness planes may contribute to defining blocks. Where the jointing is irregular, or many of the joints are discontinuous, it can sometimes be difficult to recognize the actual size and shape of individual blocks. Thus, from time to time the block size and shape therefore have to be determined using a simplification where an *equivalent block volume* is used as is described in Section A7.

Especially where irregular jointing occurs, it is time-consuming to measure all (random) joints in a joint survey. In such cases, as well as for other jointing patterns, it is often much quicker - and also more accurate - to measure the block volume directly in the field.

A2 Block volume found from joint spacings

The terms *joint spacing* and *average joint spacing* are often used in the description of rock masses. Joint spacing is the distance between individual joints within a joint set. Where more than one set occurs, this measurement is, in the case of surface observations, often given as the average of the spacings for these sets.
There is often some uncertainty as to how this average value is found; for instance, the average spacing for the following 3 joint sets having spacings S1 = 1 m, S2 = 0.5 m, and S3 = 0.2 m is Sa = 0.125 m, and not 0.85 m which initially may seem appropriate.[12]

As the term 'joint spacing' does not indicate what it includes, it is frequently difficult to determine whether a 'joint spacing' referred to in the literature represents the true joint spacing. Thus, there is often much confusion related to joint spacing recordings.

Where three regular joint sets occur, the block volume can easily be found from the joint spacings as

$$Vb = \frac{S1 \cdot S2 \cdot S3}{\sin\gamma 1 \cdot \sin\gamma 2 \cdot \sin\gamma 3} = \frac{Vb_0}{\sin\gamma 1 \cdot \sin\gamma 2 \cdot \sin\gamma 3} \qquad \text{eq. (A1)}$$

where $\gamma 1, \gamma 2, \gamma 3$ are the angles between the joint sets, and
S1, S2, S3 are the spacings between the individual joints in each set.
Vb_o is the block volume in cases where joints intersect at right angles.

For a rhombohedral block with two angles between 45° and 60°, two between 135° and 150° and the last two being 90°, the volume will be between $Vb = 1.3\ Vb_o$ and $2\ Vb_o$. Compared to the variations caused by the joint spacings, the effect from the intersection angle between joint sets is relatively small.

A3 Block volume calculated from the volumetric joint count

The volumetric joint count (Jv) has been described by Palmström (1982, 1985, 1986) and Sen and Eissa (1991, 1992). It is a measure of the number of joints within a unit volume of rock mass, defined by

$$Jv = \Sigma\,(1/S_i) \qquad \text{eq. (A2)}$$

where S_i = the joint spacing in metres for the each joint set *i*.

Also random joints can be included by assuming a random spacing for each of these. Experience indicates that this can be set to Sr = 5 m; thus, the volumetric joint count can be generally expressed as

$$Jv = \Sigma\,(1/S_i) + Nr/5 \qquad \text{eq. (A3)}$$

where Nr = the number of random joints. A more accurate determination of Nr has been shown by Palmström (1995).

[12] The average spacing is found from 1/Sa = 1/S1 + 1/S2 + 1/S3

Since both the volumetric joint count (Jv) and the size of blocks in a rock mass vary according to the degree of jointing, there is a correlation between them (Palmström, 1982). Jv varies with the joint spacings, while the block size also depends on the type of block. A correlation between the two parameters has therefore to contain adjustment factor for the block shape in addition to the angle between the joint sets, as shown by Palmström (1995):

$$Vb = \beta \cdot Jv^{-3} \frac{1}{\sin\gamma 1 \cdot \sin\gamma 2 \cdot \sin\gamma 3} \qquad \text{eq. (A4)}$$

where $\gamma 1$, $\gamma 2$, and $\gamma 3$ are the angles between the joint sets.

The factor β is expressed as

$$\beta = \frac{(\alpha 2 + \alpha 2 \cdot \alpha 3 + \alpha 3)^3}{(\alpha 2 \cdot \alpha 3)^2} \qquad \text{eq. (A5)}$$

where $\alpha 2 = S2/S1$ and $\alpha 3 = S3/S1$

In cases where all angles between the block faces are 90°, eq. (A4) is given as

$$Vb_o = \beta \cdot Jv^{-3} \qquad \text{eq. (A4a)}$$

As β depends mainly on the differences between the joint set spacings, it has been named the *block shape factor* as further described in Section A4.

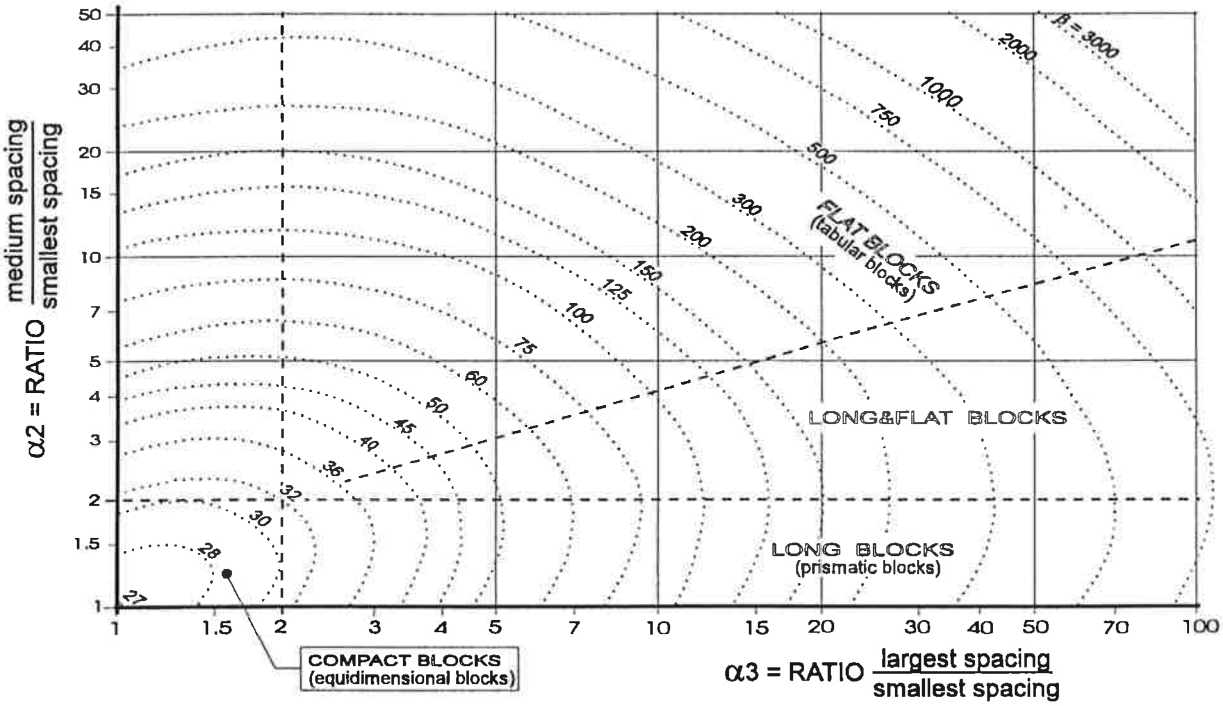

Figure A2 Block types characterized by the block shape factor (β) found from the ratio between spacings of the joint sets. The data are based on 3 joint sets intersecting at right angles (from Palmström, 1995).
Example: For $\alpha 2 = 4$ and $\alpha 3 = 15$, $\beta = 135$.

By definition the volumetric joint count (Jv) takes into account in an unambiguous way all the occurring joints in a rock mass. It is, therefore, often appropriate to use Jv in the correlation between joint frequency measurements and block volume estimates as shown by Palmström (1982 and 1995). Important here is the block shape factor β which is included in all equations to estimate the block volume. Where β is not known, it is recommended to use a 'common' value of $\beta = 40$.

A4 Block types and shapes

The *type and shape* of blocks are determined by:

- the number of joint sets;
- the differences in joint spacings; and
- the angles between the joints or joint sets.

For a rock mass with 3 joint sets intersecting at right angles the values of β are given Figure A2. The descriptive terms applied by Palmström (1995) are also shown in in this figure. For $\beta = 27$ to 32 the block term 'compact' has been introduced to include cubical, equidimensional, blocky and other existing terms for blocks not being long or flat.

The use of Figure A2 requires the presence of 3 joint sets. As blocks often have more than six faces or have irregular shape, it can be difficult to estimate β. Therefore, the following simplified method to estimate β has been developed by Palmström (1995), in which the longest and shortest dimension of the block are applied:

$$\beta = 20 + 7\ a3/a1 = 20 + 7\ \alpha 3 \qquad \text{eq. (A6)}$$

where a3 and a1 are the longest and shortest dimension of the block.

The evaluations made by Palmström (1995) have shown that eq. (A6) covers most types of blocks (where $\beta < 1000$) within reasonable accuracy (± 25%). For very flat to extremely flat blocks eq. (A6) has limited accuracy.

A5 Block volume found from joint frequency measurements

When the frequency is given for each joint set, it is possible to find the block volume directly. In other cases, when an 'average frequency' is given, it is uncertain whether this frequency value refers to one-, two- or three-dimensional measurements; hence no accurate correlation can be presented. The use of joint frequency measurements presented in the following are similar to the joint spacing measurements shown in Section A2.

A5.1 From 2-D joint frequency measurements on an area or surface

The correlation between 2-D measurements of the joint density in a rock surface and the 3-D frequency values (given as Jv) can be done using the empirical expression

$$Jv = Na \cdot ka \qquad \text{eq. (A7)}$$

where ka = a correlation factor, which varies mainly between 1 and 2.5 with an average value ka = 1.5 as shown by Palmström (1995). It has its highest value where the observation plane is parallel to the main joint set.

A5.2 From 1-D jointing frequency measurements along a scanline or drill core

This is a record of the joint frequency along a borehole or a scanline given as the number of joints intersecting a certain length. As in other core logging methods, it is important to measure the joints in sections along the line or core which shows similar joint frequency. At the start of the logging it is rational to divide the length into such sections.

The correlation between 1-D joint frequency observations in drill holes (or scanlines) and volumetric 3-D frequency (Jv) can be done using an expression similar to eq. (A-10). The joint frequency, given as the number of joints per metre, can be expressed as:

$$Jv = Nl \cdot kl \qquad \text{eq. (A8)}$$

where kl = a correlation factor, which varies between 1.25 and 6, with an average value kl = 2. Palmström (1995) has shown that there is a rather poor correlation between Jv and Nl.

A6 A correlation between RQD and the volumetric joint count (Jv)

It is not possible to obtain good correlations between RQD and Jv or between RQD and other measurements of jointing. Palmström (1982) presented the following simple expression:

$$RQD = 115 - 3.3\ Jv \qquad \text{eq. (A9)}$$

Here RQD = 0 for Jv > 35, and RQD = 100 for Jv < 4.5

Especially where many of the core pieces have lengths around 0.1 m, the correlation above may inaccurate. However, when RQD is the only joint data available, eq. (A9) has been found to be the best simple transition from RQD via Jv to block volume.

A7 Methods to find an equivalent block volume where joints do not delimit blocks

According to Section A1, a minimum of three joint sets in different directions are theoretically necessary to delimit blocks in a rock mass. There are, however, cases with irregular jointing where blocks are formed mainly from random joints, and other cases where the blocks are delimited by one or two joint sets and additional random joints. In cases where the jointing is composed of one or two joint sets with no or few random joints, the joints do not define individual blocks. In such cases an *equivalent block volume* is used in the calculations. Such block volume may be found from one of the following methods:

1. Where only *one joint set* occurs, the equivalent block volume may be considered to be similar to the area of the joint plane [13] multiplied by distance between the two joints:

   $$Vb = L^2 \cdot S \qquad \text{eq. (A10)}$$

   Here L = the joint length, and
   S = the spacing between the joints. For long joints, for which the length is often difficult to measure, it is often sufficiently accurate to use a length L = 4 m.

   (Example: For foliation partings with lengths L = 0.5 m to 2 m and joint spacing S = 0.2 m, the equivalent block volume will vary between $Vb = S \cdot L^2 = 0.2 \cdot 0.5^2 = 0.05\ m^3$ and $Vb = 0.2 \cdot 2^2 = 0.8\ m^3$)

   A more accurate method for short joints is described in method no. 4

2. For *two joint sets* the spacing for the two sets (S1 and S2) and the length (L) of the joints can be applied: $Vb = S1 \cdot S2 \cdot L$ eq. (A11)

3. For most cases the equivalent block volume can be found from eq. (A4a) which requires input from the block shape factor (β). [14] β can be estimated from eq. (A6). A more accurate estimate of β where less than three sets occur, can be found from the following expression:

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[13] Here is assumed that the joint plane is circular, i.e. $A = \pi \cdot L^2/4 \approx L^2$

[14] As the volumetric joint count can be measured also where joints do not delimit defined blocks, this approach can be applied where few joints sets are found.

$\beta = 20 + 7\ (S_{max}/S_{min})(3/n_j) = 20 + 21(S_{max}/S_{min} \cdot n_j)$ eq. (A12)

where S_{max} and S_{min} are the largest and smallest joint spacing respectively
n_j is an ajustment factor for the number of joint sets with ratings given as:

1 joint set only	n_j = 1
1 joint set + random joints	1.5
2 joint sets	2
2 joint sets + random joints	2.5
3 joint sets	3
3 joint sets + random joints	3.5

4. For small discontinuities (fissures, partings and small joints) for which the lengths can be measured or easily estimated, the ratio length/spacing = L/S can be applied in eq. (A12):
$\beta = 20 + 21\ L/(S \cdot n_j)$ eq. (A13)

Example
For one joint set (n_j = 1) spaced at S1 = 0.2 m having an average joint length L1 = 2 m, the block shape factor according to eq. (A13) is $\beta = 20 + 21\ L1/(S1 \cdot n_j) = 230$.
The volumetric joint count for this set is $Jv = 1/S1 = 5$. This gives $Vb = \beta \cdot Jv^{-3} = 1.84\ m^3$
(For a defined block limited by 3 joints sets crossing at right angles with spacings equal to S1, L1, and L1, the volume is $Vb = 0.2 \cdot 2 \cdot 2 = 0.8\ m^3$)