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

SUMMARY

"In view of the scarcity of reliable information on the strength of rock masses and of the very high cost of obtaining such information, it is unlikely that a comprehensive quantitative analysis of rock mass strength will ever be possible. Since this is one of the key questions in rock engineering, it is clear that some attempt should be made, whatever information is available, to provide some form of general guidance on reasonable trends in rock mass strength."

Evert Hoek and Edwin T. Brown (1980)

A rock mass is an inhomogeneous material built up of smaller and larger blocks/pieces composed of rock material. A great variety exists, both in the composition of the rock material and in the structure and occurrence of its discontinuities. The rock mass is, in fact, a material exhibiting a wider range in structure, composition and mechanical properties than most other construction materials. Reliable tests of strength properties of such a complex material are impossible, or so difficult to carry out with today's technique, that rock engineering is based mainly on input data determined from observations and simplified measurements of the rock mass.

As the quality of the input data determines to a great extent the success of the design, there is a general demand for better methods in rock mass descriptions, and practical guidelines to carry out numerical characterizations.

Faults and weakness zones often exhibit special types of rock masses in the crust. They may consist of weathered or altered rocks found as decomposed rock and/or as clay filling, often together with crushed rock. Such zones may cause excavation problems, which are of no concern in the volumes between the zones. Therefore, weakness zones have been given special attention in this work.

The problem with uncertainties in rock engineering

The great spatial variability and large volumes involved in rock mass utilizations result in that only a limited number of measurements can be made in a characterization. *Before* construction, the subsurface, therefore, has to be described by a limited number of imprecisely known parameters. Considerable uncertainties may be introduced from the interpretation and extrapolation made to describe the geological setting. Also, the fact that horizontal weakness zones and other features, which do not outcrop, may be overlooked, is added to these errors.

Thus, although extensive field investigation and good quality descriptions will enable the engineering geologist to predict the behaviour of a tunnel more accurately, it cannot remove the risk of encountering unexpected features. A good quality characterization of the rock mass will, however, in all cases except for incorrect interpretations, improve the quality of the geological input data to be applied in evaluations, assessments or calculations and hence lead to better designs.

After the rock mass has been "*opened*" during excavation, the actual rock masses may be studied. In these cases the quality of the input data used in evaluations, calculations and modelling mainly depends on the way they are measured and characterized.

The main purpose of this work has been to improve the quality in rock engineering by providing better input data for characterizing the rock mass based on selected, well defined geological parameters. Methods for field descriptions of outcrops, as well as logging of drill cores and geophysical measurements, have been included.

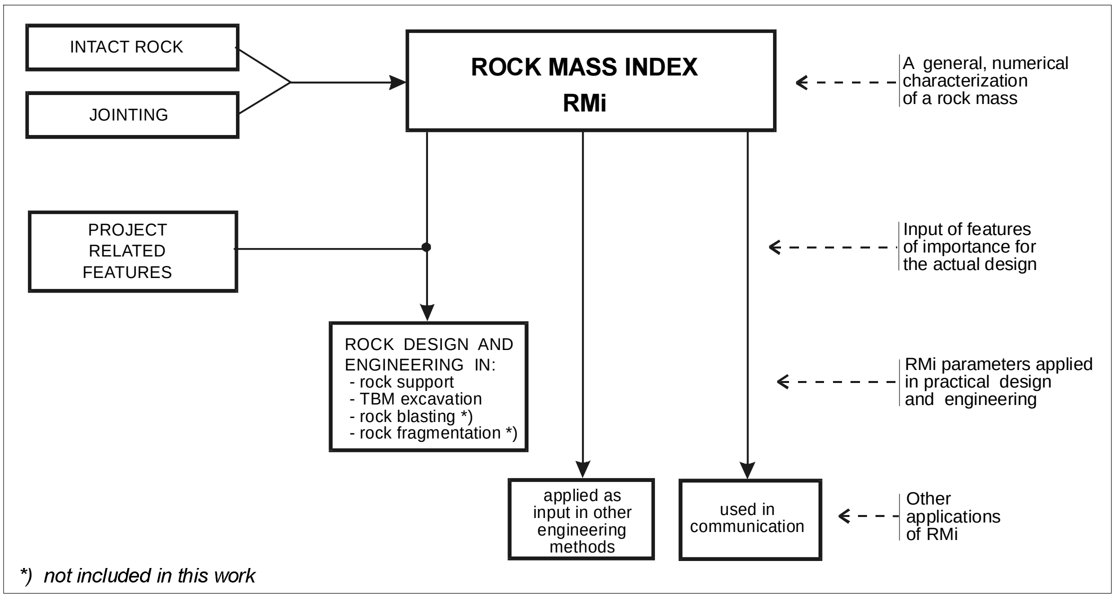

Fig. 1 General layout of the RMi system for rock engineering and design

A methodology for strength characterization of rock masses

As a rock mass basically is composed of intact rock penetrated by joints, its behaviour depends not only on the properties of the intact material and the discontinuities separately, but also on the way they are combined. Thus, there are several complications which arise when the parameters of a rock mass shall be described:

1. The intact rock is inhomogeneous and variable.
2. The discontinuities are diverse in nature (i.e. their variations involve more than ten characteristics).
3. The geometrical arrangement of discontinuities is infinitely variable.

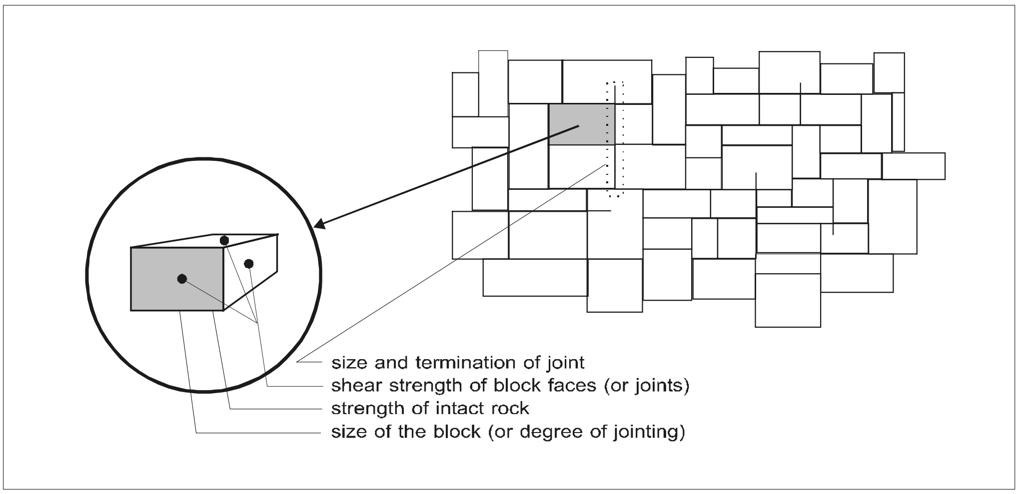

Fig. 2 The main parameters in a rock mass.

Because of these structural variations and the often large volumes involved, the properties of a rock mass have to be determined from observations backed by laboratory tests of small specimens. Thus, the evaluations and assessments applied in engineering are largely empirical, typically applied in the classification schemes.

The rock mass index (RMi) developed in this work is a general characterization of rock masses in which their main parameters are included. In principle it is based on the reduction in the strength of the intact rock due to the presence of joints. This is expressed as

$$RMi = \sigma_c \times JP$$

where σ_c is the uniaxial compressive strength of intact rock measured on 50 mm samples;
JP is the jointing parameter, i.e. the reduction factor from jointing. It consists of :
- the degree of jointing (given as block size); and
- the joint characteristics, representing the joint wall roughness and alteration, as well as the size of the joint.

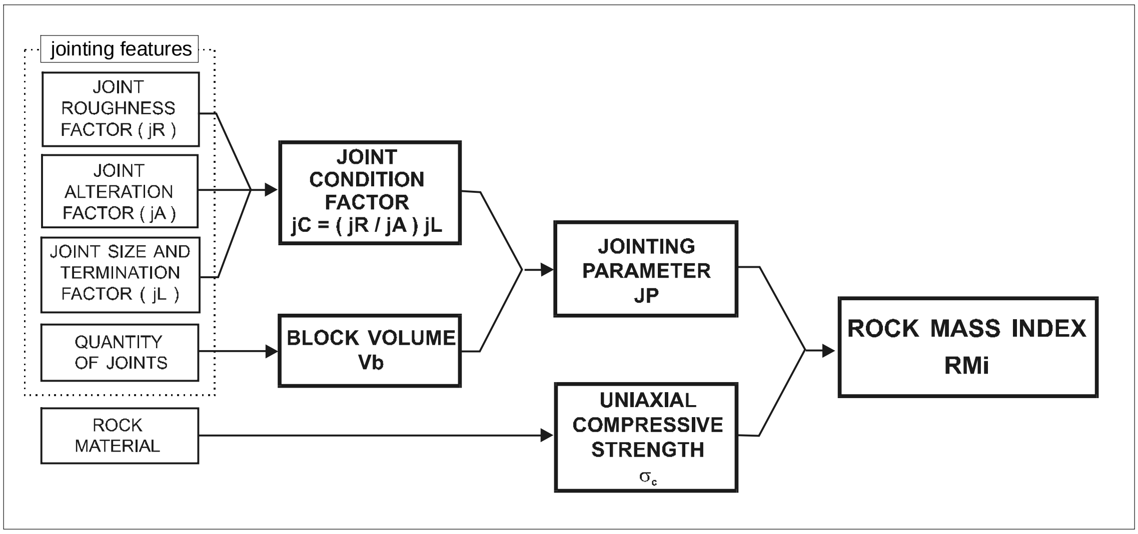

Fig. 3 Combination of the selected parameters in the Rock Mass index.

It is practically impossible to carry out triaxial or shear tests on rock masses at a scale similar to that of underground excavations. As the rock mass index, RMi, is meant to express the compressive strength of a rock mass, a calibration has been carried out. Data from 8 large-scale tests and 1 back analysis of a prototype situation have been applied. The data contained test results of the uniaxial compressive strength and the inherent parameters of the rock mass. The values for Vb and jC have been plotted in Fig. 4, and the lines representing constant jC have been drawn. In this way Vb and jC have been combined to express the jointing parameter, JP. From the lines the jointing parameter can be expressed as

$$JP = 0.2 \times \sqrt{jC} \times Vb^{D}$$

where jC = the joint condition factor, Vb = the block volume, and $D = 0.37\, jC^{-0.2}$

Significant *scale effects* are generally involved when a 'sample' is enlarged from laboratory size to field size. From the calibration described above, RMi is tied to large samples where the scale effect has been included in JP. For massive rock masses, however, where the jointing parameter JP ≈ 1, the scale effect for the uniaxial compressive strength must be accounted for, as it is related to 50 mm sample size.

This scale effect is expressed as

$$\sigma_{cf} = \sigma_{c50}\,(0.05/Db)^{0.2}$$

where σ_{c50} is the uniaxial compressive strength for 50 mm sample size, and
Db is the equivalent block diameter.

The approximate block diameter may be found from $Db = \sqrt[3]{Vb}$, or, where a pronounced joint set occurs, simply by applying the spacing (S1) of this set (Db = S1). The scale effect of compressive strength has been included in Fig. 4 to assess the jointing parameter.

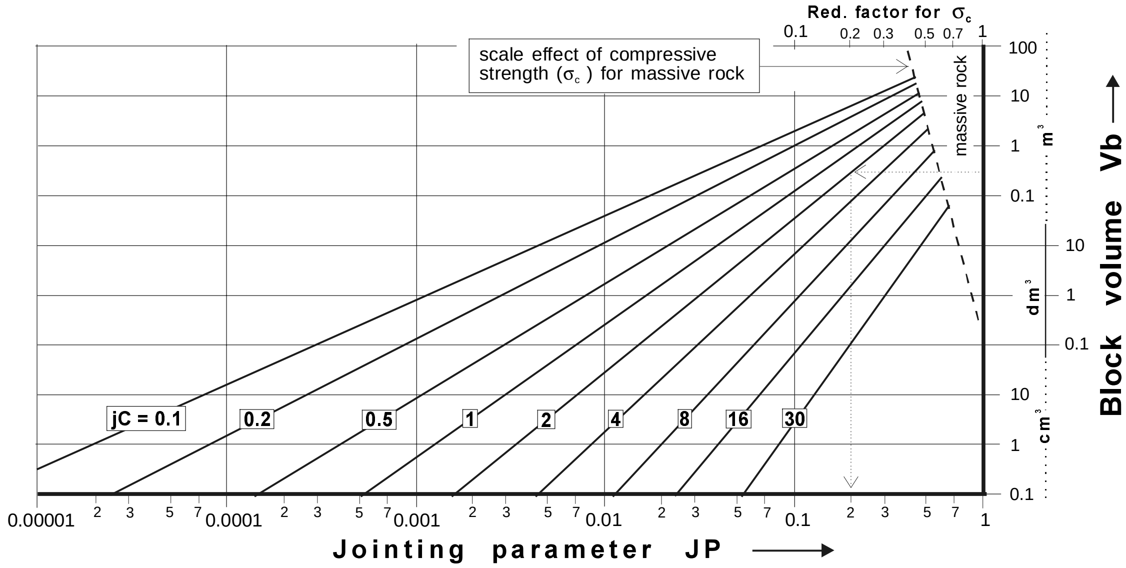

Fig. 4 Diagram for calculating the jointing parameter from block volume (Vb) and joint condition factor (jC). Example shown: from input values Vb = 0.3 m^3 and jC = 2 the value of JP = 0.2. In massive rock the approximate scale effect is shown by the dotted line and its reduction factor.

RMi may be considered as a quality index of the rock mass as a construction material. This is similar to the classification used for concrete (which is based on its uniaxial compressive strength). Both hard rocks and soft rocks can be included in the RMi characterization of rock masses for rock engineering.

The parameters used in the RMi

The *degree of jointing* (i.e. the quantity of joints) is regarded as a main feature in the strength and behaviour of a rock mass. By characterizing this parameter by the block volume (Vb) it is possible to describe most variations in jointing, and at the same time include its three-dimensional character.

Direct description of the block volume in the field is especially useful where small blocks or irregular jointing occur; further it may improve core logging of crushed rock where the size of the particles is small enough to be observed. Mathematical correlations between several methods for characterizing the quantity of joints (see Fig. 5) have been worked out.

Ideally, all the *characteristics* included in the joint condition factor, i.e.:

$$jC = \frac{\text{joint roughness factor (jR)}}{\text{joint alteration factor (jA)}} \times (\text{joint size factor (jL)})$$

should be measured accurately; jR and jA from shear strength or friction tests, and jL from the measured length and termination of the joint. Such measurements of the joints would generally be either extremely time-consuming, or in most cases, practically impossible to carry out. Generally only a limited amount of the joints can be characterized in a rock mass, and simplified methods have to be applied. Therefore, the parameters in the joint condition factor have been given ratings, which can be determined from defined descriptions. The proposed RMi system has some features similar to those of the Q-system (Barton et al., 1974). Thus, jR and jA are almost the same as Jr and Ja in the Q-system, The joint size and continuity factor (jL) is introduced in RMi as it often has a significant impact on rock mass behaviour.

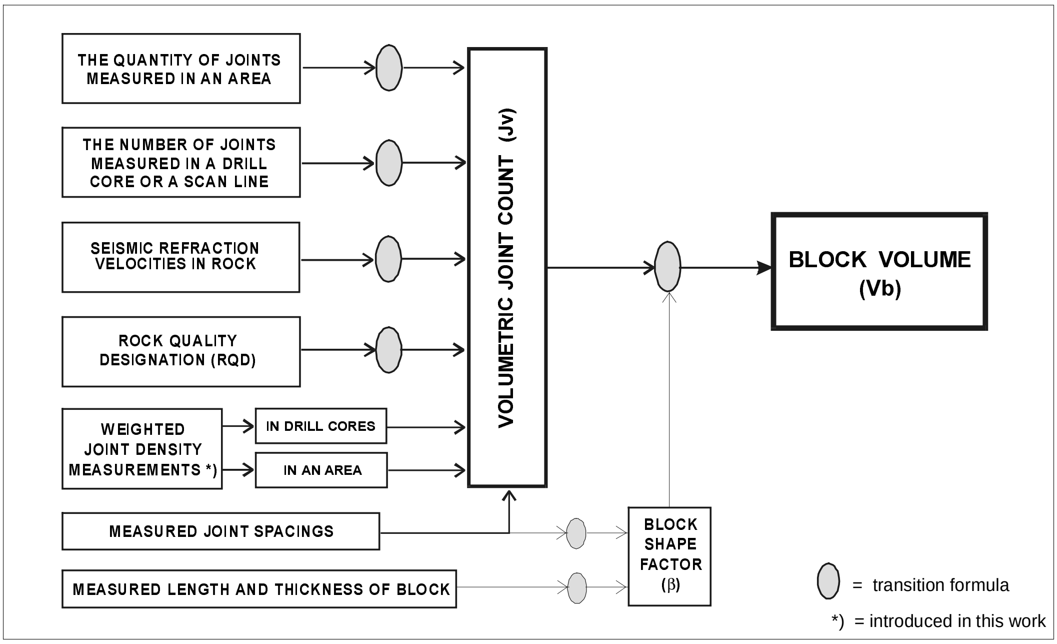

Fig. 5 Developed correlations between various methods to measure the degree of jointing and the block size.

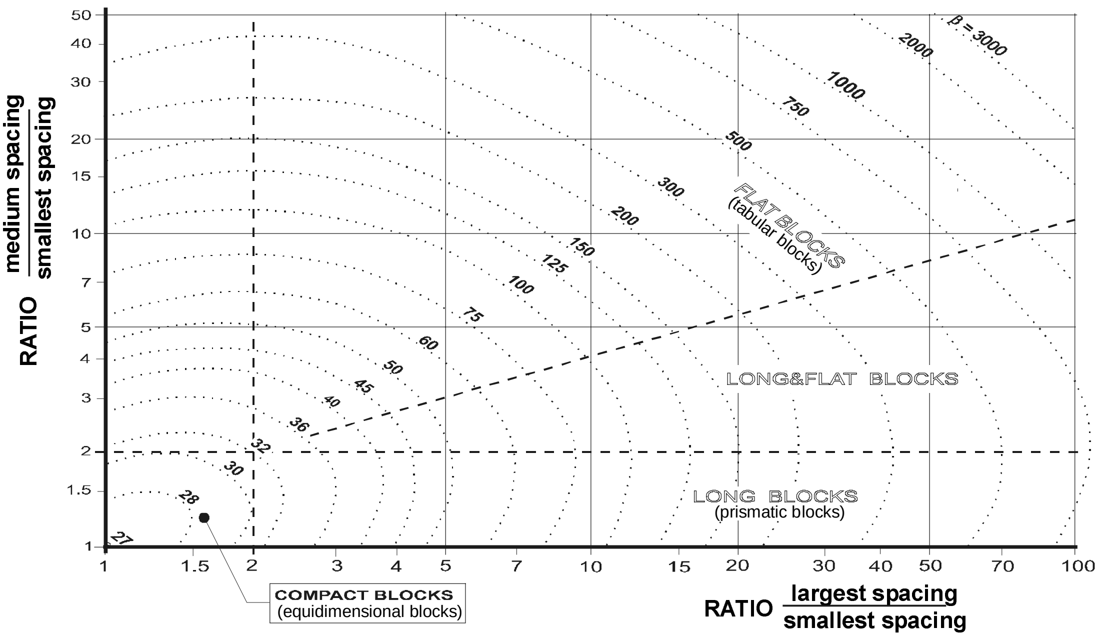

Fig. 6 The block shape factor determined from the joint spacings provided by 3 joint sets intersecting at right angles. Example: For joint spacings 0.2 m, 0.8 m, and 3 m the spacing ratios are 4 and 15, which gives β = 135 corresponding to a 'long&flat' block type.

The RMi can also be found by simplified measurements resulting, of course, in less accurate estimates. These simplifications may be:

- The compressive strength can be determined from simple field tests or from rock descriptions as is described in Appendix 3.
- If the condition of the joints has not been measured, it may be assumed that jC = 1.75 ("common" joint condition) for which the jointing parameter can be expressed as JP = 0.26 $Vb^{1/3}$, or even more simply where the 3 joint sets have similar spacings (Sj): JP = 0.26 Sj.

The strong influence that the number of joint sets has on the behaviour of rock masses is expressed in the block shape factor (β), which has been introduced to characterize the main types of blocks. It is applied in the correlation between the volumetric joint count (Jv) and block volume as

$$Vb = \beta \times Jv^{-3}$$

β may be found from Fig. 6 or more roughly from

$$\beta = 20 - 7\ S_{max}/S_{min}$$

where S_{max} and S_{min} are the longest and shortest dimensions of the block.

The methods described outline how the various parameters applied in RMi can be determined either from commonly used measurements and/or from measurements developed in this work. In variable rock masses, for instance where the block size varies and/or the joint condition factor is different for the various joint sets, some guidelines have been given on how to combine these variables to find a representative joint condition factor (JP).

The use of the RMi in rock engineering

As a relative strength index for rock masses, the RMi expresses numerically the quality of the rock mass. RMi may be applied in various methods used in practical rock engineering. In addition, some of the parameters in RMi can be used individually in classification systems on stability, where they may improve the input and/or because they may be easier and or more accurately characterized.

TABLE 1 MAIN TYPES OF WORKS CONNECTED TO ROCKS AND ROCK MASSES WITH INDICATION WHERE THE RMi CAN BE APPLIED.

TYPE	ACTUAL PROCESS OR USE
Treatment of rocks	- drilling (small holes) - boring (TBM boring, shaft reaming) *) - blasting*) - fragmentation*) - crushing - grinding - cutting*)
Application of rocks	- rock aggregate for concrete etc. - rock fill - building stone
Utilization of rock masses	- in underground excavations (tunnels, caverns, shafts) *) - in surface cuts/slopes/portals *)
Construction works in rock masses	- excavation works - rock support *) - water sealing

*) Areas where the system is of particular interest.

When applied in rock engineering, as well as in construction and utilization purposes, the RMi value or its parameters are adjusted for local features of importance to the engineering purpose as indicated in Fig. 1. Table 1 shows also other areas for applying RMi.

As the RMi expresses the inherent properties of rock masses, it is possible to compare RMi values from various locations directly. In this way RMi may contribute to improved communication between people involved in rock construction. Fig. 7 shows the main applications of RMi and/or its parameters in rock mechanics and rock engineering.

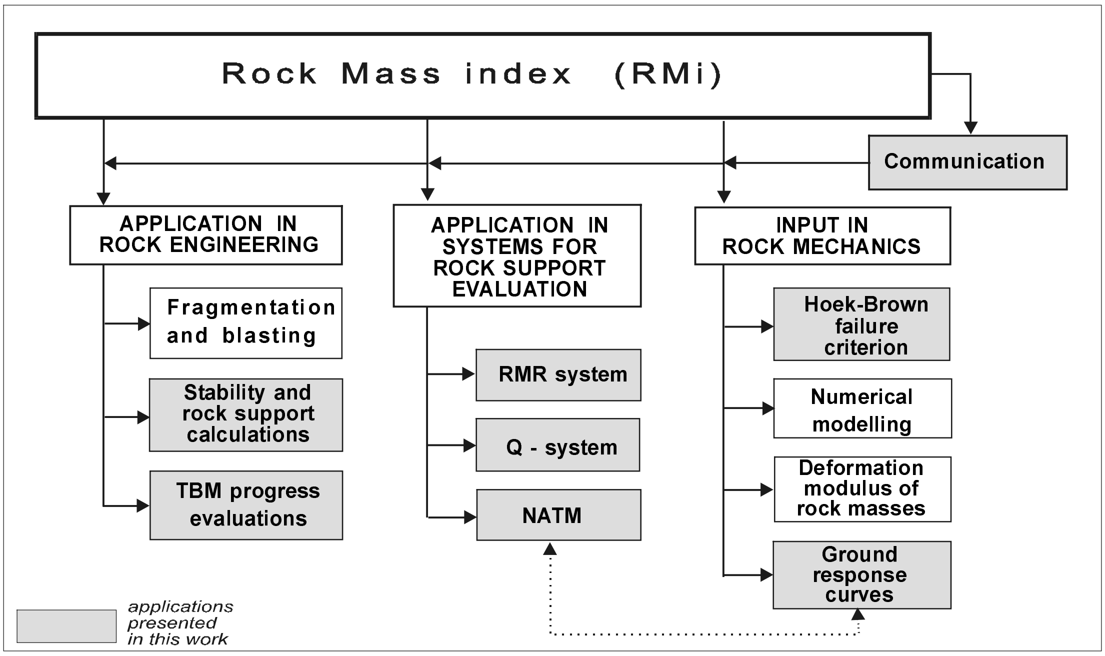

Fig. 7 Various applications of RMi and its parameters.

RMi applied as input to the Hoek - Brown failure criterion

As RMi expresses the relative compressive strength of rock masses (σ_{cm}), it represents a special case in the Hoek-Brown failure criterion of rock masses

$$\sigma_{cm} = \sigma_c \times s^{1/2}$$

where s is a constant representing the rock mass properties.

The determination of JP = $s^{1/2}$ from block size and joint characteristics introduces an easier and more direct and accurate method to find the value of s than the method presented by Hoek and Brown. Also the m factor in Hoek-Brown failure criterion for rock masses can easily be determined from JP and the type of rock. This improvement will indirectly be of benefit in rock mechanics as these factors (s and m) can be applied to assess the shear strength for continuous rock masses as well as input to ground response curves.

RMi applied in classification systems

RMi offers interesting possibilities to quantify the behaviouristic descriptions applied in the NATM. From the shear strength and - when RMi has been further developed - the deformation modulus of rock masses, it is possible to determine the 'Kennlinie-Bemessungsverfahren'. This is a type of Fenner-Pacher curves (ground response curves) from which displacements as well as rock support can be estimated in the NATM.

As the RMR system of Bieniawski (1973) is based on the sum of several parameters, while RMi and partly also the parameters involved in it are exponentially characterized, it is difficult to directly apply RMi in RMR. The NGI Q system has, however, a similar structure and partly the same parameters as RMi. It is the classification system, which is most similar to RMi, and in which the parameters applied in RMi can best be utilized. If block volume (Vb) is applied instead of RQD/Jn, the author feels that the Q system would be significantly improved. Also the use of the joint condition factor, jC, instead of Jr/Ja may improve Q because jC includes more of the joint characteristics.

RMi applied in assessment of rock support

Whereas the stability of a tunnel opening in a *continuous* material can be related to the intrinsic strength and deformation properties of the bulk material, stability in a *discontinuous* material depends primarily on the character of the joints and the block size. To assess stability in underground openings the ground has, therefore, been divided into continuous and discontinuous ground, determined by the continuity factor, CF = tunnel diameter/block diameter

Discontinuous ground occurs where CF = approx. 5 - 100, else the ground is continuous. In this manner, particle, fragment, block size, or joint spacing becomes indicative of tunnel behaviour.

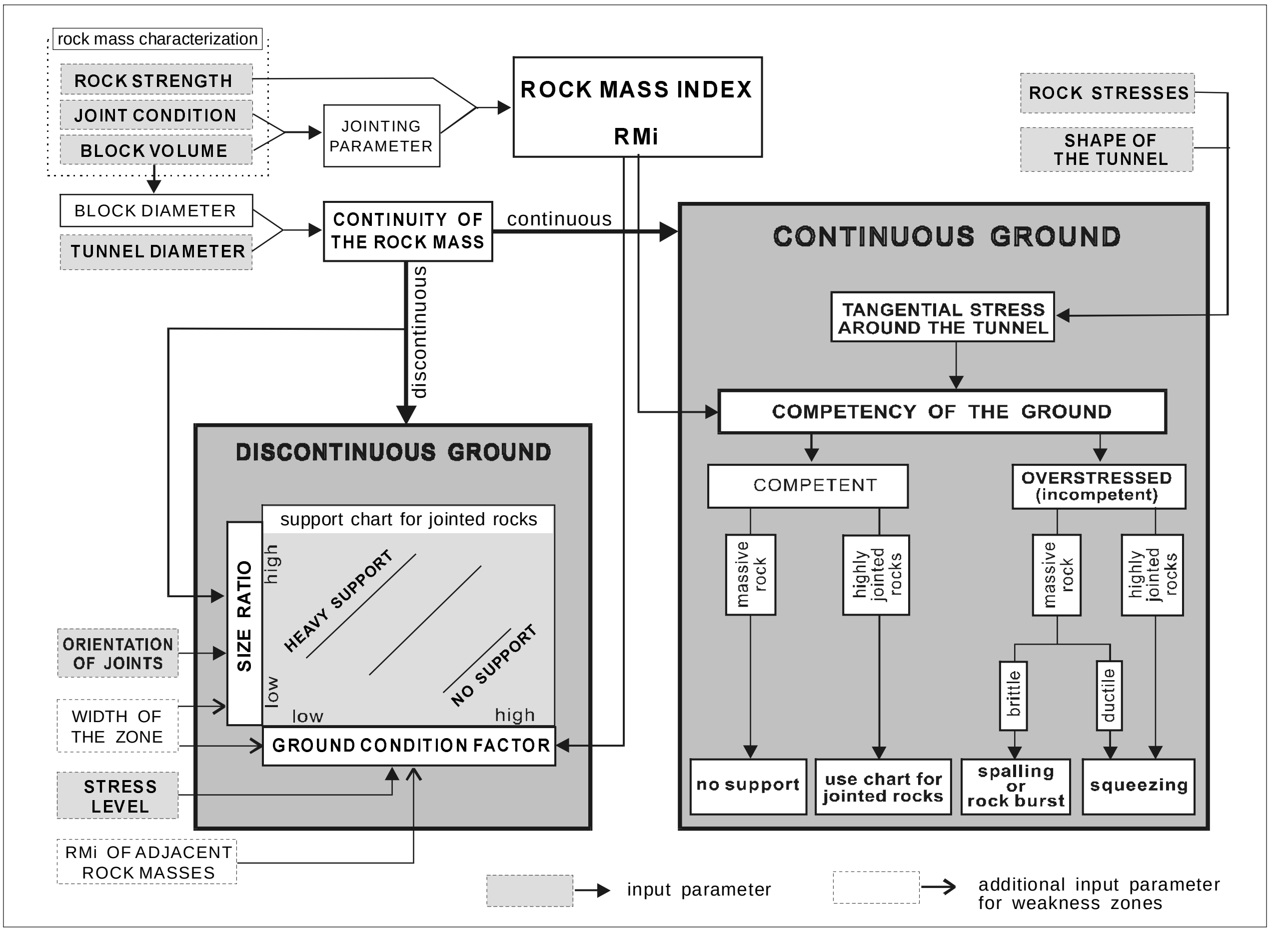

Fig. 8 The application of the RMi to assess rock support in continuous and discontinuous ground. Weakness zones are treated in the same way as discontinuous ground by adjustment to the 'size ratio' and the 'ground condition factor'.

The competency of continuous ground

The competency of continuous ground expresses whether the ground surrounding an underground opening is overstressed or not. It is expressed as the ratio

$$Cg = RMi/\sigma_\theta$$

where σ_θ is the tangential stresses set up in rock masses surrounding the opening. It depends on the overall stress level, the stress anisotropy and the shape of the opening. A method to estimate the rock stresses and σ_θ has been outlined.

In *competent* ground the compressive strength of the rock mass is higher than the rock stresses. Instability in this type of ground is mainly caused by joints or other structural weaknesses. *Incompetent* ground appears where the rock stresses are higher than the compressive strength of the rock mass. Squeezing develops in deformable rock masses, while rock burst, spalling etc. occurs in massive, brittle rocks. Here, the application of RMi to assess the competency factor opens for improved characterization and rock support of these important groups of ground.

Discontinuous (jointed) rock masses

Instability of this type of ground is mainly related to loosening and downfalls of one or more blocks in the tunnel periphery mainly determined by the properties of the joints. As these are governed by the joint condition, the block size, and the strength of the wall rock, the rock mass index, RMi, is suitable as a main input to the ground condition factor

$$Gc = SL \times RMi \times C$$

where SL is the stress level factor with ratings between 0.2 and 1.5.
C is the gravity adjustment factor to compensate for the greater stability of a vertical wall compared to a horizontal roof. It varies between 5 and 1 respectively.

The equivalent block diameter (Db) related to the size of the opening has been selected as the second parameter in the assessment of rock support in discontinuous ground. In the tunnel periphery Db depends on the orientation of the joints relative to the tunnel; thus, the 'size ratio' is expressed as

$$Sr = (Dt/Db)\ Co/Nj$$

where Dt is the diameter of the tunnel, i.e, its span or width (Wt) or height (Ht),
Db is the equivalent block diameter, the shortest side of the block, which often is the spacing of the main joint set.
Co is a factor for the orientation of the main joint set varying between 1 and 3.
Nj is a factor for the number of joint sets

Simplified expression have been derived for 'common' hard rock mass conditions, from which the factors Gc and Sr can easily be determined.

Faults and weakness zones

Compared to the 'normal' rock masses, weakness zones and faults form a special type of ground as their behaviour and requirement for support often are quite different. The ground condition factor for weakness zones is similar in structure to Gc:

$$Gc_z = SL \times RMi_m \times C$$

where RMi_m is the modified rock mass index which includes the thickness of the zone and influence from the adjacent rock masses.

Similarly, a size ratio for zones has been selected for weakness zones with width smaller than the tunnel diameter. It is expressed as

$$Sr_z = \frac{\text{thickness of zone}}{\text{block diameter (or joint spacing)}} \times \text{(orientation of zone)}$$

RMi applied in prediction of the boring rate of TBMs

Full face tunnel boring is highly influenced by the strength of the rock masses. This favours the use of RMi in assessments of TBM penetration rates. Thus, the ground condition for TBMs is characterized by the jointing parameter and the compressive strength of intact rock.

The method using RMi parameters for TBM penetration assessment has been based on the NTH prediction model for TBM excavation.[1] The drilling rate index (DRI) which represents the properties of intact rock, has been replaced by an expression including the uniaxial compressive strength (σ_c). The following expression has been developed for *massive* rock masses:

$$k_{eq} = 0.022\ E^{0.72} \times \sigma_c^{-0.43}$$

where E is a factor representing the deformation properties of the rock - varies between 500 and 1000.

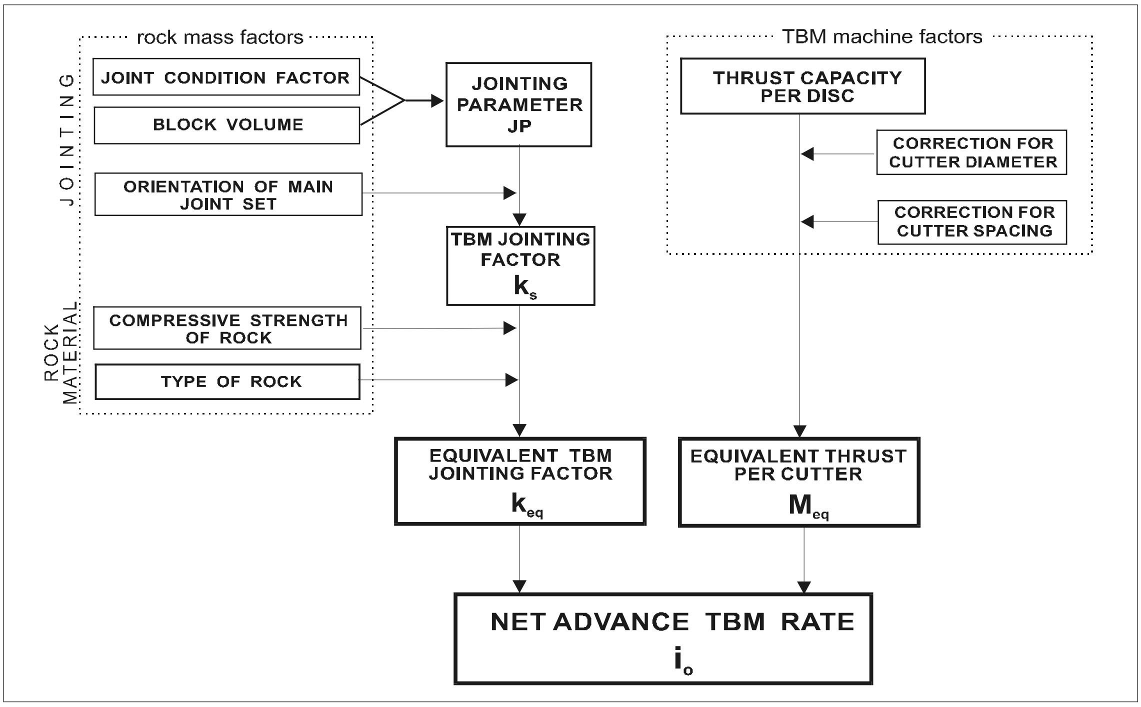

Fig. 9 Application of the parameters in RMi in the prediction of TBM boring progress.

In *jointed* rock masses the orientation of the main joint set relative to the tunnel axis and the rock characteristics (E and σ_c) have been applied in a TBM jointing factor expressed as:

[1] Refer to the method published by the Norwegian Institute of Technology (1994): Fullface boring of tunnels (in Norwegian). In PR 1-94, Trondheim Norway, 159 pp. An earlier version of the method has been published in English by Movinkel T. and Johannessen O. (1986): Geologic parameters for hard rock tunnel boring. In Tunnels & Tunnelling, April 1986, pp. 45-48.

$$k_{eq} = \frac{0.06 c_o \sqrt{E}}{JP \times \sigma_c^{0.3}}$$

where c_o is a correction factor for the angle between the tunnel and the main joint set. Its ratings vary between 1 and 1.75.

The TBM net advance rate (I) is found from the factor k_{eq} and the equivalent thrust per cutter (M_{eq}) using diagrams developed by NTH. It can also be determined from the following mathematical expressions, which have been deduced from the mentioned diagrams:

$$I = i_o \times RPM \times (60/1000) \qquad \text{(in m/h)}$$

where $i_o = F \times k_{eq}^{G}$ with $F = 0.0015\, M_{eq}^{1.5}$ and $G = 25\, k_{eq}^{-0.4} \times M_{eq}^{-0.7}$ (for $k_{eq} \geq 3.5$).

Closing remark

Caused by the variability and complex structure of rock masses, one has to accept that the data used in calculations, evaluations and assessments have limited accuracy. Part of this stems from the way descriptions of the rock masses are made and combined. The work described herein may lead to improved quality of this important feature in rock engineering.