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

Chapter 4

THE COMBINATION OF GEO-DATA INTO A ROCK MASS INDEX

> *"Try always to combine theory and practice and to confront ideas and experience."*
> Leopold Müller (1982)

Construction materials such as concrete, most metals, wood etc. used in civil and mining construction are characterized or classified according to their strength properties. This basic quality information of the material is used in the engineering and design for various construction purposes. In rock engineering, no such specific strength characterization of the rock mass is applied. Most rock engineering is carried out using various descriptions, classifications and unquantified experience. Although the various utilizations of rocks and rock masses have different purposes and are subjected to various problems, the suitability and quality of the rock mass depends largely on its strength properties.

Hoek and Brown (1980), Bieniawski (1984), Nieto (1983) and several other authors have indicated the need for a *strength characterization* of rock masses. Williamson and Kuhn (1988) are of the opinion that *"no classification system can be devised that deals with all the characteristics of all possible rock materials or rock masses. What we are aiming for, therefore, is a system that would generally group the rocks in such a way that those parameters which are of most universe concern are clearly dealt with and the number of symbols are kept to a minimum."*

The Rock Mass index, RMi, has been developed as a general strength characterization of the structural material that a rock mass represents. It therefore includes only the inherent features or parameters in the rock mass. This prerequisite is in accordance with the ideas by Patching and Coates (1968) who presented a general characterization based on the <u>intrinsic parameters</u> of a rock *"which are the same irrespective of place or circumstances. For this reason it was considered necessary to omit factors related to environment from the classification, although stress applications, pore-water and other influences have a pronounced effect on the behaviour of a rock in any given situation. Just as a structural engineer who is designing a steel structure will establish the stress distributions of the structure separately from the specifications of the steel, so in any specific problem in rock mechanics the environmental factors will be considered and established for that problem in addition to the determination of the nature or classification of the rock."* The classification of Patching and Coates (1968) was, however, descriptive and therefore not useful in calculations.

Based on the author's own experience and published papers in this context the following considerations have been important during the development of RMi and its input data:

- Few input data should be included to arrive at a simple expression.
- Existing methods should be applied for geo-data acquisition where possible.
- Simple and practical methods for finding the input values should be preferred.
- Guidelines should be developed for adequate descriptions so that they can be "translated" to numerical values.

- Correlations should be developed so that input data from various types of measurements can be used.

4.1 THE STRUCTURE OF THE ROCK MASS INDEX

From the outline in Chapters 2 and 3 and in Appendix 1 it is clear that a rock mass is a material much more complex in composition, structure, variability than most other structural materials. The presence of various defects (discontinuities) in a rock mass, which tend to reduce the inherent strength of the rock, constitutes the main feature in its behaviour. This fact is the main principle of the Rock Mass index (RMi), as explained in this chapter.

4.1.1 The input parameters selected

Although external forces acting on a rock mass, such as induced rock stresses or water pressures are justifiably included in some classification systems for designing tunnel support (see Table 2-3), the strength properties of the rock mass is not a direct function of these features. Deere et al. (1969) proposed to use parameters related to the character of discontinuities as the *main* feature for both a general and a diagnostic classification system. This use of jointing as the major input does not, however, exclude the importance of the rock material on the behaviour of the rock masses. For example, if joints are widely spaced or if the rock material is weak, the properties of the intact rock may strongly influence the gross behaviour of the rock mass. In addition, the rock material is also important if the joints are not continuous. This view of rock mass behaviour and strength has been further outlined by Wood (1991) as:

- Better quality rock masses are determined by the geometry of the rock mass structure, specially block size and block shape.
- Fair to poor quality rock masses are determined by the inter-block shear strength and deformational characteristics.
- Very poor quality rock masses mainly depends on the low strength of the intact material.

For jointed rock masses, Hoek et al. (1992) are of the opinion that the strength characteristics are controlled by the block shape and size as well as their surface characteristics determined by the intersecting discontinuities. They recommend that these parameters are selected to represent the average condition of the rock mass. Similar ideas have been set forth by Tsoutrelis et al. (1990), Matula and Holzer (1978), Coates and Patching (1968) and Milne et al. (1992). These considerations have been used in the selection of the following input parameters in a general strength characterization of a rock mass:

- the size of the blocks delineated by joints, - measured as block volume;
- the strength of the block material, - measured as uniaxial compressive strength;
- the shear strength of the block faces, - measured as friction angle, and
- the size and termination of the joints, - measured as length and continuity.

This is shown schematically in Fig. 4-1. It is considered, however, that taken together, they provide a fairly complete indication of the strength of a given rock mass. Spesific features such as faults, dykes and shear zones, should be considered separately (Bieniawski, 1984, 1989).

An additional, also important rock mass parameter, *the block shape*, is not directly included in RMi. The main reason for this is the objective to maintain a simple structure of RMi. Block shape, being a geometric delineation of the three-dimensional pattern of jointing, is, however, indirectly included in the block volume, as the block volume varies with its shape. This is further described in Appendices 3 and 4.

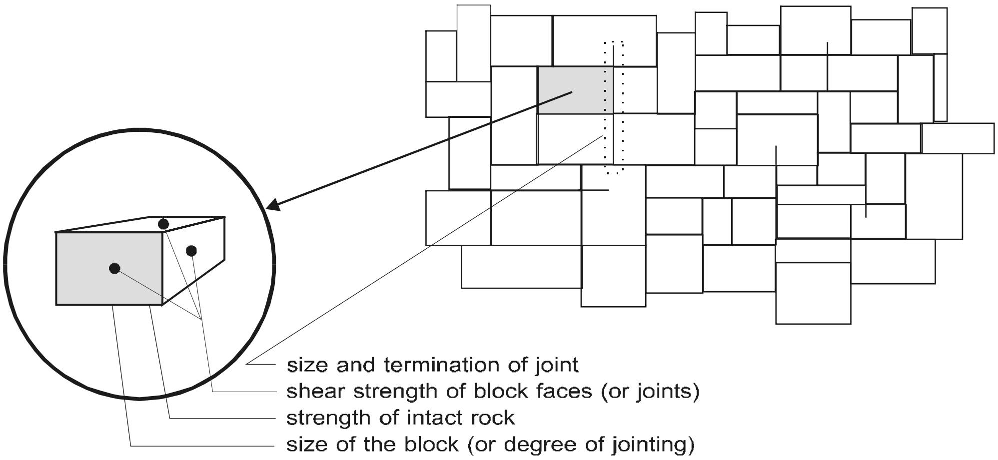

Fig. 4-1 An aggregate of blocks delineated by joints indicating the parameters selected for a general rock mass characterization

Numerical values alone are seldom sufficient for characterizing the properties of such a complex material as a rock mass. Therefore, numerical values should be accompanied by supplementary descriptions as presented in Appendix 3 where the requirements for extracting numerical values from descriptions are outlined.

4.1.2 The Rock Mass index (RMi)

The main principle in the development of RMi has been focussing on the effects of the defects in a rock mass in reducing the strength of the intact rock. The RMi is thus defined as

$$RMi = \sigma_c \times JP \qquad \text{eq. (4-1)}$$

Here σ_c = the uniaxial compressive strength of the intact rock material, and
JP = the jointing parameter, see Fig. 4-2. It is a reduction coefficient representing the block size and the condition of its faces represented by their friction properties and the size of the joints, see Fig. 4-1. The value of JP varies from almost 0 for crushed rocks to 1 for intact rock. Its value is found by combining the block size, and the joint conditions as described in the next section in this chapter.

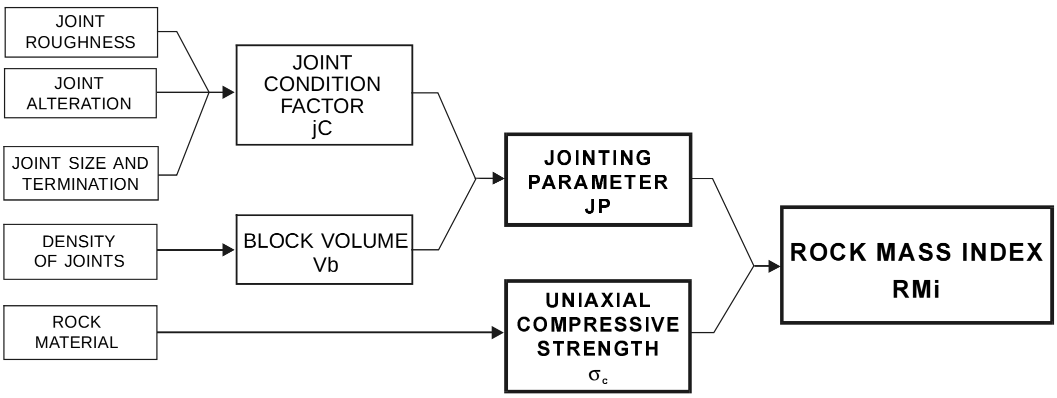

Fig. 4-2 The principle of the RMi characterizing the material properties of a rock mass.

As may be noticed, RMi is not dimensionless, but has the units of σ_c . The individual input components of RMi, the rock strength, and the jointing features are measured and combined to deduce this rock mass strength. Results from large scale and field measurements of rock mass strengths have been used to develop an expression for RMi as close as possible to reality. This is further described below in Section 4.2.

4.1.3 The combination of the input parameters

The importance of the two main contributors to RMi, the compressive strength of intact rock (σ_c) and the jointing parameter (JP) is further described in this section.

The parameter for the rock material (σ_c), the uniaxial compressive strength of intact rock, is used directly. Its value can be determined from laboratory tests. Estimates of (σ_c) can also be obtained as described in Appendix 3.

The jointing parameter (JP) is a combination of the block size, measured as its volume (Vb), and the joint condition factor (jC), see Fig. 4-2:

The block volume, Vb, is a measure of the degree of jointing or the density (amount) of joints. As it is a 3-dimensional measure, it indirectly also is an expression of the overall geometry of the rock mass. It can be determined from field measurements of the block dimensions as further described in Appendix 3.

The joint condition factor, jC, represents the inter-block frictional properties. Barton et al. (1974) have in their Q-system chosen the roughness and alteration factors (Jr and Ja) to represent the importance of dilatancy and shear strength of joints. The ratio of the two parameters (Jr/Ja) represents, with the ratings they are given in the Q-system, a fair approximation to the actual *shear strength properties* of the joint within normal variations of these factors (Barton et al, 1974; Barton and Bandis, 1990). It appears, therefore, logical to make use of the same values and combination of these parameters for the joint condition factor in the RMi. [1]

A joint size factor (jL) has been chosen as a size correction factor for joints. The reason for this is the fact that larger joints have a markedly stronger impact on the behaviour of a rock mass than smaller. In addition, the continuity or termination of the joint has been included. This part of the factor is divided between joints that terminate in massive rock, i.e. discontinuous joints, and other joints. The effect of a discontinuous joint is much less as the failure plane must partly pass through intact rock.

The factors included in the joint condition factor are combined in the following way:

$$jC = jL \times jR/jA \qquad \text{eq. (4-2)}$$

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

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

jL = the joint size and continuity factor.

The numeric values of these components of jC can be found from various field observations and measurements as described in Appendix 3.

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

4.2 CALIBRATION OF RMi FROM KNOWN ROCK MASS STRENGTH DATA

> *"The purpose of science is to simplify, not to complicate. The function of an engineering geologist, geotechnical or rock engineer is to examine and observe the complex variables of an area or project site and from this effort arrive at a set of simple, significant generalizations".*
> Douglas A. Williamson and C. Rodney Kuhn (1988)

It is practically impossible to carry out triaxial or shear tests on rock masses at a scale similar to that of surface or underground excavations (Hoek and Brown, 1988). The numerous attempts made to overcome this problem by modelling generally suffer from the limitations and simplifications, which have to be made in order to permit construction of the models. Consequently, the possibility of predicting the strength of jointed rock masses on the basis of direct in situ tests or of model studies is very limited.

This problem resulted in that Hoek and Brown (1980), during development of the Hoek-Brown failure criterion for rock masses, had very few strength data available (see Section 8.1). Their criteria for jointed rock masses are, therefore, based almost wholly on the laboratory tests carried out on Panguna andesite described by Jaeger (1969). In addition to these data on the Panguna andesite, for working out the RMi it has been possible to make use of some few more results of triaxial laboratory tests on large scale samples of rock masses, including:

- clay schist, sandstone, and siltstone from various locations in Germany, and
- granite from the Stripa test mine, Sweden.

Also results from in situ tests of quartzitic sandstone in the Laisvall mine in Sweden, and a back analysis from a large slide in quartzite and schist in the Långsele mine in Sweden have been used in the calibration.

As the rock mass index is meant to express the compressive strength of a rock mass (σ_{cm}) it can be expressed as RMi = $\sigma_{cm} = \sigma_c \times$ JP. The uniaxial compressive strength of intact rock (σ_c) is defined and can be determined within a reasonable accuracy. The jointing parameter (JP), however, is a combined parameter made up of the following features:

- the block volume (Vb) which can be found from measurements, and
- the joint condition factor (jC) which is the result of three independent joint parameters (roughness, alteration and size).

The results from the tests and back analysis have been used to determine how Vb and jC can be combined to express JP (and RMi accordingly when σ_c is known). This calibration has been performed in the following way:

1. From the known results of the tests or back analysis, namely of
   - the uniaxial compressive strength of the rock mass (σ_{cm}) and
   - the uniaxial compressive strength of the intact rock (σ_c)

   the value of the jointing parameter is by definition

   JP = RMi/σ_c = σ_{cm}/σ_c eq. (4-3)
2. Numerical characterizations have been made of the joints and the block characteristics in the actual rock mass 'sample' tested to find
   - the block volume (Vb), and
   - the joint condition factor (jC) found from eq. (4-2).
3. The data from the tests described in Appendix 6 and shown in Table 4 -1 have been plotted on the diagram in Fig. 4-3. Log. scales have been used both for the jointing parameter (JP) along the x-axis and for the block volume (Vb) along the y-axis. As joint spacing (i.e. block size) generally has an exponential distribution (see Appendix 1, Section 5), the lines representing jC are expected to be straight.

4. From the values of block volume (Vb) and jointing parameter (JP) the position of the corresponding joint condition factor (jC) is found for each of the data sets. As a best fit to these data the lines representing jC have been drawn, as shown in Fig. 4-3.

TABLE 4-1 THE RESULTS FROM LARGE SCALE TESTS ON ROCK MASSES FURTHER DESCRIBED IN APPENDIX 6.

Sample no	**Location**	**Rock type**	**σ_c MPa**	**jC**	**Vb**	**JP**
1	Panguna	andesite	265	4 - 6	2 - 6 cm^3	0.014
2	Stripa	granitic rock	200	1.5 - 2.5	5 - 15 dm^3	0.04
3	Laisvall mine	sandstone	210	0.75 -1	0.1 - 0.3 m^3	0.095
4	Långsele mine	grey schist, greenstone	110 - 1600	2 - 0.3	8 - 20 dm^3	0.01
5 a	Thüringer wald	clay-schist	55	1.5 - 2	5 - 10 dm^3	0.055 *)
5 b	"	" "	100	2 - 2.5	5 - 10 dm^3	0.08 **)
6	Hessen	sandstone/claystone	10.5/4.8	5 - 10 (?)	1 - 5 dm^3	0.17
7	Hagen	siltstone	65	3.5 - 4.5	5 - 10 dm^3	0.10

*) Tests parallel to schistosity **) Tests normal to schistosity

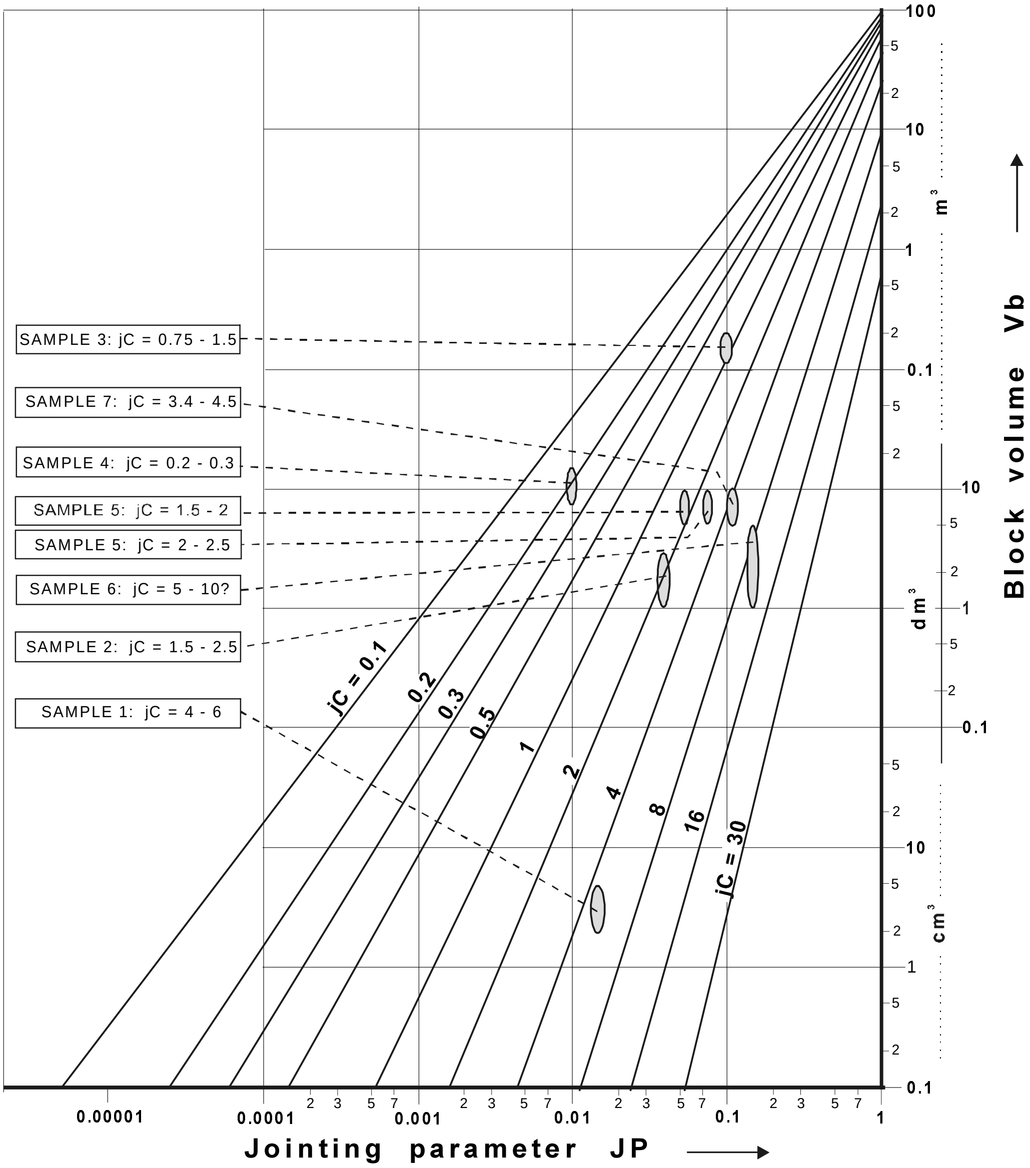

Fig. 4-3 The connection between block volume, joint condition and jointing parameter determined from plotting of the data sets described in Appendix 6.

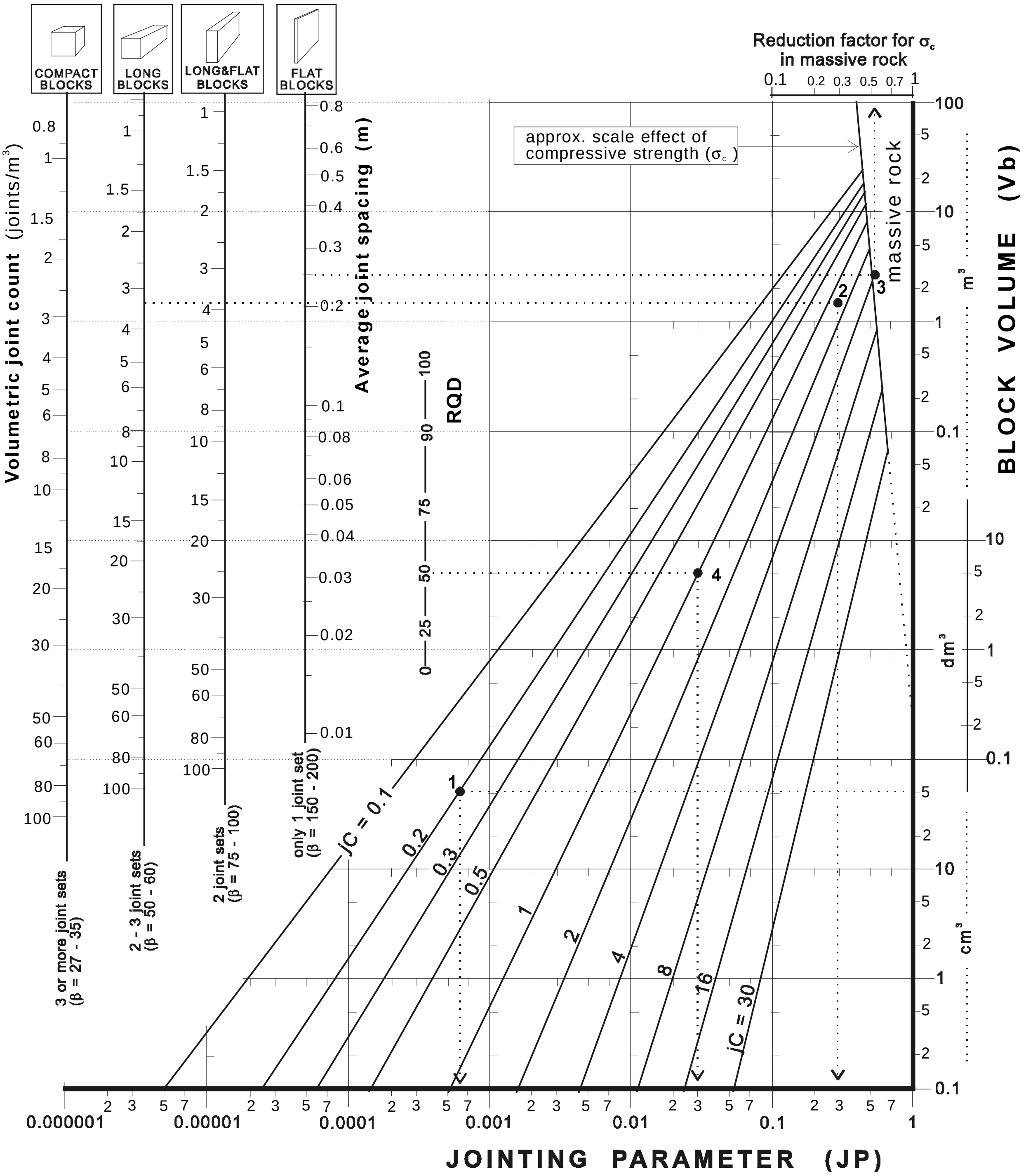

Fig. 4-4 Diagram for finding the value of the jointing parameter (JP) from the joint condition factor (jC) and various measurements of jointing density (Vb, Jv, RQD).

Examples shown in Fig. 4-4:

1: For Vb = 0.00005 m^3 (50 cm^3) and jC = 0.2 the JP = 0.0006;

2: For two joint sets with jC = 1.5 and the volumetric joint count Jv = 3.3 the JP = 0.3;

3: For one joint set with spacing S = 0.25 m and jC = 8 the JP = 0.5 (determined by the scale effect of compressive strength in massive rock)

4: For RQD = 50 and jC = 1 the JP = 0.03.

The jointing parameter can also be determined by the following expression which has been derived from the lines representing jC in Fig. 4-3:

$$JP = 0.2\sqrt{jC} \times Vb^{D} \qquad \text{eq. (4-4)}$$

where Vb = the block volume, given in m^3, and
D = 0.37 $jC^{-0.2}$, which has the following values:

jC =	0.1	0.25	0.5	0.75	1	1.5	2	2.5	3	4	6	9	12	16	20
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	0.203

Fig. 4-4 shows the same diagram where also other measurements than block volume can be applied directly. These are located in the upper left part of the figure. Here, the volumetric joint count (Jv) for various block shapes can be used instead of the block volume. Also, RQD can be used directly with the limitations of the accuracy in this measure as given in Appendix 4.

RMi is a material characterization of the structural material called "rock mass" as it involves only its inherent features. As it has a general form, RMi is not a quality characterization, but merely, within its limitations, a rock mass index strength, as further outlined in Section 4.5. The classification presented in Table 4-2 is suggested for RMi.

TABLE 4-2 CLASSIFICATION OF THE RMi

CHARACTERIZATION		**RMi VALUE** (MPa)
Term for RMi	Term related to rock mass strength	
Extremely low	Extremely weak	< 0.001
Very low	Very weak	0.001 - 0.01
Low	Weak	0.01 - 0.1
Moderately high	Moderately strong	0.1 - 1
High	Strong	1 - 10
Very high	Very strong	10 - 100
Extremely high	Extremely strong	> 100

For the most common joint conditions where jC = 1 - 2, the jointing parameter will vary between JP = 0.2 $Vb^{0.37}$ and JP = 0.28 $Vb^{0.32}$. For jC = 1.75 the jointing parameter can simply be expressed as

$$JP = 0.25\sqrt[3]{Vb} \qquad \text{eq. (4-5)}$$

and for jC = 1:

$$JP = 0.2\ Vb^{0.37} \qquad \text{eq. (4-6)}$$

The graphical solution of eq. (4-6) is presented in Fig. 4-6, from which estimates of RMi can be quickly made from the block volume and the uniaxial compressive strength of the rock

As shown in Fig. 4-5 significant *scale effects* are generally involved when a 'sample' is enlarged from laboratory size to field size. After the calibration described above, RMi is tied to large samples where the scale effect has been included in JP. The joint size factor (jL) is also a scale variable. For massive rock masses, however, where the jointing parameter JP ≈ 1 the scale effect for the uniaxial compressive strength (σ_c) must be accounted for, as σ_c is related to a 50 mm sample size. Barton (1990) suggests from data presented by Hoek and Brown (1980) and Wagner (1987) that the actual compressive strength for large 'field samples' with diameter (d is measured in mm) may be determined from

$$\sigma_{cf} = \sigma_{c50}\,(50/d)^{0.2} = \sigma_{c50}\,(0.05/Db)^{0.2} = \sigma_{c50} \times f_{\sigma} \qquad \text{eq. (4-7)}$$

where σ_{c50} = the uniaxial compressive strength for 50 mm sample size, and
$f_{\sigma} = (0.05/Db)^{0.2}$ is the scale factor for compressive strength

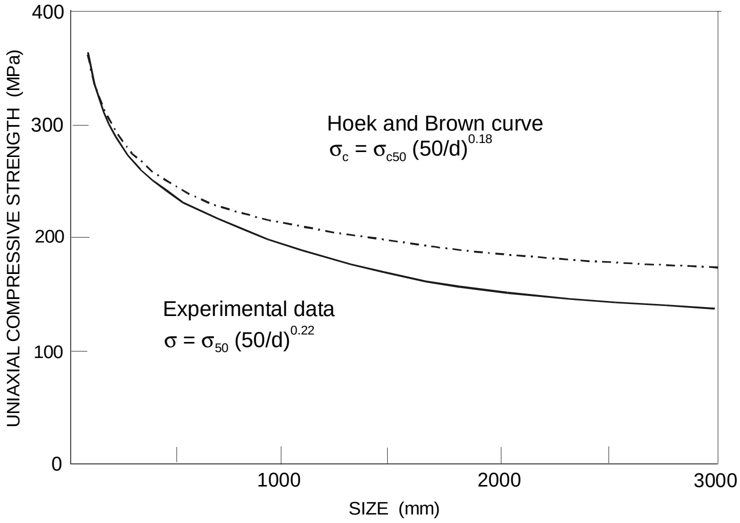

Fig. 4-5 Empirical equations for the scale effect of uniaxial compressive strength (from Barton, 1990, based on data from Hoek and Brown, 1980 and Wagner, 1987). Barton suggests to apply a value of 0.2 for the exponent.

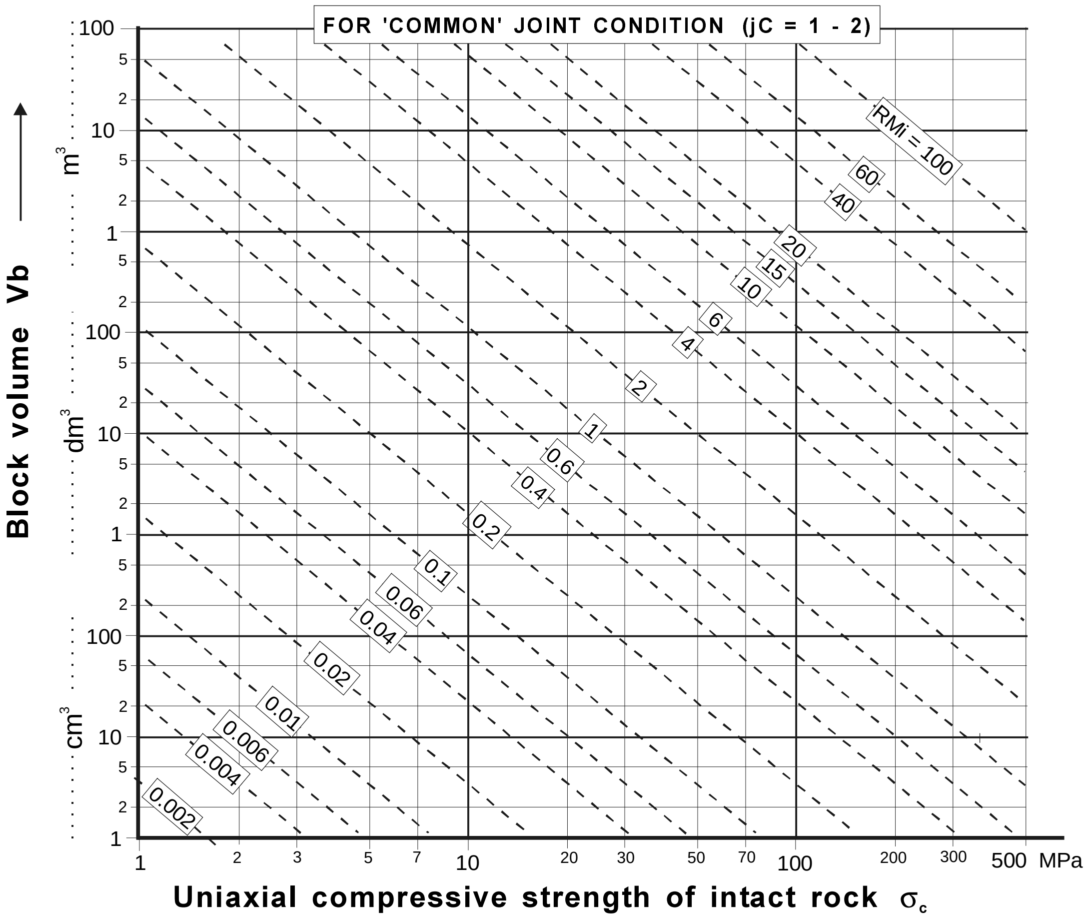

Fig. 4-6 RMi for 'common' joint condition (jC = 1 - 2) based on the uniaxial compressive strength of intact rock and the block volume. Example: for Vb = 0.2 m^3 and σ_c = 100 MPa the RMi = 25 MPa.

Eq. (4-7) is valid for sample diameter up to some metres, and may therefore be applied for massive rock masses as indicated in Fig. 4-4. The block diameter (Db) may be found from

$$Db = \frac{\beta_o}{\beta}\sqrt[3]{Vb} = \frac{27}{\beta}\sqrt[3]{Vb} \qquad \text{(eq. (6-8))}$$

as presented in Appendix 3 and in Chapter 6, or more approximately as $Db = \sqrt[3]{Vb}$ or simply by applying the spacing for the main joint set.

4.3 NUMERICAL VALUES OF THE INPUT PARAMETERS TO RMi

The various parameters used in RMi are shown in Fig. 4-2. Several simplifications had to be made in its structure to maintain an overview of the many properties of a rock mass. The volumes involved in a rock excavation and the size of the input parameters are generally so large that their numerical values mainly have to be determined from field observations. An exception is the compressive strength of intact rock. Well defined and practical usable descriptions are important for a good result. In this section it is shown how the ratings of these parameters have been determined. A more detailed description on how to find the values of these parameters is given in Appendix 3.

4.3.1 The compressive strength of intact rock (σ_c)

Several authors have stressed the importance of compressive strength of rock material as a classification parameter (Deere et al., 1969; Coates, 1964; Bieniawski, 1973, 1984, 1989; Piteau, 1970).

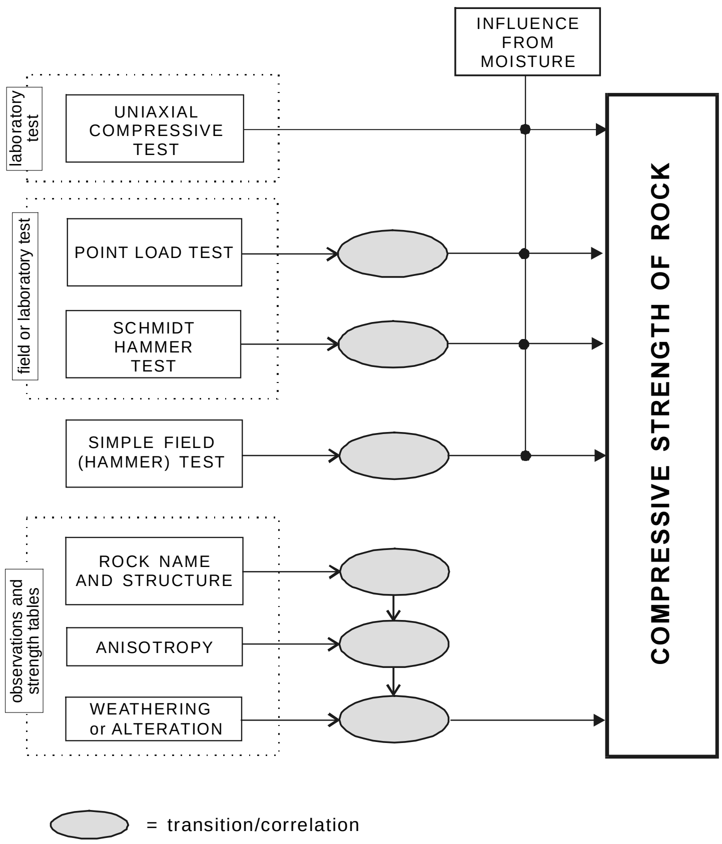

Fig. 4-7 Various methods to assess the uniaxial compressive strength.

The uniaxial compressive strength of rock can be determined in the laboratory according to the specifications given by the ISRM. Other ways of assessing this strength is indicated in Fig. 4-7. Wet specimens are used where the location of interest is below the ground water table. It should be noted whether the strength value used represents wet or dry conditions. Where no indication is given, dry specimens have normally been tested.

Where anisotropic rocks occur, *the lowest compressive strength should be applied* which generally will be a test direction at 25 - 45^o to the schistocity or layering as outlined in Appendix 3, Section 1.

The value of the uniaxial compressive strength (σ_c) in MPa from a 50 mmdiameter sample, is applied directly in RMi. For massive rocks (see Fig. 4-4) the scale effect of σ_c shown in eq. (4-7) should be applied.

4.3.2 The block volume (Vb)

The discontinuities cut the rock masses in various directions and delineate a bulk unit, which is simply referred to as the block. The block size is, therefore, intimately related to the degree of jointing. Each one of such blocks is more or less completely separated from others by various types of discontinuities. If all blocks in a rock mass volume could be measured or "sieved", a block size distribution similar to that for granular soils is found (Fig. 4-8).

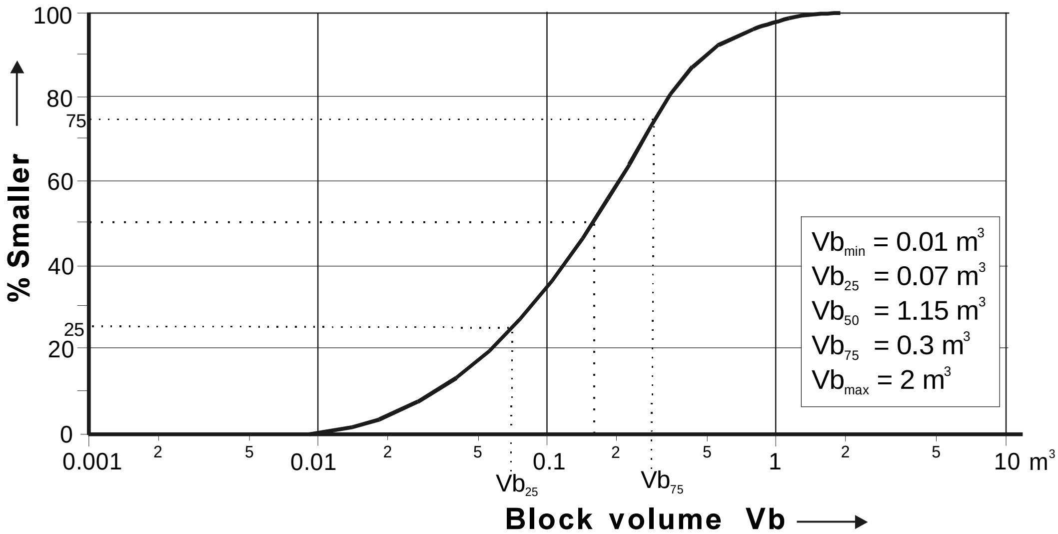

Fig. 4-8 Example of block size distribution

A great variation range will mostly be found between the sizes of the smaller and the larger blocks in a location; the characterization of the block size should, therefore, indicate their <u>size range</u>. Simplifications have often to be made during this measurement, as it is not possible to measure all blocks and their dimensions. Block size is, however, often the most important parameter in the RMi, and emphasis should be placed on this measurement. Possible ways for estimating the block volume are shown in Fig. 4-9.

The block volume (Vb) is used directly in the calculation of the jointing parameter (JP). As shown in Appendix 3, it can also be found from other measurements of the jointing density such as the volumetric joint count (Jv) and the rock quality designation (RQD). An improved technique for block size registration in surfaces and drill cores - the weighted joint measurement - is developed and described here together with a method for finding the block volume from refraction seismic measurements.

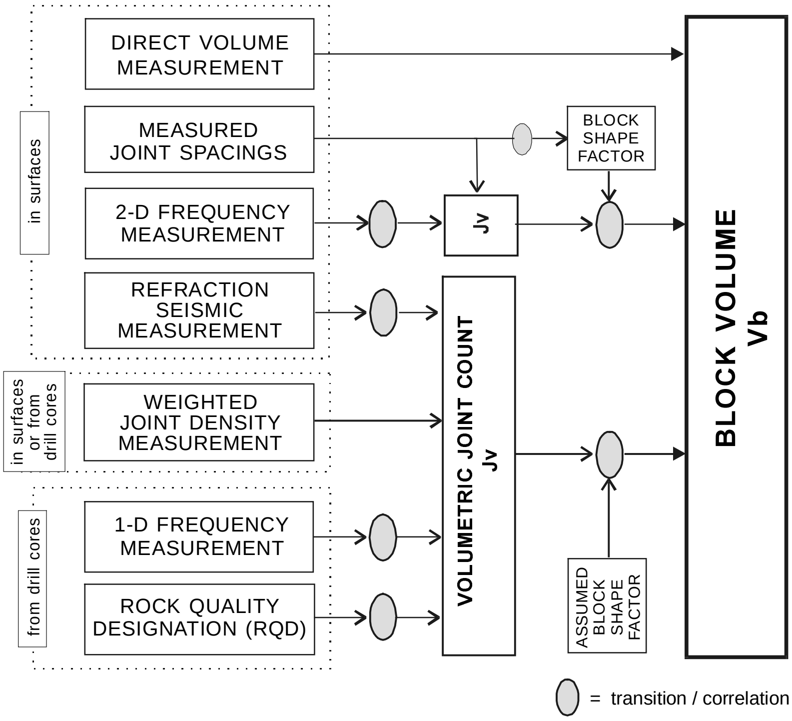

Fig. 4-9 Various methods to assess the block volume (Vb). See Appendix 3, Section 3.

4.3.3 The joint condition factor (jC)

The joint condition factor is meant to represent the friction properties of the block faces (i.e. joints) and the relative scale effect imposed by the joints.

The works of Patton (1966) have emphasized the importance of the surface characteristics of joints in determining their shear strength. Of particular importance was Patton's recognition that the shear resistance resulting from asperities on the joint surfaces had to be overcome during deformation either by sliding over or by shearing through.

The strength of the rock in which the discontinuities occur, has a direct bearing on the strength characteristics of the discontinuities, particularly where the walls are in direct rock to rock contact as in the case of unfilled joints (ISRM, 1978). The nature of *asperities*, particularly those of roughness and hardness, are likely to be dependent on the mineralogical and lithological make-up of the rock. Mineral *coatings* will affect the shear strength of discontinuities to a marked degree if the walls are planar and smooth (Piteau, 1970).

The distance between the two matching joint walls controls the extent to which these can interlock. In the absence of interlocking, the shear strength of the joint is that of the filling material. As separation decreases, the asperities of the rock wall gradually become more interlocked, and both the filling and the rock material contribute to the shear strength. According to Barton et al. (1974) the function $\tan^{-1}(Jr/Ja)$ in the Q system is a fair approximation to the friction angle of the joint. As shown in Appendix 3, the ratio jR/jA is similar to the ratio Jr/Ja.

The author has experienced during several years of geological engineering practice that the length of joints often has a significant influence on the behaviour of rock masses. Both Lardelli (1992) and

Kleberger (1992) have also stressed the importance of this observation, in particular the difference between partings and normal joints.

The properties or effects of a joint depend therefore on the following basic factors (Bieniawski, 1984; Piteau, 1970):

1. The *condition of the joint*, i.e.
   - The strength (hardness) of the wall rock material in *clean* joint surfaces, or the friction angle of the minerals in the coating.
   - Weathering of the wall rock of the planes of weakness.
   - The small scale asperities and large scale planarity of the joint surface (unevenness and waviness).
   - The distribution, thickness and nature of the gouge materials in *filled* joints.
   - Size (persistence) and termination of the joint.
2. The *external features*, such as
   - The shear movement that has occurred.
   - The presence or absence of water on the joint.

Each of the following three main parameters representing the joint condition is given a numerical value from well defined and simple field registrations based mostly on existing methods:

A. *The roughness factor* (jR) representing the unevenness of the joint surface which consists of:
   - the smoothness (js) of the joint surface, and
   - the waviness (jw) or planarity of the joint wall.

B. *The alteration factor* (jA) expressing the characteristics of the joint (Barton 1974):
   - the strength of the wall rock, or
   - the thickness and strength of a possible filling.

C. *The size factor* (jL) representing the influence of the size and termination of the joint.

The joint condition factor is found from the following expression:

$$jC = jL \times jR/jA = jL(js \times jw)/jA \qquad \text{eq. (4-8)}$$

Often, rough and inexpensive investigations are carried out where only an approximate estimate of the rock mass characteristics is sufficient. In such cases, there is often limited information on the parameters in jC. The parameters included in this factor have each been given unit values for common occurrences. Most commonly the value of jC = 1.5 - 2; using jC = 1 may generally be somewhat conservative, i.e. ’on the safe side’.

This entails that rough characterization of RMi can be made even if some of the parameters in the joint condition are absent. The RMi value will, of course, be less accurate in such cases. The benefit of this is that where the jointing condition is not known - for example in the case of refraction seismic measurements - the RMi can be estimated (of course, with limited accuracy) from input of only the rock strength and block size alone.

4.3.3.1 The joint roughness factor (jR)

The roughness factor is, as mentioned, similar to Jr in the Q-system. Roughness here includes both the small scale asperities (smoothness) on the joint surface and the large scale planarity of the joint plane (waviness). It has been found appropriate to divide the roughness into these two different features, as it is often easier to characterize them separately in the joint survey.

Surface *smoothness* or unevenness is the nature of the asperities in the joint surface, which can be felt by touch. This is an important parameter contributing to the condition of joints. Asperities that occur on joint surfaces interlock, if the surfaces are clean and closed, and inhibit shear movement along joint surfaces. Asperities usually have a wave length and amplitude measured in millimetres and are readily apparent on a core-sized exposure of a discontinuity. The applicable descriptive terms are defined in Table 4-3.

TABLE 4-3 CHARACTERIZATION OF THE SMOOTHNESS FACTOR (js). THE DESCRIPTION IS PARTLY BASED ON BIENIAWSKI (1984) AND BARTON ET AL. (1974).

TERM	**DESCRIPTION**	**factor js**
Very rough	Near vertical steps and ridges occur with interlocking effect on the joint surface.	3
Rough	Some ridge and side-angle steps are evident; asperities are clearly visible; discontinuity surface feels very abrasive (like sandpaper grade approx.< 30)	2
Slightly rough	Asperities on the discontinuity surfaces are distinguishable and can be felt (like sandpaper grade approx. 30 - 300).	1.5
Smooth	Surface appear smooth and feels so to the touch (smoother than sand-paper grade approx. 300).	1
Polished	Visual evidence of polishing exists, or very smooth surface as is often seen in coatings of chlorite and specially talc.	0.75
Slickensided	Polished and often striated surface that results from friction along a fault surface or other movement surface.	0.6 - 1.5

Waviness of the joint wall appears as undulations from planarity. It is defined by

$$U = \frac{\text{max. amplitude } (a_{max}) \text{ from planarity}}{\text{length of joint (Lj)}}$$

The maximum amplitude or offset (a_{max}) can be found using a straight edge which is placed on the joint surface. The length of the edge should be of the same size as the joint, provided that this is practically possible. As the length of the joint seldom can be observed or measured, simplifications in the determination of (U) have to be done. A procedure described by Piteau (1970) can be applied with a standard 0.9 m long edge. Barton (1982) has used a length of 200 mm for joint roughness coefficient (JRC) measurements. For the smallest joints even shorter lengths can be applied. The simplified waviness or undulation is found as

$$u = \frac{\text{measured max. amplitude (a)}}{\text{measured length along joint (L)}}$$

The ratings of the waviness factor are shown in Table 4-4. Often it is found sufficient to determine the waviness by visual observation as described in Appendix 3, because undulation measurements are time-consuming.

TABLE 4-4 CHARACTERIZATION OF WAVINESS FACTOR (jw).

TERM	**undulation (u)**	**waviness factor** (jw)
Interlocking (large scale)		3
Stepped		2.5
Large undulation	u > 3 %	2
Small undulation	u = 0.3 - 3 %	1.5
Planar	u < 0.3 %	1

The joint roughness factor is found from $jR = js \times jw$, or it can also be determined from Table 4-5. As the ratings of these parameters are based on the Q system, the joint roughness factor (Jr) in the Q system is, as mentioned, similar to jR.

TABLE 4-5 JOINT ROUGHNESS FACTOR (jR) FOUND FROM SMOOTHNESS AND WAVINESS. THE VALUES ARE SIMILAR TO Jr IN THE Q SYSTEM.

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

*) For filled joints in Table 4-6 jR = 1
**) For slickensided joints the Jr value depends on the presence and outlook of the striations the highest value is used for marked striations.

Joint roughness includes the condition of the joint wall surface both for filled and unfilled (clean) joints. For joints with filling thick enough to avoid contact of the two joint walls, any shear movement will be restricted to the filling, and the joint roughness will then have minor or no importance (See Appendix 3, Section 2). In the cases of filled joints it is often difficult or impossible to measure the smoothness and often also the waviness. Therefore the roughness factor is defined as jR = 1 as in the Q system (where Jr = 1).

4.3.3.2 The joint alteration factor (jA)

This factor is for a major part based on Ja in the Q-system. It represents both the strength of the joint wall and the effect of filling and coating materials. The strength of the surface of a joint is a very important component of shear strength and deformability where the surfaces are in direct rock to rock contact as in the case of unfilled (clean and coated) joints (Bieniawski, 1984, 1989). The strength of the joint surface is determined by the following:
- the condition of the surface in clean joints,
- the type of coating on the surface in closed joints,
- the type, form and thickness of filling in joints with separation.

When *weathering* or *alteration* has taken place, it can be more pronounced along the joint wall than in the block. This results in a wall strength that is often some fraction of what would be measured on the fresher rock found in the interior of the rock blocks. The state of weathering or alteration of the joint surface where it is different from that of the rock material, is therefore essential in the characterization of the joint condition.

TABLE 4-6 CHARACTERIZATION AND RATING OF JOINT ALTERATION FACTOR (jA)
(partly based on Ja in the Q-system)

A. CONTACT BETWEEN THE TWO ROCK WALL SURFACES		
TERM	**DESCRIPTION**	**jA**
Clean joints		
-Healed or "welded" joints .	Softening, impermeable filling (quartz, epidote etc.)	0.75
-Fresh rock walls	No coating or filling on joint surface, except of staining	1
-Alteration of joint wall:		
1 grade more altered	The joint surface exhibits one class higher alteration than the rock	2
2 grades more altered	The joint surface shows two classes higher alteration than the rock	4
Coating or thin filling		
-Sand, silt, calcite etc.	Coating of friction materials without clay	3
-Clay, chlorite, talc etc.	Coating of softening and cohesive minerals	4

B. FILLED JOINTS WITH PARTLY OR NO CONTACT BETWEEN THE ROCK WALL SURFACES			
TYPE OF FILLING MATERIAL	**DESCRIPTION**	**Partly wall contact** thin fillings (< 5 mm*)) **jA**	**No wall contact** thick filling or gouge **jA**
-Sand, silt, calcite etc.	Filling of friction materials without clay	4	8
-Compacted clay materials	"Hard" filling of softening and cohesive materials ..	6	10
-Soft clay materials	Medium to low over-consolidation of filling	8	12
-Swelling clay materials	Filling material exhibits clear swelling properties...	8 - 12	12 - 20

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

TABLE 4-7 ENGINEERING CHARACTERIZATION OF WEATHERING/ALTERATION. (from Lama & Vutukuri, 1978)

GRADE	TERM FOR WEATHERING OR ALTERATION	DESCRIPTION
I	**Fresh**	No visible signs of weathering. Rock fresh, crystals bright. Few discontinuities may show slight staining.
II	**Slightly**	Penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material. Discontinuities are discoloured and dis-coloration can extend into rock up to a few mm from discontinuity surface.
III	**Moderately**	Slight discoloration extends through the greater part of the rock mass. The rock material is not friable (except in the case of poorly cemented sedimentary rocks). Discontinuities are stained and/or contain a filling comprising altered materials.
IV	**Highly**	Weathering extends throughout rock mass and the rock material is partly friable. Rock has no lustre. All material except quartz is discoloured.Rock can be excavated with geologist’s pick.
V	**Completely**	Rock is totally discoloured and decomposed and in a friable condition with only fragments of the rock texture and structure preserved. The external appearance is that of a soil.
VI	**Residual soil**	Soil material with complete disintegration of texture, structure and mineralogy of the parent rock.

The alteration factor (jA) is, as seen in Table 4-6, somewhat different from (Ja) in the Q system. Some changes have also been made in an attempt to make field observations easier and quicker. The values of Ja can be used - provided the alteration of the joint wall is the same as that of the intact rock material.

The various classes of rock weathering/alteration that can be determined from field observations, are shown in Table 4-7.

4.3.3.3 The joint length and continuity factor (jL)

Several writers have experienced during many years of geological engineering that the size and continuity of the joints often have great influence on the properties of rock masses. Both Lardelli (1992) and Kleberger (1992) have stressed this, in particular the difference in importance between partings and normal joints upon rock mass behaviour.

The *joint length* can be crudely quantified by observing the discontinuity trace lengths on surface exposures. It is an important rock mass parameter, but is one of the most difficult to quantify in anything but crude terms. Frequently, rock exposures are small compared to the length of persistent discontinuities, and the real persistence can only be guessed. However, the difficulties and uncertainties involved in the field measurements will be considerable for most rock exposures encountered. The size or the length of the joint is often a function of the thickness or separation of the joint, and can sometimes be evaluated from this feature. This is further described in Appendix 3 (Section 2.4 and Fig. A3-18).

As the exact length of a joint seldom can be found, the most important task is to estimate the size range of the joint. Often it is no problem to observe the difference between partings and medium or larger sized joints during field observations. *Joint continuity* is divided into two main groups:
- continuous joints that terminate against other joints
- discontinuous joints that terminate in massive rock.

TABLE 4-8 THE JOINT SIZE AND CONTINUITY FACTOR (jL).

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

*) Often a singularity, and should in these cases be treated separately.

The joint size factor for continuous joints can also be expressed as

$$jL = 1.5 \times L^{-0.3} \qquad \text{eq. (4-9)}$$

where L = the length of the joint in metre. The ratings of jL are shown in Table 4-8.

4.4 POSSIBLE AREAS OF APPLICATION OF THE RMi

The main purpose during the development of the RMi has been to work out a system to characterize rock masses, which is applicable in rock engineering. As RMi is linked to the material, it represents only the inherent properties of a rock mass. Thus, it does not express external loads or forces acting on the material, such as:

- the in situ rock stresses;
- the presence of ground water;
- the orientation of:
  - loads or stresses,
  - structural elements (joints, anisotropy, etc.),
  - permeability or ground water flow; and
- the impacts from human activity.

For application of RMi in practical rock mechanics and engineering for civil or mining, one or more of these features usually have to be included where they have influence or impact on the ground conditions.

The main activities where rocks and rock masses involved are shown in Table 4-9, which also indicates the fields in which the RMi may be of interest.

As RMi is a strength index it is suitable for application in rock engineering, design or other evaluations connected with utilization of rocks. This is fully shown in Chapter 6 on stability assessments for rock support analysis and in Chapter 7 on capacity evaluation of tunnel boring machines (TBM). For these applications the RMi is adjusted for local features of importance to determine the behaviour of the rock mass.

The RMi value can not be used directly in existing classification systems as many of them are systems of their own. Some of its input parameters are sometimes similar to those used in these classifications and may then be applied more or less directly, see Chapter 8.

TABLE 4-9 A BRIEF VIEW OF MAIN INTERNAL ROCK MASS PARAMETERS AND THEIR IMPORTANCE IN ROCK AND ROCK MASS UTILITIES.

UTILITY	ACTIVITY	RELATIVE IMPORTANCE OF:		
		rock strength	jointing	singularities *)
Treatment of rocks and rock masses	· drilling (small holes) + boring (TBM boring, reaming) = blasting = fragmentation · crushing · grinding = cutting	x x x (x) x x x	-/(x) x (x) x - - x	- (x) (x) (x) - - (x)
Application of rock material	· rock aggregate for concrete · rock fill · natural stone/building stone	x x x	(x) x x	- - (x)
Utilization of rock masses	+ underground excavations + surface cuts and slopes = foundations for dams etc.	(x)/x (x) (x)	x x x	x x x
Legend:	+ Suitable for characterization = May partly be suitable for characterization · Generally not suitable	x great influence (x) limited influence - little or no influence		

*) These are seams, shears, weakness zones

Hoek (1983, 1986) and Hoek and Brown (1988) mention that further work is required to improve the Hoek-Brown failure criterion, since the use of classification systems developed for the design of tunnel support has been found to have some limitations when used for estimating rock mass strength parameters. They suggest that it may be necessary to develop a system specifically for this purpose. As described in Section 5.2 in this chapter and further dealt with in Chapter 8, the jointing parameter (JP) in the RMi is similar to one of the main parameters in the Hoek-Brown failure criterion for rock masses. The RMi may, therefore, contribute to such a future system.

The rock mass strength characteristics found from RMi can also be further applied in NATM classification and rock support design as well as in ground response curves, as demonstrated in Chapter 8. Finally, it should be mentioned that the system for characterizing block geometry (volume, shape factor, angles) may be of use in numerical models.

4.5 DISCUSSION

The following discussion is limited to the structure and development of the RMi as dealt with in this chapter. A discussion of the RMi system, its use, and a comparison with other engineering systems and methods is presented in Chapter 9.

4.5.1 Limitations of the RMi

The RMi is meant to express the relative variation in the strength between different rock masses. As determination of the strength of an in situ rock mass by laboratory type testing for many reasons is not practical, the RMi makes use of input from geological observations and test results on individual rock pieces or rock surfaces which have been removed from the actual rock mass.

RMi is restricted to expressing only the compressive strength. Hence, it has been possible to arrive at a simple expression, contrary to, for example, the general failure criterion for jointed rock masses developed by Hoek and Brown (1980) and Hoek et al. (1992). Because simplicity has been preferred in the structure and in selection of parameters in RM, it is clear that such an index may result in inaccuracy and limitations, the most important of which are connected to:

A. *The range and types of rock masses covered by the RMi.*
Both the intact rock material as well as the joints exhibit great directional variations in composition and structure, which results in an enormous range of properties of rock masses. It is, therefore, not possible to characterize all these combinations in one, single number. However, it should be added that the RMi probably may characterize a wider range of materials than most other classification systems. Characterization of rock masses by the RMi is presented in Chapter 5.

B. *The accuracy in the expression of the RMi.*
The value of the jointing parameter (JP) is calibrated from a few large scale compression tests. Both the evaluation of the various factors (jR, Ja and Vb) in JP and the size of the samples tested, which in some of the cases had less than 5x5x5 blocks, have resulted in that there certainly are errors connected to the expression developed for the JP. In addition, the test results used were partly made on dry, partly on wet samples (Stripa on wet). The influence of moisture may have reduced the accuracy of the data used.
Also, the uniaxial tests are encumbered with errors as pointed out by Farmer and Kemeny (1992) and in Appendix 3, Section 1. The value of RMi found can, therefore, be very approximate. In some cases, however, the errors in the various parameters may partly cancel out.

C. *The effect of combining parameters that vary in range.*
The input parameters to the RMi express generally a certain range of variation related to changes in the actual representative volume of the rock mass. The combination of such ranges

in RMi may cause additional errors. Chapter 5 briefly outlines some methods to reduce this effect.

The result of the foregoing is that RMi in many cases will give an inaccurate value for the strength of such a complex assemblage of different materials and defects as in a rock mass. For this reason, the RMi is regarded as a *relative* expression of the rock mass strength. It should preferably be used in communication and characterization.

Being valid for minimum 125 blocks the direct use of the RMi involves larger volumes of rock masses than is actually required. RMi can, therefore, seldom be directly applied in engineering and design. Some modification or supplementary adjustments have generally to be made as shown in Chapters 6 on rock support and Chapter 7 on TBM.

4.5.2 Other similar rock mass characterization methods

The RMi has been developed during a process that has involved a critical examination of rock mass characteristics and available literature. The main philosophy has been to take account of the effect of discontinuities in reducing the strength of intact rock.

Earlier, a similar approach to a strength characterization of rock masses has been proposed by Hansagi (1965, 1965b), who introduced a similar reduction factor to the jointing parameter (JP) to arrive at an expression for the *compressive* strength of the rock mass, given as

$$\sigma_{mc} = \sigma_c \times C_g \qquad \text{eq. (4-10)}$$

where σ_c = compressive strength,
C_g = the reduction factor which Hansagi named 'gefüge-factor' (joint factor) being "*representative for the jointed effect of a rock mass*".

The C_g factor consists of two inputs: a factor for the "structure of jointing" (core length), and a scale factor. Hansagi (1965b) mentions that the value of C_g is 0.7 for massive rock and 0.47 for jointed rock (from small joints) for two test locations in Kiruna, Sweden. Hansagi did not, however, - as far as the author knows - publish more on his method.

From Fig. 4-10 it is seen that the expression for the RMi is also similar in structure to the expression of unconfined *compressive strength of rock masses* (σ_{cm}), which is a part of the Hoek-Brown failure criterion for rock masses, and is expressed as

$$\sigma_{cm} = \sigma_c \times s^{1/2} \qquad \text{eq. (4-11)}$$

Here σ_c = the uniaxial compressive strength of the intact rock material, and
s = an empirical constant. The value of *s* ranges from 0 for jointed rock masses to 1 for intact rock. The value of s is found from the RMR or the Q classification system as described by Hoek (1983), Hoek and Brown (1980, 1988), and Wood (1991).

Thus, the jointing parameter(JP) is similar to $s^{1/2}$ in the Hoek-Brown failure criterion. The process of finding JP is, however, more direct and clear as it only involves features that have a direct impact on this parameter. This is further described in Chapter 8.

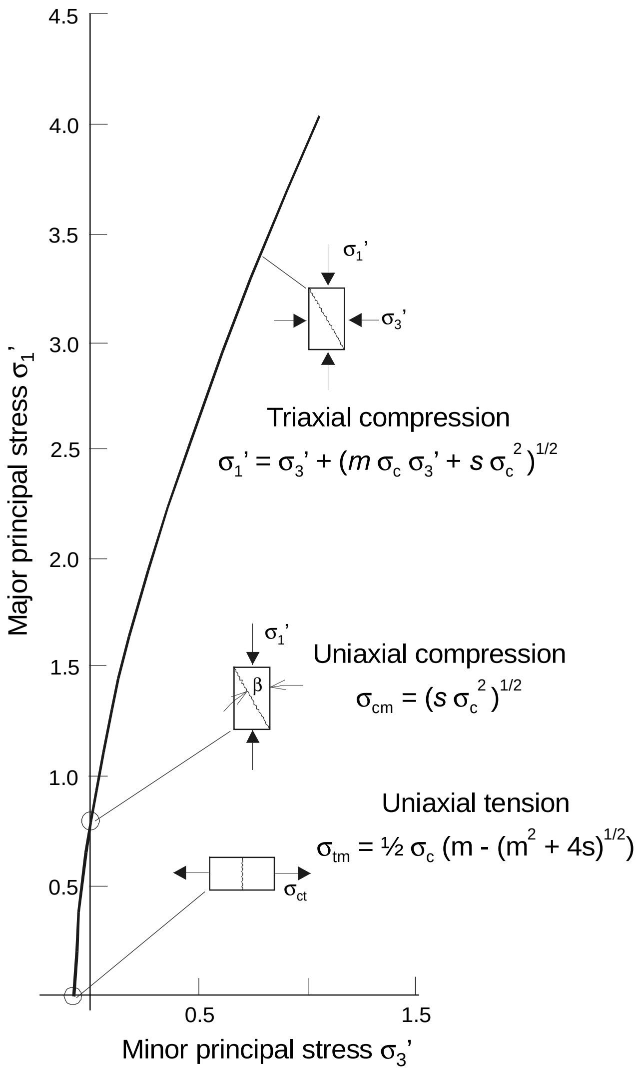

Fig. 4-10 The uniaxial compressive strength is one special mode of the Hoek-Brown failure criterion for rock masses (from Hoek, 1983).