Application of RMi System for the Design of Tunnel Supports

Naresh Kumar
Central Water Commission,
611 (S), Sewa Bhawan, R.K. Puram,
New Delhi – 110 066, India
Tel.: (011) 25176495
E-mail: kumarnaresh@hotmail.com

N.K. Samadhiya
Department of Civil Engineering,
Indian Institute of Technology Roorkee,
Roorkee – 247 667, India
Tel.: (01332) 275133
E-mail: nksamfce@iitr.ernet.in

ABSTRACT

Engineering rock mass classification is the best-known empirical approach for assessing the stability of underground openings. This approach has got enormous potential and forms the backbone of present day rock engineering. RMi-system, developed recently, is one of the several systems of rock mass classification.

Estimation of RMi and application in the design of tunnel supports have not been tested and also not fully understood. This paper offers some clarifications and suggestions for using support charts for different ground conditions and method of estimation of block volume, an important input to the RMi. Refinements have also been proposed in estimation of stress level factor in blocky ground.

Keywords: RMi-system, blocky ground, continuous ground, shape factor, block volume, volumetric joint count, support design.

1. INTRODUCTION

Reliable tests for determining strength properties of rock mass are difficult, if not impossible. Therefore, practical rock engineering is still based mainly on input data determined from observations of the rock mass in-situ. The quality of input data significantly affects the accuracy in rock engineering and design. Hoek et al. (1992) are of the opinion that the strength characteristics for jointed rock masses are controlled by the block shape and size, and their surface characteristics determined by the intersecting joints. This does not imply that the properties of the intact rock material should be disregarded in the characterization. Hoek and Brown (1980), and Bieniawski (1984) have indicated the need for a strength characterization of rock masses. Therefore, Palmstrom (1995a, b, c) has developed RMi-system to characterize the strength of the rock mass for construction purposes.

RMi-system considers only intrinsic parameters of the rock mass viz. compressive strength of intact rock, block volume and joint characteristics such as roughness, alteration and size (Fig.1). RMi-system is based principally on the reduced rock strength caused by jointing and is expressed as:

$$RMi = \sigma_c \cdot JP \tag{1}$$

where

RMi = Rock Mass index proposed by Palmstrom (1995a),

σ_c = uniaxial compressive strength of intact rock, and

JP = the jointing parameter.

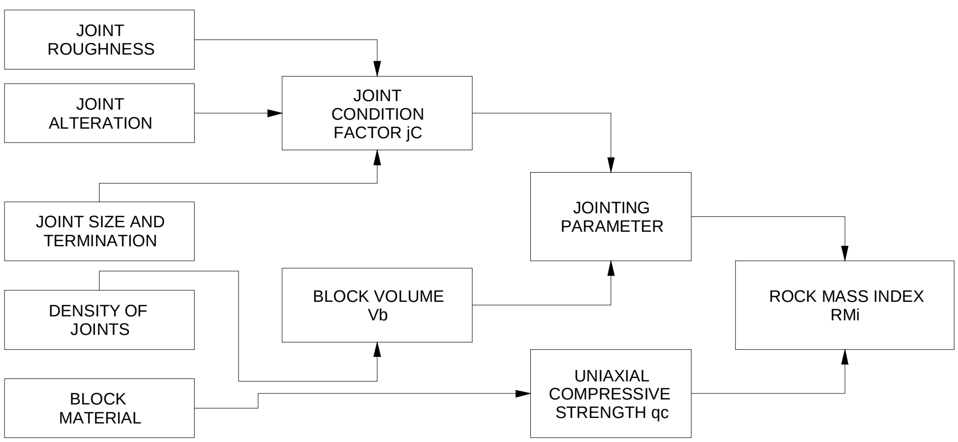

Fig. 1 – Input parameters of RMi

Here JP represents the main jointing features, namely block volume (or density of joints), roughness, alteration and size of the joints. JP can be found from Fig. 2 by using the block volume (V_b) or the volumetric joint count (J_v), the joint spacing or the RQD.

Since RMi-system has been developed only recently, its estimation and application in the design of tunnel supports have not been tested at large and also not clearly understood. Moreover, case histories throwing light on the application of this system are also not available. Palmstrom (1995a, b, c, 1996, 2000a) has explained in detail about the RMi-system but one gets the impression through these publications that the RMi-system is still in its evolution stage. Until it is applied in more and more field cases, its potential as an effective means of rock mass classification might not be fully understood and appreciated.

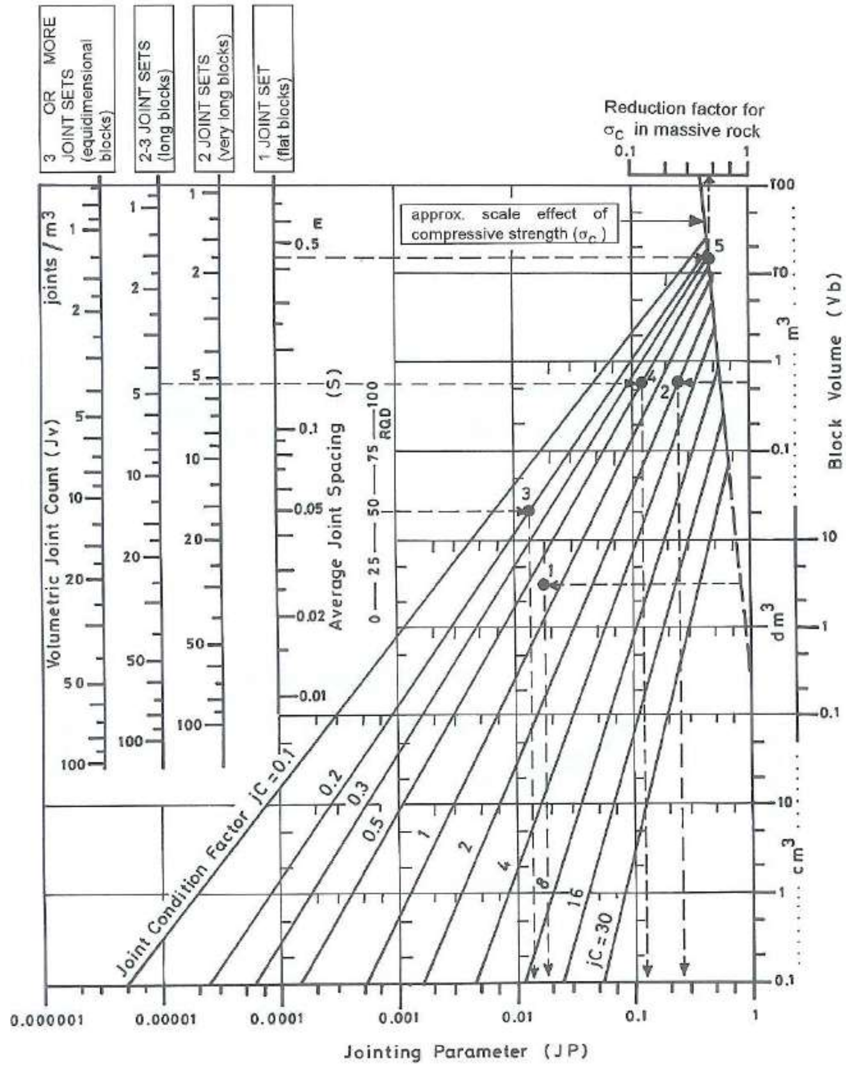

Fig. 2 – The jointing parameter J_P found from the joint condition factor jC and various measurements of jointing intensity (Vb, J_v, RQD)

Kumar (2002) has utilized RMi-system apart from various other systems for the classification of 10.15 m diameter, 27.4 km long headrace tunnel of ongoing Nathpa Jhakri Project located in higher Himalaya. Enriched from the experience of this study, the present paper attempts to throw light on some important issues related to the support design and also offers suggestions for estimation of block volume, an important input to the RMi.

2. TYPES OF GROUND

From engineering view point, the ground can be defined by various parameters. The ground parameters having major influence on stability in underground openings are as follows:

a. The inherent properties of the rock mass:
- The intact rock strength
- The jointing properties
- The structural arrangement of the discontinuities
- The properties specific to weakness zones

b. The external factors:
- The insitu stress condition
- The groundwater condition

c. The excavation parameters:
- The shape and size of the opening
- The excavation method
- The ratio, tunnel dimension / block size

The above-mentioned parameters have been combined in following manner to account for influence of stability of excavation:

A. The number of blocks on the periphery of an underground opening will largely determine whether the surrounding ground will behave: (i) as a blocky material, dominated by the individual blocks and the character of the joints; or (ii) as a continuous, bulk material where the magnitude of the rock stresses is important. Continuity of the ground has been represented by a continuity factor (CF) as follows:

$$CF = \text{tunnel diameter / block diameter} = D_t/D_b \qquad (2)$$

- For slightly jointed (massive) rock, $CF < 5$ (app.)
- For highly jointed (particulate) rock, $CF > 100$ (app.)

In case of blocky rock masses, value of CF may be assumed in between 5 and 100.

B. The condition of the ground factor comprises of inherent rock mass parameters and stress conditions. A ground condition factor (Gc) has been applied in blocky ground, and a competency factor (Cg) is introduced in continuous ground.

2.1 Blocky Ground

The stability in blocky (jointed) ground is mainly influenced by the block size and shape, the shear strength of the joints delineating the blocks, and the orientation of the same joints relative to the opening. The ground condition factor for blocky ground includes the inherent rock mass characteristics that have a significant influence on stability as well as the external stresses. Following two parameters are used in support design.

- Ground condition factor,

  G_c = RMi . SL . C (3)

- Size ratio, S_r = CF . C_o / N_j, (4)

where
SL = a stress level adjustment,
C = a gravity adjustment factor,
C_o = an adjustment factor for joint attitude, and
N_j = an adjustment factor for number of joint sets.

According to Palmstrom (1996), the influence of joint water pressure is generally difficult to incorporate in the stress level. Often, the joints around the tunnel will drain the ground water in the large quantity nearest to the tunnel, hence the influences from groundwater pressure on the effective stresses is limited. The total stresses have, therefore, been selected. In some cases, however, where unfavorable orientation of joints combined with high ground pressure, it will tend to reduce the stability by extra loading on key blocks, the stress level factor should be reduced as follows:

Divide SL by 2.5 for moderate influence (5)

Divide SL by 5 for significant influence (6)

In the Q-system, effect of joint water has been considered by using a factor called joint water factor (J_w) whose values are 1.0, 0.66, 0.5 and 0.33 corresponding to groundwater discharges. J_w is in the numerator $\left(\frac{J_w}{SRF}\right)$ for computing Q-value. From the present study, it has been found that for J_w to be in the denominator, these values will convert into 1.0, 1.5, 2.0, 3.0, and so on. Now comparing it with Eqs. 5 and 6, refinement may be done in computing SL depending upon level of influence of groundwater. One can be sure of what value has to be chosen for dividing SL, instead of first two values of 2.5 and 5 as suggested by Palmstrom (1996) in Eqs. 5 and 6.

2.2 Continuous Ground

In the continuous ground when CF < approximately 5 (massive rock), the properties of intact rock dominate, and when CF > approximately 100 (particulate or highly jointed rock), the ground behaves as a bulk material. In these types of ground the main influence on the behaviour in an underground opening comes from the stresses around the opening and the strength of the rock mass. For this purpose, a new parameter, competency factor (C_g), has been considered and defined as follows:

C_g = RMi / σ_θ (7)

where

σ_θ = the tangential stress in the rock masses around the opening.

Competent ground occurs where $C_g > 1$; else the ground is overstressed (incompetent). Massive, competent ground is generally stable and don't need any support whereas incompetent ground requires supports due to either squeezing or rock bursting.

3. DESIGN OF TUNNEL SUPPORTS

Palmstrom (2000a) has given support charts for above two types of grounds as shown in Figs. 3 and 4. For proper application of RMi support methods, the following clarifications are worth mentioning:

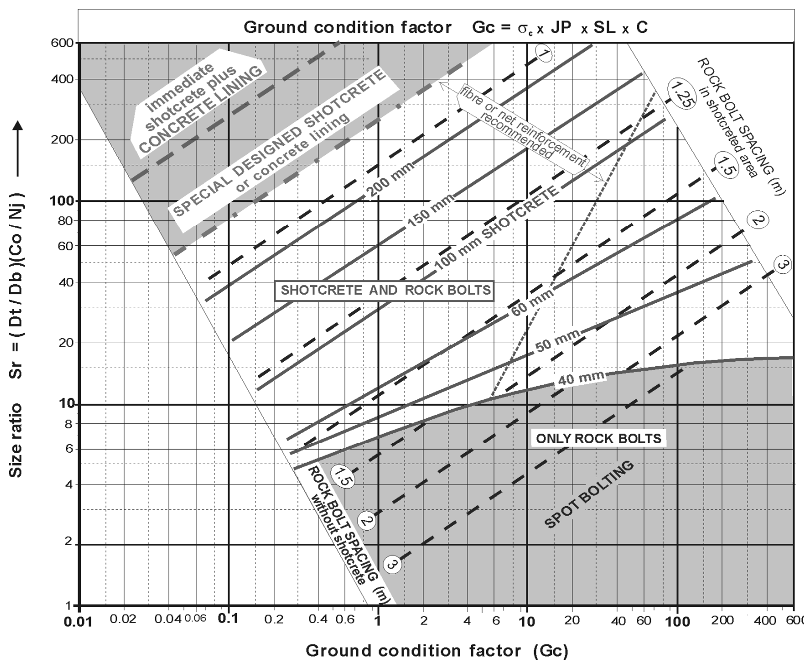

Fig. 3 – Rock support for blocky ground including weakness zones (Palmstrom, 2000a)

(A) In massive brittle rock, if $C_g < 1.0$, failure of rock mass will take place on account of rock bursting whereas in ductile rock, failure is on account of squeezing. These rock failures are termed as 'stress-controlled' failures; tangential stress in the tunnel is more than RMi (which is nothing but modified uniaxial compressive strength of rock mass). Rock bursting or squeezing occur in incompetent rocks.

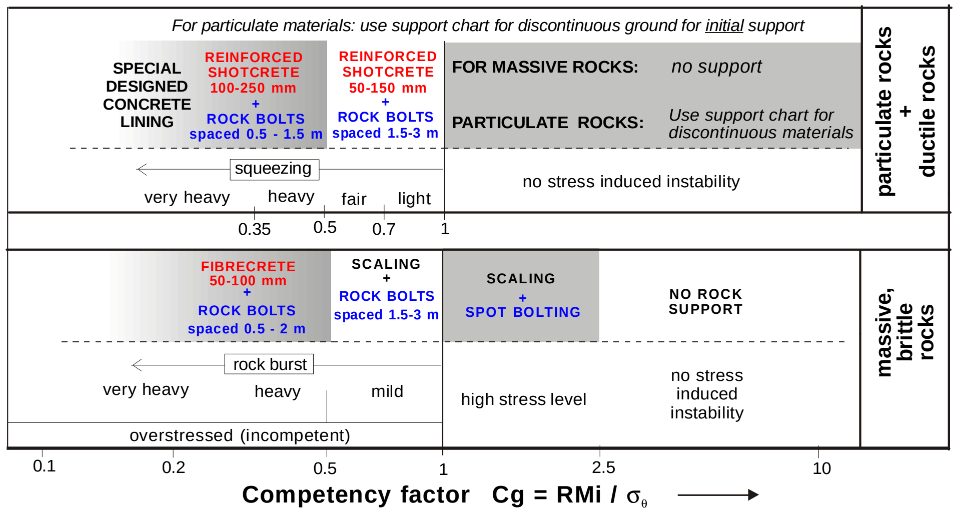

Fig. 4 – Chart for estimating support in continuous (massive and particulate) ground (Palmstrom, 2000a)

(B) Competency factor $\left(C_g = \dfrac{\text{RMi}}{\sigma_\theta} \right)$ is meant for characterization of failure modes in massive rocks only. This is done on the basis of values of σ_c used by Barton et al. (1993). Palmstrom (1995a) has used this classification but only difference being that RMi replaces σ_c. The value of σ_c is related to the compressive strength of 50 mm diameter samples. In massive rock masses the block size is significantly larger in the range 1-15 m^3 for which the factor for scale effect is in the range 0.45 - 0.55.

From above, it is clear that roughly RMi ≈ 0.5.σ_c. Thus, the values of $\dfrac{\text{RMi}}{\sigma_\theta}$ used by Palmstrom (1995a) are half of those used by Barton et al. (1993).

(C) Ground characterization for squeezing rock masses (only massive) has been done by Palmstrom (2000a) based on research of Aydan et al. (1993). On this basis, whether it is light, fair, heavy or very heavy squeezing, support system has been suggested. This condition, according to Palmstrom (2000a), is applicable for highly jointed rock (i.e. continuous ground for which D_t/D_b > 100) also. But this has to be adopted with caution since it is based on a limited amount of results for massive rocks only. It cannot be directly applicable for highly jointed ground. For initial support, however, support chart for blocky ground may be used.

(D) In highly jointed rock masses (i.e. continuous ground), failure may take place due to excessive stresses ('stress-controlled' failure as C_g < 1) or due to sliding, spalling or slabbing of wedges/blocks (this type of failure with C_g > 1 is termed as 'structurally-controlled' failure). In case of 'stress-controlled' failures in the highly jointed rock mass, squeezing will take place and support system will be determined on the basis of degree of squeezing, which in turn shall depend on C_g. In case of 'structurally-controlled' failure in the highly jointed rock mass ($C_g > 1$), support system will be selected from the chart of blocky ground.

(E) Irrespective of the types of rock masses i.e. jointed (blocky) or highly jointed (continuous) both competency factor (C_g) and ground condition factor (G_c) should be determined and support system selected accordingly from charts. Higher of the two support systems should be adopted.

4. ESTIMATION OF BLOCK VOLUME

The block volume (V_b) is intimately related to the intensity or degree of jointing. Each one of such blocks is more or less completely separated from other by various types of discontinuities. The greater the block size the smaller will be the number of joints penetrating the rock mass. Hence, there is an inverse relationship between the block volume and the number of joints.

The block volume 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 blocky ground, V_b is included both in the ground condition factor and in the size ratio. The accuracy of this measurement has a significant impact on the reliability of the RMi value. The bock size is a result of the detailed jointing in a rock mass formed mainly by the small and moderate joints. The block dimensions are determined by joint spacing and the number of joint sets.

Palmstrom (1995b, 1996, 2000a) has given charts for estimation of V_b by correlating it with type of blocks and block shape factor (β) (Fig. 5). Palmstrom (1995a, c) has also given following correlation between block volume and volumetric joint count (J_v):

$$V_b = \frac{\beta}{J_v^3} \cdot \frac{1}{\sin\gamma_1 . \sin\gamma_2 . \sin\gamma_3} \tag{8}$$

$$\beta = \frac{(\alpha_2 + \alpha_2\alpha_3 + \alpha_3)^3}{(\alpha_2 . \alpha_3)^2} \tag{9}$$

where γ_1, γ_2 and γ_3 are the angles between the joint sets; $\alpha_2 = S_2/S_1$ and $\alpha_3 = S_3/S_1$, provided $S_3 > S_2 > S_1$ and S_1, S_2, S_3 etc. are the spacing between the individual joints in each set in metres.

Considering γ_1, γ_2 and γ_3 equal to 90^0, Eq. 8 simplifies to Eq. 10:

$$V_b = \frac{\beta}{J_v^3} \tag{10}$$

Often, it is not possible to observe the whole individual block on an outcrop or on the surface of an underground opening, specially where less than three joint sets occur. Random joints or cracks formed during the excavation process often result in defined blocks. In such cases, according to Palmstrom (2000a), a spacing of random joints say 5 to 10 times the spacing of the main set can be used to estimate block volume.

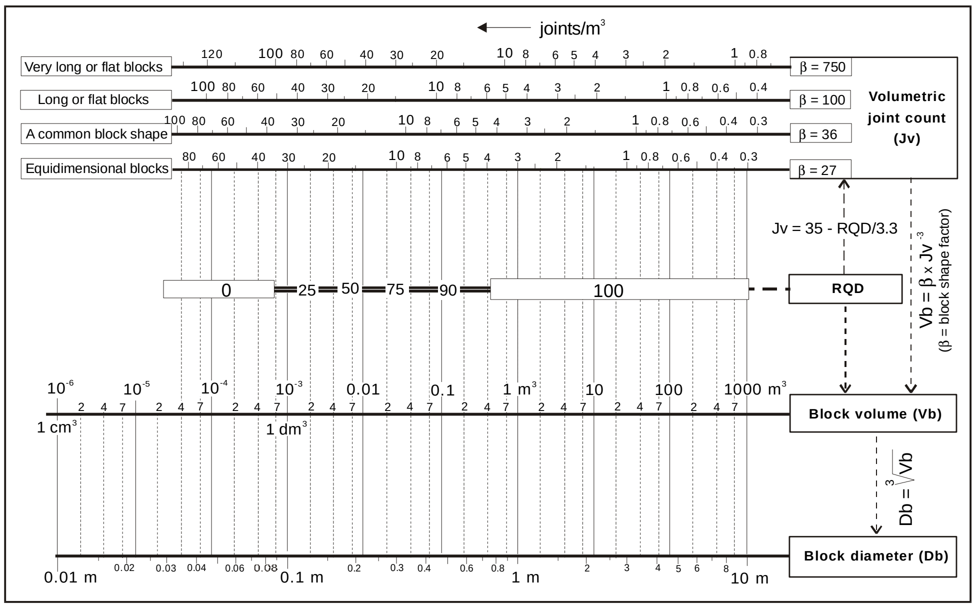

Fig. 5 – Connections between block size, block diameter and other jointing measurements (Palmstrom, 2000a)

According to Palmstrom (2000b), it is difficult to define the β-value for the various types of blocks such as flat, long, very flat, very long etc., and it has not been intended that the block shape factor should be measured accurately.

From the experience gained so far in the tunnelling, it is easy to identify number of joint sets and their spacing/frequency rather than block shapes at a particular site. From these two parameters (number of joint sets and spacings/frequencies), volumetric joint count can be easily computed which is essential to compute V_b. The following observation and suggestions are made in this regard:

(a) Average value of β for only one joint set may be taken as 180 as considered by Palmstrom (1996) (β = 150 to 200, flat blocks). With β = 180, spacing of random joints or cracks formed during excavation process comes out approximately as 13 (i.e. $\sqrt{180}$) times the spacing of only joint set present instead of 5 to 10 times as mentioned subsequently by Palmstrom (2000a).

$$V_b \approx S_l \text{ x } 13\, S_l \text{ x } 13\, S_l \approx 180\, S_l^3 \tag{11}$$

Using Eq. 9, for β as 180, $\alpha_2 = \alpha_3 \approx 10$. Therefore, it may be considered appropriate to use β-value as 180 for only one joint set.

Paradoxically, Palmstrom (2000a) considered β-value as 750 for 'very long or flat blocks', giving spacing of random joints more than 27 (i.e. $\sqrt{750}$), a very high figure. It may be designated as 'Exceptionally long or flat blocks (only one joint set, β = 750)' and one more category in between β = 750 and β = 100 may be included with the designation, 'very long or flat blocks (only one joint set, β = 180)'.

(b) Palmstrom (2000a) considered β-value as 100 for 'long or flat blocks'. It may be considered corresponding to two joint sets as suggested by Palmstrom (1996) (β = 75 to 100, long and flat blocks). In this case also, spacing of random joints or cracks formed during excavation process comes out as 13 times the spacing of the main set (joint set with closer spacing).

$$V_b \approx S_l \text{ x } S_2 \text{ x}13\, S_l = 13\, S_l^2 \text{ x } S_2 \tag{12}$$

By putting α_2 as 1 and α_3 as 10 in Eq. 9, β comes out as 93. Therefore, the value of β as 100 for two joint sets is appropriate.

(c) Palmstrom (2000a) considered β-value as 36 for 'A common block shape'. It may be considered corresponding to two plus random joint sets, close to three joint sets as β is only marginally higher than that for three or more joint sets. Earlier, Palmstrom (1996) suggested β = 50 to 80 for two - three joint sets, which seems to be somewhat better initially than the current β-value of 36. Nevertheless, the change of designation of block shape as 'A common block shape' from the earlier one of 'long block' might have justified the changes in β-values from 50 - 80 to current 36. In the absence of frequency/spacing of random joint for estimation of J_v, it has been found that for β = 36, a spacing of 2.6 times the spacing of the main set can be considered.

$$V_b \approx S_l \text{ x } S_2 \text{ x } 2.6\, S_l \approx 2.6\, S_l^2\, S_2 \tag{13}$$

By putting α_2 as 1 and α_3 as 2.6 in Eq. 9, β comes out as 35. Therefore, the value of β as 36 for two plus random joint sets is quite appropriate.

(d) 'Equidimensional blocks' (β = 27) considered by Palmstrom (2000a) corresponds to the three or more joint sets. In this situation rock blocks are clearly defined by the intersection of three joint sets and the relationship between J_v and V_b yields a value of β as 27. From Eq. 9 also, by putting $\alpha_2 = \alpha_3 = 1$, β comes out as 27.

Considering above, values of shape factor β for different shapes of blocks and number of joint sets may be obtained as given in Table 1 for the estimation of block volume.

Table 1 – Value of shape factor β for different shapes of block

S. No.	Shape of blocks	Number of joint sets	Value of β
1.	Exceptionally long or flat blocks	One	750
2.	Very long or flat blocks	One	180
3.	Long or flat blocks	Two	100
4.	A common block shape	Two plus random	36
5.	Equidimensional blocks	Three or more	27

Summarizing above, V_b may be estimated from one of the following methods (Table 2):

Table 2 – Methods to estimate block volume V_b for different joint set conditions

No. of Joint Sets	Method 1	Method 2
One Joint Set	$J_v = \frac{1}{S_1}$ $V_b = \frac{180}{J_v^3}$	$V_b = S_1 \text{ x } 13\ S_1 \text{ x } 13\ S_1$ $\approx 180\ S_1^3$
Two Joint Sets	$J_v = \frac{1}{S_1} + \frac{1}{S_2}$ $V_b = \frac{100}{J_v^3}$	$V_b = S_1 \text{ x } S_2 \text{ x } 13\ S_1$ $= 13\ S_1^2 \text{ x } S_2$
Two Joint Sets Plus Random	$J_v = \frac{1}{S_1} + \frac{1}{S_2} + \frac{1}{2.6\,S_1}$ $V_b = \frac{36}{J_v^3}$	$V_b = S_1 \text{ x } S_2 \text{ x } 2.6\ S_1$ $= 2.6\ S_1^2 \text{ x } S_2$
Three or More Joint Sets	$J_v = \frac{1}{S_1} + \frac{1}{S_2} + \frac{1}{S_3} + ...$ $V_b = \frac{27}{J_v^3}$	$V_b = S_1 \text{ x } S_2 \text{ x } S_3$

5. CONCLUSIONS

RMi differs from the existing classification systems for support design. While other prevailing methods combine all the selected parameters to directly arrive at a quality or rating for the ground conditions, RMi method applies an index (RMi) to characterize the rock mass. The index is then applied as input to determine the ground quality. The division of ground into blocky and continuous materials and the size ratio (tunnel size/block size) are also new features of the RMi support method.

Emphasis has been given to relate the block shape factor with the number of joint sets in the rock mass and their values have also been suggested. Although the RMi system has ignored the influence of groundwater in stress level, yet in the extreme conditions, it has suggested modification in stress level factors for two categories of groundwater influences only. Accordingly, further refinements have been proposed in the paper.

Approach suggested for using support charts for different ground conditions and method of estimation of block volume would be useful for reliable application of RMi system.

References

Aydan, O., Akagi, T. and Kawamoto, T. (1993). The squeezing potential of rocks around tunnels; theory and prediction, Jr. Rock Mechanics and Rock Engineering, Springer - Verlag, No.26, pp.137-163.

Barton, N. and Grimstad, E. (1993). Updating the Q-system for NMT, Proc. Int. Symp. on Sprayed Concrete, Norway, Oslo, 20p.

Bieniawski, Z.T. (1984). Rock mechanics design in mining and tunneling, Balkema, Rotterdam, 272p.

Hoek, E. and Brown, E.T. (1980). Empirical strength criterion for rock masses, ASCE, Geotech. Engineering, Vol.106, No.GT9, pp.1013-1035.

Hoek, E., Wood, D. and Shah, S. (1992). A modified Hoek–Brown failure criteria for jointed rock masses, Int. Conf. Eurock'92, London, pp.209-214.

Kumar, Naresh (2002). Rock mass characterization and evaluation of supports for tunnels in Himalaya, Ph.D. Thesis, Water Resources Development Training Centre, Indian Institute of Technology Roorkee, Roorkee, India, 295p.

Palmstrom, A. (1995a). RMi - a rock mass characterization system for rock engineering purposes, Ph.D. Thesis, University of Oslo, Norway, 400p.

Palmstrom, A. (1995b). Characterizing the strength of rock masses for use in design of underground structures, Int. Conf. on Design and Construction of Underground Structures, New Delhi, pp.43-52.

Palmstrom, A. (1995c). RMi – a system of characterizing rock mass strength for use in rock engineering, J. Rock Mechanics and Tunnelling Technology, India, Vol.1, No.2, pp.69-108.

Palmstrom, A. (1996). The rock mass index applied in rock mechanics and rock engineering, J. Rock Mechanics and Tunnelling Technology, India, Vol.2, No.1, pp.1-40.

Palmstrom, A. (2000a). Recent developments in rock support estimates by the RMi, J. Rock Mechanics and Tunnelling Technology, India,Vol.6, No.1, pp.1-24.

Palmstrom, A. (2000b). Personal communication with the first author, dated 5th October 2000.