IS 13365-1 (1998): Quantitative classification system of rock mass - Guidelines, Part 1: RMR for predicting of engineering properties [CED 48: Rock Mechanics]

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IS 13365 ( Part 1 ) : 1998
Reaffirmed 2010

*भारतीय मानक*

# शैल संहति मात्रात्मक वर्गीकरण तंत्र — मार्गदर्शी सिद्धांत

## भाग 1 इंजीनियरी गुणधर्मों के निर्धारण के लिए शैल संहति रेटिंग (आर एम आर)

*Indian Standard*

# QUANTITATIVE CLASSIFICATION SYSTEMS OF ROCK MASS — GUIDELINES

## PART 1 ROCK MASS RATING *(RMR)* FOR PREDICTING ENGINEERING PROPERTIES

ICS 93.020



BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

*July* 1998 Price Group 6

**AMENDMENT NO. 1 OCTOBER 2008**
**TO**
**IS 13365 (PART 1) : 1998 QUANTITATIVE CLASSIFICATION SYSTEMS OF ROCK MASS — GUIDELINES**

**PART 1 ROCK MASS RATING (*RMR*) FOR PREDICTING ENGINEERING PROPERTIES**

(*Page* 4, *Fig.* 1) — Substitute the following for the existing figure:

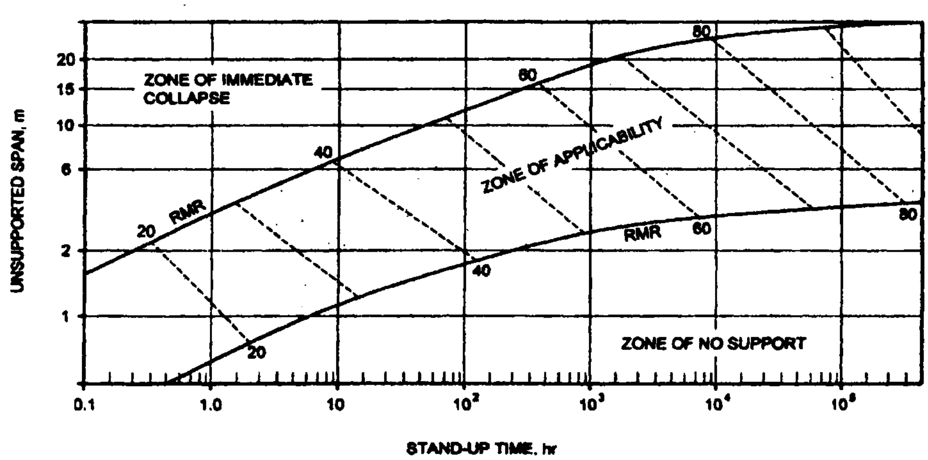


FIG. 1 STAND-UP TIME V/S UNSUPPORTED SPAN AS PER ROCK MASS RATING

(CED 48)

Reprography Unit, BIS, New Delhi, India

Rock Mechanics Sectional Committee, CED 48

## FOREWORD

This Indian Standard (Part 1) was adopted by the Bureau of Indian Standards, after the draft finalized by the Rock Mechanics Sectional Committee had been approved by the Civil Engineering Division Council.

Quantitative classification of rock masses has many advantages. It provides a basis for understanding characteristics of different groups. It also provides a common basis for communication besides yielding quantitative data for designs for feasibility studies of project. This is the reason why quantitative classifications have become very popular all over the world.

Rigorous approaches of designs based on various parameters could lead to uncertain results because of uncertainities in obtaining the correct value of input parameters at a given site of tunnelling. Rock mass classifications which do not involve uncertain parameters are following the philosophy of reducing uncertainities. Part 2 of this standard presents Quantitative Classification System, and Part 3 offers details of Slope Mass Rating.

Technical Committee responsible for the formulation of this standard is given in Annex D.

In reporting the result of a test or analysis made in accordance with this standard, if the final value, observed or calculated, is to be rounded off, it shall be done in accordance with IS 2 : 1960 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

*Indian Standard*

# QUANTITATIVE CLASSIFICATION SYSTEMS OF ROCK MASS — GUIDELINES

## PART 1 ROCK MASS RATING (*RMR*) FOR PREDICTING ENGINEERING PROPERTIES

### 1 SCOPE

This standard (Part 1) covers the procedure for determining the class of rock mass based on geomechanics classification system which is also called the Rock Mass Rating (*RMR*) system. The classification can be used for estimating the unsupported span, the stand-up time or bridge action period and the support pressures of an underground opening. It can also be used for selecting a method of excavation and permanent support system. Further, cohesion, angle of internal friction and elastic modulus of the rock mass can be estimated. In its modified form *RMR* can also be used for predicting the ground conditions for tunnelling.

It is emphasized that recommended correlations should be used for feasibility studies and preliminary designs only. *In-situ* tests are essential for final design of important structures.

### 2 REFERENCES

The Indian Standards given in Annex A contain provisions which through reference in this text, constitute provision of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standard indicated.

### 3 PROCEDURE

To apply the geomechanics classification system, a given site should be divided into a number of geological structural units in such a way that each type of rock mass present in the area is covered. The following geological parameters are determined for each structural unit:

a) Uniaxial compressive strength of intact rock material (IS 8764),
b) Rock quality designation [IS 11315 (Part 11)],
c) Spacing of discontinuities [IS 11315 (Part 2)],
d) Condition of discontinuities [IS 11315 (Part 4)],
e) Ground water condition [IS 11315 (Part 8)], and
f) Orientation of discontinuities [IS 11315 (Part 1)].

**3.1 Collection of Field Data**

Various geological and other parameters given in **3.1.1** to **3.1.6** should be collected and recorded in data sheet shown in Annex B.

**3.1.1** *Uniaxial Compressive Strength of Intact Rock Material* ($q_c$)

The strength of the intact rock material should be obtained from rock cores in accordance with IS 9143 or IS 8764 or IS 10785 as applicable based on site conditions. The ratings based on uniaxial compressive strength and point load strength are given in Annex B (Item I). However the use of uniaxial compressive strength is preferred over that of point load index strength.

**3.1.2** *Rock Quality Designation (RQD)*

Rock quality designation (*RQD*) should be determined as specified in IS 11315 (Part 11). The details of rating are given in Annex B (Item II).

Where the rock cores are not available, *RQD* can be determined with the help of following formula:

$$RQD = 115 - 3.3\,J_v$$
$$= 100 \text{ for } J_v < 4.5$$

where

$J_v$ = number of joints per metre cube.

Minimum value of *RQD* is taken as 10 even if it is zero.

**3.1.3** *Spacing of Discontinuities*

The term discontinuity covers joints, beddings or foliations, shear zones, minor faults, or other surfaces of weakness. The linear distance between two adjacent discontinuities should be measured for all sets of discontinuities. The details of ratings are given in Annex B (Item III).

**3.1.4** *Condition of Discontinuities*

This parameter includes roughness of discontinuity surfaces, their separation, length or continuity, weathering of the wall rock or the planes of weakness, and infilling (gauge) material. The details of rating are given in Annex B (Item IV).The description of the term used in the classification is given in IS 11315 (Part 4) and IS 11315 (Part 5).

3.1.5 *Ground Water Condition*

In the case of tunnels, the rate of inflow of ground water in litre per minute per 10 m length of the tunnel should be determined, or a general condition can be described as completely dry, damp, wet, dripping, and flowing. If actual water pressure data are available, these should be stated and expressed in terms of the ratio of the water pressure to the major principal stress. The latter should be either measured from the depth below the surface (vertical stress increases with depth at 0.27 kg/cm$^2$ per metre of the depth below surface). The details are given in Annex B (Item V).

Rating of above five parameters (*see* **3.1.1** to **3.1.5**) is added to obtain what is called the basic rock mass rating ($RMR_{basic}$).

3.1.6 *Orientation of Discontinuities*

Orientation of discontinuities means the strike and dip of discontinuities. The strike should be recorded with reference to magnetic north. The dip angle is the angle between the horizontal and the discontinuity plane taken in a direction in which the plane dips. The value of the dip and the strike should be recorded as shown in Annex B (Item VI) for each joint set of particular importance that are unfavourable to the structure. In addition the orientation of tunnel axis or slope face or foundation alignment should also be recorded.

The influence of the strike and the dip of the discontinuities is considered with respect to the orientation of tunnel axis or slope face or foundation alignment. To facilitate the decision whether the strike and dip are favourable or not, reference should be made to Annex C, Tables C1 and C2 which give assessment of joint favourability for tunnels and dams foundations respectively. Once favourability of critical discontinuity is known, adjustment for orientation of discontinuities is applied as per Item VII, Annex B in earlier obtained basic rock mass rating to obtain *RMR*.

## 4 ESTIMATION OF ROCK MASS RATING (*RMR*)

**4.1** The rock mass rating should be determined as an algebraic sum of ratings for all the parameters given in Items I to VI after adjustments for orientation of discontinuities given in item VII of Annex B. The sum of Items II to V is called Rock Condition Rating (*RCR*) which discounts the effect of compressive strength of intact rock material and orientation of joints. This is also called as the modified *RMR*.

**4.2** On the basis of *RMR* values for a given engineering structure, the rock mass should be classified as very good (rating 100-81), good (80-61), fair (60-41), poor (40-21) and very poor (<20) rock mass.

**4.3** *RCR* may also be obtained from *Q.SRF* value as follows:

$$RCR = 8\, l_n\,(Q.SRF)+30$$

*Q.SRF* has been named as rock mass number and denoted by *N*. By doing so, the uncertainities in obtaining correct rating of *SRF* is eliminated as explained below:

$$Q = (RQD/J_n)(J_r/J_a)(J_w/SRF)$$

or $$N = Q.SRF = (RQD/J_n)(J_r/J_a)J_w$$

It can be seen in above equation that *N* is free from *SRF*. *RQD*, $J_n$, $J_r$, $J_a$, and $J_w$ are parameters as defined in IS 13365 (Part 2).

**4.4** In the case of larger tunnels and caverns, *RMR* may be somewhat less than obtained from drifts. In drifts, one may miss intrusions of other rocks and joint sets.

**4.5** Separate *RMR* shall be obtained for different orientation of tunnels after taking into account the orientation of tunnel axis with respect to the critical joint set (Item VI, Annex B).

**4.6** Wherever possible, the undamaged face should be used to estimate the value of *RMR*, since the overall aim is to determine the properties of the undisturbed rock mass. Severe blast damage may be accounted for by increasing *RMR* and $RMR_{basic}$ by 10.

## 5 ENGINEERING PROPERTIES OF ROCK MASSES

**5.1** The engineering properties of rock masses can be obtained from this classification as given in Table 1 based on assumptions given in **5.1.1** to **5.1.3**. If the rock mass rating lies within a given range, the value of engineering properties may be interpreted between the recommended range of properties.

**5.1.1** *Average Stand-up Time*

The stand-up time depends upon effective span of the opening which is defined as size of the opening or the distance between tunnel face and the adjoining tunnel support, whichever is minimum (*see* Fig. 1). For arched openings the stand-up time would be significantly higher than that for flat roof openings. Controlled blasting will further increase the stand-up time as damage to the rock mass is decreased.

**5.1.2** *Cohesion and Angle of Internal Friction*

Assuming that a rock mass behaves as a Coulomb material, its shear strength will depend upon cohesion and angle of internal friction. Usually the strength parameters are different for peak failure and residual failure conditions.

The values of cohesion for dry rock masses of slopes are likely to be signficantly more.

For underground openings, the values of cohesion will still be higher (*see* **5.1.5** and **5.1.6**).

**5.1.3** *Modulus of Deformation*

There are three correlations for determining deformation modulus of rock mass.

**5.1.3.1** Figure 2 gives correlations between rock mass rating (*RMR*) and modulus reduction factor (*MRF*), which defined as ratio of modulus of elasticity (*see* IS 9221) of rock core to elastic modulus of rock mass. Thus, modulus of deformation of rock mass be determined as product of modulus of elasticity of rock material ($E_r$) and modulus reduction factor corresponding to rock mass rating from the equation below (for hard jointed rock).

$$E_d = E_r.MRF$$

The correlation for *MRF* is shown in Fig. 2.

**5.1.3.2** There is an approximate correlation between modulus of deformation and rock mass rating for hard rock masses ($q_c \geq 50$ MPa).

$$E_d = 2 \times RMR - 100, \text{ in GPa}$$

or

$$E_d = 10^{(RMR-10)/40}, \text{ in GPa (for all values of } RMR)$$

These correlations are shown in Fig. 3.

For dry soft rock masses ($q_c < 50$ MPa) modulus of deformation is dependent upon confining pressure due to overburden.

$$E_d = 0.3z^{\alpha}10^{(RMR-20)/38}, \text{ in GPa}$$

$\alpha$ = 0.16 to 0.30 (higher for poor rocks)

$z$ = depth of location under consideration below ground surface in metres (for depths $\geq 50$ m).

The modulus of deformation of poor rock masses with water sensitive minerals decreases significantly after saturation and with passage of time after excavation. For design of dam foundations, it is recommended that uniaxial jacking tests with bore hole extensometers, wherever feasible, should be conducted very carefully soon after the excavation of drifts particularly for poor rock masses in saturated condition.

**5.1.4** *Allowable Bearing Pressure*

Allowable bearing pressure is also related to *RMR* and may be estimated as per IS 12070.

**5.1.5** In stability analysis of rock slopes, strength parameters are needed in cases of rotational slides. The same may be obtained from *RMR* parameters which is sum of rating of Items I to IV of Annex B. The seepage condition should be considered in the analysis. The same strength parameters are also applicable in case of wedge sliding along discontinuous joint sets (*see* **5.1.6** and Table 2). However, it would be better if strength parameters are obtained from back analysis of distressed slopes in similar rock conditions near the site.

**5.1.6** *Shear Strength of Jointed Rock Masses*

The shear strength ($\tau$) for poor rock masses are given by:

$$\tau = A(\sigma + T)^B$$
$$= 0 \text{ if } \sigma < 0$$

where

constants $A$, $T$ and $B$ are given in Table 2 both for dry and saturated conditions and Natural Moisture Content (nmc) also. It may be noted that shear strength decreases significantly after saturation. Block shear tests suggest that shear strength is independent of $q_c$ for poor rock masses ($RMR < 60$ and $Q < 10$). Further, much higher shear strength is likely to be mobilised in underground openings than that obtained from block shear tests or Table 2.

Block shear tests on saturated rock blocks should be conducted for design of concrete dams and stability of abutments. For hard and massive rock masses ($RMR > 60$), shear strength ($\tau$) is governed by (*see* the first row of Table 2):

$$\tau_n = A(\sigma_n + T)^B$$
$$= 0 \text{ if } \sigma_n < 0$$

where

$\tau_n$ = $\tau/q_c$

$\sigma_n$ = $\sigma/q_c$

$q_c$ = mean uniaxial compressive strength of intact rock material, and

$A, T, B$ are constants.

In case of underground openings, the increase in strength occurs due to limited freedom of fracture propagation in openings than that in block shear test. Another reason for strength enhancement is that the *in-situ* stress along the axis of tunnels and caverns prestresses rock wedges both in roof and walls. The mobilised uniaxial compressive strength of rock mass may be estimated from the following correlations for tunnels and caverns:

$$q_{c\,mass} = 70\ \gamma\ \dot{Q}^{1/3} \text{ in kg/cm}^2;\ Q \leq 10;\ J_w = 1;\ q_c < 100 \text{ MPa}$$

$$\tan\phi = J_r/J_a \leq 1.5$$

**Table 1 Engineering Properties of Rock Mass**
(*Clause* 5.1)

| Item | Rock Mass Rating | 100-81 | 80-61 | 60-41 | 40-21 | <20 |
|---|---|---|---|---|---|---|
| 1. | Class | I | II | III | IV | V |
| 2. | Classification of rock mass | Very good | Good | Fair | Poor | Very poor |
| 3. | Average stand-up time | 10 years for 15 m span | 6 months for 8 m span | 1 week for 5 m span | 10 h for 2.5 m span | 30 min for 1 m span |
| 4. | Cohesion of rock mass ($kg/cm^2$)[1)] | >4 | 3-4 | 2-3 | 1-2 | <1 |
| 5. | Angle of internal friction of rock mass[1)] | >45 | 35-45 | 25-35 | 15-25 | 15 |

[1)] Values are applicable for saturated rock masses in slopes.

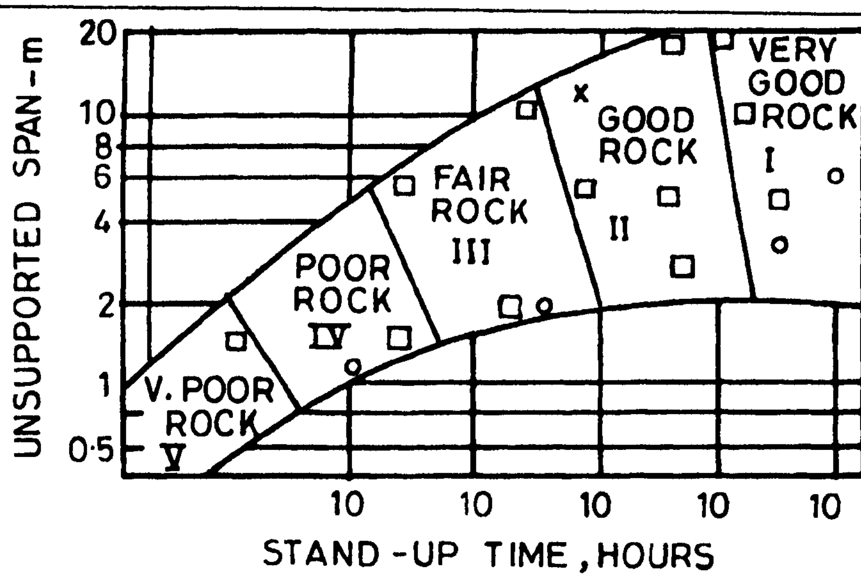


FIG. 1 GEOMECHANICS CLASSIFICATION OF ROCK MASSES IN TUNNELS

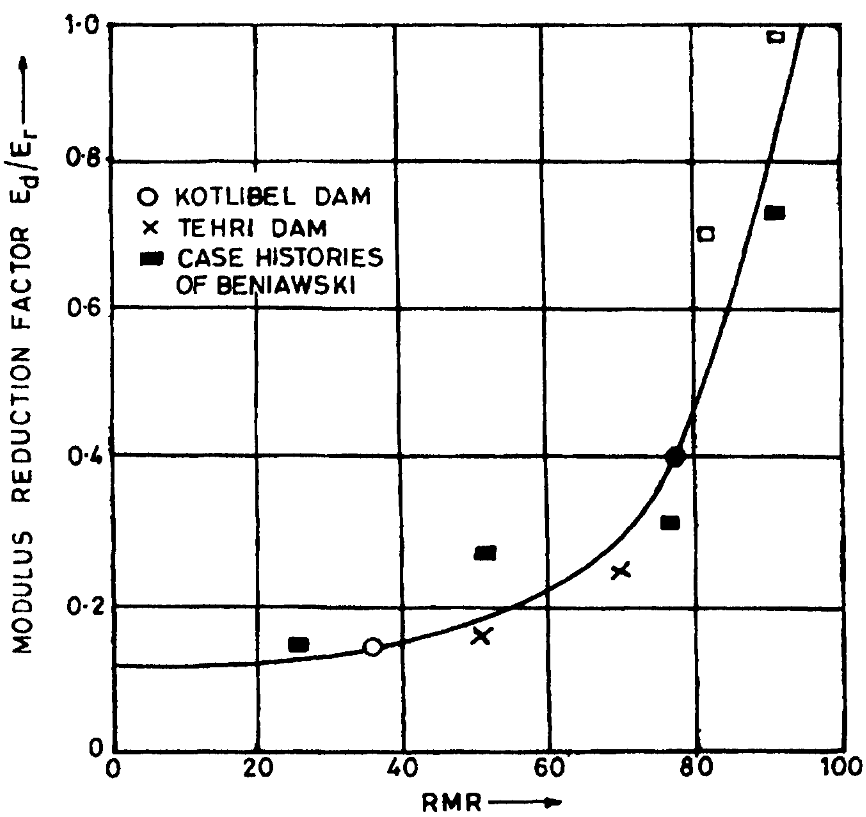


FIG. 2 RELATIONSHIP BETWEEN *RMR* AND MODULUS REDUCTION FACTOR

where

$\gamma$ = unit weight of rock mass in g/cc,

$Q$ = rock mass quality [IS 13365 (Part 2)],

$J_r$ = joint roughness number, and

$J_a$ = joint alteration number.

### 5.1.7 *Estimation of Support Pressure*

The short-term support pressures for arched underground openings in both squeezing and non-squeezing ground conditions may be estimated from the following empirical correlation in the case of tunnelling by conventional blasting method using steel rib supports:

$$P_{roof} = (7.5\, B^{0.1}\, H^{0.5} - RMR)/2RMR, \text{ in kg/cm}^2$$

where

$B$ = span of opening in metres,

$H$ = overburden or tunnel depth in metres (> 50 m), and

$P_{roof}$ = short-term roof support pressure in kg/cm$^2$.

The support pressures estimated from Q-system [IS 13365 (Part 2)] are more reliable if Stress Reduction Factor (*SRF*) is correctly obtained.

### 5.1.8 *Prediction of Tunnelling Conditions*

Ground conditions for tunnelling can be predicted by using the following correlations (*see* Fig. 4):

| *Sl No.* | *Ground Condition* | *Correlations* |
|---|---|---|
| i) | Self-supporting | $H < 23.4\, N^{0.88} . B^{-0.1}$ and $1\,000\, B^{-0.1}$ |
| ii) | Non-squeezing | $23.4\, N^{0.88}\, B^{-0.1} < H < 275\, N^{0.33}\, B^{-0.1}$ |
| iii) | Mild squeezing | $275\, N^{0.33}\, B^{-0.1} < H < 450\, N^{0.33}\, B^{-0.1}$ |
| iv) | Moderate squeezing | $450\, N^{0.33}\, B^{-0.1} < H < 630\, N^{0.33}\, B^{-0.1}$ |
| v) | High squeezing | $H > 630\, N^{0.33}\, B^{-0.1}$ |

In above correlations, $N$ is the rock mass number, as defined in 4.3. $H$ is the overburden in metres and $B$ is the tunnel width in metres.

## 6 PRECAUTIONS

It must be ensured that double accounting for parameters should not be done in analysis of rock structures and rating of rock mass. If pore water pressure is being considered in analysis of rock structures, it should not be accounted for in *RMR*. Similarly, if orientation of joint sets are considered in stability analysis of rock structures, the same should not be accounted for in *RMR*.

NOTE — For the purpose of eliminating doubts due to individual judgements, the rating for different parameters should be given a range in preference to a single value.

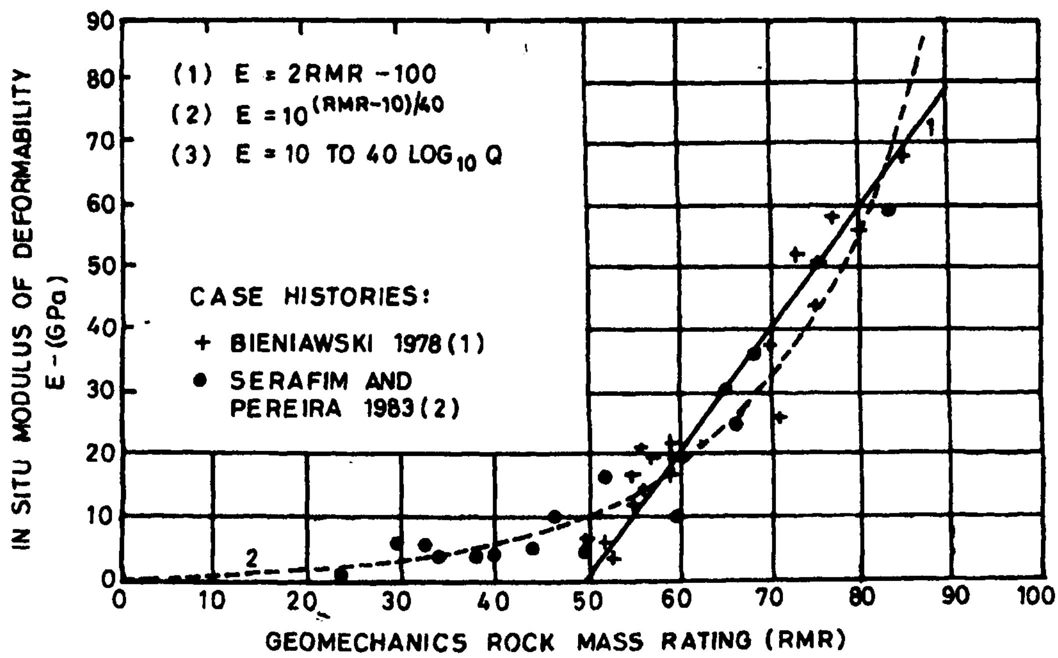


FIG. 3 CORRELATION BETWEEN THE *IN-SITU* MODULUS OF DEFORMATION AND THE GEOMECHANICS CLASSIFICATION [ROCK MASS RATING (*RMR*)] FOR HARD ROCKS (1 GPa = 10 000 kg/cm$^2$)

## Table 2 Recommended Mohr Envelopes for Jointed Rock Masses

( *Clause* 5.1.6)

$$\tau_n = \frac{\tau}{q_c},\ \sigma_n = \frac{\sigma}{q_c};\ \sigma \text{ in kg/cm}^2;\ \tau = 0 \text{ if } \sigma < 0$$

$S$ = degree of saturation [ average value of degree of saturation is shown by $S_{av}$]

= 1, for completely saturated rock mass

| Rock Type Quality | Limestone | Slate, Xenolith, Phyllite | Sandstone, Quartzite | Trap, Metabasic |
|---|---|---|---|---|
| Good Rock Mass<br>*RMR* = 61-80<br>$Q$ = 10-40 | $\tau_{n\,(nmc)} = 0.38\,(\sigma_n + 0.005)^{0.669}$<br>$\tau_{n\,(sat)} = 0.35\,(\sigma_n \div 0.004)^{0.669}$<br>[$S$=1] | $\tau_{n\,(nmc)} = 0.42\,(\sigma_n + 0.004)^{0.683}$<br>$\tau_{n\,(sat)} = 0.38\,(\sigma_n + 0.003)^{0.683}$<br>[$S$=1] | $\tau_{n\,(nmc)} = 0.44\,(\sigma_n + 0.003)^{0.695}$<br>$\tau_{n\,(sat)} = 0.43\,(\sigma_n + 0.002)^{0.695}$<br>[$S$=1] | $\tau_{n\,(nmc)} = 0.50\,(\sigma_n + 0.003)^{0.698}$ [$S_{av}$ = 0.30]<br>$\tau_{n\,(sat)} = 0.49\,(\sigma_n + 0.002)^{0.698}$<br>[$S$=1] |
| Fair Rock Mass<br>*RMR* = 41-60<br>$Q$ = 2-10 | $\tau_{(nmc)} = 2.60\ (\sigma + 1.25)^{0.662}$<br>$\tau_{(sat)} = 1.95\,(\sigma + 1.20)^{0.662}$<br>[$S$=1] | $\tau_{(nmc)} = 2.75(\sigma + 1.15)^{0.675}$<br>[$S_{av}$=0.25]<br>$\tau_{(sat)} = 2.15(\sigma + 1.10)^{0.675}$<br>[$S$=1] | $\tau_{(nmc)} = 2.85\ (\sigma + 1.10)^{0.688}$<br>[$S_{av}$=0.15]<br>$\tau_{(sat)} = 2.25\,(\sigma + 1.05)^{0.688}$<br>[$S$=1] | $\tau_{(nmc)} = 3.05(\sigma + 1.00)^{0.691}$<br>[$S_{av}$=0.35]<br>$\tau_{((sat)} = 2.45\,(\sigma + 0.95)^{0.691}$<br>[$S$=1] |
| Poor Rock Mass<br>*RMR* = 21-40<br>$Q$ = 0.5-2 | $\tau_{(nmc)} = 2.50\ (\sigma + 0.80)^{0.646}$<br>[$S_{av}$=0.20]<br>$\tau_{(sat)} = 1.50\ (\sigma + 0.75)^{0.646}$<br>[$S$=1] | $\tau_{(nmc)} = 2.65(\sigma{+}0.75)^{0.655}$<br>[$S_{av}$=0.40]<br>$\tau_{(sat)} = 1.75(\sigma{+}0.70)^{0.655}$<br>[$S$=1] | $\tau_{(nmc)} = 2.85\ (\sigma + 0.70)^{0.672}$<br>[$S_{av}$=0.25]<br>$\tau_{(sat)} = 2.00\,(\sigma + 0.65)^{0.672}$<br>[$S$=1] | $\tau_{((nmc)} = 3.00\ (\sigma + 0.65)^{0.676}$<br>[$S_{av}$=0.15]<br>$\tau_{((sat)} = 2.25(\sigma + 0.50)^{0.676}$<br>[$S$=1] |
| Very Poor Rock Mass<br>*RMR* < 21<br>$Q$ = < 0.5 | $\tau_{(nmc)} = 2.25\ (\sigma{+}0.65)^{0.534}$<br>$\tau_{(sat)} = 0.80(\sigma)^{0.534}$<br>[$S$= 1] | $\tau_{(nmc)} = 2.45\ (\sigma{+}0.60)^{0.539}$<br>$\tau_{(sat)} = 0.95(\sigma)^{0.539}$<br>[$S$= 1] | $\tau_{((nmc)} = 2.65\,(\sigma + 0.55)^{0.546}$<br>$\tau_{(sat)} = 1.05(\sigma)^{0.546}$<br>[$S$= 1] | $\tau_{((nmc)} = 2.90\,(\sigma + 0.50)^{0.548}$<br>$\tau_{((sat)} = 1.25\,(\sigma)^{0.548}$<br>[$S$= 1] |

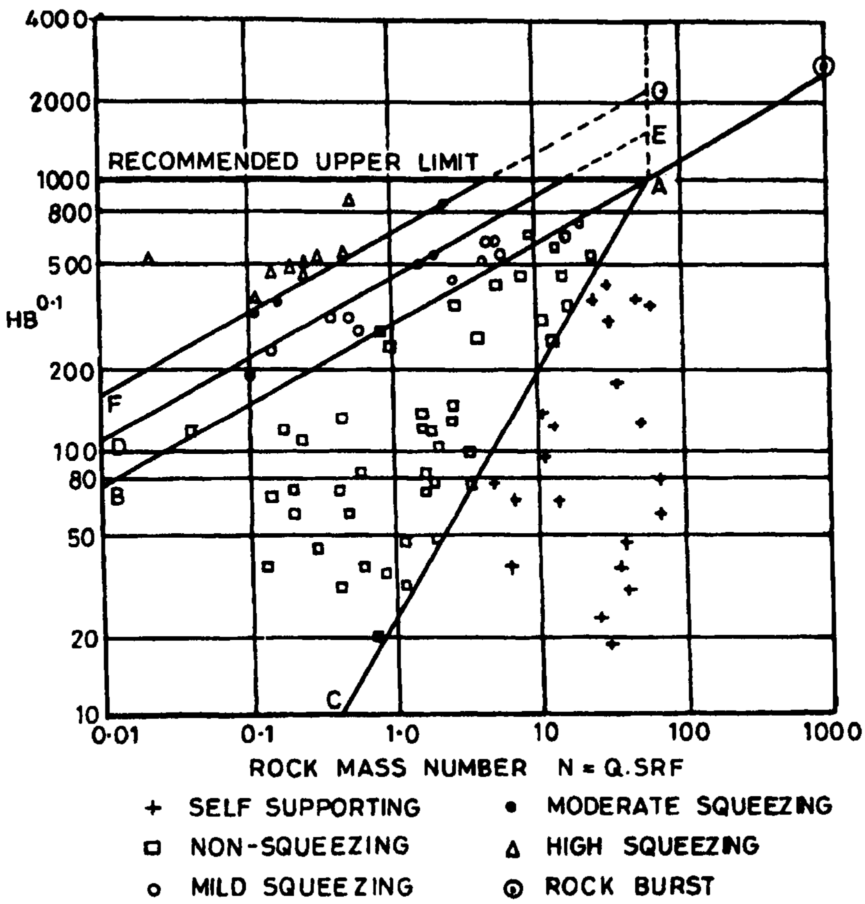


FIG. 4 CRITERIA FOR PREDICTING GROUND CONDITIONS USING ROCK MASS NUMBER, TUNNEL DEPTH AND TUNNEL WIDTH

# ANNEX A

( *Clause* 2 )

## LIST OF REFERRED INDIAN STANDARDS

| *IS No.* | *Title* |
|---|---|
| 8764 : 1978 | Method of determination of point load strength index of rocks |
| 9143 : 1979 | Method for the determination of unconfined compressive strength of rock materials |
| 9221 : 1979 | Method for the determination of modulus of elasticity and Poisson's ratio of rock materials in uniaxial compression |
| 11315 | Method for the quantitative description of discontinuities in rock mass : |
| (Part 1) : 1987 | Orientation |
| (Part 2) : 1987 | Spacing |
| (Part 3) : 1987 | Persistence |
| (Part 8) : 1987 | Seepage |
| (Part 11) : 1987 | Core recovery and rock quality |
| 12070 : 1987 | Code of practice for design and construction of shallow foundation on rock |
| 13365 (Part 2) : 1992 | Quantitative classification systems of rock mass—Guidelines: Part 2 Rock mass quality for prediction of support pressure in underground openings |

# ANNEX B

( *Clauses* 3.1, 4.1 *and* 5.1.5 )

## DATA SHEET FOR GEOMECHANICAL CLASSIFICATION OF ROCK MASSES (*RMR*)

Name of project .................................... Location of site .......................................................
Survey conducted by ................................ Date.....................................................................
Type of rock mass unit ............................ Origin of rock mass ..................................................

The appropriate rating may be encircled as per site conditions.

### I STRENGTH OF INTACT ROCK MATERIAL (MPa)

| | *Compressive Strength* | *Point Load Strength* | *Rating* |
|---|---|---|---|
| Exceptionally strong | >250 | >8 | 15 |
| Very strong | 100-250 | 4-8 | 12 |
| Strong | 50-100 | 2-4 | 7 |
| Average | 25-50 | 1-2 | 4 |
| Weak | 10-25 | Use of uniaxial compressive | 2 |
| Very weak | 2-10 | strength is preferred | 1 |
| Extremely weak | <2 | | 0 |

### II ROCK QUALITY DESIGNATION (*RQD*)

| | *RQD* | *Rating* |
|---|---|---|
| Excellent | 90-100 | 20 |
| Good | 75-90 | 17 |
| Fair | 50-75 | 13 |
| Poor | 25-50 | 8 |
| Very poor | < 25 | 3 |

### III SPACING OF DISCONTINUITIES

| | *Spacing*, m | *Rating* |
|---|---|---|
| Very wide | > 2 | 20 |
| Wide | 0.6-2 | 15 |
| Moderate | 0.2-0.6 | 10 |
| Close | 0.06-0.2 | 8 |
| Very close | < 0.06 | 5 |

NOTE — If more than one set of discontinuity is present and the spacing of discontinuities of each set varies, consider the set with lowest rating.

### IV CONDITION OF DISCONTINUITIES

| | | | | | |
|---|---|---|---|---|---|
| | Very rough and unweathered wall rock, tight and discontinuous, no separation | Rough and slightly weathered wall rock surface, separation <1 mm | Slightly rough and moderately to highly weathered wall rock surface, separation <1 mm | Slickensided wall rock surface or 1-5 mm thick gauge or 1-5 mm wide opening, continuous discontinuity | 5 mm thick soft gauge 5 mm wide continuous discontinuity |
| **Rating** | 30 | 25 | 20 | 10 | 0 |

### V GROUND WATER CONDITION

| | | | | | |
|---|---|---|---|---|---|
| Inflow per 10 m tunnel length, (litre/min) | none | <10 | 10-25 | 25-125 | >125 |
| Joint water pressure/major principal stress | 0 | 0-0.1 | 0.1-0.2 | 0.2-0.5 | >0.5 |
| General description | Completely dry | Damp | Wet | Dripping | Flowing |
| **Rating** | 15 | 10 | 7 | 4 | 0 |

### VI ORIENTATION OF DISCONTINUITIES

Orientation of tunnel/slope/foundation axis ...........

| | | |
|---|---|---|
| Set-1 | Average strike...................(from..........to..........) | Dip............. |
| Set-2 | Average strike...................(from..........to..........) | Dip............. |
| Set-3 | Average strike...................(from..........to..........) | Dip............. |

**VII ADJUSTMENT FOR JOINT ORIENTATION** (*see* **Annex C)**

| *Strike and dip orientation of joints for* | *Very Favourable* | *Favourable* | *Fair* | *Un-Favourable* | *Very Unfavourable* |
|---|---|---|---|---|---|
| Tunnels | 0 | –2 | –5 | –10 | –12 |
| Raft foundation | 0 | –2 | –7 | –15 | –35 |
| Slopes | Use slope mass rating (*SMR*) as per IS 13365 (Part 3) | | | | |

**VIII ROCK MASS RATING (*RMR*)**

# ANNEX C

(*Clause* 3.1.6)

## ASSESSMENT OF JOINT FAVOURABILITY FOR TUNNELS AND DAMS FOUNDATIONS

**Table C1 Assessment of Joint Orientation Favourability in Tunnels (Dips are Apparent Dips Along Tunnel Axis)**

| Strike Perpendicular to Tunnel Axis | | | | Strike Parallel to Tunnel Axis | | Irrespective of Strike |
|---|---|---|---|---|---|---|
| *Drive with Dip* | | *Drive Against Dip* | | | | |
| Dip 45°-90° | Dip 20°-45° | Dip 45°-90° | Dip 20°-45° | Dip 20°-45° | Dip 45°-90° | Dip 0°- 20° |
| Very favourable | Favourable | Fair | Unfavourable | Fair | Very unfavourable | Fair |

**Table C2 Assessment of Joint Orientation Favourability for Stability of Raft Foundation**

| Dip | | | | |
|---|---|---|---|---|
| 0°-10° | 10°-30° | | 30°-60° | 60°-90° |
| | Dip Direction | | | |
| | Upstream | Downstream | | |
| Very favourable | Unfavourable | Fair | Favourable | Very unfavourable |

# ANNEX D

## ( *Foreword* )

## COMMITEE COMPOSITION

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