# 3

# Rock mass classification

## 3.1 Introduction

During the feasibility and preliminary design stages of a project, when very little detailed information on the rock mass and its stress and hydrologic characteristics is available, the use of a rock mass classification scheme can be of considerable benefit. At its simplest, this may involve using the classification scheme as a check-list to ensure that all relevant information has been considered. At the other end of the spectrum, one or more rock mass classification schemes can be used to build up a picture of the composition and characteristics of a rock mass to provide initial estimates of support requirements, and to provide estimates of the strength and deformation properties of the rock mass.

It is important to understand that the use of a rock mass classification scheme does not (and cannot) replace some of the more elaborate design procedures. However, the use of these design procedures requires access to relatively detailed information on in situ stresses, rock mass properties and planned excavation sequence, none of which may be available at an early stage in the project. As this information becomes available, the use of the rock mass classification schemes should be updated and used in conjunction with site specific analyses.

## 3.2 Engineering rock mass classification

Rock mass classification schemes have been developing for over 100 years since Ritter (1879) attempted to formalise an empirical approach to tunnel design, in particular for determining support requirements. While the classification schemes are appropriate for their original application, especially if used within the bounds of the case histories from which they were developed, considerable caution must be exercised in applying rock mass classifications to other rock engineering problems.

Summaries of some important classification systems are presented in this chapter, and although every attempt has been made to present all of the pertinent data from the original texts, there are numerous notes and comments which cannot be included. The interested reader should make every effort to read the cited references for a full appreciation of the use, applicability and limitations of each system.

Most of the multi-parameter classification schemes (Wickham et al (1972) Bieniawski (1973, 1989) and Barton et al (1974)) were developed from civil engineering case histories in which all of the components of the engineering geological character of the rock mass were included. In underground hard rock mining, however, especially at deep levels, rock mass weathering and the influence of water usually are not significant and may be ignored. Different classification systems place different emphases on the various parameters, and it is recommended that at least two methods be used at any site during the early stages of a project.

### *3.2.1 Terzaghi's rock mass classification*

The earliest reference to the use of rock mass classification for the design of tunnel support is in a paper by Terzaghi (1946) in which the rock loads, carried by steel sets, are estimated on the basis of a descriptive classification. While no useful purpose would be served by including details of Terzaghi's classification in this discussion on the design of support, it is interesting to examine the rock mass descriptions included in his original paper, because he draws attention to those characteristics that dominate rock mass behaviour, particularly in situations where gravity constitutes the dominant driving force. The clear and concise definitions and the practical comments included in these descriptions are good examples of the type of engineering geology information, which is most useful for engineering design.

Terzaghi's descriptions (quoted directly from his paper) are:

- *Intact* rock contains neither joints nor hair cracks. Hence, if it breaks, it breaks across sound rock. On account of the injury to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as a *spalling* condition. Hard, intact rock may also be encountered in the *popping* condition involving the spontaneous and violent detachment of rock slabs from the sides or roof.
- *Stratified* rock consists of individual strata with little or no resistance against separation along the boundaries between the strata. The strata may or may not be weakened by transverse joints. In such rock the spalling condition is quite common.
- *Moderately jointed* rock contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. In rocks of this type, both spalling and popping conditions may be encountered.
- *Blocky and seamy* rock consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, vertical walls may require lateral support.
- *Crushed* but chemically intact rock has the character of crusher run. If most or all of the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.
- *Squeezing* rock slowly advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or clay minerals with a low swelling capacity.
- *Swelling* rock advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks that contain clay minerals such as montmorillonite, with a high swelling capacity.

### *3.2.2 Classifications involving stand-up time*

Lauffer (1958) proposed that the stand-up time for an unsupported span is related to the quality of the rock mass in which the span is excavated. In a tunnel, the unsupported span is defined as the span of the tunnel or the distance between the face and the nearest support, if this is greater than the tunnel span. Lauffer's original classification has since been modified by a number of authors, notably Pacher et al (1974), and now forms part of the general tunnelling approach known as the New Austrian Tunnelling Method.

The significance of the stand-up time concept is that an increase in the span of the tunnel leads to a significant reduction in the time available for the installation of support. For example, a small pilot tunnel may be successfully constructed with minimal support, while a larger span tunnel in the same rock mass may not be stable without the immediate installation of substantial support.

The New Austrian Tunnelling Method includes a number of techniques for safe tunnelling in rock conditions in which the stand-up time is limited before failure occurs. These techniques include the use of smaller headings and benching or the use of multiple drifts to form a reinforced ring inside which the bulk of the tunnel can be excavated. These techniques are applicable in soft rocks such as shales, phyllites and mudstones in which the squeezing and swelling problems, described by Terzaghi (see previous section), are likely to occur. The techniques are also applicable when tunnelling in excessively broken rock, but great care should be taken in attempting to apply these techniques to excavations in hard rocks in which different failure mechanisms occur.

In designing support for hard rock excavations it is prudent to assume that the stability of the rock mass surrounding the excavation is not time-dependent. Hence, if a structurally defined wedge is exposed in the roof of an excavation, it will fall as soon as the rock supporting it is removed. This can occur at the time of the blast or during the subsequent scaling operation. If it is required to keep such a wedge in place, or to enhance the margin of safety, it is essential that the support be installed as early as possible, preferably before the rock supporting the full wedge is removed. On the other hand, in a highly stressed rock, failure will generally be induced by some change in the stress field surrounding the excavation. The failure may occur gradually and manifest itself as spalling or slabbing or it may occur suddenly in the form of a rock burst. In either case, the support design must take into account the change in the stress field rather than the ‘stand-up’ time of the excavation.

### *3.2.3 Rock quality designation index (RQD)*

The Rock Quality Designation index (*RQD*) was developed by Deere (Deere et al 1967) to provide a quantitative estimate of rock mass quality from drill core logs. *RQD* is defined as the percentage of intact core pieces longer than 100 mm (4 inches) in the total length of core. The core should be at least NW size (54.7 mm or 2.15 inches in diameter) and should be drilled with a double-tube core barrel. The correct procedures for measurement of the length of core pieces and the calculation of *RQD* are summarised in Figure 4.1.

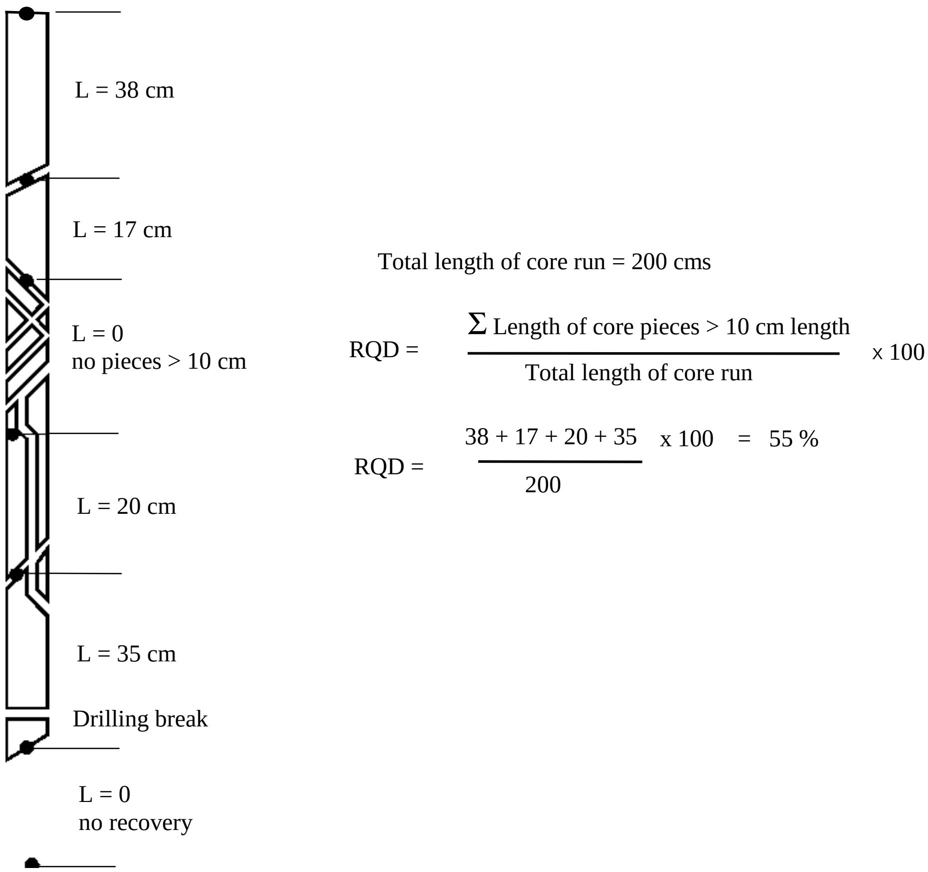



Figure 4.1: Procedure for measurement and calculation of *RQD* (After Deere, 1989).

Palmström (1982) suggested that, when no core is available but discontinuity traces are visible in surface exposures or exploration adits, the *RQD* may be estimated from the number of discontinuities per unit volume. The suggested relationship for clay-free rock masses is:

$$RQD = 115 - 3.3\, J_v \tag{4.1}$$

where $J_v$ is the sum of the number of joints per unit length for all joint (discontinuity) sets known as the volumetric joint count.

*RQD* is a directionally dependent parameter and its value may change significantly, depending upon the borehole orientation. The use of the volumetric joint count can be quite useful in reducing this directional dependence.

*RQD* is intended to represent the rock mass quality in situ. When using diamond drill core, care must be taken to ensure that fractures, which have been caused by handling or the drilling process, are identified and ignored when determining the value of *RQD*. When using Palmström's relationship for exposure mapping, blast induced fractures should not be included when estimating $J_v$.

Deere's *RQD* has been widely used, particularly in North America, for the past 25 years. Cording and Deere (1972), Merritt (1972) and Deere and Deere (1988) have attempted to relate *RQD* to Terzaghi's rock load factors and to rockbolt requirements in tunnels. In the context of this discussion, the most important use of *RQD* is as a component of the *RMR* and *Q* rock mass classifications covered later in this chapter.

### *3.2.4 Rock Structure Rating (RSR)*

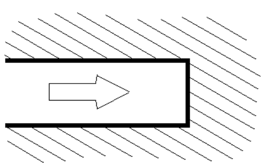
Wickham et al (1972) described a quantitative method for describing the quality of a rock mass and for selecting appropriate support on the basis of their Rock Structure Rating (*RSR*) classification. Most of the case histories, used in the development of this system, were for relatively small tunnels supported by means of steel sets, although historically this system was the first to make reference to shotcrete support. In spite of this limitation, it is worth examining the *RSR* system in some detail since it demonstrates the logic involved in developing a quasi-quantitative rock mass classification system.

The significance of the *RSR* system, in the context of this discussion, is that it introduced the concept of rating each of the components listed below to arrive at a numerical value of $RSR = A + B + C$.

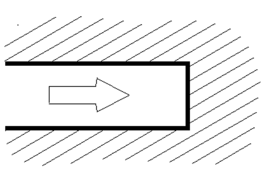
1. *Parameter A, Geology:* General appraisal of geological structure on the basis of:
   a. Rock type origin (igneous, metamorphic, sedimentary).
   b. Rock hardness (hard, medium, soft, decomposed).
   c. Geologic structure (massive, slightly faulted/folded, moderately faulted/folded, intensely faulted/folded).

2. *Parameter B, Geometry*: Effect of discontinuity pattern with respect to the direction of the tunnel drive on the basis of:
   a. Joint spacing.
   b. Joint orientation (strike and dip).
   c. Direction of tunnel drive.

3. *Parameter C*: Effect of groundwater inflow and joint condition on the basis of:
   a. Overall rock mass quality on the basis of A and B combined.
   b. Joint condition (good, fair, poor).
   c. Amount of water inflow (in gallons per minute per 1000 feet of tunnel).



Drive with dip



Drive against dip

Note that the *RSR* classification used Imperial units and that these units have been retained in this discussion.

Three tables from Wickham et al's 1972 paper are reproduced in Tables 4.1, 4.2 and 4.3. These tables can be used to evaluate the rating of each of these parameters to arrive at the *RSR* value (maximum *RSR* = 100).

For example, a hard metamorphic rock which is slightly folded or faulted has a rating of $A$ = 22 (from Table 4.1). The rock mass is moderately jointed, with joints striking perpendicular to the tunnel axis which is being driven east-west, and dipping at between 20° and 50°. Table 4.2 gives the rating for $B$ = 24 for driving with dip (defined in the margin sketch).

The value of $A + B = 46$ and this means that, for joints of fair condition (slightly weathered and altered) and a moderate water inflow of between 200 and 1,000 gallons per minute, Table 4.3 gives the rating for $C = 16$. Hence, the final value of the rock structure rating $RSR = A + B + C = 62$.

A typical set of prediction curves for a 24 foot diameter tunnel are given in Figure 4.2 which shows that, for the *RSR* value of 62 derived above, the predicted support would be 2 inches of shotcrete and 1 inch diameter rockbolts spaced at 5 foot centres. As indicated in the figure, steel sets would be spaced at more than 7 feet apart and would not be considered a practical solution for the support of this tunnel.

For the same size tunnel in a rock mass with *RSR* = 30, the support could be provided by 8 WF 31 steel sets (8 inch deep wide flange I section weighing 31 lb per foot) spaced 3 feet apart, or by 5 inches of shotcrete and 1 inch diameter rockbolts spaced at 2.5 feet centres. In this case it is probable that the steel set solution would be cheaper and more effective than the use of rockbolts and shotcrete.

Although the *RSR* classification system is not widely used today, Wickham et al's work played a significant role in the development of the classification schemes discussed in the remaining sections of this chapter.

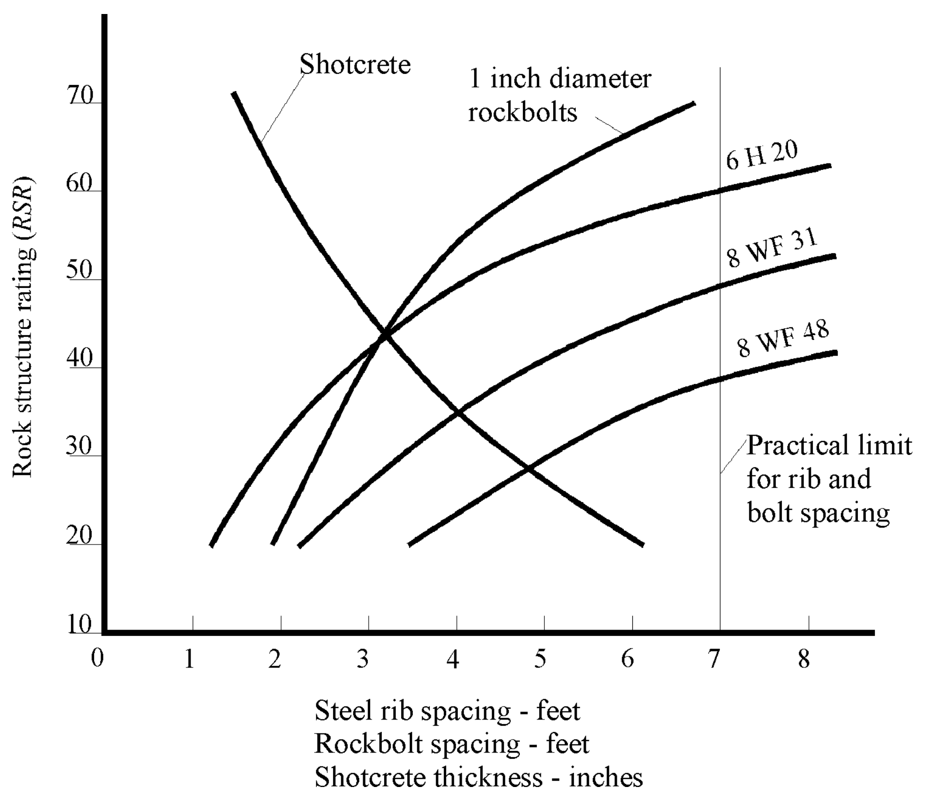



Figure 4.2: *RSR* support estimates for a 24 ft. (7.3 m) diameter circular tunnel. Note that rockbolts and shotcrete are generally used together. (After Wickham et al 1972).

Table 4.1: Rock Structure Rating: Parameter *A*: General area geology

| | Basic Rock Type | | | | Geological Structure | | | |
|---|---|---|---|---|---|---|---|---|
| | Hard | Medium | Soft | Decomposed | Massive | Slightly Folded or Faulted | Moderately Folded or Faulted | Intensively Folded or Faulted |
| Igneous | 1 | 2 | 3 | 4 | | | | |
| Metamorphic | 1 | 2 | 3 | 4 | | | | |
| Sedimentary | 2 | 3 | 4 | 4 | | | | |
| Type 1 | | | | | 30 | 22 | 15 | 9 |
| Type 2 | | | | | 27 | 20 | 13 | 8 |
| Type 3 | | | | | 24 | 18 | 12 | 7 |
| Type 4 | | | | | 19 | 15 | 10 | 6 |

Table 4.2: Rock Structure Rating: Parameter *B*: Joint pattern, direction of drive

| | Strike ⊥ to Axis | | | | | Strike \|\| to Axis | | |
|---|---|---|---|---|---|---|---|---|
| | Direction of Drive | | | | | Direction of Drive | | |
| | Both | With Dip | | Against Dip | | Either direction | | |
| | Dip of Prominent Joints [a] | | | | | Dip of Prominent Joints | | |
| Average joint spacing | Flat | Dipping | Vertical | Dipping | Vertical | Flat | Dipping | Vertical |
| 1. Very closely jointed, < 2 in | 9 | 11 | 13 | 10 | 12 | 9 | 9 | 7 |
| 2. Closely jointed, 2-6 in | 13 | 16 | 19 | 15 | 17 | 14 | 14 | 11 |
| 3. Moderately jointed, 6-12 in | 23 | 24 | 28 | 19 | 22 | 23 | 23 | 19 |
| 4. Moderate to blocky, 1-2 ft | 30 | 32 | 36 | 25 | 28 | 30 | 28 | 24 |
| 5. Blocky to massive, 2-4 ft | 36 | 38 | 40 | 33 | 35 | 36 | 24 | 28 |
| 6. Massive, > 4 ft | 40 | 43 | 45 | 37 | 40 | 40 | 38 | 34 |

Table 4.3: Rock Structure Rating: Parameter *C*: Groundwater, joint condition

| | Sum of Parameters *A* + *B* | | | | | |
|---|---|---|---|---|---|---|
| | 13 - 44 | | | 45 - 75 | | |
| Anticipated water inflow | Joint Condition [b] | | | | | |
| gpm/1000 ft of tunnel | Good | Fair | Poor | Good | Fair | Poor |
| None | 22 | 18 | 12 | 25 | 22 | 18 |
| Slight, < 200 gpm | 19 | 15 | 9 | 23 | 19 | 14 |
| Moderate, 200-1000 gpm | 15 | 22 | 7 | 21 | 16 | 12 |
| Heavy, > 1000 gp | 10 | 8 | 6 | 18 | 14 | 10 |

[a] Dip: flat: 0-20°; dipping: 20-50°; and vertical: 50-90°

[b] Joint condition: good = tight or cemented; fair = slightly weathered or altered; poor = severely weathered, altered or open

## 3.3 Geomechanics Classification

Bieniawski (1976) published the details of a rock mass classification called the Geomechanics Classification or the Rock Mass Rating (*RMR*) system. Over the years, this system has been successively refined as more case records have been examined and the reader should be aware that Bieniawski has made significant changes in the ratings assigned to different parameters. The discussion which follows is based upon the 1989 version of the classification (Bieniawski, 1989). Both this version and the 1976 version will be used in Chapter 8 which deals with estimating the strength of rock masses. The following six parameters are used to classify a rock mass using the *RMR* system:

1. Uniaxial compressive strength of rock material.
2. Rock Quality Designation (*RQD*).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities.

In applying this classification system, the rock mass is divided into a number of structural regions and each region is classified separately. The boundaries of the structural regions usually coincide with a major structural feature such as a fault or with a change in rock type. In some cases, significant changes in discontinuity spacing or characteristics, within the same rock type, may necessitate the division of the rock mass into a number of small structural regions.

The Rock Mass Rating system is presented in Table 4.4, giving the ratings for each of the six parameters listed above. These ratings are summed to give a value of *RMR*. The following example illustrates the use of these tables to arrive at an *RMR* value.

A tunnel is to be driven through a slightly weathered granite with a dominant joint set dipping at $60^{o}$ against the direction of the drive. Index testing and logging of diamond drilled core give typical Point-load strength index values of 8 MPa and average *RQD* values of 70%. The slightly rough and slightly weathered joints with a separation of < 1 mm, are spaced at 300 mm. Tunnelling conditions are anticipated to be wet.

The *RMR* value is determined as follows :

| *Table* | *Item* | *Value* | *Rating* |
|---|---|---|---|
| 4.4: A.1 | Point load index | 8 MPa | 12 |
| 4.4: A.2 | *RQD* | 70% | 13 |
| 4.4: A.3 | Spacing of discontinuities | 300 mm | 10 |
| 4.4: E.4 | Condition of discontinuities | Note 1 | 22 |
| 4.4: A.5 | Groundwater | Wet | 7 |
| 4.4: B | Adjustment for joint orientation | Note 2 | -5 |
| | | Total | 59 |

*Note 1*. For slightly rough and altered discontinuity surfaces with a separation of < 1 mm, Table 4.4.A.4 gives a rating of 25. When more detailed information is available, Table 4.4.E can be used to obtain a more refined rating. Hence, in this case, the rating is the sum of: 4 (1-3 m discontinuity length), 4 (separation 0.1-1.0 mm), 3 (slightly rough), 6 (no infilling) and 5 (slightly weathered) = 22.

*Note 2.* Table 4.4.F gives a description of 'Fair' for the conditions assumed where the tunnel is to be driven against the dip of a set of joints dipping at 60$^{o}$. Using this description for 'Tunnels and Mines' in Table 4.4.B gives an adjustment rating of -5.

Bieniawski (1989) published a set of guidelines for the selection of support in tunnels in rock for which the value of *RMR* has been determined. These guidelines are reproduced in Table 4.5. Note that these guidelines have been published for a 10 m span horseshoe shaped tunnel, constructed using drill and blast methods, in a rock mass subjected to a vertical stress < 25 MPa (equivalent to a depth below surface of <900 m).

For the case considered earlier, with *RMR* = 59, Table 4.5 suggests that a tunnel could be excavated by top heading and bench, with a 1.5 to 3 m advance in the top heading. Support should be installed after each blast and the support should be placed at a maximum distance of 10 m from the face. Systematic rock bolting, using 4 m long 20 mm diameter fully grouted bolts spaced at 1.5 to 2 m in the crown and walls, is recommended. Wire mesh, with 50 to 100 mm of shotcrete for the crown and 30 mm of shotcrete for the walls, is recommended.

The value of *RMR* of 59 indicates that the rock mass is on the boundary between the 'Fair rock' and 'Good rock' categories. In the initial stages of design and construction, it is advisable to utilise the support suggested for fair rock. If the construction is progressing well with no stability problems, and the support is performing very well, then it should be possible to gradually reduce the support requirements to those indicated for a good rock mass. In addition, if the excavation is required to be stable for a short amount of time, then it is advisable to try the less expensive and extensive support suggested for good rock. However, if the rock mass surrounding the excavation is expected to undergo large mining induced stress changes, then more substantial support appropriate for fair rock should be installed. This example indicates that a great deal of judgement is needed in the application of rock mass classification to support design.

It should be noted that Table 4.5 has not had a major revision since 1973. In many mining and civil engineering applications, steel fibre reinforced shotcrete may be considered in place of wire mesh and shotcrete.

## 3.4 Modifications to *RMR* for mining

Bieniawski's Rock Mass Rating (*RMR*) system was originally based upon case histories drawn from civil engineering. Consequently, the mining industry tended to regard the classification as somewhat conservative and several modifications have been proposed in order to make the classification more relevant to mining applications. A comprehensive summary of these modifications was compiled by Bieniawski (1989).

Laubscher (1977, 1984), Laubscher and Taylor (1976) and Laubscher and Page (1990) have described a Modified Rock Mass Rating system for mining. This *MRMR* system takes the basic *RMR* value, as defined by Bieniawski, and adjusts it to account for in situ and induced stresses, stress changes and the effects of blasting and weathering. A set of support recommendations is associated with the resulting *MRMR* value. In using Laubscher's *MRMR* system it should be borne in mind that many of the case histories upon which it is based are derived from caving operations. Originally, block caving in asbestos mines in Africa formed the basis for the modifications but, subsequently, other case histories from around the world have been added to the database.

Table 4.4: Rock Mass Rating System (After Bieniawski 1989).

**A. CLASSIFICATION PARAMETERS AND THEIR RATINGS**

| | Parameter | | Range of values | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Strength of intact rock material | Point-load strength index | >10 MPa | 4 - 10 MPa | 2 - 4 MPa | 1 - 2 MPa | For this low range - uniaxial compressive test is preferred | | |
| | | Uniaxial comp. strength | >250 MPa | 100 - 250 MPa | 50 - 100 MPa | 25 - 50 MPa | 5 - 25 MPa | 1 - 5 MPa | < 1 MPa |
| | Rating | | 15 | 12 | 7 | 4 | 2 | 1 | 0 |
| 2 | Drill core Quality *RQD* | | 90% - 100% | 75% - 90% | 50% - 75% | 25% - 50% | < 25% | | |
| | Rating | | 20 | 17 | 13 | 8 | 3 | | |
| 3 | Spacing of discontinuities | | > 2 m | 0.6 - 2 . m | 200 - 600 mm | 60 - 200 mm | < 60 mm | | |
| | Rating | | 20 | 15 | 10 | 8 | 5 | | |
| 4 | Condition of discontinuities (See E) | | Very rough surfaces Not continuous No separation Unweathered wall rock | Slightly rough surfaces Separation < 1 mm Slightly weathered walls | Slightly rough surfaces Separation < 1 mm Highly weathered walls | Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous | Soft gouge >5 mm thick or Separation > 5 mm Continuous | | |
| | Rating | | 30 | 25 | 20 | 10 | 0 | | |
| 5 | Ground water | Inflow per 10 m tunnel length (l/m) | None | < 10 | 10 - 25 | 25 - 125 | > 125 | | |
| | | (Joint water press)/ (Major principal $\sigma$) | 0 | < 0.1 | 0.1, - 0.2 | 0.2 - 0.5 | > 0.5 | | |
| | | General conditions | Completely dry | Damp | Wet | Dripping | Flowing | | |
| | Rating | | 15 | 10 | 7 | 4 | 0 | | |

**B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS** (See F)

| Strike and dip orientations | | Very favourable | Favourable | Fair | Unfavourable | Very Unfavourable |
|---|---|---|---|---|---|---|
| Ratings | Tunnels & mines | 0 | -2 | -5 | -10 | -12 |
| | Foundations | 0 | -2 | -7 | -15 | -25 |
| | Slopes | 0 | -5 | -25 | -50 | |

**C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS**

| | | | | | |
|---|---|---|---|---|---|
| Rating | 100 ← 81 | 80 ← 61 | 60 ← 41 | 40 ← 21 | < 21 |
| Class number | I | II | III | IV | V |
| Description | Very good rock | Good rock | Fair rock | Poor rock | Very poor rock |

**D. MEANING OF ROCK CLASSES**

| | | | | | |
|---|---|---|---|---|---|
| Class number | I | II | III | IV | V |
| Average stand-up time | 20 yrs for 15 m span | 1 year for 10 m span | 1 week for 5 m span | 10 hrs for 2.5 m span | 30 min for 1 m span |
| Cohesion of rock mass (kPa) | > 400 | 300 - 400 | 200 - 300 | 100 - 200 | < 100 |
| Friction angle of rock mass (deg) | > 45 | 35 - 45 | 25 - 35 | 15 - 25 | < 15 |

**E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions**

| | | | | | |
|---|---|---|---|---|---|
| Discontinuity length (persistence)<br>Rating | < 1 m<br>6 | 1 - 3 m<br>4 | 3 - 10 m<br>2 | 10 - 20 m<br>1 | > 20 m<br>0 |
| Separation (aperture)<br>Rating | None<br>6 | < 0.1 mm<br>5 | 0.1 - 1.0 mm<br>4 | 1 - 5 mm<br>1 | > 5 mm<br>0 |
| Roughness<br>Rating | Very rough<br>6 | Rough<br>5 | Slightly rough<br>3 | Smooth<br>1 | Slickensided<br>0 |
| Infilling (gouge)<br>Rating | None<br>6 | Hard filling < 5 mm<br>4 | Hard filling > 5 mm<br>2 | Soft filling < 5 mm<br>2 | Soft filling > 5 mm<br>0 |
| Weathering<br>Ratings | Unweathered<br>6 | Slightly weathered<br>5 | Moderately weathered<br>3 | Highly weathered<br>1 | Decomposed<br>0 |

**F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING****

| Strike perpendicular to tunnel axis | | Strike parallel to tunnel axis | |
|---|---|---|---|
| Drive with dip - Dip 45 - 90° | Drive with dip - Dip 20 - 45° | Dip 45 - 90° | Dip 20 - 45° |
| Very favourable | Favourable | Very unfavourable | Fair |
| Drive against dip - Dip 45-90° | Drive against dip - Dip 20-45° | Dip 0-20 - Irrespective of strike° | |
| Fair | Unfavourable | Fair | |

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

** Modified after Wickham et al (1972).

Table 4.5: Guidelines for excavation and support of 10 m span rock tunnels in accordance with the *RMR* system (After Bieniawski 1989).

| Rock mass class | Excavation | Rock bolts (20 mm diameter, fully grouted) | Shotcrete | Steel sets |
|---|---|---|---|---|
| I - Very good rock *RMR*: 81-100 | Full face, 3 m advance. | Generally no support required except spot bolting. | | |
| II - Good rock *RMR*: 61-80 | Full face , 1-1.5 m advance. Complete support 20 m from face. | Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh. | 50 mm in crown where required. | None. |
| III - Fair rock *RMR*: 41-60 | Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face. | Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown. | 50-100 mm in crown and 30 mm in sides. | None. |
| IV - Poor rock *RMR*: 21-40 | Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face. | Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh. | 100-150 mm in crown and 100 mm in sides. | Light to medium ribs spaced 1.5 m where required. |
| V – Very poor rock *RMR*: < 20 | Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting. | Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert. | 150-200 mm in crown, 150 mm in sides, and 50 mm on face. | Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert. |

Cummings et al (1982) and Kendorski et al (1983) have also modified Bieniawski's *RMR* classification to produce the *MBR* (modified basic *RMR*) system for mining. This system was developed for block caving operations in the USA. It involves the use of different ratings for the original parameters used to determine the value of *RMR* and the subsequent adjustment of the resulting *MBR* value to allow for blast damage, induced stresses, structural features, distance from the cave front and size of the caving block. Support recommendations are presented for isolated or development drifts as well as for the final support of intersections and drifts.

## 3.5 Rock Tunnelling Quality Index, *Q*

On the basis of an evaluation of a large number of case histories of underground excavations, Barton et al (1974) of the Norwegian Geotechnical Institute proposed a Tunnelling Quality Index ($Q$) for the determination of rock mass characteristics and tunnel support requirements. The numerical value of the index $Q$ varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is defined by:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (4.2)$$

where

| | |
|---|---|
| $RQD$ | is the Rock Quality Designation |
| $J_n$ | is the joint set number |
| $J_r$ | is the joint roughness number |
| $J_a$ | is the joint alteration number |
| $J_w$ | is the joint water reduction factor |
| $SRF$ | is the stress reduction factor |

In explaining the meaning of the parameters used to determine the value of $Q$, Barton et al (1974) offer the following comments:

> The first quotient ($RQD/J_n$), representing the structure of the rock mass, is a crude measure of the block or particle size, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400. If the quotient is interpreted in units of centimetres, the extreme 'particle sizes' of 200 to 0.5 cm are seen to be crude but fairly realistic approximations. Probably the largest blocks should be several times this size and the smallest fragments less than half the size. (Clay particles are of course excluded).
>
> The second quotient ($J_r/J_a$) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favour of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favourable to tunnel stability.
>
> When rock joints have thin clay mineral coatings and fillings, the strength is reduced significantly. Nevertheless, rock wall contact after small shear displacements have occurred may be a very important factor for preserving the excavation from ultimate failure.
>
> Where no rock wall contact exists, the conditions are extremely unfavourable to tunnel stability. The 'friction angles' (given in Table 4.6) are a little below the residual strength values for most clays, and are possibly down-graded by the fact that these clay bands or fillings may tend to consolidate during shear, at least if normal consolidation or if softening and swelling has occurred. The swelling pressure of montmorillonite may also be a factor here.
>
> The third quotient ($J_w/SRF$) consists of two stress parameters. $SRF$ is a measure of: 1) loosening load in the case of an excavation through shear zones and clay bearing rock, 2) rock stress in competent rock, and 3) squeezing loads in plastic

incompetent rocks. It can be regarded as a total stress parameter. The parameter $J_w$ is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. Water may, in addition, cause softening and possible out-wash in the case of clay-filled joints. It has proved impossible to combine these two parameters in terms of inter-block effective stress, because paradoxically a high value of effective normal stress may sometimes signify less stable conditions than a low value, despite the higher shear strength. The quotient ($J_w/SRF$) is a complicated empirical factor describing the 'active stress'.

It appears that the rock tunnelling quality $Q$ can now be considered to be a function of only three parameters which are crude measures of:

1. Block size ($RQD/J_n$)
2. Inter-block shear strength ($J_r/J_a$)
3. Active stress ($J_w/SRF$)

Undoubtedly, there are several other parameters which could be added to improve the accuracy of the classification system. One of these would be the joint orientation. Although many case records include the necessary information on structural orientation in relation to excavation axis, it was not found to be the important general parameter that might be expected. Part of the reason for this may be that the orientations of many types of excavations can be, and normally are, adjusted to avoid the maximum effect of unfavourably oriented major joints. However, this choice is not available in the case of tunnels, and more than half the case records were in this category. The parameters $J_n$, $J_r$ and $J_a$ appear to play a more important role than orientation, because the number of joint sets determines the degree of freedom for block movement (if any), and the frictional and dilational characteristics can vary more than the down-dip gravitational component of unfavourably oriented joints. If joint orientations had been included the classification would have been less general, and its essential simplicity lost.

Table 4.6 gives the classification of individual parameters used to obtain the Tunnelling Quality Index $Q$ for a rock mass. The use of this table is illustrated in the following example.

A 15 m span crusher chamber for an underground mine is to be excavated in a norite at a depth of 2,100 m below surface. The rock mass contains two sets of joints controlling stability. These joints are undulating, rough and unweathered with very minor surface staining. $RQD$ values range from 85% to 95% and laboratory tests on core samples of intact rock give an average uniaxial compressive strength of 170 MPa. The principal stress directions are approximately vertical and horizontal and the magnitude of the horizontal principal stress is approximately 1.5 times that of the vertical principal stress. The rock mass is locally damp but there is no evidence of flowing water.

The numerical value of $RQD$ is used directly in the calculation of $Q$ and, for this rock mass, an average value of 90 will be used. Table 4.6.2 shows that, for two joint sets, the joint set number, $J_n$ = 4. For rough or irregular joints which are undulating, Table 4.6.3

gives a joint roughness number of $J_r = 3$. Table 4.6.4 gives the joint alteration number, $J_a = 1.0$, for unaltered joint walls with surface staining only. Table 4.6.5 shows that, for an excavation with minor inflow, the joint water reduction factor, $J_w = 1.0$. For a depth below surface of 2,100 m the overburden stress will be approximately 57 MPa and, in this case, the major principal stress $\sigma_1 = 85$ MPa. Since the uniaxial compressive strength of the norite is approximately 170 MPa, this gives a ratio of $\sigma_c / \sigma_1 = 2$. Table 4.6.6 shows that, for competent rock with rock stress problems, this value of $\sigma_c / \sigma_1$ can be expected to produce heavy rock burst conditions and that the value of *SRF* should lie between 10 and 20. A value of *SRF* = 15 will be assumed for this calculation. Using these values gives:

$$Q = \frac{90}{4} \times \frac{3}{1} \times \frac{1}{15} = 4.5$$

In relating the value of the index $Q$ to the stability and support requirements of underground excavations, Barton et al (1974) defined an additional parameter which they called the *Equivalent Dimension*, $D_e$, of the excavation. This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the *Excavation Support Ratio*, *ESR*. Hence:

$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio } ESR}$$

The value of *ESR* is related to the intended use of the excavation and to the degree of security which is demanded of the support system installed to maintain the stability of the excavation. Barton et al (1974) suggest the following values:

| Excavation category | | *ESR* |
|---|---|---|
| A | Temporary mine openings. | 3-5 |
| B | Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations. | 1.6 |
| C | Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels. | 1.3 |
| D | Power stations, major road and railway tunnels, civil defence chambers, portal intersections. | 1.0 |
| E | Underground nuclear power stations, railway stations, sports and public facilities, factories. | 0.8 |

The crusher station discussed above falls into the category of permanent mine openings and is assigned an excavation support ratio *ESR* = 1.6. Hence, for an excavation span of 15 m, the equivalent dimension, $D_e = 15/1.6 = 9.4$.

The equivalent dimension, *De*, plotted against the value of *Q*, is used to define a number of support categories in a chart published in the original paper by Barton et al (1974). This chart has recently been updated by Grimstad and Barton (1993) to reflect the increasing use of steel fibre reinforced shotcrete in underground excavation support. Figure 4.3 is reproduced from this updated chart.

From Figure 4.3, a value of $D_e$ of 9.4 and a value of $Q$ of 4.5 places this crusher excavation in category (4) which requires a pattern of rockbolts (spaced at 2.3 m) and 40 to 50 mm of unreinforced shotcrete.

Because of the mild to heavy rock burst conditions which are anticipated, it may be prudent to destress the rock in the walls of this crusher chamber. This is achieved by using relatively heavy production blasting to excavate the chamber and omitting the smooth blasting usually used to trim the final walls of an excavation such as an underground powerhouse at shallower depth. Caution is recommended in the use of destress blasting and, for critical applications, it may be advisable to seek the advice of a blasting specialist before embarking on this course of action.

Løset (1992) suggests that, for rocks with $4 < Q < 30$, blasting damage will result in the creation of new 'joints' with a consequent local reduction in the value of $Q$ for the rock surrounding the excavation. He suggests that this can be accounted for by reducing the *RQD* value for the blast damaged zone.

Assuming that the *RQD* value for the destressed rock around the crusher chamber drops to 50 %, the resulting value of $Q = 2.9$. From Figure 4.3, this value of $Q$, for an equivalent dimension, $D_e$ of 9.4, places the excavation just inside category (5) which requires rockbolts, at approximately 2 m spacing, and a 50 mm thick layer of steel fibre reinforced shotcrete.

Barton et al (1980) provide additional information on rockbolt length, maximum unsupported spans and roof support pressures to supplement the support recommendations published in the original 1974 paper.

The length L of rockbolts can be estimated from the excavation width *B* and the Excavation Support Ratio *ESR*:

$$L = \frac{2 + 0.15B}{ESR} \tag{4.3}$$

The maximum unsupported span can be estimated from:

$$\text{Maximum span (unsupported)} = 2\, ESR\, Q^{0.4} \tag{4.4}$$

Based upon analyses of case records, Grimstad and Barton (1993) suggest that the relationship between the value of $Q$ and the permanent roof support pressure $P_{\text{roof}}$ is estimated from:

$$P_{\text{roof}} = \frac{2\sqrt{J_n}\, Q^{-\frac{1}{3}}}{3 J_r} \tag{4.5}$$

Table 4.6: Classification of individual parameters used in the Tunnelling Quality Index *Q* (After Barton et al 1974).

| DESCRIPTION | VALUE | NOTES | |
|---|---|---|---|
| **1. ROCK QUALITY DESIGNATION** | ***RQD*** | | |
| A. Very poor | 0 - 25 | 1. Where *RQD* is reported or measured as ≤ 10 (including 0), | |
| B. Poor | 25 - 50 | a nominal value of 10 is used to evaluate *Q*. | |
| C. Fair | 50 - 75 | | |
| D. Good | 75 - 90 | 2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently | |
| E. Excellent | 90 - 100 | accurate. | |
| **2. JOINT SET NUMBER** | $J_n$ | | |
| A. Massive, no or few joints | 0.5 - 1.0 | | |
| B. One joint set | 2 | | |
| C. One joint set plus random | 3 | | |
| D. Two joint sets | 4 | | |
| E. Two joint sets plus random | 6 | | |
| F. Three joint sets | 9 | 1. For intersections use $(3.0 \times J_n)$ | |
| G. Three joint sets plus random | 12 | | |
| H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc. | 15 | 2. For portals use $(2.0 \times J_n)$ | |
| J. Crushed rock, earthlike | 20 | | |
| **3. JOINT ROUGHNESS NUMBER** | $J_r$ | | |
| ***a. Rock wall contact*** | | | |
| ***b. Rock wall contact before 10 cm shear*** | | | |
| A. Discontinuous joints | 4 | | |
| B. Rough and irregular, undulating | 3 | | |
| C. Smooth undulating | 2 | | |
| D. Slickensided undulating | 1.5 | 1. Add 1.0 if the mean spacing of the relevant joint set is | |
| E. Rough or irregular, planar | 1.5 | greater than 3 m. | |
| F. Smooth, planar | 1.0 | | |
| G. Slickensided, planar | 0.5 | 2. $J_r$ = 0.5 can be used for planar, slickensided joints having | |
| ***c. No rock wall contact when sheared*** | | lineations, provided that the lineations are oriented for | |
| H. Zones containing clay minerals thick enough to prevent rock wall contact | 1.0 (nominal) | minimum strength. | |
| J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact | 1.0 (nominal) | | |
| **4. JOINT ALTERATION NUMBER** | $J_a$ | $\phi r$ degrees (approx.) | |
| ***a. Rock wall contact*** | | | |
| A. Tightly healed, hard, non-softening, impermeable filling | 0.75 | | 1. Values of $\phi r$, the residual friction angle, are intended as an approximate guide |
| B. Unaltered joint walls, surface staining only | 1.0 | 25 - 35 | to the mineralogical properties of the |
| C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc. | 2.0 | 25 - 30 | alteration products, if present. |
| D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening) | 3.0 | 20 - 25 | |
| E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less) | 4.0 | 8 - 16 | |

Table 4.6: (cont'd.) Classification of individual parameters used in the Tunnelling Quality Index *Q* (After Barton et al 1974).

| DESCRIPTION | VALUE | NOTES | |
|---|---|---|---|
| **4, JOINT ALTERATION NUMBER** | $J_a$ | $\phi r$ degrees (approx.) | |
| ***b. Rock wall contact before 10 cm shear*** | | | |
| F. Sandy particles, clay-free, disintegrating rock etc. | 4.0 | 25 - 30 | |
| G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick) | 6.0 | 16 - 24 | |
| H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick) | 8.0 | 12 - 16 | |
| J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of $J_a$ depend on percent of swelling clay-size particles, and access to water. | 8.0 - 12.0 | 6 - 12 | |
| ***c. No rock wall contact when sheared*** | | | |
| K. Zones or bands of disintegrated or crushed | 6.0 | | |
| L. rock and clay (see G, H and J for clay | 8.0 | | |
| M. conditions) | 8.0 - 12.0 | 6 - 24 | |
| N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening | 5.0 | | |
| O. Thick continuous zones or bands of clay | 10.0 - 13.0 | | |
| P. & R. (see G.H and J for clay conditions) | 6.0 - 24.0 | | |
| **5. JOINT WATER REDUCTION** | $J_w$ | approx. water pressure (kgf/cm$^2$) | |
| A. Dry excavation or minor inflow i.e. < 5 l/m locally | 1.0 | < 1.0 | |
| B. Medium inflow or pressure, occasional outwash of joint fillings | 0.66 | 1.0 - 2.5 | |
| C. Large inflow or high pressure in competent rock with unfilled joints | 0.5 | 2.5 - 10.0 | 1. Factors C to F are crude estimates; increase $J_w$ if drainage installed. |
| D. Large inflow or high pressure | 0.33 | 2.5 - 10.0 | |
| E. Exceptionally high inflow or pressure at blasting, decaying with time | 0.2 - 0.1 | > 10 | 2. Special problems caused by ice formation are not considered. |
| F. Exceptionally high inflow or pressure | 0.1 - 0.05 | > 10 | |
| **6. STRESS REDUCTION FACTOR** | | ***SRF*** | |
| ***a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated*** | | | |
| A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth) | | 10.0 | 1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation |
| B. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth < 50 m) | | 5.0 | |
| C. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth > 50 m) | | 2.5 | |
| D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth) | | 7.5 | |
| E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m) | | 5.0 | |
| F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m) | | 2.5 | |
| G. Loose open joints, heavily jointed or 'sugar cube', (any depth) | | 5.0 | |

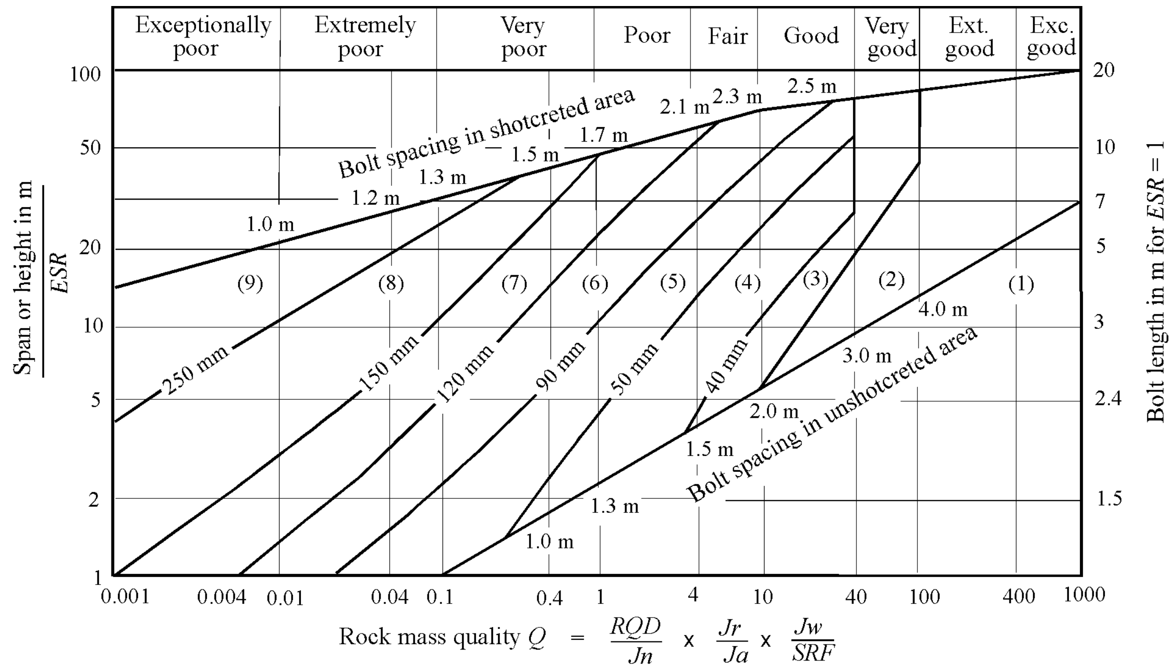
Table 4.6: (cont'd.) Classification of individual parameters in the Tunnelling Quality Index *Q* (After Barton et al 1974).

| DESCRIPTION | VALUE | | | NOTES |
|---|---|---|---|---|
| **6. STRESS REDUCTION FACTOR** | | | ***SRF*** | |
| ***b. Competent rock, rock stress problems*** | | | | |
| | $\sigma_c/\sigma_1$ | $\sigma_t\sigma_1$ | | 2. For strongly anisotropic virgin stress field |
| H. Low stress, near surface | > 200 | > 13 | 2.5 | (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce $\sigma_c$ |
| J. Medium stress | 200 - 10 | 13 - 0.66 | 1.0 | to $0.8\sigma_c$ and $\sigma_t$ to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, |
| K. High stress, very tight structure | 10 - 5 | 0.66 - 0.33 | 0.5 - 2 | reduce $\sigma_c$ and $\sigma_t$ to $0.6\sigma_c$ and $0.6\sigma_t$, where |
| (usually favourable to stability, may | | | | $\sigma_c$ = unconfined compressive strength, and |
| be unfavourable to wall stability) | | | | $\sigma_t$ = tensile strength (point load) and $\sigma_1$ and |
| L. Mild rockburst (massive rock) | 5 - 2.5 | 0.33 - 0.16 | 5 - 10 | $\sigma_3$ are the major and minor principal stresses. |
| M. Heavy rockburst (massive rock) | < 2.5 | < 0.16 | 10 - 20 | 3. Few case records available where depth of |
| ***c. Squeezing rock, plastic flow of incompetent rock*** | | | | crown below surface is less than span width. |
| ***under influence of high rock pressure*** | | | | Suggest *SRF* increase from 2.5 to 5 for such |
| N. Mild squeezing rock pressure | | | 5 - 10 | cases (see H). |
| O. Heavy squeezing rock pressure | | | 10 - 20 | |
| ***d. Swelling rock, chemical swelling activity depending on presence of water*** | | | | |
| P. Mild swelling rock pressure | | | 5 - 10 | |
| R. Heavy swelling rock pressure | | | 10 - 15 | |

**ADDITIONAL NOTES ON THE USE OF THESE TABLES**

When making estimates of the rock mass Quality (*Q*), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, *RQD* can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to *RQD* for the case of clay free rock masses: $RQD = 115 - 3.3\,J_v$ (approx.), where $J_v$ = total number of joints per $m^3$ ($0 < RQD < 100$ for $35 > J_v > 4.5$).
2. The parameter $J_n$ representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating $J_n$.
3. The parameters $J_r$ and $J_a$ (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of $J_r/J_a$ is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of $J_r/J_a$ should be used when evaluating *Q*. The value of $J_r/J_a$ should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor *SRF* appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths ($\sigma_c$ and $\sigma_t$) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.




**REINFORCEMENT CATEGORIES**
1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting with 40-100 mm unreinforced shotcrete
5) Fibre reinforced shotcrete, 50 - 90 mm, and bolting
6) Fibre reinforced shotcrete, 90 - 120 mm, and bolting
7) Fibre reinforced shotcrete, 120 - 150 mm, and bolting
8) Fibre reinforced shotcrete, > 150 mm, with reinforced ribs of shotcrete and bolting
9) Cast concrete lining

Figure 4.3: Estimated support categories based on the tunnelling quality index *Q* (After Grimstad and Barton 1993).

## 3.6 Using rock mass classification systems

The two most widely used rock mass classifications are Bieniawski's *RMR* (1976, 1989) and Barton et al's *Q* (1974). Both methods incorporate geological, geometric and design/engineering parameters in arriving at a quantitative value of their rock mass quality. The similarities between *RMR* and *Q* stem from the use of identical, or very similar, parameters in calculating the final rock mass quality rating. The differences between the systems lie in the different weightings given to similar parameters and in the use of distinct parameters in one or the other scheme
the other scheme.

*RMR* uses compressive strength directly while *Q* only considers strength as it relates to in situ stress in competent rock. Both schemes deal with the geology and geometry of the rock mass, but in slightly different ways. Both consider groundwater, and both include some component of rock material strength. Some estimate of orientation can be incorporated into *Q* using a guideline presented by Barton et al (1974): 'the parameters $J_r$ and $J_a$ should ... relate to the surface most likely to allow failure to initiate.' The greatest difference between the two systems is the lack of a stress parameter in the *RMR* system.

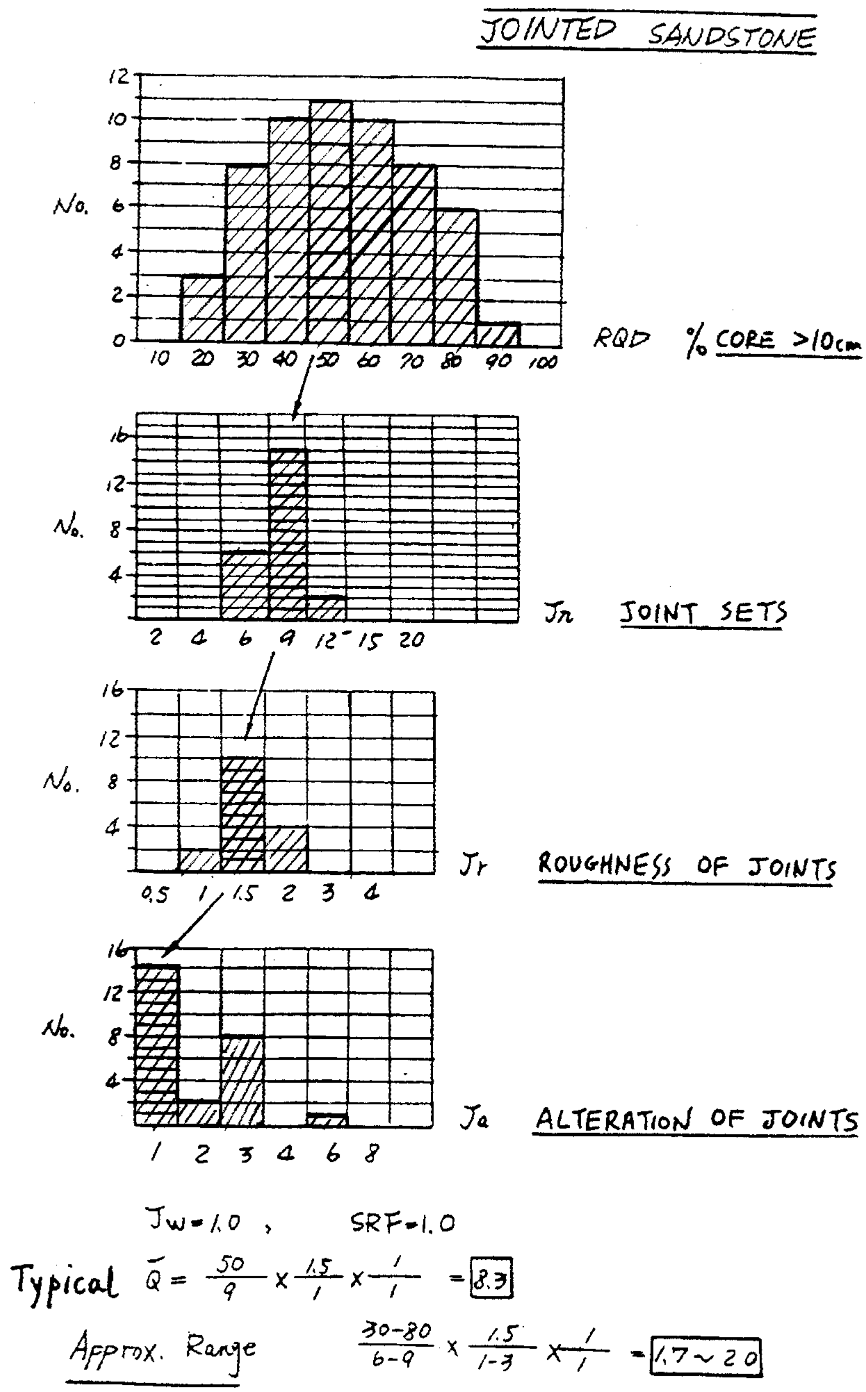



Figure 4.4: Histograms showing variations in *RQD*, $J_n$, $J_r$ and $J_a$ for a dry jointed sandstone under 'medium' stress conditions, reproduced from field notes prepared by Dr. N. Barton.

When using either of these methods, two approaches can be taken. One is to evaluate the rock mass specifically for the parameters included in the classification methods; the other is to accurately characterise the rock mass and then attribute parameter ratings at a later time. The latter method is recommended since it gives a full and complete description of the rock mass which can easily be translated into either classification index. If rating values alone had been recorded during mapping, it would be almost impossible to carry out verification studies.

In many cases, it is appropriate to give a range of values to each parameter in a rock mass classification and to evaluate the significance of the final result. An example of this approach is given in Figure 4.4 which is reproduced from field notes prepared by Dr. N. Barton on a project. In this particular case, the rock mass is dry and is subjected to 'medium' stress conditions (Table 4.6.6.K) and hence $J_w$ = 1.0 and *SRF* = 1.0. Histograms showing the variations in *RQD*, $J_n$, $J_r$ and $J_a$, along the exploration adit mapped, are presented in this figure. The average value of $Q$ = 8.9 and the approximate range of $Q$ is $1.7 < Q < 20$. The average value of $Q$ can be used in choosing a basic support system while the range gives an indication of the possible adjustments which will be required to meet different conditions encountered during construction.

A further example of this approach is given in a paper by Barton et al (1992) concerned with the design of a 62 m span underground sports hall in jointed gneiss. Histograms of all the input parameters for the $Q$ system are presented and analysed in order to determine the weighted average value of $Q$.

Carter (1992) has adopted a similar approach, but extended his analysis to include the derivation of a probability distribution function and the calculation of a probability of failure in a discussion on the stability of surface crown pillars in abandoned metal mines.

Throughout this chapter it has been suggested that the user of a rock mass classification scheme should check that the latest version is being used. An exception is the use of Bieniawski's *RMR* classification for rock mass strength estimates (discussed in Chapter 8) where the 1976 version as well as the 1989 version are used. It is also worth repeating that the use of two rock mass classification schemes is advisable.