12 Rock mass classification

12.1 Rock parameters and classification schemes

The purpose of **rock mass classification** is to establish the quality of a particular rock mass (or part of a rock mass) by assigning rating values to a **set of rock parameters**. Webster's dictionary defines 'classification' as "the act of classifying or forming into a class or classes, so as to bring together those beings or things which most resemble each other, and to separate those that differ". This definition immediately highlights two main issues in rock mass classification: the purpose of the classification has to be established and the method of classification has to be commensurate with the purpose.

For example, if we only used the uniaxial compressive strength of the intact rock and the fracture frequency of the rock mass, we could generate a rock mass classification scheme for characterizing sections of rock in a tunnel as shown in Table 12.1.

On the basis of this scheme, all rock masses must then be one of the categories, *A1*, *A2*, *B1*, *B2*. We could call this a **Rock Index** and assign the words 'Good' to *A1*, 'Fair' to *A2* and *B1*, and 'Poor' to *B2*. But what is the purpose of this classification? Perhaps, the Rock Index would indicate the excavatability and stability of the rock masses in each category. If so, is the classification the best one for that purpose?

There are **four main steps** in the development of any rock mass classification scheme:

- decide on the objective of the rock mass classification scheme;
- decide on the parameters to be used, their ranges and ratings;
- decide on the algebra to be used for the rock index (e.g. do we simply

Table 12.1 Illustrative simple rock mass classification scheme

Parameter	Ratings, R	
Uniaxial compressive strength, σ_c	If $\sigma_c \geq 100$ MPa, $R = A$	If $\sigma_c < 100$ MPa, $R = B$
Fracture frequency, λ	If $\lambda \leq 4$/m, $R = 1$	If $\lambda > 4$/m, $R = 2$

select values from a table, do we add rating values together, do we multiply ratings together, or something else?); and
- calibrate the rock index value against the objective.

The **advantage** of using a rock mass classification scheme is that it is a simple and effective way of representing rock mass quality and of encapsulating precedent practice. The **disadvantage** is that one cannot use it for a different objective or in significantly new circumstances.

The rock mass classifications that have been developed to date follow this basic approach, but include more parameters and use a greater number of classes than the simple 'good', 'fair', 'poor' example we gave above. For example, by adding a third parameter to the classification given in Table 12.1, 'thickness of the layers', and using more rating values (Vervoort and de Wit, 1997[1]), a useful rock index for rock dredging has been developed. By judicious choice of the relevant parameters, such rock mass classification schemes can be a powerful tool for rock engineering.

The two main classification systems, Rock Mass Rating and Tunnelling Quality Index (***RMR* and *Q***), have both been widely applied and there is now a large database of projects where they have been used as the main indicator of rock stabilization requirements in rock tunnelling. The systems provide a coherent method of using precedent practice experience and can now be linked to numerical analysis approaches.

With all schemes, the key issues are the objective of the classification system, choice of the optimal parameters, assigning numerical ratings to parameter values, the algebraic manipulation of the parameter ratings, and drawing conclusions from the mean and variation of the overall rock quality index values.

12.2 Questions and answers: rock mass classification

Rock mass classification schemes are designed to be used in the field, but it is possible to apply them in the office, given a description of the rock mass. Indeed, they are often used in this way, especially as they can be a means of translating a site investigation report into an input for design. In the following questions, you are asked to apply rock mass classification schemes given a description of two rock masses[2] (Bieniawski, 1989).

Q12.1 A mudstone rock mass at a depth of 200 m contains three fracture sets. One set comprises bedding planes; these are highly weathered, slightly rough surfaces, and are continuous with an orientation of 180/10. Another set is jointing; these joints are slightly weathered, slightly rough, and have an orientation of 185/75. The

[1] Vervoort A. and de Wit K. (1997) Use of rock mass classifications for dredging. *Int. J. Rock Mech. Min. Sci.*, **34**, 5, 859–864.

[2] To answer Q12.1–12.5, it is necessary to use the *RMR* and *Q* rock mass classification tables in Appendix 3 of this book. Further explanation of these tables is presented in Bieniawski Z. T. (1989) *Engineering Rock Mass Classifications*. Wiley, New York, 251pp.

third set is also jointing; again, the joints are slightly weathered and slightly rough, and have an orientation of 090/80. The strength of the intact rock has been assessed as 55 MPa, and values for the *RQD* and mean fracture spacing are reported as 60% and 0.4 m, respectively.

Use the *RMR* system to classify this rock mass, and assess the stability of a 10 m wide excavation being driven from east to west.

A12.1 In order to apply the *RMR* system, five principal parameters are assessed: intact rock strength, groundwater conditions, *RQD*, fracture spacing, and fracture condition. This provides the basic *RMR* value for the rock mass. The orientation of the fractures is then accounted for through the use of rating adjustment factors to determine the final *RMR* value.

In the example here we have three sets of fractures, and so we will need to apply the *RMR* system to each set in turn, and hence identify the set that is most critical for this particular excavation. Of course, the classification parameters of intact rock strength and groundwater conditions relate to the rock mass as a whole, rather than to specific fracture sets, and are applied identically to each set. Here, the values for *RQD* and mean fracture spacing are reported generally, rather than for specific sets, and so these will be applied equally. We will assess all of these parameters first, and then move on to the parameters that are specific to the fracture sets.

Overall rock mass

The strength of the intact rock has been assessed as 55 MPa, and so a conservative value for the strength rating is 6. The excavation is situated at a depth of 200 m, and the rock mass is a mudstone. At a depth of 200 m, the vertical stress component will be in the region of 5 MPa (assuming a unit weight of rock of 25 kN/m^3), and this stress will probably be sufficient to keep the fractures tightly closed. Taken together with the fact that mudstones have very low primary and secondary permeability, we can infer that the groundwater conditions will probably be in the range of damp to wet, with rating values ranging from 7 to 10.

For an average *RQD* value of 60%, the rating value is about 12, and for a mean fracture spacing of 0.4 m the rating value is about 10. Adding these four ratings together gives a total rating value of
$6 + (7 \text{ to } 10) + 12 + 10 = 35 \text{ to } 38$.

Classification using Set 1

These bedding planes are highly weathered, with slightly rough surfaces, and are continuous. The rating values for these specific attributes are 1, 3 and 0, respectively. An assessment of aperture can be made on the basis of the *in situ* stress state at the location of the excavation: a stress of 5 MPa will mean that the aperture is low, and so a reasonable rating value is 5. We have no information regarding infilling but, as there is no evidence to suggest the presence of any, we can assume a rating value of 6. The total rating value for these fractures is then $1 + 3 + 0 + 5 + 6 = 15$. The dip direction of the bedding is 180°, and so the excavation is being driven along the strike. This is considered to result in 'fair' conditions, with a corresponding rating adjustment value of −5.

Thus, the overall *RMR* value based on Set 1 is (35 to 38) + 15 = 50 to 53, which is classified as 'fair rock'. Taking into account the orientation rating adjustment of −5, the *RMR* value is reduced to 45 to 48, but this does not alter the classification.

Classification using Set 2

We know that this jointing is slightly weathered, slightly rough, and has an orientation of 185/75. We can therefore assign rating values of 5, 3 and −12 to these attributes. Knowing that the fractures are joints in a mudstone, their persistence will probably be in the range of 1 m to 2 m, and an appropriate rating value for this is 2. As with Set 1, we can assign rating values for the aperture and infilling of 5 and 6, respectively. The total rating value for these fractures is then $5 + 3 + 2 + 5 + 6 = 21$, giving an overall *RMR* value of (35 to 38) + 21 = 56 to 59. Again, this is classified as 'fair rock'. Taking the orientation rating adjustment of −12 into account reduces the *RMR* value to 44 to 47.

Classification using Set 3

The jointing representing Set 3 has identical mechanical characteristics to Set 2, and so has a rating value of 21, giving an overall *RMR* value of 56 to 59 and a classification of 'fair rock'. The orientation rating adjustment is now −5 (the strike is perpendicular to the excavation axis, and we are driving against the dip which is classed as 'fair'), which reduces the *RMR* value to 51 to 54.

Overall assessment

We can see that Set 2 leads to the most critical classification, with a range of probable *RMR* values of 44 to 47. Using a chart linking *RMR* and excavation span to stand-up time shows that an excavation 10 m wide in such a rock mass would suffer from immediate collapse, and so we can see that the engineering design will need to incorporate rock stabilization measures (i.e. support or reinforcement). In addition, some form of staged excavation may also be necessary, whereby a small pilot excavation is formed and then systematically opened out to the full size as the engineering behaviour of the rock mass is steadily improved as the stabilization measures are applied.

***Q12.2* A 7-m-diameter tunnel is to be driven through a sequence of shale and basalt rock at a maximum depth of 61 m. The shales dip towards the east, and the basalts form sub-vertical dykes. The bedding dips between 15° and 20°, the joints dip between 70° and 90°. The joints in the shale are rough, and most of them are thin and healed with calcite, but overall the rock is described as 'blocky and seamy'. The groundwater level is about 50 m above the invert of the tunnel. The average uniaxial compressive strength of the shale is 53 MPa, of the basalt it is 71 MPa. The vertical stress is about 1.0 MPa, and the horizontal stress is about 3.4 MPa. The snaking nature of**

the tunnel's route means that at some place along its length it will head in all directions between 090° and 180°.

Use the *RMR* system to predict how the rock will behave in the excavation.

A12.2 This is an example of a commonly occurring problem where, at first sight, much useful information is given but, when we investigate further, we find that there is little on which to base a rock mass classification. Often we are not able to obtain further information, and so it is necessary to carefully consider what are the appropriate rating values. Also, there may be dispute about the rating values we select, and so it is prudent to investigate the sensitivity of the rating value assessments to these values.

Overall environmental conditions

As the groundwater level is 50 m above the tunnel invert, a water pressure of 0.5 MPa will be induced at the tunnel level. The major principal stress is 3.4 MPa acting horizontally, and so the ratio of water pressure to major principal stress is $0.5/3.4 \approx 0.15$, which is regarded as 'wet' and therefore attracts a rating value of 7.

Shale

The shale is described as 'blocky and seamy' and, although descriptions do not enter into the *RMR* assessment directly, they can be used to help assess the *RQD* and fracture spacing values. Shales are sedimentary rocks, and tend to form distinct beds. The description 'blocky and seamy' allows us to picture a rock mass that splits easily along the bedding, but is broken into blocks by the joints. Thus, it is likely that both the *RQD* and the mean fracture spacing values will be low for such a rock, and so we may select rating values of, say, 10 and 8 for these two parameters.

We are told that the 'joints in the shale are rough, and most of them are thin and healed with calcite'. We do not know whether this description is just for the joints, or for the joints and bedding planes. If we assume it is for the joints only, then how do we assess the bedding planes? Bedding planes tend to be extensive, and in a shale will be smooth. Joints on the other hand will probably have a persistence of no more than a few metres. We can use this understanding to make preliminary rating assessments. The fact that the joints are 'thin and healed with calcite' allows us to assess the aperture rating as being about 5, and the infilling as about 5 as well. Finally, the tunnel is to be excavated at quite a shallow depth — no more than about 60 m — and on this basis it would be prudent to assume that some weathering will have taken place. A rating value of 4 for weathering is therefore suggested.

We can now assess these fracture sets as follows:

	Persistence	Aperture	Roughness	Infilling	Weathering	Total
Bedding	0	5	1	5	4	15
Joints	5	5	5	5	4	24

This shows that the rating value for the bedding is much lower than that of the jointing, and so it is likely that the bedding will be the most critical fracture set for determining tunnel stability. As the dip of the bedding is no more than 20°, this is considered to result in 'fair' conditions, and so the rating adjustment for orientation will be −5. Note that if we consider the joints, then when the tunnel is heading south these features will give rise to 'very unfavourable' conditions, with a rating adjustment of −12. Combining the fracture assessment and the rating adjustment for the bedding gives a value of $15 - 5 = 10$, and for the joints gives a value of $24 - 12 = 12$. This shows that the bedding is, as we surmised above, the most critical feature, but there is little difference between the joints and the bedding.

Finally, we know the strength of the shale is 53 MPa, and this gives a rating value of about 5. The basic *RMR* value for the shale is then as follows:

Strength	Groundwater	*RQD*	Spacing	Fracture Condition	Total
5	7	10	8	15	45

Taking the rating adjustment for orientation into account reduces this to 40.

Basalt dykes

We are given no definite geomechanical data regarding the basalt dykes, other than that they are sub-vertical features. We need to turn to our geological knowledge in order to make an assessment of this rock type. Firstly, we can assume that these dykes will be of limited thickness, say, no more than 5 m. As the cooling joints in a dyke run across the plane of the dyke — rather than parallel to it — this will set a maximum fracture persistence of 5 m, giving a rating value of 2. The aperture of these cooling joints could be as large as 1 mm, giving a rating value of about 3. The joints are liable to be 'slightly rough', with no infilling and only slight weathering. The rating values for these attributes are 3, 6 and 5, respectively. The overall assessment for the joints in the dykes is then $2 + 3 + 3 + 6 + 5 = 19$. The rating adjustment for the orientation of these joints is difficult to assess but, given the limited extent of the joints, we can perhaps assign an effect of 'fair' and a corresponding rating value of −5.

The *RQD* of the dykes will probably be high, and a rating value of around 15 will be suitable. The mean fracture spacing may be of the order of 0.5 m, and the rating value for this is 10. Finally, the strength of the basalt is 71 MPa, and the corresponding rating value is about 7.

The basic *RMR* value for the dykes is then as follows:

Strength	Groundwater	*RQD*	Spacing	Fracture condition	Total
7	7	15	10	19	58

Taking the rating adjustment for orientation into account reduces this to 53.

We must bear in mind that the dykes may have been acting as conduits for groundwater flow for a long time, and may be highly weathered.

If there is any evidence for such weathering, then another assessment should be made taking this into account.

Overall assessment

The overall *RMR* values for the two rock types are 40 for the shale and 53 for the basalt dykes. For a 7-m-diameter tunnel, we find that the shale will suffer immediate collapse, whereas the basalt will be able to stand unsupported for around 1 week. We can now suggest that the tunnel excavation should be carried out using some form of shield or tunnel boring machine to offer continuous support, followed by the installation of an immediate support system such as a pre-cast concrete lining. This overall system may not work well when a dyke is encountered (the rock may be too strong to excavate mechanically), but the additional stand-up time available in this material will allow the use of a different support or reinforcement system, say, shotcrete or rockbolts.

Q12.3 Use the Q system to assess the stability of the rock mass as described in Q12.1.

A12.3 In order to determine the *Q* value for a rock mass, we need to determine rating values for each of six parameters: *RQD*, joint set number, joint roughness number, joint alteration number, joint water reduction factor, and stress reduction factor.

***RQD* rating**

The average *RQD* value is 60%, and hence the rating value is 60.

Joint set number (J_n)

As there are three sets of fractures, the appropriate value for this parameter is 9.

Joint roughness number (J_r)

The bedding planes are slightly rough, continuous surfaces. No single rating entry fits this description exactly, but that of 'rough or irregular, planar' seems most appropriate for such a large-scale feature as a bedding plane. The rating value is then 1.5.

The jointing is slightly weathered and slightly rough. Joints are likely to be relatively small-scale features (persistence of the order of 1 m to 2 m) and so a realistic rating value is 3, appropriate for 'rough or irregular, undulating' features.

Although we could perform two calculations and use both of these values in turn, it is only appropriate to use the one that represents the most critical fracture set. Thus, as the rock is a mudstone, we should take particular account of the highly continuous bedding planes, and hence use the value of 1.5.

Joint alteration number (J_a)

Although we know that the bedding planes are highly weathered and that the jointing is slightly weathered, we have no information regarding fracture infilling materials. However, the general description of the geological environment would suggest that the fracture surfaces are in contact. This, taken in conjunction with the fact that the rock type is a mudstone, leads to the selection of '...low-friction clay mineral coatings...' as the most appropriate entry for joint alteration, giving a rating value of 4.0. It is worthwhile noting that, had we selected entries for 'rock wall contact before 10 cm shear', we may have felt that the entry for 'clay mineral fillings (continuous, <5 mm in thickness)' was the most appropriate, giving a rating value of 8.0. We should bear this in mind and examine the final classification for sensitivity to this rating value.

Joint water reduction factor (J_w)
The presence of water has not been noted in the description and, given the low primary and secondary permeability of mudstone, it is reasonable to select 'dry excavations or minor inflow, e.g. 5 l/min locally' for this parameter, and set the rating value to 1.0.

Stress reduction factor (*SRF*)
The tunnel is to be excavated at a depth of 200 m in a rock with a compressive strength of 55 MPa. The vertical stress at this depth will be in the order of 5 MPa (assuming a unit weight of rock of 25 kN/m^3), and assuming that this will be the major principal stress leading to a strength/stress ratio of $55/5 = 11$. However, at this depth it is possible that the major principal stress could be horizontal with a magnitude twice that of the vertical stress, which will lead to a strength/stress ratio of 5.5. Taking these two results together indicates that we should regard this environment as 'high-stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)' and take the rating for SRF to lie in the range of 0.5 to 2.0. An initial assessment can be made using a value of 1.0, but we should be prepared to investigate the effect of varying this rating.

***Q* value and assessment**
The Q value for the rock mass is now computed as

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\text{SRF}} = \frac{60}{9} \times \frac{1.5}{4.0} \times \frac{1.0}{1.0} \approx 2.5$$

for which the classification is 'poor'. If, as was noted above, we investigate the sensitivity of Q to our uncertainty by increasing the joint alteration number to 8.0 and increasing the stress reduction factor to 2.0, then we obtain a value of

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\text{SRF}} = \frac{60}{9} \times \frac{1.5}{8.0} \times \frac{1.0}{2.0} \approx 0.6$$

which is regarded as 'very poor'.

In order to determine the engineering ramifications of these assessments, we now need to determine the 'equivalent dimension' of the excavation. This is the actual size of the excavation scaled to account for the degree of security we require in our assessment (i.e. reducing the size gives us less required security, and is appropriate for temporary mine openings; increasing the size increases the required security and is appropriate for openings to which the general public have access). Here, we have no information on which to compute the equivalent dimension, and so we take it as the actual size, i.e. 10 m. Charts and tables are available that show how the reinforcement and support requirements vary for various combinations of Q and equivalent dimension, and using such aids leads to the following assessment for a 10 m span:

Poor rock Untensioned rockbolts, at 1 m to 1.5 m spacings, together with mesh-reinforced shotcrete applied to a thickness of 5 cm to 10 cm.

Very poor rock Untensioned rockbolts, at 1 m spacings, together with mesh-reinforced shotcrete applied to a thickness of 5 cm to 7.5 cm.

The similarity between these schemes would allow us to develop a flexible system for application underground, such that the inevitable variations in rock mass quality encountered during construction could be dealt with easily. It is interesting to see that the shotcrete thickness is lower for the very poor rock than for the poor rock. This is because the rockbolt spacing is also lower, and so the shotcrete spans smaller distances.

Q12.4 Use the Q system to assess the stability of the rock mass described in Q12.2.

A12.4 In this example we will need to assess the two principal rock types — shale and basalt dykes — separately. The lack of geomechanical data means that we will need to apply a good deal of judgement in order to generate a classification for the rock mass.

In order to determine the Q value for a rock mass, we need to determine rating values for each of six parameters: *RQD*, joint set number, joint roughness number, joint alteration number, joint water reduction factor, and stress reduction factor.

RQD rating

No values for *RQD* are given, but using the values determined as part of our assessment using *RMR* gives a rating of 50 for the shale and 75 for the basalt dykes.

Joint set number

It is appropriate to assume that the shale contains three fracture sets, for which the joint set number is 9, and to assume that the cooling joints in the dykes can best be described as '...four or more joint sets, random, heavily jointed...', for which the rating value is 15.

Joint roughness number

The bedding in the shale is likely to be a particularly extensive feature, and may therefore be the most critical feature. For a large-scale feature such as a bedding plane, the description that best fits this is 'rough or irregular, planar', and hence the rating value is 1.5.

In the basalt dykes the joints will tend to be discontinuous, small-scale features, and the appropriate rating value for them will be 4, although the slightly lower value of 3 may be suitable.

Joint alteration number

The joints in the shale are quoted as 'thin and healed with calcite', and so an appropriate classification for these is 'tightly healed, hard, non-softening, impermeable filling, i.e. quartz or epidote' which gives a rating value of 0.75. However, it is possible that the shale may degrade readily, and so we should consider the classification 'softening or low-friction clay mineral coatings', for which the rating value is 4.0.

For the basalt, a prudent classification would be 'slightly altered joint walls', giving a rating value of 2.0.

Joint water reduction factor

The groundwater level is about 50 m above the tunnel invert, and so this could lead to a water pressure of 5 kg/cm^2, for which the rating value is 0.5.

Stress reduction factor

The average uniaxial compressive strength of the shale is 53 MPa, and of the basalt it is 71 MPa. The vertical stress is about 1.0 MPa, and the horizontal stress is about 3.4 MPa. The major principal stress is horizontal with a magnitude of 3.4 MPa, and the compressive strength of the rock types is 53 MPa for the shale and 71 MPa for the basalt. The strength/stress ratio for these two cases is then $53/3.4 = 15.6$ and $71/3.4 = 20.9$, respectively, and so for both of them the stress reduction factor is 1.0.

***Q* value and assessment**

The Q value for the shale is now computed as

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\text{SRF}} = \frac{50}{9} \times \frac{1.5}{0.75} \times \frac{0.5}{1.0} \approx 5.6$$

for which the classification is 'fair', and the Q value for the basalt is

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\text{SRF}} = \frac{75}{15} \times \frac{3}{2} \times \frac{0.5}{1.0} \approx 3.8$$

for which the classification is 'poor'. We noted above that for the shale a value for J_a of 4.0 may be more suitable than 0.75, and adopting this value reduces Q for the shale to 1.0, which is on the boundary of 'very poor' and 'poor' rock.

If we take the 'equivalent dimension' of the excavation to be its actual dimension, i.e. 7 m, then by reference to charts and tables of reinforcement and support requirements we find that the following are appropriate:

Fair rock Untensioned rockbolts at 1 m to 1.5 m spacings, together with mesh.

Poor rock Shotcrete applied to a thickness of 2.5 cm to 7.5 cm, or untensioned rockbolts at 1 m to 1.5 m spacings, together with mesh-reinforced shotcrete applied to a thickness of 5 cm to 10 cm.

Very poor rock Untensioned rockbolts, at 1 m spacings, together with mesh-reinforced shotcrete applied to a thickness of 5 cm to 7.5 cm.

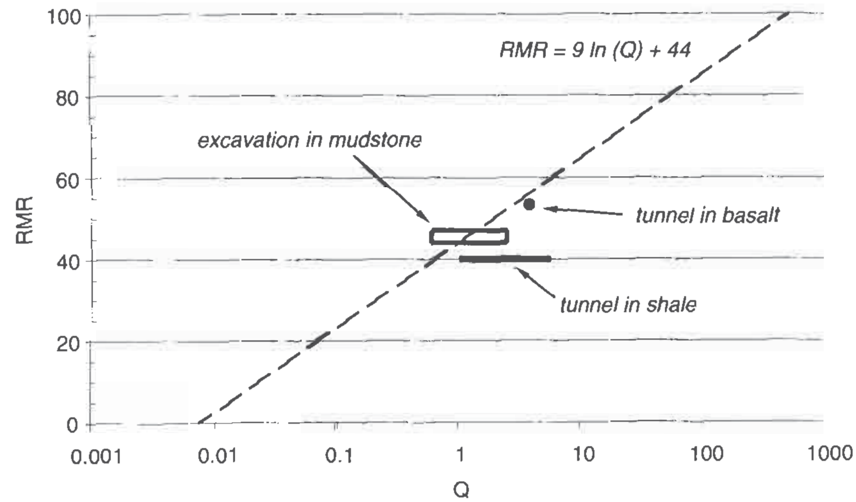
Note how the stabilization systems become heavier as the quality of the rock reduces. Once again, we will be able to exploit the similarity between these schemes to develop a flexible system for use in the tunnel, such that the inevitable variations in rock mass quality encountered during construction could be dealt with easily.

Q12.5 Using your assessments of *RMR* and Q for questions Q12.1, Q12.2, Q12.3 and Q12.4, investigate the relation between the Q and *RMR* values. Do your results correspond with a generally accepted relation, *RMR* = 9 ln Q + 44?

A12.5 From answers A12.1–A12.4, we now have the following information:

Project	*RMR*	*Q*
Excavation in mudstone	44–47	0.6–2.5
Tunnel in shale	40	1–5.6
Tunnel in basalt dykes	53	3.8

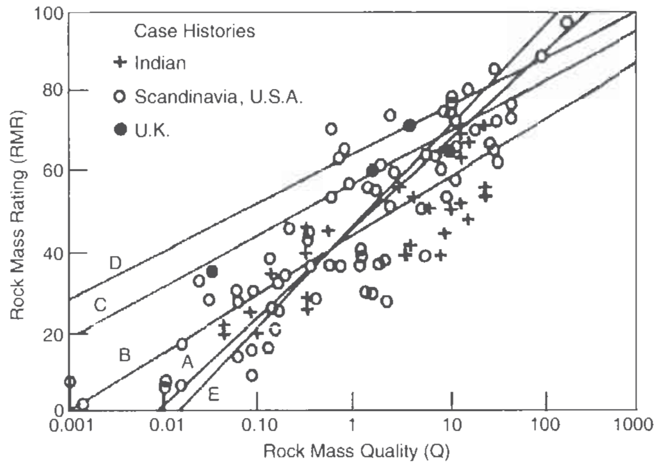
These results, together with the relation $RMR = 9 \ln(Q) + 44$, are plotted below:



Notice that a box has been plotted for the excavation in mudstone, as we have a range of values for both *Q* and *RMR*. For the tunnel in shale, we have a range of values for *Q* and a single value for *RMR* and so a line has been plotted, and for the tunnel in basalt we have a single value for both Q and RMR, which means that only a single point is plotted.

This plot shows that the values we have determined in Q12.1–Q12.4 plot close to the line representing the empirical relation between Q and RMR, despite the fact that we had to use judgement to determine many of the various rating values. This highlights one of the strengths of rock mass classification systems: they are really quite robust in application. However, it is important not to attempt to be too precise when using them. For example, trying to distinguish between Q values of, say, 1.2 and 1.3 is not a useful exercise.

Q12.6 The diagram below (Singh and Goel, 1999[3]) shows *RMR*–Q correlations for case studies in India, Scandinavia, UK and USA.



The suggested *RMR*–Q correlation lines shown on the diagram are

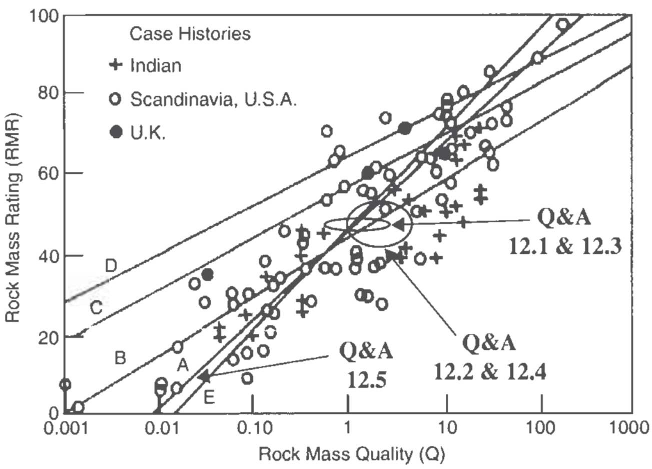
A	$RMR = 9 \ln Q + 44$
B	$RMR = 5.9 \ln Q + 43$
C	$RMR = 5.4 \ln Q + 55.2$
D	$RMR = 5 \ln Q + 60.8$
E	$RMR = 10.5 \ln Q + 41.8$

For a rock engineering design project where a correlation between *RMR* and Q is required to support the design, which of the correlations would you choose?

A12.6 In A12.1–A12.4, it is evident that the assessments of RMR and Q require some experience and engineering judgement. Moreover, the ratings apply for the specific rock mass at the site and to the project in hand. Therefore, it is better to try to extract more information from the site for direct assessment of RMR and Q than to use the correlation lines. However, if there are reasons for using a correlation

[3] The information in this question is from pp. 93–94 in Singh B. and Goel R. K. (1999) *Rock Mass Classification*. Elsevier, Oxford, 267pp.

line, then $RMR = 9 \ln Q + 44$ is the most well known line, although $RMR = 5.9 \ln Q + 43$ has the highest correlation coefficient for the 115 case studies used by Singh and Goel which include 34 from India. Singh and Goel (1999) also indicate how to improve the correlation by using a rock condition rating and a rock mass number.



The *RMR* and *Q* values given in A12.1–A12.4 are plotted on the diagram above. Line A is the one plotted in A12.5.

***Q12.7* Imagine that a rock mass classification system is required to assess the instability of natural slopes in the Italian Alps. List 15–25 parameters that you think would be most useful for such a classification scheme.**

A12.7 A scheme that was developed for this purpose (Mazzoccola and Hudson, 1996[4]) included the following nineteen parameters.

Geology	Folds	Faults	Rainfall	Freeze/thaw
Previous instability	Fracture intact wall strength	Weathering	Number of fracture sets	Fracture orientation
Fracture aperture	Fracture persistence	Fracture spacing	Mechanical properties	Rock mass strength
Hydraulic conditions	Slope orientation	Slope dimensions	*In situ* stress	

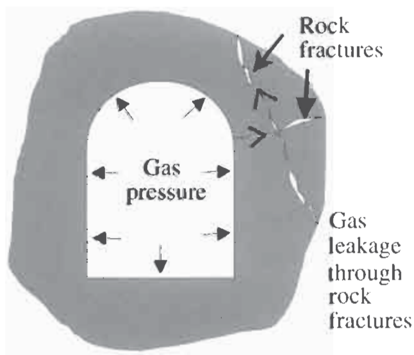
When a standard scheme, such as the *RMR* or *Q* system, is being used, the results can be compared with extensive previous experience (Q and A 12.5 and 12.6) and conclusions drawn about engineering design. In the case of a new classification for a new purpose — in this case a natural slope instability classification — this is not possible, although there is the advantage that we can include all the paramet-

[4] Mazzoccola D. F. and Hudson J. A. (1996). A comprehensive method of rock mass characterization for indicating natural slope instability. *Q. J. Eng. Geol.*, **29**, 37–56.

ers that we consider essential. However, there is the disadvantage that *a priori* assignments of ratings for the parameter values encountered on site are not straightforward (because the values have not yet been measured) and questions about the range and sensitivity of the parameter ratings can only be answered after the measurements have been made.

Nevertheless, the exercise of creating a new scheme is worthwhile because it will indicate the relative nature or quality of different sites. In the case of the Italian Alps work, it was possible to establish the relative instability of twenty natural slopes, which directly indicates which slopes are the most hazardous. The paper by Mazzoccola and Hudson (1996) also demonstrates how to establish weighting factors for the parameter ratings for any given rock mechanics or rock engineering objective.

Q12.8 A rock mass classification system is required for assessing the suitability of different rock formations for storing compressed domestic gas in unlined rock caverns along the route of a main gas transmission line. List the rock parameters that you would use in a rock mass–rock engineering classification scheme for this objective.



A12.8 The principal rock engineering objective is to reduce potential gas leakage to an acceptably low level. The primary gas leakage pathway is via rock fractures, and possibly also the intact rock. Therefore, the following parameters should be included in a rock mass classification scheme tailored to this engineering objective.

- Minimum principal stress value, σ_3, because the gas pressure can open fractures.
- Compressive strength of the rock, σ_c, because the cavern should be stable and the compressive strength indicates the intact rock quality.
- Groundwater pressure at cavern crown level (before cavern excavation), p_w, because the gas has to pass through the water-filled rock fractures.
- Stress anisotropy ratio, $(\sigma_1/\sigma_3) = \sigma_R$, because anisotropic stress conditions are destabilizing to an excavation.
- Maximal stored gas pressure, p_g, because the 'rock mass–rock engineering' composite conditions are a function of this engineering variable.
- Fracture frequency, λ, because the number of fractures will affect leakage.
- Fracture aperture, e, because the aperture will also affect leakage potential.

The next step is to establish whether an increase in each of these seven parameters is good or bad in the context of a classification index in which higher values indicate improved engineering circumstances.

Minimum principal stress value	σ_3	an increase is good
Compressive strength of the rock	σ_c	an increase is good
Groundwater pressure at cavern crown level (before cavern excavation)	p_w	an increase is good
Stress anisotropy ratio	$(\sigma_1/\sigma_3) = \sigma_R$	an increase is bad
Maximal stored gas pressure	p_g	an increase is bad
Fracture frequency	λ	an increase is bad
Fracture aperture	e	an increase is bad

To develop a 'Gas Cavern Tightness Index' (GCTI), the parameters or parameter ratings have to be algebraically arranged so that variations in the parameter values are correctly reflected in the GCTI index according to the table above. If we use the same approach as the Q system, in which the quotients have an interpretation, a suitable GCTI would be

$$\mathrm{GCTI} = \frac{\sigma_c}{\sigma_R} \times \frac{\sigma_3}{\lambda e^3} \times \frac{p_w}{p_g}.$$

The individual quotients can then be interpreted as follows:

$\frac{\sigma_c}{\sigma_R}$ is related to the stability of the intact rock around the cavern;

$\frac{\sigma_3}{\lambda e^3}$ is related to the resistance to gas leakage; and

$\frac{p_w}{p_g}$ is related to the effectiveness of the groundwater confinement.

The reader can no doubt think of improvements to this initial index. Our intention here is only to provide an example of the principles used in developing new rock mass classification indices.

Q12.9 The following parameter values have been determined for three rock mass types found along the route of a major new highway tunnel that passes at a high level through the flank of a mountain range:

	Strength (MPa)	*RQD* (%)	Mean fracture spacing (m)
Sandstone	80	45	0.4
Mudstone	20	75	0.3
Syenite intrusions	250	10	0.2

The fractures within each rock mass type have the properties shown in the following table:

	Persistence (m)	Aperture (mm)	Roughness	Infilling	Weathering
Sandstone	5–8	~1.5	rough	none	none
Mudstone	1.5–2.5	~0.5	slight	none	slight
Syenite	2	~6	very	none	none

Write down a description for each of these three rock mass types, and describe their likely engineering behaviour.

Then apply the *RMR* system to these rock mass types, and compare the assessment of their engineering behaviour made in this way with the description you wrote down earlier.

What do you conclude from this exercise about the ability of *RMR* to discriminate between the engineering behaviour of these particular rock mass types?

A12.9 Descriptions of the rock mass types.

On the basis of the rating summaries given we can describe each rock type as follows.

Sandstone

A strong rock, probably with extensive bedding planes that cause the rock to break into beds that are on average 0.4 m thick. The moderately low *RQD* indicates that a large number of these units will be thinner than 0.1 m, and so a slabby structure should be expected. The roughness of the fractures, together with the appreciable apertures, indicates that the rock mass may be loose.

Mudstone

Very weak, but no evidence of extensive bedding planes. The fractures are tight and only slightly rough, and taken together with the *RQD* of 75% this indicates that the rock mass is not highly fractured — it is probably blocky, with most of the blocks being around 0.3 m in size.

Syenite intrusions

A very strong rock, but with very low *RQD* and mean spacing values. The fractures are very rough but have large aperture. The rock mass probably has a 'sugar cube' type appearance, but may have a high degree of mechanical interlock.

In summary, we have one rock that is very strong but highly broken (syenite), another that is weak but comprising reasonably large blocks (mudstone), with the third somewhere in between (sandstone).

***RMR* ratings**

The *RMR* rating values associated with the fracture summaries for each of the three rock types in the Question are presented in the table below (rating values are given between parentheses).

	Persistence (m)	Aperture (mm)	Roughness	Infilling	Weathering	Total
Sandstone	5–8 (2)	~1.5 (3)	rough (5)	none (6)	none (6)	(22)
Mudstone	1.5–2.5 (4)	~0.5 (4)	slight (3)	none (6)	slight (5)	(22)
Syenite	2 (4)	~6 (0)	very (6)	none (6)	none (6)	(22)

As we can see, the very different fracture descriptions have led to an identical rating value of 22 across all rock types.

Before continuing to determine the basic *RMR* value for each rock type, we need to assess the groundwater conditions. We know that the tunnel passes high through the flank of a mountain range, and from this we can infer that it will probably be located above the groundwater level. Accordingly, we can assess the conditions as 'damp' and assign a rating value of 10 to the groundwater.

Taking the information we have been given about the strength and degree of fracturing, together with the fracture condition and groundwater rating values, we can now obtain the basic *RMR* for each rock type (rating values are given between parentheses):

	Strength (MPa)	*RQD* (%)	Fracture spacing (m)	Fracture condition	Ground-water	Total rating
Sandstone	80 (8)	45 (9)	0.4 (10)	(22)	(10)	(59)
Mudstone	20 (3)	75 (15)	0.3 (9)	(22)	(10)	(59)
Syenite	250 (15)	10 (4)	0.2 (8)	(22)	(10)	(59)

The basic *RMR* rating of 59 is the same for all three rock types. This value of 59 classifies each rock type as being on the boundary of 'fair rock' and 'good rock'.

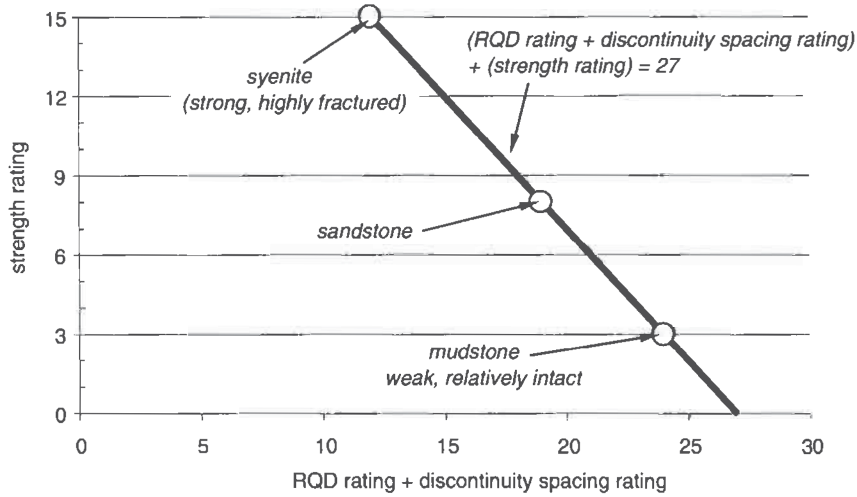
Rock type comparisons

These three rock types which, on the basis of their descriptions, should display very different engineering behaviour, all score the same *RMR* value. It appears therefore that *RMR* is not a good discriminator of engineering behaviour for these rock types.

This is not a deficiency with the *RMR* system in itself, but is more to do with the use of addition to compute a single overall value of *RMR*. This is demonstrated if we examine the ratings associated with strength, *RQD* and fracture spacing for these rock types. It is reasonable to assume that *RQD* and fracture spacing are related to each other, and that they are essentially independent of rock strength. On this basis we can produce the table given below, and then plot the results as shown in the figure.

	Strength	*RQD* + fracture spacing	Partial rating
Sandstone	8	19	27
Mudstone	3	24	27
Syenite	15	12	27

Geometrically, the use of addition to combine values induces what is known as the 'city block metric', so called because it reflects the lengths



of the routes one can walk through a city that is built on a rectangular grid. In the example here, for the partial *RMR* rating value of 27, i.e. a path length of 27 units, there are many paths of this length formed from a combination of a horizontal distance (i.e. combined *RQD* and fracture spacing rating) and a vertical distance (i.e. strength rating), represented by the solid line in the figure. Within the limit of acceptable values for the strength rating (0 to 15), the end points of these paths are shown as the solid line.

The figure shows that the rating value locus of 27 extends from extremely strong and highly fractured rock through to very weak and unfractured rock and, as a result, it is not possible to discriminate between these rock types using *RMR*. In fact, this is true for all classification schemes that depend on a single classification value computed using simple arithmetic; we have chosen *RMR* as the example here.

A technique to overcome this difficulty is to quote classification values as a vector: for *RMR*, there are five parameters and so it would be a five-dimensional vector, and for *Q* it would be a six-dimensional vector. However, one of the problems of adopting such a technique is that it would require the rock engineering community to reinterpret the large database of projects on which these schemes had been used.

Q12.10 The following measurements of mean fracture spacing (in metres) have been made on core from 12 boreholes as part of a site investigation project:

0.259	**0.304**	**0.875**	**0.292**	**0.467**	**0.412**	**0.350**	**0.368**	**0.438**	**0.389**	**0.280**	**0.318**

As the rock mass is to be characterized using the Q system, the following parameters have also been determined: $J_n = 9$; $J_r = 1.5$; $J_a = 2$; $SRF = 2.5$; and $J_w = 1$.

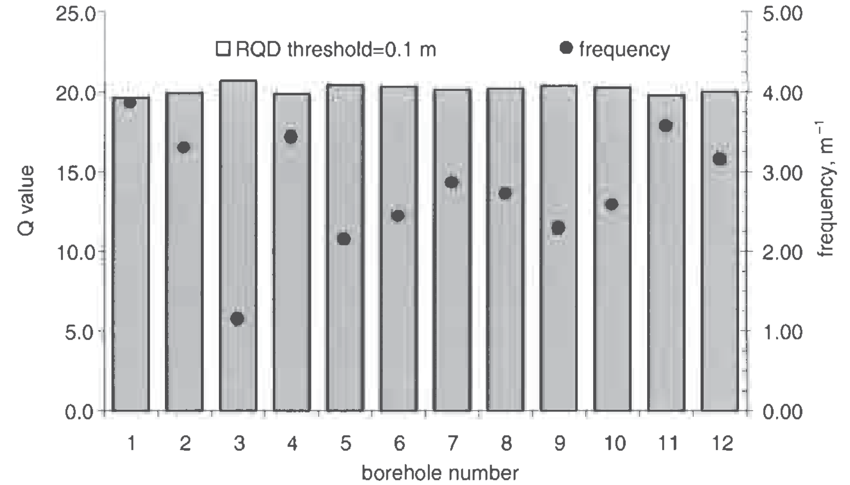
(a) Using the frequency measurements to determine *RQD* values and thence *Q* values with the additional parameters given,

comment on the inhomogeneity of the rock mass in terms of (i) fracture frequency and (ii) Q.

(b) A technique for increasing the range of *RQD* values in a given rock mass is to adopt a different *RQD* threshold value (from the usual value of 0.1 m) computed using
$t^* = 2\ln(\lambda_{max}/\lambda_{min})/(\lambda_{max} - \lambda_{min})$,
where λ_{max} and λ_{min} are the extreme values of the fracture frequency occurring in the rock mass. Use this technique to compute new values of Q, and compare the results with those found in Part (a).

A12.10 (a) In order to compute Q we use the relation $Q = (\mathrm{RQD}/J_n) \times (J_r/J_a) \times (\mathrm{SRF}/J_w)$, and to determine values for *RQD* we use the relation $\mathrm{RQD} = 100(\lambda t + 1)\exp(-\lambda t)$, where λ, the fracture frequency, is given by the reciprocal of the mean spacing. In this relation for *RQD*, the threshold value t is taken to be 0.1 m. The results of the calculations are given in the following table and plotted in the figure below.

Borehole	1	2	3	4	5	6	7	8	9	10	11	12
Spacing (m)	0.259	0.304	0.875	0.292	0.467	0.412	0.350	0.368	0.438	0.389	0.280	0.318
Frequency (m^{-1})	3.861	3.289	1.143	3.425	2.141	2.427	2.857	2.717	2.283	2.571	3.571	3.145
RQD (%)	94.2	95.6	99.4	95.3	98.0	97.5	96.6	96.9	97.8	97.2	95.0	96.0
Q	19.6	19.9	20.7	19.9	20.4	20.3	20.1	20.2	20.4	20.3	19.8	20.0



We can see immediately from the plot that the rock mass is significantly inhomogeneous in terms of fracture frequency (for these data, the ratio of maximal frequency to minimal frequency is 3.38), and that this information is lost when we consider Q values (the ratio of maximal Q to minimal Q is only 1.06). Clearly, Q is not sensitive to changes in fracture frequency (or its reciprocal fracture spacing).

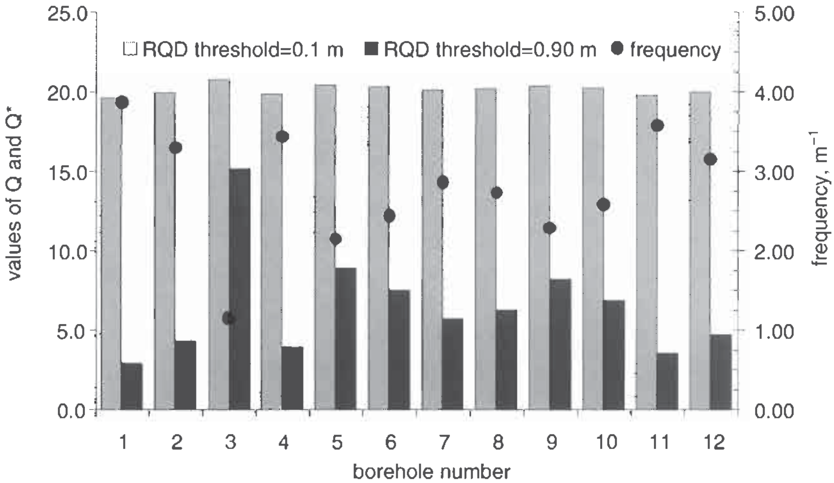
(b) On the basis of the site investigation results, the minimal and maximal values of fracture frequency in the rock mass are 1.143 m^{-1} and 3.861 m^{-1}, respectively, and from these we find that the appropriate value of RQD threshold is

$$t^* = \frac{2}{\lambda_{max} - \lambda_{min}} \ln\left(\frac{\lambda_{max}}{\lambda_{min}}\right) = \frac{2}{3.861 - 1.143} \ln\left(\frac{3.861}{1.143}\right) = 0.90\,\text{m}.$$

The new values of RQD and Q — called RQD^* and Q^* to distinguish them from the customary formulations — are as shown in the following table.

Borehole	1	2	3	4	5	6	7	8	9	10	11	12
Spacing (m)	0.259	0.304	0.875	0.292	0.467	0.412	0.350	0.368	0.438	0.389	0.280	0.318
Frequency (m^{-1})	3.861	3.289	1.143	3.425	2.141	2.427	2.857	2.717	2.283	2.571	3.571	3.145
RQD^* (%)	14.0	20.7	72.7	18.9	42.9	36.1	27.5	30.1	39.4	33.0	17.1	22.8
Q^*	2.9	4.3	15.1	3.9	8.9	7.5	5.7	6.3	8.2	6.9	3.6	4.8

These results for Q^*, together with the results from the earlier calculation, are plotted in the figure below. We can see from this that Q^* has a greater discrimination than Q with regard to the inhomogeneity, at the expense of much reduced values.



Although this technique has improved the sensitivity of the classification values, the reduced values of Q^* in comparison to Q show that we have effectively developed a completely new classification scheme, and as such we cannot use any of the customary correlations between Q and engineering behaviour. Thus, this technique must only be used to improve the sensitivity of a classification scheme when the scheme itself is being used to delineate different zones of rock properties, and not yet, for example, for selection of support and reinforcement.

12.3 Additional points

A substantial extension to the RMR system has been made by Romana (1993)[5] with his SMR system for assessing slope stability. The SMR value

[5] Romana M. R. (1993) A geomechanical classification for slopes: slope mass rating, in *Comprehensive Rock Engineering*, Vol. 3, Ch. 23 (J. A. Hudson, ed.), Pergamon Press, Oxford, pp. 575–599.

can be written in the form

$$\mathrm{SMR} = \mathrm{RMR} - F_g + F_e,$$

where F_g is a factor representing the geometry of the potential instability present in a rock slope, and F_e is a factor corresponding to the excavation method. Geometries that are intrinsically more unstable have higher values of F_g (and hence reduce *SMR*), and excavation methods that induce little perturbation in the rock mass have high values of F_e (and hence increase *SMR*).

Additional work on applying classification systems to slope stability has been reported by Sonmez and Ulusay (1999)[6]. In this paper, it is noted that the rock mass classification should refer not to the rock mass in its undisturbed condition but to the excavation-disturbed rock mass — which is the one that hosts the rock engineering structure. The authors suggest methods for assessing the excavation disturbance effect on rock mass classification values.

The **advantages** of using rock mass classification systems are that
- the rock mass quality can be assessed simply, rapidly and continuously,
- the classification values can be established by trained site personnel (i.e. a high level of general engineering expertise is not required),
- continuous rock mass assessment using logging sheets will alert contractors and consulting engineers to significant changes in rock conditions, and
- engineering design is coherently based on previous experience.

The **disadvantages** of using a rock mass classification system are that
- the systems currently in use are historical and idiosyncratic,
- the algebra and ratings values of the systems have not been scientifically considered, and
- they cannot be used for the full range of engineering objectives.

[6] Sonmez H. and Ulusay R. (1999) Modifications to the geological strength index (GSI) and their applicability to stability of Slopes. *Int. J. Rock Mech. Min. Sci.*, **36**, 743–760.