Ground classification systems in tunnel construction

Prepared for Quality Services (Civil Engineering), Highways Agency

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Ground classification systems play an important role in setting up and managing tunnel construction contracts because they may determine the level of payment to the contractor. However because of the rapid rate of development in tunnel design, currently employed methods may no longer be the most efficient. The classification of the ground for tunnel support is an area where improvements may be possible in the planning and execution of future tunnelling contracts.

This study was commenced in December 1994 to review this area and identify the best route forward. The objective is to give guidance to the Highways Agency by indicating an approach to ground classification which will assist in setting-up cost-effective contracts for new tunnelling works.

This report reviews the development and use of empirical ground classification systems for the selection of tunnel support. Recent UK and other practice is examined to identify new developments and note particular problem areas. The concept and nature of the systems is discussed in the first section. Recent literature is then examined and discussed, followed by a review of UK case histories of tunnelling where ground classification schemes were used. Final sections draw together the most important conclusions to be gleaned from the evidence and some recommendations are made on the most profitable direction of future research.

Ground classification systems have been developed over the past fifty years and have become increasingly sophisticated. They provide an empirical method, based on the analysis of case histories, of assisting with the selection of tunnel support systems. They can be of use during the site investigation, design and construction phases of a tunnelling project. During site investigation a classification system can be used to define the appropriate parameters of the ground to be measured to enable an adequate classification to be carried out. During the design phase preliminary selection of the support for sections of tunnel can then be made by the use of the chosen classification system. This should always be checked and refined by the use of an analytical method. During construction the classification system can be applied to the ground conditions encountered and used to assist with selection or modification of the support to be used for each advance of the face. Payment and settlement of any disputes between the contract parties can be facilitated by the comparison of the ground classifications agreed at the tunnel face, and specified ‘ground reference conditions’ based on the pre-construction site investigation data. A form of Contract which allows flexibility in support choice and an equitable sharing of risk, is needed to facilitate this.

The most popular systems in use are the ‘Q’ system, developed at the Norwegian Geotechnical Institute (Barton, 1974), and the Rock Mass Rating (RMR) system, or Geomechanics’ system developed by Bieniawski (1973). The ‘Q’ system has been shown to be generally less conservative and more sensitive to changes in rock quality than the RMR system. The system was originally developed using just over two hundred case histories, mainly in Scandinavia. It is now backed by a database of over one thousand case histories in a wide variety of ground types and support systems, from many different countries. The ‘Q’ system utilizes the classification parameters RQD (a measure of joint spacing), number of joint sets, joint roughness, joint condition or alteration, groundwater inflow and stress condition. The individual parameters are multiplied to give the rock quality ‘Q’, which ranges from about 0.001 to 1000. This figure can be used to give an expected stand up time for specific unsupported spans and together with the effective span and a factor of safety, used to select an appropriate support design. Correlations with ‘Q’, which allow estimation of deformation modulus and mass strength have been developed. These are properties which are needed for the use of analytical methods and are otherwise difficult to determine, requiring large scale tests.

A good correlation between ‘Q’ and seismic velocity has been observed in many types of ground. The measurement of seismic velocity during site investigation and when probing ahead from the tunnel face is a promising technique. However more data is required, in particular on the effect of overburden stress, before this technique can be used with confidence.
1 Introduction

This report reviews the development and use of empirical ground classification systems for the selection of tunnel support. Recent UK and other practice is examined to identify new developments and note particular problem areas. The basic concept and nature of the systems are discussed in the first section. The most recent literature on these systems is then examined and discussed, followed by a review of UK case histories of tunnelling where ground classification schemes were used. Final sections draw together the most important conclusions to be gleaned from the evidence and some recommendations are made on the most profitable direction of future research.

2 Ground classification systems

2.1 Introduction

The process of design of a tunnel support system to resist ground loading includes several steps:

1. Characterisation of the ground mass.
2. Definition of the ideal geometry of the scheme.
3. Consideration of the ground/lining interaction effect on support load.
4. Selection of lining type and design of lining.

The process will often be an iterative one, particularly as at early stages in the design process there may be very little site investigation data available. Where the observational method is used the design of the lining may be changed as construction proceeds, based on measured loads and deformations. The type of lining chosen may additionally be influenced by constructional and operational requirements. The former could be environmental restrictions on the construction method and physical or economic limitations on plant, and the latter a requirement for a smooth bore. Ground classification systems provide an empirical method, based on the analysis of case histories, of applying the information obtained in steps 1 and 2 above to the design considerations in steps 3 and 4. In any major tunnelling scheme the empirical design should be backed up by the use of analytical methods and engineering judgement. It is possible that such analytical methods will eventually supersede the empirical methods. However the complex nature of the problem makes this unlikely in the near future. Ground classifications can be used both at the design stage using data from the site investigation, and during construction to assist the selection or modification of support at the tunnel face, based on the conditions actually encountered.

It should be noted that the application of these systems at the design stage is strongly dependent on the quality of the site investigation data available. If the site investigation is inadequate then it is inevitable that any design using rock classification, or any other method, will be inaccurate.

2.1.1 Classification by strength

In tunnelling there is a traditional classification of the ground into soft ground and rocks of various strengths. Soft ground can usually be excavated by hand or mechanical digging equipment and requires immediate or very rapid support. Soft ground mainly comprises recent alluvium and glacial drift deposits and also the stiff fissured clays of the Eocene, Cretaceous and Jurassic periods. The scales of strength given in BS5930 (British Standards Institution, 1981), for clays and rocks, can also be useful in discriminating between soft ground and rock, although there is undoubtedly some overlap. In the British Standard ‘Hard clay (or Very weak mudstone)’, is defined as having an undrained shear strength greater than 300kPa. In the section on rocks ‘very weak rock’ is defined as having an undrained shear strength greater than 1.25 MPa.

2.1.2 Choice of support

Tunnel support systems may be chosen in several ways, the choice of which will depend on the type of ground, the size and purpose of the excavation, and the pre-existing stresses in the ground. The latter are mainly dependent on the depth of overburden, but may be affected by adjacent tunnels.

2.1.2.1 Soft ground

In ground classified as soft, tunnels are normally constructed by the use of a tunnelling shield and a segmental lining of cast iron or concrete, although recently unshielded methods using sprayed concrete linings have been used in the London Clay. Research has shown that in soft ground hoop loads in linings rarely exceed a value equivalent to the full overburden pressure, and typically reach about 75% of it in the long term. Design in these conditions is therefore usually based on the full overburden pressure. This may include an allowance for dead loading at the surface. It will not normally be necessary to consider live loading unless there is very little cover. The ‘full overburden pressure’ criterion is based on measurements made in a number of segment lined tunnels in the London Clay and in particular measurements made by TRL at Regents Park on the Jubilee line (Barratt et al, 1994). This paper concluded that it is highly unlikely that the ring loads on segmental linings of either cast iron or concrete ever exceed those corresponding to an all-round pressure equal to that due to the overburden. More recent measurements supporting this view have been made by TRL at St James’ Park on the Jubilee line extension (Bowers and Redgers, 1996). The hoop load reached, and its rate of increase in individual cases, will depend on the time between excavation and support, the stiffness of the support and its degree of contact with the ground. An overriding design consideration is likely to be the control of surface settlement, particularly as most soft-ground tunnels are at shallow depth in urban areas. This will usually require heavier support than that required to stabilize the opening when settlement control is not important. The highest and earliest hoop loads will occur...
when minimization of settlement is required and a stiff lining is erected quickly after excavation, with intimate ground contact. In order to provide a stiff lining it is likely that it will have a large cross-sectional area which will result in relatively low lining stresses.

2.1.2.2 Rock
In rock the situation is somewhat different because, depending on the engineering characteristics of the rock and the size and depth of the tunnel, the support needed may vary from none to a heavy mass concrete lining, depending on the ability of the ground to support itself. Great strides have been made in recent years in the refinement of analytical methods for tunnel design based on the principles of rock mechanics. In particular the severe shortcomings in the use of finite element and other numerical methods to analyse discontinuous rocks are being reduced by the development of discrete element methods. However, due the variable nature of the ground and difficulties with adequately defining the parameters of a ground model, empirical rock classification systems for tunnel support estimation are often resorted to both for preliminary design and to guide support decisions during construction. In effect such systems allow the systematic application of past experience to new tunnels. It is therefore important that the classification used is based on a range of case histories which encompass the conditions to be encountered. It should also be recognised that because of the highly variable nature of the ground such empirical design cannot be more than an approximate guide to support selection and should never be totally relied upon. Analytical studies, field observations and engineering judgement by experienced personnel must also be used. It should also be noted that because it is impossible to determine the factor of safety in the case histories upon which these empirical methods are based their use may tend to perpetuate excessive conservatism.

2.1.3 The concept of classification systems
Bieniawski’s book ‘Engineering Rock Mass Classifications - A complete manual for engineers and geologists in mining, civil and petroleum engineering’ (Bieniawski, 1989) provides a good overview of the history and use of rock classification systems. The concept embodied in rock classification systems for tunnel support is to identify the set of parameters of the rock mass which are best correlated with its engineering behaviour and to assign numerical weights to the likely range of each parameter. The parameters chosen must have values which are readily obtainable from site investigation data. The parameter weights are derived by adjusting them to best fit available case history data. Finally the values of parameters are combined to produce a single rating parameter representing the ‘quality’ of the rock. The full range that is possible for this quality rating figure can then be used to assign descriptive classes to the rock each associated with, in conjunction with the size and purpose of the excavation, a suitable type of support. In Barton (1988) the ‘Q’-system of rock classification is likened to an ‘expert system’ based on the knowledge and experience of an expert tunnelling consultant.

2.2 Classification parameters
A rock mass generally comprises intact blocks separated by discontinuities such as joints, bedding planes and faults. The strength and stiffness of the rock mass, and its stability when excavated, are reduced by the presence of the discontinuities. The properties of the intact rock are typically less important than the properties of the discontinuities in determining the overall properties of the rock mass, except in cases where rock stresses are approaching the intact rock strength. In shallow tunnels the strength/stress ratio is only likely to be low in the weakest materials. The quality and type of classification data available will depend on whether the work is at the site investigation phase or the construction phase. A description follows of each of the properties of a rock mass which may be quantified as part of a rock classification system, with an indication as to the method of measurement. The determination and recording of each of these parameters is covered in detail by the publications of the International Society for Rock Mechanics (ISRM) Commission on Testing Methods, most of which have been brought together in one volume (International Society for Rock Mechanics, 1981).

It should be remembered that even with the best site investigation there will be considerable uncertainty as to the values of the parameters due to the inevitable variability of the ground, the small size of the sample taken and errors in measurement. Probabilistic methods attempt to accommodate this variability by calculating the probability distribution of the output of a classification system from the measured or assumed probability distributions of the inputs. This subject will be examined in section 3.5.

2.2.1 Intact rock strength
The intact rock strength can be assessed by uniaxial compressive strength tests on prepared cores in the laboratory, or by the point load test (ISRM, 1985) on sections of cores or irregular lumps in the field. The latter test produces an index value which can be correlated to uniaxial compressive strength. The Schmidt rebound hammer and sonic velocity tests may also be used to give index values related to strength and elastic modulus.

2.2.2 Number of joint sets
There is normally more than one set of joints in a rock mass (Figure 1). Spacing data for each significant joint set is required to determine the block size. The size and orientation of blocks relative to the size and orientation of the tunnel is an important factor in determining tunnel stability. It is difficult to assess this parameter from borehole core, an exposure is needed.
2.2.3 Discontinuity spacing

The mean discontinuity spacing can be obtained from both exposures and borehole core. The inverse is often quoted as the Fracture Index or Frequency for lengths of core with a similar intensity of fracturing. The Fracture Index or Frequency is defined as the number of discontinuities per unit length of core. In the case of cores it is important that fractures caused by the drilling operation are not counted. When measuring discontinuity spacing on an exposure it is usual to count the discontinuities intersecting a straight line or lines, commonly denoted a ‘scanline’, defined on the surface (Figure 2). It is important to realise that the orientation of the borehole or scanline relative to the orientation of the sets of discontinuities present is likely to affect the result. Ideally the spacing of each set of discontinuities should be measured normal to its plane. This is likely to require boreholes or scanlines in more than one direction. Priest and Hudson (1979) suggest that scanlines, and therefore by implication borehole core, should be at least 50 times longer than the mean discontinuity spacing to yield results ‘to a reasonable precision’. In many cases, particularly in boreholes, the length available will be less than this, introducing a degree of error into the result.

The Rock Quality Designation (RQD) introduced by Deere et al (1967) is a very commonly used index of rock fracturing, which was developed for core logging. The RQD is the percentage of length of core or scanline consisting of intact pieces longer than 0.1m. Priest and Hudson (1976) have observed that the distribution of discontinuities in most rock closely follows the negative exponential distribution, so in this case there is a functional relationship between RQD and mean spacing:

$$\text{RQD} = 100e^{-0.1\lambda}(0.1\lambda + 1); \text{ where } \lambda = \text{mean discontinuity spacing}$$

They also observed that the standard RQD with a threshold value of 0.1m is insensitive to variations in rock quality when the average discontinuity spacing exceeds 0.3m. A additional RQDₚ₁₀ index with a threshold of 1.0m is suggested to extend the range to a discontinuity spacing of about 2.5m, although this idea does not seem to have been adopted in any classification system.
2.2.4 Discontinuity condition

Some of the most important characteristics of discontinuities are aperture, filling (Figure 3), roughness (Figure 4) and wall strength. Persistence may also be important. Unfortunately filling and aperture in particular cannot be obtained from cores, although downhole inspection with a miniature TV camera may yield this information.

2.2.5 Discontinuity orientation

The orientation of discontinuities can be an important parameter in tunnel stability. This can be obtained from core if special precautions are taken, but more reliably from exposures or orehole inspection. The International Society for Rock Mechanics (1981) describe three methods for obtaining orienting core. One depends on matching successive sections of a run of core, one pre-reinforcement of core with a grouted reinforcing rod and the last scribing of the core and the use of a compass photo device. Borehole inspection with a CCTV camera, yielding orientation data, is now possible to depths exceeding 1000m. A more recent technique is the ‘Acoustic Borehole Televiewer’ (Siddans, 1995) which utilises a scanning acoustic reflection technique in a water filled hole, together with computer processing of the data to provide an ‘unwrapped’ oriented chart on which, for example, a plane discontinuity shows as a sinusoidal trace.

2.2.6 Groundwater conditions

Groundwater conditions depend on the position of the water table and the permeability of the rock. The latter is dependent on the properties of the discontinuities although the intact rock may be permeable in some cases. For classification purposes groundwater conditions are often expressed as the ratio of water pressure to major principal stress or inflow rate per unit length of tunnel.

2.3 Classification systems

The empirical rock classification systems discussed in this report are intended to assist with the selection of tunnel support at both the initial design phase and during excavation. They are attempts to provide a) a standardised method, b) data based on the accumulated experience of large numbers of case histories and c) a method of defining and processing the site investigation data necessary for the selection of tunnel support. For this reason it is important that they are not used outside the range of the case histories analysed in their development, and that they are not used in isolation without appropriate engineering judgement. They are introduced here in chronological order of their development.

2.3.1 Rock load classification

Terzaghi’s rock load classification (Terzaghi, 1946) is generally accepted as the first rock classification system formulated to evaluate rock loads in tunnels supported by steel arches. Rock load was expressed in terms of the height and the width of the tunnel for nine classes of rock.

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Figure 3 Examples of discontinuity condition (after Brown, 1981)
These classes were originally purely qualitative, ranging from ‘1) Hard and intact’, which contains no discontinuities, to ‘9) Swelling rock’ which is rock containing clay minerals which expand on wetting. Later modified versions included an RQD range for each class. This system has been extensively used in the USA but has been shown to be excessively conservative in many situations. More recently developed systems are more satisfactory because they permit a more qualitative assessment of the range of factors which control rock behaviour and also permit the selection of support systems based on rock bolts and sprayed concrete as well as steel arches.

2.3.2 ‘Stand-up time’ concept
The idea that the stand-up time for an unsupported heading is dependent on the rock mass class and the unsupported span is attributed to Lauffer (Lauffer, 1958). The larger the unsupported span and the poorer the rock class, the shorter is the stand-up time. This concept is central to the more recent rock mass classification systems.

2.3.3 Rock quality designation (RQD)
The RQD concept was introduced in section 2.2.3. It was introduced by Deere (1967) as an improvement on the ‘percentage core recovery’ as an index of rock quality. The International Society for Rock Mechanics recommends that core should be at least NX (54.7mm) diameter and drilled with double-tube core barrel. Descriptive terms were assigned as shown in Table 1. Some early attempts were made to correlate RQD with support requirement. However, because of the importance of the other properties of discontinuities (see sections 2.2.4, 2.2.5) it is now used as an index property in more comprehensive classification systems. Two main drawbacks to the use of RQD identified by Priest and Hudson (1976) are:
1 calculating RQD is a time consuming process, which, when applied to borehole core, can give results unrepresentative of the rock mass;
2 the conventional RQD is insensitive to variations in rock quality when the average discontinuity spacing is greater than 0.3m.

2.3.4 Rock structure rating (RSR)
Wickham et al (1972) introduced the Rock Structure Rating (RSR), as a partly quantitative, nine-parameter, weighted classification system for determining rock quality. Support prediction is a function of the RSR and the steel arch support given by Terzaghi’s rock load. This system is essentially an enhancement to Terzaghi’s rock load method, which allows the incorporation of the quality of the rock mass and a corresponding reduction in conservatism of support predictions. The rock mass parameters included are grouped as in Table 2.

![Figure 4 Examples of joint roughness classifications](image)

These classes were originally purely qualitative, ranging from ‘1) Hard and intact’, which contains no discontinuities, to ‘9) Swelling rock’ which is rock containing clay minerals which expand on wetting. Later modified versions included an RQD range for each class. This system has been extensively used in the USA but has been shown to be excessively conservative in many situations. More recently developed systems are more satisfactory because they permit a more qualitative assessment of the range of factors which control rock behaviour and also permit the selection of support systems based on rock bolts and sprayed concrete as well as steel arches.

### Table 1 Rock quality designation

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<tr>
<th>RQD%</th>
<th>Rock Quality</th>
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<tr>
<td>&lt; 25</td>
<td>Very poor</td>
</tr>
<tr>
<td>25-50</td>
<td>Poor</td>
</tr>
<tr>
<td>50-75</td>
<td>Fair</td>
</tr>
<tr>
<td>75-90</td>
<td>Good</td>
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<tr>
<td>90-100</td>
<td>Excellent</td>
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### Table 2. Rock Structure Rating parameters

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<th>Group</th>
<th>Parameter</th>
<th>Range</th>
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<tbody>
<tr>
<td>A</td>
<td>Rock type (3 grades)</td>
<td>6 - 30</td>
</tr>
<tr>
<td></td>
<td>Hardness (4 grades)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Geological structure (folding, faulting, 4 grades)</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>Joint spacing (6 grades)</td>
<td>7 - 45</td>
</tr>
<tr>
<td></td>
<td>Joint orientation relative to drive to drive (8 grades)</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>Joint condition (3 grades)</td>
<td>6 - 25</td>
</tr>
<tr>
<td></td>
<td>Water inflow (4 grades)</td>
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</table>

All the previously considered parameters (see sections 2.2.1 to 2.2.6) are included. Each group is given a single numerical value selected from a matrix of values for all the parameters in the group. The three group values which result are then added to give the RSR, which has a maximum value of 100. An adjustment factor is tabulated to convert the standard RSR to a value applicable to machine bored tunnels. Possible criticisms are that the joint condition parameter only allows three ratings, ‘good’, ‘fair’ and ‘poor’, and the ‘rock type’ classes are qualitative and could be open to misinterpretation. Ninety percent of case histories were for steel rib supported tunnels, so the...
predictions are based on this type of support. To correlate RSR with support the Rib Ratio (RR) is defined. This is the ratio of the actual support spacing, in the 190 case records, to that given by standard steel arch tables, derived using Terzaghi’s rock load method, for the same tunnel diameter. The plot of RSR against RR for the case records yields the mean relation $(\text{RSR}+30)(\text{RR}+80)=8800$, although there is considerable scatter, suggesting the use of a cautious approach in its use.

Use of this method therefore requires determination of the RSR and the corresponding RR. The support given by Terzaghi’s method, is then modified by the RR to give the final support. Although it is possible to calculate equivalent rockbolt and/or shotcrete support from the predicted rock load this is not supported by sufficient case history data of these support types for the use of this classification system to be recommended with them.

### 2.3.5 Geomechanics classification or Rock Mass Rating system (RMR)

The Rock Mass Rating (RMR) system, sometimes also known as the Geomechanics System (Bieniawski, 1973), is one of the two systems which are currently the most used. A full description including various enhancements which have been developed since and taking account of further case histories (numbering 351 in 1989) is given in Bieniawski’s book (Bieniawski, 1989). Extensions to the original system by various authors covering mining, ripping, foundation design and slope stability, are described in the book. There are six parameters in the current system:

1. Uniaxial compressive strength.
2. RQD.
3. Discontinuity spacing.
4. Discontinuity condition.
5. Orientation of discontinuities.

The range of each of these parameters is subdivided into five classes, each of which is assigned a tabulated numerical weighting depending on its relative importance. The ranges of the rating parameters used conform to the ISRM (1978, 1981) method. The weights were originally derived from those used by Wickham et al (1972) in the RSR system. To use the system the ratings for each parameter are determined from a table and added to give the RMR, which lies in the range 0-100. A separate table, based on Wickham et al (1972), is given to assist with the classification of discontinuity strike and dip directions relative to the tunnel axis. The joint orientation ratings are different for tunnels, foundations and slopes and are given as a negative adjustment to the RMR for each of five classes. It is assumed that there are three joint sets, the most unfavourable of which should be considered. It is suggested that when there are only two joint sets that the ratings for joint spacing should be increased by 30%. A subsidiary chart is presented which subdivides the ratings for joint condition across the sub-parameters; persistence, aperture, roughness, infilling and weathering. Other charts present smooth curves for the interpolation of values of strength, RQD and spacing between those given in the main table. For mining it is stated that further adjustments for stress, blasting damage and major faults or fractures may be called for.

A peculiarity of this classification system is that both RQD and discontinuity spacing are included as parameters, although they are strongly correlated, as described by Priest and Hudson (1976). Bieniawski goes as far as to present a chart based on Priest and Hudson’s relationship which is to be used if either RQD or spacing is missing.

The final rating (0-100) puts the rock into one of five categories (Table 3), each with a span/stand-up-time characteristic which is plotted on a span-stand-up time chart (Figure 5). This chart could be used, for example to assess the maximum unsupported advance which could be made in a given class of rock. Guidelines for excavation and support are tabulated for steel ribs, rock bolts and sprayed concrete, for each of the five classes. It is specifically stated that these support recommendations are for permanent support in tunnels constructed by conventional drill and blast. A similar stand-up time/unsupported span chart is presented for tunnel boring machine (TBM) excavation, adjusted for the difference in rock damage between these two excavation modes. This highlights a potential problem with the use of rock classification systems, that the rock quality assessed from site investigation boreholes and surface exposures is likely to differ from that measured during construction, particularly by drill and blast. Bieniawski suggests a blasting damage adjustment to RMR of 0.8 to 1.0 for mining applications which should perhaps also be applied to civil engineering tunnelling.

### Table 3 RMR classes

<table>
<thead>
<tr>
<th>Class</th>
<th>Rating no.</th>
<th>Rock Description</th>
<th>Average stand-up time</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100-81</td>
<td>Very good</td>
<td>20 years (15m span)</td>
</tr>
<tr>
<td>2</td>
<td>80-61</td>
<td>Good</td>
<td>1 year (10m span)</td>
</tr>
<tr>
<td>3</td>
<td>60-41</td>
<td>Fair</td>
<td>1 week (5m span)</td>
</tr>
<tr>
<td>4</td>
<td>40-21</td>
<td>Poor</td>
<td>10 hours (2.5m span)</td>
</tr>
<tr>
<td>5</td>
<td>&lt;20</td>
<td>Very Poor</td>
<td>30 min (1m span)</td>
</tr>
</tbody>
</table>

The following useful correlations are also presented:

1. between deformation modulus and RMR;
2. between rock mass strength (Hoek and Brown, 1980,1988) and RMR for both ‘disturbed’ (eg. blast damaged) and ‘undisturbed’ (eg. machine bored) rock;
3. between rock load and a function of RMR rock density and tunnel width;
4. between RMR and ‘Q’ based on 111 case histories;
5. between RSR and RMR based on 7 case histories in New Zealand.

These correlations should be used with caution as the scatter is considerable. The output of the RMR method is stated to tend to be conservative, which should be addressed by monitoring deformations during construction and adjusting the design as necessary.
2.3.6 Q - system

Barton et al (1974) developed the ‘Q’ system based originally on an analysis of 212 mainly Norwegian case histories of tunnelling in hard rock. Since that time there has been considerable development and revision to include new support types and about 1050 extra case history records (Barton and Grimstad, 1994a), bringing the total to about 1262. These cover a wide range of rock types and support methods, in many different countries. However the system has required virtually no re-calibration of the rock quality ratings since its original development. There are six parameters in this scheme.

1. RQD.
2. Number of joint sets (Jn).
3. Joint roughness (Jr).
4. Joint condition - or alteration (Ja).
5. Groundwater inflow (Jw).
6. Stress condition. (Stress reduction factor(SRF)).

The value for each rating parameter is selected from the table shown in Figure 6. The rock quality, ‘Q’, is calculated as follows:

$$Q = \frac{RQD \cdot J_r \cdot J_w}{J_n \cdot SRF}$$

and lies in the approximate range 0.001 to 1000. The three ratios represent block size, inter-block shear strength and active stress. Barton (1988) considers that the ‘Q’ system provides a much more detailed assessment of joint roughness, filling (alteration) and relative orientation than any other system. The joint roughness is rated on a seven point scale from ‘discontinuous joints’ to ‘slickensided planar’ and the joint condition is rated on a sixteen point scale from ‘tightly healed’ to ‘thick continuous’ zones of swelling clay’. Although joint orientation is not included it is implicit because the joint parameters are to be applied to the most unfavourable joint set.

The ‘stress reduction factor’ has 16 classes falling into four groups:
1. zones of weakness causing loosening or fallout.
2. rock stress problems in competent rock.
3. squeezing or flow of incompetent rock.
4. swelling rock.

The ratio of rock stress to rock strength is only considered in the ‘competent rock - rock stress problems’ group. This is appropriate because, as previously discussed in section 2.2, the rock stress is only important when it is a significant proportion of the rock strength.

The ‘excavation support ratio’ (ESR) reflects the desired stand-up-time and can be thought of as a risk-related safety factor. It is selected from a table according to the use of the excavation. Suggested figures range from approximately 3 to 5 for ‘temporary mine openings etc.’ to approximately 0.8 for public facilities, although the confidence in these extreme values is stated not to be high because there are only two case histories relating to each. The ‘equivalent dimension’ (ED) is then given by

$$ED = \frac{\text{span or height}}{ESR}$$

In this equation the choice of span (or diameter) is appropriate to roof support, and height to wall support. Support is then chosen from 38 tabulated basic categories.
### 2. Joint Set Number

<table>
<thead>
<tr>
<th>Joint Set Number</th>
<th>J&lt;sub&gt;j&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.5 - 1.0</td>
</tr>
<tr>
<td>B</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>3</td>
</tr>
<tr>
<td>D</td>
<td>4</td>
</tr>
<tr>
<td>E</td>
<td>6</td>
</tr>
<tr>
<td>F</td>
<td>9</td>
</tr>
<tr>
<td>G</td>
<td>12</td>
</tr>
</tbody>
</table>

Note: For intersections, use (0.3 x J<sub>1</sub> + J<sub>2</sub>). For parallel, use 2.0 x J<sub>1</sub>.

### 3. Joint Roughness Number

<table>
<thead>
<tr>
<th>J&lt;sub&gt;r&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
</tr>
<tr>
<td>B</td>
</tr>
<tr>
<td>C</td>
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<tr>
<td>D</td>
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<td>E</td>
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<td>F</td>
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<tr>
<td>G</td>
</tr>
</tbody>
</table>

Note: Descriptions refer to small scale features and intermediate scale features, in that order.

### 4. Joint Alteration Number

<table>
<thead>
<tr>
<th>J&lt;sub&gt;a&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
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<tr>
<td>B</td>
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<tr>
<td>C</td>
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<td>D</td>
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<td>H</td>
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<td>I</td>
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<tr>
<td>J</td>
</tr>
<tr>
<td>K</td>
</tr>
<tr>
<td>L</td>
</tr>
</tbody>
</table>

Note: Add 1.0 if the mean spacing of the relevant joint set is greater than 3m. J<sub>a</sub> = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented for minimum strength.

### 5. Joint Water Reduction Factor

<table>
<thead>
<tr>
<th>J&lt;sub&gt;w&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
</tr>
<tr>
<td>B</td>
</tr>
<tr>
<td>C</td>
</tr>
<tr>
<td>D</td>
</tr>
<tr>
<td>E</td>
</tr>
<tr>
<td>F</td>
</tr>
</tbody>
</table>

Note: Factors C to F are crude estimates. Increase J<sub>w</sub> if drainage measures are installed.

### 6. Stress Reduction Factor

<table>
<thead>
<tr>
<th>SRF</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
</tr>
<tr>
<td>B</td>
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<tr>
<td>C</td>
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<td>D</td>
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<tr>
<td>J</td>
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<tr>
<td>K</td>
</tr>
<tr>
<td>L</td>
</tr>
<tr>
<td>M</td>
</tr>
<tr>
<td>N</td>
</tr>
</tbody>
</table>

Note: Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.

### Figure 6 Ratings for the six 'Q' - system parameters (after Barton and Grimstad, 1984a)

- **Rock Quality Designation**
  - A: Very poor
  - B: Poor
  - C: Fair
  - D: Good
  - E: Excellent

- **Joint Water Reduction Factor**
  - J<sub>w</sub>

- **Stress Reduction Factor**
  - SRF
which depend on ‘Q’ and ED. The support categories are further subdivided depending on tabulated critical values of combinations classification parameters. Barton et al (1974) and Barton, (1988) both include a chart of ED versus ‘Q’ on which the 38 support categories are plotted. Barton and Grimstad (1994a) give a similar chart (Figure 7) showing the applicable regions for the various general support types including the more recent fibre reinforced sprayed concrete. The chart clearly shows the boundary beyond which no support is required. Relationships for bolt length and maximum unsupported span in terms of ‘Q’, and roof pressure in terms of ‘Q’ and Jr., can be found in Barton et al (1974) and Bieniawski (1989).

Several authors have correlated ‘Q’ values with RMR values with widely differing results. A graph of the regression RMR=9 ln ‘Q’ + 44 for the data from 111 case histories is given by Bieniawski (1976), from which it is apparent that there is considerable scatter. The 90% confidence limits are at ±18. It is evident that the use of such correlations is not likely to yield a very accurate conversion from one system to the other.

2.3.7 New Austrian Tunnelling Method (NATM) classification

Bieniawski (1989) includes a section on the ‘NATM classification’. The NATM is stated to be a design philosophy rather than a ‘method’. An essential component is the continuous observation of lining loads and deformations and subsequent modification of the support. Thus the NATM is an application of the ‘Observational method’ to tunnelling. The NATM also aims to mobilise the strength of the rock mass by placing suitable support at the correct time. These support measures are determined by the application of a rock mass classification system. In order for such a system to be workable contractually, payment must be based on rock mass classification at the working face after each advance.

It is stated that when using the NATM the ground should be classified according to a scheme drawn up for each individual site and which should form part of the contract. There would be a small number of classes, each with its appropriate basic support type. An example of seven classes is tabulated in which the ground is described.

![ROCK MASS CLASSIFICATION](image)

**REINFORCEMENT CATEGORIES:**

1) Unsupported
2) Spot bolting, sb
3) Systematic bolting, B
4) Systematic bolting, (and unreinforced shotcrete, 4-10 cm), B(+S)
5) Fibre reinforced shotcrete and bolting, 5-9 cm, Sfr+B
6) Fibre reinforced shotcrete and bolting, 9-12 cm, Sfr+B
7) Fibre reinforced shotcrete and bolting, 12-15 cm, Sfr+B
8) Fibre reinforced shotcrete >15 cm, reinforced ribs of shotcrete and bolting, Sfr,RRS+B
9) Cast concrete lining, CCA
qualitatively and a basic support type assigned to each. This approach, which is still relatively novel in the UK, would be facilitated by a type of contract which defines a simple method for the resolution of any disagreements over support. Contractors and engineers experienced in these techniques are also essential.

Austrian Standard Önorm B2203 (Österreichisches Normungsinstitut, 1994) contains a purely qualitative rock classification. This describes three main classes of rock, subdivided into ten sub-classes from ‘Stable Rock’ to ‘Flowing Ground’. This classification system is based on Austrian, and in particular Alpine, tunnelling conditions. It is the only formalised system specifically developed for NATM tunnels. For each of the classes qualitative advice is given on the generic type of support required, and the effect of the support on progress. No information is given on how the classifications should be measured, either at the site investigation stage, or during tunnelling.

Sauer (unpublished report to TRL) considers that there are three subjects to be classified for NATM tunnels. These are rock or soil, excavation and support. It is stated that classes are normally developed which are project specific. Parameters relevant to support classes are given. The summary of all the site investigation data should lead to a reliable estimate of stand-up-time, deformation, overbreak, support load, settlement and ground water flow. However no methods are proposed for the determination of these from the site investigation data. During construction, continuous geotechnical documentation is recommended. This should comprise groundwater observation and face logging. However no instructions are given on how such face logging should be carried out. Especially in the case of soft ground tunnels (as noted in section 2.1.2.1), settlement may be an overriding factor affecting the requirement for support. This is not considered in any of the classification systems. It is stated that it is not practicable to produce a universally applicable, formalised classification system for tunnelling in general and soft ground tunnelling in particular.

Sauer also recommends that, if a general rock classification system is used, both of the two most popular systems, RMR and ‘Q’, should be considered. It is emphasised that these systems should be used with great caution, and preferably should not be included in the contract specification. Defects in these systems are considered to be as follows:

1. Inherently conservative.
2. Each omit key elements of rock mass characterisation.
3. Certain measurements can be subjective and not repeatable.
4. The values obtained depend on how and where the measurements are taken, and on the technique and experience of geotechnical staff.
5. There may be a tendency to accept the values obtained in favour of the actually observable condition of the ground.
6. The values obtained are subject to statistical uncertainty.

It is suggested that the results of the two systems, obtained by both the contractor’s and the consultant’s geologists, should be compared, to take account of the neglect of some of the possible parameters by the individual systems. Most importantly the results should be checked against the experience of as many tunnelling staff working at the face as possible. It is particularly emphasised that measurements on a relatively small number of scanlines may not reveal as much about the ground as a good look at the whole face.

2.3.8 Contractual matters

The successful use of rock classification systems for the selection of tunnel support requires both sufficient good quality site investigation data and a Contract form which is flexible enough to allow rapid variation in the support type used according to the conditions found in the tunnel. The Contract specification should allow the parties to the Contract to rapidly agree on the support to be provided for each advance of the face without costly disputes and delays.

The Institution of Civil Engineers (ICE) 6th edition of the Conditions of Contract (1991) in a modified form, the ICE Design and Construct Conditions (1992), the New Engineering Contract (NEC)(1993, 1994), and the Institution of Chemical Engineers (IChemE) Form of Contract (1992) (the ‘Green Book’) may each be appropriate depending on the type of project management. This subject is fully examined by Attewell (1995), in a comprehensive book on tunnelling contract and site investigation practice, which includes an appendix on the use of rock classification systems.

A key document is Construction Industry Research and Information Association (CIRIA) Report 79 (1978), ‘Tunnelling - improved contract practices’. This report is aimed to increase efficiency and reduce costs of tunnelling by recommending practices which would identify, eliminate or reduce risk connected principally with the behaviour of the ground. It also established appropriate means to evaluate and allocate the responsibility for the risk that remains.

It is recommended that a possible tunnelling system should be defined at the time of Tender and the limits determined for the ground conditions expected. Subsequent variations can then be compared with these ‘ground reference conditions’. It is emphasised that the Engineer’s role is crucial in safeguarding the interests of both parties to the Contract. Areas and incidence of possible risk should be explicitly defined in the contract documents, along with liabilities in the event of their occurrence.

Site investigation is identified as fundamental, the ground being the source of the largest risk in tunnelling. The recommendation of the Harris committee (National Economic Development Council, 1968) that the Engineer should be responsible for the selection of the ground investigation contractor on the basis of expert knowledge of ground investigation firms, rather than purely by price competition, is endorsed. The investigation should be designed and controlled by engineers or geologists.
conversant with the difficulties of tunnelling. The site investigation should ideally (although it is currently uncommon) provide the information required by both the Designer and the Contractor. To this end, the CIRIA report suggests that allowing expected Tenderers to influence the site investigation could avoid this deficiency.

It is recommended that the Engineer ‘should have the responsibility for supplying the Tenderer with the full report of the site investigation and for making the final assessment and judgement of any specialists findings’. It is also suggested in the CIRIA report that, where an interpretation of the site investigation data has been made which is fully accepted by the Engineer and has a particular bearing on the design and construction of a tunnel, that this interpretation should be made available to tenderers. It is also recommended that no disclaimers or warranties should be applied under the Contract to any site investigation information, and that Clauses 11 and 12 of the ICE Form should be fully accepted. The Tenderer must use the information provided to make its own judgement of the construction method and expected rate of progress. If any site investigation information is factually inaccurate, especially due to the negligence of the Promoter or Engineer, then the Contractor may have a valid claim. The Tenderer’s assessment of the site investigation should be fully reflected in his detailed method statement, and the Engineer should be satisfied that the Tenderer has fully understood and utilised the information.

The ground encountered during construction should be monitored, recorded, agreed and compared with the reference conditions previously determined by the Engineer. It should then be possible to apply Clause 12 without dispute. The reference conditions may contain elements from one or more of: a) geological; b) method of construction; c) response of the ground; d) rate of progress.

Commenting on the Round Hill project (see section 4.1), Mott MacDonald (1994) noted that the Client was meeting most of the risk in that Contract, through the instructed support clauses. The suggestion is made that the NEC or IChemE forms of Contract may hold some advantages for this type of work, but that a modified 5th edition of the ICE Conditions of Contract should also succeed. It is also commented that the alternative approach of ‘design and build’ contracts does not necessarily guarantee that there will be no claims. This is stated to be particularly true unless the client imposes no design changes, which is rarely the case. There will also be a danger under this arrangement that there will be problems with ensuring quality. Napthine and Smart (1995), reviewing the lessons to be learned from the use of design and build at the UK Channel Tunnel Terminal, state that the main disadvantage is a constant tendency to look for minor cost savings which gradually erode the final quality. It is also worth noting that lack of Client supervision in tunnel construction may allow defects to be hidden which do not come to light for many years.

Recommendations are also made by Mott MacDonald for beneficial changes to future tunnelling specifications. In the area of ground support it is recommended that all parameters for rock mass classification should be defined in detail. The split of support types should not be shown on the drawings but quantified in the Bill of Quantities, thus giving the Engineer more freedom to design final usage.

3 Literature review

A large number of papers relevant to the subject of rock classification systems have been published worldwide. This section reviews the most recent of these to provide an up-to-date discussion of the state of development of the systems. The reviews are grouped under six subheadings according to the topic covered.

3.1 Comparisons of and observations on ‘Q’ and RMR systems

Goel, Jethwa and Paithankar (1995) have evaluated the ‘Q’ and RMR systems with reference to measured support pressures on steel arch ribs at 25 tunnel sections in India. Results are claimed to show that ‘Q’ is unsafe for the prediction of rock loads for large tunnels in squeezing ground (squeezing being the plastic flow of the ground where the stress is high compared with the strength). However this finding is based on a very small sample in which the observed pressure exceeded that predicted in only two of 10 squeezing cases. Support pressure predictions based on Unal’s (1983) correlation between support pressure and RMR are even poorer, with observed pressures exceeding those predicted in all the squeezing cases and four of the non-squeezing cases. Equations based on the rock mass number (N), a ‘stress-free Q’ value as defined below, tunnel depth and radius, are presented for both squeezing and non-squeezing cases. These equations are shown to be much better predictors of the support pressure for the small number of case histories examined.

In order to improve the comparability between RMR and ‘Q’, new parameters are defined which omit the ratings for joint orientation and intact rock strength from RMR and rock stress in the ‘Q’ system. The modified RMR, is designated RMR_mod, and the modified ‘Q’ is designated Rock Mass Number (N). Correlations between RMR and ‘Q’ by Bieniawski (1976) and Rutledge and Preston (1978) are claimed unsafe and an improved correlation between RMR_mod and N from 61 case histories, is demonstrated.

Tarkoy (1995) considers the problems associated with the use of the RMR system with tunnel boring machines (TBMs) and provides guidance as to acceptable ranges of RMR for their use. He states that rock mass characterisation systems have often been used blindly in the past, whereas they should only be used as a design aid. They are useful for assessment of support requirements based on past experience, mainly from drill and blast tunnels. Although the predictions of support required may be reliable it should not be assumed that the use of a TBM is appropriate for the ground conditions. There is no easy way to adjust the RMR reliably for TBM excavation. The main problems are the distance from the face that support is installed and the resulting delay, the effect of gripper loads on the tunnel walls, and the effects on the excavation.
system of the face condition. It is observed that the walls of a TBM tunnel may initially look more stable than they are. Limitations of the RMR are that it does not apply to soil and is insensitive in the lower ranges of values, particularly of RQD and intact strength.

It is recommended that RMR only be used for:
1. Drill and blast excavation;
2. TBM support assessment, but only if RMR > 20;
3. TBM excavation assessment if RMR < 45 and then only with judgement and experience and an appropriate TBM design.

Barton and Grimstad (1994a) present an updated ‘Q’-system with 1050 new case records and the inclusion of fibre-reinforced shotcrete as support. This stainless-steel fibre-reinforced shotcrete is corrosion resistant, dense and of low permeability with compressive strengths of 35MPa to 45MPa. To avoid corrosion problems, triple-sleeved epoxy-coated rock bolts can be used, which are grouted after shotcreting to fully protect them. Single-pass linings are proposed using this material, which has been shown to save costs in 160km of Norwegian road tunnels. ‘Q’-system tables are presented. These are identical to the 1974 tables except for some changes to the SRF for rock stress problems. A graphical ‘Q’-logging chart is presented which is convenient for recording geotechnical data during a survey. An updated Equivalent Span (ED) v. ‘Q’-chart is presented, with support-type zones, including bolts and fibrecrete, superimposed (Figure 7). A new feature of this chart is that contours of shotcrete thickness are included. As in the original system the safety factor is governed by the value of ESR chosen by the user (high for temporary openings, low for permanent). A table of suggested adjustments to ESR and ‘Q’ to determine temporary support requirements and wall support requirements is given. This suggests that for temporary support ‘Q’ may be increased by a factor of 5 and the ESR increased by a factor of 1.5.

Equations relating critical depth for squeezing and compressive strength to ‘Q’, after Singh et al (1992) are given. An approximate relation between ‘Q’ and P-wave velocity (Vp) is also given, based on case histories from several countries in a range of rocks, but not corrected for depth. More data is required to refine this concept. This suggests the possible use of ‘design-as-you-drive’ using ‘Q’ derived from Vp obtained by probing ahead of the face, particularly with long and deep tunnels where site investigation boreholes are sparse.

The use of the ‘Q’ system with TBMs is discussed. The use of a TBM is considered likely to reduce the need for support only in the mid range of ‘Q’ from 3 to 30. An example is given in which the support needs increased by a factor of 5 and the ESR increased by a factor of 1.5.

Choubey and Dhawan (1990) present a case history of a 300m long x 7m span, D-shaped access tunnel through quartzitic phyllite, for a hydroelectric project. Some 40 exposures 300m-500m from the tunnel were mapped, giving 219 joint observations. Stereographic plots revealed four prominent joint sets. Each set was characterised and the rock mass classified according to both the ‘Q’ and the RMR systems. The tunnel faces were logged during tunnel construction. There were some differences between the surface exposures and the in-tunnel measurements. Table 4 shows the resulting average ratings for both ‘Q’ and RMR.

Table 4 ‘Q’ and RMR ratings

<table>
<thead>
<tr>
<th>RMR</th>
<th>‘Q’</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface exposure</td>
<td>51</td>
</tr>
<tr>
<td>Tunnel face</td>
<td>45</td>
</tr>
<tr>
<td>Description</td>
<td>fair rock</td>
</tr>
</tbody>
</table>

In either system the surface to tunnel range falls within one support class but overall the ‘Q’ predictions for support were nearer to the actual requirement than those based on RMR. It is stated that the number of rockbolts could be reduced by attention to potential loose blocks. By this means bolt spacing was gradually increased up to 3m in rock for which ‘Q’ suggested 1m spacing.

Kirsten (1988a) emphasises that classification systems are no substitute for engineering judgement. He highlights, in this discussion on Bieniawski’s (1988) paper, a lack of sensitivity of the RMR system, particularly to joint condition. This is largely due to the additive nature of the RMR rating system, which is overcome in Barton’s ‘Q’-system by the multiplication of individual parameters. Scatter in correlations between RMR and ‘Q’ is stated to be largely due to this lack of sensitivity. RMR also measures discontinuity spacing twice by including RQD and spacing as separate parameters, which gives block size an undue predominance in the final rating. The priority given to rock strength in the RMR system is also denigrated as it is only critical in cases of high stress relative to strength. This is much better accommodated in the ‘Q’ system. It is considered that the RMR system, which predicts final support requirements, is not adequate for the design of immediate support as required by the NATM.

Tallon (1982) compares RSR, RMR and ‘Q’ systems in the construction of seven tunnels on the Campomanes-Leon highway in Spain. The circular tunnels had a diameter of 11.6m with up to 600m of cover of Palaeozoic rocks (shale limestone, sandstone and quartzite). Classification indices were established at the tunnel faces and used to assist with the selection of support. A problem identified was that where rock quality is variable over the tunnel face, no standard method of face logging is laid down by any of the classification systems. Shoul zones be averaged, or the worst taken? This will depend on whether the poorer zones are judged significant in terms of stability. Problems with evaluating the various classification parameters are discussed. For example, uniaxial compressive strength can only easily be assessed at the face by the Schmidt hammer index, which is likely to be highly unreliable. A great deal of experience is required to perform a reliable assessment of most of the parameters. One hundred and fifty determinations of each of the indices were made. The RSR was found not to
differentiate clearly between average quality rocks and those of better and poorer quality. The RMR showed a better distinction between average and good quality rocks. The ‘Q’ index was found to be the most sensitive in its definition of rock quality, particularly for rocks of other than average quality. Correlations are presented between the three systems for four tunnels and compared with correlations published by Bieniawski (1976) and Rutledge and Preston (1979). These correlations are quite good considering that the data was from different places, rocks and observers, suggesting that all three systems are broadly comparable.

Cameron-Clarke and Budvari (1981) examined the applicability of both ‘Q’ and RMR systems to the selection of tunnel support, for three South African tunnels, based on both borecore and in-tunnel observations. The tunnel cross sections were of a 3m wide x 3m high inverted ‘U’ shape. A wide variety of rock types and support systems were covered. In both cases borecore tended to indicate poorer rock than in-situ observations. For both systems 82% of the results from borecore and in-situ measurements were within one rock class. For the in-situ measurements of discontinuities 10m long scanlines were used. Both classification systems were considered to be useful but should not be regarded as providing more than a preliminary assessment of the support required. Great care is necessary when interpolating between boreholes. The correlation between borecore and in-situ rock classes was found to be better for the ‘Q’ system than the RMR system. The tables presented in the paper show that the ‘Q’ system’s predictions generally agreed much better with the support actually installed than those of the RMR system, which were often excessively conservative. In most cases the ‘Q’ system was slightly conservative. The correlation between RMR and ‘Q’ for these sites was poor and very different from that presented by Bieniawski (1976).

3.2 Face logging

Rock mass classification systems are often used during construction of tunnels to guide the choice of support system. For this purpose the rock exposed in the heading must be assessed. In a drill and blast tunnel some or all of the face and a short length of sidewall will normally be available, whereas in a full-face machine bored tunnel the only available exposure may be the tunnel periphery behind the shield. Unless the geotechnical assessment (‘logging’) of the ground is comprehensive, and reflects the condition of the most critical areas of the face, the support predicted by the application of a rock classification system may not be appropriate.

An example of this is given by Garrett (1993) who describes the construction of the twin Cumberland Gap highway tunnels through the Appalachian mountains. The northbound bore is 11.43m wide, 7.8m high, 1249m long and the southbound bore is 12.1m wide, 9.55m high and 1259m long. A pilot tunnel 3m wide x 3m high formed the crown of the southbound bore. The geology was shale, mudstone, limestone, sandstone and coal in which voids up to 27.4m high were encountered. Geological mapping of both the face and the periphery was carried out during each shift. Excavation was by heading and bench using drill and blast. RMR and ‘Q’ were both evaluated and the temporary support was intended to be chosen from 5 pre-determined temporary support categories (supposedly NATM). The support details for each category are tabulated, but not the appropriate ‘Q’ - range. However the support used was generally chosen daily by the tunnel project engineer and the Client’s consultant. The Consultant’s assessment of the support required often differed from the result of using the rock classification systems on the shift geologists’ interpretations of the geology. The two shift geologists often rated the same ground very differently. It is not recorded how the ‘experienced NATM consultants’ decided on the support category. Sprayed concrete was by the wet process sprayed by robot. Both steel fibre reinforced and non-reinforced silica fume mixes were used together with dowels and bolts. Lattice girders were used in the lowest grade ground. The final support was cast in-situ concrete.

3.3 Block fallout

Several authors have considered the problem of the stability of potentially unstable blocks in a tunnel excavation and how this might be taken account of in rock classification systems.

Barrett and McCreath (1995) consider that empirical rules such as RMR and ‘Q’ are a useful starting point in the design of tunnel support but that uncertainty over the factor of safety achieved is an inherent problem. They examine the action of shotcrete in stabilizing the ground. Wet-mix shotcrete reduces rebound and increases its compressive strength, particularly with the addition of silica fume. The replacement of weldmesh by steel fibres in the mix saves the time required for its erection, with no loss of post-peak strength. There are, however, doubts about its performance at large deformations. A basis for design is presented based on four mechanisms of failure applicable to cases where rock mass failure is controlled by discontinuities rather than rock stress:

1. Adhesive strength.
2. Direct shear.
3. Flexure.
4. Punching shear.

The structural supporting ring function of a shotcrete lining in weak ground is not considered here. The main function of shotcrete and bolts in rock is to restrain ‘key blocks’ which if allowed to fall out can lead to progressive collapse. Shotcrete should be applied as soon as possible except in high stress zones where large deformations are allowed to occur. Particularly in blasted tunnels fibrecrete can be applied more quickly than mesh reinforced shotcrete. The worst case shotcrete load from block loosening is considered to be defined by weight of the 60° apex prism with base defined by the rockbolts. No friction is assumed as this might be reduced by blasting disturbance.

Estimates are given of various strengths of unreinforced silica fume shotcrete at a range of ages. Fibre
reinforcement is stated to impart no greater peak strength but gives it some post-peak strength in flexure and punching shear (when cracking has occurred). Calculation methods for the rock load between bolts are given based on the four modes listed above. Stability charts for these modes of failure are plotted based on bolt spacing and factor of safety for assumed values of lining thickness and strength.

Hatzi (1993) discusses the difficulties in defining critical blocks in a tunnel through joints rock, which lead rock engineers to resort to empirical rock classification schemes for support design. A major drawback of conventional rock classification systems is considered to be their ‘disregard for block motion into the newly created space’. Not all the removable blocks identified by block theory actually fall out in construction, because of differences between idealised geometry and reality, leading to a concept of block failure likelihood. Current rock mass classification systems do not adequately account for the interaction between the tunnel orientation and rock structure. The block failure likelihood $P(B)$ was originally defined as the product of the three parameters: Joint Combination Probability ($P(JC)$), Block Instability Parameter ($F$) and Shape Parameter ($K$). Correlations between the prediction parameters and observations in two tunnels suggest that the inclusion of shape parameter is not justified. $B'$, the cumulative block failure likelihood, is defined as the area of the $P(B)$ histogram and represents the overall tendency of the rock mass to produce block failures from a free face of known attitude. It is proposed that $B$, which will vary with tunnel orientation, could replace parameters such as spacing, orientation and condition of joints in rock classification schemes.

Insufficient information is given on the practical use of the system, in particular it is not at all clear how the factor $F$ is calculated from the field data, and no suggestions are made as to how the concept would be integrated with any specific rock classification system.

Ward (1978) Discusses three-dimensional block loosening mechanisms when tunnelling in weak rock, the importance of restraining ‘key’ blocks and the use of physical models and field observations. In cases where surface settlements are not critical he highlights the need to allow a degree of rock yield combined with monitoring and to consider the placement of support in relation to the advance of the face. ‘Proper assessment of and control of the many variables which combine to produce a satisfactory and efficient support system cannot be done at the design stage. It is strongly recommended that construction should be aided and guided by monitoring of performance carried out by engineers relieved of contractual responsibilities as the only positive means of taking sensible, economic and safe decisions during the progress of the work.’ Much reference is made to the Kielder Water experimental tunnel in support of these arguments. He denigrates the rock classification approach to design because it perpetuates existing practice, which may be excessively conservative, and takes no account of the fact that the same supports can be both satisfactory or unsatisfactory in the same rock depending on construction procedures. This is true however many case histories are correlated. The problem with the alternative analytical approach is incomplete information on the rock mass and the supports. The ‘characteristic line’ or ‘ground reaction curve’ is discussed in detail. It is stated that the ‘rising portion of this curve postulated by Pacher (1964) and others has not been found in the field’. He considers that sprayed concrete applied close to the face may prove to be too stiff and attract excessive loading even though it has a high creep and shrinkage capacity when green. A method of controlling the circumferential stiffness of a shotcrete lining by the use of longitudinal slots is described. This was used in the Tauern and Arlberg tunnels in Austria, in conjunction with steel ribs with frictional joints. Construction monitoring is highlighted as essential, also load and stress measurements are of little value without displacement measurement.

3.4 Norwegian method of tunnelling (NMT)

Barton and Grimstad (1994b) define a Norwegian Method of Tunnelling, which they claim to be more appropriate than NATM in harder, jointed rocks driven by drill and blast, where overbreak makes mesh erection difficult and causes excessive concrete consumption. In Norway wet process steel fibre reinforced shotcrete totally supplanted mesh reinforced shotcrete by 1984, due to its improved characteristics and ease of robot application. It is quicker to apply, has less rebound and thus reduces concrete volume. It has also replaced cast concrete, even in fault and clay bearing zones. Here sprayed fibrecrete with rebar reinforcement is used, with cost savings of approximately 50%. Drill and blast driving rates of 40m to 70m a week are achieved compared to 60m to 100m, with only minor reinforcement being necessary. 160km out of a total of 460km of road tunnels in Norway have an unreinforced or fibre-reinforced final lining. Fibre corrosion does not appear to be a problem, even in 10 year old sub-sea tunnels.

In Barton and Grimstad (1994a) the Norwegian Method of Tunnelling (NMT) is defined as following the following principles:

1. Usually applied to jointed rock, with or without clay bearing zones and or stress slabbing. ‘Q’ from 0.001 to 10 or more.
2. Excavation usually by drill and blast or hard rock TBM.
3. Support using fibrecrete, possibly with reinforced shotcrete ribs, bolting or cast concrete. No lattice girders or dry process shotcrete.
4. Contractor chooses temporary support.
5. Owner/Consultant chooses permanent support, which is often fibre reinforced shotcrete and not cast concrete.
6. Rock mass classification is used to predict rock mass quality and support needs, and both are updated during construction with monitoring of lining loads and deformations only in critical cases.

In contrast the NATM is quoted as being suitable for soft ground (‘Q’ from 0.001 to 0.01), machine excavated tunnels, with a closed invert in very weak ground. NMT is
most appropriate in drill and blast tunnels in harder rocks where overbreak is a problem. Design is often based on the ‘Q’-system of rock classification. Monitoring is generally not performed unless ‘Q’ is less than 0.01, unless the span is particularly great, as at the Gjøvik cavern. On the new support chart (Figure 7) NATM overlaps with NMT in categories 8 and 9. A possible system for soft rock combining the NMT and NATM principles is given as:

1. Support prediction using ‘Q’.
2. Temporary support close to the face with bolts and fibrecrete.
3. Adjustment of support class for final support well back from the face.

3.5 Probabilistic methods

The inherent variability of the ground, the relatively small amount of ground sampled in even the best site investigation and measurement errors inevitably result in a degree of uncertainty in both the assessed rock classification at the measurement points and that interpolated between them. This can be addressed by the use of risk analysis techniques by which the probability distribution of the output of a classification system can be estimated from the measured or assumed probability distributions of the inputs. The technique may be useful at the design stage, as detailed below. Alternatively it could be used during construction to provide a continuously updated prediction of the probability of encountering various rock classes in the tunnel yet to be constructed, based on the measured distributions of the classification parameters encountered to date. A detailed appraisal of the potential application of risk analysis to all aspects of highway tunnel construction is provided by Conway et al (1995).

Conway (1993) describes risk assessment techniques applied to the choice of alignment of a tunnelled river crossing. As it was a feasibility study there were no detailed investigation results. Minimum, most likely and maximum values were assigned to the geological parameters by a ‘best-guess’ approach, using a triangular probability density function (PDF) in the @RISK add-in function to the Lotus 1-2-3 spreadsheet. The trial alignments were divided into 100m long segments, each assumed to have constant characteristics. Triangular PDFs were used to model a) the depth of weathering of volcaniclastic rocks, b) thickness of alluvium c) the width of faults and d) depth to water table. Locations of faults were modelled using a Poisson distribution. The PDFs for the cost of ground treatment to reduce water inflow at faults was based on combining estimates of the probabilities of intersecting a fault, the specific discharge, the length and the wetted perimeter.

Tunnel support prediction was based on a probabilistic implementation of the ‘Q’ system. Estimated triangular PDFs were substituted for each ‘Q’-parameter. A simulation method using Latin Hypercube sampling within the spreadsheet was then used to generate a PDF for ‘Q’. Ninety-five percent confidence limits profiles for ‘Q’ could then be plotted along each alignment. Four support classes were developed from the ‘Q’ system recommendations. The cost of each alignment was calculated, based on a bill of quantities model using estimated PDFs for unit costs, and the extra costs due to fault zones. Cumulative frequency distributions of cost for each alignment were plotted, based on the ‘Q’ profiles. A sensitivity analysis could then have been performed to give further confidence in the model.

This method provides a systematic way of applying engineering judgement to the assessment of geological data for tunnel construction. Where the number of variables is small this approach may not be necessary, but with a larger number this approach provides a consistent basis for the application of engineering judgement.

A risk-based prediction model using these techniques was developed and applied during the construction of the Pen-y-Clip tunnel, which is described in section 4.3.

3.6 Numerical modelling

A number of authors consider that improved analytical methods will in time supplant the empirical rock classification schemes currently in use. Most major tunnel designs will involve the use of both methods. However, until recently the techniques available for numerical modelling did not include the analysis of discontinuous media, which was a severe handicap as in most practical cases it is the properties of the discontinuities which predominate in controlling the behaviour of the rock mass. Because of such difficulties with the characterization of the rock mass all numerical analyses for tunnelling need very cautious application. Ways in which conventional numerical analyses are being developed to make them more reliable are described in this section. It is likely that such calculations will always need checking by other means, including empiricism.

Pan and Trenter (1992) discuss the application of finite element analysis (FEA) and discrete element analysis (DEA), including block theory, to tunnels. It is asserted that a numerical modelling technique can assist the engineer in many practical problems, but failure to select the correct model could lead to expensive or even dangerous mistakes. However no specific numerical modelling packages are named. The basic idea is that the rock mass can be modelled as a continuum if the block size of the rock is either very large or very small relative to the size of the excavation. It is suggested that rock classification systems could be useful in determining whether to use a continuum (FEA) or discontinuum approach (DEA). On the ‘Q’ rating scale it is asserted that DEA is applicable in the central band of rock classes from ‘poor’ to ‘good’, while FEA is restricted to ‘very good’ and above, or in a pseudo-continuous form of FEA, to conditions worse than ‘poor’. Hudson has introduced the representative elemental volume (REV) concept, which is designed to help with this problem (see below). The strength and stiffness properties of the rock mass and discontinuities are required for the DEA approach and those of the rock mass for the FEA approach, together with the properties of the support system. These properties can be estimated from laboratory tests or rock mass

Barton et al (1990) in 'Tunnelling by numbers' describe the use of UDEC-BB, a discrete element method using the Barton-Bandis (BB) (Barton and Bandis, 1990) joint model, with qualitative examples. It is stated that numerical modelling may be required in some cases as a supplement to the use of the 'Q' system. There are severe shortcomings in the use of continuum analyses such as the finite element method (FEM), finite difference method (FDM) or boundary element method (BEM) when deformation and failure are controlled by discontinuities. The Norwegian Geotechnical Institute (NGI) has made extensive use of the code UDEC by Peter Cundall of Itasca Consulting. This has been extended by the use of the BB non-linear joint model. The input data required for the model can be acquired from drill core and/or outcrops, by the use of index tests. The case history used is described in more detail in Makurat et al (1990) (see section 3.6 below).

Barton and Bandis (1990) introduce the 'joint roughness coefficient (JRC) - joint compressive strength (JCS)' model for rock joints for use with the UDEC-BB program.

\[ \tau = \sigma_n \tan \left[ JRC \log \left( \frac{JCS}{\sigma_n} \right) + \phi_r \right] \]

where

\[ \tau = \text{shear strength} \]
\[ \sigma_n = \text{normal stress} \]
\[ \phi_r = \text{joint friction angle} \]

The derivation of the constitutive model and the input data required for UDEC-BB is described. A laboratory tilt test on jointed cores is described which has been found to be a good index test for finding peak strength of joints. The results show remarkably good agreement with direct shear tests at five orders of magnitude higher normal stress. JRC can be evaluated directly from tilt tests. Both shear strength and stiffness are affected by scale effects, confirmed by model tests. Scale correction curves are presented for JRC and JCS for extrapolation from laboratory tests to in-situ block sizes. Methods are given of obtaining JRC by the use of asperity depth measured with laboratory tests to in-situ block sizes. JRC-JCS joint model. The input data required for the model can be acquired from drill core and/or outcrops, by the use of index tests. The case history used is described in more detail in Makurat et al (1990) (see section 3.6 below).

There is no comparison in the paper between the predicted effects of tunnelling and those which actually occurred so it is not useful as a case history.

Wood (1991) discusses the estimation of Hoek-Brown rock mass strength parameters from rock mass classifications. This would allow rock mass strength and failure conditions to be assessed from a knowledge of intact strength and rock classification. Hoek and Brown (1980) developed a relationship between the principal stresses at failure in rock and the uniaxial compressive strength of intact rock which involved the empirical constants \( m \) and \( s \), very approximately analogous to \( \phi \) and \( c' \) in the Mohr-Coulomb failure criterion. The values of \( m \) and \( s \) for intact rocks are determined from curves fitted to triaxial test results. A table of values of \( m \) and \( s \) for discontinuous rock, from Hoek and Brown (1988), is given which was generated using a set of empirical relationships with RMR which are also given. The RMRs in this case are not adjusted for discontinuity orientation. Experience has since shown that these relationships underestimate the strength of the rock mass at low confining stresses.

For this reason Wood proposes that the RMR should be subdivided into partial classification parameters \( \text{RMR}_m \) and \( \text{RMR}_s \), very analogously analogous to \( m \) and \( s \) in the Mohr-Coulomb failure criterion. The values of \( m \) and \( s \) for intact rocks are determined from curves fitted to triaxial test results. A table of values of \( m \) and \( s \) for discontinuous rock, from Hoek and Brown (1988), is given which was generated using a set of empirical relationships with RMR which are also given. The RMRs in this case are not adjusted for discontinuity orientation. Experience has since shown that these relationships underestimate the strength of the rock mass at low confining stresses.

1 surface exposure mapping of discontinuities and borehole logging;
2 simple index tests on samples. (Point load, Schmidt hammer, tilt tests, in-situ profiling, strain-gauged uniaxial compression; for the JRC-JCS joint model).
3 generate joint performance curves of normal load/ (aperture or conductivity) and shear strain/ (shear stress, dilation, conductivity) from above data using the one dimensional program Lotus-BB;
4 build numerical model UDEC-BB. It is practically impossible to model every joint so a representative joint pattern is chosen and the elastic moduli correspondingly reduced. The model can be run in several stages with the effects of, for example, bolting and lining applied at appropriate times.

Output from the model includes:
1 changes in joint conducting aperture;
2 principal stresses;
3 deformations;
4 joint shear deformation.
point load tests or a field estimate based on ISRM recommendations.

**Hudson (1989)** introduced the concept of a representative elemental volume (REV) as an aid to determining whether the rock in a rock engineering problem should be analysed as a continuum or a discontinuum. The REV is the minimum volume of a discontinuous rock which must be tested to reduce the scatter in the measurements of a particular property to an effectively constant value. It is suggested that in modelling structures in rock that the near-field, up to the REV, be treated as a discontinuum and the far-field as a continuum.

### 4 Case histories

#### 4.1 Round Hill tunnels

The twin bores of the Round Hill tunnels were the first road tunnels in the UK to be designed using the basic principles of the NATM. They were constructed between November 1990 and their opening in December 1993 to take the re-aligned A20 through the Lower Chalk escarpment north of Folkestone. There are two tunnels approximately 350m long, with a centreline separation which varies but is never less than the 30m separation at the portals. The excavated cross-section is about 11.7m wide and 10.3m high, with an arched crown and a fairly flat invert. The faces were excavated by top heading and bench with the benches following after completion of the top headings. Primary support was by rockbolts and mesh reinforced shotcrete with or without lattice arches. Final support was by a cast concrete arch. A drainage layer and waterproofing membrane was placed between the two linings.

**Ground investigation**

Ground investigation borehole logs and the results of laboratory testing formed part of the Contract documents available to tenderers. The ground investigation provided five boreholes near the tunnel alignment, using rotary coring, and one shell and auger borehole. The tunnel was entirely within the Grey Chalk and the Upper Chalk Marl of the Lower Chalk. Initially both ‘Q’ and RMR systems were used for rock mass classification, based on the borehole data (Table 5). However it was later decided to rely on the ‘Q’ system alone. The Lower Chalk at this location comprises 15m of grey/white massive to blocky chalk known as the White Chalk underlain by some 25m of Grey Chalk which is in turn underlain by around 40m of the Chalk Marl. The Chalk Marl is a dark grey marly limestone with a high clay content. At the south-western portals the tunnel cross-section is entirely within the Upper Chalk Marl with the cover in the Grey Chalk, whereas at the north eastern portals both the tunnel and the cover are within the Grey Chalk.

**Design**

Design of the support systems was by continuum analysis (Murphy and Butfield, 1991) using both two-dimensional finite element programs and a finite difference analysis program for soils and rocks. Rock mass strength parameters for this purpose were derived from the rock mass classification systems using the Hoek-Brown (1980, 1988) failure criterion. (This technique allows for the degradation of rock mass strength caused by discontinuities but cannot model the kinematic behaviour of jointed rock.) Four combinations of ground and primary support were analysed using this model, covering the three support classes (Table 6). Support from the primary lining was ignored in the design of the 300mm thick cast concrete secondary lining (Murphy et al, 1993).

**Contract and specification**

The conditions of contract were the ICE 5th edition. Specifications, method of measurement and cost control systems were those of DOT. Three types of primary support had been designed by the Engineer. Each support type was assigned to a range of rock mass quality as determined by the ‘Q’ system. For tendering purposes the length of tunnel in each rock class was estimated. The contract provided for the final support type to be chosen by the Engineer, based on rock mass quality and deformation measurements in the tunnel (the latter being an essential component of NATM). The Engineer was also empowered to vary the details of any of the support types to suit the ground conditions.

**Construction**

Both top headings were driven from the eastern portals. Excavation of the westbound tunnel started first and was always considerably in advance of the eastbound. The top heading in the westbound tunnel was completed before its bench was excavated, as was the top heading in the eastbound tunnel. Table 7 below compares the proportions of each support type actually installed with those originally anticipated from the site investigation data.

Difficulties were encountered with overbreak and instability of the crown which necessitated the use of spiling and reductions in the advance length with both Type 2 and Type 3 support. Type 1 was little used because the lattice girder was necessary to facilitate spiling and maintain the profile of the crown. As seen from the table above it was generally found possible to use a lower class of support in the bench than in the crown. This may in part have been due to better quality rock in the lower part of the tunnel, but also highlights that block fallout problems are likely to be more critical in the crown than the sidewalls.

There was initially a problem with the contract in distinguishing temporary (cost allowed for in the

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**Table 5 Rock mass classification**

<table>
<thead>
<tr>
<th>Chalk RMR Grade</th>
<th>Geomechanics Classification</th>
<th>Q</th>
<th>Description</th>
<th>Support Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>II 40-50</td>
<td>Fair</td>
<td>0.7-2</td>
<td>Poor</td>
<td>1</td>
</tr>
<tr>
<td>III 25-40</td>
<td>Poor</td>
<td>0.12-0.7</td>
<td>Very poor</td>
<td>2</td>
</tr>
<tr>
<td>IV 17-25</td>
<td>Very poor - poor</td>
<td>0.05-0.12</td>
<td>Extremely poor</td>
<td>3</td>
</tr>
</tbody>
</table>
contractor’s bid) and permanent (measured cost reimbursed), overcome with the Client’s agreement by the Engineer directing most of the spiling, which then became part of the measured permanent support.

The design model produced expected displacements for comparison with those measured in the tunnel during construction, but not those at the ground surface. The predictions of crown settlement were 15% to 45% less than those actually measured for top heading excavation and 1.4 to 5.6 times greater than those measured for excavation of the bench. However the overall crown settlement predictions for both heading and bench were much closer for three of the sections, being generally within 10%, except for the section modelled with type 1 support where the total crown settlement was over predicted by 42 to 48%.

Seismic investigation
As discussed in section 3.1 Barton and Grimstad (1994a) propose an approximate correlation between compressive (P) wave velocity (V_p) and rock quality ‘Q’ which shows promise as a site investigation tool. The relationship is affected by stress (depth) and porosity and more data is needed to refine it. To this end a seismic survey was carried out during the construction of the Round Hill tunnels by TRL (Bowers and Hiller, 1994) to assess the variation of V_p along the drive and compare it with the assessed ‘Q’ values predicted by the site investigation and measured in the tunnels. This permitted an independent measure of the extent to which the support types used reflected the ground conditions measured by V_p. The investigation was carried out by measuring the time taken for a seismic wave to travel between the two bores at a number of locations along the tunnels. A critical depth of cover of 22m was found above which the seismic velocity was approximately proportional to the overburden. At shallower depths, the depth of cover had little influence on velocity, probably due to insufficient overburden stress to seismically close the joints. The results tended to confirm the distribution of ground quality indicated by the site investigation. There was also a reasonable correlation with the support used. However the differences between support type predicted and that used did not only reflect ground quality variation. Lattice girders were installed at some locations at which the ground conditions did not necessitate their use since the efficiency of the construction technique benefitted from their presence.

This was the first time that the NATM had been used under British highway contract conditions. Tunnelling generally went well, with the support methods providing rapid stabilisation in poor ground conditions.

### 4.2 Penmaenbach tunnel

The Penmaenbach tunnel is one of a series of improvements to the North Wales Coast Road constructed in 1986-7 through a headland of strong igneous rock (Ordovician Rhyolite). Excavation was by drill and blast except for a short length at the Eastern portal through loose ground. Temporary support was to comprise either steel ribs or rockbolts and sprayed concrete with and without fabric reinforcement depending on the ground conditions encountered. This temporary support was defined as forming part of the permanent works.

Site investigation
The contract documents provided a wealth of site investigation information including the results of mapping of discontinuities shears and folds in outcrops and the parallel unlined rail tunnel. Good access for geological mapping was available, in the existing unlined rail tunnel running roughly parallel about 100m distant from the new tunnel, and surface outcrops. These data were provided in the form of tables, drawings and polar discontinuity plots. The site investigation interpretive report reveals that tunnelling conditions were assessed by both ‘Q’ and RMR rock mass classification systems. The indices were calculated from both borehole logs and surface mapping. Ranges of ‘Q’ and RMR are given in detail for boreholes in the portal areas and the tunnel zone, at tunnel level, at 3m above the crown and 3-8m above the crown. They are also given for coastal exposures and at intervals along the rail tunnel. Expected ground water inflows were reported as 35 L/min/10m length at the east portal zone, negligible in the west portal zone and less than 50 L/min/10m in the central part. The hardness of the Rhyolite was expected to

### Table 6 Support description

<table>
<thead>
<tr>
<th>Support type</th>
<th>Lattice girder</th>
<th>Heading</th>
<th>Bench</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Advance (m)</td>
<td>Dowels (m x mm)</td>
<td>Dowel spacing (m)</td>
</tr>
<tr>
<td>1</td>
<td>No</td>
<td>1.5</td>
<td>3 or 4 x 25</td>
</tr>
<tr>
<td>2</td>
<td>No</td>
<td>1.5</td>
<td>3 or 4 x 25</td>
</tr>
<tr>
<td>3</td>
<td>Yes</td>
<td>1.0</td>
<td>2 x 25</td>
</tr>
</tbody>
</table>

### Table 7 Round Hill tunnel-actual v. predicted support class

<table>
<thead>
<tr>
<th>(Percentages are of total length)</th>
<th>Actual (Eastbound)</th>
<th>Actual (Westbound)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Support type</td>
<td>Heading</td>
<td>Bench</td>
</tr>
<tr>
<td>1</td>
<td>20%</td>
<td>1%</td>
</tr>
<tr>
<td>2</td>
<td>64%</td>
<td>92%</td>
</tr>
<tr>
<td>3</td>
<td>16%</td>
<td>7%</td>
</tr>
</tbody>
</table>
necessitate drill and blast excavation, and high rates of
drill tool wear were expected. An unfavourably orientated
joint set striking parallel to the tunnel with a steep dip was
identified as a likely cause of overbreak and roof-falls, as it
was in the rail tunnel. The rail tunnel also allowed the
identification of the position of shear zones at which rapid
support would be necessary in the new tunnel. Design
advice was given for three possible east portal schemes
and for the west portal in general terms for both temporary
and permanent support. The expected lengths of the near-
portal support type were given. Design advice for the main
tunnel support drew heavily on observation of the 100 year
old rail tunnel which is largely unlined. Rock dowels or
tensioned bolts with shotcrete were suggested for the
fracture zones, together with spot bolting of unstable
blocks.

**Contract and specification**

The Conditions of Contract were based on the ICE 5th
dition form to the general requirements of the
Specification for Highway Works. It was specified that the
excavated face was to be geologically mapped after
blasting and scaling of the face and the installation of
temporary support. Initial rockbolt and sprayed concrete
support was to be installed between 5m and 7m from the
face to avoid blasting damage. Steel rib support was fully
specified and shown on the drawings. Full details of
rockbolt installation tensioning and proof testing were
specified. A detailed specification covering all materials
and admixtures, mix design, setting times and strengths,
operators, test panels and testing was given. Steel fibre
reinforcement was allowed at the Resident Engineer’s
discretion.

Alternatives for the permanent lining, required to
minimize maintenance and for safety from minor falls,
were suggested as shotcrete with dowels or bolts and mesh
reinforcement where required to control cracking in
discontinuity zones, or shuttered mass concrete to give a
smooth profile. In the event the unreinforced mass
cement lining was constructed using a rail mounted steel
shutter.

4.3 Pen-y-Clip

The Pen-y-Clip tunnel is also on the North Wales Coast
Road and was the next to be constructed after
Penmaenbach. It was opened in October 1993 at the
completion of a 4 year contract period of which tunnelling
took 2 years. It passes through an intrusion of Ordovician
microdiorite into mudstone which is covered with fossil
scree and quarry debris. The total length is 930m including
the old rail tunnel which is largely unlined. Rock dowels or
rockbolts tensioned with shotcrete were suggested for the
fracture zones, together with spot bolting of unstable
blocks. Discovery in 1983 of a fissure large
enough for man entry, after cleaning to rockhead at the
portal site of the originally proposed northern tunnel,
resulted in the abandonment of this line and the adoption of
a line further into the headland. Access for drilling on
the tunnel line was very difficult due to the very steep
slope covered with scree. Drilling difficulties (later
repeated in the tunnel) were such that one borehole took
seven weeks to complete 22.2m through fossil scree and
was aborted before reaching bedrock, due to the nature of
the ground. Due to the complexity of the geotechnics it
was still necessary to incorporate further site investigation
works into the main tunnel construction contract. It was
expected from the site investigation data that the
excavation would be by drill and blast in hard rock for
most of the tunnel length.

As had been the case for the Penmaenbach tunnel the
tender documents included detailed factual site
investigation data. Again the adjacent unlined rail tunnel
provided ready access to exposures of similar rock. Joint
measurements from outcrops in nine areas were presented
as stereograms and tables. The tables include data on rock
type, RQD, weathering, groundwater, intact strength, joint
spacing of each set, orientation of each joint set, continuity,
separation, infilling, shape, roughness and strength of joint wall rock for each set and the presence of
faults or shear zones. RQD measurements were made
along scanlines in several directions using the techniques
of Priest and Hudson (1976). RMR values were calculated
from the collected data and were tabulated for 11 zones.
The nature of the jointing is described in detail.

Discontinuity logging of borehole core from 46
boreholes in the region of the tunnel was also used to
assess both ‘Q’ and RMR values. The records include, core
recovery, RQD, fracture index (discontinuities/m), position
(depth), orientation, roughness and condition and number
of sets. The entire West Quarry, close to the tunnel line,
was mapped in 60m² strips to provide RQD and fracture
index measurements and computer generated stereoplots.

**Contract and specification**

As at Penmaenbach the Contract was based on the ICE 5th
dition form, to the general requirements of the
Specification for Highway Works. Exploratory fully-cored
drilling in the tunnel was specified to be carried out as
determined by the Engineer. Full details of the coring
method, storage and core logging were specified. It was
also specified that the arrangement of permanent supports
described in the Contract might be modified after
geological mapping of the exposed and cleaned bedrock.
The initial supports were specified and were part of the
permanent works. Rockbolts and shotcrete were to be
installed between 3m and 7m from the face. Other
temporary support would not be accepted as part of the
permanent works. Details of advance probe drilling were
given as were procedures for dealing with water if
detected. Full details of the installation of steel rib support
were given. Details of the support systems are given
below.

Geological mapping of the face was to be completed
before drilling for the next round, by the Contractor’s
specialist engineering geologist, in the presence of the Engineer’s geologist. This shall comprise a detailed examination of the tunnel face, crown and sidewalls for the purpose amongst other things of determining the rock mass class. The geological logs were to include, lithology, degree of weathering and details of discontinuities. RQD, RMR and ‘Q’ values were to be determined and the rock mass class identified in accordance with the classes described in the Contract. The Engineer’s assessment was to prevail in the event of a disagreement.

Subsequent clauses similar to those for Penmaenbach described the materials and processes for the various tunnel support elements and grouting. Instrumentation was specified to be installed at prescribed intervals to measure convergence, rock strain and support load, and the times of reading.

Construction

In September 1991, after about 12 months of excavation, the tunnel was 25 weeks behind the Clause 14 programme and extrapolation suggested a total expected delay of 88 weeks. Eight weeks extensions to time had already been granted. The main difficulty was in tunnel support. The contract specified 3 rock classes in terms of ‘Q’ and RMR with associated support for each class (see Table 9 in Catling and Scholey, below). The expected occurrences for each class, as a percentage of the total length of tunnel were 36% in Class 1, 56% in Class 2 and 8% in Class 3.

Tunnel support was reviewed and conditions compared with those expected, as work progressed. The disparity between the conditions expected and those found in the tunnel resulted in a need to revise the predictions for the remainder of the work. Revised support classes are shown in Table 8. A risk-based prediction model was developed. This approach used the distributions of the rock classification system input parameters, measured in the tunnel completed to date, to predict the probability of the output distributions, using a numerical simulation technique. In this case the output required was the ‘Q’ value at any point of interest in the tunnel. In certain mixed-face conditions consideration of infilled fissure widths overrode the ‘Q’ value, so the model included this as an extra parameter to the six input parameters to ‘Q’.

The model divided the tunnel into 168 five metre segments and calculated the probability of each support class for each segment, based on the input data. In the tunnel, the support category was based on an agreed overall ‘Q’ value from ‘all exposed rock faces’. In mixed face conditions a controlling ‘Q’ value was identified, usually for the worst conditions at the face, or extrapolated just ahead of it.

<table>
<thead>
<tr>
<th>Table 8 ’Q’ based support selection</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Q</strong> value</td>
</tr>
<tr>
<td>&lt;0.01</td>
</tr>
<tr>
<td>0.01 to 0.1</td>
</tr>
<tr>
<td>0.1 to 0.4</td>
</tr>
<tr>
<td>&gt; 0.4</td>
</tr>
<tr>
<td>Widely fissured</td>
</tr>
</tbody>
</table>

Unexpected geotechnical problems encountered, giving rise to Clause 12 claims, included: infilled fissures; highly disturbed rock with clay-filled joints; loose ground and shear zones. These conditions often required a spiled canopy or in some cases, a micro-heading. Small blocks had given difficulties with rock-bolting. Loose ground and wide jointing were expected from the ground investigation, but some ‘multiple wide fissures’ had been accepted as unforeseen conditions. An assessment of the need to order further steel ribs was based on a ‘broad overview of the geological structures exposed in viaduct bay and the West Quarry and ground investigation boreholes’. It was assumed that Class 3 could continue to the eastern boundary structure where the more competent rock was expected at chainage 1250. The revised requirement for steel ribs was based on this. This chainage was found to agree quite well with the 65% confidence limit for Class 3 support predicted by the most recent risk-based analysis, and was not seen as unduly conservative. Further details are provided by Haycock (1996).

Operational problems had given rise to much lower rates of advance than anticipated, mainly caused by difficulties with achieving clear drilled holes for rockbolts and in the face similar to those experienced in drilling from the surface during the ground investigation. These were caused by small fragments of rock jamming the drill string, and the resulting holes being unsuitable for resin capsules without further treatment, such as grouting and re-drilling. These problems were mainly in the lower Class 2 rock (‘Q’ = 0.01 to 0.1). Delays caused by difficulties with shotcreting were not related to the geological conditions. Extensive trials of rockbolting were performed to identify the best procedures for overcoming the problems. Various support modification options were considered to speed up construction in the light of these difficulties. A modified Class 3 support (Class 3m) was adopted from chainage 1040, with steel ribs at 1.5m spacing and 100mm mesh reinforced shotcrete between the ribs. An alternative design was considered comprising 370mm shotcrete with one layer of mesh and lattice arches at 750mm centres, installed by heading and bench working, as for the Class 3 steel support. However frequent switching between Class 2 support (necessarily constructed full-face) and this alternative would incur excessive delay. The various support options throughout the remainder of the tunnel were assessed on a time and cost calculation based on both the 50% and 65% confidence limits of expected rock classes and locations. It was concluded that the introduction of lattice girder and shotcrete support in Classes 2 and/or 3 should produce a significant reduction in excavation times even if conditions were worse than then expected. It was expected that, although the direct costs would increase, the cost of prolongation would substantially reduce, giving a net gain. In the event this was not accepted and the Class 3 part of the tunnel was completed in steel arches.

Catling and Scholey (1994) provide a detailed account of lining the tunnel. The ‘Q’ system and the RMR were both used for rock classification. The ‘Q’ system proved to be the more accurate and sensitive, particularly in poor
rock. The Contract specified the primary support systems described in Table 9 below. These were designed to provide the full permanent support.

The length of bolts was determined from wedge analysis (the specific method is not specified) and the steel arches were designed for the full overburden pressure at the portals. Class 3 sections were driven by heading and bench and Classes 1 and 2 full-face. Permanent works were not allowed closer than 3m to the face in Classes 1 and 2. Sprayed concrete was by the dry process. Some steel fibre reinforced shotcrete was used for re-profiling but was not pursued for long. Re-profiling to fill overbreak was intended to be by shotcrete, but problems were expected with obtaining the required tolerance (25mm) on the surface profile. Wet process shotcrete was rejected after observation of European practice. Finally this concrete was placed behind a 10m long shutter. Rockbolts and mesh were used to tie this concrete to the shotcrete so that no load was transferred to the secondary lining. This secondary final lining was cast in-situ, over a full-profile drainage membrane, to provide a smooth surface but was not necessary as structural support. This lining is 300mm thick in the rockbolted sections and 600mm thick in the steel rib sections.

On the completion of the tunnel Fowler (1993) reported that 68% of the tunnel was built with steel ribs, rather than the 8% originally envisaged. This was caused by a much greater length of poor quality rock than expected from the site investigation. This ground was mostly able to excavated without blasting and the tunnel had to be driven by heading and bench to avoid face instability. Progress in the rib sections averaged 12m per week, less than half that achieved in the full-face central section. Arches at 1500mm spacing were reverted to after 35m of difficulties with drilling holes for bolts. The Contract was granted a ten week extension.

The difficulties with poor rock are highlighted by Watson (1991) who reported a crown collapse 60m from the west portal. Loose rock flowed from the crown and face of the top heading, when miners were excavating up to 2m ahead of the heavy steel ribs at 750mm centres used to support continuous poor ground. The geotechnical consultant insisted that the conditions were consistent with rock classification into 6 groups was appended to the specification, based on rock strength and discontinuities. Control water inflow, was specified. A provisional rock classification into 6 groups was appended to the specification, based on rock strength and discontinuities. Items for each rock group were included in the Bill of Quantities. Two rules were applied:
1. The length of tunnel in each rock group was defined by the characteristic rock group at the tunnel face.
2. Where two or more rock groups were exposed at the face, the characteristic rock group was to be taken as the highest ranking of those present, provided its total thickness at the face was not less than 0.6m.

The expected proportions of each group were inserted in the Bill with allowed pre-grouting time for each.

This case history highlights the need for adequate site investigation, without which no rock classification scheme is likely to be of any assistance. After one year the work was 6 months behind, mainly due to hard and massive rock and water ingress at higher rates than expected. Boom machines had to be replaced by drill and blast in 3 of 4 headings, and compressed air was necessary at two portals.

### Table 9 Support classes for the Pen-y-Clip tunnel

<table>
<thead>
<tr>
<th>Q’ value</th>
<th>RMR</th>
<th>Support Class</th>
<th>Support</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 0.01</td>
<td>&lt;20</td>
<td>3</td>
<td>Steel arches at 750mm centres 305x305x158kg/m arch on wall beam; 305x305x137kg/m walls</td>
</tr>
<tr>
<td>0.01 - 0.4</td>
<td>(extremely - very poor)</td>
<td>2</td>
<td>Rockbolts (.85m x 1m array; typ. 6 x 7m long + 13 x 3.5m long) + full mesh + shotcrete (100mm min)</td>
</tr>
<tr>
<td>&gt; 0.4</td>
<td></td>
<td>1</td>
<td>Rockbolts (1.3m x 1.5m array; typ. 4 x 7m long + 13 x 3.5m long)+ crown mesh + shotcrete (100mm min)</td>
</tr>
</tbody>
</table>
to control water. Additional boreholes were sunk to determine the optimum measures to be adopted for tunnelling work.

### 4.5 Tyne-Tees Aqueduct

Davies et al (1981) examined the effectiveness of site investigation in 32.2km of tunnelling, in three sections, for the Tyne-Tees water transfer scheme. This tunnel was of 3.5m excavated diameter through a varied sequence of sedimentary carboniferous strata with limited occurrence of intrusive igneous rocks. Most of the tunnel length was driven by full face tunnelling machine with limited sections by roadheader and by drill and blast. Temporary support methods included: none; rockbolts and mesh or steel sheets; sprayed concrete; and in the worst conditions, steel arches. The ground investigation for the Derwent-Wear-Tees tunnels (27.7km) included 18 main boreholes cored in the lower sections (75 and 100mm diameter), with 18 further boreholes in the portal areas. Spacing between boreholes in the deeper tunnels was up to 3km and many were over 500m from the tunnel line because of route uncertainty at the time of the investigation. Cores were logged for RQD, Schmidt rebound and fracture spacing. Packer tests and piezometers gave indications of permeability and water pressures at tunnel level. Strength, deformation and swelling characteristics of over 350 samples were measured in laboratory tests. Based on the site investigation data, which included results from an experimental tunnel at Rogerley Quarry, TBM was selected for the excavation of the two long tunnels and drill and blast for the shorter ones. It was expected that rockbolts, with or without steel mesh would suffice as support for most of the tunnel, with lagged steel arches or shotcrete being necessary near portals, in some mudstone areas and in faulted ground.

A five-zone rock classification system for payment was developed specially for the long tunnels of the scheme, based on the long sections derived from the investigation results (Table 10). The system was designed to allow adjustment of the contract value to take account of the actual rock conditions.

#### Table 10 Site-specific rock classification for the Tyne Tees Aqueduct tunnels

<table>
<thead>
<tr>
<th>Class</th>
<th>Rock</th>
<th>Quality</th>
<th>Support</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sandstone/Limestone</td>
<td>Good</td>
<td>Little or none</td>
</tr>
<tr>
<td>2</td>
<td>Sandstone/Limestone/Mudstone/Shale</td>
<td>Poor</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Mudstone/Mixed beds</td>
<td>Good</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Mudstone/Mixed beds/Fault boundaries</td>
<td>Poor</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Fault conditions</td>
<td>Heavy (eg. full circle steel arches)</td>
<td></td>
</tr>
</tbody>
</table>

Ground conditions were continuously logged in the tunnel by the RE’s inspectors. In the machine bored tunnels the face was not accessible so the sidewalls and crown were logged on specially designed logsheets. These were assessed by the Resident Engineer’s geologist who determined the classifications.

**Drill and blast tunnel.** Overall the site investigation predictions were substantially correct with the main structural features occurring at the expected locations. One major unanticipated fault zone was encountered, and minor variations in the bedding dip caused greater lengths than anticipated to be excavated with unfavourable roof conditions.

**Machine bored tunnels.** Overall there was less Class 1 and 2 and more Class 3 and 4 than predicted. However this led to an increase of only 5.5% in payment, confirming the generally accurate predictions. There were some significant deviations including locally greater faulting than expected, solution cavities in limestone into which mudstone had collapsed, an upward transgression of a hard dolerite sill, and some lengths in less favourable strata at the crown than expected.

The level of support required was generally underestimated in the drill and blast and overestimated in the machine bored lengths. It was concluded that there will be uncertainties in detail in any investigation, due to limitations on numbers and positions of boreholes, and likely shortcomings in core and borehole logging. In this case the pre-defined rock classification system was flexible enough to permit adjustment to the Contract value to take account of deviations from the predicted geology. The most important factor in improving site investigation was stated to be the positioning of boreholes close to the tunnel line.

#### 5 Conclusions

**General**

The empirical rock classification systems discussed in this report are useful to assist with the selection of tunnel support at both the initial design phase and during excavation. However they are not and cannot be precise tools and cannot replace experience and engineering judgement. In most cases it will also be necessary to use analytical methods at the design stage to analyse support options.

**Choice of system**

It is apparent that in recent years the use of general rather than site specific rock classification schemes has become more common. The general consensus appears to be that the ‘Q’ system of Barton et al (1974) is the most successful of all the systems. This is also the only scheme which is being actively developed and extended by its originator. It is now backed up by a database of over one thousand case histories covering a wide range of ground types and support systems. The RMR system of Bieniawski (1989) is the only other popular contender. In
many cases the ‘Q’ system has been shown to be less conservative than the RMR system and more sensitive to changes in rock quality. For example Barton (1988) reports that Einstein et al. (1979) found the RMR system to predict considerably support in many of the unsupported cases analysed by Cecil (1970) which were part of the database upon which the ‘Q’ system was originally based.

Site investigation
It is vitally important that thorough site investigation is carried out. Data should be obtained from as many boreholes as possible on or very close to the tunnel line. The acquisition of adequate data for the use of rock classification systems may require more detailed analysis of boreholes than is commonly made, such as the use of oriented downhole CCTV or ‘acoustic televiewer’ surveys to determine joint condition and orientation, which cannot normally be obtained from cores. Full use should also be made of exposures close to the tunnel line. Particularly where the number of boreholes is limited, investigation ahead of the tunnel face by drilling may be justified.

Seismic investigation
Both at the site investigation stage and during tunnelling seismic velocity measurements show promise as a means of deriving the rock classification. At the site investigation stage seismic cross-hole tomography has been used with success (Barton, 1991; Barton et al, 1992). Barton (1994) has suggested the use of seismic velocity logging in probe holes drilled ahead of the tunnel face for this purpose.

Access for face logging
When rock classification schemes are used during tunnel construction to determine support at the face it is vitally important that they are applied by engineering geologists experienced in their use. The whole face should be logged with attention to the critical areas near the periphery, in particular near the crown. To this end it is important that the Contract specifies that the Contractor should provide the means of access to these areas.

Contracts
Because of the variability of the ground and the inevitable lack of complete knowledge of the ground conditions prior to tunnelling it is important that Contracts allow flexibility in support selection based on the conditions actually encountered. The establishment of the ‘ground reference conditions’ recommended by CIRIA report 79 (1978) (see section 2.3.8) could be facilitated by the use of a specified rock classification system. The parameters for this rock classification system should be defined in detail. The classification agreed at each advance during construction would then, in accordance with the CIRIA recommendations, be compared with the reference class for the determination of payment. Flexibility would be facilitated by the split of support types not being shown on the drawings but quantified in the Bill of Quantities, thus giving the Engineer more freedom to design final usage.

As recommended by CIRIA, cost reimbursable and target type Contracts are likely to be the most appropriate when limited information is available about the ground, there is insufficient time to prepare an admeasurement type of Contract, there is a wish to use innovative methods for which little cost experience is available, or Contractors are not willing to respond to a high risk venture. Possible Contract forms include a modified version of the ICE 6th edition, the ICE New Engineering Contract and the IChemE (Green Book) Contract.

Generating analysis parameters
As previously discussed it is vitally important that at the design stage the use of rock classification systems for the determination of support requirements should be supported by analytical methods. Where the scale of the discontinuity structure relative to the excavation is such that the properties of the discontinuities control the behaviour of the ground, it is likely that the use of a discrete element method which can model the discontinuities will provide a more reliable solution than continuum methods. As discussed, a major difficulty in the use of these methods is the determination of appropriate ground properties and improvements need to be made. However methods have been developed of estimating some of the ground properties needed for input to these methods using rock classification schemes.

6 Recommendations
Further work to refine the relationship between rock quality and seismic velocity could render this a very useful predictive tool. Measurement of seismic velocity could then be used during site investigation and construction to assess the likely rock quality. The development of a standard method of measuring the seismic velocity would facilitate this.

A particular problem of which TRL staff have been made aware of is how such irregularities in a tunnel cross-section as recesses and cross-passage junctions might be accommodated at the support selection stage of the use of a rock classification system. As this is in effect a change in the tunnel cross-sectional shape or span it does not appear logical that the rock quality should be adjusted to accommodate this. In the ‘Q’ system it would appear that the most appropriate parameters to adjust would be the span or the ‘excavation support ratio’ (ESR) which together are used to define the effective span to be taken for the selection of the support. The appropriate adjustment could in principle be determined by the collection and analysis of case history data. However because of the difficulties of data collection and of assessing the factor of safety in existing construction a more satisfactory route would be to calculate the adjustment which would be required for a representative selection of cases using one of the numerical modelling techniques described in this report.

A suggested outline sequence for use of rock classification system in tunnelling projects is given below:
1 Site investigation and laboratory testing.
2 Preliminary design using both rock classification and an appropriate numerical modelling techniques to define a range of possible support types.
3 Contract specification to suggest an appropriate tunnelling method and specify ground reference conditions and measurement of classification parameters.
4 Continuous face logging, possibly with probe drilling ahead of the face, to assess actual ground quality.
5 Selection or modification of support type based on 2 and 4 above combined with engineering judgement and experience.
6 Payment based on agreed support system actually used.

7 Acknowledgement
The author is grateful for the assistance of Dr J Perry in reviewing the draft of this document.

8 References
The references which follow are divided into two sections covering first rock classification and second, case histories. The Appendix contains further useful references which have been consulted but are not referenced in the text.

8.1 Rock Classification


Barton N and Grimstad E (1994a) The ‘Q’-system following twenty years of application in NMT support selection. 43rd Geomechanics Colloquy Salzburg.


Institution of Chemical Engineers (1992) Model form of conditions of contract for process plant suitable for reimbursable contracts, second edition, IChemE, Rugby.


8.2 Case history references


Appendix: Other useful references


Abstract

Recent UK and other practice in the use of empirical ground classification systems for the selection of tunnel support is examined to identify new developments and note particular problem areas. The concept and nature of the systems are discussed in the first section. Recent literature is then examined and discussed, followed by a review of UK case histories of tunnelling where ground classification schemes were used. Final sections draw together the most important conclusions to be gleaned from the evidence and some guidance is provided on the practical application of the systems. Recommendations are also made on the possible direction of future research.

Related publications

PR60  Study of the efficiency of site investigation practices by Mott MacDonald and Soil Mechanics Ltd. 1994 (price code J)

TRL192  Sources of information for site investigations in Britain (revision of TRL Report LR403) by J Perry and G West. 1996 (price code L)

TRL209  Field evaluation of the TRL load cell pressuremeter by P Darley, D R Carder, M D Ryley and P G Hawkins. 1996 (price code E)