

Recent experiences with the Q-system of tunnel support design

NICK BARTON, PhD Norwegian Geotechnical Institute, P.O. Box 40 – Taasen, Oslo 8 – Norway.

SUMMARY

The Q-system of rock mass classification and support design is based on a numerical assessment of the rock mass quality using six different parameters. The six parameters consist of the RQD, the number of joint sets, the roughness of the most unfavourable joint or discontinuity, the degree of alteration or filling of the most unfavourable joint or discontinuity, the degree of water inflow, and the stress condition. Another classification system, the Geomechanics Classification (Bieniawski, 1973, 1974) is also based on six parameters. Qualitative differences between the two methods are discussed.

The 200 case records that were analysed when developing the Q-system, included more than 30 cases of permanently unsupported openings. An analysis of the rock mass characteristics involved has shown that certain characteristics are essential if an excavation is to be left permanently unsupported. If the maximum unsupported span for a given Q-value is exceeded, the safe life of the excavation may be shortened. A preliminary attempt is made to correlate stand-up time, rock mass quality Q, and span width.

The Q-system has been applied on several projects in Scandinavia and abroad since its development in 1973/1974. An example of a recent application is given in detail. The preliminary estimates of permanent support for a 19 metres span underground power house were obtained from an analysis of corelogs. In a subsequent site visit the Q-system was applied *in-situ*. The final estimates of permanent support were found to compare well with the preliminary estimates. Core logs, seismic profiles and surface mapping were used as a basis for preliminary design of permanent support for the 9 metres span tailrace tunnel, again using the Q-system. This tunnel is presently under construction so comparison of predicted and actual support is not yet possible.

KEY WORDS

Rock mass, classification, tunnel, powerhouse, support, borecore, case record.

INTRODUCTION

The six parameters chosen to describe the rock mass quality Q are as follows:

RQD = rock quality designation (Deere, 1963)

J_n = joint set number

J_r = joint roughness number

J_a = joint alteration number

J_w = joint water reduction factor

SRF = stress reduction factor

These parameters are combined in pairs and are found to be crude measures of:

1. relative block size (RQD/J_n)
2. inter-block shear strength $(J_r/J_a) (\cong \tan \phi)$
3. active stress (J_w/SRF)

The overall quality Q is equal to the product of the three pairs:

$$Q = (RQD/J_n) \cdot (J_r/J_a) \cdot (J_w/SRF) \quad (1)$$

Thus, the following rock mass would be most favourable for tunnel stability: high RQD-value, small number of joint sets, appreciable joint roughness, minimal joint alteration of filling, minimal water inflow, and favourable stress levels.

Individual ratings of the six parameters have been published previously, together with detailed support tables from which estimates of appropriate permanent support can be obtained. In view of the fact that no changes have been found necessary, the support tables are not repeated in this paper, and readers should consult two earlier publications for such details (Barton, Lien and Lunde 1974, 1975). However the classification ratings are given here (see Appendix) so that the following examples of support prediction and classification philosophy may be more easily followed. These classification ratings are also unchanged from the original.

COMPARISON WITH THE GEOMECHANICS

CLASSIFICATION SYSTEM

It is not the intention here to make a quantitative comparison between the Q-system and Bienawski's (1974) Geomechanics Classification since this is done in the general review paper in this symposium. However, certain qualitative differences can be mentioned which serve as a useful basis for discussion.

Bieniawski (1974) rates the following six parameters in his system:

1. Uniaxial compressive strength of rock material.
2. Drill core quality RQD.
3. Spacing of joints.
4. Condition of joints.
5. Groundwater conditions.
6. Orientation of joints.

It can be seen that stress is not used specifically as a parameter though it is apparently when selecting support measures. In the Q-system, the ratio (σ_c/σ_1) (unconfined compression strength/major principal stress) is considered a more realistic method of treating this "rock burst" factor, and in fact the onset of rock bursting and slabbing problems can be quite accurately predicted (see Appendix, Table 6b). The Q-system also accounts for *loosening* caused by shear zones and faults, and *squeezing* and *swelling* ground.

A common factor between the two systems is the use of Deere's (1963) RQD. However Bienawski also includes joint *spacing* and *orientation*, while the Q-system only considers the *number of joint sets*. The significance of the number of joint sets, particularly in the case of unsupported tunnels has been discussed at some length by Barton (1976).

The exclusion of *orientation* as a separate parameter in the Q-system has been criticised quite widely, but possibly the basic philosophy of the Q-system has not been fully appreciated by those concerned.

In all publications it has been emphasised that the parameters J_r , *joint roughness number*, (Appendix: Table 3) and J_a , *joint alteration number*, (Appendix: Table 4) should apply to the joint set or single discontinuity most likely to *allow failure to initiate*. The orientation of the feature relative to the excavation is implicit in these instructions. A practical example may be useful here. The Q-system was recently used for estimating the support requirements of a 19 meters span hydro power cavern and a parallel gate gallery of 3.5 meters span. A vertical narrow shear zone intersected the axis of both excavations, more or less perpendicularly. Besides other joint sets there was also a set of unfavourably orientated smooth, undulating joints dipping at about 50° from the downstream walls. The minimum value of J_r/J_a is obviously obtained from the shear zone. However, due to its favourable orientation this was ignored in the classification and the slightly higher value of J_r/J_a for the unfavourably orientated joints was considered more relevant. If the shear zone had been looser and clay bearing, then clearly it would re-establish itself as the potential source of failure, and a lower Q-value and heavier support would result.

Bieniawski (1974) appears to have favoured the *mean* rating for *spacing* and *orientation* of the different joint sets according to the case record given in his paper.

The very detailed treatment of joint *roughness* and *alteration* which is perhaps the strongest feature of the Q-system is not particularly emphasised in the

Geomechanics Classification. In his original version Bienawski (1973) considered the condition of joints under three descriptive terms: *weathering* (5 ratings), *separation of joints* (5 ratings, <0.1 mm up to >5 mm) and *continuity of joints* (5 ratings, not continuous up to continuous with gouge). In his 1974 publication Bienawski condensed these three terms to *condition of joints* which again has five ratings; from *very tight*, *separation <0.1 mm, not continuous*, up to *open >5 mm, continuous gouge >5 mm*. In his most recent publication (general review paper, this symposium) Bienawski also includes joint roughness in his fourth parameter *condition of joints*.

MAXIMUM SPANS FOR UNSUPPORTED EXCAVATIONS

A very interesting area of application for the Q-system is in the recognition of rock mass characteristics required for safe operation of permanently unsupported openings. A detailed analysis of all the available case records of unsupported excavations (Barton, 1976) revealed the following requirements. (The ratings of the various parameters should be checked against the descriptions given in the Appendix).

General requirements for permanently unsupported openings.

1. $J_n \leq 9$, $J_r \geq 1.0$, $J_a \leq 1.0$, $J_w = 1.0$, $SRF \leq 2.5$

Conditional requirements.

2. If $RQD \leq 40$, should have $J_n \leq 2$
3. If $J_n = 9$, should have $J_r \geq 1.5$ and $RQD \geq 90$
4. If $J_r = 1$, should have $J_n < 4$
5. If $SRF > 1$, should have $J_r \geq 1.5$
6. If $SPAN > 10$ m, should have $J_n < 9$
7. If $SPAN > 20$ m, should have $J_n \leq 4$ and $SRF \leq 1$

Existing natural and man-made openings indicate that very large unsupported spans can be safely built and utilized if the rock mass is of sufficiently high quality. The case records that describe unsupported man-made excavations have spans ranging from 1.2 to 100 metres. If there are only a limited number of discontinuous joints and the rock mass quality Q is up to 500 or 1000 the maximum unsupported span may only be limited by the ratio of rock stress/rock strength. Naturally, if this ratio becomes unfavourable (see Appendix, Table 6b) the quality Q will not remain at this high value, and the maximum safe span will be reduced.

All the available case records of unsupported spans are plotted in Fig. 1. The tentative curved envelope is the assumed *maximum design span* for *man-made* openings based on these available cases. The five square data points plotting above this curve were obtained from the huge *natural* openings of the Carlsbad limestone caverns in New Mexico. If the data for man-made and natural openings is combined, it is seen that the limiting envelope is approximately linear.

It can be represented by the following simple equation:

$$SPAN = 2Q^{0.66} \quad (2)$$

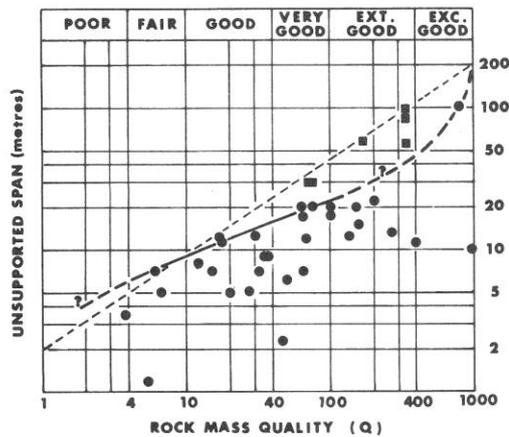


Fig. 1 Circles represent the man-made unsupported excavations reported in the literature. Squares represent some natural openings from Carlsbad, New Mexico. The curved envelope is an estimate of the maximum design span for permanently unsupported man-made openings.

For design purposes the evidence of the natural caverns is ignored. Suggested *maximum design spans* for different types of excavations are based on the curved envelope. Details are given by Barton (1976), and also in the last section of this paper.

STAND-UP TIMES FOR UNSUPPORTED EXCAVATIONS

The man-made openings which plot closest to the curved envelope in Fig. 1 were the following types of excavations; 7 m span major road tunnel (slow lane for lorries), 8 m span water collector tunnel for hydro scheme, 11.2 m span tailrace tunnel for hydro electric station, 12 m span waste water treatment plant, 12.5 m span head race tunnel for hydro scheme, 20 m span mine openings (two cases), 22 m span subway station, 100 m span mine opening.

In view of the type of excavations involved, and the fact that the mine openings in question have been perfectly stable for many years, it is certainly *conservative* to assume that the above excavations have a stand-up time in excess of 10 years. For practical purposes they can probably be regarded as *permanent*. Certainly the Carlsbad caverns must be considered as "permanent" unsupported openings. No appreciable rock fall has been observed in more than 50 years of public visits, and more than 1 million people pass through the caverns each year. Classification details and approximate dimensions of these caverns are given by Barton (1976).

The closeness with which an unsupported opening can be designed to the envelope of *maximum design span* will depend on the type of excavation, the degree of safety, and the stand-up time required. If the *maximum design span* is exceeded, or if some of the above conditional factors are not satisfied the stand-up time may be less than "permanent".

A group of excavations which are probably frequently designed with spans exceeding the maximum design envelope are *temporary mine openings*. As a group, they can be subdivided since the required stand-up times will depend on the time it takes to finish extracting ore in the vicinity of, or in the excavation in question. The stand-up time actually available with a given span will depend both on the shape of the roof, and on the rock mass quality *Q*, and it will also depend on the care with which blasting is carried out, although this effect should be automatically incorporated in the estimate of *Q*.

It has been assumed here that the excavations that plot closest to the curved envelope in Fig. 1 (the *maximum design span*) have stand-up times in excess of 10 years. In view of the type of excavations involved it is obviously expected that they will stand unsupported for at least a "life-time", in other words more than 50 years. This conservative range of 10 to 50 years to represent "permanency" is used to obtain Fig. 2, which is a preliminary attempt at correlating stand-up time, *Q*, and unsupported span.

The envelopes have been truncated at various time intervals as a concession to the approximate minimum construction periods of the various dimensions of excavation. The equivalent unsupported span at any one time can be considered as the length from the face to the supported zone, or as the span itself, whichever is the smaller. Except for the smallest spans there will be a significant stand-up period concurrent with the advance of the successive blasting rounds.

The actual inclination of the shaded zones drawn for various spans is unknown. In other words for a given span the relationship between stand-up time and rock mass quality is unknown. However, it seems quite likely that future case records will show that stand-up time reduces more abruptly and unexpectedly in the poorer qualities of rock. The shaded zones would then tend to bend down towards the vertical as suggested in Fig. 2.

The envelopes presented in Fig. 2 have been used by Bieniawski (general review paper, this symposium) to compare the Geomechanics Classification and the *Q*-system. The Geomechanics Classification was based initially on Lauffer (1958), which is now acknowledged to be excessively conservative. Despite later modification based on South African case records, Bieniawski's chart of stand-up time versus *unsupported span* is still seen to be very conservative compared to the *Q*-system. In the best qualities of rock mass it is extremely conservative. This is clearly a reflection of the different tunneling practice in Scandinavia compared to South Africa. The greater confidence apparently exhibited in Scandinavian tunneling projects is clearly a function of the generally excellent rock, and the longer experience with excavations for civil engineering purposes.

The value of case records of tunnels that failed due to inadequate stand-up times cannot be overemphasised. The tunneling profession is constantly asked to assess the "factor-of-safety" of a given design. If we are honest we have to admit that our present state of knowledge is inadequate to allow us to come anywhere near the correct value. For this reason the back-analysis of a failed length of tunnel; the stand-up

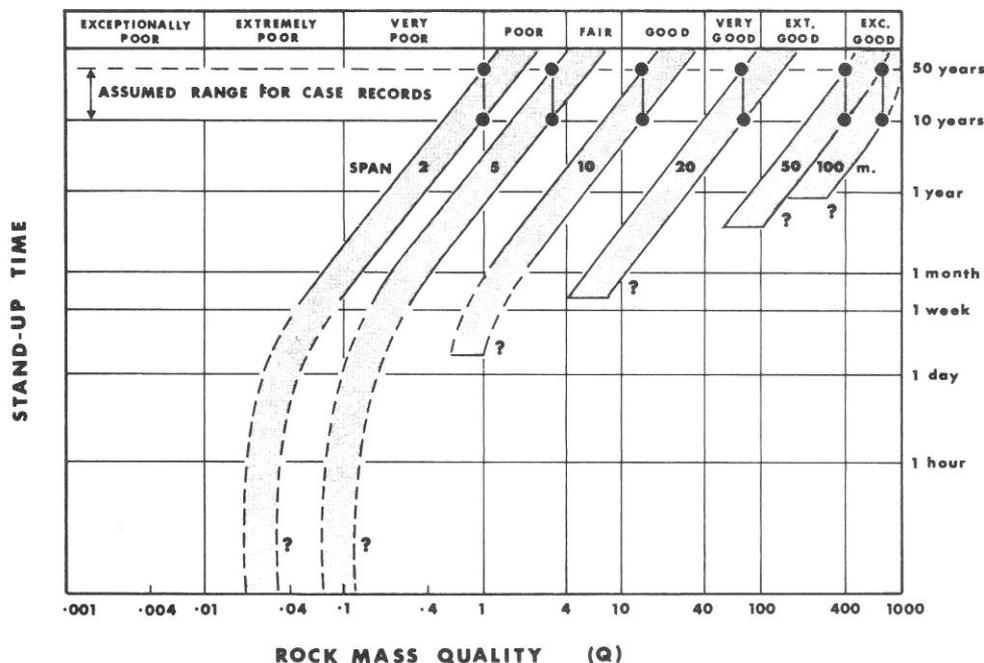


Fig. 2 The envelopes represent a preliminary attempt at predicting how much the stand-up time reduces when the span of an unsupported excavation is increased beyond the maximum design span (Fig. 1).

time, the Q-value and the span, can give us some indication of how conservative our present designs really are. We have to start on the "simple" case of unsupported openings, before attempting to assess the "factor-of-safety" of supported excavations, with all the associated complications of rock-support interaction.

When considering safety it should be remembered that the Q-system itself has a built in safety factor since it is firmly based on an engineering tradition that results in very few failures. Moreover, the majority of tunneling case records on which it was based were under construction or already built before 1970.

EXAMPLES OF APPLICATION OF Q-SYSTEM

The practical examples to be described here concern a power house and associated excavations and tunnels, which are presently being excavated at a depth of between 100 and 150 metres, mostly in quite massive biotite gneiss. The consultants claimed to have used the Q-system in their preliminary estimates of support requirements. However, the contractors, who were

widely experienced, doubted that the Q-system could have been used correctly, since the designed support was considered by them to be excessively conservative. This situation resulted in the contractor requesting an independent assessment of the rock mass conditions, and a re-assessment of the support requirements based on the Q-system.

1. Estimating support requirements from borecore logs.

The first re-assessment was based on geological reports and core logs made available by the contractor. No photographs were available, either of the core or of the existing excavations. Only later was the site visited and the Q-system applied *in-situ* in the existing tunnels and power house roof heading.

Five boreholes had been drilled in the neighbourhood of the power station. Four were vertical and one was inclined. The relevant core logs were studied, in particular the records of core recovered from between about 110 and 160 metres, which correspond to roughly ten metres above the roof down to the base of the excavation. "Best", "medium" and "poorest"

qualities were estimated from the relevant depths of each core. The following example shows the ratings estimated from the borehole that was most typical of the five holes. (The Appendix should be checked to obtain the appropriate verbal descriptions).

	Best	Medium	Poorest
RQD	100	90	70
J_n	3	4	9
J_r	2	2	1
J_a	1	2	4
J_w	1	1	0.66
SRF	1	1	2.5
Q	67	22	0.5

For the purpose of estimating the approximate overall support requirements, the average values obtained from the five lengths of borehole were used, equally weighted for each hole since there did not appear to be any hole with a particular advantage as regards location. The mean Q-values were 67, 20 and 1.2.

Geological engineering judgement was used to estimate the overall number of joint sets (J_n). The geological report contained descriptions of up to three joint sets on individual surface outcrops, though several additional joint orientations were plotted in polar diagrams. It appears that this may have been one source or error in the consultant's estimates of J_n , in other words the value of J_n was overestimated. The unbroken nature of most of the core made it unlikely that there were four or five joint sets in any one location. Therefore three sets were chosen to represent the "poorest" quality, since this corresponded to what was found at the surface in strongly weathered outcrops. The extrapolation to two sets for the "medium" quality, and one set plus random for the "best" was considered realistic in view of the excellent core recovery (mostly 100%) and the high RQD (mostly 100%) at the appropriate depths. (This assumption that the jointing was markedly less persistent at depth proved to be essentially correct on the subsequent site visit).

The joint roughness number (J_r) was generally guessed to be 2 (smooth, undulating) in view of the foliated nature of the gneiss, while for the "poorest" quality it was assumed to be 1 (smooth, planar).

The joint alteration number (J_a) was assumed to be 1 (unaltered joint walls) for the "best" quality, and down to 4 (chlorite coatings) for the "poorest" quality, since occasional chlorite and limonite coated joints were recorded in the corelog.

The joint water reduction factor (J_w) was generally assumed to be 1 (dry excavations, or minor inflow) though for the "poorest" quality it was assumed to be 0.66 (medium inflow or pressure, occasional outwash of joint fillings). Many of the Lugeon pumping tests showed "zero" permeability, exceptions generally corresponding to zones where the RQD values were lower than 80 or thereabouts.

Finally, the stress reduction factor (SRF) was assumed to be 1 (medium stress, σ_c/σ_t between 10 and 200) for the "best" and "medium" qualities. The

assumed value of the maximum principal stress (σ_1) was 50 kg/cm², based on a depth of 150 m, a level topography and a geological history that suggested that high horizontal stresses would be absent. The assumed value of σ_c was 800 kg/cm² for the biotite gneiss. This value was actually measured, but an informed guess would probably have been in this region anyway. (According to Table 6b of the Appendix, mild rock burst problems would not be encountered unless the ratio σ_c/σ_t dropped to between 5 and 2.5). The value of SRF assumed to best represent the "poorest" zones was 2.5 (see Table 6a, description C, Appendix).

The three mean values of Q (67, 20 and 1.2) were used to obtain estimates of permanent roof and wall support for the 19 m span, 31 m high power house using the support tables given by Barton et al. (1974, 1975).

	roof arch	walls
"Best" (Q=67)	untensioned, grouted bolts, 5m long, c/c 2.0m	spot bolts
"Medium" (Q=20)	tensioned, grouted bolts, 5m long, c/c 1.7m	spot bolts
"Poorest" (Q=1.2)	tensioned, grouted bolts, 5m long, c/c 1.0m + shotcrete, mesh reinforced, 15cm thick	tensioned, grouted bolts, 7m long, c/c 1.4m + shotcrete, mesh reinforced, 12cm thick

The above estimates of support were apparently in line with those considered appropriate by the contractor.

Note: It is general practice to use alternating bolt lengths in caverns of this size (B=19 m). For example, 4 m and 6 m long bolts could be used in the roof arch on an intermeshed pattern, while 4 m and 8 m long bolts could be used in the walls. It is also general practice - though possibly of questionable value - to use long tensioned anchors when the rock mass quality is as low as the poorest value (Q=1.2). However, since these zones were likely to be relatively narrow, with quite massive rock surrounding them, there did not seem to be any necessity for anchors. Careful orientation ("stitching") of the bolts across the weakness zones was recommended.

2. Estimating support requirements from in situ classification.

The site in question was visited approximately one month after the above estimates were made. Nine locations were selected in and around the power station. The roof arch was shotcreted at this stage though some 3 to 6 metres of the walls were excavated and parts were not shotcreted. Both end walls were bare. Other unsupported locations were selected in the immediate vicinity of the power house in an attempt to predict conditions likely to be encountered when the cavern height was increased to the maximum 31 metres.

The six classification parameters were estimated at each location. In the case of the end wall of the power house three separate assessments were made, one for the localised silty-clay bearing fracture

zone (which had the worst quality of all), and the other two assessments for the medium and better rock also found in the end wall.

The separate assessments fell into three groups. For statistical purposes these were simply averaged:

	RQD/J _n	J _r /J _a	J _w /SRF	Q
BEST	98/4.3	1.7/1.0	1/1	39
POORER ZONES	72/7	1.9/1.8	1/1	11
WORST	40/9	2/6	1/2.5	0.6

In terms of expected frequency of occurrence, it was estimated that more than 90% of the excavated surface in the power house (including roof and walls) would be of "best" quality, less than 10% of "poorer" quality, and probably only 1 or 2 % of "worst" quality. A careful assessment of available borecore suggested that only the existing top part of the end wall would be affected by the "worst" quality zones. The remainder yet to be excavated could well be up to the "best" quality.

The mean ratings for the majority of the rock mass (BEST, Q=39) can be translated into the following descriptions :

1. RQD = 98 (excellent)
2. J_n = 4.3 (approx. two joint sets)
3. J_r = 1.7 (rough-planar to smooth-undulating)
4. J_a = 1.0 (unaltered joints, surface staining)
5. J_w = 1.0 (dry excavations)
6. SRF = 1.0 (medium stress, no rock bursting)

(A very favourable quality was the non-planarity of the joints. The slight displacement resulting from excavation allows joints to shear slightly thereby increasing the favourable interlock effect. A non-planar joint dilates strongly when sheared, especially if the normal stress level is not too high.

The three mean values of Q (39, 11, and 0.6) estimated from the *in situ* classification are each about half the value estimated from the earlier classification of bore core logs (Q=67, 20 and 1.2). However, due to the logarithmic arrangement of the Q rating (i.e. POOR = 1 - 4, FAIR=4 - 10, GOOD=10 - 40 etc. see Figure 2) the two-fold discrepancy has a relatively small effect on the recommended permanent support. The support recommendations, which were again obtained from Barton et al.(1974, 1975), were as follows :

BEST	ca. 90%	Q=39	Roof: B 1.7m c/c + c1m Walls: sb
POORER ZONES	ca. 10%	Q=11	Roof: B 1.5m c/c + S(mr) 7cm Walls: B 1.6m c/c + c1m
WORST	1-2% ?	Q=0.6	Roof: B 1.0m c/c + S(mr)15cm Walls: B 1.2m c/c + S(mr)12cm
KEY :	B = systematic bolting with given c/c spacing sb = spot bolts S(mr) = mesh reinforced shotcrete c1m = chain link mesh or steel bands		

The above recommendations for support, especially those for the majority of the rock mass (Q=39) will obviously appear grossly under-conservative in

countries where a concrete lining has been a common feature of final tunnel support. However, it should not be forgotten that the support recommendations obtained from the Q-system were based on the analysis of about 200 case records, and 79 of these were in the power house category. Underground excavations are supported with some confidence primarily because many others have been supported before them and they have performed satisfactorily.

The particular support method recommended by the Q-system depends on the rock mass quality Q, the span or wall height (whichever is relevant), and the *type of excavation*. Power houses are naturally amongst the most important excavations, where safety has to be permanently assured. The support recommendations are therefore inherently conservative, and the factor of safety against collapse is likely to be quite high.

If Figure 1 is examined, it will be seen that the Q value of 39 (BEST) and the span of 19m, lie some 3 to 4m above the maximum design span for permanently unsupported openings. The recommended systematic bolting (c/c 1.7m) and the steel banding (a single layer of shotcrete might be preferred for aesthetic reasons) do indeed seem to be overdesign considering that the joint spacing was 1 to 2m and the existing joints relatively discontinuous anyway. In addition it may be noted that the mean ratings of the six rock mass parameters for the BEST quality (Q=39) rock satisfy all the *conditional factors* apparently needed for an excavation to be left permanently unsupported. These were listed earlier.

3. Estimating support requirements for tunnels

Estimating the support requirements for a tunnel that has yet to be excavated is obviously a difficult task, even if a large number of boreholes have been drilled. The problem is reduced somewhat if seismic measurements are available, although if the tunnel lies below the interface between the weathered zone and hard fresh rock, it is easy to underestimate the quality from seismic profiles. (In the present example the weathered zone extended down to a maximum depth of 40m.)

The problem of extrapolating the results of surface or near surface mapping to tunnel depth is clearly of considerable importance if cost estimates are to have any meaning. To take an example, one can consider a fault mapped at the surface. It might correctly be given the following classification :

$$RQD/J_n = 10/20, J_r/J_a = 1.0/8.0, J_w/SRF = 0.5/10$$

These ratings combine to give almost the worst possible quality Q=0.003 (EXCEPTIONALLY POOR), and correspondingly heavy support (cast concrete lining). The value of J_n = 20 represents "crushed rock, earth like" which may be a good description of the surface condition of many faults and weakness zones. However, at the tunnel depth of say 100m, the same fault might only be a relatively narrow zone of weakness, and the classification and resulting support should then also reflect the quality of the surrounding rock.

In the present example a planned 5km long tailrace tunnel trace was investigated with 15 irregularly spaced boreholes. As a first attempt at support prediction, the corelogs were examined between the

appropriate depths, which in this case ranged from about 150-160m at the upstream (powerhouse) end, down to only 10-20m close to the downstream portal. Estimates of "best", "poorer zones", and "worst" qualities were made from examination of each corelog. The nearest 10m both above and below the planned 8.8m span tunnel were considered. The classification took into account the expected looser and more weathered state of the rock mass where the depth of cover was less than 30-40m, as was the case near the portal.

The average Q values for the 15 holes were as follows

BEST Q=42
 POORER ZONES Q=12
 WORST Q=1.1

The variation from borehole to borehole was quite marked, as can be seen from the following maximum and minimum values :

BEST max. Q=100
 min. Q= 19
 POORER ZONES max. Q= 50
 min. Q=4.1
 WORST max. Q=19
 min. Q=0.03

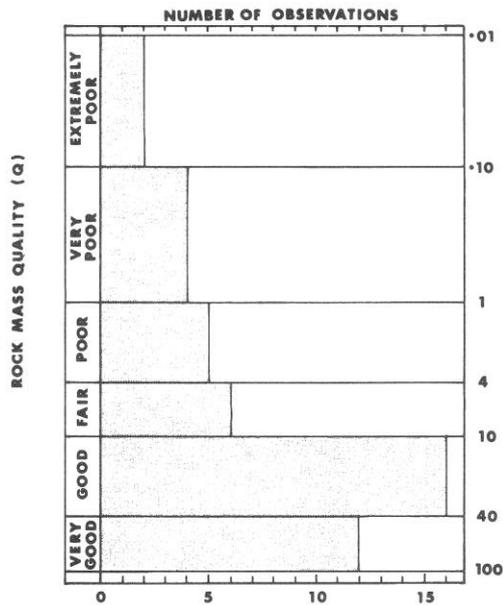


Fig. 3 The distribution of Q values from analysis of 15 corelogs for 5km long tailrace tunnel.

In view of the scatter the results were plotted as a histogram, as shown in Figure 3. The two minimum quality WORST zones had Q values of 0.07 and 0.03, and these were assumed to represent the quality of weakness zones at tunnel depth. Between 20 and 25 weakness zones were suspected from surface mapping and/or low seismic velocities.

The various estimates of permanent support are given below, based on a tunnel span and height of 8.8m and an ESR value equal to 1.3 appropriate to the relative importance of a tailrace tunnel. (ESR represents the type of excavation in terms of its relative safety requirement. The use of ESR values is described fully by Barton et al. 1974, 1975, and is summarised in the last section of this paper.)

BEST Q=42 Roof : none
 Wall : none
 POORER ZONES Q=12 Roof : B 1.5m c/c
 Wall : none
 WORST Q=1.1 Roof : B 1.0m c/c + S(mr) 5cm
 Wall : B 1.0m c/c + S 3cm,
 (or : S(mr) 5cm alone,
 depends on block size)
 FAULTS or WEAKNESS ZONES Q=0.05 Roof : S(mr) 20-25cm
 Wall : S(mr) 20-25cm
 (include invert)

KEY : B = systematic bolting with given c/c spacing
 S(mr) = mesh reinforced shotcrete
 sb = spot bolts

(Note : There was no evidence of swelling clays, therefore the Q values and recommended support are not exceptional.

Rock mass classification *in situ* in an existing unsupported tunnel clearly gives a much more reliable estimate of support than the above extrapolation of surface mapping and borehole data. Experience with the Q-system in many kilometers of tunnels shows it to be a very rapid method both of mapping essential parameters and of estimating support requirements on site. The input data is listed on a simple form for each length of tunnel considered to require different support from the adjacent length.

If the engineering geologist prefers to consult the support tables (Barton et al. 1974, 1975) in the luxury of a site office, then a short verbal description of the different zones needing support is helpful. Alternatively, the number and letter coding appropriate to each of the six parameters can be recorded. From the appendix it will be seen that a rockmass with the following characteristics is extremely favourable for tunnel stability :

1.E/2.A, 3.A/4.A, 5.A/6.K

CRITICAL Q CONCEPT IN TUNNEL MAPPING

In this last section another problem of extrapolation is considered. The problem is one of extrapolating

observations from pilot tunnels or access tunnels to the full scale excavation. Apart from the great advantage of more reliable mapping that a pilot tunnel affords, the performance of the tunnel itself can provide useful pointers to full scale behaviour. If light support is just needed to maintain a section of the pilot opening, then the Q-value can be *back analysed* and checked with the value estimated from the tunnel mapping. The reliability of the input data can then be assessed.

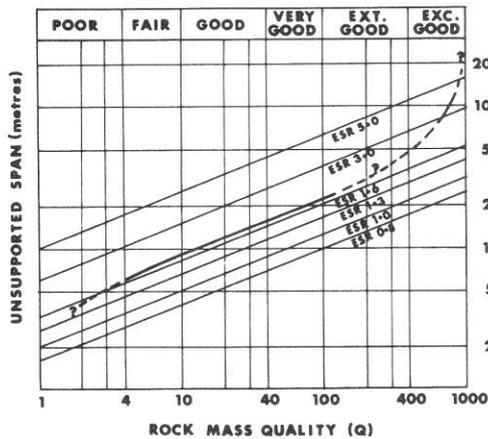


Fig. 4 Suggested design limits for the various types of excavation listed in the table opposite. ESR values greater than 1.6 apply to temporary openings, for which less stringent requirements concerning stand-up times are acceptable. Support may be required if spans in excess of the relevant design limits are excavated.

The maximum design span (curved envelope, Figure 1), has been redrawn in Figure 4, together with a set of parallel lines that represent the suggested span limits for unsupported excavations of various types. The table shows the suggested classes of excavation and their corresponding ESR values. The span width divided by ESR (SPAN/ESR) and the rock mass quality Q give a combined measure of the degree of support that is required. The actual span limits for permanently unsupported openings can be expressed as follows :

$$\text{SPAN} = 2 \cdot \text{ESR} \cdot Q^{0.4} \quad \dots \dots \dots (3)$$

The six parallel lines in Figure 4 correspond to this equation.

The vast majority of case records with spans exceeding these suggested design limits were supported in some way - for instance with various combinations of bolting, shotcrete and reinforcing mesh. Returning to the *pilot tunnel extrapolation*, it will be appreciated that a section that *almost/just* requires support gives a spot check of the Q value, if this in situ evidence of support needs is evaluated according to Figure 4. Thus a 2m span pilot tunnel (ESR=1.6) will not require support unless Q lies in the VERY POOR

TYPE OF EXCAVATION	ESR	NO. CASES
A. Temporary mine openings	ca. 3-5 ?	(2)
B. Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks), pilot tunnels, drifts and headings for large excavations etc.	1.6	(83)
C. Storage rooms, watertreatment plants, minor road and railway tunnels, surge chambers, access tunnels, etc. (hemispherical caverns)	1.3	(25)
D. Power houses, major road and railway tunnels, civil defence chambers, portals, intersections etc.	1.0	(79)
E. Underground nuclear power stations, sports and public facilities, factories etc.	0.8	(2)

category (Q=0.1 to 1). The theoretical Q value according to equation 4 would be 0.3, based on the rearranged form of equation 3 :

$$Q = \left(\frac{\text{SPAN}}{2 \cdot \text{ESR}} \right)^{2.5} \quad \dots \dots \dots (4)$$

In routine mapping of tunnels, the *critical* Q value should be determined from Figure 4 (or equation 4) at the outset, so that sections requiring support can be more rapidly distinguished from the sections that can be left permanently unsupported.

In the case of a pilot tunnel the range of Q values obtained from mapping and back analysis provides an invaluable and specific range of values for estimating support for the full scale excavation. This would be selected from the support chart and support tables given by Barton et al.(1974, 1975), using the same range of Q values, but the value of SPAN/ESR relevant to the full scale excavation. Naturally, if geological mapping suggested different conditions in parts of the full scale excavation, perhaps due to the nearness of a fault zone, then the Q values obtained from the pilot tunnel would have to be modified accordingly.

The support recommendations for the large scale excavation would generally incorporate thicker shotcrete or cast concrete arches, and of course longer bolts. However, the increase in thickness of the shotcrete or concrete only goes up in approximate proportion to the span width. The bolt spacing and theoretical support pressure remain roughly the same. This appears to be in line with present practice in large excavations, and is justified because of the efficiency of modern temporary support methods (i.e. shotcrete and bolting). It is only under extremely difficult ground conditions, where even temporary support is "too late" that a large span excavation is likely to require a higher designed support pressure than a pilot tunnel through the same ground. A careful multiple heading technique can presumably reduce the discrepancy.

CONCLUSIONS

Tunnel mapping and support prediction have been performed at a rate of up to several kilometers per day using the Q system. While it is extremely unwise to rush this important task, it does illustrate that the method is certainly not "too complicated to be generally acceptable in practice", as has been claimed recently by Pells (1975). The method is in fact embarrassingly simple, once the user becomes experienced.

The Q system is essentially a *weighting process*, in which the positive and negative aspects of a rockmass are assessed. A store of experience (case records), which is itself based on earlier experience, is searched to try to find the most appropriate support measures for the given excavations and rock mass conditions. The whole procedure is probably not dissimilar to the mental process occurring when a very experienced tunneling consultant is asked for his support recommendations. While the assessment of most of the parameters is admittedly subjective, the process of support selection is organised and reasonably consistent.

ACKNOWLEDGEMENTS

The detailed descriptions of rock conditions in some Scandinavian tunneling projects given by Cecil (1970) provided an invaluable store of data for stage by stage improvement of the classification system during its development in 1973. The interest and practical advice given by Reidar Lien and Johnny Lunde of NGI is sincerely acknowledged.

REFERENCES

- Barton, N. (1976)
Unsupported underground openings. *Rock Mechanics Discussion Meeting*, BeFo, Stockholm, February 1976.
- Barton, N., Lien, R. and Lunde, J. (1974)
Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics*, Springer Verlag, Vol. 6, pp. 189-236.
- Barton, N., Lien, R. and Lunde, J. (1975)
Estimation of support requirements for underground excavations. *Design Methods in Rock Mechanics*, Proc. 16th. Symp. on Rock Mech., Univ. of Minnesota.
- Bieniawski, Z.T. (1973)
Engineering classification of jointed rock masses. *The Civil Engineer in South Africa*, Dec. 1973, pp. 335-343.
- Bieniawski, Z.T. (1974)
Geomechanics classification of rock masses and its application in tunneling. *Advances in Rock Mechanics*, Proc. of 3rd. Cong. of Int. Soc. Rock Mech., Denver, 1974, Vol. II.A, pp. 27-32.
- Cecil, O.S. (1970)
Correlations of rock bolt - shotcrete support and rock quality parameters in Scandinavian tunnels. *Ph.D. Thesis*, Univ. of Illinois, Urbana, pp. 1-414. (Now published as Swedish Geotechnical Institute, Proceedings No. 27, Stockholm, 1975.)
- Deere, D.U. (1963)
Technical description of rock cores for engineering purposes. *Felsmekanik und Ingenieurgeologie*, Vol. 1, No. 1, pp. 16-22.
- Lauffer, H. (1958)
Gebirgsklassifizierung für den Stollenbau. *Geologie und Bauwesen*, Vol. 24, pp. 46-51.
- Palmstrøm, A. (1975)
Characterization of degree of jointing and rock mass quality. (In Norwegian) *Internal Report*, Ing. A.B. Bernal A/S, Oslo, pp. 1-26.
- Pells, P.J.N. (1975)
Discussion (of Barton, Lien, and Lunde, 1974), *Rock Mechanics*, Springer Verlag, Vol. 7, No. 4, pp. 246-248.

APPENDIX

Table 1. Descriptions and ratings for the parameter RQD.

1. ROCK QUALITY DESIGNATION (RQD)	
A. Very poor	0 - 25
B. Poor	25 - 50
C. Fair	50 - 75
D. Good	75 - 90
E. Excellent	90 - 100

Note: (i) Where RQD is reported or measured as ≤ 10 , (including 0) a nominal value of 10 is used to evaluate Q in equation (1).

(ii) RQD intervals of 5, i.e. 100, 95, 90, etc. are sufficiently accurate.

Table 2. Descriptions and ratings for the parameter J_n

2. JOINT SET NUMBER (J_n)	
A. Massive, no or few joints	0.5-1.0
B. One joint set	2
C. One joint set plus random	3
D. Two joint sets	4
E. Two joint sets plus random	6
F. Three joint sets	9
G. Three joint sets plus random	12
H. Four or more joint sets, random, heavily jointed, "sugar-cube" etc.	15
J. Crushed rock, earthlike	20

Note: (i) For intersections use $(3.0 \times J_n)$

(ii) For portals use $(2.0 \times J_n)$

Table 3. Descriptions and ratings for the parameter J_r

3. JOINT ROUGHNESS NUMBER	
(a) Rock wall contact and (b) Rock wall contact before 10 cms shear	(J_r)
A. Discontinuous joints	4
B. Rough or irregular, undulating	3
C. Smooth, undulating	2
D. Slickensided, undulating	1.5
E. Rough or irregular, planar	1.5
F. Smooth, planar	1.0
G. Slickensided, planar	0.5

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

(c) No rock wall contact when sheared

H. Zone containing clay minerals thick enough to prevent rock wall contact	1.0
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0

Note: (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.

(iii) $J_r = 0.5$ can be used for planar slickensided joints having lineations, provided the lineations are orientated for minimum strength.

Table 4. Descriptions and ratings for the parameter J_a

4. JOINT ALTERATION NUMBER (J_a)		(ϕ_r)
(a) Rock wall contact		(approx.)
A. Tightly healed, hard, non-softening, impermeable filling i.e. quartz or epidote	0.75	(-)
B. Unaltered joint walls, surface staining only	1.0	(25-35 ⁰)
C. Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	2.0	(25-30 ⁰)
D. Silty-, or sandy-clay coatings, small clay fraction (non-soft.)	3.0	(20-25 ⁰)
E. Softening or low friction clay mineral coatings, i.e. kaolinite or mica. Also chlorite, talc, gypsum, graphite etc., and small quantities of swelling clays	4.0	(8-16 ⁰)
(b) Rock wall contact before 10 cms shear		
F. Sandy particles, clay-free disintegrated rock etc.	4.0	(25-30 ⁰)
G. Strongly over-consolidated non-softening clay mineral fillings (continuous, but <5 mm thickness)	6.0	(16-24 ⁰)
H. Medium or low over-consolidation, softening, clay mineral fillings (continuous but <5 mm thickness)	8.0	(12-16 ⁰)
J. Swelling -clay fillings, i.e. montmorillonite (continuous, but <5 mm thickness). Value of J_r depends on percent of swelling clay-size particles, and access to water etc.	8-12	(6-12 ⁰)
(c) No rock wall contact when sheared		
K. Zones or bands of disintegrated L, or crushed rock and clay (see M, G, H, J for description of clay condition)	6, 8, or 8-12	(6-24 ⁰)
N. Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	5.0	(-)
O. Thick, continuous zones or P. bands of clay (see G, H, J for 10, 13, R. description of clay condition) or 13-20		(6-24 ⁰)

Table 5. Descriptions and ratings for the parameter J_w

5. JOINT WATER REDUCTION FACTOR (J_w)		Approx. water pres. (kg/cm ²)
A. Dry excavations or minor inflow, i.e. <5 l/min. locally	1.0	<1
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1-2.5
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5-10
D. Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5-10
E. Exceptionally high inflow or water pressure at blasting, decaying with time	0.2-0.1	>10

F. Exceptionally high inflow or water pressure continuing without noticeable decay (J_w) 0.1-0.05 >10
 Note: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
 (ii) Special problems caused by ice formation are not considered.

(c) Squeezing rock plastic flow of incompetent rock under the influence of high rock pressure (SRF)
 N. Mild squeezing rock pressure 5-10
 O. Heavy squeezing rock pressure 10-20
 (d) Swelling rock chemical swelling activity depending on presence of water
 P. Mild swelling rock pressure 5-10
 R. Heavy swelling rock pressure 10-15

Table 6. Description and ratings for parameter SRF

6. STRESS REDUCTION FACTOR

(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated. (SRF)

A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10
B. Single weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤ 50 m)			5
C. Single weakness zones containing clay or chemically disintegrated rock (depth of excavation >50 m)			2.5
D. Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)			7.5
E. Single shear zones in competent rock (clay-free) (depth of excavation ≤ 50 m) ..			5.0
F. Single shear zones in competent rock (clay-free) (depth of excavation >50 m) ..			2.5
G. Loose open joints, heavily jointed or "sugar cube" etc. (any depth)			5.0

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

(b) Competent rock, rock stress problems (SRF)

	σ_c/σ_1	σ_t/σ_1	(SRF)
H. Low stress, near surface	>200	>13	2.5
J. Medium stress	200-10	13-0.66	1.0
K. High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10-5	0.66-.33	0.5-2
L. Mild rock burst (massive rock)	5-2.5	0.33-.16	5-10
M. Heavy rock burst (massive rock)	<2.5	<0.16	10-20

Note: (ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c and σ_t to $0.8 \sigma_c$ and $0.8 \sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6 \sigma_c$ and $0.6 \sigma_t$, where: σ_c = unconfined compression strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.

(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

ADDITIONAL NOTES ON THE USE OF TABLES 1 TO 6.

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in Tables 1 to 6:

1. When borecore is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay-free rock masses (Palmström, 1975):

$$RQD = 115 - 3.3 J_v \text{ (approx.)}$$

where

$$J_v = \text{total number of joints per m}^3 \\ (RQD = 100 \text{ for } J_v < 4.5)$$

2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as an complete joint set. However, if there are few "joints" visible, or only occasional breaks in bore core due to these features, then it will be more appropriate to count them as "random joints" when evaluating J_n in Table 2.

3. The parameters J_r and J_d (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_d) is favourably oriented for stability, then a second, less favourably orientated joint set or discontinuity may sometimes be of more significance, and its higher value of J_r/J_d should be used when evaluating Q from equation 1. The value of J_r/J_d should in fact relate to the surface most likely to allow failure to initiate.

4. When a rock mass contains clay, the factor SRF appropriate to Loosening Loads should be evaluated (Table 6a). In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength (Table 6b). A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in Note (ii), Table 6b.

5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to present or future in situ conditions. A very conservative estimate of strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

