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# Rock Mass Classification and Tunnel Reinforcement Selection Using the Q-System

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**ABSTRACT:** This paper provides an overview of the Q-system and documents the scope of case records used in its development. A description of the rock mass classification method is given using the following six parameters: core recovery (RQD), number of joint sets, roughness and alteration of the least favorable discontinuities, water inflow, and stress-strength relationships. Examples of field mapping are given as an illustration of the practical application of the method in the tunneling environment, where the rock may already be partly covered by a temporary layer of shotcrete. The method is briefly compared with other classification methods, and the advantages of the method are emphasized.

**KEY WORDS:** rock mass, classification, tunnels, rock support, shotcrete, rock bolts jointing

This paper provides an analysis of the Q-system of rockmass characterization and tunnel support selection. The 212 case records utilized in developing the Q-system (Barton et al, 1974) are reviewed in detail, so that application to new projects can be related to the extensive range of rock mass qualities, tunnel sizes, and tunnel depths that constitute the Q-system data base.

Ultimately, a potential user of a classification method will be persuaded of the value of a particular system by the degree to which he can identify his site in the case records used to develop the given method. The most comprehensive data base of the seven or eight classification systems reviewed is utilized in the Q-system. This body of engineering experience ensures that support designs will be realistic rather than theoretical, and more objective than can be the case when few previous experiences are utilized to develop a support recommendation.

## Classification Systems Currently in Use

Table 1 is an abbreviated listing of most of the rock mass classification systems currently in use internationally in the field of tunneling. These are:

- *Terzaghi (1946) Rock Load Classification*—This has been used extensively in the United States for some 40 years. It is used primarily to select steel supports for rock tunnels. However, it is unsuitable for modern tunnelling methods in which rock bolts and shotcrete are used.

- *Lauffer (1958) Stand-Up Time Classification*—This introduced the concept of an unsupported span and its equivalent stand-up time, which was a function of rock mass quality. It appears excessively conservative when compared with present-day tunneling methods.

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TABLE 1—Major rock mass classification systems.

Name of Classification	Originator and Date <sup>a</sup>	Country of Origin	Applications
Rock Loads	Terzhagi (1946)	USA	tunnels with steel supports
Stand-Up-Time	Lauffer (1958)	Austria	tunneling
RQD	Deere et al (1967) and Deere et al (1970)	USA	core logging, tunnelling
RSR Concept	Wickham et al (1972)	USA	tunnels with steel supports
Geomechanics (RMR System)	Bieniawski (1973)	S. Africa	tunnels, mines, etc.
Q-System	Barton et al (1974)	Norway	tunnels, large chambers

<sup>a</sup> See Bibliography for details.

- *Deere et al (1967) Rock Quality Designation (RQD)*—This is a simple description of the condition of recovered drill core. It has been successfully adopted as part of subsequent classification systems. On its own, it fails to account for the condition of joint surfaces and filling materials, and may be overly sensitive to orientation effects. Deere et al (1970) utilized RQD to develop support recommendations for 6 to 12 m span tunnels, but pointed out that the details of jointing, weathering, and groundwater should also be taken into account when selecting support. Fourteen case records were utilized in developing these recommendations.

- *Wickham, Tiedemann, and Skinner (1972) Rock Structure Rating (RSR)*—This concept introduced numerical ratings and weightings to relate rock mass quality, excavation dimensions, and steel support requirements. The method was an immediate forerunner to the two methods now used most frequently on an international basis (the RMR and *Q* systems).

- *Bieniawski (1973) RMR Geomechanics Classification*—This evolved from several earlier systems and has undergone several changes (1974, 1975, 1976, and 1979) since its first introduction in 1973. The method was eventually based on 49 case records, though details of these cases with their relation to support recommendations have not been published. Recent applications of the RMR system have been made in mining, which has extended the data base considerably.

- *Barton, Lien, and Lunde (1974) Q-System*—This classification system was developed independently of the Wickham et al (1972) and Bieniawski (1973) methods, but it builds extensively on the RQD method of Deere et al (1967), introducing five additional parameters to modify the RQD value to account for the number of joint sets, the joint roughness and alteration (filling), the amount of water, and the various adverse features associated with loosening, high stress, squeezing, and swelling. The classification method and the associated support recommendations were based on an analysis of 212 case records. Full details of these cases are given later in this paper.

#### NATM

One further tunnel support concept which should be mentioned here is the New Austrian Tunneling Method (NATM), which was developed by Rabcewicz, Packer, and Müller (Rabcewicz, 1963 and Rabcewicz and Packer, 1975). As acknowledged by Müller (Salzburg), almost everyone using this method has a different conception of it, and numerous economic and technical failures in past years demonstrate the amount of confusion that prevails in this field. In NATM's defense, it must be understood that the method is principally utilized in squeezing ground conditions, which could present technical and economic problems for any tunneling method.

The NATM relies on performance monitoring for prediction and classification of ground conditions. It is adapted to each new project based on previous experience. The classification is also adapted during a single project based on performance monitoring. A particular classification is therefore only applicable to the one case for which it was developed and modified, so use by others on other projects may be difficult.

The NATM is essentially a design method in which the rock mass is allowed to yield only enough to mobilize its optimum strength, by utilizing light temporary support. With correct timing of final support, this initial yielding is arrested in time to prevent loss of strength. On occasion, the desire to allow deformation to occur by installing canals of deformable material within the shotcrete, and steel ribs with sliding joints, has resulted in loss of ground control and severe damage to final concrete linings and bolt arrays (Barton, 1982).

#### *Updating Case Records*

Some of the support methods recommended by the above classification methods are quite labor intensive and will need updating as new support methods become more generally available. For example, the development of high strength, but highly ductile, steel fiber reinforced microsilica shotcrete is a revolutionary advance in tunnel support. It can be applied by one robot operator and one back-up person right at the tunnel face. The extra strength of this product removes the need for mesh in shotcrete, and it has sufficient early strength to replace steel arches and cast concrete under a large range of tunnelling conditions.

#### **Comparison of RMR and *Q* Systems**

The two classification systems that appear to be in widest use in tunneling that do not rely on performance monitoring (though they can be used in conjunction with monitoring) are the RMR and *Q* systems. These two systems are therefore compared in some detail here.

Bieniawski (1976) rates the following six parameters in his RMR 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.

In contrast, the *Q*-system (Barton et al, 1974) rates the following six parameters:

1. RQD.
2. Number of joint sets.
3. Joint roughness.
4. Joint alteration.
5. Joint water.
6. Stress factor.

The common parameter used in both systems is Deere's RQD. Bieniawski also includes joint spacing and orientation, while the *Q*-system considers the number of joint sets. Orientation is included implicitly in the *Q*-system by classifying the joint roughness and alteration of only the most unfavorably oriented joint sets or discontinuities.

Bieniawski (1975) appears to have favored the mean rating for spacing and orientation of the

different joint sets according to the example given in his paper. He also indicates (1979) that when only two joint sets are present the average spacing will prove conservative, since the rating is usually based on the presence of three sets of joints.

The very detailed treatment of joint roughness and alteration, perhaps the strongest feature of the  $Q$ -system, is not particularly emphasized in the RMR Geomechanics Classification. In his original version Bieniawski (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 Bieniawski condensed these three terms to “condition of joints” which again had five ratings; from *very tight*, *separation <0.1 mm, not continuous*, up to *open >5 mm, continuous gouge >5 mm*. In his later publications (1976, 1979), Bieniawski also includes joint roughness in his fourth parameter “condition of joints.”

In the RMR system, rock stress is not used specifically as a parameter though it is apparently when selecting support measures. In his 1975 paper Bieniawski gives support recommendation for a tunnel of 5 to 12 m span in which the vertical stress should be less than 30 MPa. In his 1976 version, the same support recommendations are given specifically for 10 m span tunnels, with the vertical stress limited to 25 MPa.

In the  $Q$ -system, the ratio  $(\sigma_c/\sigma_1)$  (unconfined compression strength/major principal stress) is evaluated when treating rock stress problems. The onset of popping, slabbing, and rock burst problems can be quite accurately predicted in hard rocks. The  $Q$ -system also accounts for loosening caused by shear zones and faults, and squeezing and swelling ground. However, very few case records could be utilized in the squeezing category, so support recommendations are tentative.

The Geomechanics Classification was based initially on Lauffer (1958), which is now acknowledged to be excessively conservative. Bieniawski increased Lauffer’s maximum unsupported span of 8 m (in his 1973 version) to 20 m (1975 version) and finally to 30 m (1979 version). Despite these later modifications, Bieniawski’s chart of stand-up time versus unsupported span is still seen to be very conservative compared with the  $Q$ -system.

In a detailed comparison of various classification methods, Einstein et al (1979) compared each method’s support estimates with Cecil’s (1970) unsupported cases. They showed that Bieniawski’s (1976) method predicted considerable support in all cases, the Deere et al (1969) method even larger amounts of support. The  $Q$ -system predicted no-support, since Cecil’s cases formed the backbone of the method. Such discrepancies reflect the important differences in support philosophies between Scandinavia, South Africa, and the United States. These differences appear to be narrowing gradually.

The Einstein et al (1979) comparison of each method with Cecil’s supported cases indicated that the  $Q$ -system support recommendations also agree well with the actual support. The Bieniawski (1976) and Deere et al (1969) RQD method were more conservative.

One of the most recent reports of comparisons between rock mass classification systems for tunneling was published by Einstein et al (1983) from work performed at the Porter Square Station, a 168 by 14 by 21 m excavation located 20 to 30 m below the surface in argillite, in Cambridge, Massachusetts. Five of the classification methods listed in Table 1 were compared in detail during several stages of the project and by several observers.

The methods were compared while classifying drill core, mapping an inspection shaft and a pilot tunnel, and when the rock was exposed in the final excavation. Only the  $Q$ -system was found to be applicable to the worst conditions encountered in the main excavation. Category 32 support predicted by the  $Q$ -system, consisting of systematic bolts at 1 m centers, and 40 to 60 cm of shotcrete, was “practically identical to the actual support” used. Careful monitoring using MPBX (multiple position borehole extensometers) and convergence measurements revealed maximum deformations of only 8 mm; in other words, stable conditions were established.

## Description of the *Q*-System

### *Rationale*

The vast majority of the thousands of kilometers of tunnels constructed world-wide every year do not have the benefit of performance monitoring. Design decisions are nevertheless required both before and during construction. No matter how many sophisticated rock mechanics test programs and finite element analyses are performed, design engineers will come back to the basic question: "Is this bolt spacing, shotcrete thickness, or unsupported span width reasonable in the given rock mass?"

At present we have to rely on engineering judgment, or on classification methods, where the design is based on precedent, and where a good classification method will allow us to extrapolate past designs to different rock masses and to different sizes and types of excavation. Underground excavations can be supported with some confidence primarily because many others have been supported before them and have performed satisfactorily.

### *Method for Estimating Rock Mass Quality (Q)*

The six parameters chosen to describe the rock mass quality (*Q*) are combined in the following way:

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

where

RQD = rock quality designation (Deere et al, 1967),

$J_n$  = joint set number,

$J_r$  = joint roughness number (of least favorable discontinuity or joint set),

$J_a$  = joint alteration number (of least favorable discontinuity or joint set),

$J_w$  = joint water reduction factor, and

SRF = stress reduction factor.

The three pairs of ratios ( $RQD/J_n$ ,  $J_r/J_a$  and  $J_w/SRF$ ) represent block size, minimum inter-block shear strength, and active stress, respectively. These are fundamental geotechnical parameters.

It is important to observe that the values of  $J_r$  and  $J_a$  relate to the joint set or discontinuity most likely to allow failure to initiate. The important influence of orientation relative to the tunnel axis is implicit.

Detailed descriptions of the six parameters and their numerical ratings are given in Table 2. The range of possible *Q* values (approximately 0.001 to 1000) encompasses the whole spectrum of rock mass qualities from heavy squeezing ground up to sound unjointed rock. The case records examined included 13 igneous rock types, 26 metamorphic rock types, and 11 sedimentary rock types. More than 80 of the case records involved clay occurrences. However, most commonly the joints were unfilled and the joint walls were unaltered or only slightly altered.

The *Q*-system is more detailed than any of the other methods as regards the factors joint roughness (or degree of planarity), joint alteration (filling), and relative orientation. The classification of "least favorable features" (for  $J_r$  and  $J_a$ ) represents one of the strongest features of the method. It also seems to be a factor that is virtually ignored in the other classification schemes. For example, in Bieniawski's RMR method, although data for all joint set and discontinuities are collected, only the average data are incorporated in the numerical ratings. Furthermore, in the RMR it is impossible to separately vary the degree of joint roughness and the degree of infilling, as obviously may occur in practice.

TABLE 2—Ratings for the six Q-system parameters  
(table continues on pp. 65 and 66).

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  $\leq 10$ , (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.

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  $(3.0 \times J_n)$

Note: (ii) For portals use  $(2.0 J_n)$

3. JOINT ROUGHNESS NUMBER		( $J_r$ )
(a) Rock wall contact and (b) Rock wall contact before 10 cms shear		
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 3m.

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

TABLE 2—(continued).

4. JOINT ALTERATION NUMBER	( $J_a$ )	( $\phi_r$ )
(a) <i>Rock wall contact</i>		(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')
C. Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc. ....	2.0	(25-30')
D. Silty-, or sandy-clay coatings, small clay fraction (non-soft.)	3.0	(20-25')
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')
(b) <i>Rock wall contact before 10 cms shear</i>		
F. Sandy particles, clay-free disintegrated rock etc. ....	4.0	(25-30')
G. Strongly over-consolidated non-softening clay mineral fillings (continuous, but <5 mm thickness) .....	6.0	(16-24')
H. Medium or low over-consolidation, softening, clay mineral fillings. (continuous but <5mm thickness) .....	8.0	(12-16')
J. Swelling -clay fillings, i.e. montmorillonite (continuous, but <5mm thickness) Value of $J_a$ depends on percent of swelling clay-size particles, and access to water etc. ....	8 - 12	(6-12')
(c) <i>No rock wall contact when sheared</i>		
K, L, Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition) .....	6, 8, or 8-12	(6-24')
N. Zones or bands of silty- or sandy-clay, small clay fraction (non-softening) ..	5.0	( - )
O, P, Thick, continuous zones or bands of clay (see G, H, J for description of clay condition) .....	10, 13, or 13-20	(6-24')

5. JOINT WATER REDUCTION FACTOR	( $J_w$ )	Approx. water pres. (kg/cm <sup>2</sup> )
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 .....	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.

TABLE 2—(continued).

6. STRESS REDUCTION FACTOR				
(a) <i>Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.</i>				
			(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 $\leq$ 50m) .....		5	
C.	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50m) .....		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 $\leq$ 50m) .....		5.0	
F.	Single shear zones in competent rock (clay-free) (depth of excavation > 50m) .....		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) <i>Competent rock, rock stress problems</i>				
	$\sigma_c/\sigma_1$	$\sigma_t/\sigma_1$	(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 $\sigma_c$ and $\sigma_t$ to $0.8\sigma_c$ and $0.8\sigma_t$ . When $\sigma_1/\sigma_3 > 10$ , reduce $\sigma_c$ and $\sigma_t$ to $0.6\sigma_c$ and $0.6\sigma_t$ , where: $\sigma_c$ = unconfined compression strength, and $\sigma_t$ = tensile strength (point load), and $\sigma_1$ and $\sigma_3$ 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).				
(c) <i>Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure</i>				
			(SRF)	
N.	Mild squeezing rock pressure .....		5 - 10	
O.	Heavy squeezing rock pressure .....		10 - 20	
(d) <i>Swelling rock: chemical swelling activity depending on presence of water</i>				
P.	Mild swelling rock pressure .....		5 - 10	
R.	Heavy swelling rock pressure .....		10 - 15	

### *Method of Selecting Suitable Support*

The  $Q$ -system is essentially a weighting process, in which the positive and negative aspects of a rock mass 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 organized and reasonably objective. The trial-and-error adjustment and readjustment of parameter ratings necessary during the development of the  $Q$ -system was an important factor in reducing the need for subjective judgments on the part of the developers. The large number of case records made it possible to generate the support recommendations quite objectively.

Figure 1 shows that the tunnel or excavation span width and the rock mass quality ( $Q$ ) are the decisive parameters for placing an excavation in a given support category (Boxes 1 to 38). However, there is an important user requirement for different degrees of safety. The excavation support ratio (ESR), which reduces the effective span in Fig. 1, reflects construction practice in that the degree of safety and support demanded by an excavation is determined by the purpose, presence of machinery, personnel, etc. The list of ESR values in Table 3 was developed through exhaustive trial and error, and seems to be the most workable solution to the problem of variable safety requirements.

Increased safety can be selected at will by reducing the ESR value (e.g. by using  $ESR = 1.3$  for an important permanent mine opening in place of 1.6). Similarly, support for oil storage caverns could be selected by using  $ESR = 1.6$  instead of 1.3, assuming the occasional fall of small stones (from the walls) were acceptable. The use of  $ESR = 1.0$  for power station chambers and major road tunnels ensures a high factor of safety, as obviously required.

Most of the 38 numbered "support boxes" shown in Fig. 1 contain case records. The support recommendations for cases that plot in these same boxes are listed in Table 4. Further details of the use of these support recommendations are given by Barton, Lien, and Lunde (1975). Roof support, wall support, and temporary support can be selected as required.

### *Examples of Case Records*

A considerable data base for developing the  $Q$ -system was provided by Cecil (1970), who described numerous tunneling projects in Sweden and Norway, including detailed evaluations of the rock, the jointing, the type of support, and the apparent stability. Figure 2 shows three examples of Cecil's cases, and Table 5 gives the abbreviated descriptions of the key rock mass parameters and describes the support actually used. Note that the alphabetic descriptions given in brackets in Table 5 can be checked directly against the same letters given in Table 2. This convenient shorthand method can be used during tunnel mapping, when writing conditions are unfavorable.

### *Examples of Tunnel Mapping*

Examples of some tunnel projects in which the  $Q$ -system has been used extensively in day-to-day follow-up mapping are shown in Figs. 3a and 3b. It will be noticed that the treatment of crushed zones and major discontinuities shown in Figs. 3a and 3b is often on an individual basis. The quality of the rock mass between the zones is a decisive factor in deciding between individual "stitching" and general support.

One of the examples (Fig. 3b) is a tunnel excavated by a full-face tunnel boring machine (TBM). The minimal disturbance caused by TBM excavation makes it particularly important to map as close to the advancing face as possible (for optimal joint definition), followed by repeated

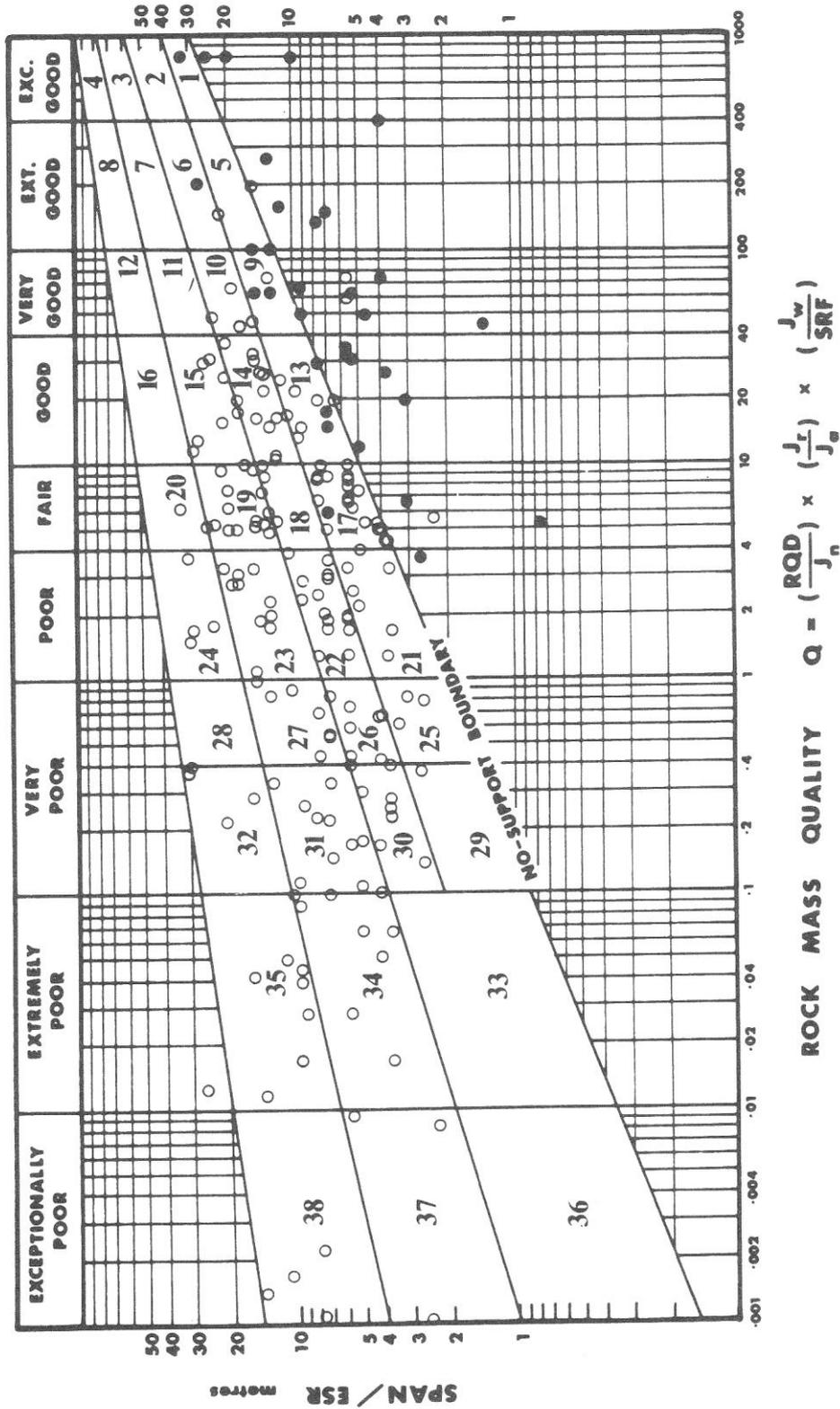


FIG. 1—Rock support category is given by the box numbers (1 to 38); refer to Table 4.

TABLE 3—Excavation Support Ratio (ESR) for a variety of underground excavations.

Type of Excavation	ESR	Number of Cases
A. Temporary mine openings etc.	ca. 3–5?	2
B. Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large openings	1.6	83
C. Storage caverns, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels, etc.	1.3	25
D. Power stations, major road and railway tunnels, civil defense chambers, portals, intersections	1.0	79
E. Underground nuclear power stations, railway stations, sports and public facilities, factories	ca. 0.8?	2

mapping before permanent support is chosen. There will then be improved possibilities for observing the character of narrow clay-bearing discontinuities. The effective RQD of the zone of rock around a TBM excavated tunnel will generally be higher than that around a blasted tunnel owing to the relatively slight disturbance of incipient joints and tight structures.

Figure 4 illustrates ten parallel (100 m long) sewage treatment caverns constructed near Oslo. At the feasibility and planning stage, surface mapping and drill core analysis were interpreted in terms of the  $Q$ -system parameters. Support requirements were predicted on this basis. During construction, support decisions were also guided by the method outlined. The general improvement in rock conditions as the parallel caverns advanced from the shale into the nodular limestone were clearly reflected in the six parameters and support was reduced accordingly:

**Shale:** B 1.25 m c/c, L = 3.5 m + S (mr) 12-15 cm

**Nodular limestone:** B 1.5 m c/c, L = 3.5 m + S 5 cm

(B = bolting, c/c = spacing, L = length, S = shotcrete, mr = mesh reinforced)

The  $Q$ -system has also been used in the area of mine stability. In a recent assessment of stability in two limestone mines with 13 to 15 m span rooms or drifts, respectively, the quality of the limestone varied as follows:

$$Q = \frac{80 - 100}{4 - 9} \times \frac{1 - 1.5}{1 - 2} \times \frac{1}{1} = 4 - 38 \text{ (fair - good)}$$

In the great majority of the drifts the quality was "good" ( $Q = 18 - 38$ ). By comparing these qualities with the permanently unsupported cases (Fig. 1, black circles) it was possible to demonstrate satisfactory conditions for the great majority of the excavations. However, in places, stability was apparently nearer the "temporary mine openings" category (ESR = 3–5, Table 1). There were in fact limited areas in the mines where fall-out occurred from the pillars or walls. A limited number of pillars in one of the mines were instrumented as a precaution.

#### *Rock Mass Requirements for Permanently Unsupported Excavations*

An important area of application for the  $Q$ -system is the recognition of rock mass characteristics required for safe operation of permanently unsupported openings. The relationship between the maximum unsupported span and the  $Q$ -value is clearly seen in Fig. 1. Detailed analysis of the available case records reveals the following requirements:

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TABLE 4—Support recommendations for the 38 categories shown in Fig. 1 (see Barton et al, 1974, 1975 for notes; table continues on pp. 71 and 72).

Support category	Conditional factors			Type of support	Notes
	RQD $J_n$	$J_r$ $J_a$	SPAN ESR		
1*	—	—	—	sb(utg)	—
2*	—	—	—	sb(utg)	—
3*	—	—	—	sb(utg)	—
4*	—	—	—	sb(utg)	—
5*	—	—	—	sb(utg)	—
6*	—	—	—	sb(utg)	—
7*	—	—	—	sb(utg)	—
8*	—	—	—	sb(utg)	—
Note: The type of support to be used in categories 1 to 8 will depend on the blasting technique. Smooth wall blasting and thorough barring-down may remove the need for support. Rough-wall blasting may result in the need for single applications of shotcrete, especially where the excavation height is >25 m. Future case records should differentiate categories 1 to 8.					
9	$\geq 20$	—	—	sb(utg)	—
	<20	—	—	B(utg) 2.5–3 m	—
10	$\geq 30$	—	—	B(utg) 2–3 m	—
	<30	—	—	B(utg) 1.5–2 m + clm	—
11*	$\geq 30$	—	—	B(tg) 2–3 m	—
	<30	—	—	B(tg) 1.5–2 m + clm	—
12*	$\geq 30$	—	—	B(tg) 2–3 m	—
	<30	—	—	B(tg) 1.5–2 m + clm	—
13	$\geq 10$	$\geq 1.5$	—	sb(utg)	I
	$\geq 10$	<1.5	—	B(utg) 1.5–2 m	I
	<10	$\geq 1.5$	—	B(utg) 1.5–2 m	I
	<10	<1.5	—	B(utg) 1.5–2 m + S 2–3 cm	I
14	$\geq 10$	—	$\geq 15$	B(tg) 1.5–2 m + clm	I, II
	<10	—	$\geq 15$	B(tg) 1.5–2 m + S(mr) 5–10 cm	I, II
	—	—	<15	B(utg) 1.5–2 m + clm	I, III
15	>10	—	—	B(tg) 1.5–2 m + clm	I, II, IV
	$\geq 10$	—	—	B(tg) 1.5–2 m + S(mr) 5–10 cm	I, II, IV
16* See note XII	>15	—	—	B(tg) 1.5–2 m + clm	I, V, VI
	$\geq 15$	—	—	B(tg) 1.5–2 m + S(mr) 10–15 cm	I, V, VI
17	>30	—	—	sb(utg)	I
	( $\geq 10$ , $\geq 30$ )	—	—	B(utg) 1–1.5 m	I
	<10	—	$\geq 6$ m	B(utg) 1–1.5 m + S 2–3 cm	I
	<10	—	<6 m	S 2–3 cm	I
18	>5	—	$\geq 10$ m	B(tg) 1–1.5 m + clm	I, III
	>5	—	<10 m	B(utg) 1–1.5 m + clm	I
	$\geq 5$	—	$\geq 10$ m	B(tg) 1–1.5 m + S 2–3 cm	I, III
	$\geq 5$	—	<10 m	B(utg) 1–1.5 m + S 2–3 cm	I

TABLE 4—(continued).

Support category	Conditional factors			Type of support	Notes
	$\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{SPAN}{ESR}$		
19	—	—	$\geq 20$ m	B(tg) 1–2 m + S(mr) 10–15 cm	I, II, IV
	—	—	<20 m	B(tg) 1–1.5 m + S(mr) 5–10 cm	I, II
20* See note XII	—	—	$\geq 35$ m	B(tg) 1–2 m + S(mr) 20–25 cm	I, V, VI
	—	—	<35 m	B(tg) 1–2 m + S(mr) 10–20 cm	I, II, IV
21	$\geq 12.5$	$\leq 0.75$	—	B(utg) 1 m + S 2–3 cm	I
	<12.5	$\leq 0.75$	—	S 2.5–5 cm	I
	—	>0.75	—	B(utg) 1 m	I
22	( $>10,$ $<30$ )	>1.0	—	B(utg) 1 m + clm	I
	$\leq 10$	>1.0	—	S 2.5–7.5 cm	I
	<30	$\leq 1.0$	—	B(utg) 1 m + S(mr) 2.5–5 cm	I
	$\geq 30$	—	—	B(utg) 1 m	I
23	—	—	$\geq 15$ m	B(tg) 1–1.5 m + S (mr) 10–15 cm	I, II, IV, VII
	—	—	<15 m	B(utg) 1–1.5 m + S(mr) 5–10 cm	I
24* See note XII	—	—	$\geq 30$ m	B(tg) 1–1.5 m + S(mr) 15–30 cm	I, V, VI
	—	—	<30 m	B(tg) 1–1.5 m + S(mr) 10–15 cm	I, II, IV
25	>10	>0.5	—	B(utg) 1 m + mr or clm	I
	$\leq 10$	>0.5	—	B(utg) 1 m + S(mr) 5 cm	I
	—	$\leq 0.5$	—	B(tg) 1 m + S(mr) 5 cm	I
26	—	—	—	B(tg) 1 m + S(mr) 5–7.5 cm	VIII, X, XI
	—	—	—	B(utg) 1 m + S 2.5–5 cm	I, IX
	—	—	—	B(tg) 1 m + S(mr) 7.5–10 cm	I, IX
27	—	—	$\geq 12$ m	B(utg) 1 m + S(mr) 5–7.5 cm	I, IX
	—	—	<12 m	B(utg) 1 m + S(mr) 5–7.5 cm	I, IX
	—	—	>12 m	CCA 20–40 cm + B(tg) 1 m	VIII, X, XI
	—	—	<12 m	S(mr) 10–20 cm + B(tg) 1 m	VIII, X, XI
28* See note XII	—	—	$\geq 30$ m	B(tg) 1 m + S(mr) 30–40 cm	I, IV, V, IX
	—	—	( $\geq 20,$ $<30$ m)	B(tg) 1 m + S(mr) 20–30 cm	I, II, IV, IX
	—	—	<20 m	B(tg) 1 m + S(mr) 15–20 cm	I, II, IX
	—	—	—	CCA(sr) 30–100 cm + B(tg) 1 m	IV, VIII, X, XI
29*	>5	>0.25	—	B(utg) 1 m + S 2–3 cm	—
	$\leq 5$	>0.25	—	B(utg) 1 m + S(mr) 5 cm	—
	—	$\leq 0.25$	—	B(tg) 1 m + S(mr) 5 cm	—

TABLE 4—(continued).

Support category	Conditional factors			Type of support	Notes
	$\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{SPAN}{ESR}$		
30	$\geq 5$	—	—	B(tg) 1 m + S 2.5–5 cm	IX
	$< 5$	—	—	S(mr) 5–7.5 cm	IX
	—	—	—	B(tg) 1 m + S(mr) 5–7.5 cm	VIII, X, XI
31	$> 4$	—	—	B(tg) 1 m + S(mr) 5–12.5 cm	IX
	$\geq 4, \geq 1.5$	—	—	S(mr) 7.5–25 cm	IX
	$< 1.5$	—	—	CCA 20–40 cm + B(tg) 1 m	IX, XI
	—	—	—	CCA(sr) 30–50 cm + B(tg) 1 m	VIII, X, XI
32 See note XII	—	—	$\geq 20m$	B(tg) 1 m + S(mr) 40–60 cm	II, IV, IX, XI
	—	—	$< 20m$	B(tg) 1 m + S(mr) 20–40 cm	III, IV, XI IX
	—	—	—	CCA(sr) 40–120 cm + B(tg) 1 m	IV, VIII, X, XI
33*	$\geq 2$	—	—	B(tg) 1 m + S(mr) 2.5–5 cm	IX
	$< 2$	—	—	S(mr) 5–10 cm	IX
	—	—	—	S(mr) 7.5–15 cm	VIII, X
34	$\geq 2$	$\geq 0.25$	—	B(tg) 1 m + S(mr) 5–7.5 cm	IX
	$< 2$	$\geq 0.25$	—	S(mr) 7.5–15 cm	IX
	—	$< 0.25$	—	S(mr) 15–25 cm	IX
	—	—	—	CCA(sr) 20–60 cm + B(tg) 1 m	VIII, X, XI
35 See note XII	—	—	$\geq 15m$	B(tg) 1 m + S(mr) 30–100 cm	II, IX, XI
	—	—	$\geq 15m$	CCA(sr) 60–200 cm + B(tg) 1 m	VIII, X, XI, II
	—	—	$< 15m$	B(tg) 1 m + S(mr) 20–75 cm	IX, III, XI
	—	—	$< 15m$	CCA(sr) 40–150 cm + B(tg) 1 m	VIII, X, XI, III
36*	—	—	—	S(mr) 10–20 cm	IX
	—	—	—	S(mr) 10–20 cm + B(tg) 0.5–1.0 m	VIII, X, XI
	—	—	—	S(mr) 20–60 cm + B(tg) 0.5–1.0 m	IX VIII, X, XI
37	—	—	—	S(mr) 20–60 cm	IX
	—	—	—	S(mr) 20–60 cm + B(tg) 0.5–1.0 m	VIII, X, XI
	—	—	$\geq 10m$	CCA(sr) 100–300 cm	IX
	—	—	$\geq 10m$	CCA(sr) 100–300 cm + B(tg) 1 m	VIII, X, II, XI
38 See note XIII	—	—	$< 10m$	S(mr) 70–200 cm	IX
	—	—	$< 10m$	S(mr) 70–200 cm + B(tg) 1 m	VIII, X, III, XI
* Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.					
Key to Support Tables:					
sb = spot bolting					
B = systematic bolting					
(utg) = untensioned, grouted					
(tg) = tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; see Note XI)					
S = shotcrete					
(mr) = mesh reinforced					
clm = chain link mesh					
CCA = cast concrete arch					
(sr) = steel reinforced					

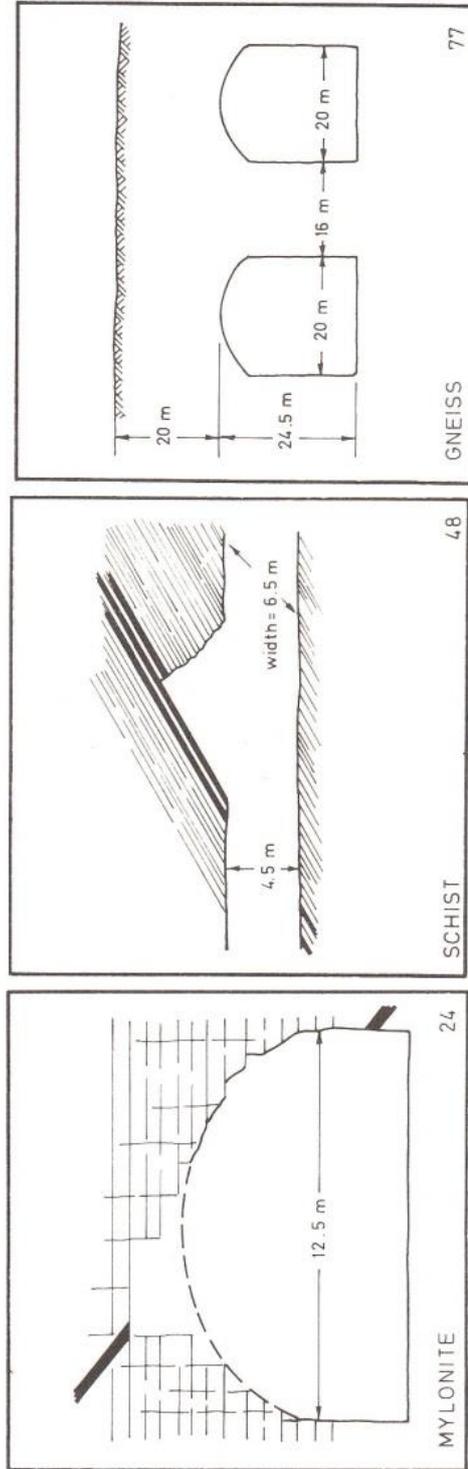


FIG. 2—Examples of three cases mapped by Cecil (1970).

TABLE 5—Comparison of support used and support recommended for three case records described by Cecil (1970).

Case No.	DESCRIPTION OF ROCK MASS	SPAN (m)	Height (m)	Depth (m)	Support used	$\frac{RQD}{J_n}$ (Code : Tables 1 to 6)	$\frac{J_a}{J_a}$	$\frac{J_w}{SRF}$	Q	ESR	$\frac{SPAN}{ESR}$	Estimate of permanent roof support
24	1. 60 m length, including a 1 m wide shear zone in mylonite. Crushed clay seams and non-softening fillings. Intersecting joint set. 2 joint sets plus random, 5-30 cm spacing. Minor water inflows (<3l/min). RQD = 60 2. Wedge shaped roof fall. 3. Headrace tunnel, Vietas Hydro, N.Sweden (ref. Cecil 1970).	12.5	6.5	60	Rock bolts, wire mesh and shotcrete	60 6	1.0 6	1.0 2.5				Category 22 =B 1 m +S (m <sub>r</sub> ) 2.5-5 cm
									1.3	1.6	7.8	

<p>48</p> <p>1. 15 m length, overthrust shear zone in schist, in which there was a 3 cm thick clay (non softening) and graphite seam. Shear zone was 50-100 cm wide and contained smooth, slickensided graphite-coated joint surfaces, 1 joint set, 5-30 cm spacing. Insignificant water inflow. RQD = 10</p> <p>2. Wedge-shaped roof fall.</p> <p>3. Tailrace tunnel, Bergvattnet, Hydro, N.Sweden (ref. Cecil 1970)</p>	<p>6.5 4.5 50</p>	<p>Rock bolts, wire mesh and two shotcrete applications</p>	<p>10 1.0 1.0 5 2 10 <math>(\frac{1A}{2B})</math> <math>(\frac{3H}{40})</math> <math>(\frac{5A}{6B})</math></p>	<p>0.10 1.6 4.1</p>	<p>Category 31 =B 1 m +S(mr) 5 cm</p>
<p>77</p> <p>1. 300 m length, massive gneiss, few joints. Planar, rough-surfaced, unaltered joints. 3 m spacing. Insignificant water inflow. RQD = 100</p> <p>2. Minor overbreak, no falls or slides.</p> <p>3. Wine and liquor storage rooms. Stockholm (ref. Cecil 1970).</p>	<p>20 24.5 18</p>	<p>50 spot bolts in about 300 m of chamber</p>	<p>100 5 1.0 2.5 1.0 1.0 <math>(\frac{1E}{2A})</math> <math>(\frac{3E}{4B})</math> <math>(\frac{5A}{6H})</math></p>	<p>200 1.3 15.4</p>	<p>Category 0,5 =None or sb</p>

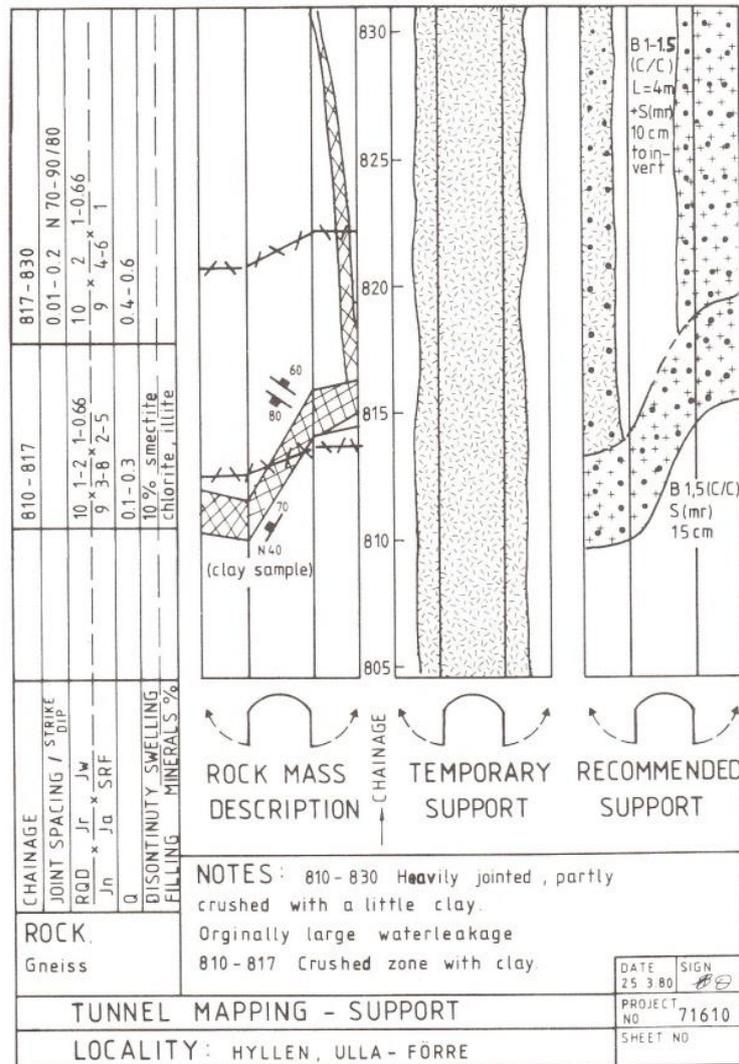
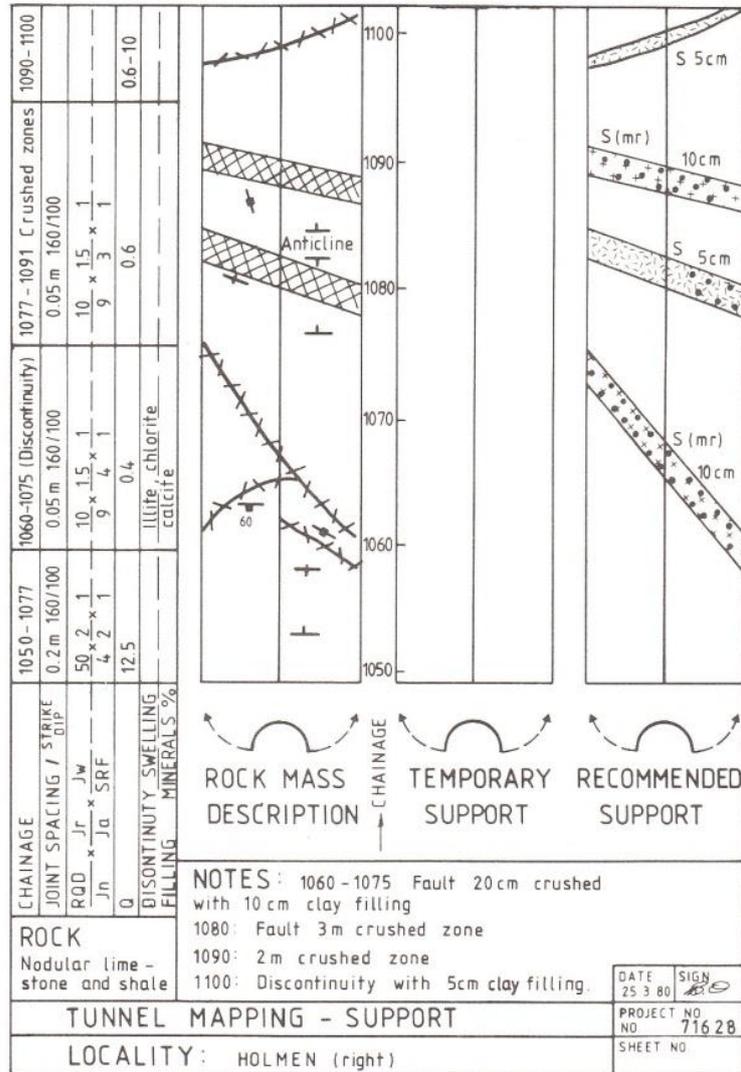


FIG. 3a—Example of tunnel mapping using the Q-system: 160 m<sup>2</sup> headrace tunnel.



- SUPPORT
- B Expansion bolts
  - o--o-- Bolts with bands
  - B Grouted bolts
  - S Shotcrete
  - S(mr) Mesh reinforced shotcrete with grouted bolts
  - Cast concrete arch

FIG. 3b—Example of tunnel mapping using the Q-system: 3.3 m diameter TBM driven sewage tunnel.

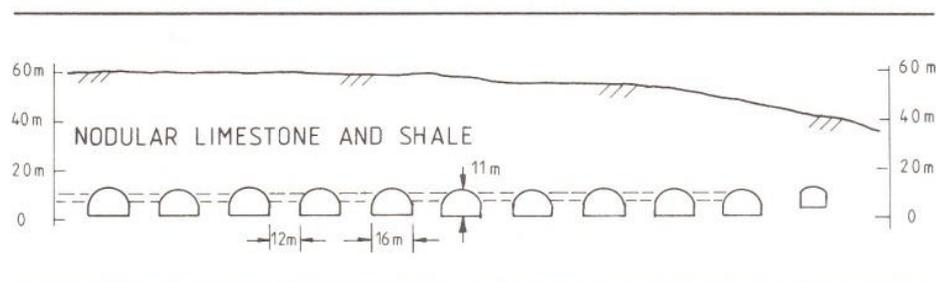


FIG. 4—Vertical section through the VEAS sewage treatment plant, Oslofjord.

General requirements for permanently unsupported openings (i.e., it is preferable that):

1.  $J_n \leq 9$ ,  $J_r \geq 1.0$ ,  $J_a \leq 1.0$ ,  $J_w = 1.0$ ,  $SRF \leq 2.5$  (see Table 2).

Conditional requirements for permanently unsupported openings:

2. If  $RQD \leq 40$ , need  $J_n \leq 2$ .
3. If  $J_n = 9$ , need  $J_r \geq 1.5$  and  $RQD \geq 90$ .
4. If  $J_r = 1.0$ , need  $J_n < 4$ .
5. If  $SRF > 1$ , need  $J_r \geq 1.5$ .
6. If  $SPAN > 10$  m, need  $J_n < 9$ .
7. If  $SPAN > 20$  m, need  $J_n \leq 4$  and  $SRF \leq 1$ .

The shorthand in No. 3 gives the following recommendations:

3. If there are as many as three joint sets ( $J_n = 9$ ), then one needs joint roughness ( $J_r$ ) at least equivalent to rough-planar or smooth-undulating ( $J_r \geq 1.5$ ) and one needs  $RQD \geq 90$  (i.e., "excellent").

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. Our case records describe unsupported man-made excavations having spans from 1.2 to 100 m.

#### Analysis of Q-System Case Records

Ultimately, a potential user of a classification method will be persuaded of the value of a particular system by the degree to which he can identify his site in the case records used to develop the given method. In this section, the characteristics of the 212 case records used to develop the Q-system are analyzed by means of histograms. The potential user can evaluate to what extent his site fits with the data base, or whether he would be relying on few, extreme value cases, which would inherently reduce the reliability of associated support recommendations.

Histograms of the principal parameters used to codify the 212 case records used in the Q-system have been developed. The following brief summary describes the extreme values and the most common values of the various factors affecting tunnel stability in the case records considered.

#### Support Method

The large majority (180) of the 212 case records were supported excavations. Only 32 cases were permanently unsupported. Support ranged from spot bolting (as little as 50 bolts over a roof area of 6000 m<sup>2</sup>) to very heavy rib-and-rock-bolt-reinforced concrete of 2 to 3 m thickness, poured

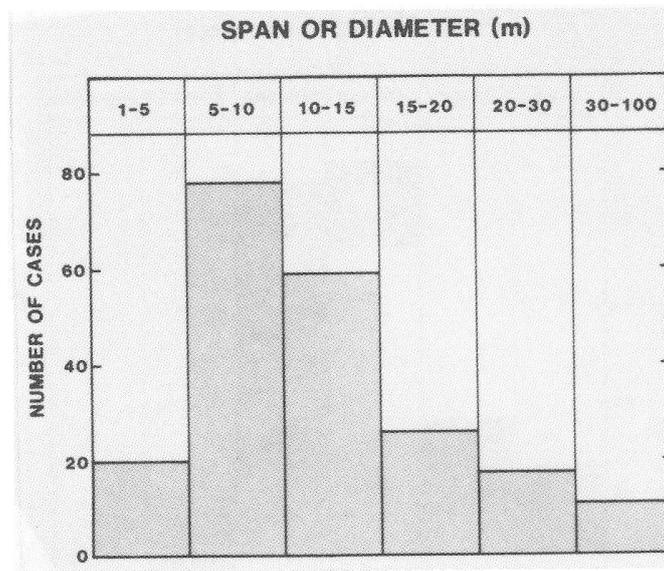


FIG. 5—Histogram of tunnel spans.

in multiple arch and wall drifts. The predominant form of support in the case records was rockbolts, or combinations of rockbolts and shotcrete, often mesh reinforced. Occasionally, extreme conditions called for extreme varieties of support (e.g., 9.8 m long rockbolts on 0.9 m centers together with 14.6 m long bolts on overlapped 0.9 m centers) and mesh-reinforced gunite.

#### Dimensions

The cases studied ranged from unsupported 1.2 m wide pilot tunnels to unsupported 100 m wide mine caverns (Fig. 5). The predominant tunnel dimensions (span or diameter) were 5 to 10 m (78 cases) and 10 to 15 m (59 cases). Excavation heights ranged from extreme values of 1.8 to 100 m. A significant body of the case records came from hydroelectric projects; consequently there were some 40 cases of large caverns with spans in the range of 15 to 30 m and wall heights in the range of 30 to 60 m.

#### Depths

Excavation depths ranged from 5 to 2500 m, though most were commonly in the range of 50 to 250 m. The Scandinavian bias caused predominantly by Cecil's (1970) case records is shown in Fig. 6. Note that the other case records contribute most of the data on the deeper-seated excavations, including numerous hydropower caverns.

#### RQD

The "rock quality designation" (RQD) ranged in a quite uniform manner from 0 up to 100%. Forty (40) cases lay in the "very poor" category (0 to 25%) and fifty-three (53) cases in the "excellent" category (90 to 100%).

#### Number of Joint Sets ( $J_n$ )

The number of joint sets was most commonly in the range of one set (plus random) to three sets (plus random). Fifty-two (52) cases, the largest group, had exactly three joint sets. Extreme cases consisted of massive, unjointed rock and completely crushed, disintegrated rock.

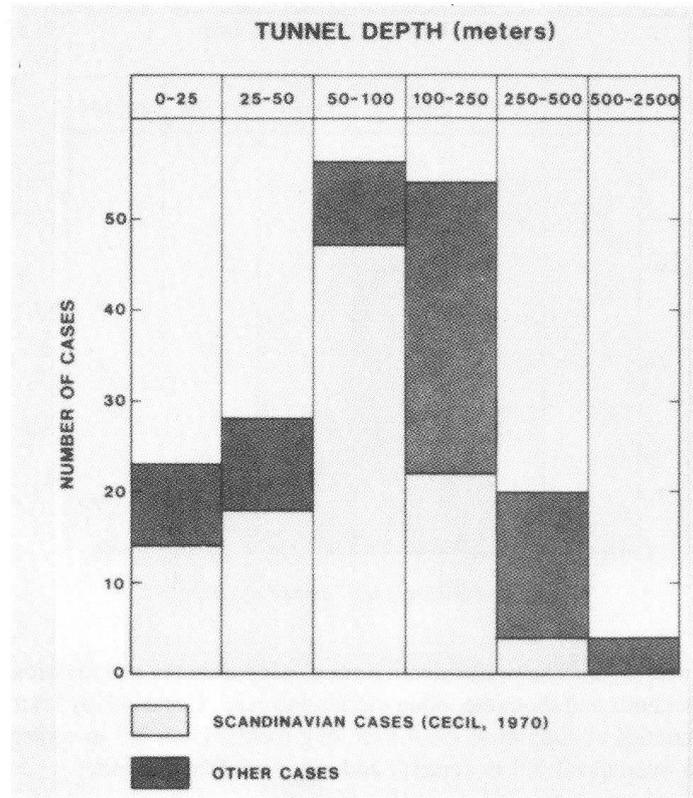


FIG. 6—Histogram of tunnel depths.

#### *Joint Roughness ( $J_r$ )*

The joint roughness numbers most commonly found in the 212 case records were 1.0-1.5-2.0, which represent smooth-planar, rough-planar, and smooth-undulating surfaces, respectively. Extreme values consisted of discontinuous joints in massive rock (16 cases) and plane slickensided surfaces (17 cases) typically seen in faulted rock with clay fillings.

#### *Joint Alteration ( $J_a$ )*

The joint alteration parameter most commonly seen in the case records was represented by the number 1.0 (unaltered or unweathered). One hundred and three (103) cases were in this class. However, more than eighty (80) of the case records involved clay mineral joint fillings of various kinds; these included twelve (12) swelling clay occurrences. Thirteen (13) cases consisted of healed joints, which are obviously very favorable for stability and for their inherently low permeability.

#### *Joint Water ( $J_w$ )*

The joint water reduction factor describing the degree of water inflow was strongly biased in the direction of "dry excavations or minor inflow" (<5 L/min locally). Eighty-one percent (81%) of the cases fell in this category. Twenty-four (24) cases had "medium inflow, occasional outwash of joint fillings" ( $J_w = 0.66$ ). Twelve (12) cases were classed as "large inflow in competent rock," or "considerable outwash of joint fillings," or "exceptionally high inflow decaying with time" ( $J_w = 0.66$  to 0.33).

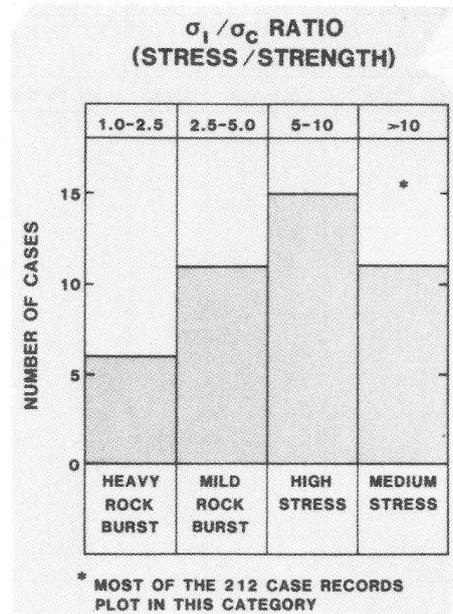


FIG. 7—Histogram of stress/strength ratios.

#### Stress Reduce Factor (SRF)

The stress reduction factor has sixteen (16) classes. These are divided into four broad groups: (a) weakness zones causing loosening or fall-out, (b) rock stress problems in competent rock, (c) squeezing (flow of incompetent rock), and (d) swelling (chemical effect due to water uptake). Seventy-three (73) cases fell in group (a) in which clay fillings were the direct cause of loosening and fall-out. This can be compared with the statistic for  $J_a$  in which eighty-one (81) cases were identified as having “mineral coatings, thin clay fillings,” or “thick clay fillings, swelling clay.” In other words, the majority of cases with these features were classed as “weakness zones causing loosening or fall-out.”

Eighty-one (81) cases were classified as having moderate stress in essentially competent rock; i.e., with  $\sigma_c/\sigma_1$  (unconfined compression strength/major principal stress) in the moderate range of 10 to 200, which is neither too high nor too low.

Thirty-two (32) cases were classified as having rock stress problems (group (b)) with ratios of  $\sigma_c/\sigma_1$  less than 10. This statistic is shown in Fig. 7. The data for this histogram were obtained from the case records in which stress and strength data were specifically described. Eleven (11) of the cases were found to have satisfactory ratios of  $\sigma_c/\sigma_1$  (i.e.,  $> 10$ ).

The mean and extreme values of the  $\sigma_c/\sigma_1$  data plotted in Fig. 7 can be summarized by the following values:

- $\sigma_1$  (range) = 3.5 to 50 MPa
- $\sigma_1$  (mean) = 15.1 MPa
- $\sigma_c/\sigma_1$  (range) = 1.0 to 62
- $\sigma_c$  (range) = 7 to 300 MPa
- $\sigma_c$  (mean) = 112 MPa
- $\sigma_c/\sigma_1$  (mean) = 8.8

(It should be noted that several of the case records with  $\sigma_1$  and  $\sigma_c$  data gave ranges for at least one of these parameters (e.g.,  $\sigma_c$  equal to 120 to 200 MPa). These ranges, and their extreme value ratios, were plotted in Fig. 7. The actual case records describing high stress, slabbing, or rock burst problems numbered only twenty (20). Use of the extreme values expanded this data base to the 32 cases with  $\sigma_c/\sigma_1 \leq 10$  shown in Fig. 7.)

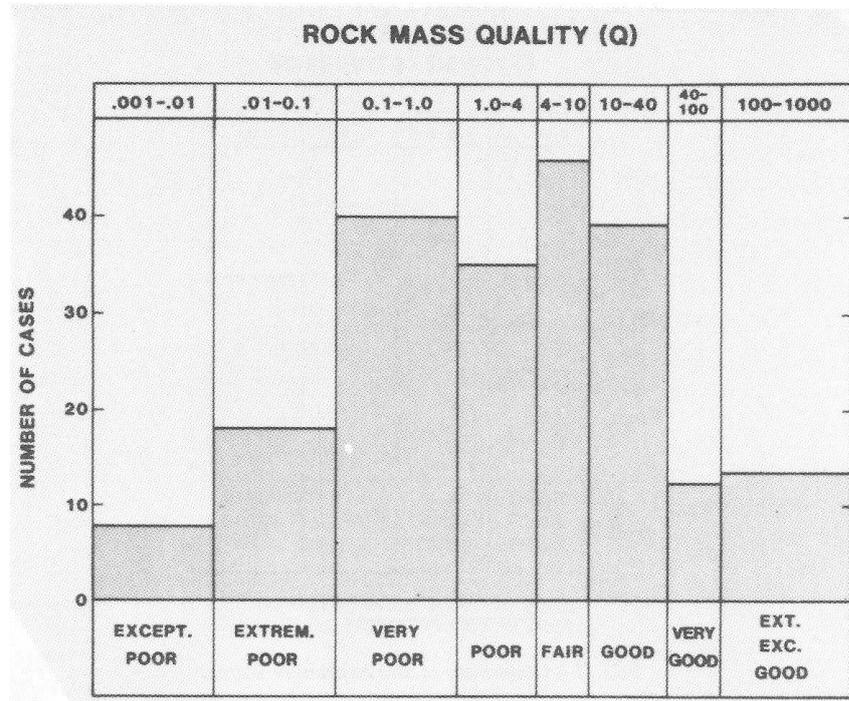


FIG. 8—Histogram of Q-value for all 212 case records.

The predominant rockmass characteristic in the cases with popping, slabbing, or rockbursting was relatively massive rock, with few joint sets and/or wide joint spacing. The mean RQD for these cases was 91% (range 70 to 100%), and the mean  $J_n$  value was 5.0 (two joint sets—two plus random). Jointing ranged from three widely spaced sets ( $J_n = 9$ ) to massive intact rock ( $J_n = 1.0$ ).

Squeezing or swelling problems (groups (c) and (d)) were encountered in only nine of the case records, although a total of twelve cases were listed as rock containing swelling clay such as montmorillonite.

#### Q-Value

The whole spectrum of rock mass qualities exhibited by the case records is shown in Fig. 8. As expected, the majority of cases (76%) fall in the central categories “very poor” ( $Q = 0.1$  to 1.0), “poor” ( $Q = 1.0$  to 4), “fair” ( $Q = 4$  to 10), and “good” ( $Q = 10$  to 40). The whole spectrum of case records utilized in the  $Q$ -system ranges from qualities of 0.001 (extreme squeezing) to 800 (essentially unjointed, massive rock).

#### Rock Types

The distribution of rock types represented in the case records can be summarized as follows: igneous rock (13 types), metamorphic rock (26 types), and sedimentary rock (11 types). Table 6 provides a complete breakdown on the rock types and the number of case record occurrences of each type. The statistics are dominated by granite (48) and gneiss (21). However, there are significant numbers of case records involving schist (21), quartzite (13), leptite (11), and amphibolite (8). Sedimentary rocks are relatively poorly represented with only 19 cases.

TABLE 6—Frequency of occurrence of rock types in examined case records.

I. Igneous		II. Metamorphic		III. Sedimentary	
Basalt	1	Amphibolite	8	Chalk	1
Diabase	4	Anorthosite (meta-)	1	Limestone	3
Diorite	2	Arkose	1	Marly Limestone	1
Granodiorite	1	Arkose (meta-)	3	Mudstone	1
Quartzdiorite	1	Claystone (meta-)	2	Calcareous Mudstone	1
Dolerite	1	Dolomite	1	Sandstone	4
Gabbro	2	Gneiss	14	Shale	2
Granite	46	Biotite Gneiss	1	Clay Shale	2
Aplitic Granite	1	Granitic Gneiss	4	Siltstone	2
Monzonitic Granite	1	Schistose Gneiss	2	Marl	1
Quartz Monzonite	2	Graywacke	1	Opalinus Clay	1
Quartz Porphyry	2	Greenstone	1		
Tuff	2	Schistose meta Graywacke	1		
		Quartz Hornblende	1		
		Leptite	11		
		Marble	1		
		Mylonite	4		
		Pegmatite	2		
		Syenite	1		
		Phyllite	1		
		Quartzite	13		
		Schist	17		
		Biotite Schist	1		
		Mica Schist	2		
		Limestone Schist	1		
		Sparagmite	2		

## Conclusions

1. The large number of case records utilized to develop the  $Q$ -system ensures that reliable support recommendations are provided for a very wide range of tunnel sizes, types of excavation, depths, and rock mass qualities.

2. Detailed analysis of the case records has revealed the overall distribution of individual rock mass parameters such as joint roughness, alteration, and stress-strength ratios, so that extreme value cases can be readily identified.

3. Squeezing ground is the only class of problems that is inadequately represented in the original data base. Swelling, slabbing, and rock bursting problems are represented in a sufficient number of case records for reliable determination of support requirements. General tunneling conditions are extremely well represented, with 160 case records in the range of  $Q$ -values from 0.1 (very poor) to 40 (good).

4. Fifty individual rock types are represented in the case records. Their characteristics are quantified in such a manner that the individuality exhibited by many rock types is carried all the way through to support selection. Application of the  $Q$ -system to other rock types than those described in the case records can be performed with confidence, provided that any special characteristics of the new rock type are adequately represented in the six parameters. A case in point would be the susceptibility to alteration by exposure to moisture. The environment expected under tunnel use must always be carefully considered.

5. The  $Q$ -system has been used for several years in conjunction with Norwegian tunnels supported with fiber-reinforced microsilica shotcrete, a revolutionary new material that is rapidly replacing labor-intensive mesh-reinforced shotcrete. Very poor rock qualities previously requiring

cast concrete arches are being successfully supported by fiber-reinforced shotcrete and rock bolts. Updating of the  $Q$ -system for use in countries with access to this new temporary and permanent support method is underway at NGI.

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