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Engineering Classification of Rock Masses for the Design of Tunnel Support

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With 8 Figures

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Summary — Zusammenfassung — Résumé

Engineering Classification of Rock Masses for the Design of Tunnel Support. An analysis of some 200 tunnel case records has revealed a useful correlation between the amount and type of permanent support and the rock mass quality Q, with respect to tunnel stability. The numerical value of Q ranges from 0.001 (for exceptionally poor quality squeezing-ground) up to 1000 (for exceptionally good quality rock which is practically unjointed). The rock mass quality Q is a function of six parameters, each of which has a rating of importance, which can be estimated from surface mapping and can be updated during subsequent excavation. The six parameters are as follows; the *RQD* index, the number of joint sets, the roughness of the weakest joints, the degree of alteration or filling along the weakest joints, and two further parameters which account for the rock load and water inflow. In combination these parameters represent the rock block-size, the inter-block shear strength, and the active stress. The proposed classification is illustrated by means of field examples and selected case records.

Detailed analysis of the rock mass quality and corresponding support practice has shown that suitable permanent support can be estimated for the whole spectrum of rock qualities. This estimate is based on the rock mass quality Q, the support pressure, and the dimensions and purpose of the excavation. The support pressure appears to be a function of Q, the joint roughness, and the number of joint sets. The latter two determine the dilatency and the degree of freedom of the rock mass.

Detailed recommendations for support measures include various combinations of shotcrete, bolting, and cast concrete arches together with the appropriate bolt spacings and lengths, and the requisite thickness of shotcrete or concrete. The boundary between self supporting tunnels and those requiring some form of permanent support can be determined from the rock mass quality Q.

Key words: Classification, rock mass, joints, shear strength, tunnels, support pressure, shotcrete, bolts.

Technische Klassifikation von Gebirgsqualität zwecks Projektierens von Hohlraumsicherungen im Fels. Eine Untersuchung von Daten aus etwa 200 fertiggestellten Tunnelbauten ergab einen nutzbaren Zusammenhang zwischen Umfang und Typ des permanenten Verbaues und der Gebirgsqualität *Q*. Die numerische Leitziffer erfaßt Werte von 0,001 (äußerst schlechter, langsam rutschender oder quellender Boden) bis auf 1000 für hochwertigen, fast bruchfreien Fels. Die Gebirgsqualität *Q* ist eine Funktion von sechs Parametern, die aus Oberflächenbeobachtungen und nach skalierten Gewichten bestimmte Leitziffern erteilen werden. Die Werte können während des Bauvortriebes justiert werden. Die sechs Parameter sind: *RQD*-Leitziffer, Anzahl der Kluftsysteme, Rauhigkeit (für schwächste oder ungünstigste Spaltebene), Umwandlungsgrad (Charakter der Risse oder Füllung längs der schwächsten Spalten) und des weiteren zwei Parameter, die Spannungsniveau und Wasserzufluß berücksichtigen. Wenn man diese Parameter koordiniert, vertreten sie den Einfluß der Körnung, der Scherfestigkeit an den Anschlußflächen zwischen den Felsblöcken und den einwirkenden Spannungen. Die vorgeschlagene Klassifikation wird mittels Beispielen im Felde und einer Auswahl der Berichte aus fertiggestellten Anlagen erläutert.

Detaillierte Analysen der Gebirgsqualität und der entsprechenden Sicherungsmaßnahmen haben erwiesen, daß es möglich ist, einen angemessenen Ausbau fürs ganze Spektrum der Gebirgsqualität zu veranschlagen. Die Bemessung ist auf die Qualität *Q* des Gebirges, den Ausbaudruck und die Dimensionen und den Zweck des Hohlraumes ausgerichtet. Der Ausbaudruck ist scheinbar eine Funktion von *Q* und von der Rauhigkeit und Anzahl der Spaltsysteme. Die beiden letzteren entscheiden die Dilatanz der Felsmasse und den Freiheitsgrad der Felsblöcke.

Detaillierte Anleitungen für Sicherungsmaßnahmen umfassen verschiedene Kombinationen von Nägeln, Ankern, Spritzbeton und Ortsbetongewölben sowie auch Angaben über Ankerabstände und erforderliche Stärke des Spritz- oder Gußbetons. Die Grenze zwischen selbsttragenden Tunnels und denjenigen, die irgend eine Art permanenten Verbaues benötigen, kann aus der Gebirgsqualität *Q* ermittelt werden.

Classification technique des roches en vue de l'étude des soutènements a prévoir dans les cavités creusées dans la roche. Une analyse de données provenant de quelque 200 cavitées creusées a permis d'établir une relation utile entre, d'une part, l'envergure et le type de soutènements permanents et, d'autre part, la qualité *Q* des masses rocheuses, en ce qui concerne la stabilité. La valeur numérique de *Q* s'étend de 0,001 (roche particulièrement mauvaise, fluante ou gonflant) jusqu'à 1000 pour une roche d'excellente qualité, pratiquement exempte de fissurations. La qualité *Q* de la roche est une fonction de six paramètres dont chacun, dans des échelles données, s'est vu attribuer un coefficient pondéré déterminé qu'on peut estimer en se basant sur des observations faites en travaillant à ciel ouvert et qui pourra être ajusté et mis à jour au cours de l'avancement des travaux. Ces paramètres sont: l'indice *RQD*, le nombre de systèmes de fissuration, la rugosité (celle du plus faible plan de fissuration), le degré d'altération (caractéristiques de ce dont les fissures sont remplies), et, en outre, deux paramètres qui tiennent compte du niveau de tension et de l'afflux d'eau. Dans leur ensemble, ces paramètres représentent l'influence qu'exercent la grandeur des pierres, la résistance au cisaillement existant sur les surfaces de contact entre les pierres, et les tensions actives. La classification suggérée est mise en évidence à l'aide d'exemples tirés de l'expérience acquise sur le terrain ou tirés d'une sélection de rapports concernant des ouvrages exécutés.

Des analyses détaillées de la qualité, accompagnée d'une prise en considération de la pratique de soutènement utilisée, ont permis de démontrer qu'il est possible d'estimer un soutènement approprié pour toute la variété de qualités de roche. Cette estimation est basée sur la qualité *Q* de la roche, sur la pression supportée par le soutènement, sur la taille de la cavité et sur la destination de celle-ci. La pression supportée par le soutènement semble être une fonction de *Q* et de la rugosité et du nombre des systèmes de fissuration. Ces deux derniers paramètres semblent déterminer la dilatance et le degré de liberté (liberté de mouvement) des pierres dans la roche.

Des recommandations détaillées de mesure de sûreté englobent différentes combinaisons de béton projeté, de boulonnage et d'arcs en béton coulés, accompagnées de l'indication de la distance appropriée entre boulons, de la longueur de ces derniers et de l'épaisseur à respecter tant pour le béton projeté que pour le béton coulé. La limite séparant les cavités autoportantes de celles nécessitant un soutènement permanent d'une manière ou d'une autre, peut être déterminée à partir de la qualité *Q* de la roche.

Introduction

„... when you can measure what you are speaking about, and express it in numbers, you know something about it, but when you cannot express it in numbers, your knowledge is of a meagre and unsatisfactory kind ..."

Lord Kelvin (1824—1907)

The Symposium on Large Permanent Underground Openings held in Oslo in 1969 focussed attention on two important gaps in our ability to design the correct support for excavations in rock masses. Denkhaus (1970) pointed out the existence of a missing link between the acquisition of rock mechanics data and the final decisions as to whether an opening should be lined, rock bolted, or kept unlined. Bjerrum (1970) noted that the dilatent property of many rock masses seemed to have been ignored when designing rock bolt systems. He also doubted that the *RQD* index (Deere, 1963) could give a sufficiently complete description of a rock, since two rocks with the same *RQD* index could show entirely different behaviour in a rock cavity.

The last criticism could also be levelled against other widely used rock mechanics parameters, for instance: unconfined compressive strength, shear strength, rock stress, joint frequency, etc. It is essential that such parameters should each to be allowed to contribute in the final decision of tunnel support requirements. The *RQD* index happens to be one of the better single parameters since it is a combined measure of joint frequency and degree of alteration and discontinuity fillings, if these exist. However, it is relatively insensitive to several important properties of rock masses, in particular the friction angle of altered joint fillings (Cording and Deere, 1972), and the roughness or planarity of joint walls.

Despite the known limitations of *RQD*, several attempts have been made to correlate it with the degree of tunnel support, as for instance by

Cecil (1970), Deere et al. (1970), and Merritt (1972). In regularly jointed and clay free rocks these attempts seem to be partly successful. However, a one-parameter description of a rock mass is inevitably limited to a relatively small number of geological environments, if it is to be reliable.

A more general method of numerically classifying rock masses and estimating support has been described by Wickham et al. (1972). This includes a larger number of parameters, each having a numerical scale of importance. Bieniawski (1973) has recently modified this system and combined it with some other proposals for classification. The end result is an eight-parameter description of jointed rock masses, each parameter having five ratings of importance. The proposed parameters were: *RQD*, degree of weathering, intact rock strength, spacing of joints, separation of joints, continuity of joints, ground water inflow, strike and dip orientations. In retrospect it would appear that both, Wickham et al. (1972) and Bieniawski (1973), have almost ignored three important properties of rock masses, namely the roughness of joints, the frictional strength of joint fillings, and the rock load.

The method of classifying rock masses to be described in this paper was developed independently from that described by Wickham et al. (1972) and Bieniawski (1973). However, it is interesting to find that there are several points in common. A special feature of the method is that it was developed through exhaustive analysis of more than two hundred case records. The recommendations for support are therefore detailed, and are also based on estimates of support pressure, which can apparently be quite closely estimated for the whole spectrum of rock mass environments.

Part I

Estimating the Rock Mass Quality

(A) Development of the Classification System

The tunnel case records described by Cecil (1970) provided a comprehensive source of data for the initial development of the method. One of Cecil's figures showed span width plotted against *RQD* for unsupported tunnels. The trend for wider unsupported spans with higher *RQD* values was recognizeable, although the scatter was large. The authors found that this correlation was improved if the relevant *RQD* values were divided by a number representing the number of joint sets measured at each location. As pointed out by Cecil the number of joint sets is an important indication of the degree of freedom of a rock mass.

The modified *RQD* had improved sensitivity to tunnel support requirements, since one important anomaly was removed. For example, a blocky granitic rock mass having three joint sets and an *RQD* of 90 might give equal tunnel stability to a tightly jointed phyllite, having only one joint set, but an *RQD* of only 30.

The importance of dilatancy and shear strength suggested further improvements to the modified *RQD*. Joint roughness (small- and intermediate-scale) was a potentially positive contribution to rock mass quality, while joint alteration and filling materials were potentially negative. Two simple numerical scales of joint roughness and alteration were therefore developed. Finally, numerical scales for rock load and water pressure were added, to further modify the original *RQD* value.

Several months were spent in evaluating case records in the literature, and developing improved numerical scales, until a consistent picture of rock mass quality and tunnel support was obtained. Both the size of excavation (span, diameter or height) and the purpose of the excavation (power house, water tunnel, pilot heading, etc.) were additional important parameters for determining the type and degree of support. However, these two parameters were not included in the estimation of rock mass quality. As suggested by Coates (1964), it is preferable that the estimate of rock mass quality should be independent of both the type and size of excavation if it is to be widely accepted as a classification system.

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

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

$$Q = (RQD/J_n) \cdot (J_r/J_a) \cdot (J_w/SRF) \tag{1}$$

where

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

The rock mass description and ratings for each of the six parameters are given in Tables 1, 2 and 3. The range of possible *Q* values (approx. 0.001 to 1000) encompasses the whole spectrum of rock mass qualities from heavy squeezing-ground right up to sound unjointed rock. (In fact more than 300 000 different geological combinations can theoretically be represented.) The case records examined included 13 igneous rock types, 24 metamorphic rock types, and 9 sedimentary rock types. More than 80 of the case records involved clay mineral joint fillings of various kinds, including 12 swelling clay occurrences. However, most commonly the joints were unfilled and the joint walls were unaltered or only slightly altered. Further details of the range of case records studied can be found in the report by Barton et al. (1974).

Table 1. Descriptions and Ratings for the Parameters RQD, J_n, and J_r

1.	ROCK QUALITY DESIGNATION	(RQD)	
A.	Very poor	0—25	Note:
B.	Poor	25—50	(i) Where RQD is reported or measured as $\leqq 10$ (including 0) a nominal value of 10 is used to evaluate Q in Eq. (1)
C.	Fair	50—75	(ii) RQD intervals of 5, i. e. 100, 95, 90, etc. are sufficiently accurate
D.	Good	75—90	
E.	Excellent	90—100	
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	Note:
H.	Four or more joint sets, random, heavily jointed, "sugar cube", etc.	15	(i) For intersections use ($3.0 \times J_n$)
J.	Crushed rock, earthlike	20	(ii) For portals use ($2.0 \times J_n$)
3.	JOINT ROUGHNESS NUMBER	(J_r)	
	(a) *Rock wall contact* and (b) *Rock wall contact before 10 cms shear*		
A.	Discontinuous joints	4	Note:
B.	Rough or irregular, undulating	3	(i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
C.	Smooth, undulating	2	
D.	Slickensided, undulating	1.5	
E.	Rough or irregular, planar	1.5	
F.	Smooth, planar	1.0	(ii) $J_r = 0.5$ can be used for planar slickensided joints having lineations, provided the lineations are favourably orientated
G.	Slickensided, planar	0.5	
	(c) *No rock wall contact when sheared*		
H.	Zone containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	
J.	Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)	

Table 2. Descriptions and Ratings for the Parameters J_a and J_w

4.	JOINT ALTERATION NUMBER	(J_a)	φ_r (approx.)	
	(a) *Rock wall contact*			
A.	Tightly healed, hard, non-softening, impermeable filling i. e. quartz or epidote	0.75	(—)	Note: (i) Values of $(\varphi)_r$ are intended as an approximate guide to the mineralogical properties of the alteration products, if present
B.	Unaltered joint walls, surface staining only	1.0	$(25^0$—$35^0)$	
C.	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	2.0	$(25^0$—$30^0)$	
D.	Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	$(20^0$—$25^0)$	

Table 2. Continued

E.	Softening or low friction clay mineral coatings, i. e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1—2 mm or less in thickness)	4.0	(8^0—16^0)	
	(b) *Rock wall contact before 10 cms shear*			
F.	Sandy particles, clay-free disintegrated rock etc.	4.0	(25^0—30^0)	
G.	Strongly over-consolidated, non-softening clay mineral fillings (Continuous, <5 mm in thickness)	6.0	(16^0—24^0)	
H.	Medium or low over-consolidation, softening, clay mineral fillings. (Continuous, <5 mm in thickness)	8.0	(12^0—16^0)	
J.	Swelling clay fillings, i. e. montmorillonite (Continuous, <5 mm in thickness). Value of J_a depends on percent of swelling clay-size particles, and access to water etc.	8.0—12.0	(6^0—12^0)	
	(c) *No rock wall contact when sheared*			
K, L, M.	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6.0, 8.0 or 8.0—12.0	(6^0—24^0)	
N.	Zones or bands of silty- or sandy clay, small clay fraction (non-softening)	5.0		
O, P, R.	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	10.0, 13.0 or 13.0—20.0	(6^0—24^0)	
5.	JOINT WATER REDUCTION FACTOR	(J_w)	Approx. water pressure (kg/cm^2)	
A.	Dry excavations or minor inflow, i. e. <5 l/min. locally	1.0	<1	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
B.	Medium inflow or pressure occasional outwash of joint fillings	0.66	1.0— 2.5	
C.	Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5—10.0	
D.	Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5—10.0	
E.	Exceptionally high inflow or water pressure at blasting, decaying with time	0.2—0.1	>10.0	
F.	Exceptionally high inflow or water pressure continuing without noticeable decay	0.1—0.05	>10.0	

Table 3. Descriptions and Ratings for the Parameter *SRF*

		σ_c/σ_1	σ_t/σ_1	
6.	STRESS REDUCTION FACTOR			*(SRF)*
	(a) *Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated*			
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10.0
B.	Single weakness zones containing clay, or chemically disintegrated rock (depth of excavation $\leqq 50$ m)			5.0
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 $\leqq 50$ 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
	(b) *Competent rock, rock stress problems*	σ_c/σ_1	σ_t/σ_1	
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 to wall stability)	10—5	0.66—0.33	0.5—2.0
L.	Mild rock burst (massive rock)	5—2.5	0.33—0.16	5—10
M.	Heavy rock burst (massive rock)	< 2.5	< 0.16	10—20
	(c) *Squeezing rock;* plastic flow of *incompetent rock under the influence of high rock pressures*			
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

Note:

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

(ii) For strongly anisotropic stress field (if measured): when $5 \leqq \sigma_1/\sigma_3 \leqq 10$, 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, σ_t = tensile strength (point load), σ_1 and σ_3 = 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)

Notes on the Use of Tables 1, 2 and 3

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in Tables 1, 2 and 3:

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, 1974),

$$RQD = 115 - 3.3\, J_v \text{ (approx.)} \tag{2}$$

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, schistocity, slatey cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible, or 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 1.

3. The parameters J_r and J_a (representing shear strength) should be relevant to the *weakest significant joint set or clay filled discontinuity* in a given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favourably orientated 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_a) should be used when evaluating Q from Eq. (1).

4. When a rock mass contains clay, the factor *SRF* appropriate to *loosening loads* should be evaluated (Table 3, 6 a). 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 3, 6 b). A strongly anisotropic stress field is unfavourable to stability and is roughly accounted for as in note (ii), Table 3.

5. In general the compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the direction that is unfavourable for stability. This is especially important in the case of strongly anisotropic rocks. In addition, the test samples should be saturated if this condition 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.

When the rock mass quality varies markedly from place to place it will obviously be desirable to map and classify these zones separately. In general the rock mass quality Q will be evaluated separately in two adjacent zones if it is considered that a change in support will be justified in practice. (A four-fold increase or reduction in Q, caused by a change in joint frequency, roughness or degree of alteration etc., will normally qualify for changed support). However, if the variable zones intersect the excavations for only a few metres, it will normally be most economical to map the overall quality, and estimate a compromise value of Q, for eventual design of compromise support. It is normally uneconomic to change support measures over very short tunnel lengths, and in any case the overall stability has to be assured.

However, swelling and softening clay zones may often require individual sealing treatment, even if the affected discontinuities are quite narrow. The type of treatment will depend on the clay content, the access to water, and the quality of the wall rock (Selmer-Olsen, 1970). In some cases the latter may be sufficiently high and the zone sufficiently narrow (i. e. <20 cms for it to be left unsealed. This will also depend on the use to which the tunnel will be put. In general, individual classification and sealing treatment for swelling or softening clay zones should be supplemented with a compromise classification and support, so that the zones between the clay are adequately supported.

Cases sometimes arise where unfavourably dipping shear zones delineate exceptionally large unstable wedges requiring special support. This may take the form of specially dimensioned tensioned anchors positioned to allow for the variously orientated forces. A surge chamber wall at Churchill Falls (Benson et al., 1972) and a power house wall at Morrow Point (Brown et al., 1971) were both stabilized in this manner. In view of the special nature of such problems, no attempt should be made to relate the relevant rock mass quality Q to special-purpose support of this type.

(C) Examples of Rock Mass Quality Q from Surface Exposures

Fig. 1 illustrates the method of classifying rock masses for their quality Q. All the photographs are of surface exposures, but imaginary tunnel depths of about 40 m have been assumed. Therefore, water pressures and rock pressures of medium values have been assumed for each of the eight examples.

Beneath each photograph the following are listed:

1. Rock type.
2. Rock mass quality Q and values of the six parameters: RQD/J_n, J_r/J_a, J_w/SRF.
3. Numerical and alphabetical key to the classification descriptions given in Tables 1, 2 and 3.

The classification of the six samples should be self explanatory. Each numerical value can be checked against the relevant descriptions listed in Tables 1, 2 and 3. The following list of observations may help to clarify some of the special features of the method.

1. The positive contribution of irregular, undulating joints ($J_r = 3$) in example 2, gives this heavily jointed rock mass almost the same quality (Q) as example 1.

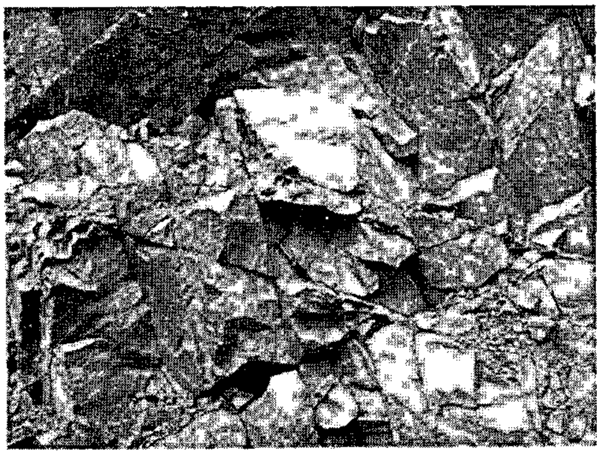
Fig. 1. Six examples of rock mass classified according to their tunnel stability

Sechs Beispiele von Felsmassen, mit Rücksicht auf Tunnel-Stabilität klassifiziert

Six exemples de roches, classifiées selon leur stabilité dans le cas de cavités creusées

1. GRANITE

Q = 90/9 x 1.5/1.0 x 0.66/1.0
= 10 (fair/good)
(1E/2F, 3E/4B, 5B/6J)



2. GRANITE

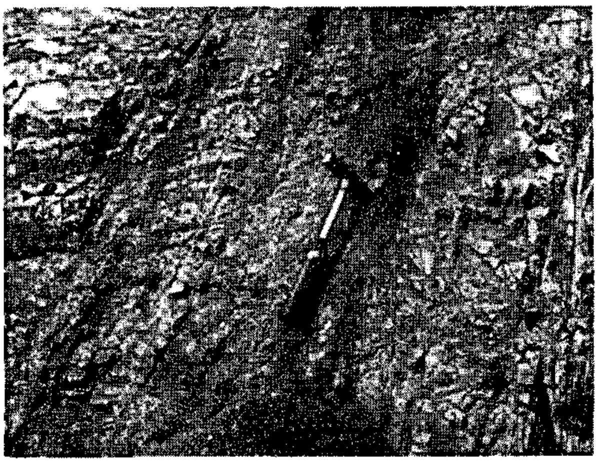
Q = 70/15 x 3.0/1.0 x 0.66/1.0
= 9.2 (fair)
(1C/2H, 3B/4B, 5B/6J)

3. SANDSTONE-CLAYSTONE

Q = 40/9 x 1.0/2.0 x 0.66/1.0
= 1.5 (poor)
(1B/2F, 3F/4C, 5B/6J)

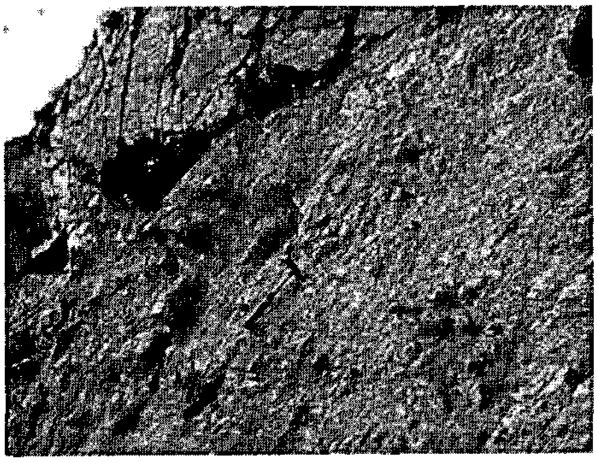
4. NODULAR-LIMESTONE

Q = 80/9 x 1.0/5 x 0.66/5
= 0.24 (very poor)
(1D/2F, 3J/4N, 5B/6G)



5. MUDSTONE (overall RQD = 30)

Q = 30/9 x 1.0/5 x 0.66/5
= 0.09 (extremely poor)
(1B/2F, 3J/4N, 5B/6B)



6. GRANITE (decomposed) RQD = 0

Q = 10/20 x 1.0/6 x 0.66/6
= 0.009 (exceptionally poor)
(1A/2J, 3J/4K, 5B/6N)

2. The relatively widely spaced bedding joints in example 4 would normally produce a higher rock mass quality Q than for example 3. However, the presence of layers of unconsolidated volcanic ash causes the rock mass to be loose and unfavourable for tunnel stability.

3. The weakness zone in example 5 does not contain swelling or softening clay and therefore is not wide enough for individual classification. The values of RQD, J_n, J_w and SRF are relevant to the overall rock mass quality. However, the weakness zone does provide the minimum shear strength parameters J_r/J_a.

4. The decomposed granite shown in example 6 has a very low strength. It is probable that at 40 metres depth, with a rock pressure in the region of 10—15 kg/cm^2, the material will exhibit some mild squeezing, hence the estimate of $SRF = 6$.

(D) Special Features of the Six Classification Parameters

Each of the parameter ratings listed in Tables 1, 2 and 3 are, with the exception of RQD, the end product of successive modifications made during analysis of the available case records. The successive modifications and reanalyses were needed to improve the relation between the rock mass quality Q and the support actually used. The final numerical ratings are therefore more than just arbitrary descriptive scales such as poor (1), fair (2), good (3) etc., and actually give some clue as to the principal properties controlling tunnel stability in rock masses.

1. The first quotient appearing in Eq. (1) (RQD/J_n) represents the overall structure of the rock mass, and it happens to be a crude measure of the relative *block size*, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400. If as an example the quotient is interpreted in units of centimeters, the extreme "particle sizes" of 200 cm and 0.5 cm are seen to be crude but recogniseable approximations. Probably the largest block should be several times this size and the smallest rock fragments less than half the size. (Clay particles are of course excluded.)

2. The second quotient (J_r/J_a) represents the roughness and degree of alteration of the joint walls or filling materials. Quite by chance it was found that the function $\tan^{-1}(J_r/J_a)$ is a fair approximation to the actual *shear strength* that one might expect of the various combinations of wall roughness and alteration products. Table 4 shows values of $\tan^{-1}(J_r/J_a)^0$ tabulated for the three categories of rock wall contact given in Tables 1 and 2. It will be noticed that the "friction angles" are weighted in favour of rough, unaltered joints in direct contact [category (a)]. It is to be expected that such surfaces will be close to peak strength, that they will tend to dilate strongly when sheared, and that they will therefore be especially favourable to tunnel stability.

These high "friction angles" are very similar to the total friction angles (combined cohesion and friction $= \tan^{-1}\tau/\sigma$, where τ = shear strength,

σ = normal stress), measured and predicted for such surfaces (Barton, 1973). Joint spacing or block sizes larger than 3 m will increase these estimates further, thereby allowing for a possible scale effect [see note 3 (i), Table 1].

When rock joints have thin clay mineral coatings and fillings [category (b)], the strength is reduced significantly. Nevertheless, renewed rock wall contact after small shear displacements have occurred may be a very important factor for preserving the excavations from ultimate failure. These effects have been discussed by Barton (1974).

Table 4. Estimate of Apparent "Shear Strength" from the Parameters J_r and J_a

(a) Rock wall contact	J_r	$\tan^{-1}(J_r/J_a)^0$				
		J_a = 0,75	1.0	2	3	4
A. Discontinuous joints	4	79°	76°	63°	53°	45°
B. Rough, undulating	3	76°	72°	56°	45°	37°
C. Smooth, undulating	2	69°	63°	45°	34°	27°
D. Slickensided, undulating	1.5	63°	56°	37°	27°	21°
E. Rough, planar	1.5	63°	56°	37°	27°	21°
F. Smooth, planar	1.0	53°	45°	27°	18°	14°
G. Slickensided, planar	0.5	34°	27°	14°	9.5°	7.1°

(b) Rock wall contact when sheared	J_r	$\tan^{-1}(J_r/J_a)^0$			
		J_a = 4	6	8	12
A. Discontinuous joints	4	45°	34°	27°	18°
B. Rough, undulating	3	37°	27°	21°	14°
C. Smooth, undulating	2	27°	18°	14°	9.5°
D. Slickensided, undulating	1.5	21°	14°	11°	7.1°
E. Rough, planar	1.5	21°	14°	11°	7.1°
F. Smooth, planar	1.0	14°	9.5°	7.1°	4.7°
G. Slickensided, planar	0.5	7°	4.7°	3.6°	2.4°

(c) No rock wall contact when sheared	J_r	$\tan^{-1}(J_r/J_a)^0$		
		J_a = 6	8	12
Disintegrated or crushed rock and clay	1.0	9.5°	7.1°	4.7°
Bands of silty- or sandy-clay	1.0	J_a = 5		
		11°		
Thick continuous bands of clay	1.0	J_a = 10	13	20
		5.7°	4.4°	2.9°

The third category involving no rock wall contact appears extremely unfavourable to tunnel stability. The "friction angles" tabulated are a little below residual strength values for most clays, and are possibly downgraded by the fact that thick clay bands or fillings may tend to consolidate during shear, at least if normally consolidated or if softening and swelling has occurred. The swelling pressure of montmorillonite may also be a factor here.

3. The third quotation (J_w/SRF) consists of two stress parameters. The parameter J_w is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. Water may in addition cause softening and possible outwash in the case of clay filled zoints. The parameter SRF is a measure of: (1) loosening load in the case of excavation through shear zones and clay bearing rock, (2) rock stress in competent rock, (3) squeezing or swelling loads in plastic incompetent rock. It can be regarded as a total stress parameter. It has proved impossible to combine these two parameters in terms of inter-block effective normal stress, because paradoxically a high value of effective normal stress may sometimes signify less stable conditions than a low value, despite the higher shear strength. The quotient (J_w/SRF) is a complicated empirical factor describing the "active stresses".

It appears that the rock mass quality Q can therefore be considered a function of only three parameters which are crude measures of:

1. *block size* (RQD/J_n)
2. *inter-block shear strength* (J_r/J_a)
3. *active stress* (J_w/SRF)

Undoubtedly, there are several other parameters which could be added to improve the accuracy of the classification system. One of these would be joint orientation. Although many case records included the necessary information on structural orientation in relation to excavation axis, it was not found to be the important general parameter that might be expected. The parameters J_n, J_r and J_a appear to play a more important general role than orientation, because the number of joint sets determines the degree of freedom for block movement (if any), and the frictional and dilational characteristics can vary more than the down-dip gravitational component of unfavourably orientated joints. If joint orientation had been included, the classification system would be less general, and its essential simplicity lost.

However, it is recognised that orientation is an important parameter in cases involving major clay-bearing weakness and fault zones. As suggested earlier, it is not recommended that the classification system is extended to cases involving special-purpose support, as would often be required in these cases. Large unstable wedges, both underground and in rock slopes, require specially orientated cable anchor or bolt systems. Special problems will inevitably require special classification systems. The six parameters chosen to define the rock mass quality Q with respect to tunnel stability, will need to be re-evaluated if the problem is one of drillability, boreability, ease of excavation, slope stability etc. It seems very likely that the first four parameters (RQD, J_n, J_r, J_a) can form the basis for many rock mass classification systems. However, the ratings may need to be modified, and other parameters added.

Part II

Estimating the Support Pressure

It is inevitable that all methods of tunnel excavation and support presently in use allow some degree of deformation in the surrounding rock. In most of the poorer qualities of rock mass (squeezing and swelling rock excluded), the final rock load tends to be greater if the initial support is *excessively* soft (i. e. steel ribs and wooden blocking), or if the application of support is delayed. The unchecked deformation may loosen a deeper zone of rock above and around the excavation and the final loads will be greater than they need be. The European approach using an immediate shotcrete and/or rock bolt temporary support system therefore tends to minimise final loads compared to rib and block methods, because it allows a controlled amount of deformation sufficient to develop arching, but insufficient to allow loosening.

(A) Terzaghi's Estimates of Support Pressures

The support pressure criteria developed by Terzaghi (1946) were mostly based on experiences in railway tunnels supported by steel ribs with wooden blocking. For this reason his criteria tend to be over-conservative in the better qualities of rock, if shotcrete and/or bolting is used as immediate support in place of the steel ribs and wooden blocks. However, in the poorest qualities of rock it may be difficult to apply any type of support sufficiently quickly to prevent significant deformation. As a result Terzaghi's criteria appear quite relevant to present day practice when excavating medium-size tunnels in very difficult rock conditions, and are in fact quite widely used.

It is unlikely that a large range of tunnel sizes were involved in Terzaghi's observations of the adequacy of support methods. Spans of between 5 and 10 m probably cover most of the tunnel sizes studied. It is therefore appropriate in the first instance to consider his estimates of support pressure relevant to this approximate size range. In Table 5 the support pressures have been tabulated for each of the nine classes of rock mass loosely defined by Terzaghi.

Although the accuracy of the above estimates of support pressures will vary with the degree of deformation allowed, they do serve as a useful guide as to the possible range that are likely to be encountered in practice. Case records describing design pressures, or better still measured support pressures, can be used to supplement and check these ranges. In each case the support pressures will depend on both the rock mass quality and the type of support method used.

Table 5. Estimates of Roof Support Pressures for Tunnels of 5 m and 10 m Span after Terzaghi (1946)
Assume: span = height, rock density $\gamma = 2.6$ t/m³

Description	Rock load estimates (m)	Support pressures kg/cm² B=H=5 m	B=H=10 m
1. Hard and intact	zero	0	0
2. Hard stratified or schistose	0 to 0.5 B	0 to 0.6	0 to 1.3
3. Massive, moderately jointed	0 to 0.25 B	0 to 0.3	0 to 0.6
4. Moderately blocky and seamy	0.25 B to 0.35 (B+H)	0.3 to 0.9	0.6 to 1.8
5. Very blocky and seamy	(0.35 to 1.10) (B+H)	0.9 to 2.9	1.8 to 2.9
6. Completely crushed but chemically intact	1.10 (B+H)	2.9	5.7
7. Squeezing rock, moderate depth	(1.10 to 2.10) (B+H)	2.9 to 5.5	5.7 to 10.9
8. Squeezing rock, great depth	(2.10 to 4.50) (B+H)	5.5 to 11.7	10.9 to 23.4
9. Swelling rock	up to 80 m any (B+H)	up to 20.8	up to 20.8

As a preliminary effort to relate rock mass quality Q to support pressure, the authors translated Terzaghi's nine rock mass descriptions into

Table 6. Estimates of Rock Mass Quality Q for the Nine Classes of Rock Mass Listed in Table 5

No.	*RQD*	*Jn*	*Jr*	*Ja*	*Jw*	*SRF*	*Q* (range)
1	100	≦2	4	1	1	1	≧200
2	≧30	3	1	1	1	1	20—10
3	100	6	≧1.5	1	1	1	50—25
4	80	9	1	≦3	0.66	1	6—2
5	50	12	1	≧3	0.66	1	1—0.4
6	20	15	1	2	≦0.66	5	0.08 —0.04
7	20	20	1	≧6	0.66	5—10	0.03 —0.01
8	0	20	1	≧6	0.33	10—20	0.004—0.001
9	0	20	1	12	≦0.66	10	0.003—0.001

values of the six classification parameters, as shown in Table 6. There is obviously room for alternative interpretation. However, the resulting ranges of Q were a useful starting point.

(B) Effect of Dimensions

There is a further important factor which should not be overlooked when attempting to estimate the required support pressure for a given excavation using Terzaghi's method. This concerns excavation dimensions. Fig. 2, reproduced from Cording et al. (1972), shows the support pressures actually designed for a number of large caverns excavated during the last two decades or so. These case records are numbered in the figure as below:

1. Cavities I and II (NTS)	15. El Toro
2. Cavity II NTS (stabilized)	16. Norad
3. Cavity II NTS (at failure)	17. Tumut I
4. Poatina	18. Tumut II
5. Poatina (initial)	19. Tuloma
6. Poatina (final)	20. Outardes
7. Churchill Falls	21 Cruachan
8. Hoos	22. Vlanden
9. Harspranget	23. Northfield
10. Sackingen	24. Boundary
11. Hongrin	25. Ronco Val Grande (upper half of wall)
12. Morrow Point	26. Ronco Val Grande (lower half of wall)
13. Woh	
14. Oroville	

There does not appear to be any trend or necessity to increase the support pressure with increasing dimensions of cavern. For the most part, roof support pressures range from approximately 0.5 to 1.5 kg/cm², and wall support pressures from approximately 0.3 to 0.7 kg/cm². In general these pressures are less than half the value they would be if Terzaghi's (1946) design criteria had been rigidly followed.

Fig. 3 is a convenient illustration of this apparent discrepancy between Terzaghi's design criteria and the support capacity currently designed for large rock bolted caverns. Improvements in support methods over the years are undoubtedly part of the reason for the discrepancy. However, it is believed that the widely different dimensions are of equal or more importance.

Terzaghi's (1946) recommendations were based on two types of observations; firstly on model arching experiments in sand which he compared to blocky and seamy rock having "very large grains and little or no cohesion", and secondly on observations of failure of the wooden blocks inserted behind the steel ribs in various railway tunnels in the eastern Alps. It is unlikely that a large range of tunnel sizes was involved in these in situ experiments, and it is unlikely that Terzaghi ever intended his recommendations to be extrapolated to excavations approaching one order of magnitude larger. It seems extremely unlikely that with present-day support methods, doubling the span width would have the effect of doubling the pressure on the supports, as implied by column 2 of Table 5, and by the rock load factors (n) and (m) illustrated in Figs. 2 and 3. Provided the rock around an excavation is held in place in a "closed ring" (using shotcrete if necessary), the required support *pressures* should be more or less independent of moderate increases dimensions, though strongly dependent on unchanged rock

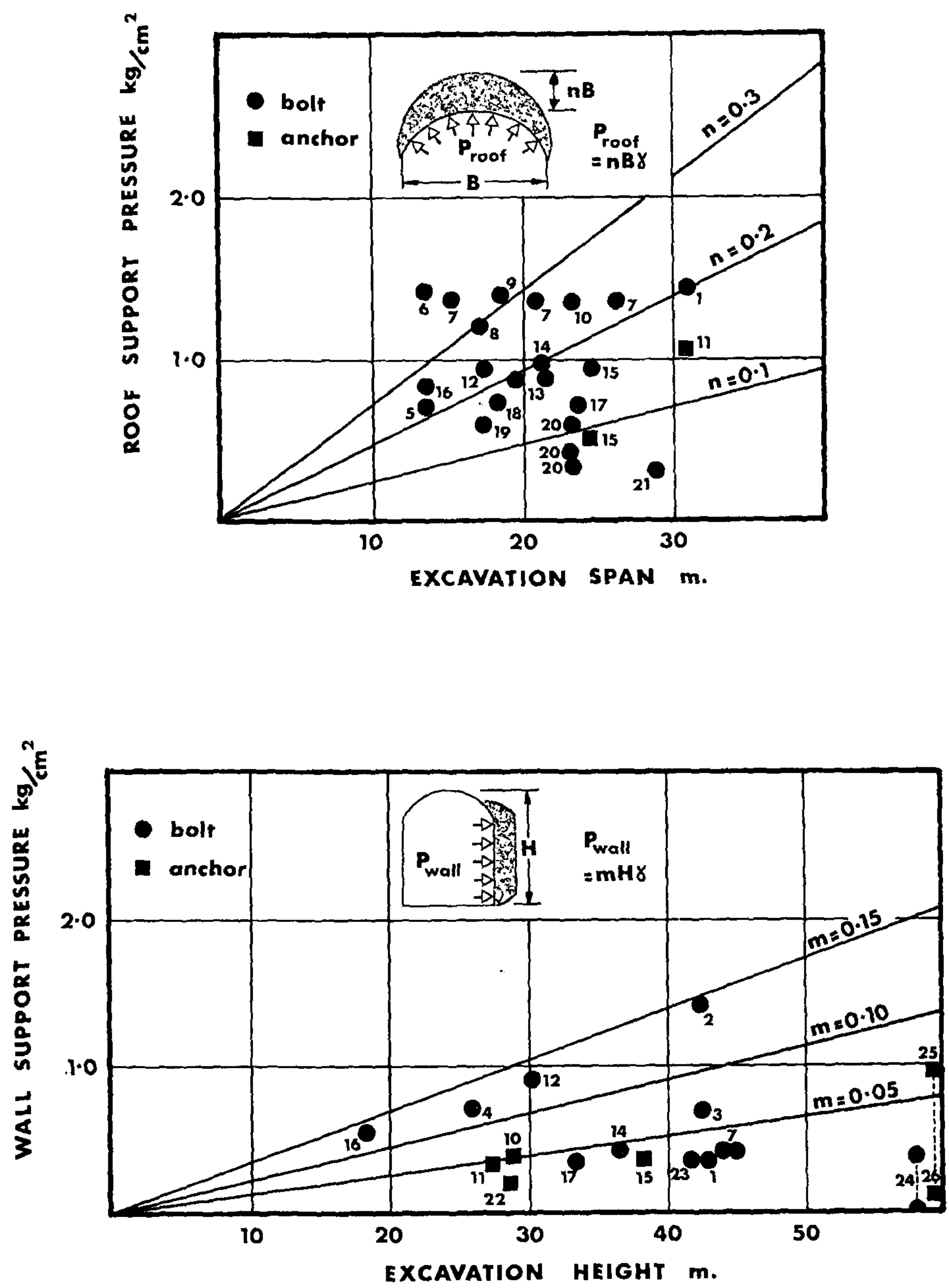


Fig. 2. Design support pressures for the roofs and walls of some large caverns, after Cording et al. (1972). (γ represents the rock density)

Projektierter Ausbaudruck für Gewölbe und Wände in einigen Kavernen, laut u. a. Cording 1972. (γ = Raumgewicht des Felses)

Pression supportée par le soutènement, prévue pour les voûtes et parois de quelques grandes cavités creusées dans la roche, selon Cording et autres, 1972. (γ représente la densité de la roche) mass quality. Obviously the thickness of shotcrete or cast concrete arches needs to be increased when, for a given rock mass quality, the dimensions are increased. However, this does not necessarily imply an increase in support pressure. Bolt spacing, though not bolt length, usually remains unchanged. In fact, the total *load*

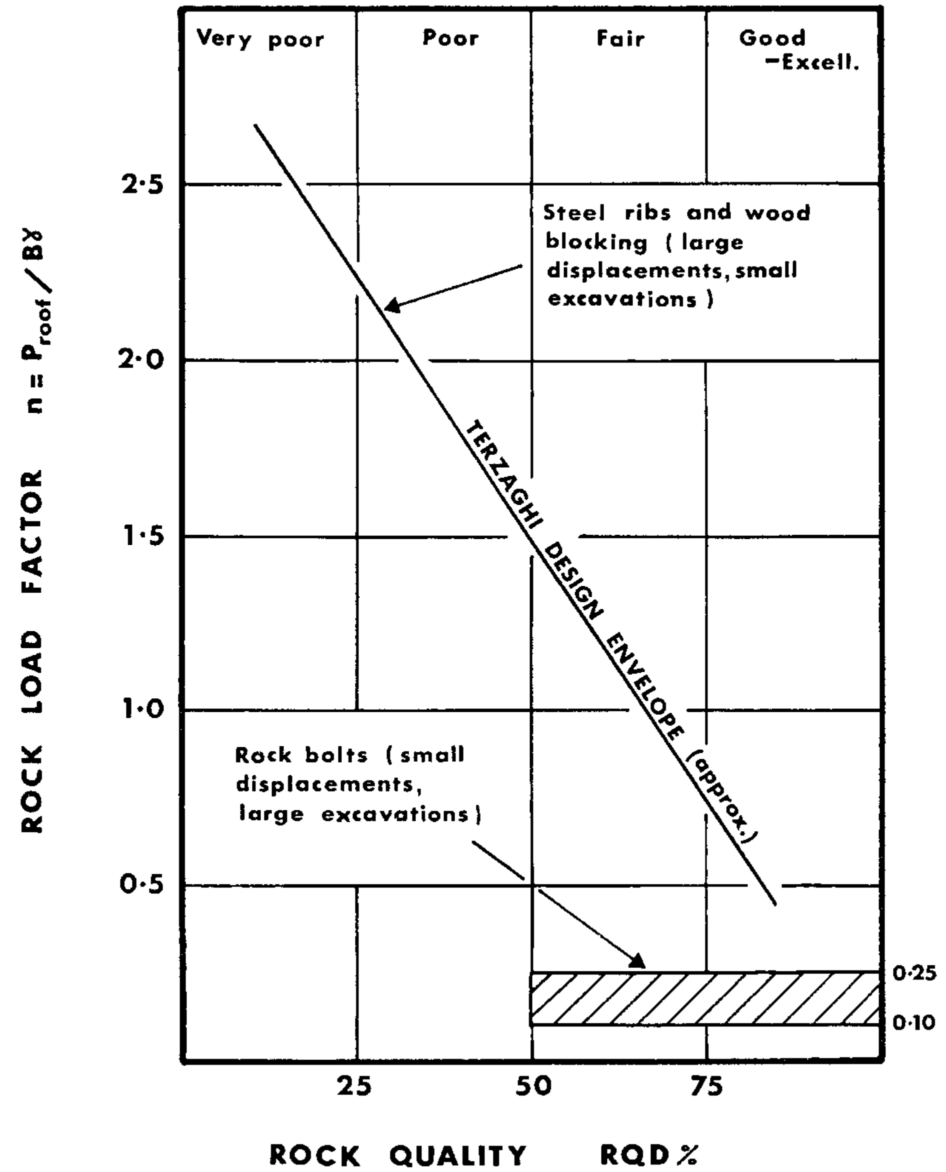
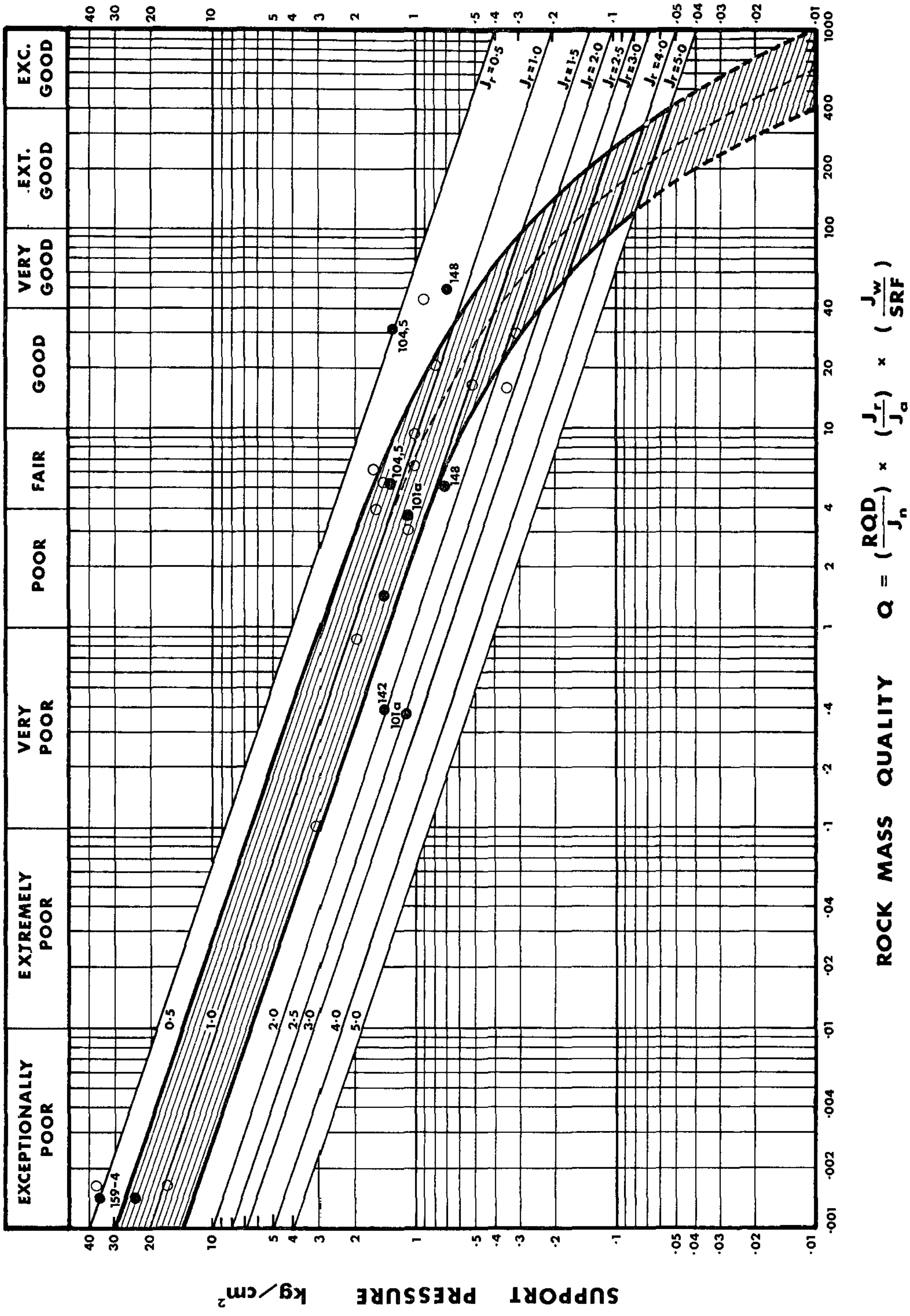


Fig. 3. Comparison of roof support designs for steel rib supported tunnels (large displacements, small excations) and for rock bolted caverns (small displacements, large excavations) after Monsees (1970), and Cording and Deere (1972)

Vergleich zwischen projektiertem Gewölbeausbau mit Stahlbogenstütze (große Verschiebungen, kleine Querschnitte) und Ankerausbau (kleine Verschiebungen, große Querschnitte) laut Monsees 1970, Cording und Deere 1972

Comparaison entre, d'une part, le soutènement de voûtes prévu sous forme d'arcs métalliques (grands déplacements, petites cavités) et, d'autre part, sous forme de boulonnage (petits déplacements, grandes cavités), selon Monsees 1970 et Cording et Deere 1972

capacity of the support system is increased, but not the *pressure*. It will therefore be assumed that under most conditions found in practice, excavation dimensions can be largely ignored where support pressures are concerned. This appears to be in line with present-day practice.



EXCEPTIONALLY POOR
EXTREMELY POOR
VERY POOR
POOR
FAIR
GOOD
VERY GOOD
EXT. GOOD
EXC. GOOD
$J_r = 0.5$
$J_r = 1.0$
$J_r = 1.5$
$J_r = 2.0$
$J_r = 2.5$
$J_r = 3.0$
$J_r = 4.0$
$J_r = 5.0$
SUPPORT PRESSURE kg/cm²
ROCK MASS QUALITY
$Q = \left(\frac{RQD}{J_n}\right) \times \left(\frac{J_r}{J_a}\right) \times \left(\frac{J_w}{SRF}\right)$

(C) Relationship between Support Pressure and Rock Mass Quality Q

(i) Roof Support Pressure

An empirical equation relating permanent support pressure and rock mass quality Q, which fits available case records quite well, was found to be the following:

$$P_{\text{roof}} = \left(\frac{2.0}{J_r}\right) Q^{-1/3} \tag{3}$$

where

P_{roof} = permanent roof support pressure in kg/cm²
J_r = joint roughness number
Q = rock mass quality

The diagonal lines drawn in Fig. 4 and numbered with their respective J_r values were plotted directly from this equation. The shaded envelope is the authors' estimate of the range to be expected in practice according to available case records. The double dependence of support pressure on joint roughness number J_r was deliberate and it appears to be realistic according to available case records. The poorer qualities of rock mass are dominated by more or less non-dilatent clay filled joints ($J_r = 1.0$ nominal), while the better qualities tend to receive their improved Q values from the dilatent property of interlocked non-planar rock joints. Accordingly, the shaded envelope curves downwards, and for the very best qualities, drops below $J_r = 5$, which signifies discontinuous joints having a spacing of more than 3 metres.

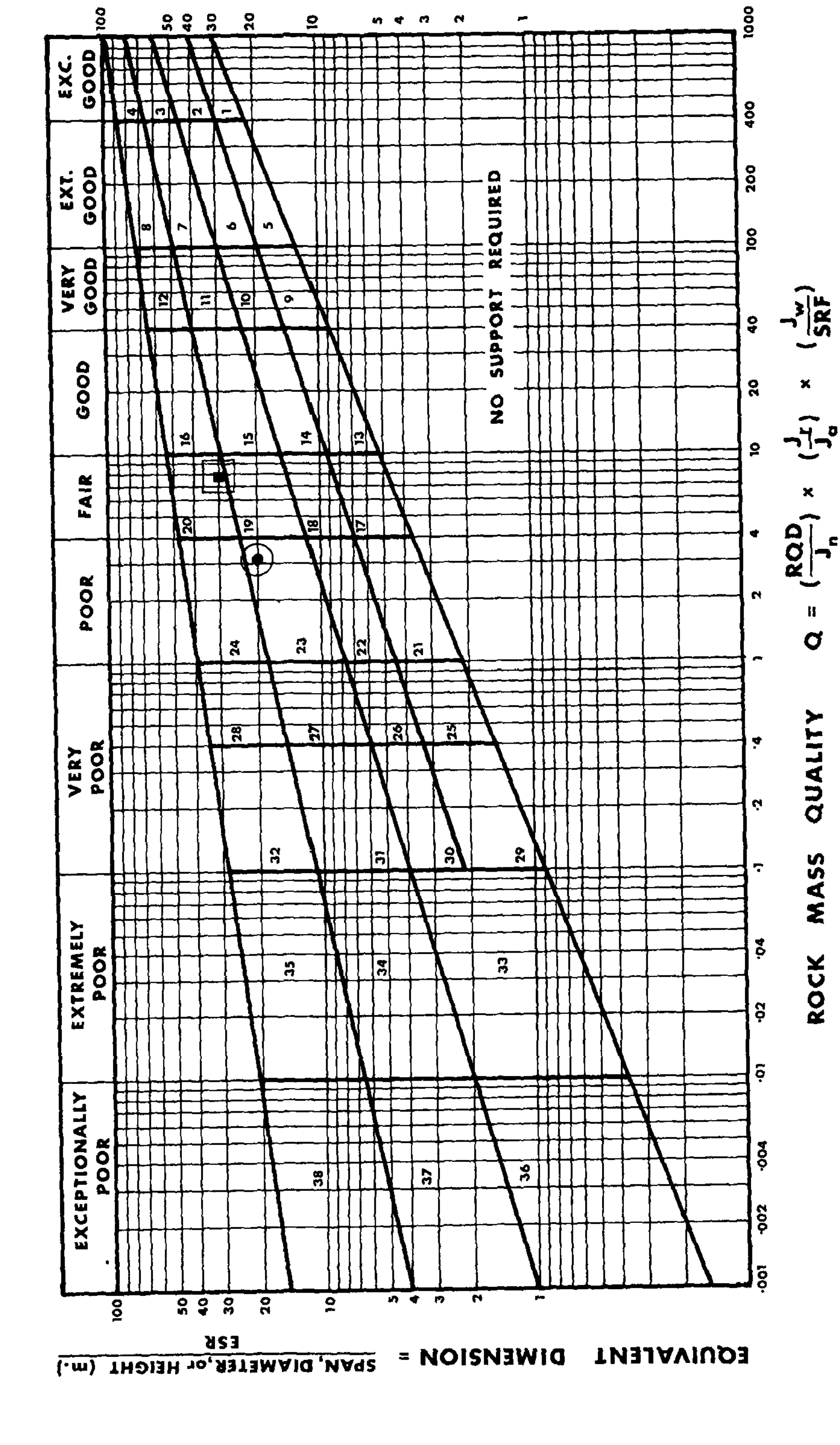
It is not possible to introduce more variables in the chart shown in Fig. 4. Nevertheless, Eq. (3) can be improved if the number of joint sets (joint structure number J_n) is also included separately, besides its contribution to Q. When the number of joint sets falls below three, the degree of freedom for block movement is greatly reduced since three joint sets (or two plus random) is the limiting case for three-dimensional rock blocks. The equation below is offered as an improved version of Eq. (3).

$$P_{\text{roof}} = \frac{2\, J_n^{1/2}\, (Q)^{-1/3}}{3\, J_r} \tag{4}$$

Fig. 4. Empirical method for estimating the support pressure. Plotted points refer to case records describing measured or designed roof support pressures. Case records for each of the numbered points have been described by Barton et al. (1974)

Empirische Methode zur Berechnung des Ausbau-Druckes. Die eingetragenen Punkte beziehen sich auf beschriebene Fälle, wo gemessene oder projektierte Ausbaudrucke angezeigt sind. Die Daten der einzelnen, numerierten Punkte wurden u. a. von Barton (1974) gesammelt

Méthode empirique pour le calcul de la pression supportée par le soutènement. Les points appliqués se réfèrent à des cas décrits, où lesdites pressions, mesurées ou projetées, ont été indiquées. Les données pour chacun des points numérotés ont été réunies par Barton et autres 1974


EXCEPTIONALLY POOR
EXTREMELY POOR
VERY POOR
POOR
FAIR
GOOD
VERY GOOD
EXT. GOOD
EXC. GOOD
NO SUPPORT REQUIRED
EQUIVALENT DIMENSION = SPAN, DIAMETER, or HEIGHT (m.) / ESR
ROCK MASS QUALITY $Q = (\frac{RQD}{J_n}) \times (\frac{J_r}{J_a}) \times (\frac{J_w}{SRF})$
·001 ·002 ·004 ·01 ·02 ·04 ·1 ·2 ·4 1 2 4 10 20 40 100 200 400 1000
100 50 40 30 20 10 5 4 3 2 1

It should be noted that Eqs. (3) and (4) will give an identical estimate of roof support pressure when the rock mass is intersected by three joint sets ($J_n = 9$). Eq. (4) will give a lower estimate of support pressure than Eq. (3) when there are less than three joint sets (no three-dimensional blocks), and a higher estimate when there are more than three joint sets. This would seem to be a realistic improvement, since it provides estimates of support pressure largely in agreement with actual practice, and generally within the shaded envelope in Fig. 4.

(ii) Wall Support Pressure

Several large excavations have been supported with almost equal pressures on the walls and roof arch (see for instance Barth, 1972, concerning Waldeck II). In other cases the wall pressure may be less than one third that used in the roof arch. In the Churchill Falls power cavern (Benson et al., 1972), the support pressures applied were approximately 1.3 and 0.4 kg/cm^2 in the roof and walls respectively, despite the presence of unfavourably dipping foliation joints in the 45 m high walls. The trend towards higher roof support pressure is shown unmistakeably in Fig. 2.

In view of the more favourable position of excavation walls as opposed to roofs, it seems appropriate to consider a hypothetically increased "wall quality" which will be some function of the general rock mass quality Q for a given excavation. Analysis of case records to compare the permanent roof and wall support pressures used in a given quality of rock mass provide the necessary guidelines.

It is recommended that a hypothetical "wall quality" equal to $5\,Q$ be regarded as the maximum for use in the better qualities of rock mass when $Q > 10$. (This may lead to a recommendation of zero support for the walls of small excavations as shown later.) In intermediate qualities, i. e. $0.1 < Q < 10$, in which the wall pressure is of more consequence, a value of $2.5\,Q$ could be used. In the worst qualities, i. e. $Q < 0.1$, where the wall pressure (and floor pressure) can be almost equal to the vertical pressure, a minimum value $1.0\,Q$ should probably be used. In exceptional cases of invert swelling due to water uptake, the floor and lower walls might require

Fig. 5. Tunnel support chart showing the box numbering for 38 categories of support. The two plotted points refer to the worked example given on page 230

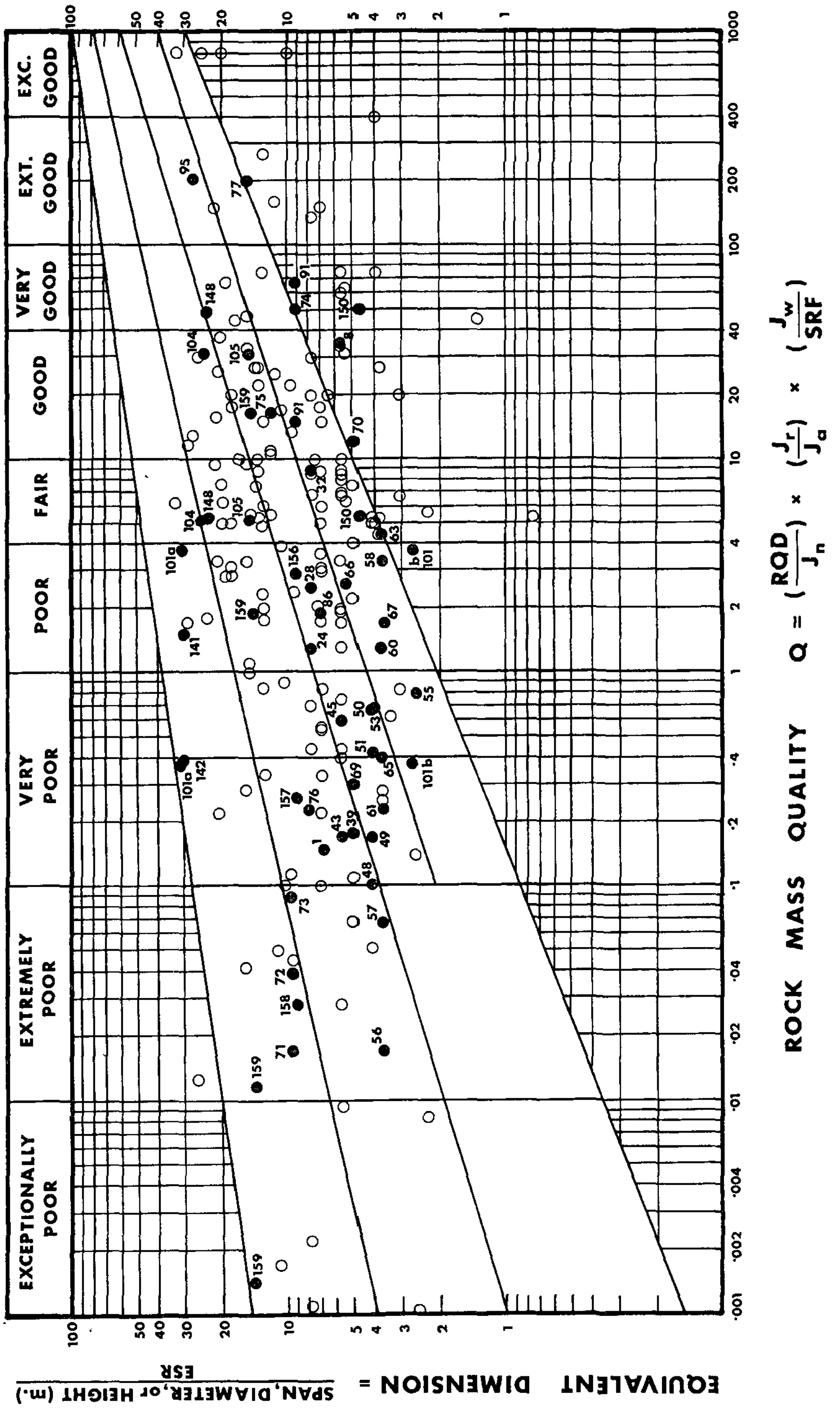
● = roof ■ = wall

Diagramm, welches die 38 Ausbaukategorien veranschaulicht. Die beiden markierten Punkte beziehen sich auf das Arbeitsbeispiel, Seite 230

● = Gewölbe ■ = Wand

Graphique montrant 38 catégories de soutènements. Les deux points margués se réfèrent à l'exemple de travail reproduit à la page 230

● = voûte ■ = paroi


EXCEPTIONALLY POOR
EXTREMELY POOR
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ROCK MASS QUALITY Q = (RQD/J_n) × (J_r/J_a) × (J_w/SRF)
EQUIVALENT DIMENSION = SPAN, DIAMETER, or HEIGHT (m.) / ESR
·001
·002
·004
·01
·02
·04
·1
·2
·4
1
2
4
10
20
40
100
200
400
1000

a lower value of Q than used for the roof. These modified "wall qualities" can be substituted directly in Eqs. (3) and (4), or read directly into the support pressure chart shown in Fig. 4.

Part III
Design of Support Based on Case Records

(A) Tunnel Support Chart for Analysis of Case Records

The method of classifying a rock mass for its quality Q was developed by successive re-analysis of case records, until a consistent relationship was obtained between Q, the excavation dimension, and the support actually used. These three variables were inter-related by means of a support chart. The final version of this chart is given in Fig. 5. It was arrived at after several alterations and re-analyses of the case records. The box numbering 1 to 38 is used as a reference to the *support category*. Support measures that are appropriate to each category are tabulated later.

Table 7. The Excavation Support Ratio (*ESR*) Appropriate to a Variety of Underground Excavations

	Type of excavation	*ESR*	No. of cases
A.	Temporary mine openings etc.	ca. 3—5?	(2)
B.	Vertical shafts: (i) circular section	ca. 2.5?	(0)
	(ii) rectangular/square section	ca. 2.0?	(0)
C.	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks), pilot tunnels, drifts and headings for large excavations etc.	1.6	(83)
D.	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels, etc. (cylindrical caverns?)	1.3	(25)
E.	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections etc.	1.0	(79)
F.	Underground nuclear power stations, railway stations, sports and public facilities, factories etc.	ca. 0.8?	(2)

The left-hand axis of the support chart gives the equivalent dimension (D_e), which is a function of the size and purpose of the excavation. The span or diameter are used as dimensions when analysing roof support, and the diameter or height for wall support. The excavation support ratio (*ESR*)

Fig. 6. Support recommendations are based on the analyses of more than 200 case records. Numbered case records are described by Barton et al. (1974)

Die Ausbauanleitungen sind auf Untersuchung von über 200 beschriebenen Anlagen basiert. Numerierte Fälle sind u. a. von Barton (1974) beschrieben

Les recommandations pour le soutènement se basent sur un examen de plus de 200 ouvrages décrits. Les cas numérotés ont été décrits par Barton et autres (1974) which modifies these dimensions, reflects construction practice in that the degree of safety and support demanded by an excavation is determined by the purpose of the excavation, the presence of machinery, personell etc.

The list of *ESR* values given in Table 7 was developed through trial and error as the most workable solution to the problem of variable support practice. The number of case records relevant to each class of construction are given in brackets. The degree of confidence in these figures will be roughly in proportion to the number of relevant case records, hence the question marks.

(B) Examples of Case Record Analysis

More than two hundred case records were evaluated, and the relevant values of *Q* and *SPAN/ESR* are plotted in Fig. 6. All the numbered points refer to case records that are described in detail by Barton et al. (1974). In all, more than ninety of the case records were obtained from Cecil (1970), who visited and mapped a wide variety of tunnel conditions in Scandinavia. In view of their importance to the development of the classification system, a selection have been reproduced in Tables 8 and 9, with relevant sketches in Figs. 7 and 8. The case record numbering used by Cecil is unchanged.

The twelve case records have been chosen to illustrate a variety of rock mass environments. The six-parameter classification (Tables 1, 2 and 3) should be checked to verify the various ratings used. The values of rock mass quality *Q* and *SPAN/ESR* are plotted in the tunnel support chart (Fig. 6), and the relevant support category can be found from Fig. 5 (box numbers 1 to 38 represent support category numbers).

The extreme right-hand columns of Tables 8 and 9 are termed "roof support recommendation", and apart from category number, contain in abbreviated form a description of the recommended roof support for the given tunnel. This is based on the support used in all those case records that plot within the same support category. A complete list of the recommended support for each category is given in a later section (Tables 11, 12, 13 and 14).

In order to make support recommendations consistent and continuous between categories, some simple design concepts were used to rationalize the bolt spacings and shotcrete or concrete arch thickness for each category. This compromise solution was tailored to fit those case records giving detailed dimensions of bolt patterns and shotcrete or concrete linings. It also supplied a reasonable estimate of support dimensions for case records where the support was referred to in vague terms, i. e. "systematic bolting and shotcrete". The simple design concepts for rationalizing the support recommendations are given in an appendix at the end of the paper.

(C) Self-Supporting Tunnels

The lower diagonal line of the tunnel support chart (Figs. 5 and 6) was found to be the approximate boundary between self-supporting excavations,

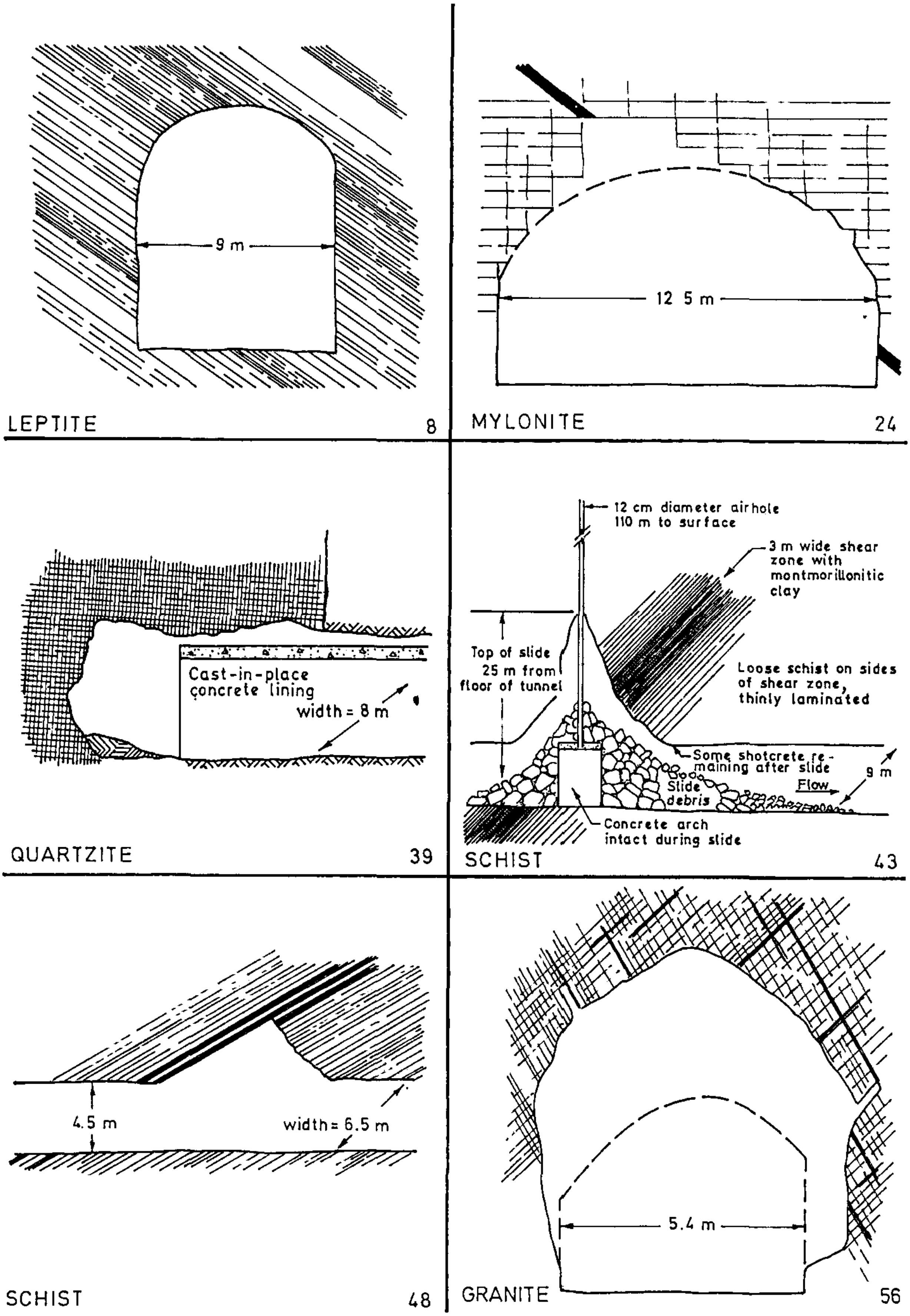

Fig. 7. Sketches of the six case records described in Table 8, after Cecil (1970)
Skizzen der sechs Fälle, welche nach Cecil (1970) in Tabelle 8 beschrieben sind
Croquis représentant les six cas décrits dans le tableau 8, selon Cecil (1970)

Table 8. Classification and Prediction of Support for Six of the Case Records Described by Cecil (1970)

Case No.	1. DESCRIPTION OF ROCK MASS 2. Nature of instability 3. Purpose of excavation, location, reference	*SPAN* m	Height m	Depth m	Support used	*RQD*/*Jn*	*Jr*/*Ja*	*Jw*/*SRF*	*Q*	*ESR*	*SPAN*/*ESR* m	Roof Support Recommendation
8	1. 50 m length of closely spaced, tight diagonal joints in leptite. Planar, smooth joints. 1 joint set, 5—30 cm, spacing. No water present. 2. Minor overbreak when blasting. 3. Tailrace tunnel, Seitevare Hydro. N. Sweden (ref. Cecil 1970).	9	9	140	None	70/2	1.0/1.0	1.0/1.0	35	1.6	5.6	Category 0
24	1. 60 m length, including a 1 m wide shear zone in mylonite. Crushed mylonite and non-softening clay seams and joint fillings. Intersecting joint set. 2 joint sets plus random, 5—30 cm spacing. Minor water inflows (<31/min). 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	1.3	1.6	7.8	Category 22 =B 1 m +S (mr) 2.5—5 cm
39	1. 50 m length, shear zone in quartzite, "sugar cube" rock structure. Planar, smooth, unaltered joints. 3 joint sets, <5 cm, spacing. 5—10 l/min water inflow. 2. Major roof falls, progressive formation of dome- and vault-shaped crown. Also falls from the face. 3. Headrace tunnel, Rendal Hydro, Norway (ref. Cecil 1970).	8	6	200	Cast concrete arch, immediately after mucking out	20/15	1.0/1.0	0.66/5	0.18	1.6	5.0	Category 31 =CCA 20—30cm +B 1 m
43	1. 25 m length, 3 m wide shear zone in thinly laminated schist, swelling montmorillonitic clay seam in shear zone, some chlorite joint coatings. Planar slickensided joint walls. 1 joint set, 5—30 cm spacing. Ground water seepage along cased de-air hole may have contributed to swelling process.	9	8	110	Original 6—8 cm shotcrete failed. Permanent support after failure with cast concrete arches	20/2	0.5/12	1.0/2.5				Category 31 =CCA (sr) 30 cm +B 1 m

	2. Complete collapse of tunnel during operation of power plant. Vault-shaped crown opening. 3. Tailrace tunnel, Sällsjö Hydro, N. Sweden (ref. Cecil 1970).								0.17	1.6	5.6	
48	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. 2. Wedge-shaped roof fall. 3. Tailrace tunnel, Bergvattnet Hydro, N. Sweden (ref. Cecil 1970)	6.5	4.5	50	Rock bolts, wire mesh and two shotcrete applications	10 2	1.0 10	1.0 5	0.10	1.6	4.1	Category 31 =B 1 m +S (mr) 5 cm
56	1. 20 m length, 10 m wide vertical shear zone in granite. Rock crushed and frequently altered to earthy-gravel. Some remnant joint surfaces coated with clay (non-softening). Rock adjacent to zone blocky and loose. Irregular slickensided joint surfaces, 5—30 cm spacing. Large water inflows after blasting carried fault zone debris into tunnel, left open voids up to 1 m wide. *Note:* Tunnel located within 10 km of a major overthrust sheet, locally vertical and low angle shear zones occur. 2. Progressive roof fall-out to form a large vault-shaped opening. 3. Headrace tunnel, Stensjöfallet Hydro, N. Sweden (ref. Cecil 1970).	5.9	4.3	100	No support immediately after blasting. Eventually two shotcrete applications	10 20	1.5 6	0.33 2.5	0.017	1.6	3.7	Category 34 =S (mr) 7.5 cm

Note: Right-hand column "Roof Support Recommendation" is obtained from Tables 11, 12, 13, and 14

Key: *S* = shotcrete, B = systematic bolting, sb = spot bolting, CCA = cast concrete arches, mr = mesh reinforced, sr = steel reinforced, clm = *chain link mesh.*

Bolt spacing is given in metres. — Shotcrete or concrete thickness is given in centimeters.

Table 9. Classification and Prediction of Support for Six of the Case Records Described by Cecil (1970)

Case No.	1. DESCRIPTION OF ROCK MASS 2. Nature of instability 3. Purpose of excavation, location, reference	SPAN m	Height m	Depth m	Support used	$\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{J_w}{SRF}$	Q	ESR	SPAN/ESR m	Roof support recommendation
60	1. 20 m length, 1 m wide zone of sheared granite with clay seams (non-softening) slide boundary is a thin (< 1 cm) clay seam and thinly sheared material that lie in contact with massive rock. Planar, slickensided joints. 1 joint set, 5—30 cm spacing. Insignificant inflow of water. See note, case 56. 2. Wedge-shaped roof fall. 3. Headrace tunnel. Stensjöfallet Hydro. N. Sweden (ref. Cecil 1970).	5.9	4.3	85	Rock bolts, and shotcrete	$\frac{80}{2}$	$\frac{0.5}{6}$	$\frac{1.0}{2.5}$	1.3	1.6	3.7	Category 21 = B 1 m + S 2.5 cm
66	1. 80 m length, open horizontal sheeting joints in granite, partially filled with sand sized material. Planar, rough surfaced joints. 2 joint sets, 5—30 cm spacing. Insignificant water inflow. See note, case 56. 2. Overbreak above springline. 3. Access tunnel, Stensjöfallet Hydro. N. Sweden (ref. Cecil 1970).	7	4.5	15—20	Rock bolts and shotcrete	$\frac{70}{4}$	$\frac{1.5}{2}$	$\frac{1.0}{5}$	2.6	1.3	5.4	Category 21 = B 1 m + S 2.5 cm
67	1. 50 m length, close vertical jointing cutting across schistose rock structure in schistose metagreywacke. Sandy, gravelly joint fillings. Planar smooth surface joints. 1 joint set plus random (for schistocity planes), 5—30 cm spacing. Water inflows up 1000 l/min.	5.9	4.8	100	Shotcrete	$\frac{20}{3}$	$\frac{1.0}{2}$	$\frac{0.2}{1.0}$				Category 21 = S 2.5 cm

	2. Large overbreak in intrados, some roof falls.											
	3. Railrace tunnel, Stensjöfallet Hydro. N. Sweden (ref. Cecil 1970).								1.7	1.6	3.7	
70	1. 10 m length, strongly sheared granite, very tight vertical structure. Planar, rough surfaced, unaltered joints. 1 joint set, 5—30 cm spacing. Insignificant water inflow.	8	5.7	15	None	40 2	1.5 1.0	1.0 2.5				Category 0 Note: Very tight structure may imply higher stress,
	2. Stable, minor overbreak, no roof falls.											i. e. *SRF* = 1.0 Hence $Q = 30$
	3. Collector tunnel, Mo i Rana Hydro. N. Norway (ref. Cecil 1970).								12	1.6	5.0	
74 75	1. Approx. 2 km length, massive granite, widely spaced, tight, vertical joints. Planar, smooth-surfaced unaltered joints. 1 joint set, 1—3 m *spacing. Insignificant water inflow.*	12	12.5	≦100	None in chambers	100 2	1.0 1.0	1.0 1.0	50	1.3	9.2	Category 0,9 = NONE or *sb*
	2. No overbreak in chambers, but overbreak at intersections.											Category 14 = B 1.5—2 m + clm
	3. Waste water treatment plant, Käppala. Sweden (ref. Cecil 1970).				Bolts at intersections	100 2 × 3	1.0 1.0	1.0 1.0	16.7	1.0	12.0	
77	1. 300 m length, massive gneiss, few joints. Planar, rough-surfaced, unaltered joints. > 3 m spacing. Insignificant water inflow.				50 spot bolts in about 300 m of chamber	100	5	1.0				Category 0,5 = None or sb
	2. Minor overbreak, no falls or slides.	20	24.5	18		1.0	1.0	2.5				
	3. Wine and liquor storage rooms. Stockholm (ref. Cecil 1970).								200	1.3	15.4	

Note: Right-hand column "Roof Support Recommendation" is obtained from Tables 11, 12, 13, and 14.

Key: S = shotcrete, B = systematic bolting, sb = spot bolting, CCA = cast concrete arches, mr = mesh reinforced, sr = steel reinforced, clm = chain link mesh.

Bolt spacing is given in metres. — Shotcrete or concrete thickness is given in centimeters.

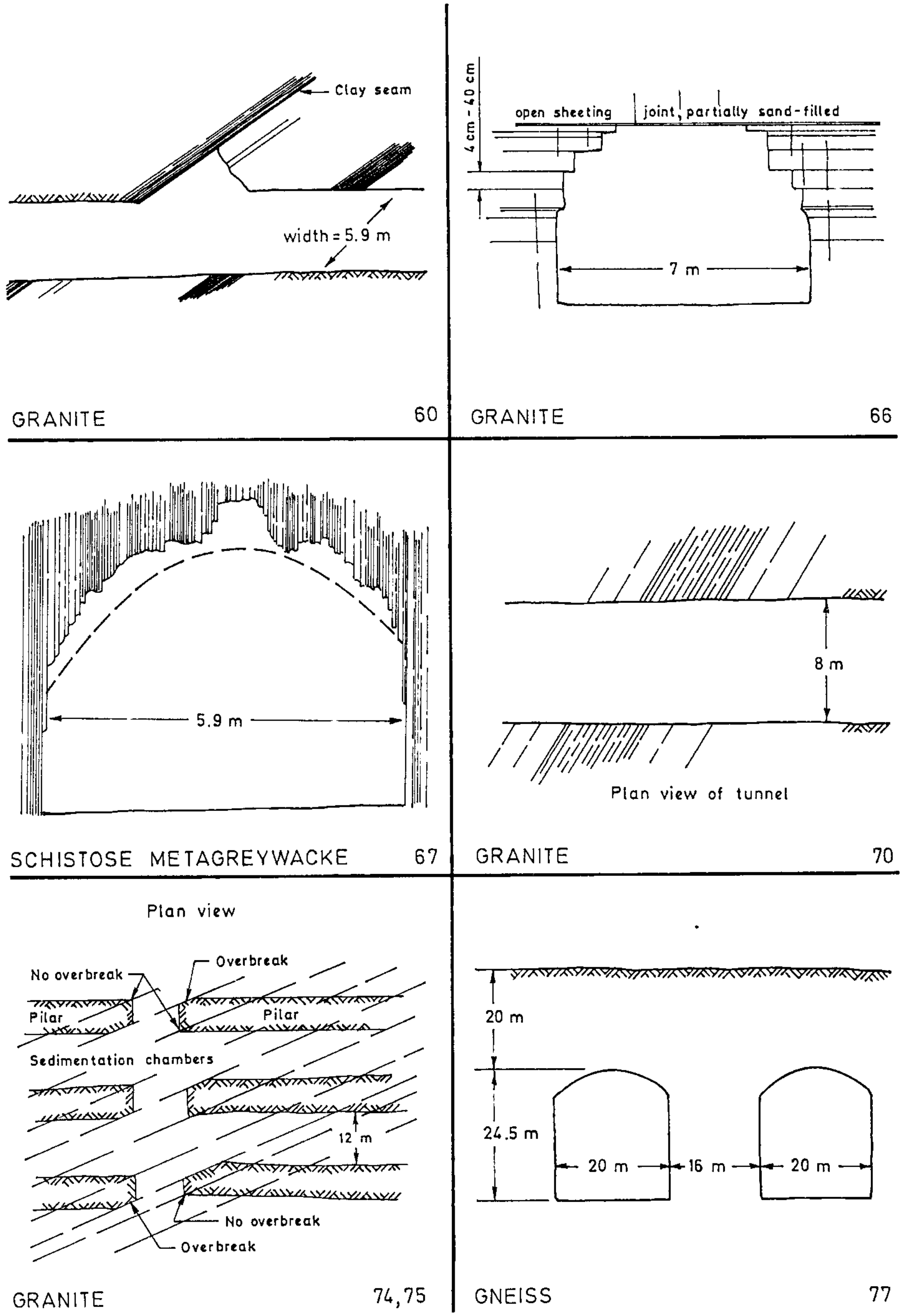

Fig. 8. Sketches of the six case records described in Table 9, after Cecil (1970)
Skizzen der sechs Fälle, welche nach Cecil (1970) in Tabelle 9 beschrieben sind
Croquis représentant les six cas décrits dans le tableau 9, selon Cecil (1970)

and those requiring some form of permanent support. Classification data for the thirty case records that plot on or below the boundary in Fig. 6 are given in Table 10.

As can be seen from the table, out of the thirty case records there are only six supported tunnels that plot below the boundary. The remaining one hundred and ninety case records contain only six further exceptions to the rule; i. e. unsupported tunnels that plot *above* the diagonal boundary. It therefore appears that self-supporting tunnels can be predicted with acceptable accuracy. The linear boundary can be approximated by the following equation:

$$D_e' = 2\,Q^{0.4} \tag{5}$$

where

D_e' = limiting value of *SPAN/ESR*

Q = rock mass quality

The unsupported spans listed in Table 10 range from 1.2 to 100 metres. Thus it does not appear that span-width need be a limiting factor, provided the rock mass quality is sufficiently high. In fact the Carlsbad limestone caverns of New Mexico have unsupported spans of up to 190 metres, presumably due both to the absence of joints and to a favourable stress field.

The classification data listed in Table 10 gives a good indication of the "vital statistics" of self-supporting tunnels. It appears that a high *RQD* value (mean $RQD = 85\%$) is common but not without exception. One joint set is also a common characteristic, although the mean value of J_n is 2.9, which represents one joint set plus random. None of these unsupported tunnels have more than three joint sets. In general the joints tend to be discontinuous or undulating (mean $J_r = 2.6$), though there are several examples with smooth-planar joints. The two most important requisites appear to be unaltered joints ($J_a \leqq 1$) and dry excavations ($J_w = 1$). There are very few exceptions to these two observations.

(D) Tunnel Support Recommendations

Different personal, national and continental engineering practices lead inevitably to variations in methods of support, even for the same quality of rock. The majority of data has been obtained from European case records due in particular to the ninety or so case records from Scandinavia (Cecil, 1970) and other Norwegian case records known to the authors. As a result of this European-Scandinavian bias, and the belief that shotcrete and bolting methods deserve most attention, many well documented case records have been ignored. These include those describing steel rib support methods, free span concrete arch roofs, and pre-cast sectional linings.

Small variations in support methods also occur in each category and are due to rock mass differences, since a given value of Q is not unique, but a combination of several variables. In order to separate the more im-

Table 10. Classification Data for Self-Supporting Tunnels

Support category	Case No.	Description of support used	RQD/J_n	J_r/J_a	J_w/SRF	$SPAN/ESR$ ($=D_e$)	Q
No. 0	6	none, *S* (1 app.) for protection from small stones	60/2	2/1	1/1	9/1.6	60
(no	8	none	70/2	1/1	1/1	9/1.6	35
support)	17	*sb* + *S* (1 app.) for protection from small stones	100/2	1.5/1	1/1	9/1.6	75
	20	none	70/2	1/1	1/1	9/1.6	35
	21	none	100/1	4/1	0.66/1	13/1.0	266
	27	(near category 13) none	90/3	1/1	1/1	12.5/1.6	30
	29	none	90/2	3/1	1/1	12.5/1.6	135
	35	none	10/3	2/1	1/1	5/1.6	6.7
	36	none	20/2	2/1	1/1	5/1.6	20
	63	(near category 17) *B*	100/9	1/1	1/2.5	5.9/1.6	4.4
	68	none	$100/{}^1/_2$	5/1	1/1	10/1.0	1000
	70	none	40/2	1.5/1	1/2.5	8/1.6	12
	74	(near category 9) none	100/2	1/1	1/1	12/1.3	16.7
	77	(near category 5) *sb* (50 bolts per 300 m)	100/1	5/1	1/2.5	20/1.3	200
	78	none	90/2	1.5/1	1/2.5	5/1.3	27
	87	none	100/1	4/1	1/1	11.25/1.6	400
	91	none	90/2	1.5/1	1/1	12/1.3	67.5
	96	none	100/1	4/1	?1/2.5	15/1.3	160
	101b	none	75/9	2/3	0.66/1	3.5/1.3	3.7
	112	none	80/2	2/1	1/15	1.2/1.6	5.3
	113	none	100/1	4/1	1/7.5	2.3/1.6	46
	115	(near category 13) *B* (1.0 m)	100/1	4/1	1/20	6.4/1.0	20
	119b	none	100/1	4/1	1/0.5	100/4	800
	119c	none	100/1	4/1	1/0.5	100/5	800
	120a	none	95/9	3/1	1/1	7/1.3	31.6
	120b	none	95/9	3/1	1/0.5	7/1.3	63
	127a	none or *sb*	100/4	3/1	1/0.5	20/5	75
	127b	none or *sb*	100/4	3/1	1/0.5	20/3	150
	144	*sb*, 2 m long	90/4	1/4	1/1	3/1.3	5.6
	150	none	100/4	2/1	0.5/0.5	6.1/1.3	50

Key: *S* = shotcrete (number of applications in brackets)
B = systematic bolting (mean spacing in brackets)
sb = spot bolting

portant variations in support practice, the conditional factors RQD/J_n and J_r/J_a should be evaluated in addition to the overall quality Q. Two excavations having the same rock mass quality Q, may in one case be bolted, and in the other case only shotcreted. The conditional factor RQD/J_n describing *block size* will normally separate these two cases. For instance, rock masses with RQD/J_n values larger than 10 will tend to be massive to blocky requiring only bolting, while values less than 10 are likely to represent blocky and jointed rock, which can often be adequately treated with shotcrete. In other examples, the conditional factor J_r/J_a describing *inter-block shear strength* may play a more important role. In some cases the equivalent dimension (D_e) which is equal to *SPAN/ESR* can be used as a third conditional factor to explain differences in support practice.

Tables 11, 12, 13 and 14 contain the authors' recommendations for permanent support for all 38 categories. It should be noted that the support tables have been designed in the first instance to supply estimates of permanent *roof* support. Methods for estimating permanent *wall* support are based on the hypothetical "wall quality" (range 1.0 Q to 5.0 Q) that was discussed on p. 213. A complete worked example is given at the end of the paper to illustrate the whole method.

It will have been noticed that no recommendations for temporary support have been discussed up to this point. Only a limited number of the case records contained such details. Therefore any recommendation given here will be an approximation, without the necessary back-up from case records. Nevertheless in principle, a tunnel with given values of *SPAN/ESR* and quality Q will obviously require reduced overall measures where temporary support is concerned. Appropriate reductions in support can be obtained by increasing the value of *ESR* to 1.5 *ESR*, and by increasing Q to 5 Q. In other words, shifting a plotted point downward and to the right hand side of Fig. 5, in the general direction of the NO SUPPORT boundary. These factors would be applied equally to both the roof and wall, such that any differences in roof and wall support would also be in operation for temporary support.

It should finally be emphasised that the support recommendations contained in this paper are based for the most part on general engineering practice for a given type of excavation. If for some reason the quality of drilling and blasting is better or worse than that in normal practice, then the recommended support will tend to be over-conservative or inadequate respectively. However, there is an additional complication in that the appearance of the excavated surfaces (apparent rock mass quality) tends to suggest either an increased or a decreased Q value for these two cases. For instance, when the drilling is poorly executed and hole alignment is bad, the degree of overbreak and need for support may increase considerably. Therefore, where possible, the rock mass quality Q should be estimated from exposures excavated in a similar manner to that used in the final excavations. Where this is not possible, allowances should be made, in particular with regard to the value of J_n (joint set number) and to a lesser extent *RQD*.

Table 11. Support Measures for Rock Masses of "Exceptional", "Extremely Good", "Very Good", and "Good" Quality (Q range: 1000—10)

Support category	Q	Conditional factors: RQD/J_n	J_r/J_n	$SPAN/ESR$ (m)	P kg/cm² (approx.)	$SPAN/ESR$ (m)	Type of support	Note see p. 229
1*	1000—400	—	—	—	< 0.01	20—40	sb (utg)	—
2*	1000—400	—	—	—	< 0.01	30—60	sb (utg)	—
3*	1000—400	—	—	—	< 0.01	46—80	sb (utg)	—
4*	1000—400	—	—	—	< 0.01	65—100	sb (utg)	—
5*	400—100	—	—	—	0.05	12—30	sb (utg)	—
6*	400—100	—	—	—	0.05	19—45	sb (utg)	—
7*	400—100	—	—	—	0.05	30—65	sb (utg)	—
8*	400—100	—	—	—	0.05	48—88	sb (utg)	—
9	100—40	≧ 20	—	—	0.25	8.5—19	sb (utg)	—
		< 20	—	—			B (utg) 2.5—3 m	—
10	100—40	≧ 30	—	—	0.25	14—30	B (utg 2—3 m	—
		< 30	—	—			B (utg) 1.5—2 m + clm	—
11*	100—40	≧ 30	—	—	0.25	23—48	B (tg) 2—3 m	—
		< 30	—	—			B (tg) 1.5—2 m + clm	—
12*	100—40	≧ 30	—	—	0.25	40—72	B (tg) 2—3 m	—
		< 30	—	—			B (tg) 1.5—2 m + clm	—
13	40—10	≧ 10	≧ 1.5	—	0.5	5—14	sb (utg)	I
		≧ 10	< 1.5	—			B (utg) 1.5—2 m	I
		< 10	≧ 1.5	—			B (utg) 1.5—2 m	I
		< 10	< 1.5	—			B (utg) 1.5—2 m + S 2—3 cm	I
14	40—10	≧ 10	—	≧ 15	0.5	9—23	B (tg) 1.5—2 m + clm	I, II
		< 10	—	≧ 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	40—10	> 10	—	—	0.5	15—40	B (tg) 1.5—2 m + clm	I, II, IV
		≦ 10	—	—			B (tg) 1.5—2 m + S (mr) 5—10 cm	I, II, IV
16* See note XII	40—10	> 15	—	—	0.5	30—65	B (tg) 1.5—2 m + clm	I, V, VI
		≦ 15	—	—			B (tg) 1.5—2 m + S (mr) 10—15 cm	I, V, VI

* Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

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.

Key to Support Tables:

sb = spot bolting
B = systematic bolting
(utg) = untensioned, grouted

Table 12. Support Measures for Rock Masses of "Fair" and "Poor" Quality (Q range: 10—1)

Support category	Q	Conditional factors RQD/J_n	J_r/J_a	$SPAN/ESR$	P Kg/cm² (approx.)	$SPAN/ESR$ (m)	Type of support	Note See p. 229
17	10—4	> 30	—	—	1.0	3.5—9	sb (utg)	I
		≧ 10, ≦ 30	—	—			B (utg) 1—1.5 m	I
		< 10	—	≧ 6 m			B (utg) 1—1.5 m + S 2—3 cm	I
		< 10	—	< 6 m			S 2—3 cm	I
18	10—4	> 5	—	≧ 10 m	1.0	7—15	B (tg) 1—1.5 m + clm	I, III
		> 5	—	< 10 m			B (utg) 1—1.5 m + clm	I
		≦ 5	—	≧ 10 m			B (tg) 1—1.5 m + S 2—3 cm	I, III
		≦ 5	—	< 10 m			B (utg) 1—1.5 m + S 2—3 cm	I
19	10—4	—	—	≧ 20 m	1.0	12—29	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	10—4	—	—	≧ 35 m	1.0	24—52	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	4—1	≧ 12.5	≦ 0.75	—	1.5	2.1—6.5	B (utg) 1 m + S 2—3 cm	I
		< 12.5	≦ 0.75	—			S 2.5—5 cm	I
		—	> 0.75	—			B (utg) 1 m	I
22	4—1	> 10, < 30	> 1.0	—	1.5	4.5—11.5	B (utg) 1 m + clm	I
		≦ 10	> 1.0	—			S 2.5—7.5 cm	I
		< 30	≦ 1.0	—			B (utg) 1 m + S (mr) 2.5—5 cm	I
		≧ 30	—	—			B (utg) 1m	I
23	4—1	—	—	≧ 15 m	1.5	8—24	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 m	I
24* See note XII	4—1	—	—	≧ 30 m	1.5	18—46	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

* Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

(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

Bolt spacings are given in metres (m). Shotcrete, or cast concrete arch thickness is given in centimetres (cm).

Table 13. Support Measures for Rock Masses of "Very Poor" Quality (Q range: 1.0—0.1)

Support category	Q	Conditional factors RQD/J_n	J_r/J_a	$SPAN/ESR$ (m)	P kg/cm² (approx.)	$SPAN/ESR$ (m)	Type of support	Note See p. 229
25	1.0—0.4	> 10	> 0.5	—	2.25	1.5—4.2	B (utg) 1 m + mr or clm	I
		≦ 10	> 0.5	—			B (utg) 1 m + S (mr) 5 cm	I
		—	≦ 0.5	—			B (tg) 1 m + S (mr) 5 cm	I
26	1.0—0.4	—	—	—	2.25	3.2—7.5	B (tg) 1 m + S (mr) 5—7.5 cm	VIII, X, XI
		—	—	—			B (utg) 1 m + S 2.5—5 cm	I, IX
27	1.0—0.4	—	—	≧ 12 m	2.25	6—18	B (tg) 1 m + S (mr) 7.5—10 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	1.0—0.4	—	—	≧ 30 m	2.25	15—38	B (tg) 1 m + S (mr) 30—40 cm	I, IV, V, IX
		—	—	≧ 20, < 30			B (tg) 1 m + S (mr) 20—30 cm	I, II, IV, IX
		—	—	< 20 m			B (gt) 1 m + S (mr) 15—20 cm	I, II, IX
		—	—	—			CCA (sr) 30—100 cm + B (tg) 1 m	IV, VIII, X, XI

29*	0.4—0.1	> 5	> 0.25	—	3.0	1.0—3.1	B (utg) 1 m + S 2—3 cm	—
		≦ 5	> 0.25	—			B (utg) 1 m + S (mr) 5 cm	—
		—	≦ 0.25	—			B (tg) 1 m + S (mr) 5 cm	—
30	0.4—0.1	≧ 5	—	—	3.0	2.2—6	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	0.4—0.1	> 4	—	—	3.0	4—14.5	B (tg) 1 m + S (mr) 5—12.5 cm	IX
		≦ 4, ≧ 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	0.4—0.1	—	—	≧ 20 m	3.0	11—34	B (tg) 1 m + S (mr) 40—60 cm	II, IV, IX, XI
		—	—	< 20 m			B (tg) 1 m + S (mr) 20—40 cm	III, IV, IX, XI
		—	—	—			CCA (sr) 40—120 cm + B (tg) 1 m	IV, VIII, X, XI

* Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

Table 14. Support Measures for Rock Masses of "Extremely Poor" and "Exceptionally Poor" Quality
(Q range: 0.1—0.001)

Support category	Q	Conditional Factors RQD/J_n	J_r/J_a	$SPAN/ESR$ (m)	P Kg/cm² (approx.)	$SPAN/ESR$ (m)	Type of support	Note See p. 229
33*	0.1—0.01	≧2	—	—	6	1.0—3.9	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	0.1—0.01	≧2	≧0.25	—	6	2.0—11	B (tg) 1 m +S (mr) 5—7.5 cm	IX
		<2	≧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	0.1—0.01	—	—	≧15 m	6	6.5—28	B (tg) 1 m +S (mr) 30—100 cm	II, IX, XI
See note XII		—	—	≧15 m			CCA (sr) 60—200 cm +B (tg) 1 m	VIII, X, XI, II
		—	—	<15 m			B (tg) 1 m +S (mr) 20—75 cm	IX, XI, III
		—	—	<15 m			CCA (sr) 40—150 cm +B (tg) 1 m	VIII, X, XI, III
36*	0.01—0.001	—	—	—	12	1.0—2.0	S (mr) 10—20 cm	IX
		—	—	—			S (mr) 10—20 cm +B (tg) 0.5—1.0 m	VIII, X, XI
37	0.01—0.001	—	—	—	12	1.0—6.5	S (mr) 20—60 cm	IX
		—	—	—			S (mr) 20—60 cm +B (tg) 0.5—1.0 m	VIII, X, XI
38 See note XIII	0.01—0.001	—	—	≧10 m	12	4.0—20	CCA (sr) 100—300 cm	IX
		—	—	≧10 m			CCA (sr) 100—300 cm +B (tg) 1 m	VIII, X, II, XI
		—	—	<10 m			S (mr) 70—200 cm	IX
		—	—	<10 m			S (mr) 70—200 cm +B (tg) 1 m	VIII, X, III, XI

* Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

Supplementary Notes for Support Tables

I. For cases of heavy rock bursting or "popping", tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when "popping" activity ceases.

II. Several bolt lengths often used in same excavation, i. e. 3, 5 and 7 m.

III. Several bolt lengths often used in same excavation, i. e. 2, 3 and 4 m.

IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2—4 m.

V. Several bolt lengths often used in some excavations, i.e. 6, 8 and 10 m.

VI. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4—6 m.

VII. Several of the older generation power stations in this category employ systematic or spot bolting with areas of chain link mesh, and a free span concrete arch roof (25—40 cm) as permanent support.

VIII. Cases involving swelling, for instance montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.

IX. Cases not involving swelling clay or squeezing rock.

X. Cases involving squeezing rock. Heavy rigid support its generally used as permanent support.

XI. According to the authors' experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i. e. > 1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i. e. $RQD/J_n < 1.5$, for example a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete (or shotcrete) arch to reduce the uneven loading on the concrete, but it may not be effective when $RQD/J_n < 1.5$, or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick setting resin anchors in these extremely poor quality rock-masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.

XII. For reasons of safety the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 ($SPAN/ESR > 15$ m only).

XIII. Multiple drift method usually needed during excavation and support of arch, walls and floor in cases of heavy squeezing. Category 38 ($SPAN/ESR > 10$ m only).

(E) Worked Example
20 m Span Machine Hall in Phyllite

(i) Rock Mass Classification

Joint set 1	strongly developed foliation	
	smooth-planar	($J_r = 1.0$)
	chlorite coatings	($J_a = 4.0$)
	ca. 15 joints/metre	
Joint set 2	smooth-undulating	($J_r = 2$)
	slightly altered joint walls	($J_a = 2$)
	ca. 5 joints/metre	
	$J_v = 15 + 5 = 20$	$RQD = 50$ (Eq. 2)
	$J_n = 4$. Minimum $J_r/J_a = 1/4$	

Minor water inflows: $J_w = 1.0$

Unconfined compression strength of phyllite $(\sigma_c) = 400$ kg/cm²

Major principal stress $(\sigma_1) = 30$ kg/cm²

Minor principal stress $(\sigma_3) = 10$ kg/cm²

$(\sigma_1/\sigma_3) = 3$

$\sigma_c/\sigma_1 = 13.3$ (medium stress) $SRF = 1.0$

$$Q = \frac{50}{4} \cdot \frac{1}{4} \cdot \frac{1}{1} = 3.1 \text{ (poor)} \qquad \text{(Eq. 1)}$$

(ii) Support Recommendation

Type of excavation	Machine hall	$B = 20$ m, $H = 30$ m
	($ESR = 1.0$)	$B/ESR = 20$, $H/ESR = 30$
Support category	(a) Roof	$Q = 3.1$; category 23 (Fig. 5)
	(b) Walls	"Q" $= 3.1 \cdot 2.5$; category 20
Recommended Support	(a) Category 23 Table 12	B (tg) 1.4 m (Roof) + S (mr) 15 cm Notes: II, IV, VII
	(b) Category 20 Table 12	B (tg) 1.7 m (Walls) + S (mr) 10 cm Notes: II, IV

Mean length of bolts and anchors	(a) Roof	bolts	5.0 m	(Appendix)
		anchors	8.0 m	(Appendix)
	(b) Walls	bolts	6.5 m	(Appendix)
		anchors	10.5 m	(Appendix)

Support pressure estimates

(a) Roof $Q = 3.1$
1. (Fig. 4, shaded envelope) Approx. range for $P_{\text{roof}} = 0.9$—$2.0\ \text{kg/cm}^2$
2. (Eq. 3) $P_{\text{roof}} = 1.37\ \text{kg/cm}^2$
3. (Eq. 4) $P_{\text{roof}} = 0.91\ \text{kg/cm}^2$

(b) Walls "Q" $= 3.1 \cdot 2.5$
1. (Fig. 4, shaded envelope) Approx. range for $P_{\text{wall}} = 0.6$—$1.4\ \text{kg/cm}^2$
2. (Eq. 3) $P_{\text{wall}} = 1.01\ \text{kg/cm}^2$
3. (Eq. 4) $P_{\text{wall}} = 0.67\ \text{kg/cm}^2$

Commentary

1. Note the use of the minimum value J_r/J_a for calculating Q. The properties of the joint set having the lowest shear strength should always be used, unless the user considers the orientation is entirely favourable such that a second joint is more unfavourable to stability, despite having a higher value of J_r/J_a.

2. The choice of 1.4 m and 1.7 m spacing for roof and wall bolts from the empirical listed ranges of 1—1.5 m and 1—2 m was made in accordance with the specific value of Q, in relation to the range for the given category (i. e. $Q = 1$—4). These bolt spacings are approximate and need to be checked against required support pressures.

3. When using Tables 11, 12, 13 and 14 for *wall support*, the relevant *span* should be used when the conditional factor (*SPAN/ESR*) is listed. Hence the choice of the minimum 10 cm of mesh reinforced shotcrete from a possible range of 10—20 cm.

4. The mean bolt and anchor lengths should be coordinated with the recommendation given under *Note II* (p. 229). Thus, for the roof, variable (intermeshed) bolt lengths of 3, 5 and 7 m appear reasonable, while for the wall 5, 6.5 and 8 m might be more appropriate. The recommendation for using tensioned cable anchors *(Note IV)* is based on current practice in most caverns of this size. The effectiveness of such widely spaced (2—4 m) reinforcement is perhaps open to question.

5. The range of estimates of support pressure give room for choice. The estimates obtained from Eq. (4) are especially dependent on the absence of additional joint sets. Should some additional random joints be discovered when access tunnels are driven into this hypothetical rock mass, both J_n and Q will be affected, and this will have a multiple effect on Eq. (4). The value of J_n will increase to 6, Q will reduce to 2.1, and the estimate of roof support pressure would rize from 0.91 to 1.28 kg/cm².

Appendix

Design Concepts for Rationalizing the Support Tables

The simple theory used to rationalize the support dimensioning can be conveniently divided into three parts: bolting, concrete lining, shotcrete lining.

1. Bolting

The support pressure capacity of tensioned or grouted bolts is equal to the yield capacity of one bolt (if adequately anchored) divided by the square of the bolt spacing. If a 10 tons working load is assumed for a 20 mm diameter bolt, the support pressure is as follows:

$$P = 1/a^2 \tag{6}$$

where

P = support pressure capacity in kg/cm²

a = bolt spacing in metres

Eq. (6) and the support pressure chart (Fig. 4) were used in combination with the case records, and this helped to provide a rational and reasonably continuous spectrum of bolt spacings. When a range of spacings is quoted in Tables 11, 12, 13 and 14, for instance 1.5 to 2.0 m, the lower limit applies to the lowest rock mass quality Q, and the upper limit to the highest rock mass quality in each given support category. In cases where anchors were noted as a supplementary reinforcement method, the given bolt spacings could be increased, provided the total support pressure generated by the combined bolting and anchoring was not reduced.

Bolt and anchor lengths depend on the dimensions of the excavations. Lengths used in the roof arch are usually related to the span, while lengths used in the walls are usually related to the height of the excavations. The ratio of bolt length to span tends to reduce as the span increases. This trend has been demonstrated by Benson et al. (1971). Accordingly, the following recommendations are given as a simple rule-of-thumb, to be modified as in situ conditions demand.

Roof: bolts $L = 2 + 0.15\, B/ESR$ (7)

anchors $L = 0.40\, B/ESR$ (8)

Walls: bolts $L = 2 + 0.15\, H/ESR$ (9)

anchors $L = 0.35\, H/ESR$ (10)

where

L = length in metres

B = span in metres

H = excavation height in metres

ESR = excavation support ratio

2. Concrete Lining

The theory of thin walled cylinders provides a simple expression between lining thickness, resultant stress in lining, and uniform internal or external pressure at equilibrium. For external loading the following expression is obtained:

$$t = \frac{P \cdot R}{\sigma} \tag{11}$$

where

P = externally applied pressure (kg/cm^2)

σ = compressive stress in lining (kg/cm^2)

R = internal radius of lining (cm)

t = wall thickness for equilibrium (cm)

The above expression is based on the assumption that bending and shear stresses are absent.

When a concrete lining is used in combination with systematic bolting, stresses caused by uneven loading or non-circular linings can presumably be minimized and the above equation used with a conservative value for allowable stress. If bolt tensions could be guaranteed, some sharing of support pressure would occur and lining thickness could be reduced. However, some form of internal steel reinforcement may be required to reduce the unfavourable effect of uneven stresses. A conservative value of σ (allowable) equal to 50 kg/cm^2 was assumed when rationalizing Tables 11, 12, 13 and 14. The appropriate range of pressure (P) was estimated using Fig. 4, in combination with available case records.

Support pressure load sharing by systematic bolting was ignored, therefore concrete thickness may be too conservative if bolts are added and anchorage is effective. However, it should be emphasised that concrete lining is only recommended in the poorest qualities of rock mass, where the effectiveness of bolt anchorage is relatively uncertain.

3. Shotcrete Lining

When single (2—3 cm) or double (5 cm) applications of shotcrete are applied — usually in combination with systematic bolting (i. e. support categories 21 and 25, Tables 12 and 13) — the function of the shotcrete is to prevent loosening, especially in the zone between bolts. In such cases no attempt was made to use Eq. (11) for design thicknesses. The mode of failure of thin layers of shotcrete is one of shear, not bending or compression, as emphasised by Rabcewicz (1969) and Müller (1970). In fact, the support tables are based on a wealth of case records in these support categories, and any attempt to incorporate theory would be superfluous, even if the relevant theory was reliable.

Conclusions

1. The method of classifying rock masses for tunneling stability incorporates six parameters which can be estimated in the first instance using an inexpensive combination of field mapping and geological engineering judgement. Should bore core be available together with the results of rock mechanics tests such as point load strength, natural rock stress etc., then the estimate of rock mass quality Q will be that much more reliable, though not necessarily more accurate. At a more advanced stage of a project when exploratory adits are available, the estimates of Q can, and should be updated further. Support requirements may be re-evaluated in the light of the in situ conditions revealed.

2. The support recommendations contained in this paper are based for the most part on general engineering practice for a given type of excavation. If for some reason the quality of drilling and blasting is better or worse than that in normal practice, then the recommended support will tend to be over-conservative or inadequate respectively. The most reliable estimates of rock mass quality Q and support measures will therefore be obtained from exposures excavated by the same methods as those to be used in the final excavations. Where this is not possible, allowance should be made, particularly with regard to the value of J_n (joint set number) and RQD.

3. The use of past and present case records as a basis for future design introduces the danger of perpetuating over-conservative (and occasionally under-conservative) practice. Consequently, case records describing failure of temporary support, or the necessity for additional support are especially valuable for indicating what the present safety margins are.

4. A further danger of using past and present case records as a basis for future design is that excavation techniques are changing. More and more smooth wall blasting is used and more and more tunnels are machinebored. The support pressure required will reduce as improved excavation techniques result in less disturbance of the surrounding rock. These trends must be incorporated as they occur.

5. Readers in a position to supply detailed case records, especially in areas where the authors' data is sparse, could make a valuable contribution, enabling the updating and improvement of the support tables.

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