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ESTIMATION OF SUPPORT REQUIREMENTS FOR UNDERGROUND EXCAVATIONS

ESTIMATION DES SOUTÈNMENTS NÉCESSAIRES POUR LES EXCAVATIONS SOUTERRAINES

ABSCHÄTZUNG DES NÖTIGEN FELSAUSBAUES IM HOHLRAMBAU

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An analysis of some 200 case records has revealed a useful correlation between the amount and type of permanent support and the rock mass quality Q, with respect to excavation stability. 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 interblock shear strength, and the active stress. 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. 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.

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 et, d'autre part, la qualité Q des masses rocheuses, en ce qui concerne la stabilité. 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. Des analyses 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. Les mesures 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.

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 Gebirgsqualität Q ist eine Funktion von sechs Parametern, die aus Oberflächenbeobachtungen und nach skalierten Gewichten bestimmte Leitziffern erwerfet 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, dei Spannungsniveau und Wasserzufluss berücksichtigen. Wenn man diese Parameter koordiniert, vertreten sie den Einfluss der Körnung, der Scherfestigkeit an den Anschlussflächen zwischen den Felsblöcken und der einwirkenden Spannungen. Analysen der Gebirgsqualität und der entsprechenden Sicherungsmassnahmen haben erwiesen, dass es möglich ist, einen angemessenen Ausbau fürs ganze Spektrum der Gebirgsqualität zu veranschlagen. Die Sicherungsmassnahmen umfassen verschiedene Kombinationen von Nägeln, Ankern, Spritzbeton und Ortsbetongewölben sowie auch Angaben über Ankerabstände und erforderliche Stärke des Spritz- oder Gussbetons.

INTRODUCTION

Two important factors for the stability of underground excavations are their location and orientation relative to unfavourable geological conditions. Both factors are weighed to minimise difficult rock conditions for the case of large span openings of limited length. However there is little opportunity to choose the orientation of tunnels, and generally only the location can be changed significantly. The amount of support required will be strongly dependent on orientation if poor rock conditions are encountered.

Estimates of support are required at three stages in a project: for the feasibility studies, for the detailed planning, and finally during excavation itself. In view of the economic importance of support costs it is vital that the support estimates are as accurate as possible for all three stages. The accuracy will depend partly on the success of the geological investigations, and partly on the success of extrapolating past experiences of support performance to new rock mass environments.
When beginning this work of support estimation a literature survey directed towards related excavations

in similar rocks can be extremely useful. Subsequently several site visits to related projects will further contribute to the familiarization process for the engineers concerned with the new design. No matter how many sophisticated rock mechanics test programmes and /or finite element analyses are performed, the design engineers will come back to the basic question - "is this bolt spacing, shotcrete thickness, or unsupported span width reasonable in the given rock mass?" Their opinions are likely to be based mainly on past experience in such projects and on their recent literature and case record study. Rock mechanics testing and finite element analyses will probably contribute little to the final decision of bolt spacing and shotcrete thickness, although the excavation shape and layout may of course benefit from such analyses. *Underground excavations are supported with some confidence primarily because many others have been supported before them and they have performed satisfactorily.*

Empirical design is likely to persist for a long time in the planning of underground support, due to the enormous complexity of the problem. It is therefore all the more important to have an objective method of analysing case records, so that this past experience can be used logically in the planning of support for new excavations in different rock mass environments.

Approximately two hundred case records have been analysed for the purpose of finding out what type and amount of support is used for a given type and size of excavation in given rock mass conditions. The quality of the rock mass is described numerically using a six parameter classification which can encompass more than 300,000 combinations of geotechnical conditions. The method appears to have great promise, although its reliability could obviously be improved by putting it to test in further projects. This paper is written in the hope of stimulating engineers and geologists to try the method, and to provide both critical and positive feedback especially in areas where the authors' case record data is sparse or non-existent. The following steps are involved in testing the method

1. Classify the revelant rock mass quality (or qualities) by means of surface mapping, bore core analysis, trial adits, etc. The method of classification, which is explained fully in the following pages (Tables 1 to 6) consists of numerically rating the following rock mass parameters: joint density (RQD) number of joint sets, roughness of most unfavourable joint set, degree of alteration or filling of most unfavourable joint set, rock load resistance, water inflow.

2. Choose optimum dimensions of excavations, keeping in mind the purpose of each excavation and the degree of safety required, i.e. power house, water tunnel, road tunnel, access tunnel etc.

3. Estimate the appropriate permanent support (shotcrete thickness, bolt spacing, cast concrete arch thickness etc.) for each excavation using the support tables (Tables 8,9,10,11).

The method is essentially a weighting process in which the positive and negative aspects of a rock mass are assessed. A store of experience (case records) is searched to try to find the most appropriate support measures for the given excavations and rock mass conditions. The whole proceedure is probably not dissimilar to the mental process occuring when a very experienced tunneling consultant is asked for his support recommendations.

METHOD FOR ESTIMATING ROCK MASS QUALITY Q

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

$$Q = (RQD/J_n).(J_r/J_a).(J_w/SRF) \qquad (1)$$

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 three pairs of parameters are found to be crude measures of .

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

The rock mass descriptions and ratings for each of the six parameters are given in Tables 1 to 6. 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. 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 occurences. 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. (1974a). Three examples are given later in this paper.

Table 1. Descriptions and ratings for the parameter RQD.

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

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

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

Table 2. Descriptions and ratings for the parameter J_n

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

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

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

Table 3. Descriptions and ratings for the parameter J_r

3. JOINT ROUGHNESS NUMBER	
(a) *Rock wall contact* and (b) *Rock wall contact before 10 cms shear*	(J_r)
A. Discontinuous joints	4
B. Rough or irregular, undulating	3
C. Smooth, undulating	2
D. Slickensided, undulating	1.5
E. Rough or irregular, planar	1.5
F. Smooth, planar	1.0
G. Slickensided, planar	0.5
Note: (i) Descriptions refer to small scale features and intermediate scale features, in that order.	
(c) *No rock wall contact when sheared*	
H. Zone containing clay minerals thick enough to prevent rock wall contact	1.0
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0
Note: (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3m. (iii) J_r =0.5 can be used for planar slickensided joints having lineations, provided the lineations are orientated for minimum strength	

Table 4. Descriptions and ratings for the parameter J_a

4. JOINT ALTERATION NUMBER	(J_a)	(ϕ_r)
(a) *Rock wall contact*		(approx.)
A. Tightly healed, hard, non-softening, impermiable filling i.e. quartz or epidote	0.75	(-)
B. Unaltered joint walls, surface staining only	1.0	(25-35°)
C. Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	2.0	(25-30°)
D. Silty-, or sandy-clay coatings, small clay fraction (non-soft.)	3.0	(20-25°)
E. Softening or low friction clay mineral coatings, i.e. kaolinite or mica. Also chlorite, talc, gypsum, graphite etc., and small quantities of swelling clays.	4.0	(8-16°)
(b) *Rock wall contact before 10 cms shear*		
F. Sandy particles, clay-free disintegrated rock etc.	4.0	(25-30°)
G. Strongly over-consolidated non-softening clay mineral fillings (continuous, but <5 mm thickness)	6.0	(16-24°)
H. Medium or low over-consolidation, softening, clay mineral fillings. (continuous but <5mm thickness)	8.0	(12-16°)
J. Swelling -clay fillings, i.e. montmorillonite (continuous, but <5mm thickness) Value of J_a depends on percent of swelling clay-size particles, and access to water etc.	8 - 12	(6-12°)
(c) *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, 8, or 8-12	(6-24°)
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, 13, or 13-20	(6-24°)

Table 5. Descriptions and ratings for the parameter J_w

5. JOINT WATER REDUCTION FACTOR	(J_w)	Approx. water pres. (kg/cm^2)
A. Dry excavations or minor inflow, i.e. < 5 l/min. locally.	1.0	<1
B. Medium inflow or pressure, occasional outwash of joint fillings.	0.66	1 - 2.5
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5-10
D. Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5-10
E. Exceptionally high inflow or water pressure at blasting, decaying with time	0.2-0.1	>10
F. Exceptionally high inflow or water pressure continuing without noticeable decay	0.1-0.05	>10
Note: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed. (ii) Special problems caused by ice formation are not considered.		

Table 6. Descriptions and ratings for parameter (SRF)

6. STRESS REDUCTION FACTOR	
(a) *Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.*	(SRF)
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B. Single weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤ 50m)	5
C. Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50m)	2.5
D. Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5
E. Single shear zones in competent rock (clay-free) (depth of excavation ≤ 50m)	5.0

		(SRF)
F.	Single shear zones in competent rock (clay-free) (depth of excavation > 50m)	2.5
G.	Loose open joints, heavily jointed or "sugar cube" etc. (any depth)	5.0

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

(b) *Competent rock, rock stress problems*

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

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

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

(c) *Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure*

		(SRF)
N.	Mild squeezing rock pressure	5 - 10
O.	Heavy squeezing rock pressure	10 - 20

(d) *Swelling rock: chemical swelling activity depending on presence of water*

P.	Mild swelling rock pressure	5 - 10
R.	Heavy swelling rock pressure	10 - 15

ADDITIONAL NOTES ON THE USE OF TABLES 1 *to* 6.

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

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

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

where

J_v = total number of joints per m^3
($RQD = 100$ 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 2.

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 the 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 equation 1. *The value of* (J_r/J_a) *should in fact relate to the surface most likely to allow failure to initiate.*

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

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

ORIENTATION AND WEAKNESS ZONES

Potential users of this classification method will have noted that the only mention of *joint orientation* is in Note 3 above. Most of the case records that were analysed included the necessary information on structural orientation relative to the excavation axes. However the information was not found to be sufficiently important to justify the use of a seventh parameter. No doubt this was in some cases due to the fact that excavation axes were already orientated favourably with respect to weakness zones. It is certainly necessary to orientate important excavations favourably with respect both to stress anisotropy and to weakness zones, as usually attempted.

However, the weakness zone poses a threat to stability not only because of its potential orientation, but also because of its weakness. A rough unfilled joint having identical orientation might not even be noticed and would certainly pose no threat to stability.

It is probable that engineers and geologists who note the presence of "unfavourably orientated" discontinuities in an excavation - and this is admittedly an important observation - do so because these surfaces are visible. One of the reasons that they are visible is that overbreak occurs preferentially along their surfaces. This is partly a consequence of planarity and/or filling material. In fact the surfaces in question are relatively non-dilatant, so offer little resistance to continued shearing. Discontinuities with $J_r/J_a \le 1$ would probably come under this category.

It is in fact difficult to separate the observation "unfavourably orientated" from the implication of low dilatancy and low shearing resistance. The number of joint sets may also play an important role here, since this number controls the degree of freedom for block

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

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

3. GRANITE RQD = 0
$Q = (10/20).(1.0/6).(0.66/6)$
$= 0.009$ (exceptionally poor)
(1A/2J, 3J/4K, 5B/6N)

Figure 1. Examples of classification for three dissimilar granitic rock masses.

fall-out, if any, whatever the orientation or shearing resistance of the discontinuities or joints. Most of the influence of orientation is automatically reflected in the value of Q since the parameters J_n, J_r, J_a and SRF are indirectly weighted by "unfavourably orientated" features.

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. 1971), 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.

EXAMPLES OF ROCK MASS CLASSIFICATION

Figure 1 illustrates the method of classifying rock masses for their quality Q. The three photographs are of surface exposures, but imaginary tunnel depths of around 40m have been assumed. Therefore water pressures and rock pressures of medium values have been assumed for each of the examples. Beneath each photograph the following are listed:

1. Rock type. RQD.
2. Rock mass quality Q and values of the 6 parameters: RQD/J_n, J_r/J_a, J_w/SRF.
3. Numerical and alphabetical coding to the classification descriptions given in Tables 1 to 6. *(This coding may be used for concise recording of rock conditions in routine tunnel mapping).*

The following points can be noted from the classification of the three granitic rock masses:

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

2. The decomposed granite shown in example 3 has a very low srtength. It is probable that at 40m 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.

ESTIMATION OF SUPPORT BASED ON CASE RECORDS

(A) *EXCAVATION SUPPORT CHART FOR ANALYSIS OF CASE RECORDS*

The method of classifying a rock mass to obtain 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 shown in Figure 2. 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 listed in Tables 8, 9, 10, and 11.

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

Table 7. The excavation support ratio (ESR) appropriate to a variety of underground excavations.

Type of excavation	ESR	No.
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. (hemispherical 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)

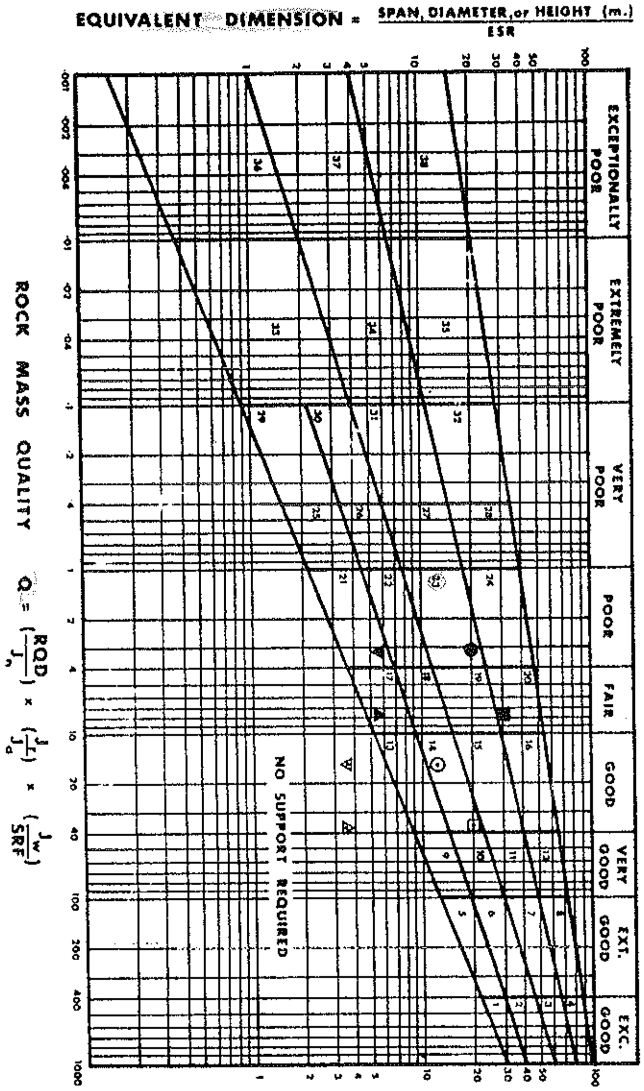

Figure 2. Excavation support chart showing the box numbering for 38 categories of support. The plotted points refer to the worked examples given in the appendix.

Machine Hall	Tailrace Tunnel	
●	▼	roof (permanent)
■	▲	walls "
⊙	▽	roof (temporary)
⊡	△	walls "

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 revelant 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 cases, hence the question marks.

More than 200 case records were evaluated, and the relevant values of Q and SPAN/ESR are plotted in Fig.3 In all, more than 90 of the case records were obtained from Cecil (1970), who visited and mapped a wide variety of excavation conditions in Scandinavia.

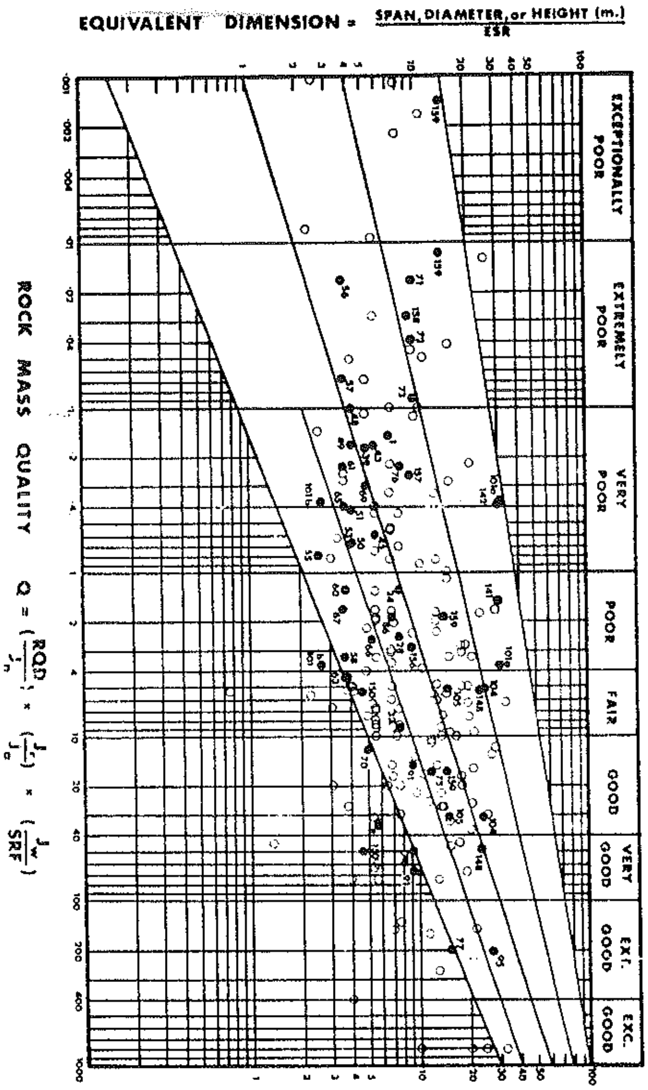

Figure 3. Support estimates are based on analysis of more than 200 case records. Numbered points refer to case records that are described in detail by Barton et al.(1974a)

(B) *GENERAL EXCAVATION SUPPORT ESTIMATES*

Different engineering practices inevitably lead 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 to the Norwegian cases known to the authors. As a result of this European - Scandinavian bias, and the belief that bolting and shotcrete methods deserve most attention, several 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. However a large number of the several hundred case records that were reviewed could not be included, as some aspect of the rock mass or support was inadequately described.

The general estimates of support for each of the 38 *support categories* (Figure 2) are given in Tables 8, 9, 10, and 11. They have been tailored to fit the largest number of case records possible, that plot within the same support category. (See Figure 3). Exceptionally conservative or (occasionally) unsafe designs are automatically excluded from consideration since it is impossible to accomodate them in a generally applicable support recommendation for a given category.

However, small variations in support methods do occur in each category 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 important 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. In other examples the *conditional factor* J_r/J_a describing *inter-block shear strength* may play a more important role, and occasionally the value of SPAN/ESR also helps to differentiate support methods.

In cases involving swelling or squeezing rock, the Notes appearing in the right hand columns of Tables 8, 9, 10 and 11 are also used to differentiate support requirements (see Notes VIII, IX and X).

The support recommendations listed in Tables 8, 9, 10 and 11 have been designed in the first instance to give estimates of *permanent roof support*, since they are based on the roof support methods quoted in the case records. However, Figure 2 and the tables can also be used to estimate the wall support, and the temporary support. The suggested methods are given in the appendix, together with recommendations for bolt and anchor lengths, and complete worked examples to illustrate the method.

Key to Support Tables:

- sb = spot bolting
- B = systematic bolting
- (utg) = untensioned, grouted
- (tg) = tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; see Note XI)
- S = shotcrete
- (mr) = mesh reinforced
- clm = chain link mesh
- CCA = cast concrete arch
- (sr) = steel reinforced

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

Table 8. Support Measures for Rock Masses of "Exeptional", "Extremely Good", "Very Good" and "Good" Quality (Q range: 1000-10)

Support category	Conditional factors RQD/J_n	J_r/J_a	$SPAN/ESR$	Type of support	Notes
1*	-	-	-	sb(utg)	-
2*	-	-	-	sb(utg)	-
3*	-	-	-	sb(utg)	-
4*	-	-	-	sb(utg)	-
5*	-	-	-	sb(utg)	-
6*	-	-	-	sb(utg)	-
7*	-	-	-	sb(utg)	-
8*	-	-	-	sb(utg)	-
9	≥20	-	-	sb(utg)	
	<20	-	-	B(utg) 2.5-3 m	-
10	≥30	-	-	B(utg) 2-3 m	-
	<30	-	-	B(utg) 1.5-2 m +clm	-
11*	≥30	-	-	B(tg) 2-3 m	-
	<30	-	-	B(tg) 1.5-2 m +clm	-
12*	≥30	-	-	B(tg) 2-3 m	-
	<30	-	-	B(tg) 1.5-2 m +clm	-
13	≥10	≥1.5	-	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	≥10	-	≥15	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	>10	-	-	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	>15	-	-	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.

Note: The type of support to be used in categories 1 to 8 will depend on the blasting technique. Smooth wall blasting and thorough barring-down may remove the need for support. Rough-wall blasting may result in the need for single applications of shotcrete, especially where the excavation height is >25 m. Future case records should differentiate categories 1 to 8.

Table 9. Support Measures for Rock Masses of "Fair" and "Poor" quality (Q range: 10-1).

Support category	Conditional factors RQD/J_n	J_r/J_a	$SPAN/ESR$	Type of support	Note
17	>30	-	-	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	>5	-	≥10 m	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	-	-	≧20 m	B(tg) 1-2 m +S(mr) 10-15 cm	I,II,IV
	-	-	<20 m	B(tg) 1-1.5 m +S(mr) 5-10 cm	I,II
20* See note XII	-	-	≧35 m	B(tg) 1-2 m +S(mr) 20-25 cm	I,V,VI
	-	-	<35 m	B(tg) 1-2 m +S(mr) 10-20 cm	I,II,IV
21	≧12.5	≦0.75	-	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	(>10, <30)	>1.0	-	(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) 1 m	I
23	-	-	≧15 m	B(tg) 1-1.5 m +S(mr) 10-15 cm	I,II,IV, VII
	-	-	<15 m	B(utg) 1-1.5 m +S(mr) 5-10 cm	I
24* See note XII	-	-	≧30 m	B(tg) 1-1.5 m +S(mr) 15-30 cm	I,V,VI
	-	-	<30 m	B(tg) 1-1.5 m +S(mr) 10-15 cm	I,II,IV

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

Table 10. Support Measures for Rock Masses of "Very poor" Quality (Q range: 1.0-0.1)

Support category	Conditional factors $\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{SPAN}{ESR}$	Type of support	Note
25	>10	>0.5	-	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	-	-	-	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	-	-	≧12m	B(tg) 1 m +S(mr) 7.5-10cm	I,IX
	-	-	<12m	B(utg) 1 m +S(mr) 5-7.5 cm	I,IX
	-	-	>12m	CCA 20-40 cm +B(tg) 1 m	VIII,X, XI
	-	-	<12m	S(mr) 10-20 cm +B(tg) 1 m	VIII,X, XI
28* See note XII	-	-	≧30m	B(tg) 1 m +S(mr) 30-40 cm	I,IV,V, IX
	-	-	(≧20, <30m)	B(tg) 1 m +S(mr) 20-30 cm	I,II,IV, IX
	-	-	<20m	B(tg) 1 m +S(mr) 15-20 cm	I,II,IX
	-	-	-	CCA(sr)30-100cm +B(tg) 1 m	IV,VIII, X,XI
29*	>5	>0.25	-	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	≧5	-	-	B(tg) 1 m +S 2.5-5 cm	IX
	<5	-	-	S(mr) 5-7.5 cm	IX
	-	-	-	B(tg) 1 m +S(mr) 5-7.5 cm	VIII,X, XI
31	>4	-	-	B(tg) 1 m +S(mr) 5-12.5cm	IX
	≦4,≧1.5	-	-	S(mr) 7.5-25 cm	IX
	<1.5	-	-	CCA 20-40 cm +B(tg) 1 m	IX
	-	-	-	CCA(sr)30-50 cm +B(tg) 1 m	VII,X, XI
32 See note XII	-	-	≧20m	B(tg) 1 m +S(mr) 40-60 cm	II,IV, IX
	-	-	<20m	B(tg) 1 m +S(mr) 20-40 cm	III,IV, IX
	-	-	-	CCA(sr)40-120cm +B(tg) 1 m	IV,VIII, X,XI

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

Table 11. Support Measures for Rock Masses of "Extremely Poor" and "Exceptionally Poor" Quality (Q range: 0.1-0.001)

Support category	Conditional factors $\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{SPAN}{ESR}$	Type of support	Note
33*	≧2	-	-	B(tg) 1 m +S(mr) 2.5-5 cm	IX
	<2	-	-	S(mr) 5-10 cm	IX
	-	-	-	S(mr) 7.5-15 cm	VIII,X
34	≧2	≧0.25	-	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 See note XII	-	-	≧15m	B(tg) 1 m +S(mr) 30-100cm	II,IX
	-	-	≧15m	CCA(sr)60-200cm +B(tg) 1 m	VIII,X, XI,II
	-	-	<15m	B(tg) 1 m +S(mr) 20-75 cm	IX,III
	-	-	<15m	CCA(sr)40-150cm +B(tg) 1 m	VIII,X, XI,III
36*	-	-	-	S(mr) 10-20 cm	IX
	-	-	-	S(mr) 10-20 cm +B(tg) 0.5-1.0m	VIII,X, XI
37	-	-	-	S(mr) 20-60 cm	IX
	-	-	-	S(mr) 20-60 cm +B(tg) 0.5-1.0m	VIII,X, XI
38 See note XIII	-	-	≧10m	CCA(sr)100-300cm	IX
	-	-	≧10m	CCA(sr)100-300cm +B(tg) 1 m	VIII,X, II,XI
	-	-	<10m	S(mr) 70-200 cm	IX
	-	-	<10m	S(mr) 70-200 cm +B(tg) 1 m	VIII,X, III,XI

*Authors' estimates of support. Insufficient case records available for 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. (Selmer-Olsen, 1970)

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 same 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. See Selmer-Olsen (1970). Drainage measures are used where possible.

IX. Cases not involving swelling clay or squeezing rock.

X. Cases involving squeezing rock. Heavy rigid support is 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, 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).

(C) *EXAMPLES OF CASE RECORD ANALYSIS AND SUPPORT COMPARISON*

Application of the classification-support method is illustrated in Table 12. The three case records and the sketches given in Figure 4 were obtained from Cecil (1970) and illustrate a wide range of conditions and dimensions. The authors' estimates of *permanent roof support* found in tables 8, 9, 10 and 11 are compared in each case with the support actually used. The classification ratings obtained from Tables 1 to 6 can be checked against descriptions using the code letters listed in Table 12. More detailed worked examples are given in an appendix. These include estimates for wall support and for temporary support.

PRELIMINARY ANALYSIS OF FAILURES

It seems unlikely that conventional safety factors can ever be specified for structures as complex as lined underground excavations in jointed rock. There are too many uncertainties concerning the interacting modes of failure between the support and the surrounding rock mass.

A statistical analysis might at first sight appear to provide a promising approach. Ideally the analysis should incorporate the uncertainties in the input parameters and the uncertainties in the mathematical models of the failure modes. The theoretically optimum design could be determined based on the probabilities of failure in the different modes and on the costs of construction and failure. The end result would be superior to design based on conventional safety factors since paradoxically the designs having the highest safety factors might nevertheless incorporate higher probabilities of failure, as for instance shown by Höeg and Murarka (1974).

In underground excavation in rock, statistical design of this form is probably a very long way off, as we know almost nothing about the modes and mathematics of failure. Some engineers might object that we do know that shotcrete fails in shear, not compression, and that a rock mass behind the support will usually slide on pre-existing joints, unless retained by bolts. These are indisputable facts, but they help very little in actually formulating the mathematical analyses for general failure modes in a medium as variable as a rock mass. *It is therefore that we have at present to fall back on a classification method, where the design is based on precedent, and where a good classification method will allow us to extrapolate past designs to different rock masses and to different sizes and types of excavation.*

A valid objection to design based on precedent is that the general safety margin is virtually unknown. Very few failures occur and those that do can be so time dependent that it is difficult to be certain whether the "factor of safety" of the failed design

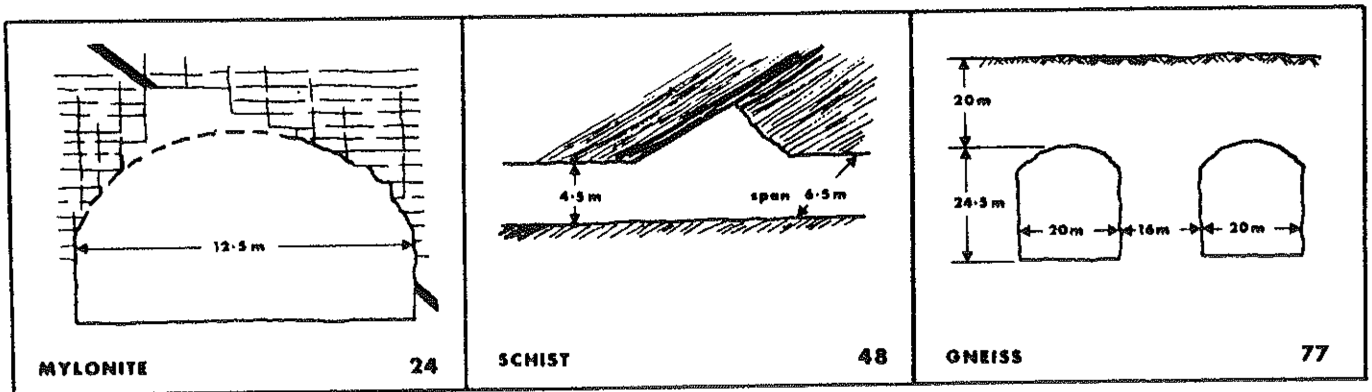

Figure 4. Sketches of three case records described in Table 12, after Cecil (1970).

Table 12. Comparison of support used and support recommended, for three 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	$\frac{SPAN}{ESR}$	Estimate of permanent roof support
						(Code : Tables 1 to 6)						
24	1. 60 m length, including a 1 m wide shear zone in mylonite. Chrushed mylonite and non-softening clay seams and joint fillings. Intersecting joint set. 2 joint sets plus random, 5-30 cm spacing. Minor water inflows (<3l/min). RQD = 60 2. Wedge shaped roof fall. 3. Headrace tunnel, Vietas Hydro, N.Sweden (ref. Cecil 1970).	12.5	6.5	60	Rock bolts, wire mesh and shotcrete	60/6 (1C/2E)	1.0/6 (3H/4K)	1.0/2.5 (5A/6C)	1.3	1.6	7.8	Category 22 =B 1 m +S(mr) 2.5-5 cm
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. RQD = 10 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 (1A/2B)	1.0/10 (3H/4O)	1.0/5 (5A/6B)	0.10	1.6	4.1	Category 31 =B 1 m +S(mr) 5 cm
77	1. 300 m length, massive gneiss, few joints. Planar, rough-surfaced, unaltered joints. 3 m spacing. Insignificant water inflow. RQD = 100 2. Minor overbreak, no falls or slides. 3. Wine and liquor storage rooms. Stockholm (ref. Cecil 1970).	20	24.5	18	50 spot bolts in about 300 m of chamber	100/1.0 (1E/2A)	5/1.0 (3E/4B)	1.0/2.5 (5A/6H)	200	1.3	15.4	Category 0,5 =None or sb

Note: Right-hand column "Estimate of roof support" is obtained from Tables 8, 9, 10 and 11.

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.

was 0.99, or considerably smaller in the long term. However, an attempt has to be made to investigate those case records describing preliminary failure and subsequent redesign that worked. Care must be taken to recognise the engineers reaction to failure. The redesign could be grossly conservative compared to general practice, or it could be a balanced redesign, depending on the confidence or otherwise of the engineers concerned.

Only six of the two hundred case records that were analysed contained useable descriptions of failure of the support that was first designed. Four of these records of failure unfortunately included no mention of design support pressures and therefore had to be analysed in the following way. The relevant value of SPAN/ESR was marked on Figure 2, and the support categories intersected by this line were searched by examining Tables 8, 9, 10 and 11 in order to find the support estimate identical to the one that failed. The corresponding rock mass quality was termed Q_o and was the initial *over-estimated* rock mass quality. The real rock mass quality Q obtained from correct classification was considerably lower. The ratio Q_o/Q is a measure of the *safety ratio* with respect to failure caused by incorrect rock mass classification.

Table 13. Apparent safety ratio when estimating Q.

Case record No.	*ESR*	Q_o	*Q*	*Safety ratio* (Q_o/Q)
18	1.6	0.37	0.0094	40
19	1.6	0.36	0.028	13
45	1.6	≧ 14	0.60	≧23
79	1.0	≧ 4	0.05	≧80

The two case records of failure that did include details of support pressures were described by Endersbee and Hofto (1963) and by Cording et al. (1972). In both cases the cause of bolt support failure was slabbing due to insufficient rock strength relative to the high in situ rock stresses*. Both cavities belong in the power station group with ESR = 1.0.

**The ratio of rock compressive strength/major principal stress (σ_c/σ_1) was from 2.1 to 2.5 for Poatina power station, (Endersbee and Hofto, 1963), and 1.5 for the Nevada test cavity (Cording et al., 1972). This places them in the "mild" to "heavy" rock burst categories according to Table 6, descriptions L and M. (SRF= 10 to 20.)*

The 13.7 m span Poatina power station described by the first authors had a design support pressure of 0.7 Kg/cm^2, which had to be increased locally (round the haunches) to 1.4 Kg/cm^2 by an overlapping 1 m pattern of 3.7 m long bolts. One of the Nevada test site cavities (hemispherical, span ca. 30.5 m) had a design support pressure of 0.35 Kg/cm^2 on the planar wall. The bolts yielded and failed when spaced at 1.8 m (yield pressure = 0.7 Kg/cm^2) and the design pressure was therefore increased locally to 1.4 Kg/cm^2 by an additional 200 bolts of 14.6 m length and 0.9 m spacing.

The two-fold and four-fold increases in support pressures described above for estimated rock mass qualities of 5.3 and 0.4 are equivalent to safety ratios (Q_o/Q) of approximately 5 and 40 respectively. The apparent correlation between support pressure and rock mass quality Q is discussed in the next section.

The *safety ratios* listed in Table 13 and those discussed above are clearly inadequate for drawing reliable conclusions. One might expect that excavations of the power station variety (ESR = 1.0) had inherently larger *safety ratios* than for instance pilot tunnels (ESR = 1.6). However, important excavations are usually more thoroughly investigated than small span tunnels, so the chance of a serious overestimation of Q should be minimal.

In general therefore, large values of safety ratios Q_o/Q are unlikely to be found *in case records of important excavations that failed.* However, the inherent over-design of important excavations unquestionably does ensure that there is more room for making errors in estimating Q, without actually bringing the inadequate support to failure.

In very approximate terms it would appear that over-estimating Q by a factor of about 5 to 10 (e.g. by failing to anticipate high rock pressure, or by failing to distinguish swelling clay from inactive clay) might perhaps result in failure of the support. Overestimation by a factor of about 30 might cause an even chance of failure. It is to be hoped that others will be able to improve upon these crude conclusions, so that safety can be better evaluated.

EFFECT OF ERRONEOUS EVALUATION OF Q

The problem of failing to anticipate unfavourable rock mass parameters, for example: slickensided joints, swelling clay, high rock pressure, squeezing ground, large water inflows etc. may cause individual errors ranging from factors of 1.5 to 2 up to a maximum of about 20. Two or more large errors out of the six parameters will be virtually certain of causing failure, if both errors are "unfavourable" (causing an overestimate of Q and an underestimate of support). However, there is room for several minor errors, especially since both "unfavourable" and "favourable" judgementsof the rock mass may be made, thereby balancing out to some extent. Total errors amounting to a factor of between 2.5 and 4 will be likely to change the support recommendation, since the "width" of most categories is of this order as can be seen from Figure 2. Smaller errors than this will only be reflected in slight adjustments to bolt spacing.

One of the most serious errors of engineering judgement that can be made is failure to anticipate a clay-filled weakness zone. This may have a "snowball error" effect on Q and therefore result in inadequate support, especially if the clay concerned is of the swelling variety. A hypothetical but realistic example is given below to illustrate this situation.

1. Assumed rock mass quality Q_0 = 70/9x1.5/3x1.0/1.0= 3.9 (POOR)
   Code to descriptions, Tables 1 to 6 (1C/2F, 3E/4D, 5A/6J)

2. Actual rock mass quality revealed upon excavation
   Q = 20/9x1.0/15x0.66/2.5 = 0.039 (EXT. POOR)
   Code to descriptions, Tables 1 to 6 (1A/2F, 3H/4R, 5B/6C)

According to the limited data of Table 13, a *safety ratio* (Q_0/Q) equal to 100, as above, will be virtually certain of causing failure in the unlikely event that support is not redesigned. The two Q values can be translated into engineering terms by imagining a water tunnel (ESR = 1.6) with both span and height equal to 9 metres. The two classifications given above lead to the following estimates for
a) permanent roof support
b) permanent wall support
c) temporary roof support
d) temporary wall support
(The method of estimating b, c and d is given in the appendix.

1. (a) Category 21 = S(5 cm)
   (b) Category 17 = S(2-3 cm)
   (Note: Q_w(wall) = 3.9x2.5)
   (c) Category 0 = NONE
   (d) Category 0 = NONE
   (Temporary support: 1.5 ESR, 5Q)

2. (a) Category 34 = CCA(sr) 35 cm
   +B(tg) 1 m
   Notes: VIII, XI
   (b) Category 34 = CCA(sr) 35 cm
   +B(tg) 1 m
   Notes: VIII, XI
   (Note: Q_w (wall) = 0.039x1.0)
   (c) Category 30 = B(tg) 1 m
   +S(mr) 5 cm
   Notes: VIII, XI
   (d) Category 30 = B(tg) 1 m
   +S(mr) 5 cm
   Notes: VIII, XI

The safety ratio of 100 in the above example is by no means the largest that can occur. For instance if the rock mass was essentially crushed in the weakness zone the safety ratio would exceed 200. However, it is a useful illustration of the "snowball error" that can occur through faulty engineering-geological judgement. **All six parameters can be altered unfavourabely by an unexpected clay zone.**

In conclusion it should be emphasised that sensitivity analyses of this type can be very informative for the design engineer since there is quite a large store of case records coded in Tables 8, 9, 10 and 11. The economic consequences of *pessimistic* assumptions of rockmass conditions can be compared with those resulting from *expected* conditions, and the consequences of individual parameter errors can be investigated. It may even be of value to investigate the economic consequences of changing the span of an excavation, if such a choice is available in the design.

SUPPORT PRESSURE ESTIMATES

Figure 5 shows an empirical method for estimating the permanent radial support pressure apparently required to stabilize the roof or walls of an excavation. The pressure to be expected for a given value of Q is likely to be dependent on the dilational properties of the weakest joint set, which is described by the J_r value. According to the limited number of case records available the range of support pressures to be expected generally lie within the shaded envelope. However, a closer estimate may perhaps be obtained from the following empirical relationships.

$$P_{roof} = \left(\frac{2.0}{3J_r}\right) J_n^{\frac{1}{2}} (Q)^{1/3} \qquad (3)$$

$$P_{wall} = \left(\frac{2.0}{3J_r}\right) J_n^{\frac{1}{2}} (Q_w)^{1/3} \qquad (4)$$

where
P_{roof} = permanent roof support pressure in Kg/cm^2
P_{wall} = permanent wall support pressure in Kg/cm^2
J_r = joint roughness number
J_n = joint set number
Q = rock mass quality
Q_w = wall factor (= 5, 2.5 or 1.0xQ, see appendix)

Estimates of support pressure obtained from Figure 5 are identical to those obtained from equation 3 and 4 when there are exactly three joint sets, which is the limiting case for three-dimensional block movement. If there are a greater number of joint sets the support pressure is likely to increase. Equations 3 and 4 are weighted accordingly. (The reasons for ignoring excavation dimensions when estimating support pressures have been discussed fully by Barton et al. (1974b) and will not be repeated here).

It will be found that the support pressure estimates obtained from Figure 5 (or equations 3 and 4) are reasonably consistent with the range of support measures listed in Tables 8, 9, 10 and 11. However, when the rock mass quality Q is higher than about 100, the estimate of pressure obviously loses its meaning, since excavations are almost certain to be self-supporting, with the exception of occasional blocks that require spot bolting.

The proposed relationship between support pressure and rock mass quality provides a convenient means for developing classification rules for dynamic as well as static loading of underground excavations. The

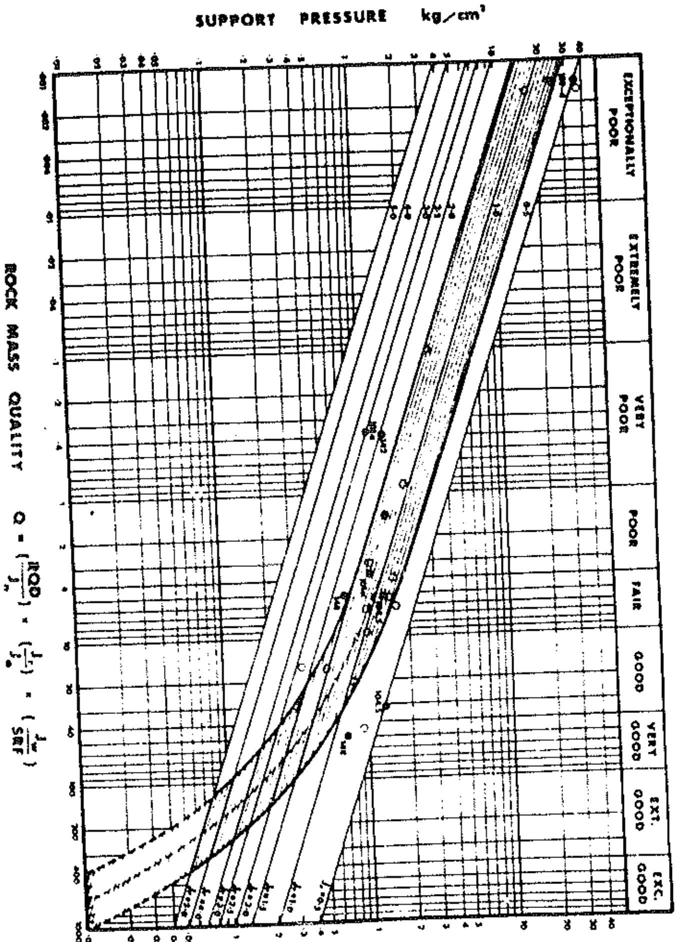

Figure 5. Empirical method for estimating permanent support pressures. Numbered points refer to case records described by Barton et al. (1974a)

dynamic stresses resulting from the passage of seismic waves will presumably exceed the static stresses by some unknown factor. (An increase of up to 20% has been suggested in recent work reported by Glass, 1973 for the case of lined excavations).

An increase in support pressure can be allowed for in the rock mass classification. For example the *stress reduction factor* SRF (Table 6) could be doubled for the case of dynamic loading. This would reduce Q by 50% and thereby allow for a dynamic/static stress ratio of approximately 1.25. In some cases this would have the effect of changing the support category, and in all cases would lead to reduced bolt spacing.

CONCLUSIONS

1. The method of rock mass classification and support estimation described in this paper can be of great value in the planning stage, when knowledge of the rock mass is limited. Sensitivity analyses of the various parameters can be performed, and cost estimates can probably be given with a little more confidence than hitherto. At a later stage, when excavation is underway, the rock mass parameters can be updated and the classification method used as a continuous record of rock conditions and a guide to support requirements. Should better methods of support design be available during excavation, then obviously the support recommendations contained in this paper should be overridden. Engineering judgement must be used at all times to prevent the recommendations being followed blindly.

2. Engineers and geologists who are in a position to supply the authors with the necessary classification and support data from projects with which they are familiar could make a valuable contribution, enabling the updating and improvement of the support tables when sufficient new data has been received. This would be especially valuable in categories where the authors' data is sparse or non-existent, and where initial support failed.

APPENDIX

The support recomndations listed in Tables 8, 9, 10, and 11 were derived from the description of *permanent roof support* given in the numerous case records. The methods of estimating *permanent wall support* and *temporary support* that are summarised in this appendix are unlikely to give as reliable an estimate of support as that for permanent roof support. However, in the feasibility and planning stages, estimates of permanent wall support and temporary support also play a part in the cost predictions, so some form of support estimate is required. In the excavation stage of a project the estimates of roof support can continue to serve as a useful guide to actual practice. However, at this stage the less reliable wall support estimate should be critically reviewed. The temporary support will be largely in the competent hands of the engineer in charge at the face.

1. *Permanent wall support*

An approximate rule of thumb for estimating wall support in medium rock conditions is to use 1.5 times the roof bolt spacing (= approx. half the support pressure) and 2/3 times the thickness of roof shotcrete. However in difficult rock conditions the wall (and invert) support may need to be similar to that of the roof arch. Conversely, in very favourable conditions there may be no need for any general wall support. Exceptions to these general assumptions may be encountered in the case of high walls. Special support might be required to stabilise deep-seated wedges.

An empirical method of modifying the roof support estimates is to multiply the rock mass quality Q by a factor which ranges in value from 1 to 5. The resulting *wall factor* Q_w is used in place of Q for determining wall support from Figure 2 and Tables 8 to 11.

Range of Q	*Wall factor Q_w*
$Q>10$	$5.0\ Q$
$0.1<Q<10$	$2.5\ Q$
$Q<0.1$	$1.0\ Q$

The *equivalent dimension axis* of Figure 2 is evalauted in terms of the *total* excavation height for the case of wall support (HEIGHT/ESR ≠ wall height/ESR). The worked examples given in this appendix illustrate the above method.

2. *Temporary support* (feasibility and planning only)

The method of modifying the estimates of permanent support to take care of temporary support is to select a *support category* (box numbers 1 to 38, Figure 2) closer to the "no support" diagonal given in Figure 2. It has been found from trial and error that the following modifications to Q and ESR give reasonable estimates:

a) Increase ESR to 1.5x ESR
b) Increase Q to 5Q (roof arch)
c) Increase Q_w to $5Q_w$ (walls)

These factors are applied equally to both roof and walls such that any differences in the permanent roof and wall support will also be in operation for temporary support. The worked examples given in this appendix illustrate the method.

3. *Recommended bolt and anchor lengths*

Bolt and anchor lengths for permanent support 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 illustrated 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
	anchors	L = 0.40 B/ESR
WALLS:	bolts	L = 2 + 0.15 H/ESR
	anchors	L = 0.35 H/ESR

where

L = length in metres
B = span in metres
H = excavation height in metres
ESR = excavation support ratio

(Bolt lengths used as temporary support will usually be only loosely dependent on excavation dimensions. Lengths of between 1.5 and 3.0 metres seem to be used in many types of excavations).

4. *WORKED EXAMPLES*

Two hypothetical examples are now given to illustrate the various stages of the method outlined in this paper. It is assumed that estimations of permanent and temporary support are required for a machine hall of 20m span, and a tailrace tunnel of 9m span, both to be excavated in the same phyllitic rock mass. It is assumed that the estimates are required for the planning stage of a project. At this stage the following geotechnical information has been produced : surface mapping and bore core analyses, rock stress estimates, rock compression tests.

I. *Rock mass classification*

Joint set 1. strongly developed foliation likely to act as fully developed joint set

smooth, planar (J_r = 1.0)
chlorite coatings (J_a = 4.0)
ca. 15 joints / m

Joint set 2. smooth, undulating (J_r = 2)
slightly altered walls (J_a = 2)
ca. 5 joints /m

J_v = 15 + 5 = 20 RQD = 50 (Eqn. 2)

J_n = 4

most unfavourable J_r/J_a = 1/4

Minor water inflows : J_w = 1.0
Unconfined compression strength of phyllite (σ_c) = 400 kg/cm^2
Major principal stress (σ_1) = 30 kg/cm^2
Minor principal stress (σ_3) = 10 kg/cm^2
(these are the virgin stress levels)

(σ_1/σ_3) = 3

σ_c/σ_1 = 13.3 (medium stress) SRF = 1.0

Q = 50/4 x 1/4 x 1/1 = 3.1 (poor) (Eqn. 1)

II. *Estimates for 20m span machine hall*

(i) permanent support

type of excavation : machine hall B = 20m H = 30m
(ESR = 1.0) B/ESR=20, H/ESR=30

(a) ROOF Q = 3.1 : category 23 (Fig.2)

Table 9 : B(tg) 1.4m + S(mr) 15cm (Notes II, IV, VII.)

(b) WALLS Q_w = 3.1 x 2.5 : category 20 (Fig.2)

Table 9 : B(tg) 1.7m + S(mr) 10cm (Notes II, IV.

mean length of bolts and anchors :

(a) roof bolts 5.0m
anchors 8.0m
(b) walls bolts 6.5m
anchors 10.5m

(ii) temporary support

B/1.5xESR = 13.3, H/1.5xESR = 20

(a) ROOF "Q" = 3.1x5 : category 14 (Fig.2)

Table 8 : B(utg) 1.6m + clm (Notes I, III.)

(b) WALLS "Q_w"= (3.1x2.5)x5 : category 14 (Fig.2)

Table 8 : B(utg) 2.0m (Notes I, III.)

III. *Estimates for 9m span tailrace tunnel*

(i) permanent support

type of excavation : tailrace tunnel B = 9m H = 9m
(ESR = 1.6) B/ESR = H/ESR = 5.6

(a) ROOF Q = 3.1 : category 21 (Fig.2)

Table 9 : B(utg) 1.0m + S 2-3cm (Notes I.)

(b) WALLS Q_w = 3.1 x 2.5 : category 17 (Fig.2)

Table 9 : B(utg) 1.4m (Notes I.)

mean length of bolts :

(a) roof 2.9m
(b) walls 2.9m

(ii) temporary support

(a) ROOF "Q" = 3.1x5 : category 0 (no support)

(b) WALLS "Q_w" = (3.1x2.5)x5 : category 0
= (no support)

5. *COMMENTARY*

The numbered *support categories* given in Figure 2 are shaped like parallelipipeds and have "widths" in units of Q (i.e. 0.01 - 0.1, 4 - 10 etc.) and "vertical" dimensions in units of SPAN/ESR. For example, category 23 has the following "dimensions": Q= 1-4, SPAN/ESR = 8-24.

1. When the estimated support listed in Tables 8 to 11 advises a range of bolt spacings i.e. 1-1.5m or 1-2 meters, the specific value to be chosen (and it will only be approximate) will depend on the value of Q relative to the given range for that category. Considering the worked example II(i): Q = 3.1, range for category 23 = 1 - 4. Hence the choice of B(tg) 1.4m from the range 1.0-1.5m. The higher the rock mass quality the wider the bolt spacing. The value of SPAN/ESR need not influence this choice.
2. The choice of shotcrete thickness or cast concrete arch thickness from an estimated range i.e. S(mr) 10-15cm will depend on the value of SPAN/ESR relative to the given range for that category. Considering the worked example II(i): SPAN/ESR = 20,range of SPAN/ESR for category 23 = 8-24. Hence the choice of S(mr) 15cm (approx.) from the range S(mr) 10 -15cm. The larger the value of SPAN/ESR the thicker the shotcrete or concrete.
3. The lengths of bolts and anchors obtained from Appendix 3 should be coordinated with the recommendations given under Notes II or III. Thus for the roof, variable (intermeshed) bolt lengths of 3, 5, and 7m appear reasonable,while for the walls 5, 6.5 and 8m might be more appropriate. The recommendation for using long tensioned cable anchors (Note IV) is based on current practice in most excavations of more than 15 to 20m span. The efficiency of long anchors spaced as widely as 4 to 6m (Note VI) is perhaps open to question as a general method of excavation support.
4. The relevant category for wall support is found by plotting the *equivalent dimension* HEIGHT/ESR versus Q_w in Figure 2, instead of SPAN/ESR versus Q. However, the *conditional factor* SPAN/ESR that is occasionally listed in Tables 8 to 11 is still used to differentiate between possible wall support alternatives, assuming that the other two *conditional factors* (RQD/J_n and J_r/J_a)are inapplicable.
5. The approximate estimate of temporary support is obtained by plotting SPAN/ESR versus 5Q for roofs, and HEIGHT/1.5xESR versus $5Q_w$ for walls. The *conditional factor* is SPAN/1.5xESR for temporary roof and wall support, assuming that the other two *conditional factors* are inapplicable.

Some engineers may prefer to modify the estimates of *permanent roof support* themselves, to obtain wall support and temporary support estimates, instead of following the worked example and notes 4 and 5 above. In all cases engineering judgement should be used so that the estimates of support are not applied blindly

For example, it is possible to point out at least one exception to the general rule that temporary support need have only limited capacity compared to permanent support. In rock bursting situations the temporary bolting should have at least equal capacity to that of the permanent bolting. The case of Siso power station that was described by Selmer-Olsen (1970) is a useful example.

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