Nr 23

NBatum

Föredrag och diskussioner vid BERGMEKANISKT DISKUSSIONSMÖTE I STOCKHOLM 10 FEBRUARI 1976

Papers presented at Rock Mechanics meeting in Stockholm 10 feb. 1976

Stiftelsen Bergteknisk Forskning - BeFo

Swedish Rock Mechanics research foundation

STOCKHOLM 1976

Jag har inte själv använt detta klassificeringssystem i praktiken och kan själv se en hel del svårigheter ifråga om tolkning av instruktioner, tillämpning m.m.

Min avsikt har dock endast varit att här presentera eller informera om detta arbete i hopp om att få konstruktiva synpunkter och kritik.

Den eller de som har synpunkter på detta klassificeringssystem är mycket välkomna att vid tillfälle kontakta antingen Carl-Olof Morfeldt, Hans Helfrich eller mig själv.

SUMMARY

A common deficiency of both geologic and geotechnical reports on particular rock mechanics problems as well as of published papers is the lack of an adequate and generally accepted means to transmit an overall assessment of the nature of the rock mass concerned to those who did not observe it. A language common to the experts from different fields who deal with rock mechanics must be made available.

For this reason the ISRM in its Council Meeting of October 3rd, 1971, in Nancy, decided to appoint a Commission on Classifications of Rock and Rock Masses.

The first task of the Commission was an exhaustive gathering of documents, in particular on the numerous classification systems which have been proposed. These documents were carefully studied, inasmuch as the Commission preferred to follow, as far as possible, classification criteria already in practice, rather than to innovate.

It was considered advisable to have, first of all, a "Basic Geotechnical Classification" of a general nature for rock masses; this classification is described in this document. In a preliminary form which is designed to be used as a "tentative standard", with the hope that sufficient data can be accumulated to permit adequate subsequent revision based on actual engineering experience.

The intention of this basic classification is to describe the various zones which constitute a given rock mass based on the available information coming either from outcrop observation, or from trenches, adits or boreholes. This classification may be used for written and oral description of rock masses or in borehole logs, trench logs and adit logs or even in maps and geological sections.

Classification especially devised for different types of engineering works, but obeying the Basic Classification, would be subsequently studied. Among these special classifications the Commission will consider the following fields of application: underground works, foundations, slopes, fragmentation and comminution. Another classification series will possibly be considered in which the type of geologic formation concerned will be taken into account.

UNSUPPORTED UNDERGROUND OPENINGS

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SUMMARY

Underground openings can often be left permanently unsupported. The decision of "support" versus "no-support" depends on factors such as the rock mass properties, the span width and the type of excavation. Since unsupported openings range in span from less than 2 metres up to 100 metres, it is clearly the rock mass properties that are of prime importance. The NGI "Q-system" of rock mass classification is used to analyse published case records describing unsupported spans, in an attempt to recognise those rockmass properties which seem to be essential if an opening is to be permanently safe, yet unsupported. Among the essential properties the following were noteworthy; there should not be more than three joint sets, the joints should have some degree of roughness or non-planarity, there should be no alteration or clay filling of the joints, there should be minimal or zero water inflow, and stress levels should be medium. The RQD is usually high, but exceptions (i.e. RQD = 40) are sometimes found.

These published case records are supplemented with rockmass descriptions and Q-classification of the famous Carlsbad limestone caverns of New Mexico, where natural spans ranging from 50 to 100 metres are found. The paper is concluded with a discussion of stand-up times for unsupported openings.

INTRODUCTION

A question that arises many times during the driving of tunnels in good quality rockmasses is the following: can this span remain unsupported, or must bolting or shotcrete be used to ensure permanent stability? The answer obviously depends on many factors, of which the following are probably most important; the rockmass properties, the span width, the type of excavation, and the method of excavation.

An idealized arch such as that illustrated in Figure 1 is a useful starting point for considering the influence of rock mass properties on tunnel stability. For example, if the joints between the blocks were interlocked and rough instead of smooth and planar, the arch would clearly be much more stable. For a block to fall out it would first have to dilate against its neighbours, thereby increasing

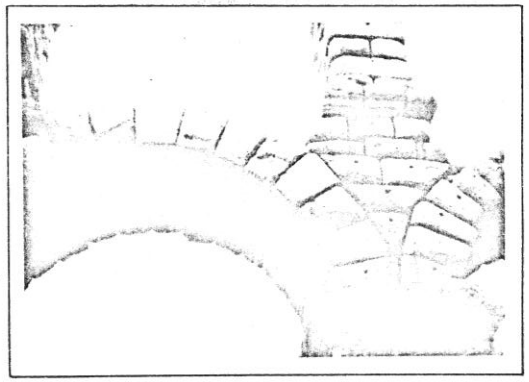

Figure 1. The importance of joint properties is illustrated by a classical Roman arch (Segovia, Spain).

the tangential stress and improving stability. Assuming the rock itself was strong enough, the arch might even be more stable if there was an increased load from overlying rock.

The effect of block-size and the number of joint sets is also of importance, if an arch is to remain permanently unsupported without the risk of block-falls. There is clearly a critical stage when the radius of curvature of an arch is sufficiently large compared to the block-size, such that tension and finally shear displacement can occur between adjacent rock blocks. The fact that *maximum* unsupported spans may range from less than 1 metre to more than 100 metres is proof of the huge range of rockmass qualities that are encountered in practice.

The influence of the above joint properties can be quantified by means of a recently developed rockmass classification system. This was developed through exhaustive analysis of approximately 200 case records of tunnels and large underground openings. Ninety of these cases were described by Cecil (1970) based on his detailed observations of tunneling projects in Sweden and Norway.

The method of rock mass classification has been described in detail by Barton, Lien and Lunde (1974, 1975). For convenience, the essential elements of the "Q-system" are given below. Potential users can then follow the analysis of unsupported openings and the associated rock mass requirements given here, without the need for immediate reference to the more detailed publications found elsewhere.

METHOD OF ROCK MASS CLASSIFICATION

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 rock mass description and ratings for each of the six parameters are given below. 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.

TABLE 1. ROCK QUALITY DESIGNATION (RQD)

A.	Very poor.....	0- 25
B.	Poor.....	25- 50
C.	Fair.....	50- 75
D.	Good.....	75- 90
E.	Excellent.....	90-100

Note:

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

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

TABLE 2. JOINT SET NUMBER

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

Note:

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

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

TABLE 3. JOINT ROUGHNESS NUMBER

		(J_r)
	(a) Rock wall contact and (b) Rock wall contact before 10 cms shear	
A.	Discontinuous joints.....	4
B.	Rough or irregular, undulating.....	3

Note:

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

TABLE 3, cont.

		(J_r)
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
	(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)

Note:

(ii) J_r = 0.5 can be used for planar slickensided joints having lineation, provided the lineations are favourably orientated.

(iii) Descriptions B to G refer to small scale features and intermediate scale features, in that order.

TABLE 4. JOINT ALTERATION NUMBER

		(J_a)	ϕ_r (approx.)
	(a) Rock wall contact		
A.	Tightly healed, hard, nonsoftening, impermeable filling i.e. quartz or epidote.....	0.75	(-)
B.	Unaltered joint walls, surface staining only.....	1.0	(25°-35°)
C.	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.....	2.0	(25°-30°)
D.	Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20°-25°)
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°-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, <5 mm in thickness)...	6.0	(16°-24°)

TABLE 4, cont.

		(J_a)	ϕ_r(approx.)
H.	Medium or low over-consolidation, softening, clay mineral fillings. (continuous, <5 mm in thickness)...	8.0	(12°-16°)
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°-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).....	6.0, 8.0 or 8.0-12.0	(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).....	10.0, 13.0 or 13.0-20.0	(6°-24°)

Note:

(i) Values of $(\phi)_r$ are intended as an approximate guide to the mineralogical properties of the alteration products, if present.

TABLE 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
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

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. STRESS REDUCTION FACTOR

		σ_c/σ_1	σ_t/σ_1	(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 ≤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 ≤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.			
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

Note:

(i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do notintersect the excavation.

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

TABLE 6, cont.

		(SRF)
	(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:

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

ADDITIONAL NOTES ON THE USE OF THE CLASSIFICATION METHOD

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

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).

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

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.

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 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 6(b).) A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in Note (ii); (Table 6(b).)

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 the Note 3 above. 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 potential threat to stability not only because of its 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 $J_r/J_a \leq 1$ 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 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.

EXAMPLES OF ROCK MASS CLASSIFICATION

Figures 2 and 3 illustrate two granitic rock masses which have significantly different jointing. The photographs are both of surface exposures, but for purposes of classification, imaginary tunnel depths of about 50 metres can be assumed. Medium water inflow and medium rock stresses will therefore be assumed in both cases. The classification of these two examples is given below. It will be noted that the positive contribution of irregular, undulating joints ($J_r = 3$) in case 2, gives this rock mass almost the same quality (Q) as in case 1, despite the greater number of joint sets and the lower RQD.

CASE 1. (Figure 2) GRANITE RQD = 90

Q = (90/9).(1.5/1.0).(0.66/1.0)

= 10 (fair/good)

Code: 1E/2F, 3E/4B, 5B/6J.

CASE 2. (Figure 3) GRANITE RQD = 70

Q = (70/15).(3.0/1.0).(0.66/1.0)

= 9.2 (fair)

Code: 1C/2H, 3B/4B, 5B/6J.

(The numerical and alphabetical coding refers to the listing of descriptions given earlier for each of the six parameters. This coding may be used for concise recording of rock conditions in routine tunnel mapping).

Figure 2. Example of three joint sets, with rough-planar joint surfaces.

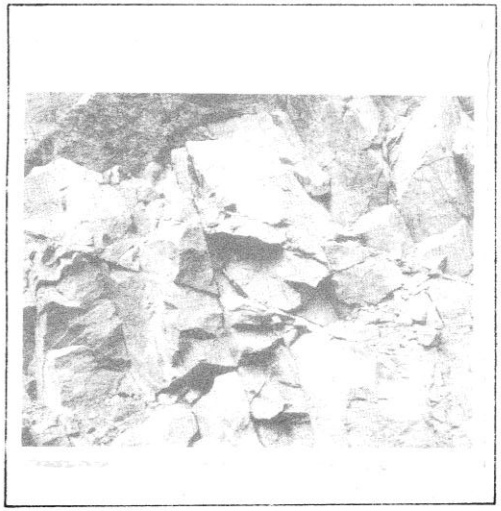

Figure 3. Example of four joint sets, with rough-undulating joint surfaces.

SPECIAL FEATURES OF THE CLASSIFICATION METHOD

The various ratings for the six parameters listed earlier 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 degree of 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.

For the purpose of analysing the unsupported cases it is convenient to summarize these features.

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.

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 arctan (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. As an example Table 7 shows values of arctan $(J_r/J_a)^0$ tabulated for the category "rock wall contact" (a). The descriptions of the various ratings of J_a (0.75 to 4) are given under the section listing the classification descriptions (see Table 4a).

TABLE 7. Estimate of Apparent "total friction angles from the Parameters J_r and J_a.

(a) ROCK WALL CONTACT	J_r	arctan $(J_r/J_a)^0$				
		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°

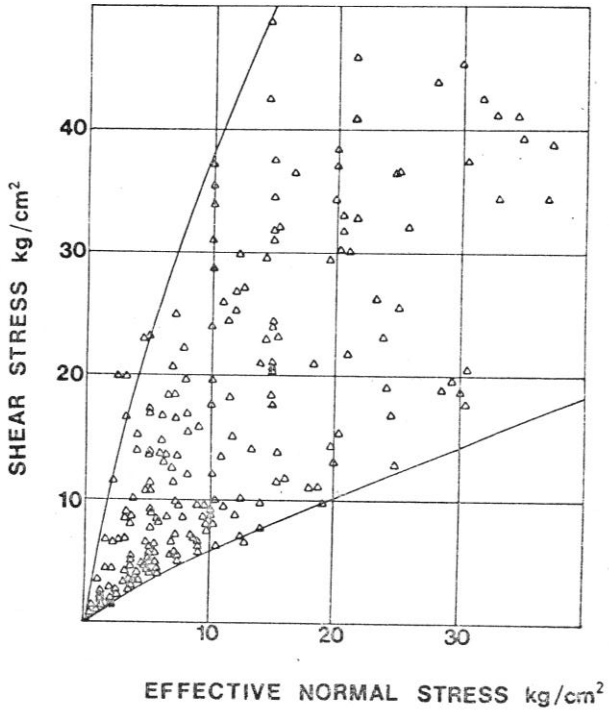

Figure 4. The peak shear strength of unfilled rock joints from the published results of direct shear tests performed in the laboratory and in situ (see Barton 1973 for references).

The above range of values of arctan $(J_r/J_a)^0$ for joints that are unweathered and unfilled ($J_a \leq 1.0$) is very similar to values of arctan $(\tau/\sigma_n)^0$ found in practice (see Figure 4). With increasing degrees of weathering, and softening of clay-mineral fillings, the "total friction angles" may reduce below 20^0, especially when clay fillings are thick or when the joint walls are smooth and planar. The great problem with clay filled joints in tunnels is that they may allow block movement to occur without the stabilizing effect of rock to rock contact and dilation. The degree of freedom increases as the ratio of wall roughness to filling thickness reduces (Barton, 1974), assuming that sufficient joint sets are present to form a discontinuous rock mass with clearly defined blocks.

The apparent "total friction angles" given in the top left hand corner of Table 7 obviously favour unsupported openings since they represent joints that are unfilled and that dilate strongly due to their non-planar surfaces. Values in the bottom right hand corner represent joints or discontinuities that are much less favourable since they are so smooth that even a thin coating of clay may allow block movement without rock to rock contact. It will be shown later that all but one of the unsupported excavations that have been analysed - some thirty cases - have values of $(J_r/J_a) \geq 1$, representing "total friction angles" $\geq 45^0$. This is probably the lower limit for dilatant surfaces, and it is obviously very significant for tunnel stability if no support is to be used.

THE TYPE OF EXCAVATION

Apart from the obvious influence of rock mass properties, there is another important variable - the type of excavation - that determines whether or not an excavation can be left unsupported. It is obvious that an unsupported span of 20 metres housing an underground railway station will require a greater degree of engineering safety than the same span housing a stockpile of broken ore near an underground crusher plant.

Table 8 lists six groups of underground openings. The number of case records that were analysed in each group are given in brackets. It was found that the given values of ESR - excavation support ratio - which range from 5 to 0.8 were convenient numbers for comparing support requirements of dissimilar types of excavation. The span width divided by ESR (SPAN/ESR) and the rock mass quality (Q) which is evaluated from six parameters, give a combined measure of the degree of support that is required. The higher the quality (Q) and the lower the value of (SPAN/ESR), the more likely that the given excavation can be left unsupported with acceptable safety.

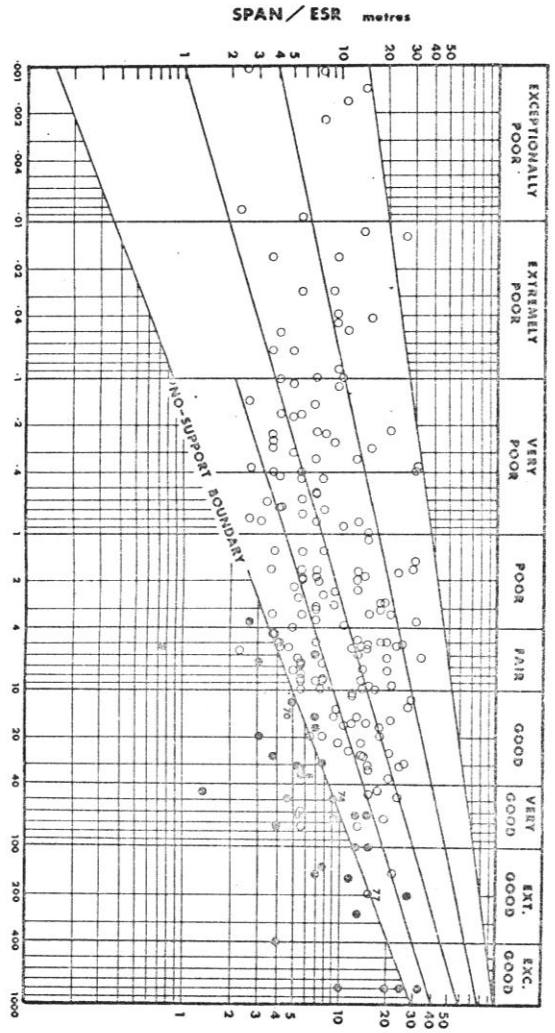

Figure 5. Analysis of case records indicates the approximate boundary between supported (o) and unsupported (•) excavations. The type of support, if any, depends on the rock mass quality (Q), the span, and the type of excavation (ESR).

TABLE 8. 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. (hemispher. 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, sports and public facilities, factories etc.....	ca. 0.8?	(2)

Figure 5 shows how the three variables are interrelated. The 38 open boxes were found by trial and error to be convenient divisions for distinguishing between the various types and degrees of support. (Details are given by Barton et.al., 1974, 1975). The great majority of case records plotting below the lower diagonal were found to be self-supporting. These will be analysed shortly.

EXAMPLES OF CASE RECORD ANALYSIS

Table 9 gives descriptions of four case records taken from Cecil (1970) that are self-supporting excavations or nearly so. Cases 74 and 77 requiring slight support plot exactly on the no-support boundary, as shown in Figure 5. Sketches of the four excavations, also taken from Cecil (1970) are reproduced in Figure 6.

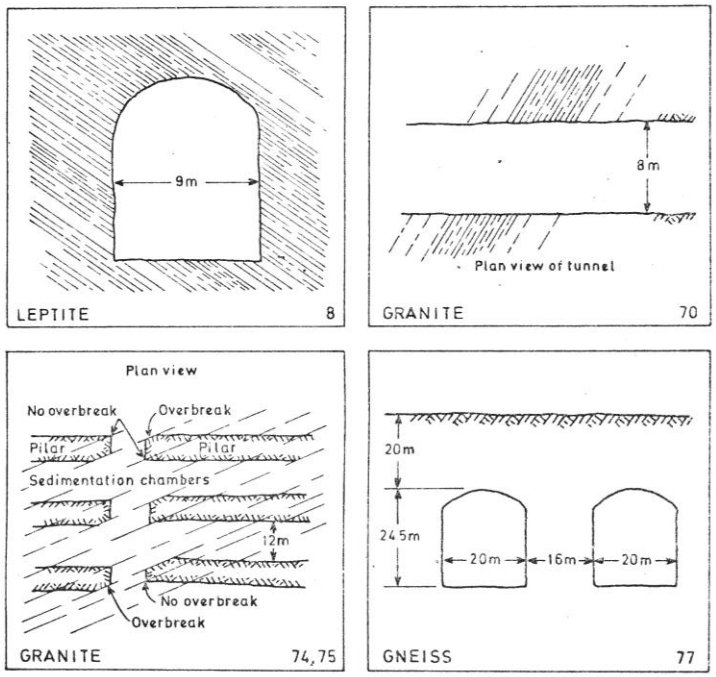

Figure 6. Sketches of the four case records described in Table 9 taken from Cecil (1970).

TABLE 9. Examples of case record analysis.

Case No.	1. DESCRIPTION OF ROCK MASS 2. Nature of instability 3. Purpose of excavation, location, reference	SPAN/ESR (m)	Height (m)	Depth (m)	Support used	RQD/J_n	J_r/J_a	J_w/SRF	Q
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 Seitvare Hydro. N.Sweden (ref. Cecil, 1970)	9/1.6	9	140	None	70/2	1.0/1.0	1.0/1.0	35
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. 2. Stable, minor overbreak, no roof falls. 3. Collector tunnel, Mo i Rana Hydro. N.Norway (ref. Cecil, 1970)	8/1.6	5.7	15	None	40/2	1.5/1.0	1.0/2.5	12
74	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. 2. No overbreak in chambers, but overbreak at intersections. 3. Waste water treatment plant, Käppala, Sweden (ref. Cecil, 1970)	12/1.3	12.5	≤100	None in chambers Bolts at intersections	100/2	1.0/1.0	1.0/1.0	50
77	1. 300 m length, massive gneiss, few joints. Planar, rough-surfaced, unaltered joints. >3 m spacing. Insignificant water inflow. 2. Minor overbreak, no falls or slides. 3. Wine and liquor storage rooms. Stockholm (ref. Cecil, 1970)	20/1.3	24.5	18	50 spot bolts in about 300 m of chamber	100/2	5/1.0	1.0/2.5	200

TABLE 10. Classification data for case records that plot below the no-support boundary.

Case No.	Description of support	RQD/J_n	J_r/J_a	J_w/SRF	SPAN/ESR	Q
8	none	70/2	1/1	1/1	9/1.6	35
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
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
68	none	100/½	5/1	1/1	10/1.0	1000
70	none	40/2	1.5/1	1/2.5	8/1.6	12
78	none	90/2	1.5/1	1/2.5	5/1.3	400
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
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
150	none	100/4	2/1	0.5/0.5	6.1/1.3	50
	Mean =	(87/2.9	2.7/1.1	0.9/2.0)		
6	S(1 app.) for protection from small stones	60/2	2/1	1/1	9/1.6	60
17	sb+S (1 app.) for protection from small stones	100/2	1.5/1	1/1	9/1.6	75
144	sb, 2 m long	90/4	1/4	1/1	3/1.3	5.6
	Mean =	(83/2.7	1.5/2	1/1)		

Key: S = shotcrete
B = systematic bolting
sb = spot bolting

VITAL STATISTICS OF UNSUPPORTED EXCAVATIONS

The classification data for the case records that plot within the no-support area in Figure 5 are summarized in Table 10. The three exceptional cases that are supported are given at the bottom of this table. Cases that plot above the no-support boundary - numbering approximately 190 cases - include only five exceptions; that is to say spans that were unsupported but which still plot outside the no-support area. The following general conclusions can be drawn concerning all the unsupported excavations:

1. A high RQD is most common (mean = 88%) but not without exception. One of the unsupported spans had RQD = 0 (nominal value = 10 for classification purposes).

2. One joint set plus random joints (J_n = 3) is the mean, although there are several unsupported excavations with three joint sets. In such cases the joints are rough or undulating.

3. The mean roughness of the joints (J_r = 2.7) signifies a condition between "smooth, undulating) and "rough or irregular, undulating". However, there are cases with "smooth, planar" joints (J_r = 1.0), but this characteristic is only present when there are two joint sets or less. In other words a rock mass having three sets of joints with one of the sets "smooth, planar" does not seem to be adequate for unsupported excavations. This is significant since three joint sets is the limiting case for three-dimensional rock blocks. Higher shear strength or increased dilatancy between the blocks is required in such cases.

4. Unaltered joints (J_a = 1) without any mineral coatings seem to be important prerequisites for unsupported excavations.

5. Minor or negligible water inflow (J_w = 1) is another important condition.

6. Rock stresses are mostly medium to high (tight structure, favourable for arching) but obviously not high enough to initiate rock bursting or squeezing conditions. The largest spans of all have high horizontal stress levels and very competent rock - the type of conditions which in mining can occasionally cause pillars to "hang" from the roof so that stability is improved by removing the pillars, despite increased span (Lunde, 1975.

It is constructive at this stage to examine the rock mass differences between the twentytwo unsupported spans plotting below the no-support boundary, and the supported spans which plot in the 8 boxes lying right next to the boundary. (See Figure 5). Forty eight cases plot in these support boxes. (Details of the types of support used are given by Barton et.al., 1974, 1975). They include various combinations of bolting and shotcrete.

The forty eight excavations are supported despite the fact that their mean span is 11.8 m compared to 16.9 m for the twenty two unsupported cases. The mean values of the six parameters used to classify the rock masses give the clue to the need for support. They are as follows:

$$RQD/J_n = 76/5.8,\ J_r/J_a = 1.6/2.2,\ J_w/SRF = 0.95/2.5$$

The lower RQD, increased number of joint sets, less rough and more altered or filled joints combine to make support necessary. The "block size" (RQD/J_n) falls from 30 to 13, and the apparent "total friction angle" (arctan J_r/J_a)° falls from 68° to 36°.

It is noteworthy that the three case records that plot below the no-support boundary but which nevertheless required support had on average similarly low "total friction angles", i.e. 37°.

Only one of the twenty nine unsupported cases had a "total friction angle" of less than 45°. This value is equivalent to unaltered, planar, smooth joints and probably represents the lower limit for dilatant joint surfaces. Dilatancy may be the single most important parameter controlling the stability of an unsupported excavation, assuming that sufficient joint sets are present to constitute a potential failure mechanism. Since no bolts or shotcrete are present to hold blocks in position the integrity of the rock arch may depend on the dowel effect of non-planar joints, especially if rock stresses are low as may be the case in near-surface excavations.

This section on rock mass parameters can be concluded by summarizing the conditional factors apparently required for safe performance of unsupported spans. The following list of conditions is based on a careful analysis of the rock mass parameters found in the case of unsupported excavations. (One or more of these conditions are usually *not satisfied* in the case of excavations requiring some form of support).

General requirements for permanently unsupported openings

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

Conditional requirements

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

In the last two sections of this paper the relationship between maximum span and rock mass quality Q will be considered, together with the question of stand-up time. If the "maximum" recommended span for a permanently unsupported excavation is exceeded, or if some of the above conditional factors are not satisfied the stand-up time may be less than "permanent".

MAXIMUM SPANS FOR UNSUPPORTED EXCAVATIONS

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

All the available case records of unsupported spans are plotted in Figure 7. The tentative curved envelope is the assumed *maximum design span* for *man-made* openings based on these available cases. The five square data points plotting above this curve were obtained from the huge *natural* openings of the Carlsbad limestone caverns in New Mexico. If the data for man-made and natural openings is combined, it is seen that the limiting envelope is linear. Until more case records are available this can perhaps be taken as representing the *limiting span* for permanently unsupported openings. It can be represented by the following simple equation:

$$\text{SPAN} = 2Q^{0.66} \qquad (3)$$

As will be seen later, for design purposes a more conservative approach is taken. The evidence of the natural caverns is in fact ignored. However, as

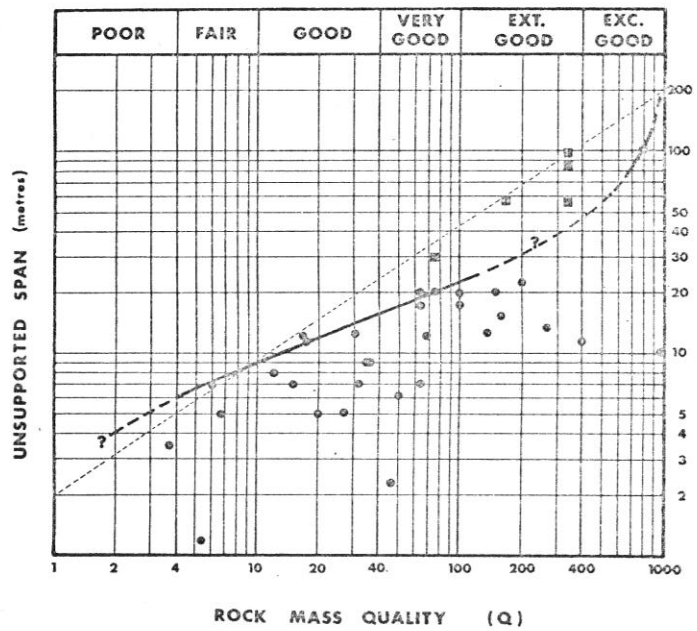

Figure 7. Circles represent the man-made unsupported excavations reported in the literature. Squares represent some natural openings from Carlsbad, New Mexico. The curved envelope is an estimate of the *maximum* design span for permanently unsupported man-made openings.

more information becomes available this conservative approach may no longer be justified. A description of the Carlsbad caverns is given here, in the hope of stimulating others to assess the rock mass quality Q of other large natural openings, so that this natural source of "case records" can be utilized more fully.

Analysis of the Carlsbad Caverns

Descriptions of the dimensions of Carlsbad caverns to be found in books on physical geology are somewhat misleading from a strictly engineering point of view. For example Holmes (1965, p. 422) describes the famous Carlsbad "Big Room" as being "nearly 4000 feet long, with walls over 600 feet apart, and a ceiling rising to a height of 300 feet". It is true that the trail round the perimeter of the "Big Room" is more than 1½ kilometers, all of it basically under one roof. However, the shape is anything but regular and it is only in the area known as the "Top of the Cross" that the effective span is at its maximum, in fact about 95 to 100 metres according to presently available plans.

Most of the caverns are situated some 200 metres below the surface in a massive limestone reef dating from the Permain period. Stratification or bedding is absent. However, closer to the surface the caverns are formed in lagoonal sediments. The extensive flat ceiling to be found close to the natural entrance to the caverns is in fact basically one huge bedding plane, spanning some 50 to 60 metres and extending at least 150 metres in an almost unbroken plane. An approximate cross-section of this room is given in Figure 8. The roof is about 50 to 60 metres below the surface. No vertical cross joints seem to be present. The rock mass was classified as follows:

$$RQD/J_n = 100/2,\ J_r/J_a = 1.5/1.0,\ J_w/SRF = 1/0.5$$

$$Q = 150$$

In the "Top of the Cross" area of the "Big Room" there is a dog-leg intersection between spans of 60 and 85 metres. At the intersection itself the maximum effective span of about 100 metres is found. Approximate cross-section are given in Figure 9. Two photographs of the roof area corresponding to cross-section BB are reproduced in Figure 10.

An important feature to be noted from these photographs is the vertical jointing running along the roof of this north-south oriented cave. There is a less strongly developed system at right angles to the first one. The two sets are in fact parallel and perpendicular to the original reef escarpment, and probably formed as a result of differential compaction (Bullington, 1968). The caverns formed by phreatic

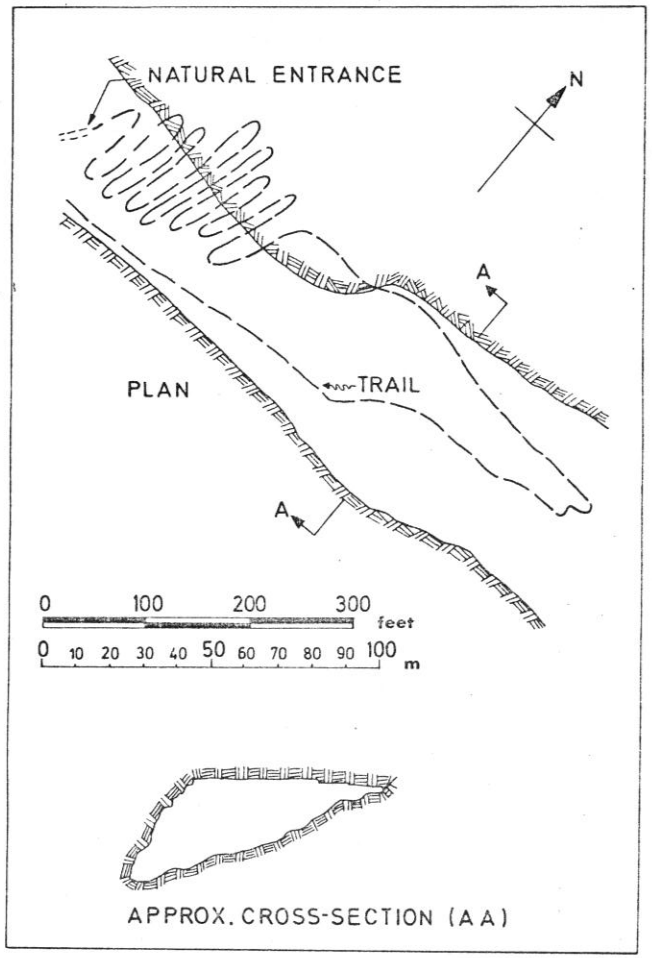

Figure 8. Approximate dimensions of one of the natural caves at Carlsbad, New Mexico, showing the flat bedding-plane roof.

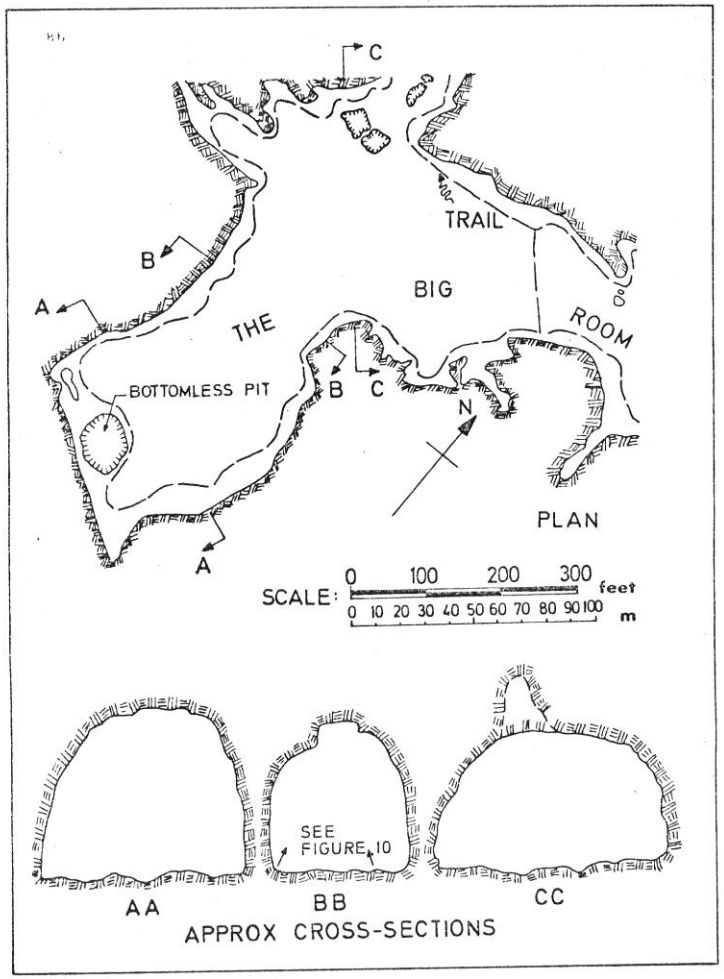

Figure 9. Approximate dimensions of some of the "Big Room" spans at Carlsbad, New Mexico.

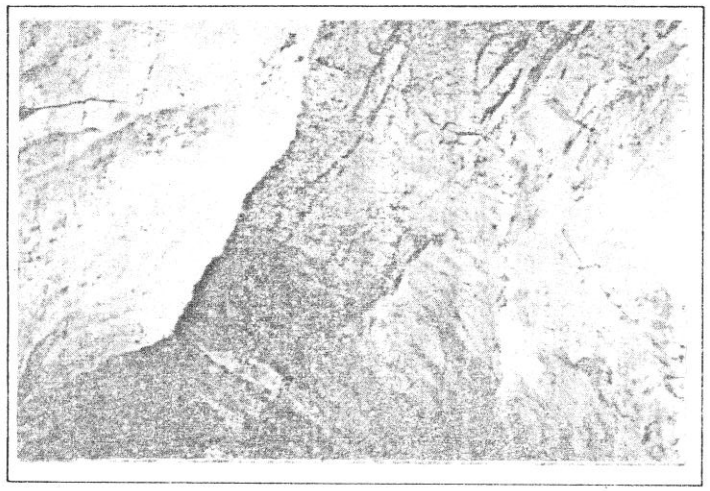

Figure 10. Photographs of the roof area (section BB, Figure 9) showing two vertical joint systems.) Carlsbad, New Mexico.

solution along the two joint sets, and later collapse of large masses, especially during re-flooding of the already formed caverns during the Pleistocene glaciation. This is one of the few parts of the caverns not decorated extensively with stalactites and stalagmites.

Since no bedding or stratification is present the three spans drawn in Figure 9 have been classified as having one joint set plus random ($J_n = 3$). The second set has such wide spacing that it probably has no more effect on the overall stability than a random joint. The rock mass was classified as follows:

$$RQD/J_n = 100/3,\ J_r/J_a = 5/1,\ J_w/SRF = 1.0/0.5$$

$$Q = 333$$

Returning to Figure 7 it will be seen that both the curved envelope representing *man-made* openings, and the dotted straight line envelope encompassing the *natural* openings suggest an "ultimate span" in the region of 200 metres in rock with Q = 1000. Such a rock might have the following classification parameters:

$$RQD/J_n = 100/0.5,\ J_r/J_a = 5/1.0,\ J_w/SRF = 1.0/1.0$$

For all practical purposes it would need to be intact, unjointed competent rock. If there were a few joints J_n would increase to 1.0, and SRF would need to be reduced to 0.5 representing "high stress, very tight structure" (Table 6b, K.)

STAND-UP TIMES FOR UNSUPPORTED EXCAVATIONS

The man-made openings which plot closest to the curved envelope in Figure 7 were the following types of excavations; 7 m span major road tunnel (slow lane for lorries), 8 m span water collector tunnel for hydro scheme (see case 70, Table 9, Figure 6), 11.2 m span tailrace tunnel for hydro electric station, 12 m span waste water treatment plant (see case 74, Table 9, Figure 6), 12.5 m span head race tunnel for hydro scheme, 20 m span mine opening (two cases), 22 m span subway station, 100 m span mine opening.

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

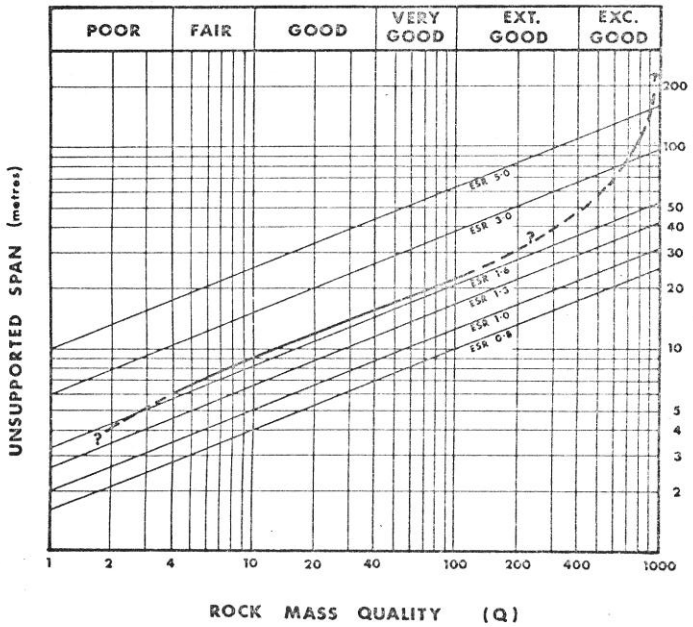

Figure 11. Suggested design limits for the various types of excavations listed in Table 8. ESR values greater than 1·6 apply to temporary openings, for which less stringent requirements concerning stand-up times are acceptable. Support may be required if spans in excess of the relevant design limits are excavated.

The closeness with which an unsupported opening can be designed to the envelope of *maximum design span* will depend on the type of excavation, the degree of safety and stand-up time required. In other words it will depend on the ESR value.

The no-support boundary drawn in Figure 5 appears to be the present design limit for man-made *permanently unsupported spans*. The span, Q and ESR values associated with this boundary can be obtained from the following relationships:

$$\text{SPAN/ESR} = 2Q^{0.4} \qquad (4)$$

$$\text{SPAN} = 2\text{ESR}\ Q^{0.4} \qquad (5)$$

When equation 5 is evaluated for various ESR values (see Table 8) the assumed safe limits for various types of unsupported excavations are obtained. These limits are drawn in Figure 11. The various lines are suggested by case record analysis as *present design limits*. The three lines representing excavation types ESR = 1.0, 1.3 and 1.6 are based on analysis of 79, 25 and 83 case records respectively. The vast majority of cases with spans exceeding these limits were supported in some way - for instance with various combinations of bolting, shotcrete and reinforcing mesh.

If these suggested limits for unsupported spans are exceeded in a given type of excavation this will clearly reduce the reliability of the given design. However, it seems unlikely that the stand-up time will be reduced to a significant degree (i.e. beneath the assumption of permanency) *unless the maximum envelope is exceeded*.

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

It has been assumed here that the excavations that plot closest to the curved envelope in Figure 7 (the *maximum design span*) have stand-up times in excess of 10 years. In view of the type of excavations involved it is obviously expected that they will stand unsupported for at least a "life-time", in other words more than 50 years. This conservative range of 10 to 50 years to represent "permanency" will now be used for interpreting the *maximum design span* of Figure 7.

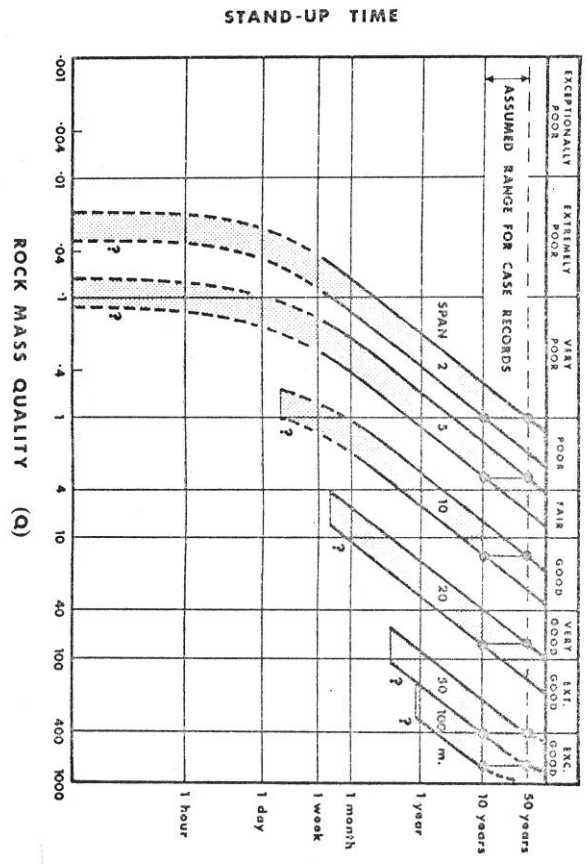

Figure 12. The envelopes represent a preliminary attempt at predicting how much the stand-up time reduces when the span of an unsupported excavation is increased beyond the maximum design span (Figure 7).

A preliminary attempt at correlating stand-up time, Q, and unsupported span is shown in Figure 12.

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

The actual inclination of the shaded zones drawn for various spans is unknown. In other words for a given span the relationship between stand-up time and rock mass quality is unknown. However, it seems quite likely that future case records will show that stand-up time reduces more abruptly and unexpectedly in the poorer qualities of rock. The shaded zones would then tend to bend down towards the vertical as suggested in Figure 12.

The value of case records of tunnels that failed due to inadequate stand-up times cannot be overemphasised. The tunneling profession is constantly asked to assess the "factor-of-safety" of a given design. If we are honest we have to admit that our present state of knowledge is inadequate to allow us to come anywhere near the correct value. For this reason the back-analysis of a failed length of tunnel; the stand-up time, the Q-value and the span, can give us some indication of how conservative our present designs really are. We have to start on the "simple" case of unsupported openings, before attempting to assess the "factor-of-safety" of supported excavations, with all the associated complications of rock-support interaction.

ACKNOWLEDGEMENTS

The research described in this report has been sponsored by the Royal Norwegian Council for Scientific and Industrial Research (NTNF). This support is gratefully acknowledged. The detailed descriptions of rock conditions in some Scandinavian tunneling projects given by Cecil (1970) provided an invaluable store of data for improving the classification method during its early development. The interest and practical advice given by Reidar Lien and Johnny Lunde of NGI is gratefully acknowledged. Finally the author would like to thank members of the Carlsbad Caverns and Guadalupe Mountains National Park Service for providing some physical and geological data concerning the Carlsbad Caverns.

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