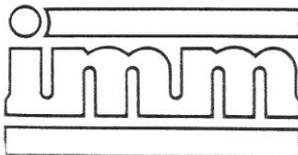


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Cavern design for Hong Kong rocks

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Abstract

The paper describes an integrated Q-system and discrete element design philosophy that can ensure the safe design of very large caverns in Hong Kong's excellent quality granites and welded tuffs. Principal cavern reinforcement methods should consist of fully-grouted rock bolts and fiber reinforced shotcrete. Predicted loading of reinforcement can be checked with numerical sensitivity analyses. The principal activities required to obtain the all-important input data for the empirical and numerical analyses will be described. These include stress measurement by hydraulic fracturing, cross-hole seismic tomography to identify fault zones and joint swarms, characterization of joints in drill core to obtain input for Q-system and discrete element (UDEC-BB) modelling, and follow-up mapping during construction to confirm designs.

Introduction

Slope stability problems in Hong Kong's weathered granites give a misleading picture of the potentially excellent rock qualities available for underground construction. Very large span caverns can be constructed at moderate cost to produce valuable additions to Hong Kong's high priced real estate. The especially favourable economy of large spans should be utilized to the full, to gain greatest benefit from the all-important area/volume ratio that favours minimum supported cavern surface area and maximum cavern volume.

How can one be so sure that large span caverns can be safely constructed and utilized in Hong Kong's granites and volcanics? The initial answer to this important question can be found in NGI's Q-system of rock mass classification and cavern support selection (Barton et al. 1974). Caverns of 20 to 30 m span have been successfully excavated and safely utilized in rock masses of equivalent quality to Hong Kong's granites and welded tuffs. In fact they have been successfully excavated and safely utilized in markedly poorer rock qualities than those available in Hong Kong's underground terrain.

The quality of Hong Kong's rock according to the Q-system

Case record statistics

More than 200 case records were utilized in the original development of the Q-system. Since that time NGI has designed almost 1000 km of tunnels and numerous large caverns based on this method. The level of precedent is therefore high, and it is apparently being added to by successful application in many other countries.

Figures 1 and 2 illustrate the statistics concerning excavation span and depth in the original 212 case records incorporated in the Q-system. It is obvious that the precedent for spans of greater than 15 m, and for depths in the economic 25 to 100 m range is quite solid. However, statistics of this kind do not tell us for instance, if spans of 30 m span can be safely constructed at 60 m depth. The routines developed within the Q-system will provide concrete answers to such uncertainties as our investigations progress.

The Q-system is based on a six-parameter scale of qualities which are listed in Table 1. The Q-value itself is based on the product of three pairs of these parameters which basically describe:

1. block size (RQD/Jn)
2. inter-block shear strength (Jr/Ja)
3. active stress (Jw/SRF)

If we take typical Hong Kong rock qualities for the granites we obtain:

$$Q \approx \left(\frac{90}{9}\right) \times \left(\frac{2}{1-2}\right) \times \left(\frac{0.7-1.0}{1}\right) \approx 7 - 20$$

Figure 3 illustrates where this quality lies in relation to the majority of the 212 Q-system case records. Occasional joint swarms, deeply weathered joints and faults will cause local reductions in Q and require individual treatment.

Preliminary support estimates

The method of support selection is based on a simple graphical routine. The dimensions of the opening and the Q-value are plotted on a diagram containing 38 separate "boxes" representing specific support recommendations. The support recommendations appear in simplified format in Figure 4.

Our typical "Hong Kong quality" of $Q \approx 7$ to 20 suggests that systematic bolting will be the principal form of arch support in a 30 m span cavern. However, the Q-system support tables inform us in more detail that a superficial (thin) layer of mesh reinforced (or fiber reinforced) shotcrete should also be used to supplement the systematic bolting, if the "block size" factor (RQD/Jn) is less than or equal to 10.

We can specify our preliminary cavern support as follows:

Arch: B 1.5 - 1.75 m
 S (mr) 100 - 150 mm or S(fr) 50 mm - 100 mm

where B represents systematic bolting of given spacing
 (mr) represents mesh reinforcement
 (fr) represents fiber reinforcement

Recommendations for wall support will depend strongly on cavern height and will be discussed later.

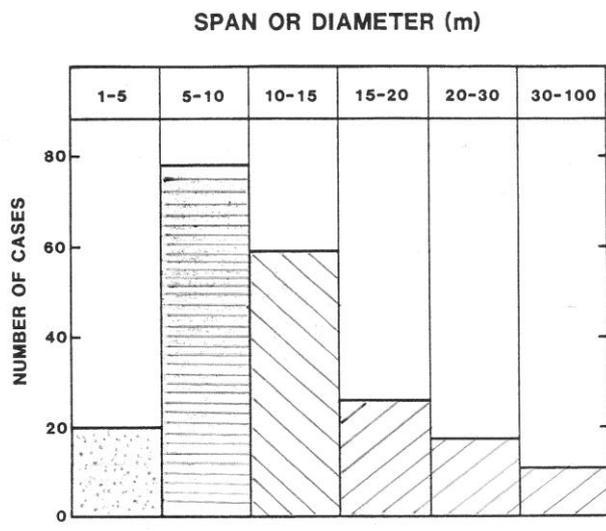


Figure 1. Q-system case records include nearly 60 caverns of span larger than 15 m.

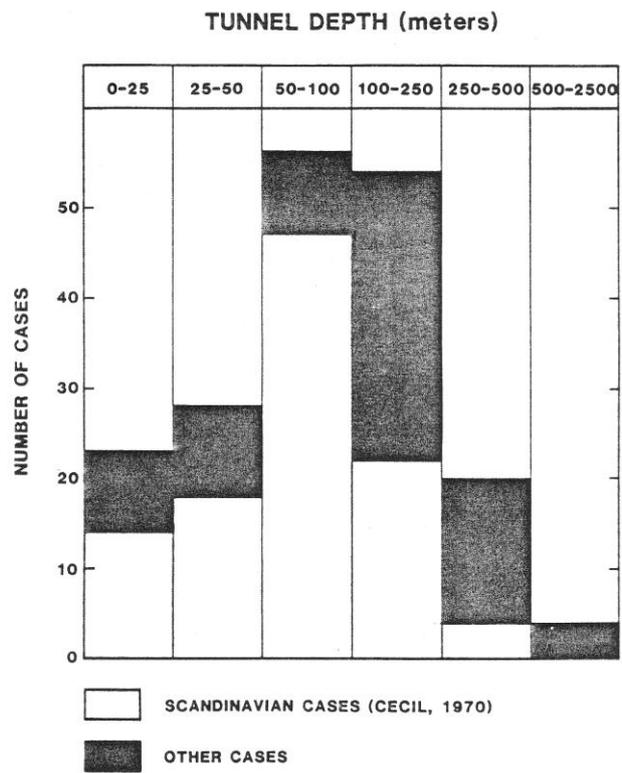


Figure 2. Q-system case records include more than 80 excavations in the depth range 25 to 100m.

Table 1. Descriptions and ratings for the six parameters.

1. ROCK QUALITY DESIGNATION (RQD)

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

Note: (i) Where RQD is reported or measured as ≤ 10 , (including 0) a nominal value of 10 is used to evaluate Q in equation (1).
(ii) RQD intervals of 5, i.e. 100,95,90, etc. are sufficiently accurate.

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 $(2.0 J_n)$

3. JOINT ROUGHNESS NUMBER

(a) Rock wall contact and (b) Rock wall contact before 10 cm 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

4. JOINT ALTERATION NUMBER (J_a) (e_j)

(a) Rock wall contact (approx.)

A. Tightly healed, hard, non-softening, impermeable filling i.e. quartz or epidote	0.75	(-)
B. Unaltered joint walls, surface staining only	1.0	(25-35')
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 cm 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 < 5 mm thickness)	8.0	(12-16')
J. Swelling clay fillings, i.e. montmorillonite (continuous, but < 5 mm 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. Zones or bands of disintegrated or crushed rock and claylike G, H, J for description of clay 6, 8, or 8-12 (6-24')

N. Zones or bands of silty or sandy-clay, small clay fraction (non-softening) .. 5.0 (-)

O, P. Thick, continuous zones or bands of claylike G, H, J for description of clay condition 10, 13, or 13-20 (16-24')

5. JOINT WATER REDUCTION FACTOR (J_w) Approx. water pres. (kg/cm²)

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.

6. STRESS REDUCTION FACTOR

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

A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B. Single weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤ 50 m)	7
C. Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m)	2.5
D. Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5
E. Single shear zones in competent rock (clay-free) (depth of excavation ≤ 50 m)	5.0
F. Single shear zones in competent rock (clay-free) (depth of excavation > 50 m)	2.5
G. Loose open joints, heavily jointed or "sugar cube" etc. (any depth)	5.0

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

(b) Competent rock, rock stress problems (σ_2/σ_1 σ_3/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 well stability)	10-5	0.66-0.33 0.5-2
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: (ii) For strongly anisotropic virgin stress field (if measured): when $5\sigma_1/\sigma_3 \leq 10$, reduce σ_2 and σ_3 to $0.8\sigma_2$ and $0.8\sigma_3$. When $\sigma_1/\sigma_3 > 10$, reduce σ_2 and σ_3 to $0.6\sigma_2$ and $0.6\sigma_3$, where σ_u = 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
G. 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

ROCK MASS QUALITY (Q)

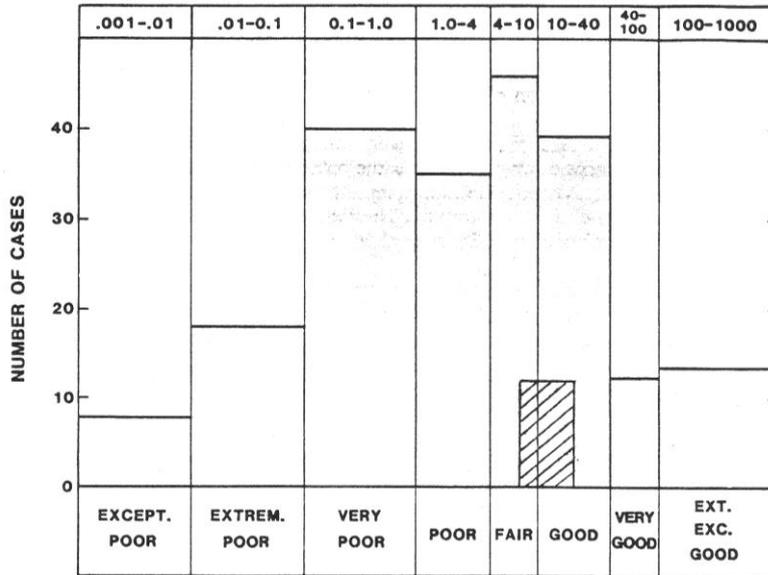


Figure 3. Q-system statistics for Q-values, indicate that much of Hong Kong's granites and welded tuffs lie in the "good" quality range of 10 to 40.

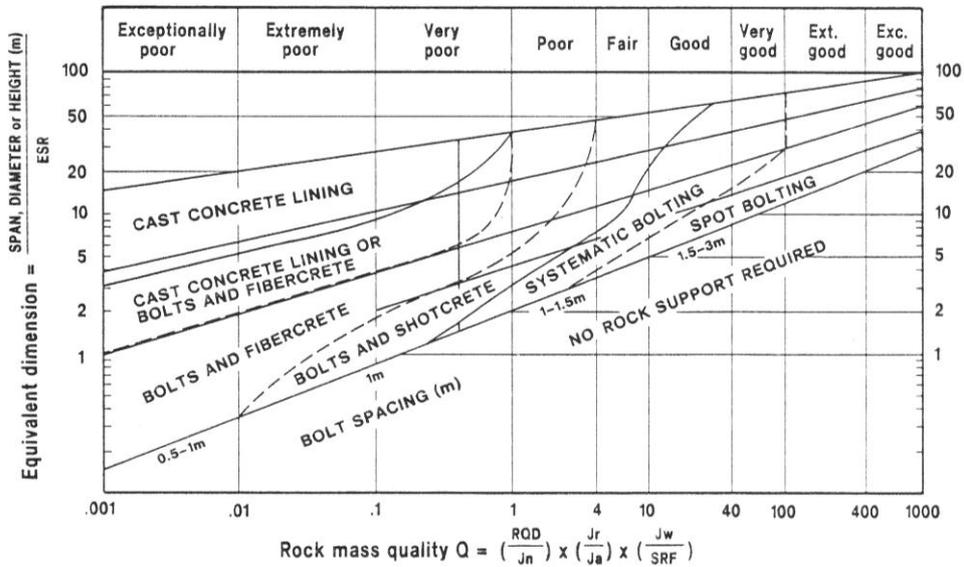


Figure 4. Principal rock support methods recommended in the Q-system. The support tables (Barton et al. 1974) should be referred to for details. After Grimstad et al. (1986).

Preliminary assessment of costs of excavation and support

We now have a rough idea of the principle rock reinforcement and cavern support methods that may be needed if Hong Kong granites and tuffs prove, upon closer examination, to have rock mass qualities in the region of $Q \approx 7$ to 20.

At this stage in an iterative approach to cavern design it may be appropriate to make preliminary assessments of whether 30 m is the optimum cavern size, with due attention to projected cavern uses. A hypothetical study, reproduced in Figure 5, was reported by Barton et al (1980) for the case of oil storage caverns. The 1980 prices (in Norwegian Kroner) need to be divided by five to obtain the US\$ equivalent.

Four hypothetical rock qualities were chosen in this cost estimate exercise. Site C happens to have a similar rock mass characteristic to our Kong Kong granites and tuffs which we are assuming have the following character:

RQD	= 90%	
Jn	= 9	(three joint sets)
Jr	= 2	smooth undulating joints
Ja	= 1-2	no or slight weathering or alteration (at 60 m depth)
Jw	= 1-0.7	no or slight water inflow problems (at 60 m depth)
SRF	= 1	no special stress problems (at 60 m depth)
		(i.e. σ_c / σ_1 in range 10 to 200)

$Q \approx 7$ to 20

It will be noted from Figure 5 that the cost of caverns of 24 m span appears to be approaching a minimum in our Site C rock quality but may involve a cost increase (per m³) if most of the quality is actually closer to 7 than 20.

Note that even in poorer quality rock (e.g. $Q = 2$, Site A) where clay coated jointing is prevalent, it appears to be economic to utilize large spans. Thus far in our iterative design of large caverns, it appears favourable to continue planning with 30 m spans.

Detailed investigations of bedrock at proposed site

It is probable that surface mapping of exposed bedrock in nearby slopes and building sites will give a good first impression of the general structure of the rock mass. Individual joint sets will have been identified by means of pole concentrations in stereographic nets (e.g. Hoek and Brown, 1980). Appropriately oriented core drilling will then be designed to extend the information on the various joint sets to greater depth, probably by use of both vertical and deviated holes.

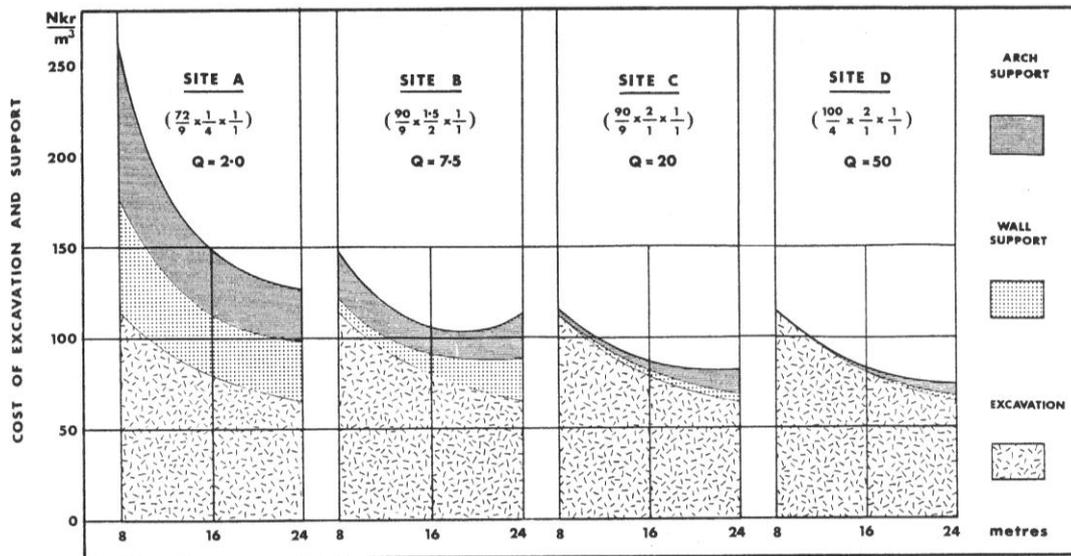


Figure 5. Relative costs of excavation, arch and wall reinforcement for storage caverns of 8, 16 and 24 m span. Barton et al. (1980).

A large amount of useful information can be obtained from the cores if the drilling is carefully executed.

- * dip and strike of most joints
- * spacing of each joint set
- * verbal descriptions of joint surface roughness, coatings and infillings
- * tilt tests for JRC (roughness) (Barton and Choubey, 1977)
- * Schmidt hammer tests for JCS (wall strength, same ref.)
- * tilt tests for ϕ_b (basic friction angle)
- * RQD, Jr, Ja; estimates of Q (Barton 1976)
- * unconfined compression tests of prepared cylinders (σ_c)

Each of these measurements or estimates will be used to complement or update the data obtained from surface investigations.

The boreholes themselves should be used for the following fundamental investigations:

- * televiewer surveys of borehole walls
- * systematic Lugeon testing (3 to 5 m packer spacing)
- * cross-hole seismic tomography
- * rock stress measurement (σ_1) and (σ_3) using hydraulic fracturing

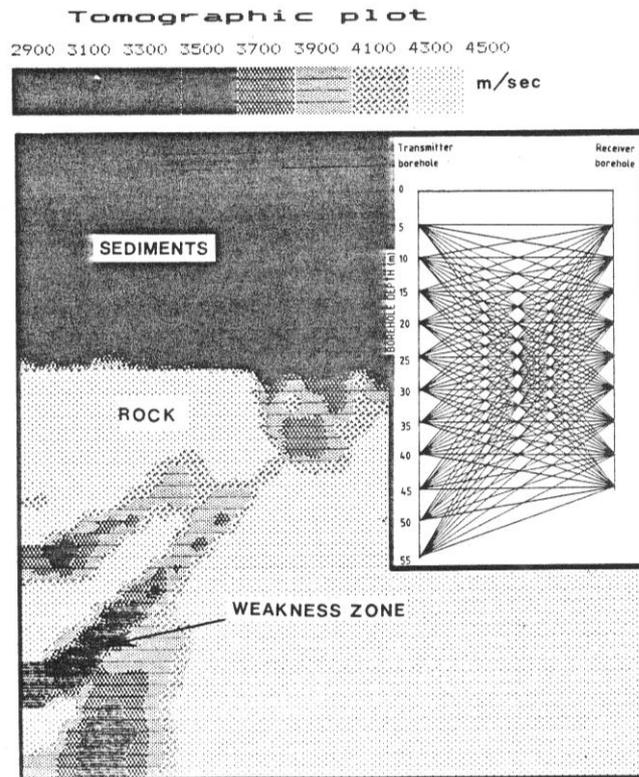


Figure 6 Cross-hole tomography for mapping and characterizing weakness zones and faults between pairs of boreholes (By, 1987).

The zones of poor core recovery will indicate where greatest use can be made of televiewer surveys. The three-dimensional structure of weakness zones and faults can be inferred from cross-hole seismic tomography (By, 1987). An example of one result from the investigation for the 13 m span Oslo motorway tunnels is shown in Figure 6. The result shown represents the rockmass in the plane between two of the five boreholes drilled from the surface in down-town Oslo. Cross-hole seismic tomography was subsequently utilized ahead of one of the tunnel faces for a particularly critical fault crossing, where overburden consisted of decomposed alum shale and marine clays. Appropriate adjustments to the velocity contouring interval (for example to 50 m/second intervals) allow the user to see the trends of joint sets and other finer features of the rock mass.

Rock mass classification statistics

Core logging should include the identification of specific variations in rock type and joint structure, with special attention to faulted material or sections of core loss (if encountered). A zoning of the rock mass should include engineering terms, such as the four Q-system parameters statistically represented in Figure 7. The engineering geologist should indicate the inter-relationships between the predominant rock qualities, by using symbols (such as A: most common 85% and B: infrequent 10%).

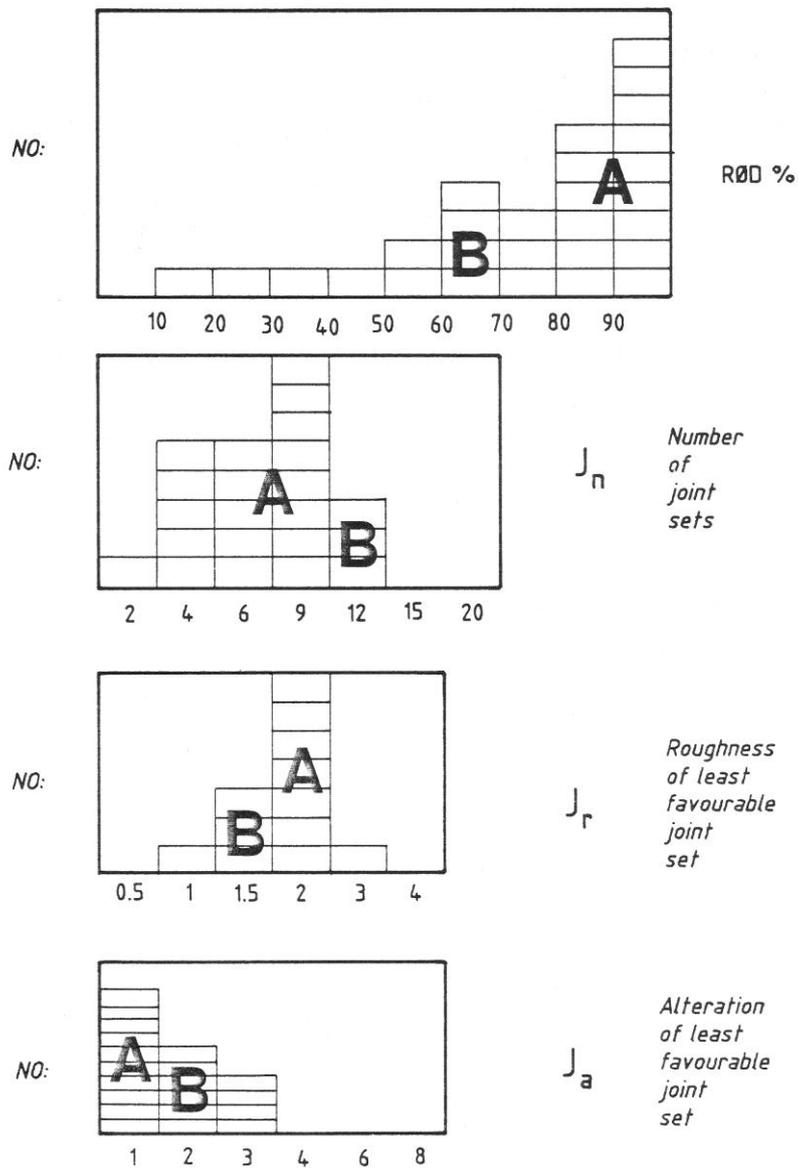


Figure 7. Preliminary Q-system statistic for granites and welded tuffs in Hong Kong's Quarry Bay and Mt. Davis.

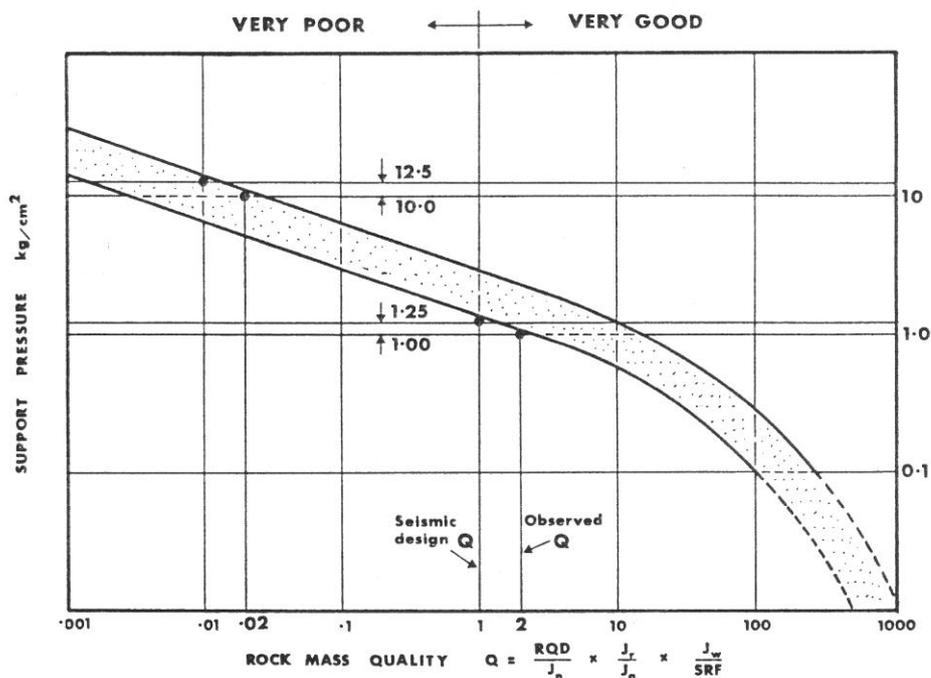


Figure 8. Support pressure diagram with allowance for seismic loading.

Intergration of support type and support pressure

The preliminary support recommendations specified bolt spacing and shotcrete thickness, but not bolt capacity. The shaded region shown in Figure 8, envelopes most of the designed or measured support loads in the Q-system case records. A basic trend of increased support capacity with reduced rock quality is of course noted.

An additional feature of the diagram, which the author has utilized in seismically active regions (e.g. Taiwan, Peru) is the provision for a 25% increase in designed support capacity by selecting a less favourable SRF stress category than that utilized in the "static" case. This conservative approach, where $SRF(\text{dynamic}) = 2 \text{ SRF}(\text{static})$, provides for a 25% increase in bolt (or anchor capacity) and may result in a "leftward" shift into a more conservative support category in Figure 4.

The 25% increase in pressure is a conservative allowance for the maximum 10 to 20% increase noted in FEM continuum analyses when comparing dynamic and static stress levels in cavern models subjected to dynamic loading. A review of dynamic loading effects on tunnels and rock caverns nevertheless confirms the favourable nature of underground construction in seismically active regions (Barton, 1985). The combination of rock bolting and fiber reinforced shotcrete is also noted as an extremely favourable rock reinforcement technique for seismic design.

Our assumed Hong Kong rock mass quality Q of 7 to 20, qualifies for an arch support pressure of approximately 0.04 to 0.05 MPa or 4 to 5 tons per m. If we utilize the recommended bolt spacing of 1.5 to 1.75 m centre to centre, each bolt will be required to reinforce 2.3 to 3.1 m of rock surface. We are therefore looking at working loads per bolt in the range of 10 to 15 tons; a moderate figure reflecting the generally good quality of the rock. The recommended 50 to 100 mm of fiber reinforced shotcrete does little to supplement the support pressure per se, but has a very beneficial effect on the stability of superficially loosened material.

These reinforcement loads can be set in perspective by referring to a recent integrated Q -system design of a large power cavern in Taiwan which resulted in support pressures as high as 0.3 to 0.4 MPa (30 to 40 tons/m) for the extensively faulted rock. In this case Q values were locally as low as 0.15 to 0.3 ("dynamic" Q and "static" Q , respectively). These design values have proved appropriate for the extensive cable and bolt reinforcement, which was specifically oriented to resist seismic deformation of the dipping fault planes crossing the cavern. Measured deformations were large during construction but appear to have stabilized satisfactorily, as expected.

Similar routines to the above are followed when designing appropriate wall reinforcement. The height of the cavern, and the uses to which it will be put, are important variables in the selection of bolt spacing, length and load capacity (Barton et al. 1975).

Bolt lengths can be selected from the empirical relationships given by Barton et al. (1975). These fit accepted practice reasonably well:

Roof:	bolts	$L = 2 + 0.15 (\text{SPAN/ESR}) \text{ m}$
	anchors	$L = 0.4 (\text{SPAN/ESR}) \text{ m}$
Walls:	bolts	$L = 2 + 0.15 (\text{HEIGHT/ESR}) \text{ m}$
	anchors	$L = 0.35 (\text{HEIGHT/ESR}) \text{ m}$

For large caverns ESR will generally be equal to 0.8 to 1.0.

Construction follow-up

Once construction of access tunnels (or shafts) begins, the interpretations of geology and structure made from core logging and cross-hole seismic tomography can be updated. A field mapping scheme used extensively in Norway is illustrated in Figure 9. The example is a 160 m² tunnel excavation. Background (most typical) rock quality should be recorded as a matter of routine. Joint swarms, faults and marked clay-filled discontinuities should be mapped with special care, and may warrant individual support and reinforcement additional to the general design, or as a local alternative to the general design. The B + S rock reinforcement method is flexible, and should be used as such if best economies are to be achieved in the overall cavern support.

A collection of measured deformation data from Q-system case records is shown in Figure 10. The shaded envelope has been found to be a useful guide to interpret measurements during cavern construction. If our assumed "Hong Kong rock quality" of 7 to 20 is representative, we can expect arch deformations in a 30 m span cavern of no more than 5 to 20 mm, and wall deformations of about the same magnitude if 30 m high caverns are chosen.

Measured deformations lying outside the shaded envelope probably represent inadequate reinforcement if lying to the right, and over-conservative reinforcement if lying to the left. Classification parameters could also have been in error, i.e. due to a clay bearing discontinuity that was missed under follow-up mapping. Checking observations with precedent can therefore be an added security measure.

Permanently unsupported spans

In view of the good quality of Hong Kong granites and welded tuffs it is relevant to mention the potential use of permanently unsupported spans in Hong Kong. Q-system case records that plotted beneath the "no rock support required" diagonal in Figure 4, have been plotted in Figure 11, together with the large natural caverns in limestone at Carlsbad, New Mexico. Analysis of each of the 31 man-made openings reveal the following conditional factors for selecting or checking on the suitability of rock conditions, when leaving excavations permanently unsupported.

Conditional factors for improving the reliability of permanent unsupported spans.

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

Table 1 should be referred to when interpreting these conditional factors. Note that the #1 factors are "preferable" features, but they are not essential if other factors are favourable. The "preferable" list informs us that we should seek rock masses with:

- a) less than or equal to three joint sets
- b) joint roughness more favourable than "smooth, planar"
- c) no joint alteration or clay fillings
- d) no water inflow problems
- e) no special high or low stress problems

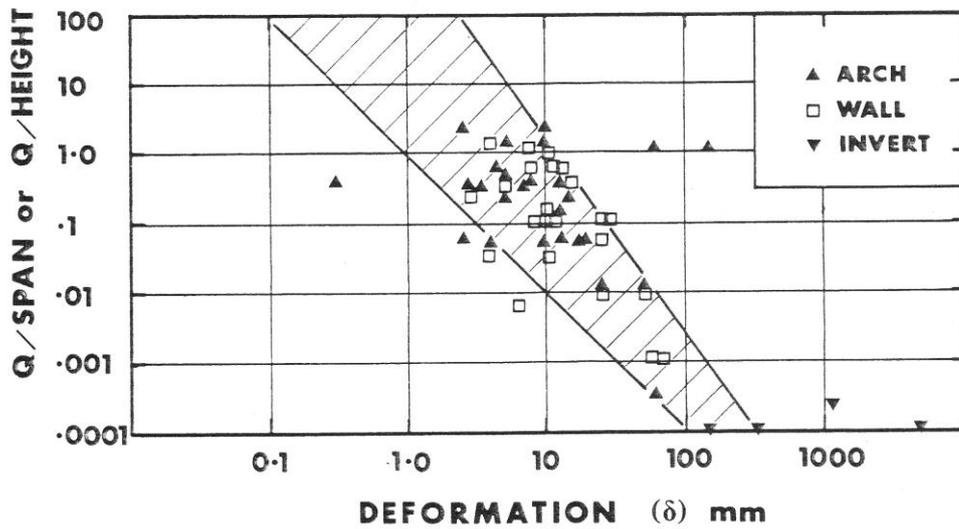


Figure 10. Q-system case records that included deformation measurements. (SPAN or HEIGHT in metres.)

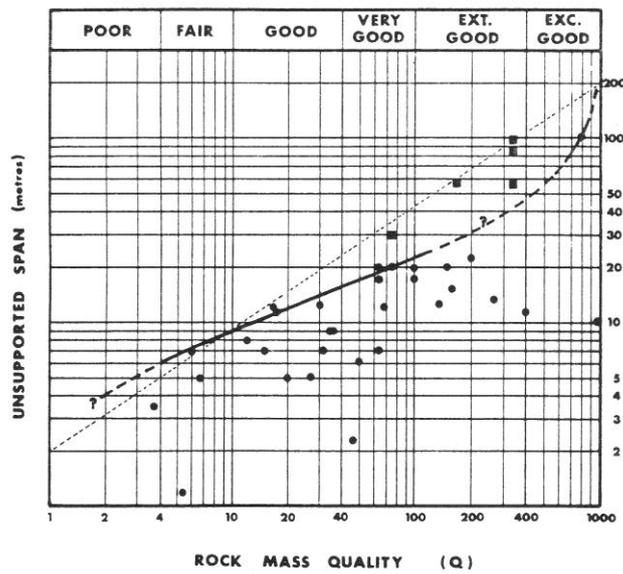


Figure 11. Permanently unsupported excavations represented in the Q-system case records.

The other conditional factors (#2 to #7) tell us among other things that if a) is not satisfied, spans should be limited to less than 10 m, that if b) is not satisfied then only one joint set should be present, that if e) is not satisfied joints with significant roughness will be required. Each of these conditional factors prove to have a sound mechanical basis, and can be demonstrated in numerical analyses with discrete element (jointed) programmes such as UDEC (Cundall, 1980).

Our assumed typical rock quality repeated here for reference:

$$Q = \frac{90}{9} \times \frac{2}{1-2} \times \frac{1-0.7}{1} \approx 7 \text{ to } 20$$

satisfied the great majority of the conditional factors #1 to #5 inclusive. However, it "fails" on factors #6 and #7 unless we limit the permanently unsupported span to less than 10 m. Reference to Figure 11 shows that the upper envelope of precedent actually lies between 8 m and 12 m for Q values ranging from 7 to 20. Permanent access tunnels larger than 10 m span should therefore be bolted locally.

Checking empiricism against numericism

The range of rock qualities that can be represented and "quantified" in an empirical design method such as the Q-system, is enormous. Until recent years, no computer programme has come in the neighbourhood of the needed sophistication, either in two or three dimensional analysis.

Discrete element models

The development of rigid block analyses by Cundall (Cundall et al. 1975) heralded the arrival of a discrete element analysis with deformable blocks called UDEC, (Cundall, 1980). More recently a three-dimensional version (3DEC) has been developed by Cundall, and is beginning to be utilized with simplified representation of joint properties (Hart et al. 1988).

Use of two or three dimensional discrete element analyses represents a giant step towards reality, after many years of over-reliance on unrealistic elastic, isotropic continuum analyses, with their unrepresentative input data.

The presence of joints surrounding a tunnel or large excavation causes a quite different stress redistribution than that calculated in elastic continuum analyses. Joints deform in a subtle manner, and may close slightly, open, or shear. The latter is often accompanied by dilation, but this will usually cause normal stress build-up and will arrest further shearing. When no dilation occurs during shear (due to extreme joint planarity or clay filling) stability may be severely compromised and reliance on reinforcement will be increased.

Acquisition of input data for discrete element analyses

Some of the input data that is required for cavern design studies using the version of UDEC operated by NGI (termed UDEC-BB) is summarized in the left hand side of Figure 12. All of it can be obtained from logging of oriented core and permeability testing, if joint characterization and constitutive modelling follows the outline given below.

- Step 1. Representation of dip, and dip azimuth of joints, as pole concentrations on lower hemisphere stereographic projections.
- Step 2. Representation of joint spacing statistics for the identified sets of joints, for selection of representative block size (L_n).
- Step 3. Double packer testing of investigation boreholes using the Snow (1968) statistical method of interpreting conducting apertures (e) and their spacing (S) (α = dip, β = azimuth, e = conducting aperture, k = conductivity = $e^2/12$). UDEC-BB can also be operated with empirically estimated joint apertures, if permeability tests are unavailable.
- Step 4. Tilt testing of drill core joints (for JRC), Schmidt hammer testing (for JCS) and core stick tilt testing (for Φ_c) to provide shear strength statistics for the critical joint sets. Figure 13 shows the simple techniques that can be used to extract this data from the cores. Figure 14 shows examples of measured data, and non-linear strength envelope construction (Barton and Choubey, 1977). Median values of JRC, JCS and Φ_c are used to represent "typical" jointing. Extra reality can be obtained by incorporating lowest values of these three parameters with the joints of greatest persistence, and highest values with the joints of least persistence.
- Step 5. Data from step 4 are used as input in the Barton-Bandis constitutive model for joint behaviour, to generate shear stress-displacement ($\tau - dh$), dilation-displacement ($dv - dh$) and conductivity-displacement ($k - dh$) curves (Barton et al. 1985). These curves are block size (L_n) dependent, as demonstrated in the examples shown in Figure 15 (see inset).
- Step 6. Data from step 4 are also used to generate normal stress-closure and stress-conductivity data. An example of measured stress-closure behaviour for a rock joint is shown in Figure 16 (I). The combination of joint deformation components N and S represented in the three sketches of rock masses in this figure, result in the typically non-linear, hysteresis behaviour.
- Step 7. All the above features of joint and rock mass behaviour are incorporated in sub-routines in UDEC-BB. The problem geometry illustrated in step 7 (Figure 12) incorporates measured data from core logging, surface mapping and permeability testing of the shale-limestone site. Model stress boundary conditions were obtained from hydraulic fracturing stress measurements.

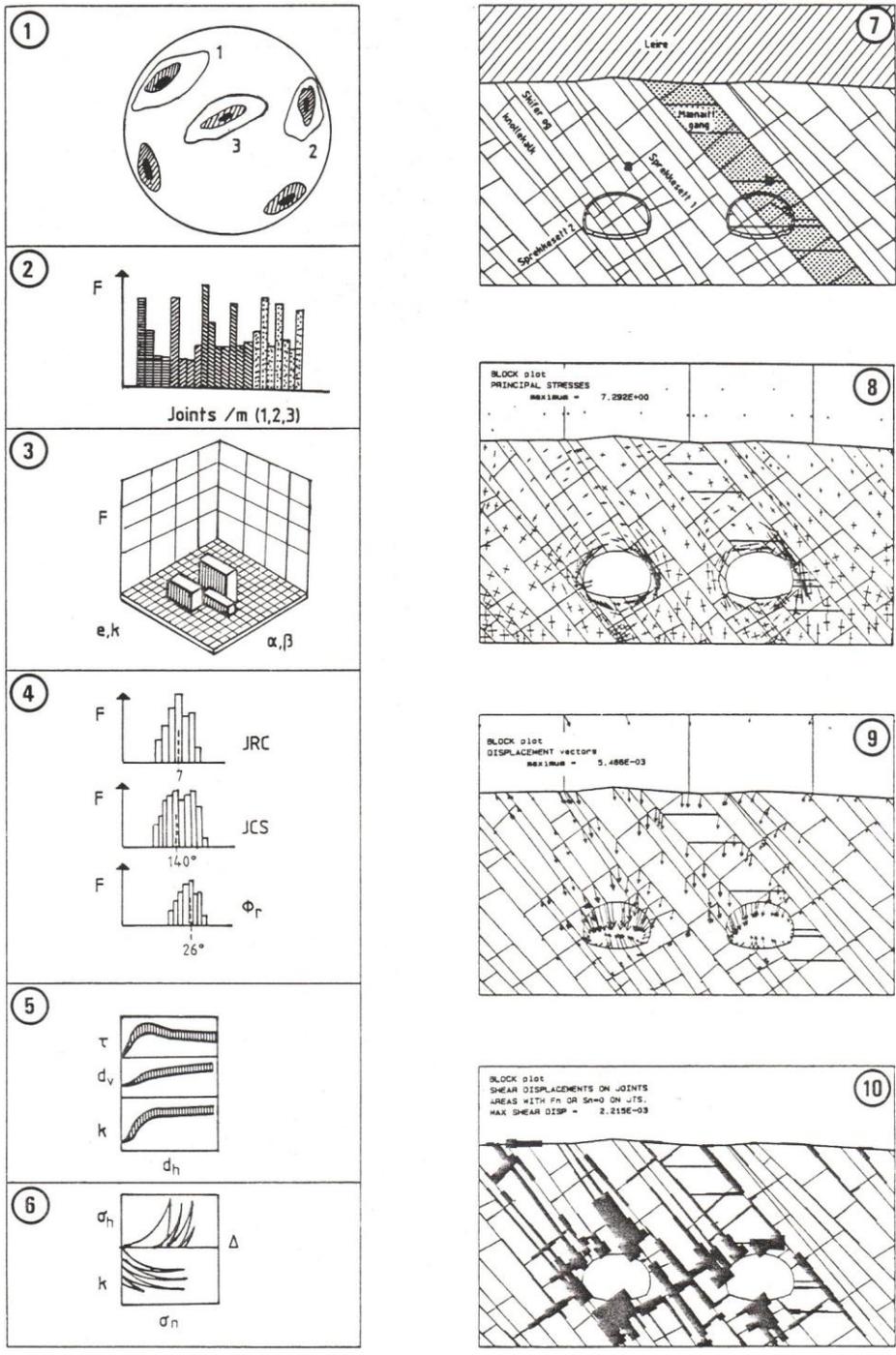


Figure 12. Acquisition and representation of input data to represent jointing (1 to 6) and execution of UDEC-BB analyses for twin tunnels (7 to 10).

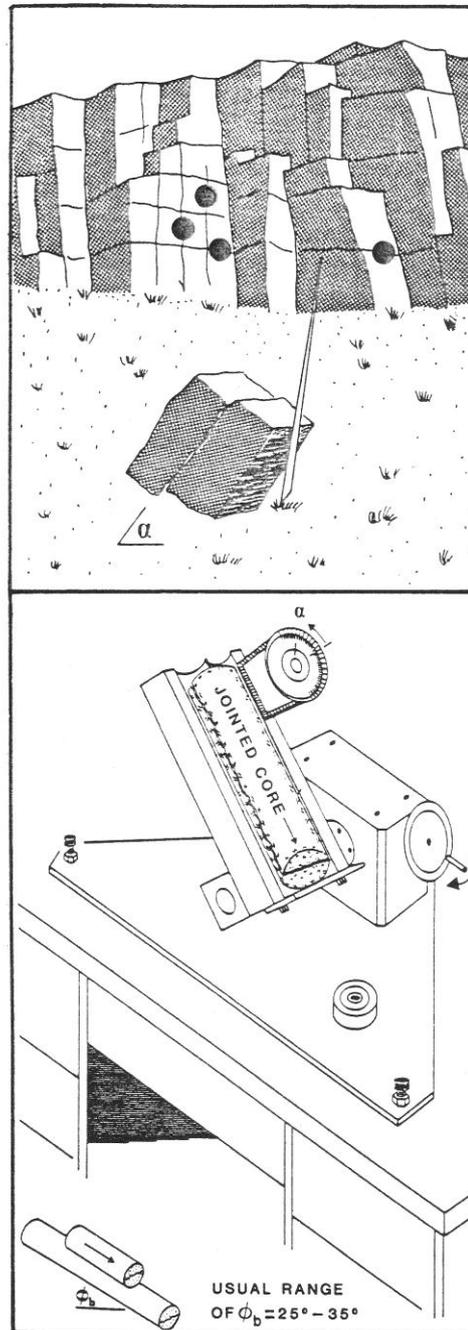


Figure 13. Schematic representation of tilt testing for measuring JRC (joint roughness) and ϕ_b (basic friction angle of planar surfaces).

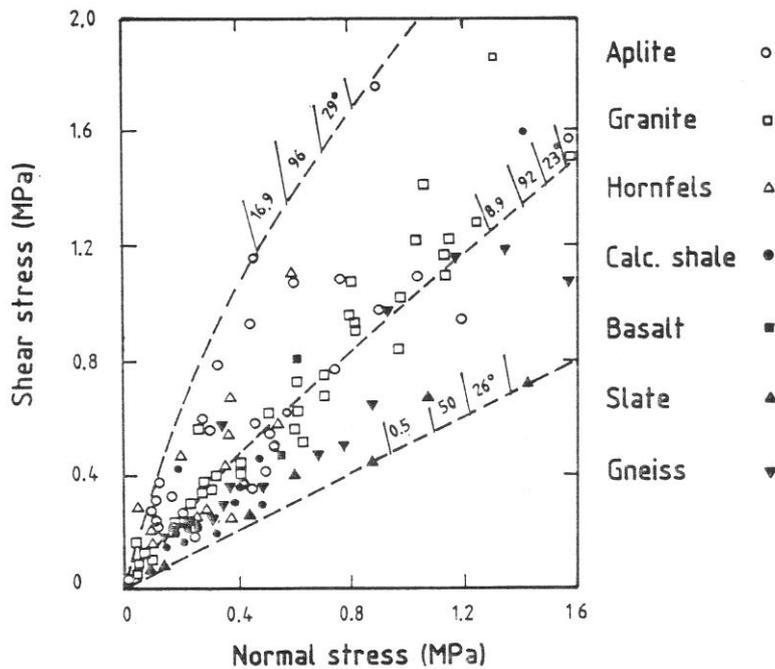
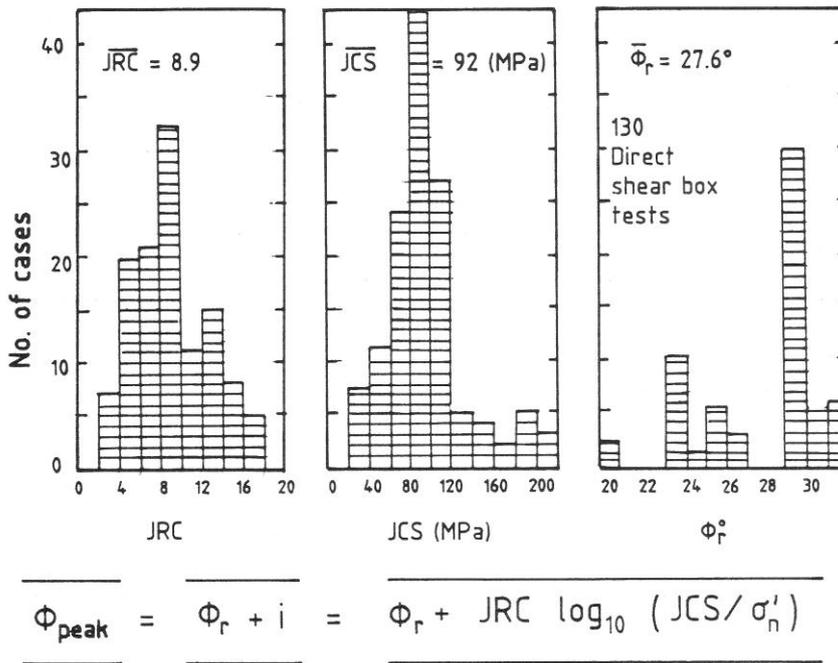


Figure 14. Examples of joint shear strength statistics for a range of rock types. Note equation for generating peak shear strength envelopes.

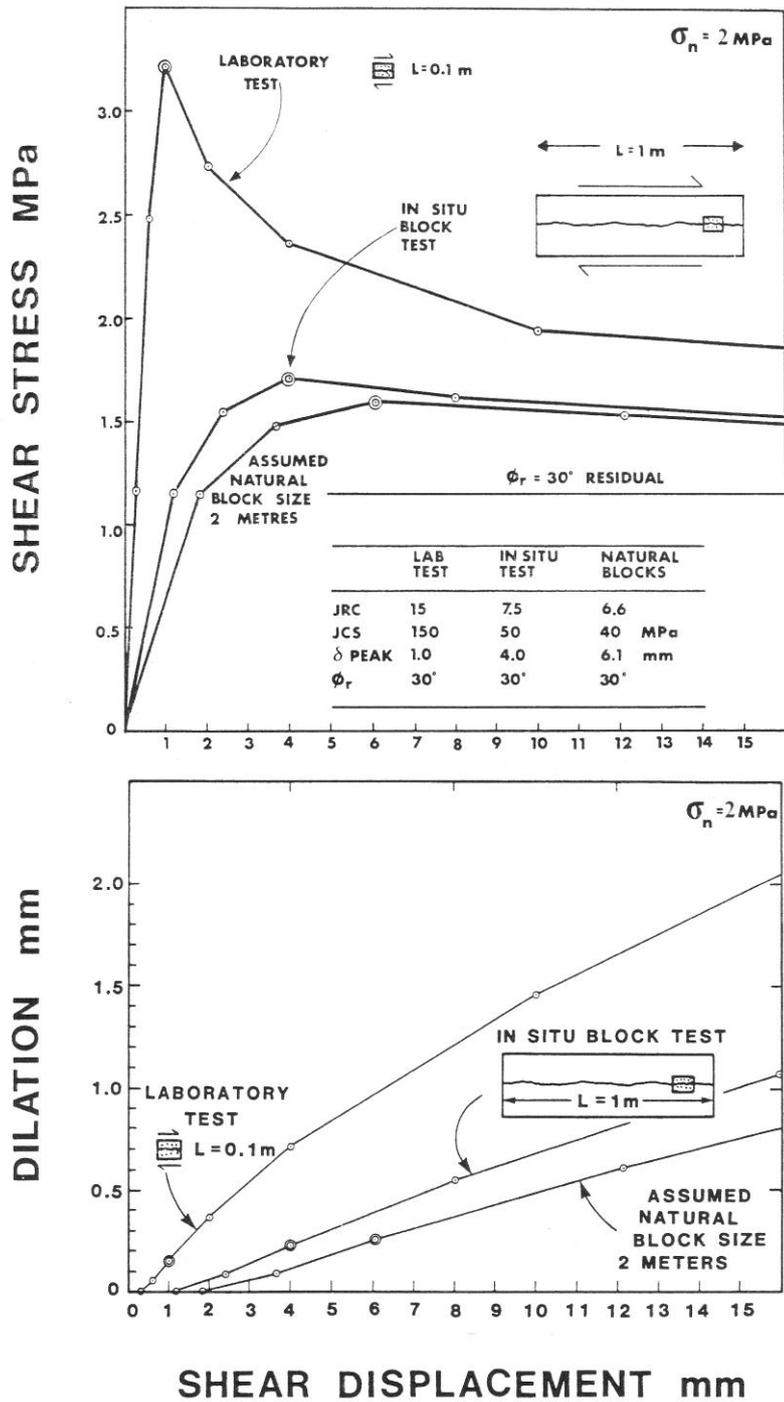


Figure 15. Predicted shear-stress-displacement and dilation behaviour for block sizes of 0.1, 1.0 and 2.0 m length.

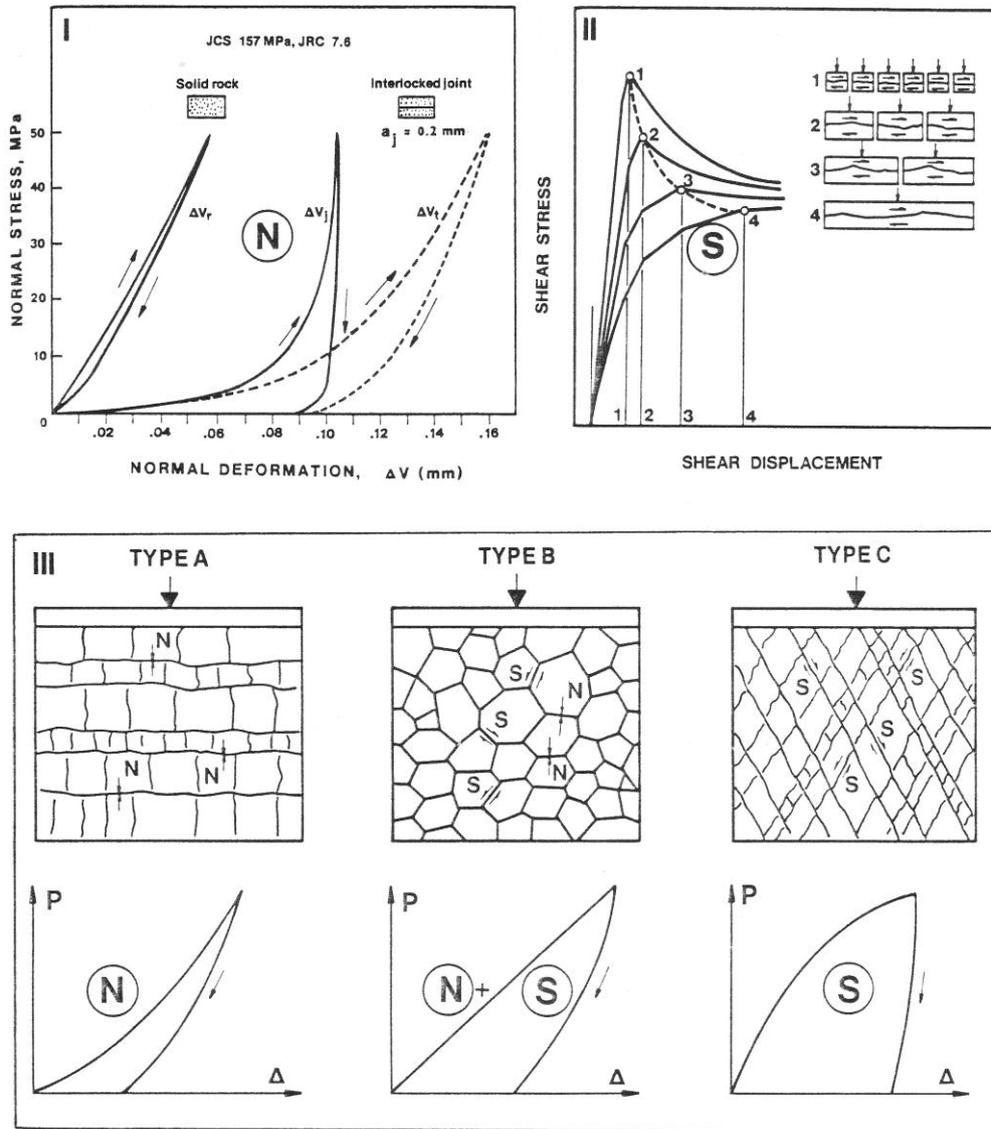


Figure 16. Normal and shear deformation components N and S for single joints (I and II) cause concave, convex or linear stress-deformation response for differently jointed rock masses (III). (Bandis et al. 1981, 1983 and Barton 1986).

- Step 8. Deletion of blocks (finite difference zones) to represent twin excavations (a motorway beneath Oslo) causes redistribution of stresses. High tangential stresses are registered in the menaitt dyke (shaded). Stress distribution is different from that predicted by elastic continuum FEM. Maximum tangential stresses were 7.3 MPa. (Makurat et al. 1989.)
- Step 9. Resulting deformations in step 9 show anisotropic distribution and smallest magnitudes in the right hand excavation, which passes through stiffer material. Note that systematic bolting of 1.5 and 2.0 m spacing has been numerically applied in both these 14 m span excavations. Bolts were incorporated at an early stage in the numerical time-stepping to equilibrium (equivalent to 1 m behind the face). Maximum bolt loads were in the range 6 to 13 tons at certain joint crossings. Maximum rock deformation (left arch) was 5.5 mm.
- Step 10. Shear deformations marked by line thickness in step 10 show the significant influence of large excavations on the surrounding rock mass. The maximum shear of 2.2 mm occurs close to the left hand excavation. Some changes in permeability are generally caused by such shearing.
- Step 11. Other computer output includes diagrams that show the distribution of joint apertures (physical and conducting apertures), bolt loads at joint crossings, and the radial and tangential stress in eventual concrete liners, if applied as permanent support.
- Step 12. Parameter sensitivity studies should subsequently be run. These should include changes in stress, effective block size, JRC, JCS, bolt spacing and capacity. A comprehensive set of sensitivity calculations run with these conservative two-dimensional models of the joint structure serve as important checks on the empirical design of reinforcement described earlier.

It should be noted that two-dimensional models represent the cavern axis as being parallel to the strike of the jointing or faulting represented in the model. This is generally an unfavourable orientation. The two-dimensional discrete element modelling should therefore represent "worst-case scenarios".

Conclusions

1. A preliminary classification of rock mass quality for Hong Kong's granites and welded tuffs indicates eminently suitable rock conditions for utilization of large caverns of 20 to 40 span at depths of 50 to 100 metres below the surface. Methods of investigations and data acquisition are outlined.
2. Analysis of Q-system case records of large caverns indicate that caverns of similar span have been successfully excavated and safely utilized. Combinations of systematic rock bolting and fiber reinforced shotcrete are found to be the most suitable reinforcement. Cost estimates favour spans of at least 25 to 30 m for optimal volume/area ratios.

3. An integrated Q-system approach to reinforcement design is demonstrated. This includes as a first step, the selection of bolt spacing and shotcrete thickness, and as a second step the estimation of the necessary reinforcement capacity. Estimates of expected deformations and limits for unsupported excavations are discussed.
4. The integrated approach to cavern design demonstrated in the Q-system should be compared with numerical modelling using discrete element (jointed) representations of the rock mass, to provide a "second opinion" concerning the stability of extra large spans. The most recent advances in these methods provide a degree of reality that is not available with elastic, isotropic continuum analyses.

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