Application of the Q-system to tunnels and caverns in weak or *faulted* rock

Nick Barton
NB&A, Oslo, Norway

Šuklje Lecture

Thank you Prof Vojkan Jovičić and
the Slovenian Geotechnical Society
for this great honour!

Akademik prof.dr. Lujo Šuklje s soprogo Boženko ob svoji 70.letnici
Academician Prof. Dr. Lujo Šuklje with his wife Boženka at his 70th anniversary

Your famous Prof Šuklje-

(who looks like a very kind person).

He lived from 1910 to 1997

UVODNO PREDAVANJE

Bojan MAJES
prof. dr., univ. dipl. inž. gradb.
Prof. Dr., Univ. Grad. in Civ. Eng.

Univerza v Ljubljani
Fakulteta za gradbeništvo in geodezijo

University of Ljubljana
Faculty of Civil and Geodetic Engineering

O ŽIVLJENJU IN DELU PROF. DR. LUJA ŠUKLJETA

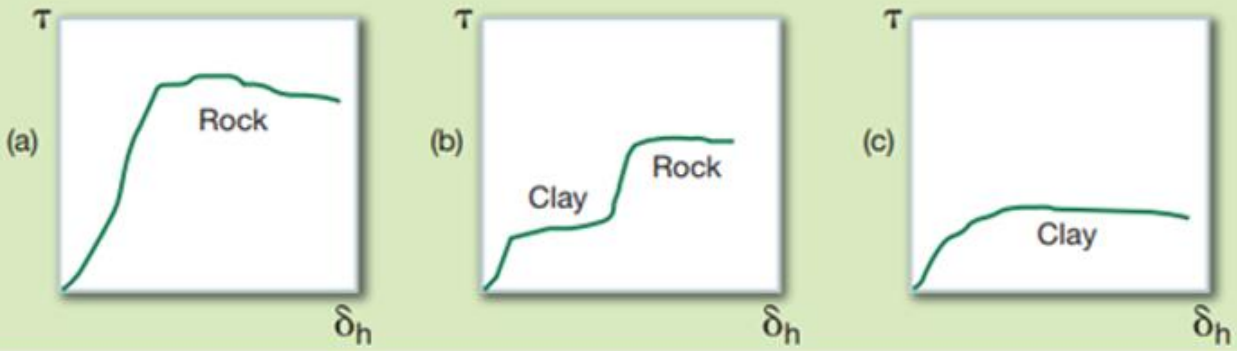
ON LIFE AND WORK OF PROF. DR. LUJO ŠUKLJE

In 1961 his first graduate, Mr. Jože Brodnik, wrote a thesis entitled: »The level of activation of the shear strength between concrete and soils depending on the allowable displacements need to be tested with laboratory model tests «.

In his extensive journal editorial work Prof Šuklje also encountered his first graduate student's publication on the same subject. His topic is close to the Q-system ratio Jr/Ja.

ACTA GEOTECHNICA, Univerza v Ljubljani, No. 1 (1962) do No. 101 (1989)

- **Deformation conditions of the mobilization of the friction between concrete and soil.** Univerza v Ljubljani, Acta Geotechnica 4, 1-4, Ljubljana 1963. (Soavtor: J. Brodnik.)



(a) Rock wall contact			(thin coatings)			
	Ja =	0.75	1.0	2	3	4
	Jr		tan⁻¹ (Jr/Ja)°			
A. Discontinuous joints	4	79°	76°	63°	53°	45°
B. Rough, undulating	3	76°	72°	56°	45°	37°
C. Smooth, undulating	2	69°	63°	45°	34°	27°
D. Slickensided, undulating	1.5	63°	56°	37°	27°	21°
E. Rough, planar	1.5	63°	56°	37°	27°	21°
F. Smooth, planar	1.0	53°	45°	27°	18°	14°
G. Slickensided, planar	0.5	34°	27°	14°	9.5°	7.1°

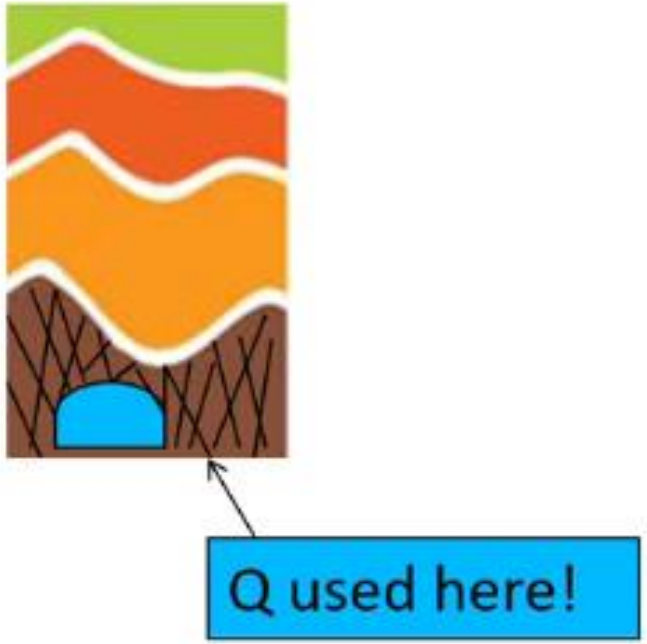
(b) Rock wall contact when sheared			(thin fillings)		
	Ja =	4	6	8	12
	Jr		tan⁻¹ (Jr/Ja)°		
A. Discontinuous joints	4	45°	34°	27°	18°
B. Rough, undulating	3	37°	27°	21°	14°
C. Smooth, undulating	2	27°	18°	14°	9.5°
D. Slickensided, undulating	1.5	21°	14°	11°	7.1°
E. Rough, planar	1.5	21°	14°	11°	7.1°
F. Smooth, planar	1.0	14°	9.5°	7.1°	4.7°
G. Slickensided, planar	0.5	7°	4.7°	3.6°	2.4°

In 1968 Prof Šuklje held a lecture at the Imperial College in London on the results of his work and the work of his colleagues. (It likely included his *non-linear* soil mechanics interests?)

(AT THIS TIME I WAS 100m AWAY(!) IN THE ROYAL SCHOOL OF MINES, WORKING ON A PhD ABOUT ROCK SLOPE STABILITY AND THE *NON-LINEAR* SHEAR STRENGTH OF ROCK JOINTS.

So we nearly met and nearly had the same interests! Just that it was soil and rock respectively. ***But non-linear interests for both of us!***

THE 'Q-system' ?

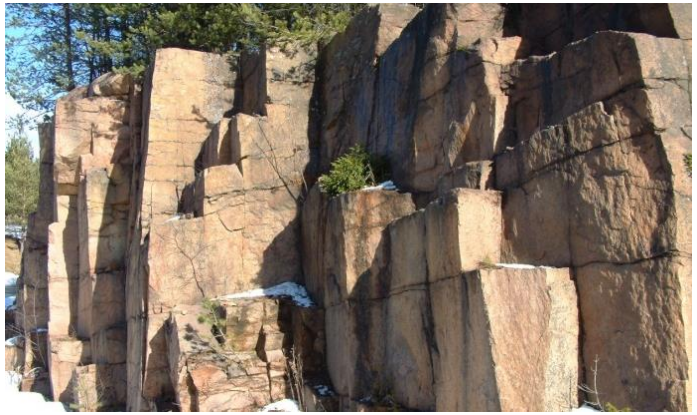


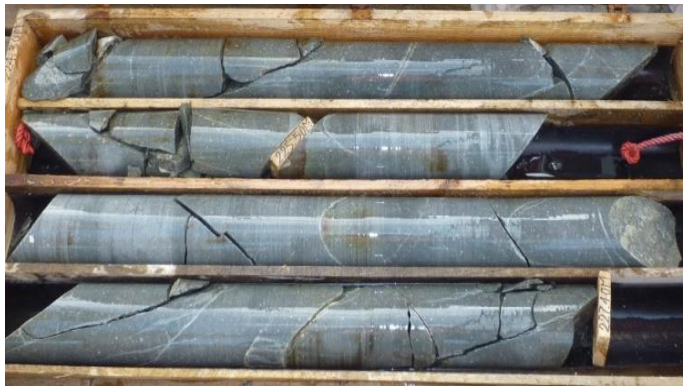
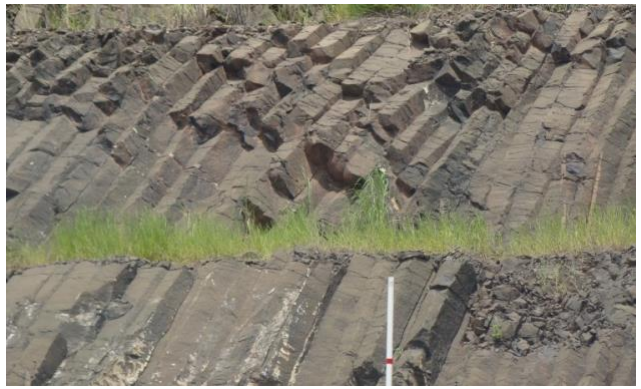
As a briefest introduction:

Q means *rock mass quality*.

Q consists of *ratings for six parameters*.

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$ = **(Block size) x (friction) x ('active stress')**



VARIABLE WORLD NEEDS *BROAD-REACH CLASSIFICATION*

ROCK MASS 'SAMPLES' useful for design are much larger than for steel, concrete, soil.

A 'Q-system' saves us from impossibly large and expensive in situ tests of the rock mass.

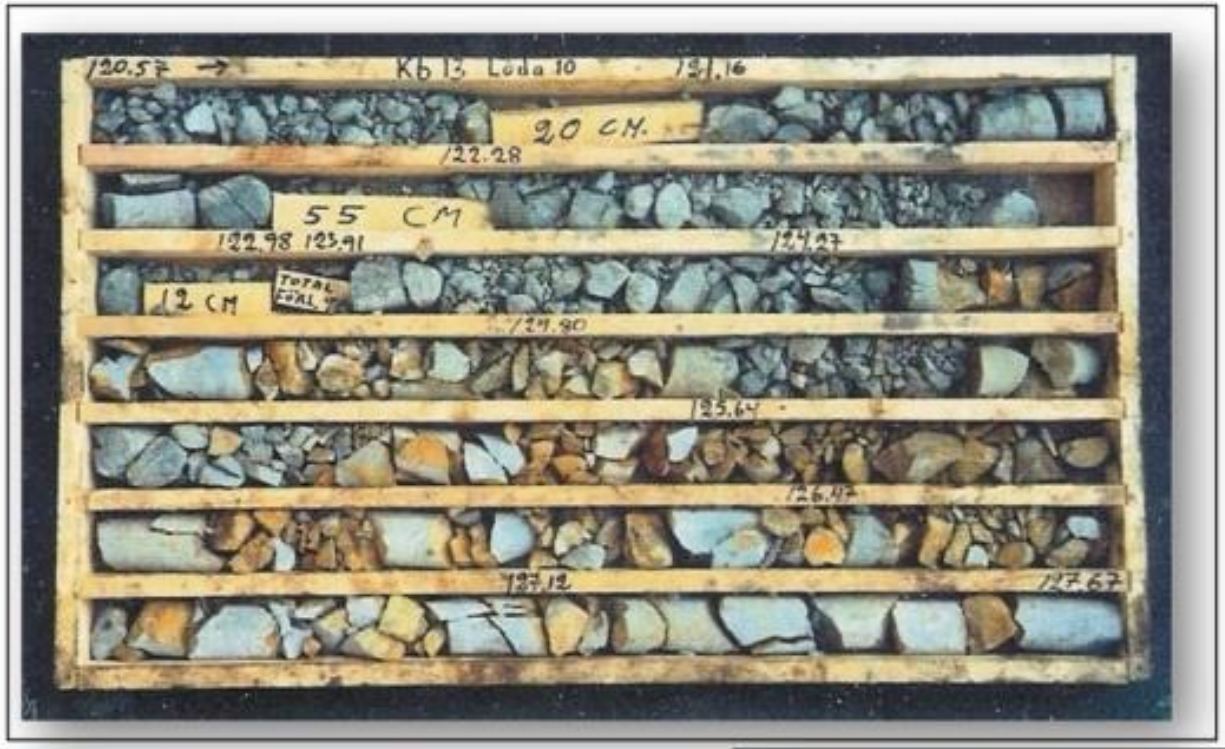
We have 'numbers' for describing the quality of all these rock masses.

Q = 1000 (or better)
(Q ≈ 100/0.5 x 4/0.75 x 1/1)

Q = 0.001 (or worse)
(Q ≈ 10/20 x 1/8 x 0.5/20)

THE WIDE NUMERICAL RANGE OF Q REFLECTS THE HUGE POTENTIAL DIFFERENCES IN *ROCKMASS* PROPERTIES

Q = 0.001 and 1000, or RMR = 5 and 95 ?



REGIONAL SHEAR ZONE, S.W. SWEDEN. ***VERY LOW Q!***
(HALLANDSÅS RAIL TUNNELS)

VERTICAL HOLE, ADMIRALTY, HK
MISSED SUB-VERTICAL JOINT SETS)
APPARENT HIGH Q

3. SANDSTONE-CLAYSTONE

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

4. NODULAR-LIMESTONE

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

5. MUDSTONE (overall RQD=30)

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

6. GRANITE (decomposed) RQD=0

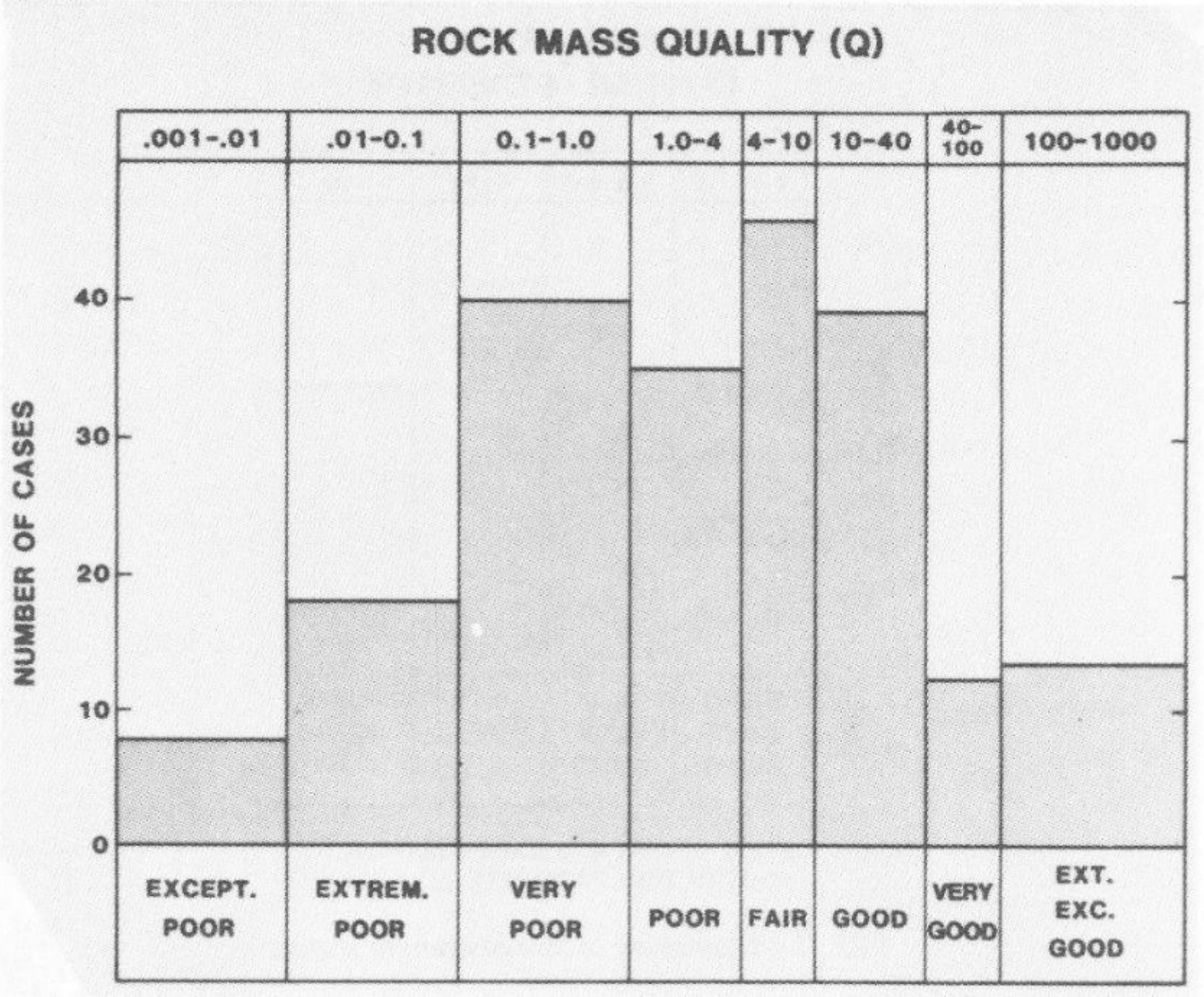
Q = 10/20 x 1.0/6 x 0.66/6
= 0.009 (exceptionally poor)
(1A/2J, 3J/4K, 5B/6N)

50 rock types in 212 Q-case records. Numerous clay-filled and faulted cases. NB et al. 1974.

Even though granite and gneiss were most common in the 60% case records from Norway and Sweden, such rocks were mostly selected due to clay problems……

Otherwise (if no clay) no support would be needed and they would be of little help to develop the Q-system.

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



The wide range of Q-values in the original 212 case records.

Barton, 1988 (ASTM).

Case records with ***significant jointing and clay-filling*** were sought because then there was data on tunnel or cavern support B+S(mr) as well.

Today mostly B+S(fr) for reinforcement and support.

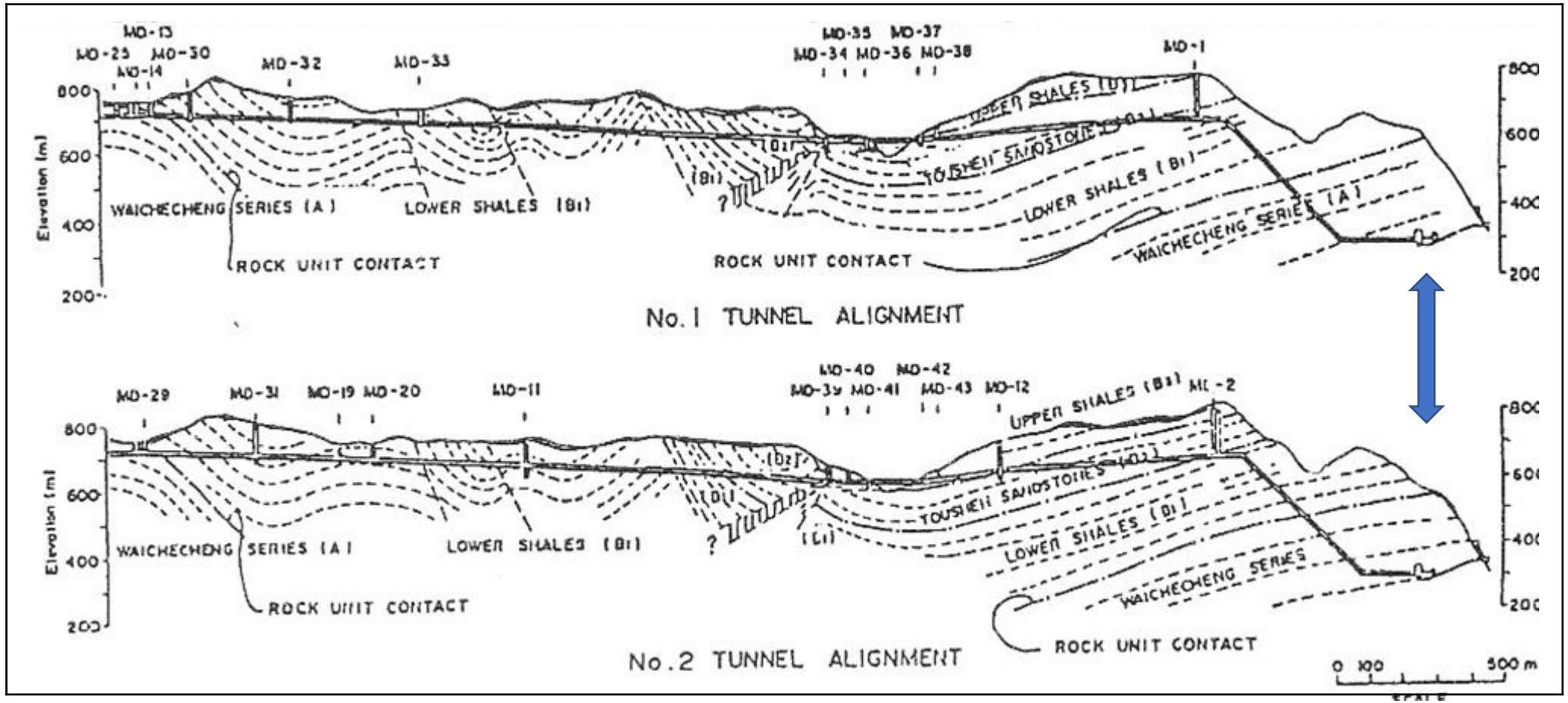
TWO CASE RECORDS WILL BE DESCRIBED

- **Mingtan Pumped Storage Project** in Taiwan. Numerous (thirteen) clay-filled bedding-plane-faults crossing the cavern axis. **(Maximum clay thickness > 1m). Q-logging gave** satisfactory cavern support design that later resisted a very ***major shallow earthquake*** close to the site.

- **Channel Tunnel(s)** between England and France driven by TBM in chalk marl with UCS of 4 to 7MPa. **(Well-developed jointing unexpected). I**ndependent rock mass **Q-classification** which perfectly explained the costly overbreak problems for the TBM.

CASE RECORD #1

Q-APPLICATION IN FAULTED ROCK AT MINGTAN PUMPED HYDRO IN TAIWAN

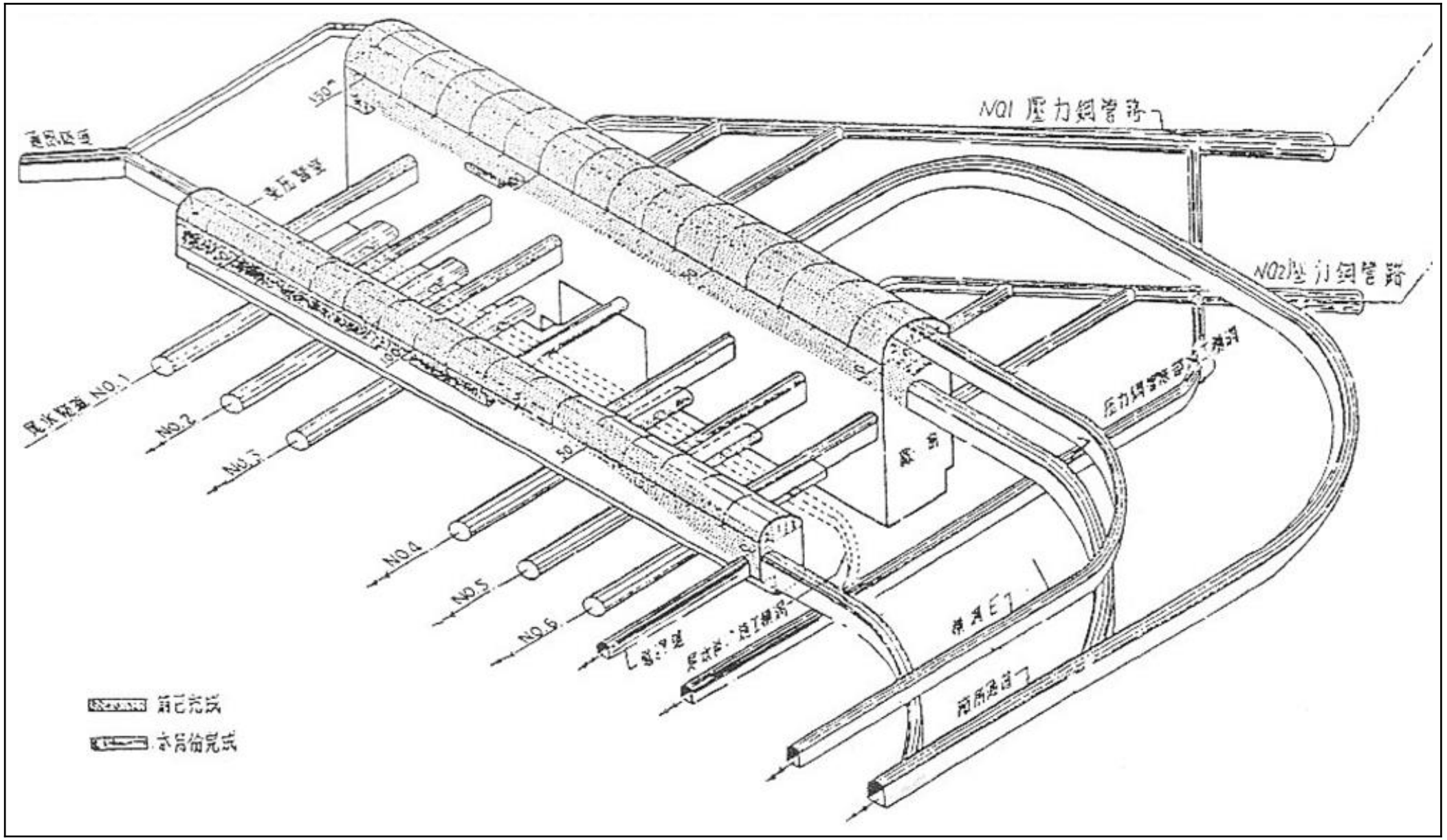


NB's CONSULTING BRIEF

- The chief design company Sinotech (head offices in Taipei) invited the writer to perform an independent assessment of rock mass qualities ***(for arch and wall reinforcement)*** in the MACHINE HALL that was under construction near Sun Moon Lake. Prof. Hoek and Golders also involved.

- This was close to the site of a severe (Chi-Chi) earthquake experienced without cavern damage some years later. More details of this are given later.

Layout for the 23m wide machine hall and transformer hall. Mingtan Pumped Hydro. Sinotech Consulting Co. Taipei.

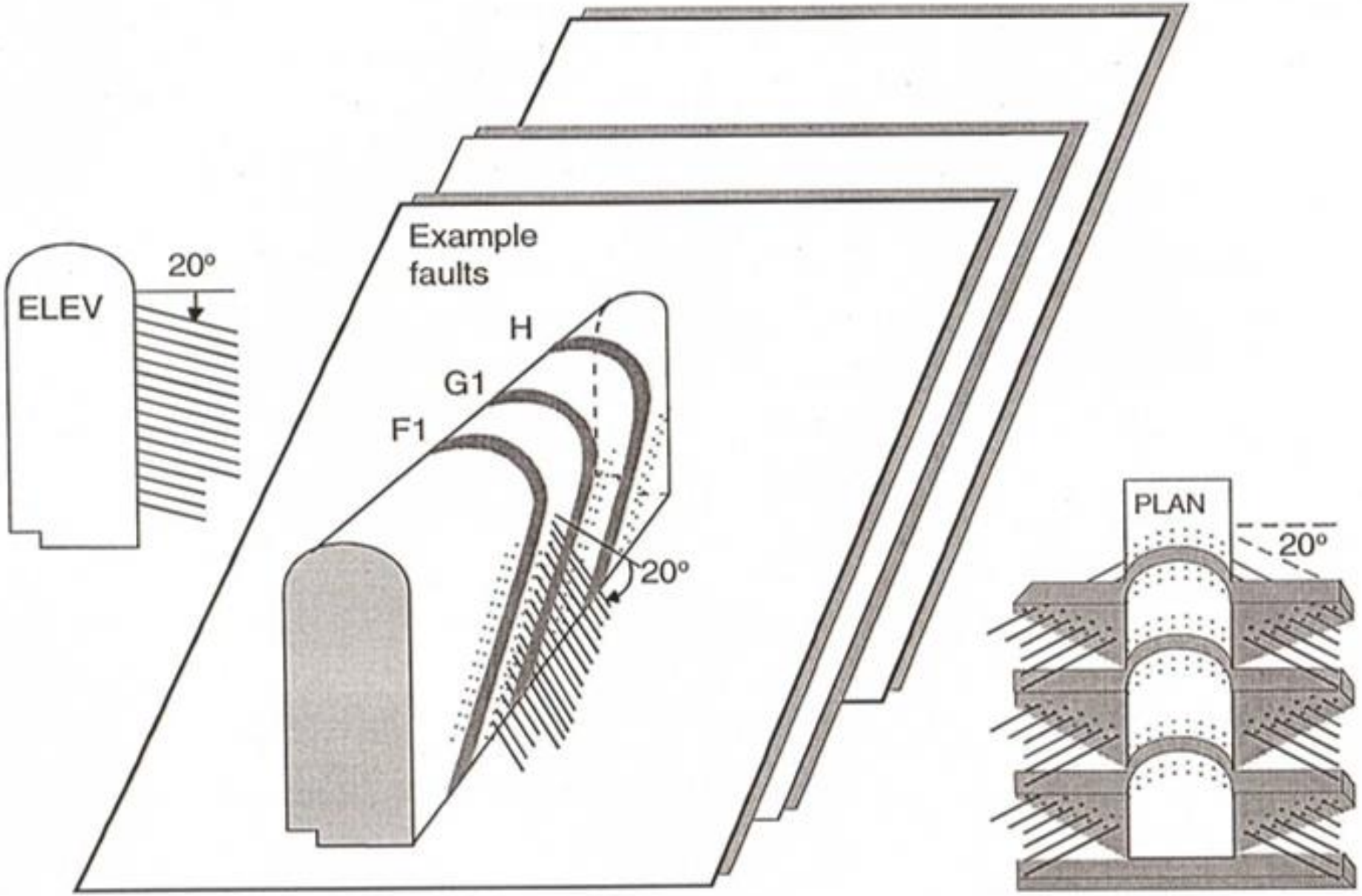


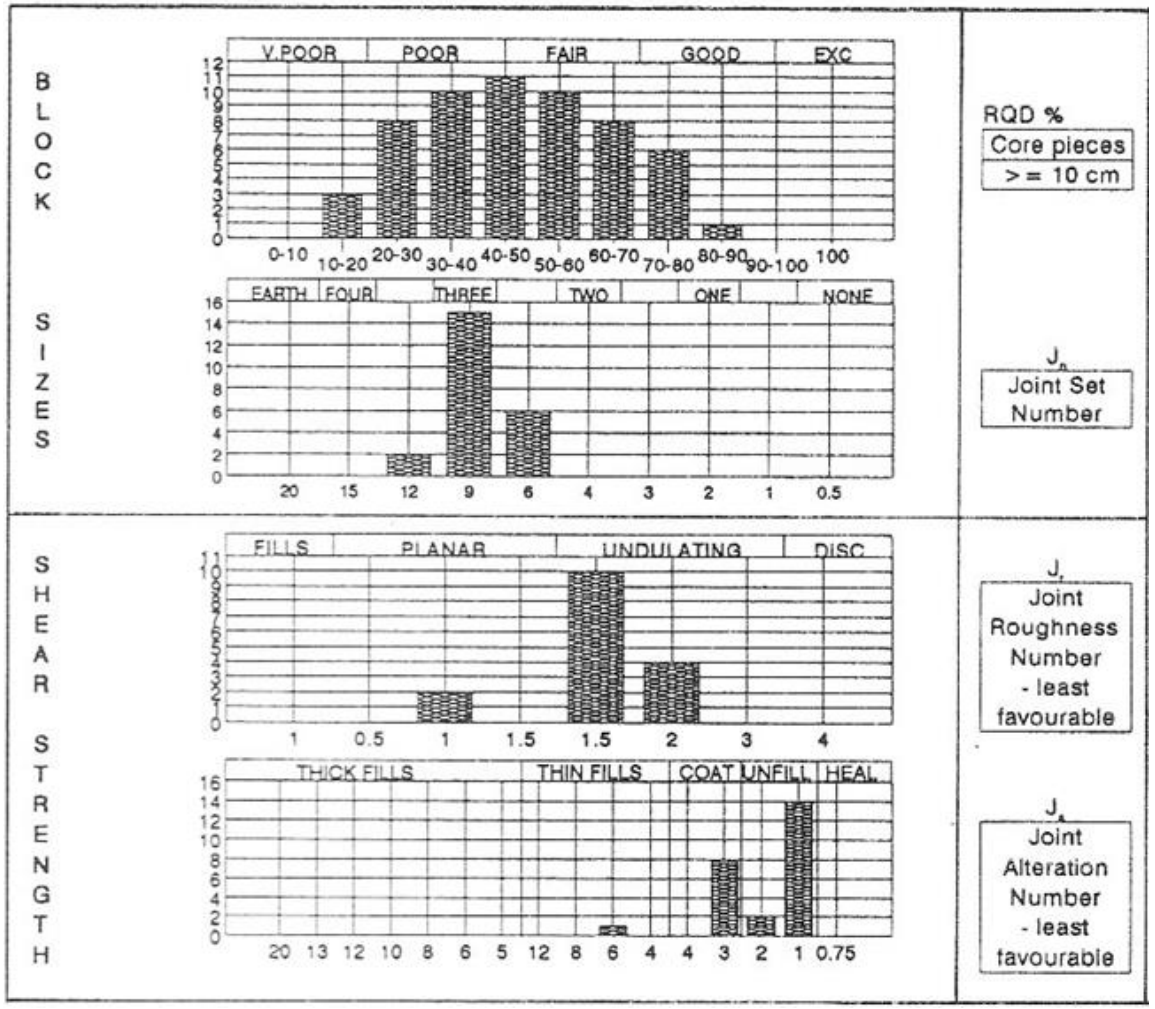
CLAY-FILLED BEDDING-PLANE FAULTS

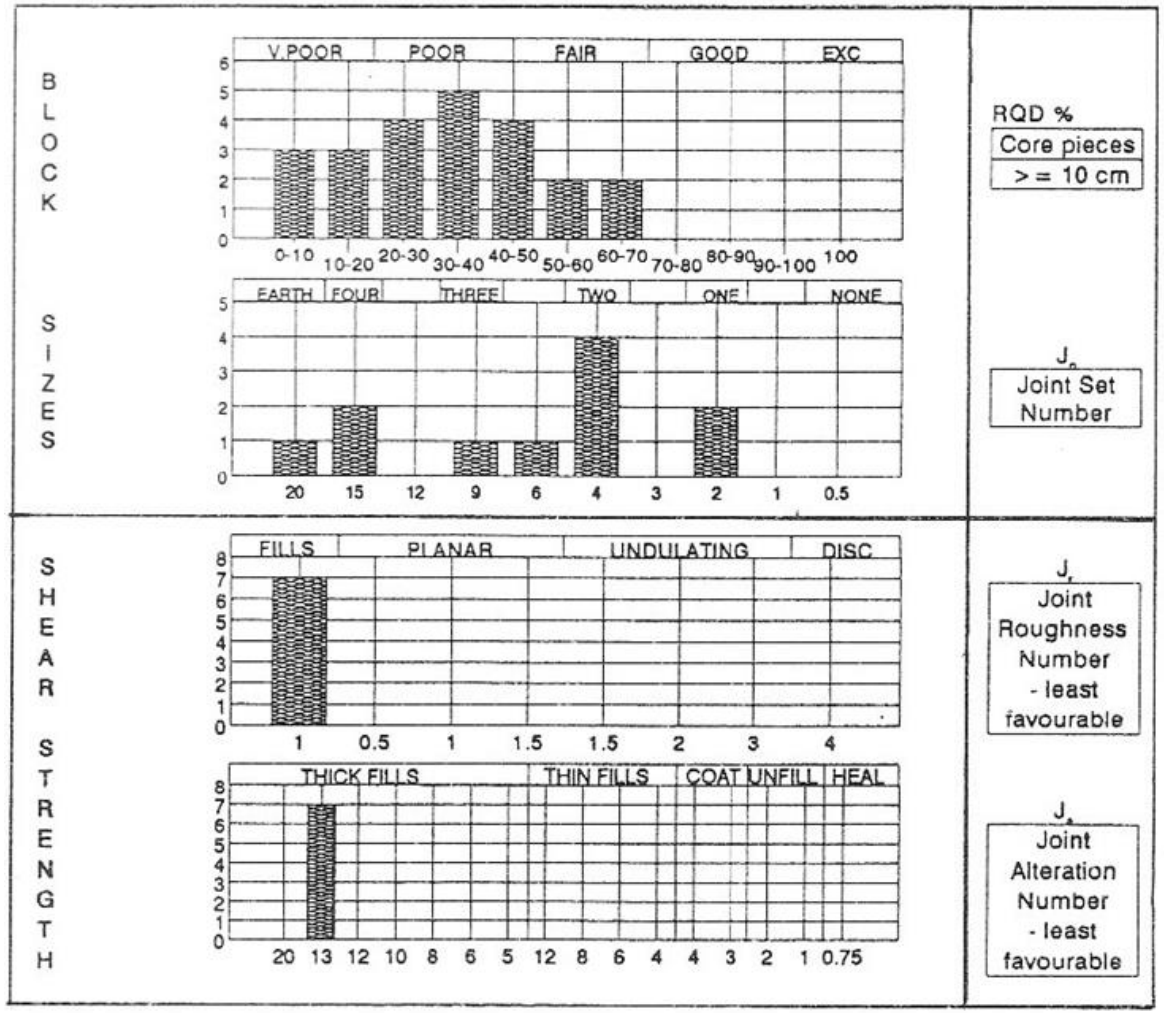
- A very special feature of the site was that folding and the tectonic history of the sedimentary interbeds had caused ‘bedding plane faults’ to develop, and some of these had thick (> 1m) clay fills.

- A prominent consultant had previously insisted on re-orientation of cavern axes, so that these low friction discontinuities were not adversely dipping out of one wall as originally. They ‘now’ dipped across the re-oriented axis.

THIS '3D' IMAGE (OF WALL REINFORCEMENT PRINCIPLES) SHOWS THE BASIC FAULT STRUCTURE ORIENTATION (13 in all !)



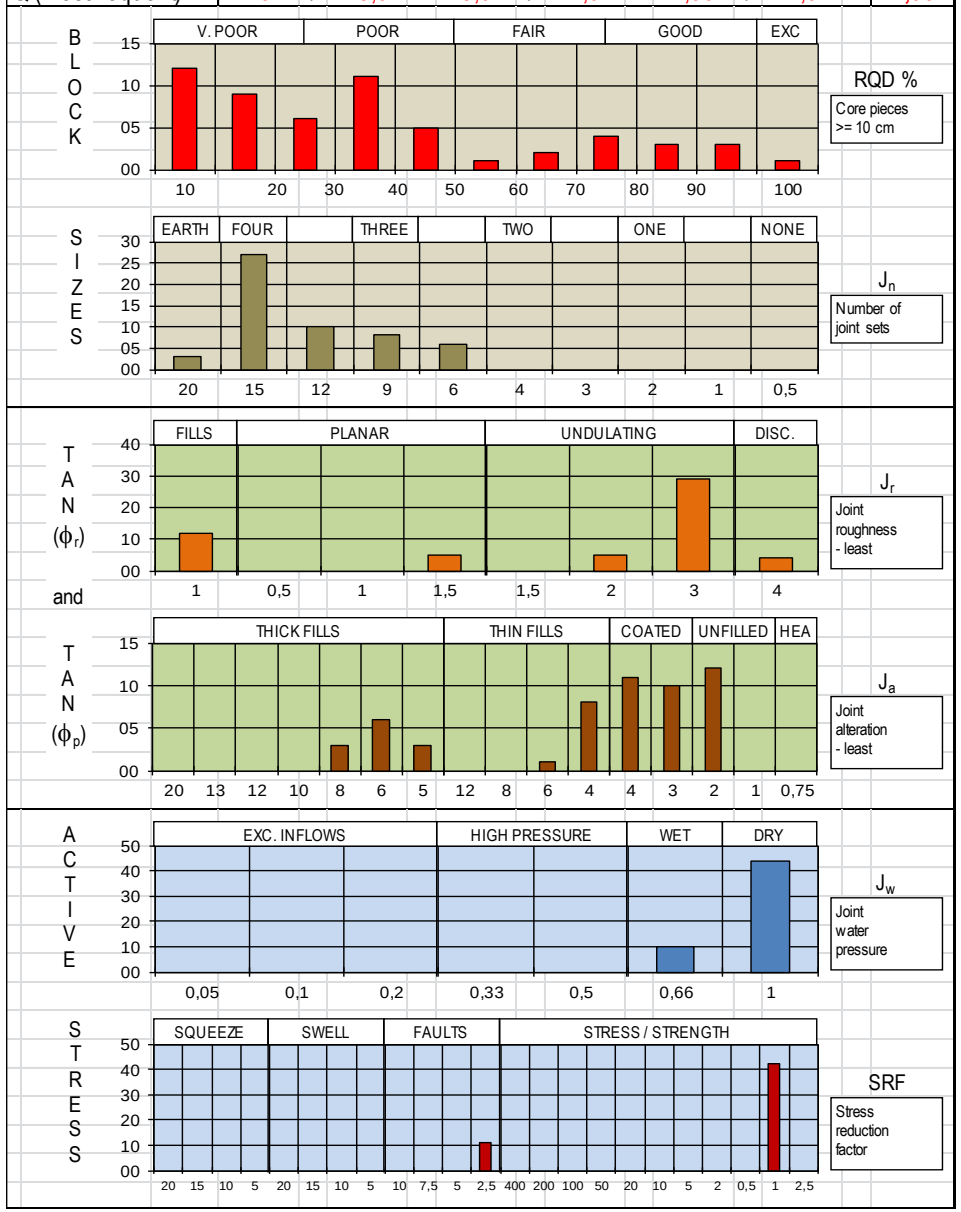




For maybe the first time (it was 1987) ‘Q-histogram’ logging was used.

Left: bedded sandstone, Right: ‘bedding-plane faulted’ rockmass.

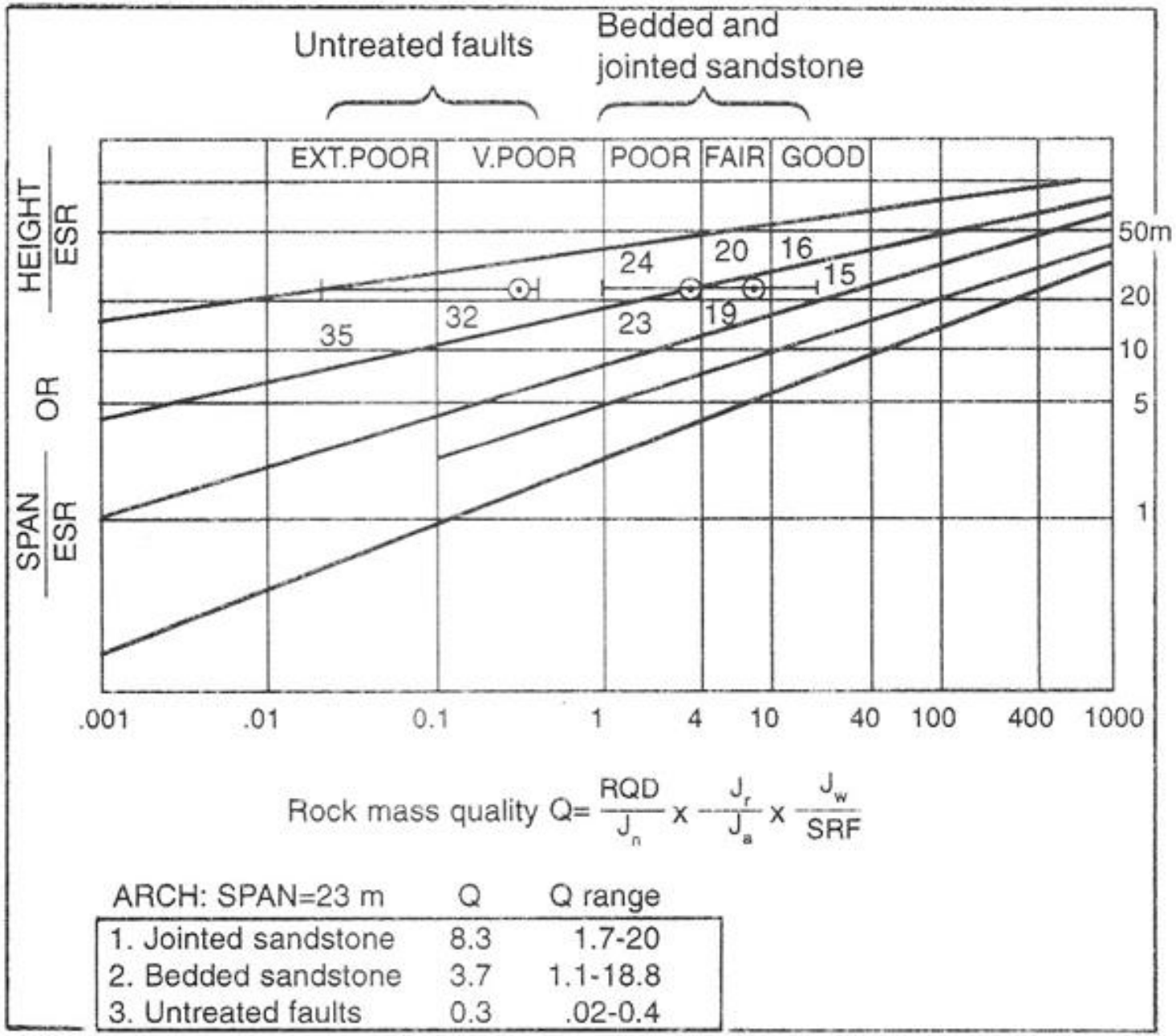
Q - VALUES:	(RQD	/	Jn)	*	(Jr	/	Ja)	*	(Jw	/	SRF)	=	Q
Q (typical min)=	10	/	15,0	*	1,0	/	6,0	*	0,66	/	2,5	=	0,029
Q (typical max)=	75	/	6,0	*	4,0	/	2,0	*	1,00	/	1,0	=	25,0
Q (mean value)=	38	/	12,8	*	2,4	/	3,9	*	0,94	/	1,3	=	1,29
Q (most frequent)=	10	/	15,0	*	3,0	/	2,0	*	1,00	/	1,0	=	1,00



Q IS ONLY PART OF A ROCK MASS DESCRIPTION EXERCISE

Also Jw and SRF

I	**ROCK MASS STRUCTURE**			
1	RQD	Deere et al., 1967)	block size	Q
2	J_n	= joint set number		Q
3	F	= joint frequency (per metre)		
4	J_v	= volumetric joint count (Palmström, 1982)		
5	S	= joint spacing (in metres)		
6	L	= joint length (in metres)		
7	w	= weathering grade (ISRM, 1978)		
8	α/ß	= dip/dip direction of joints (Schmidt diagram)		
II	**JOINT CHARACTER**			
9	J_r	= joint roughness number	shear strength	Q
10	J_a	= joint alteration number		Q
11	JRC	= joint roughness coefficient		
12	a/L	= roughness amplitude of asperities per unit length (mm/m)		
13	JCS	= joint wall compressive strength		
14	ϕ_r	= residual friction angle		
15	r,R	= Schmidt rebound values for joint and rock surfaces		
III	**WATER, STRESS, STRENGTH**			
16	J_w	= joint water reduction factor	active stress	Q
17	SRF	= stress reduction factor		Q
18	K	= rock mass permeability (m/s)		
19	σ_c	= compressive strength		
20	σ_1	= major principal stress		



ARCH: SPAN=23 m	Q	Q range
1. Jointed sandstone	8.3	1.7-20
2. Bedded sandstone	3.7	1.1-18.8
3. Untreated faults	0.3	.02-0.4

The SPAN vs Q and 'support class' (#15….#35 range) is in the *numbered format* following Barton et al. 1974.

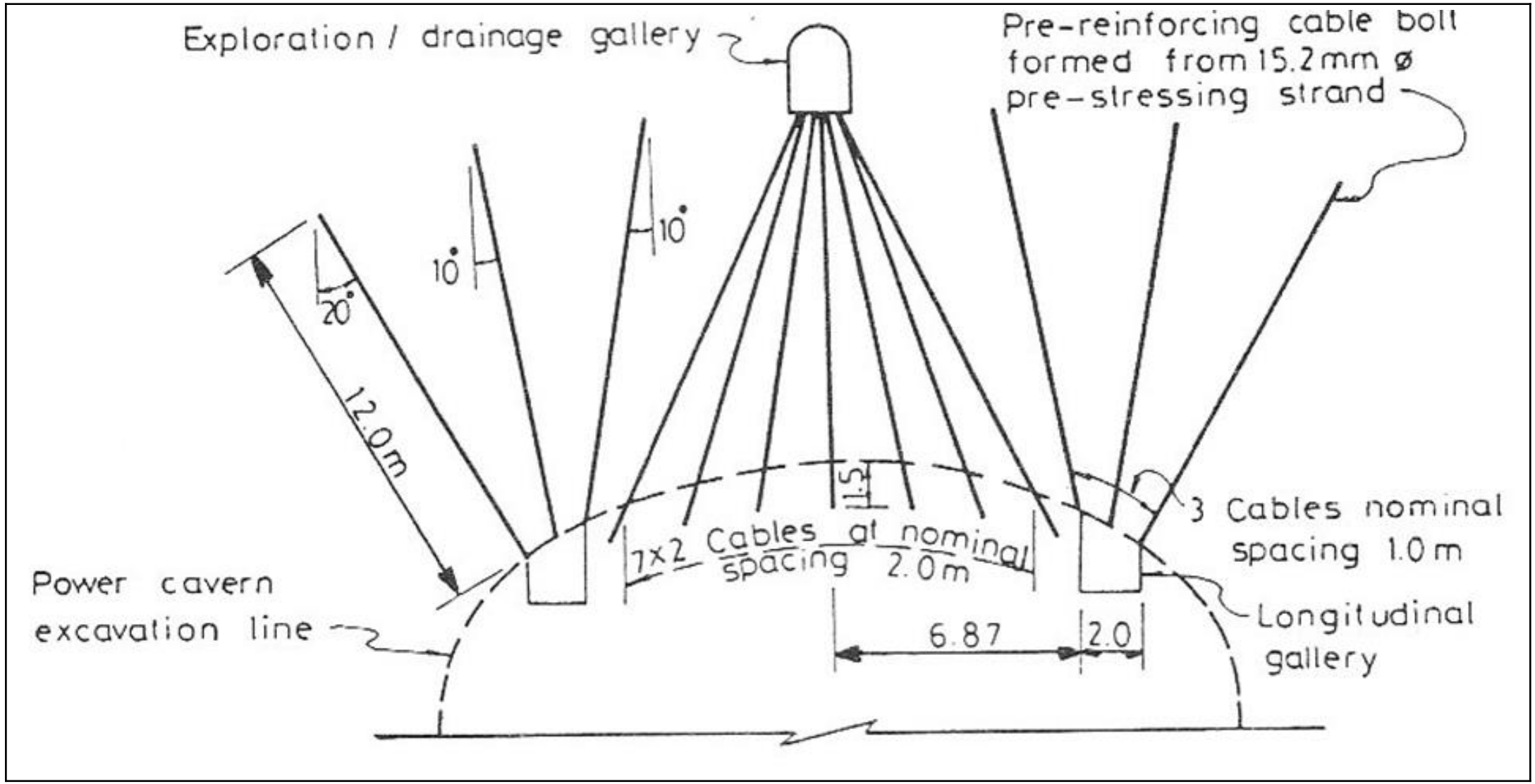
The *most typical ranges of values* obtained, which in a few minor aspects differed from those previously assessed by Sinotech and Golders, were as follows:

1. jointed sandstone $Q \approx \frac{30-80}{6-9} \times \frac{1.5}{1-3} \times \frac{1}{1} \approx 1.7 \text{ to } 20$

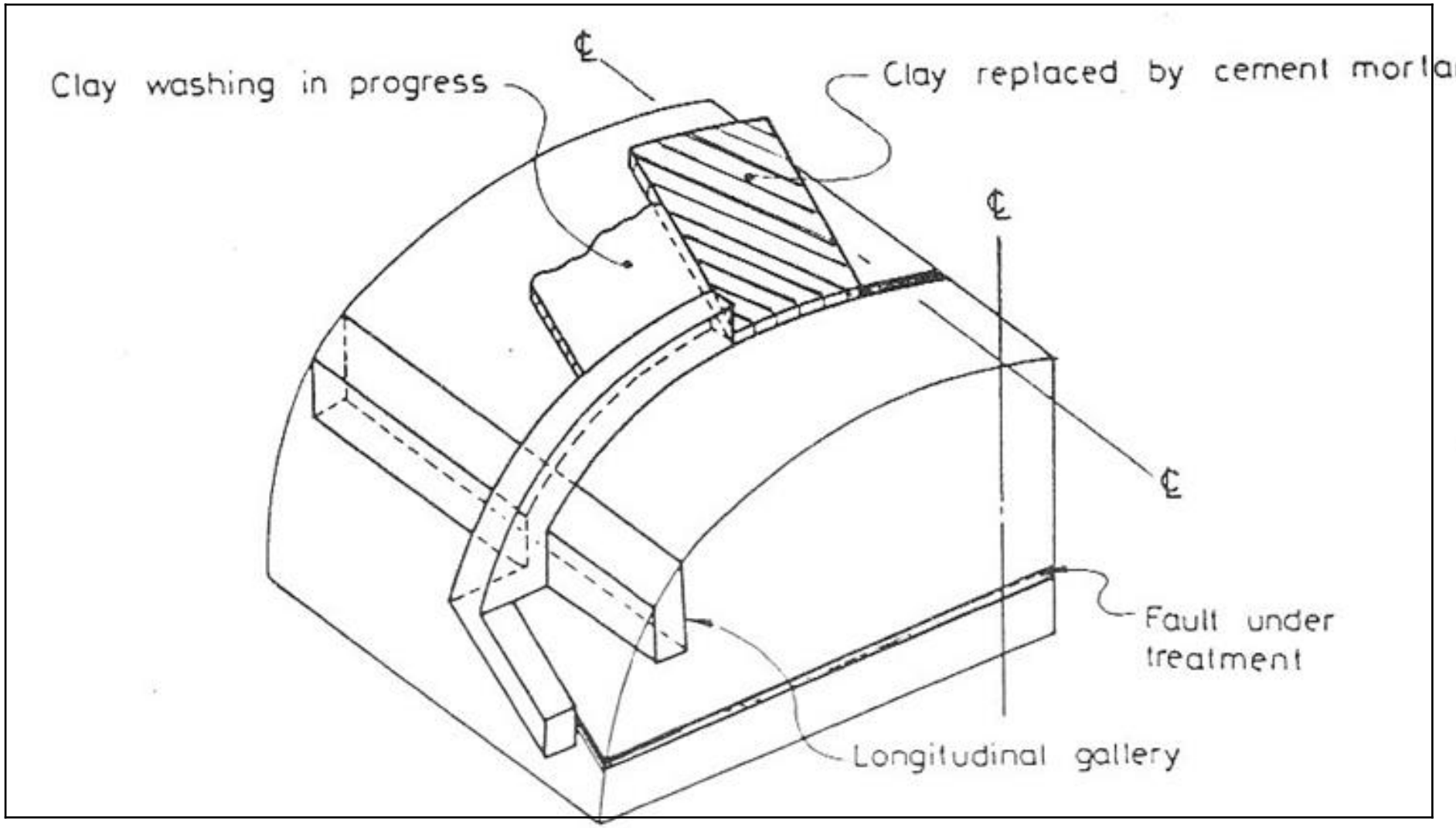
2. bedded sandstone $Q \approx \frac{30-50}{4-9} \times \frac{1-1.5}{1-3} \times \frac{1}{1} \approx 1.1 \text{ to } 18.8$

3. fault zones $Q \approx \frac{10-50}{4} \times \frac{1}{13} \times \frac{1}{2.5-10} \approx 0.02 \text{ to } 0.4$

Designers SINOTECH wanted confirmation of Q and confirmation/additions for arch support. Later they wanted wall support recommendations too.

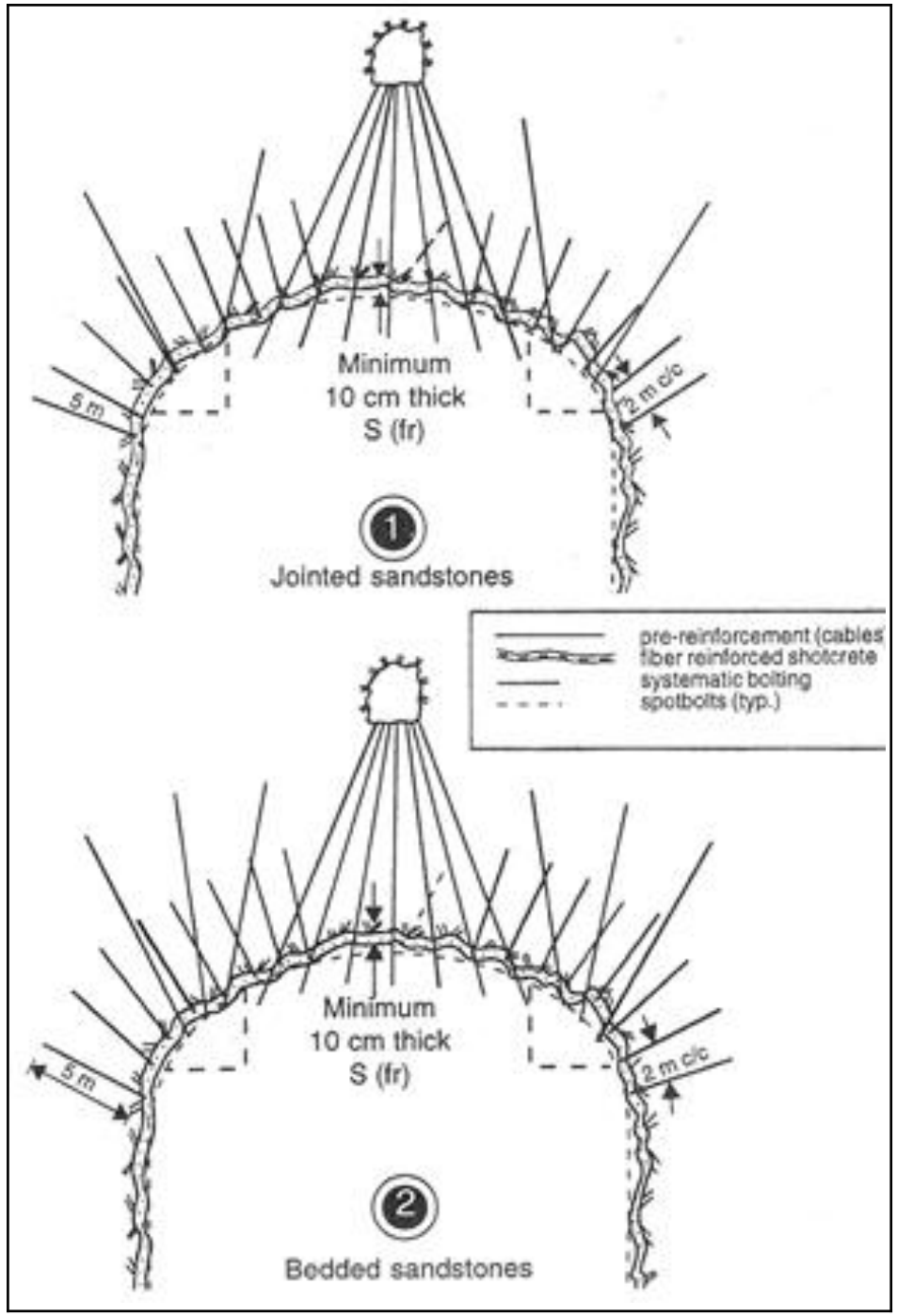


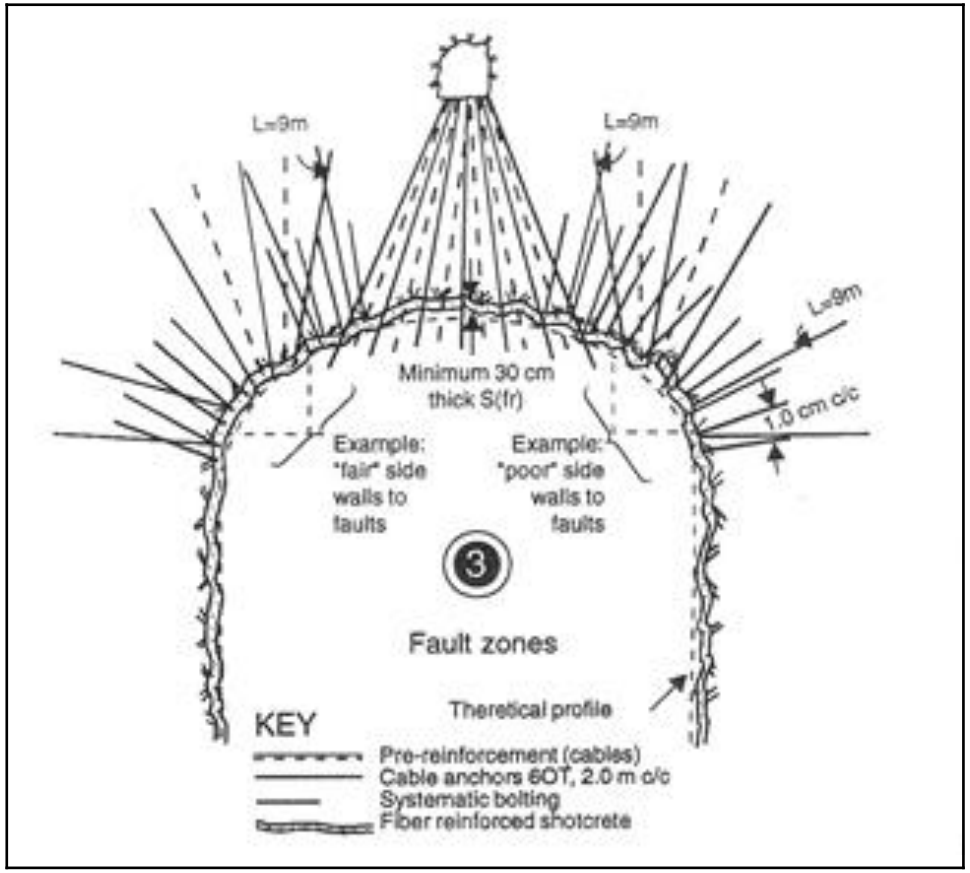
So far there was ‘pre-reinforcement’ from galleries



Highly unusual (but effective) ‘clay replacement.’

A technique used by Sinotech at a large dam in Taiwan.



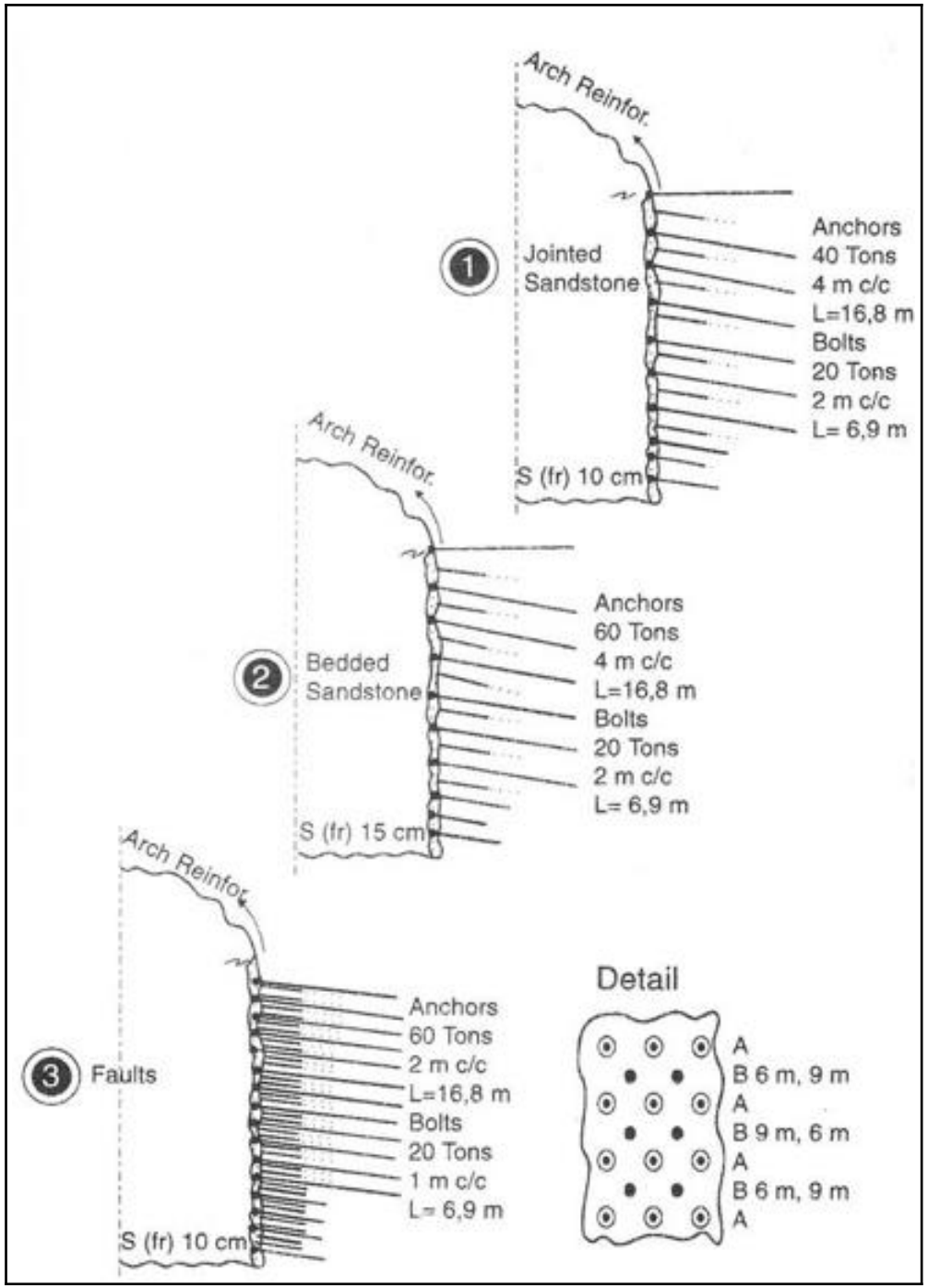


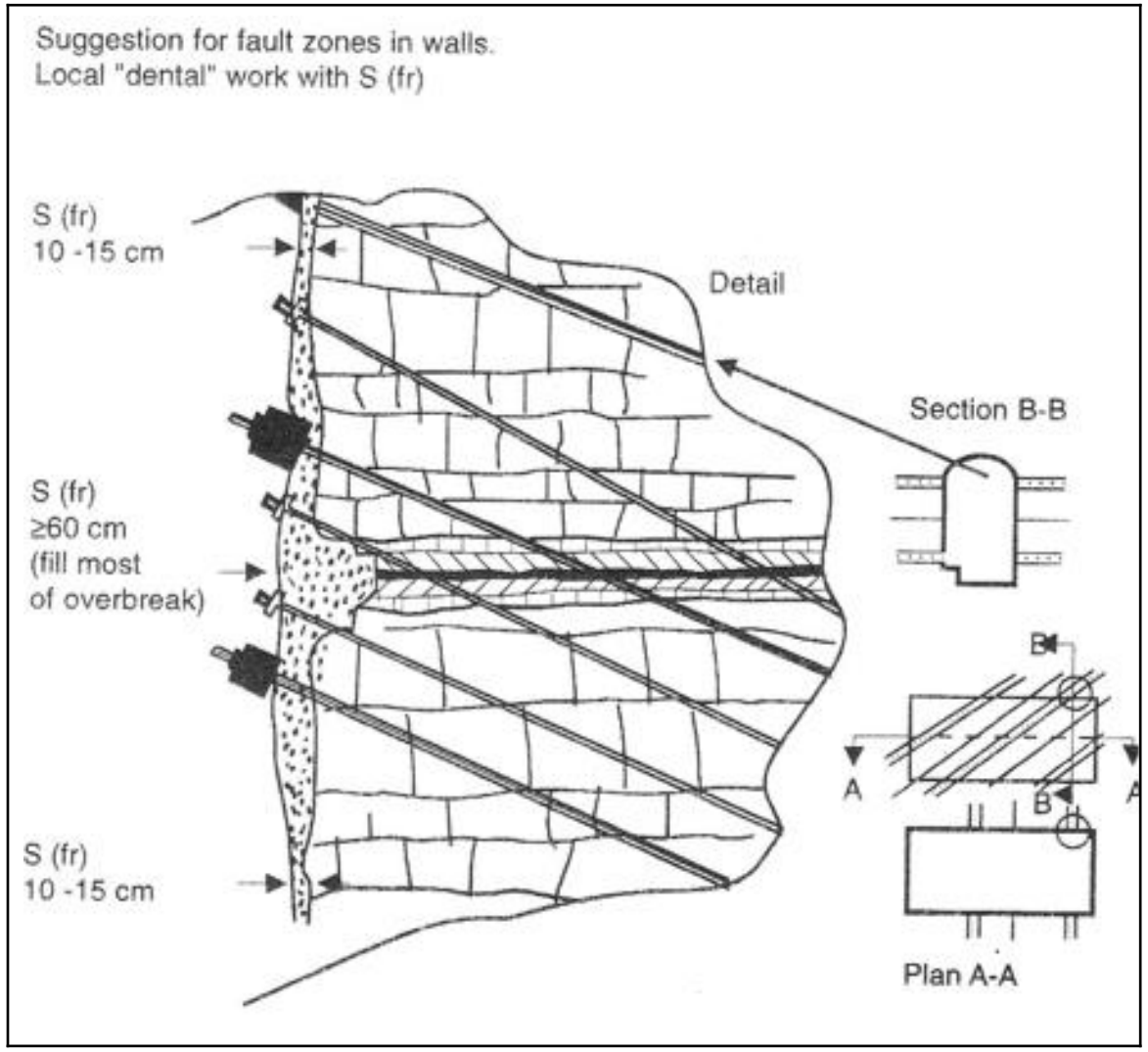
Additional Q-based support and reinforcement from inside the machine hall arch.

Rock mass TYPES 1, 2, 3.

Various bolts, cables, and 10 to 30cm S(fr).

The S(fr) was first time – in Taiwan (1987).





Wall reinforcement was drilled 20° *'downwards-and-forewards'* in order to cross and reinforce the dipping, clay-filled faults which were *left untreated in the long walls* of the cavern.

SEISMIC DESIGN PHILOSOPHY

Taiwan is an island subject to frequent earthquakes and consideration of potential effects on large underground openings at about 300m depth was required. The following seismic design philosophy was followed:

1) Seismic wave lengths are very long compared to the dimensions even of large caverns.

2) Significant differential strains across the caverns are therefore unlikely.

3) Amplification of seismic loading mostly occurs as softer surface sediments are reached.

4) Tunnel and cavern support should be designed to reinforce the rock mass and provide minimum contrast in stiffness compared to the rock mass (i.e. avoid thick, rigid concrete linings in softer rocks; they will not be needed in harder rocks!).

5) Earthquake loading has occasionally been known to cause shear strains in underground openings (i.e. increased permeability and flow into a mine with a dipping structure of bedded quartzite, non-parallel to principal stresses).

6) Faults and weak seams dipping at 31°—39° across the axis of the cavern (strike axis) may be under some residual shear stress (i.e. due to $\sigma_v > \sigma_h$, recent geomorphological adjustments etc.).

7) Cable and bolt reinforcement should therefore be designed to cross the faults and weak seams obliquely (i.e. non-parallel to strike direction) to minimise the consequences of shear strains (i.e. liner cracking) under severe earthquake loading.

Seismic Design Logic

Note: #10 ≥ 1.2

8) Surficial support (i.e. shotcrete) should be flexible and tolerate strains without significant cracking (i.e. apply good quality fiber reinforced shotcrete).

9) Conventional thick concrete linings will provide a poor solution to this special problem, due to their unwanted stiffness and their poor rock mass reinforcing character.

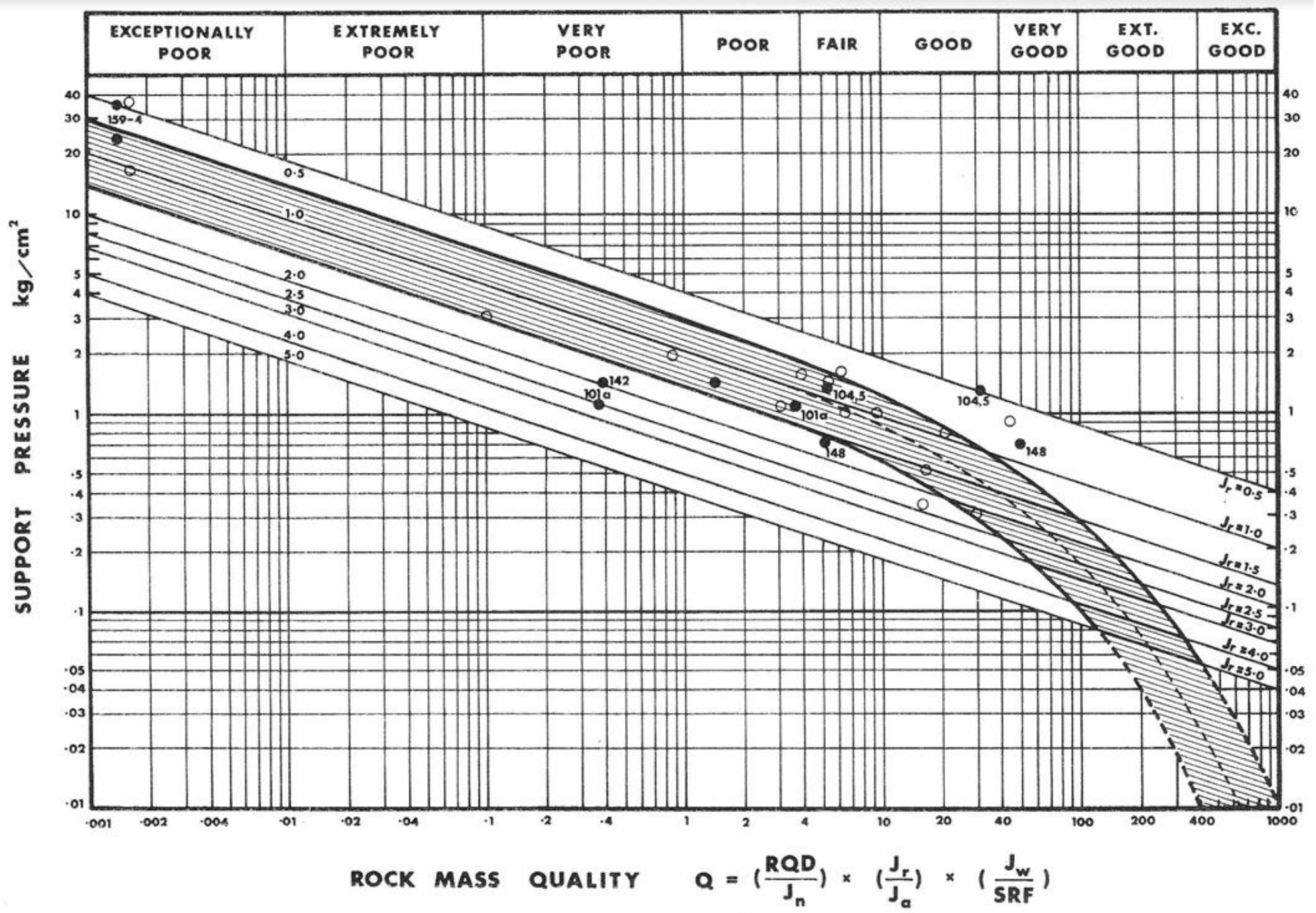
10) Dynamic, elastic, isotropic FEM analyses of lined and unlined caverns have demonstrated 10—20% (max.) amplification of stress levels (i.e. $\sigma_{dyn} \leq 1.2\ \sigma_{stat}$) due to the passage of P + S waves.

11) Assume, conservatively, that the present geological structure does not attenuate seismic waves any more than in isotropic, elastic FEM analyses.

12) Supplement the conservative approach adopted (high capacity cables, bolts, shotcrete with fiber reinf.) by using a seismic modification of the rock mass quality classification.

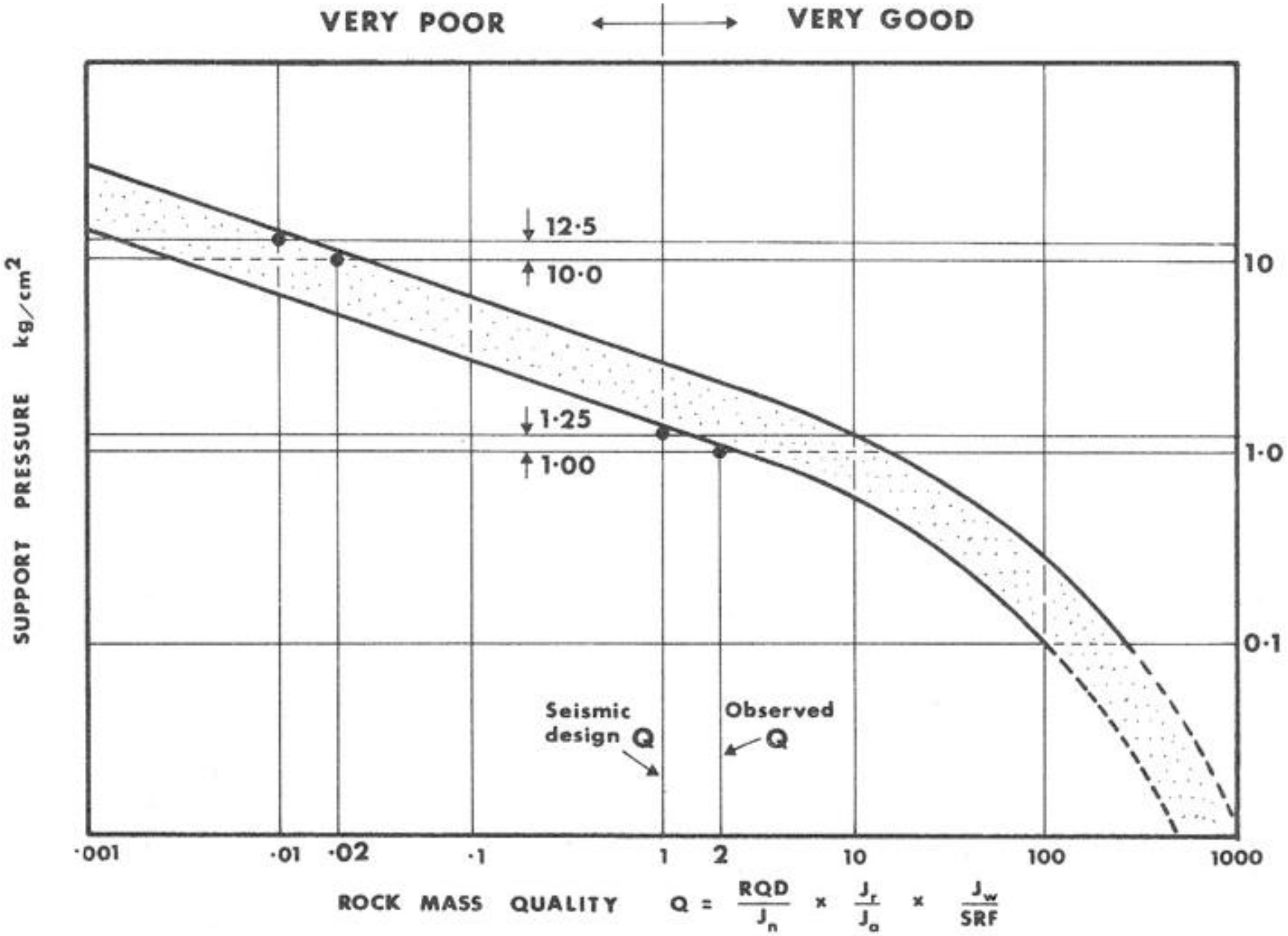
13) Assume $Q_{seis.} = ½\ Q_{stat}$ provides for a conservative increase in support capacity of about 25% (i.e. P of 1 kg/cm² → 1.25 kg/cm²) to account for the possible increases in $\sigma_{dynamic}$, i.e. assume that:

$$\underline{Q_{seis}} \approx \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{2.SRF} \approx ½\ Q_{static} \qquad (1)$$



NB et al. 1974 support pressure estimation

Used for bolt + cable capacity



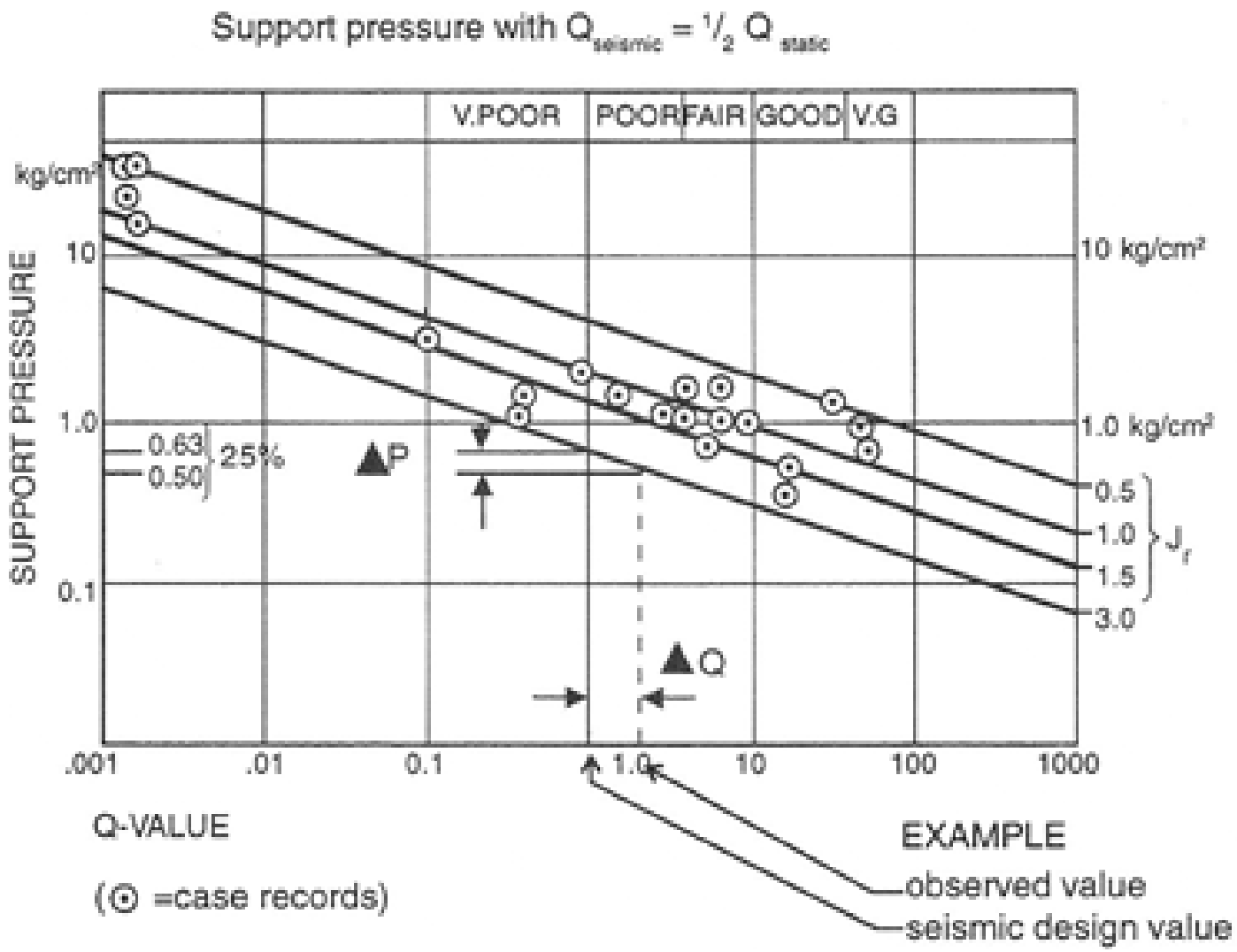
Assume, as FEM model, up to 25% increase in stress level with passage of seismic waves.

If we use 2 x SRF, get ½ x Q and this gives the wanted approximation:

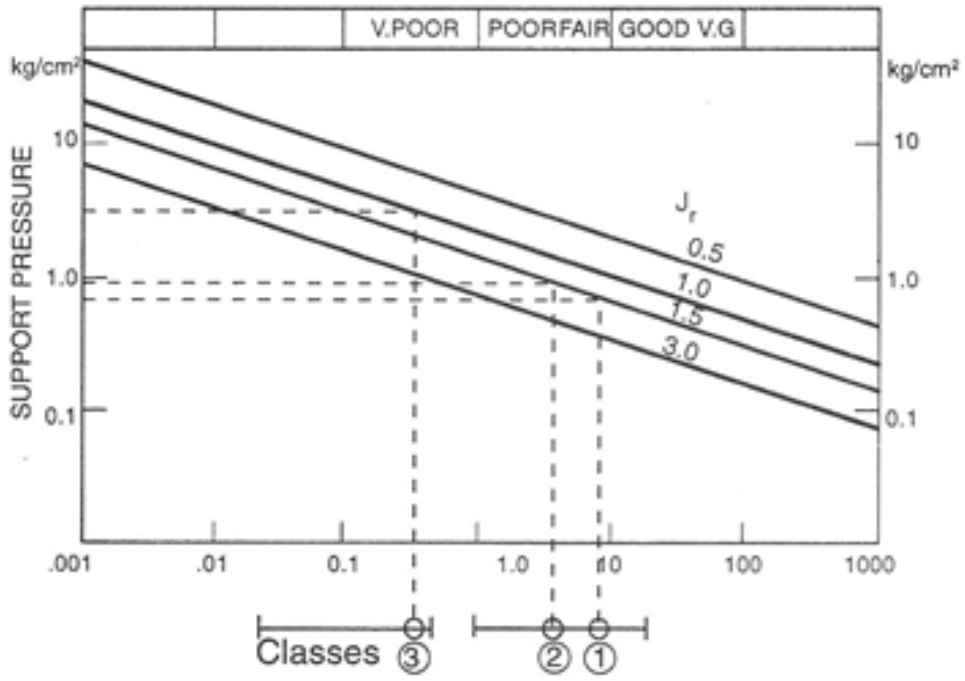
25% support pressure increase.

$Q_{seis} = ½ Q_{stat}$

$Q_{seis} = ½ Q_{stat}$



Surface peak ground accelleration PGAs were in the high range of 0.48 to 0.63g at Mingtan, yet there was no cavern damage.



The September 21earthquake's shallow epicenter was at Taiwan's geographic center and near Sun Moon Lake, a well-known vacation spot in Taiwan's less-populated central mountains.

Chi-Chi

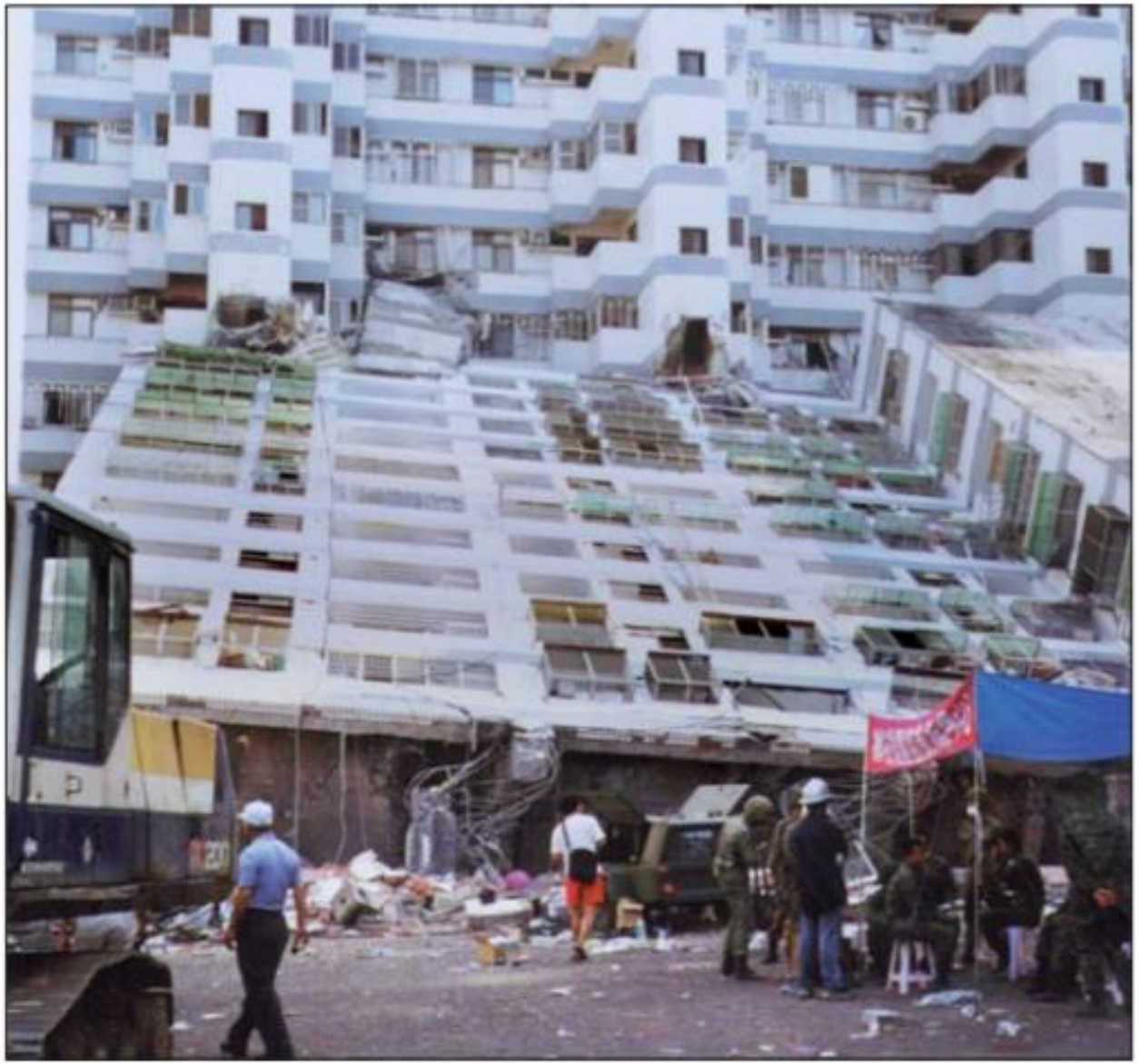
M = 7.7

This reproduction of the Central Weather Bureau's preliminary shaking map shows intensities of 0.8 g or greater in the epicentral region.

The epicenter of the earthquake was Chichi Township. The 921 Earthquake happened along the Chelungpu fault line in western part of the island of Taiwan. The fault is located along the foothills of the Central Mountains in Nantou and Taichung counties. Some sections of land near the fault were changed in elevation by as much as 7 meters (23 feet). Near the northern end of the fault line, a 7-meter tall waterfall was created by the earthquake. In the middle-western part of the island, bridges were destroyed. This stopped traffic for weeks.

- **(Other reports record M 7.6 and 7.7 with four aftershocks greater than M 6.5, peak ground accelerations as high as 1.0 to 1.2g at the southern end of the fault (near Sun Moon Lake and Mingtan) and velocities up to 300cm/s. Two tall buildings even in 150km distant Taipei were toppled. There was 30 seconds of intense shaking. A large section of mountain slid 2 to 3km.)**

  - 2,416 people died or were never found
  - 11,441 people were badly hurt
  - US$9.2 billion worth of damage
  - 44,338 houses were completely destroyed
  - 41,336 houses were badly damaged



The combined vertical weight of the building and earthquake overturning forces exceeded the load-bearing capacity of the columns, completely toppling some buildings, such as this one in Dali.

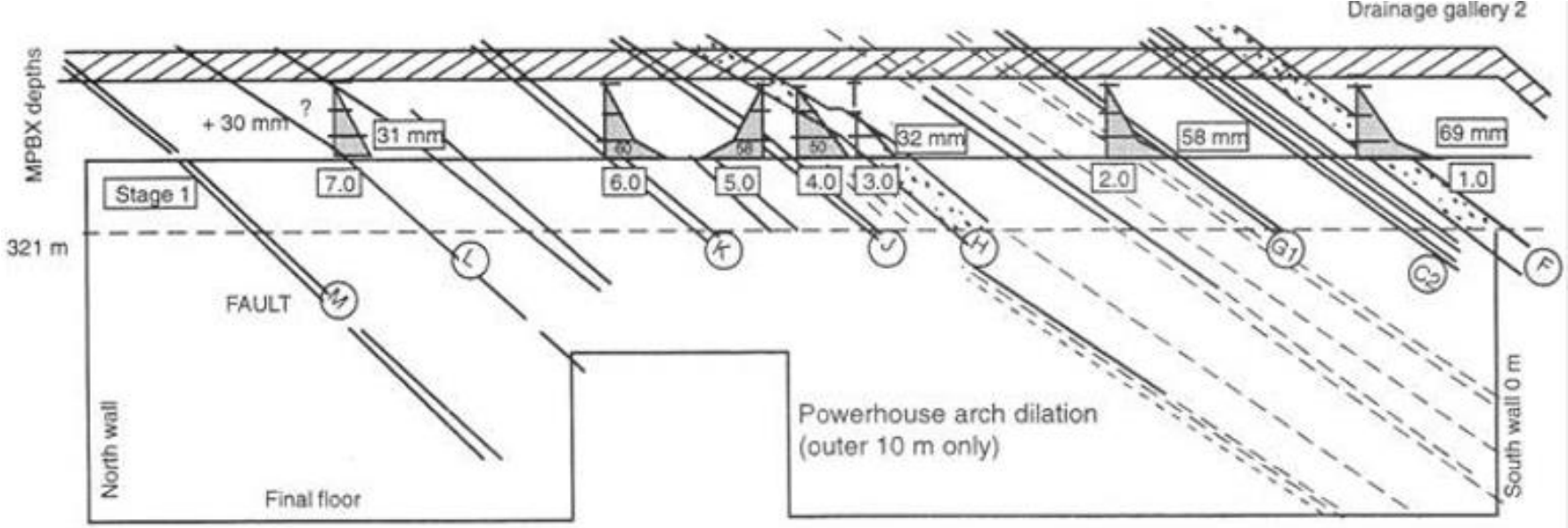
The severe loss of life and costs of building damage were each enormous.

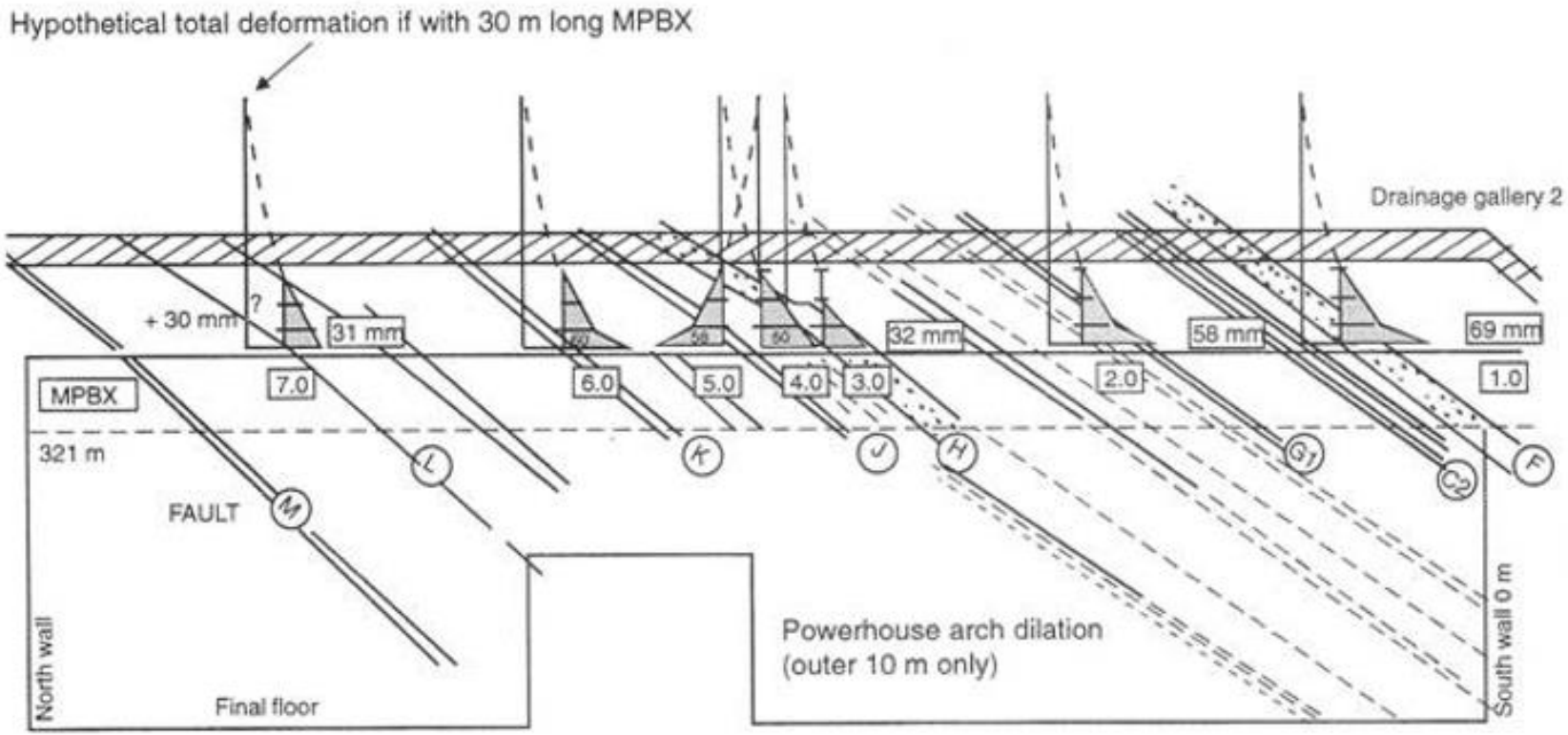
Two tall buildings in Taipei, 150km distant were toppled.

'The underground powerplant was in operation at the time of the earthquake and was undamaged although there was a loss of power supply and lighting underground.

- ***Several people were working in the plant at the time and apparently felt only minor shaking.***
- ***This is consistent with the notion that seismic ground motions at depth are less than those at surface'.***

- **From: A review of the performance of two large sub-stations and eight large dams during the Chi Chi Taiwan earthquake. R. G. Charlwood T.E. Little J.K. Lou April 2000.**
- **Institute for Catastrophic Loss Reduction, ICLR, U. of Western Ontario, info@iclr.org**





The pre-installed MPBX in the arch (from the drainage gallery) were not long enough.

Maybe should be 30m long?

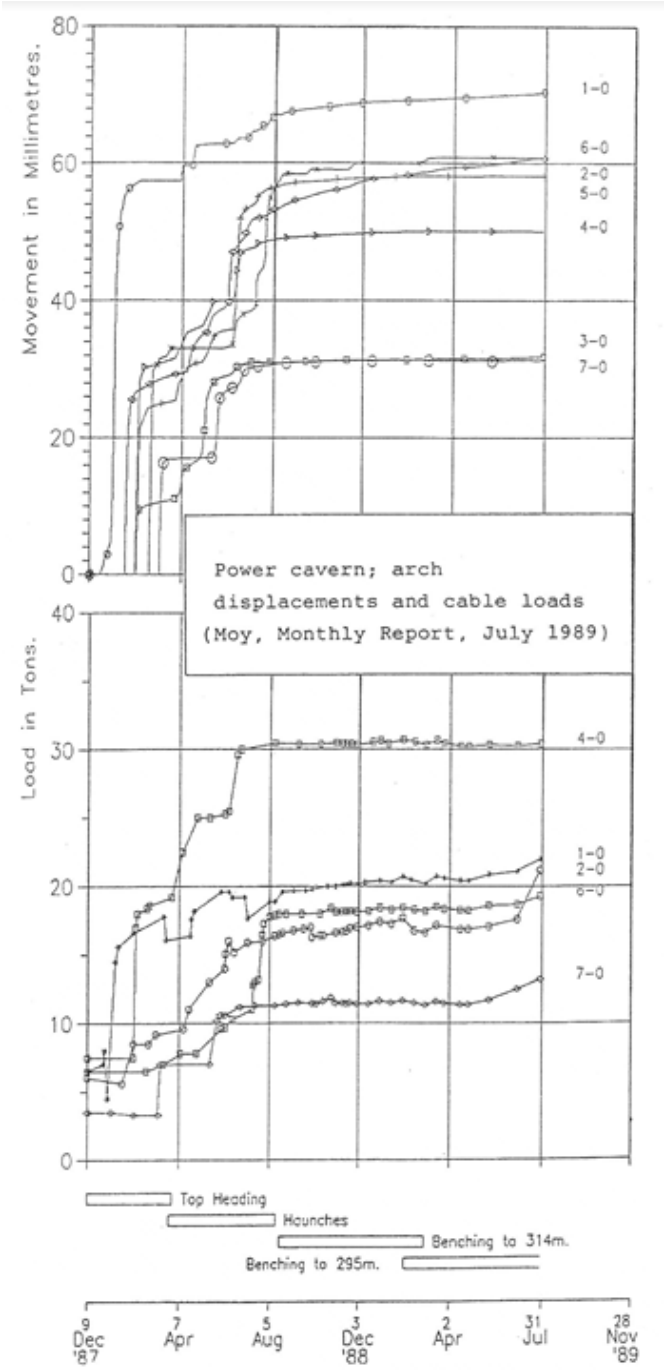
Special effects of anisotropic structure on deformation.

STRONGLY ANISOTROPIC STRUCTURAL GEOLOGY and therefore shear strength (in this case *weakness in shear*). Not only the rockmass reinforcement needed special treatment (delayed tensioning) but also the interpretation of the MPBX results.

In the arch a tendency for bedding-plane shear-related *shortening,* which counteracted the usual tendency for inward-directed radial deformation. In the walls it was the 'usual' inward-directed deformation.

Because of the unusual anisotropy in the arch there was no/little self-tensioning of the subsequently grouted anchor reinforcement in the arch, and more than normal anchor tensioning was therefore needed.

In the walls it was the opposite, with (presumed) slight but controlled down dip shear adding to the tension. This anchor loading was likely needed to protect the shotcrete from cracking.



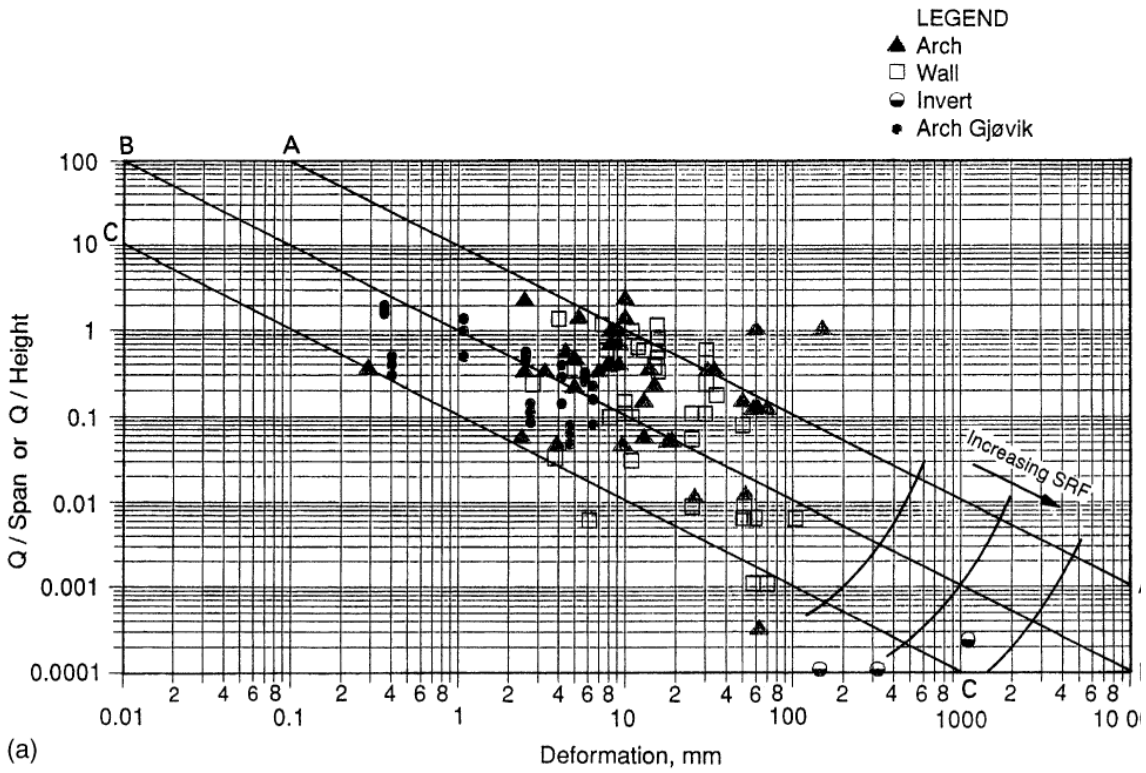
Some of the details of arch deformation control and anchor loading in this extremely anisotropic site.

Note the stabilization in the arch as the anchor load is increased.

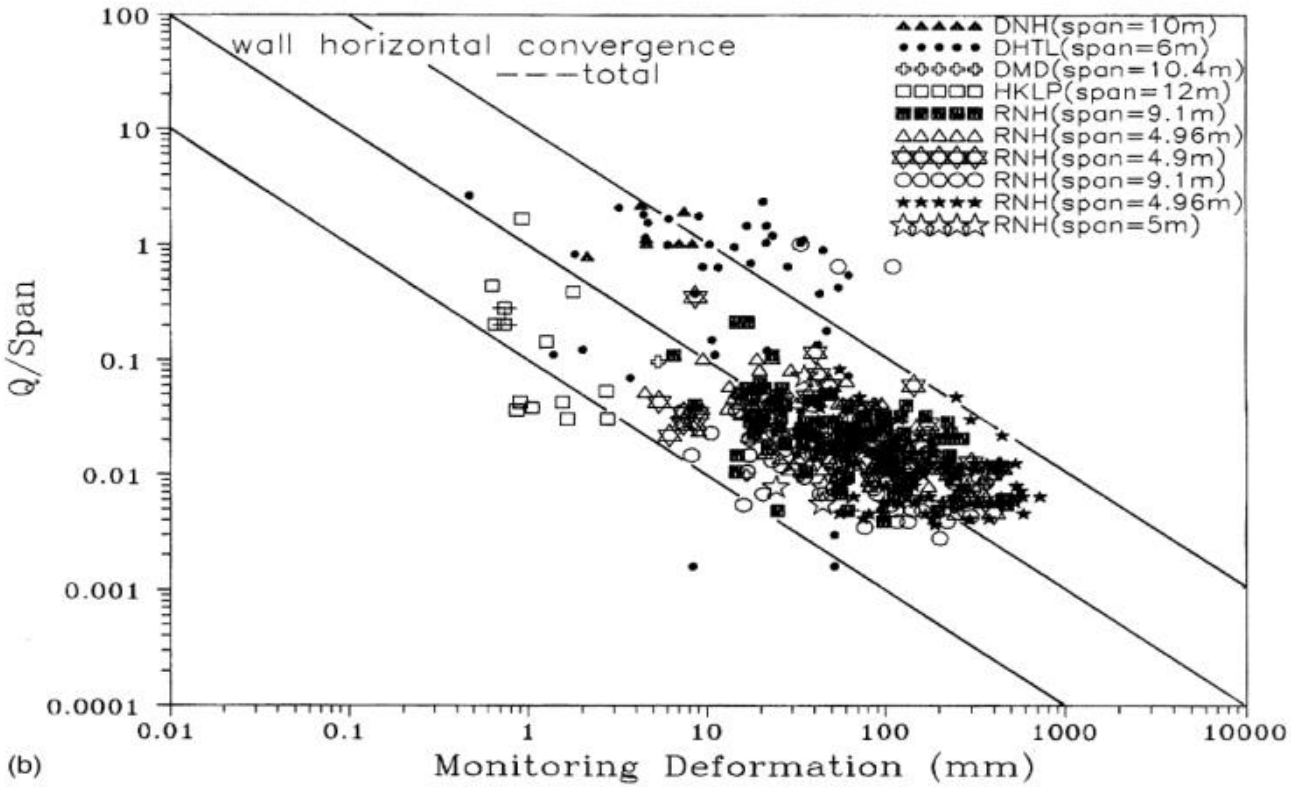
In the haunches (the arch/wall transition) anchor loads reached 50 to 70 tons, while in the walls anchor loads ranged from 50 to 90 tons and deformations from 10 to 40mm.

Timing of the cable tensioning was difficult to plan when crossing major potentially/or actually shearing faults, and may have been too early in some cases.
Sinotech/Golders, 1989.

Equation of central trend line (BB). Barton, 2002. RHS data: Taiwan.



$$\Delta = \frac{SPAN}{Q}$$



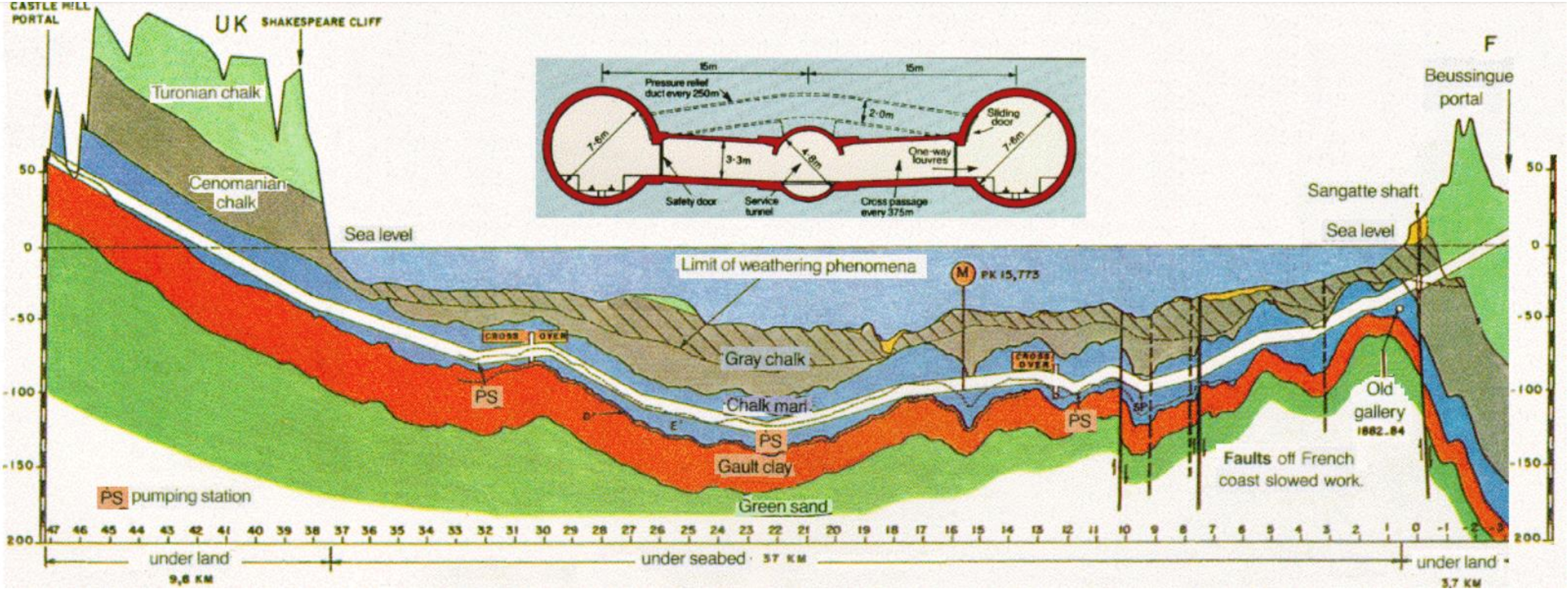
$$\Delta = \frac{SPAN}{Q}$$

Implied ‘integral’ Q-value range: 0.3 to 1.0

(Maximum Δ = 25 to 70mm. SPAN = 23m)
(Minimum Q of sandstone 1.1 to 1.7, maximum Q of bedding-plane-faults 0.4…….reinforced effect?)

CASE # 2

Q-APPLICATION ON THE UK SIDE OF THE CHANNEL TUNNEL



The ≈ 50km long Channel Tunnel (UK to France).

Jointing was not expected in the Chalk Marl since UCS only 4 to 7 MPa.

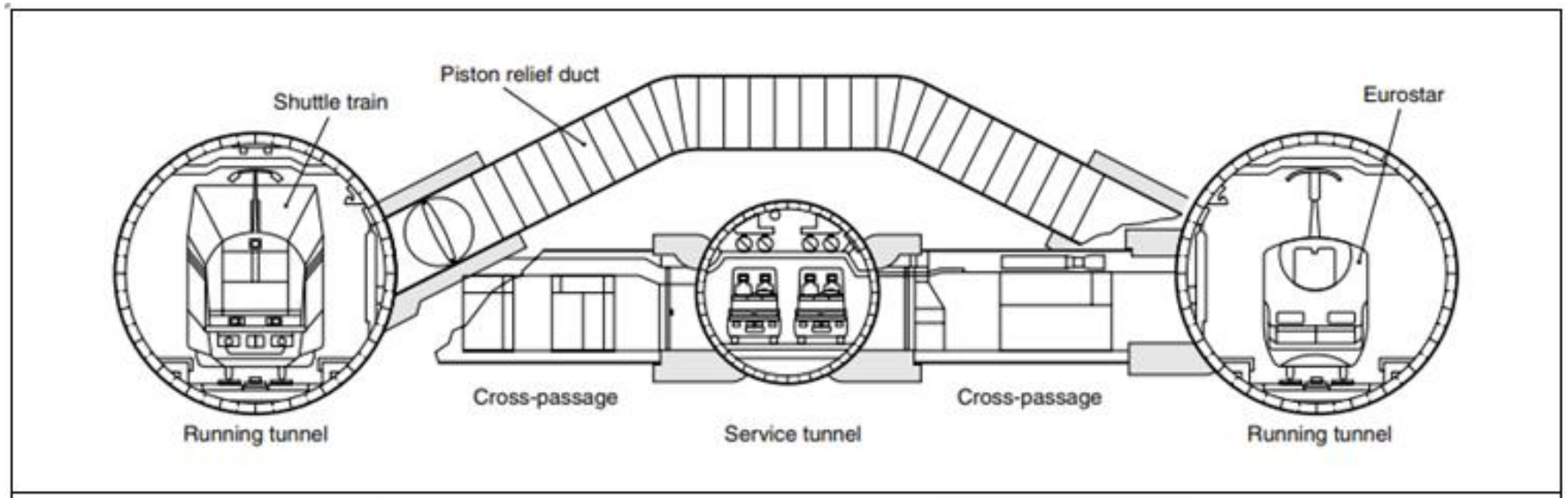
(Figure from P-J. Pompée.)

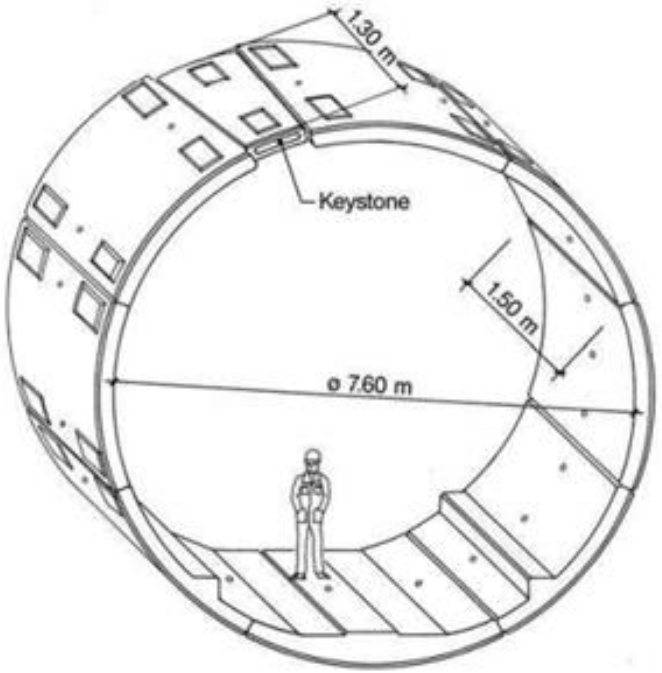
NB's Consulting Brief

As a result of difficulties and initial delays caused by overbreak and by corrosive seawater inflow in some of the UK sub-sea TBM drives (specifically ch. 20-24km) the writer was requested to independently assess the rock quality in existing tunnels in chalk marl in the neighbourhood of the Channel Tunnel which was now under construction. (NB shown TML Q-logging only later).

The contractor consortium Trans Manche Link (TML) were preparing a big claim about 'unexpected conditions'. What rockmass conditions could have been expected – in the chosen chalk marl?

The work was performed during 1990 and 1991 under contract to Dr John Sharp of GeoEngineering whose company was conducting a major review for Eurotunnel.





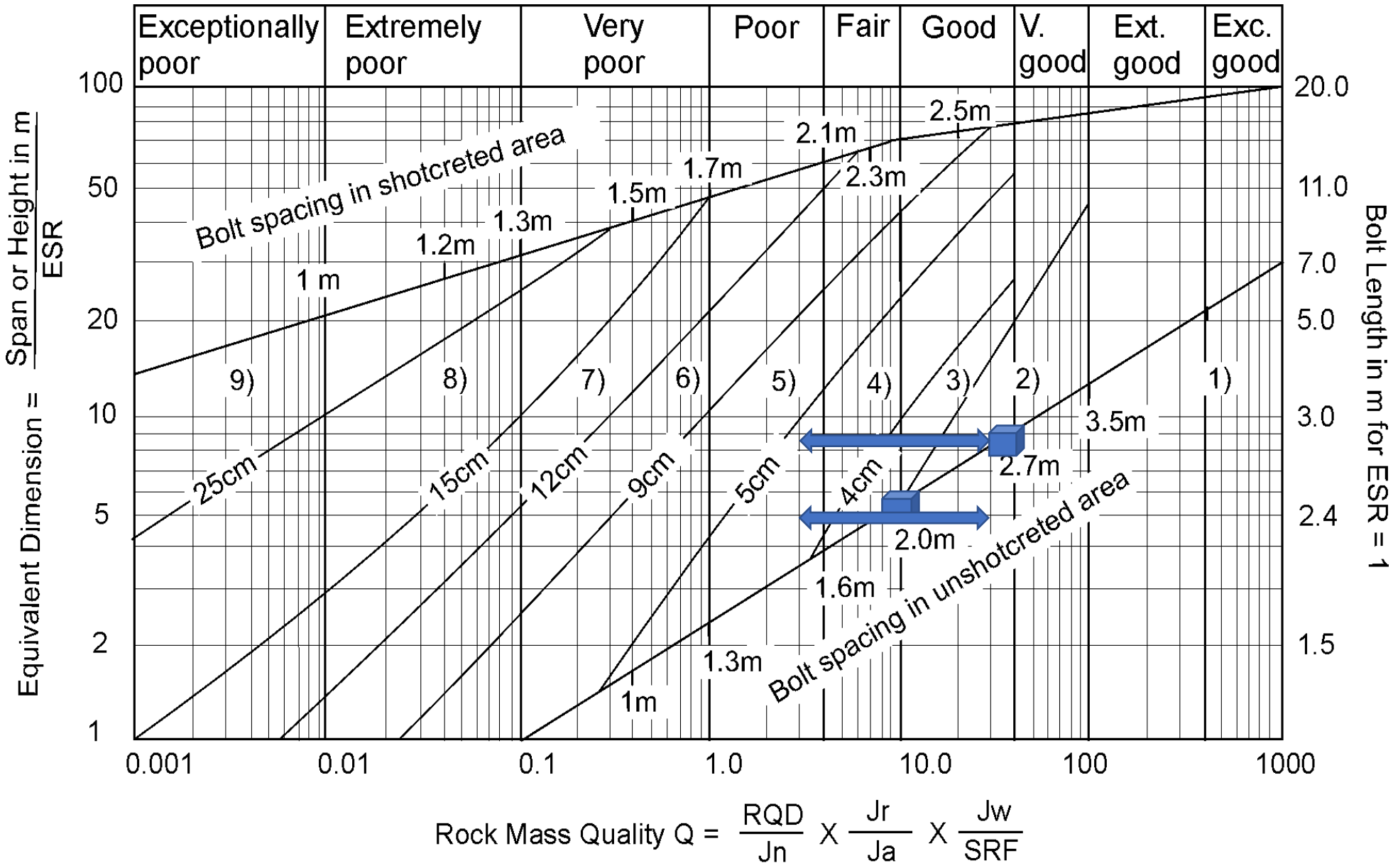
OVER-BREAK DUE TO JOINTING WAS 'NOT EXPECTED'.

THEREFORE, *UNBOLTED PC ELEMENTS* WERE USED TO LINE THE TBM TUNNELS ON THE UK SIDE.

Sea-water infiltration was thought about as there were measures to pre-inject the ground ahead of the running tunnels – from Service Tunnel.

Note adverse length of unsupported ground: (15-18m from cutter-head).

Problems where there were lower Q-values, and larger ratios of **Jn/Jr (km 20-24).**

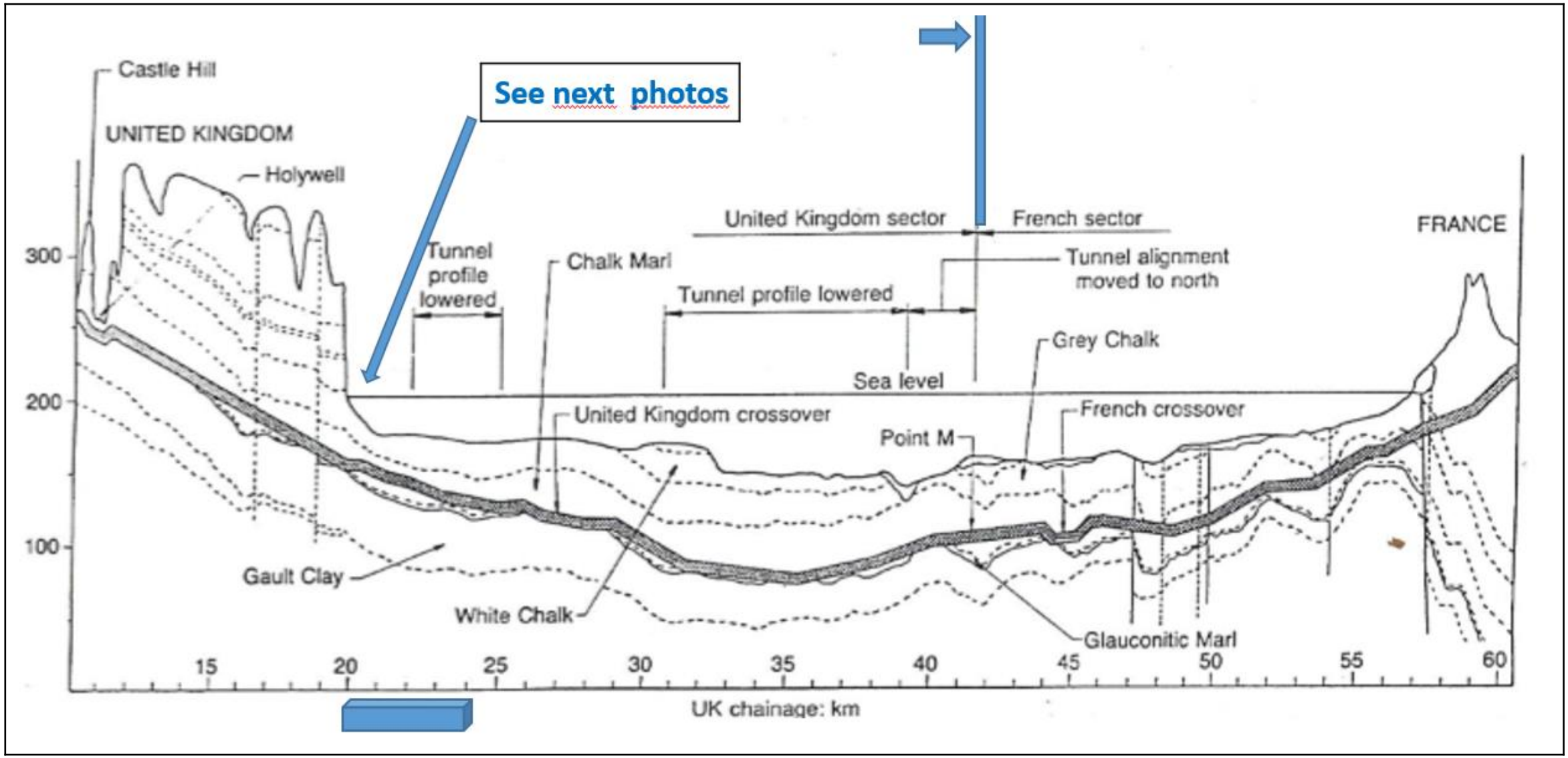


Tunnels of 5.3m and 8.4m span require Q ≈ *10* and ≈*40* for no support to be required.

Typical range of Q = 3 to 30.

So actual stability could be in doubt for 15 to 18m from cutter-head.

Simplified Q-chart Grimstad and Barton, 1993



The weak chalk marl was strong enough to resist erosion at low-tide, where long and very planar, steeply-dipping joint traces could be seen below Shakespeare Cliff. These joints do not seem to have been taken as serious threats to over-break, but hindsight is easier than foresight.

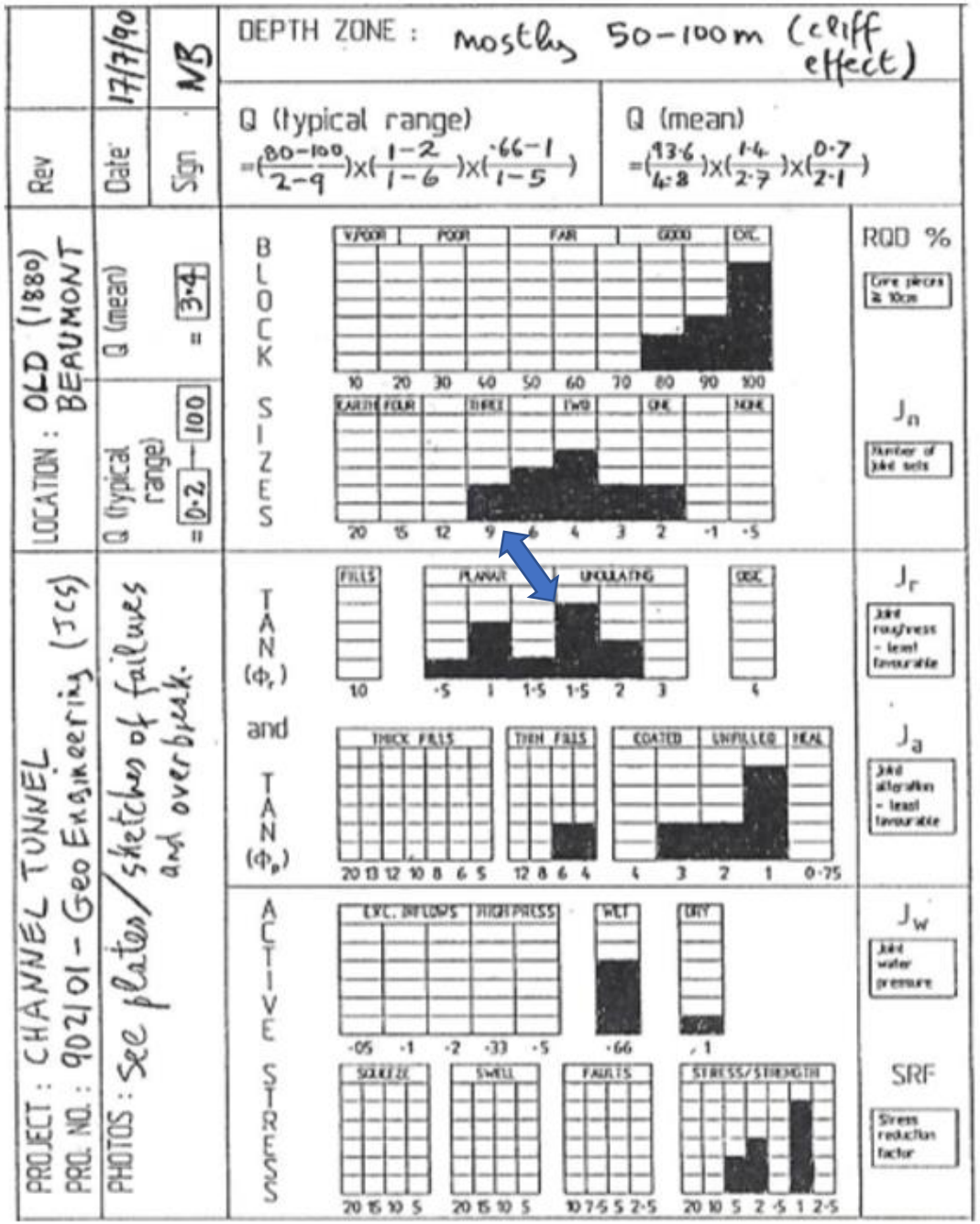
INDEPENDENT LOGGING WAS KEY INSTRUCTION

- The regular Q-system of rock mass classification was used (several years earlier) by TransManche Link (TML) in the Marine Service and Running Tunnels – also in the difficult chainages.

- The writer's classification of the grey chalk at Shakespeare Cliffs and of the **chalk marl in the Beaumont and Terlingham Tunnels** was performed prior to knowledge of conditions in the Marine Service Tunnel (MST) or in the Marine Running Tunnels (MRT).

- The PB series of core logs and photographs for marine drill core PB1 to PB8 was also classified by NB without prior knowledge of the MST or MRT conditions.

EVEN THE WEAK CHALK MARL OF THE CHANNEL TUNNEL *WAS JOINTED* AND THEREFORE Q-LOGGED (BY TML, later by NB). **Jw 0.66**



TOO MANY JOINT SETS, TOO PLANAR JOINTS

- The remarkably planar and continuous jointing in the chalk marl can be seen at low-tide beneath Shakespeare Cliffs on the north side of the English Channel.
- It can also be seen where there is over-break in pre-existing tunnels.

- Top-centre photo is the old (1880) Beaumont/English (part-pilot) Tunnel.
- Below that is similar jointing in a pressure-relief duct. (Warren and Sharp in red).
- The hand-filled (black) Q-parameter histogram on the right for Beaumont:
- $Q_{\text{most frequent}} = 100/4 \times 1.5/1 \times 0.66/1 = 25$.

- **But when the combination Jn/Jr is for instance 9/1.5 (three sets/rough-planar) or *9/1.0* overbreak is inevitable. *This was the problem in 20-24 km.***

JOINTING *WAS* EVIDENT IN NEAR-BY TUNNELS!

Top-left, clockwise:

Terlingham Water tunnel,

Beaumont TBM Tunnel, 1880: tidal influence, stress/STRAIN-failure, wedge-failure.

Three photos separated by 150 m.

(BOTH TUNNELS CLOSE TO CHANNEL TUNNEL)

An example of sub-sea-bed jointing in the chalk marl. Three sets of joints are suggested with marked planarity (Jr = 1.0?).

Jn/Jr = 9/1.0 (= inevitable overbreak)

For unknown reasons, this adverse jointing did not alert designers to the risk of over-break and the need for *bolted* (TBM) tunnel segments.

As usual the benefit of hindsight makes cause and effect much easier to appreciate after the problem has occurred.

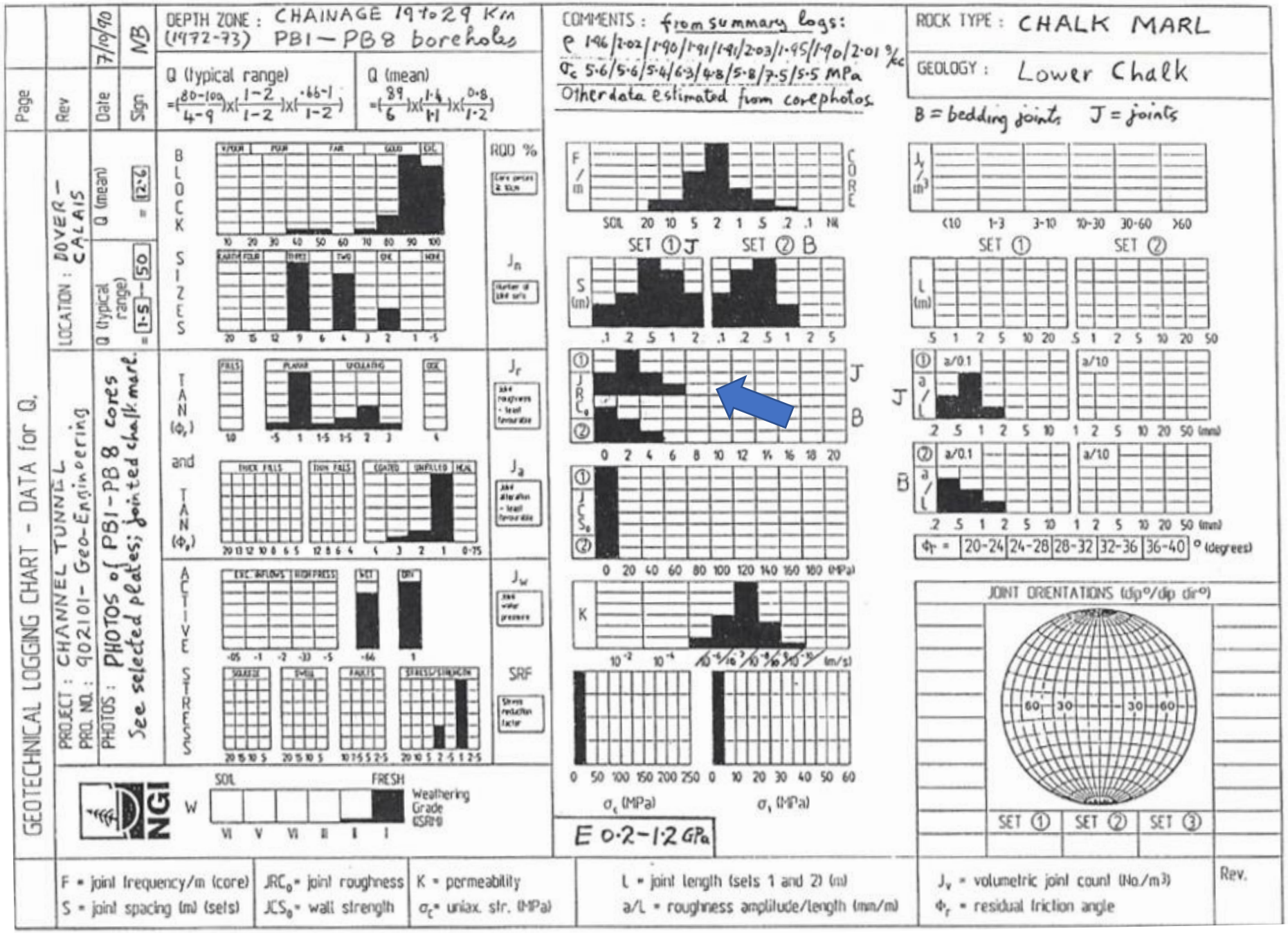
Comparison of independent Q-logging

TML (*Trans Manche Link contractor consortium*) logging of MST in the problematic sub-sea chainage 20 to 30km, and especially problematic 20 to 24 km.

Author (NB) logging of precedent sources: Terlingham Tunnel, Beaumont/English Tunnel, marine core PB1 through PB8 (the latter was photo-logging of Q).

On a six-orders of magnitude Q-scale the results are rather close.

TML mapping during construction of the MST:		
km 20–30	Q (range) = 0.3–100	Q (mean) = 22.9
km 20–24	Q (range) = 0.3–40	Q (mean) = 9.9
km 20.5–21.3	Q (range) = 0.7–20	Q (mean) = 5.6
Author's estimates from pre-construction sources:		
PB1 to PB8	Q (range) = 1.5–50	Q (mean) = 12.6
Terlingham Tunnel	Q (range) = 1.3–50	Q (mean) = 10.6
Beaumont Tunnel	Q (range) = 0.2–100	Q (mean) = 3.4



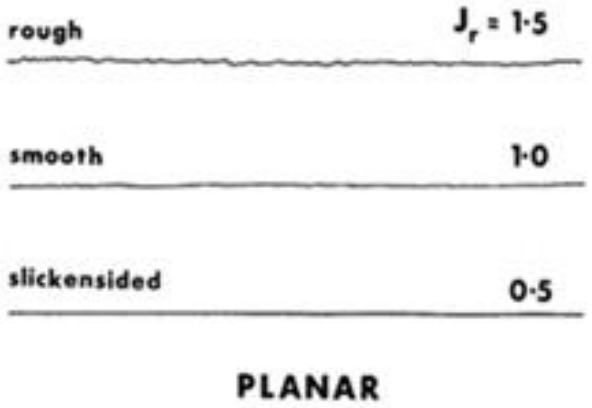
Eight marine borehole cores (Q-log in left column).

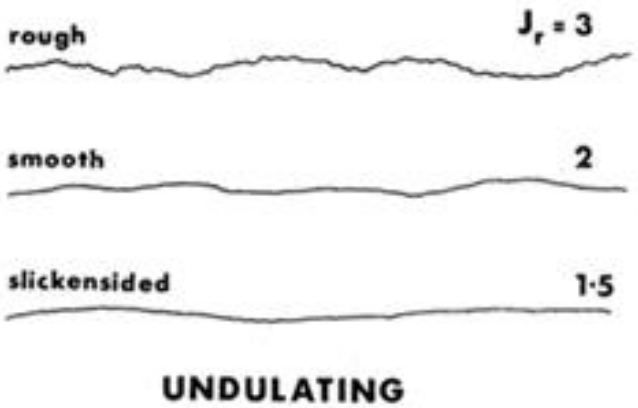
Most frequent Jn/Jr = 9/1 implies over-break.

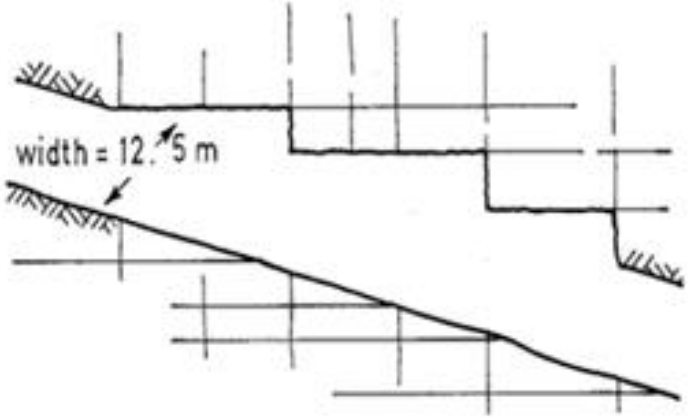
This is known with the benefit of hindsight, and dates to the Barton, 2007 Jn/Jr theory which also applies to caving in the mining industry.

Note also the low (quite planar) JRC statistics in the central area.

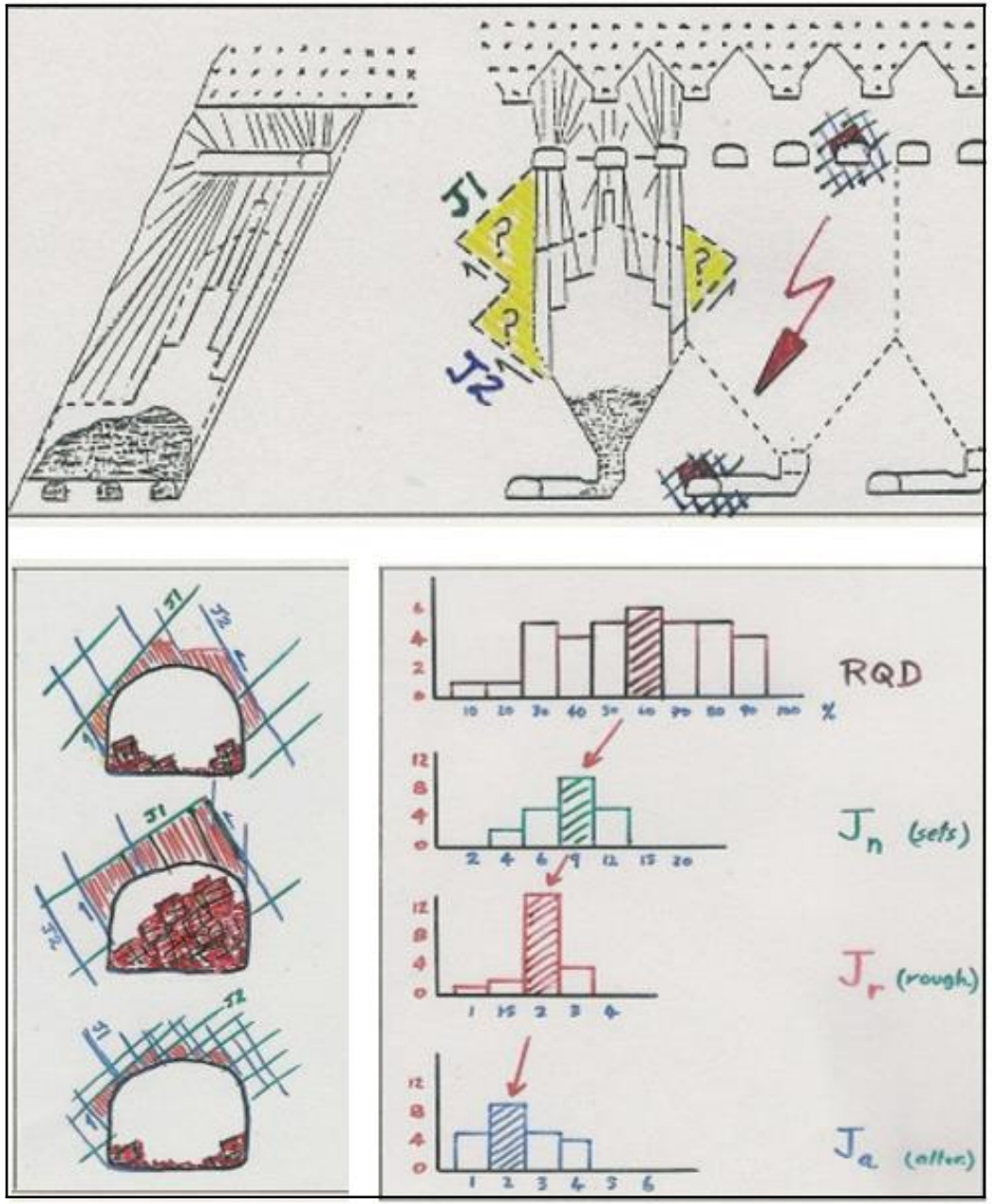
Joint roughness and a tendency for dilation will have been minimal.





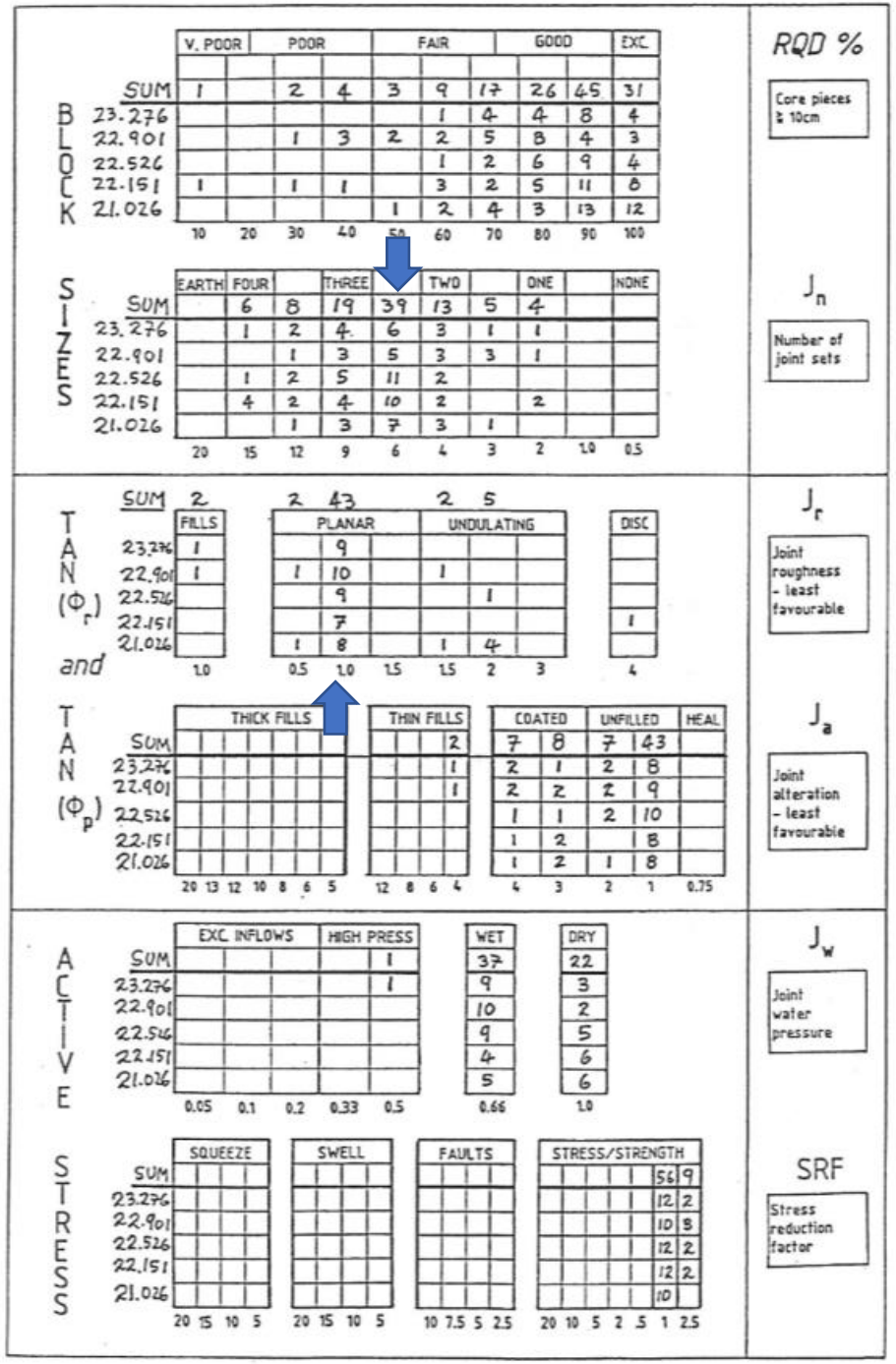


4/2….6/2….9/2….9/3….12/3….15/3…….6/1….9/1.5….12/2….15/2



- **Joint sets J1 and J2 had adverse *Jr/Ja* ratios in some cases (see outliers in the histograms).**
- **The adverse ratios of *Jn/Jr* were of most importance.**
- **<u>Jn/Jr ≥ 6 meant over-break</u>**

Demonstration of overbreak in mining (long-hole-drilling) drifts, and potentially at stope scale.



The Q-parameter statistics for some of the well-documented *TML logging* in the MST and the MRTs. (Marine Service and Running Tunnels)

At the top of each column of parameter ratings, the sum of frequencies is given.

The most frequent combination of Q-parameters is likely to be:

Q = 90/6 x 1/1 x 0.66/1 = 10

(Most frequent: Jn/Jr = 6/1)
two sets plus random/smooth-planar joints

TML problems explained. Precedent info from tunnels in chalk marl: also

PRE-CONSTRUCTION SOURCES OF ROCK MASS QUALITY FOR THE CHALK MARL.

THE WRITER USED Q-LOGGING OBSERVATIONS IN THE OLD BEAUMONT TUNNEL, TERLINGHAM TUNNEL.

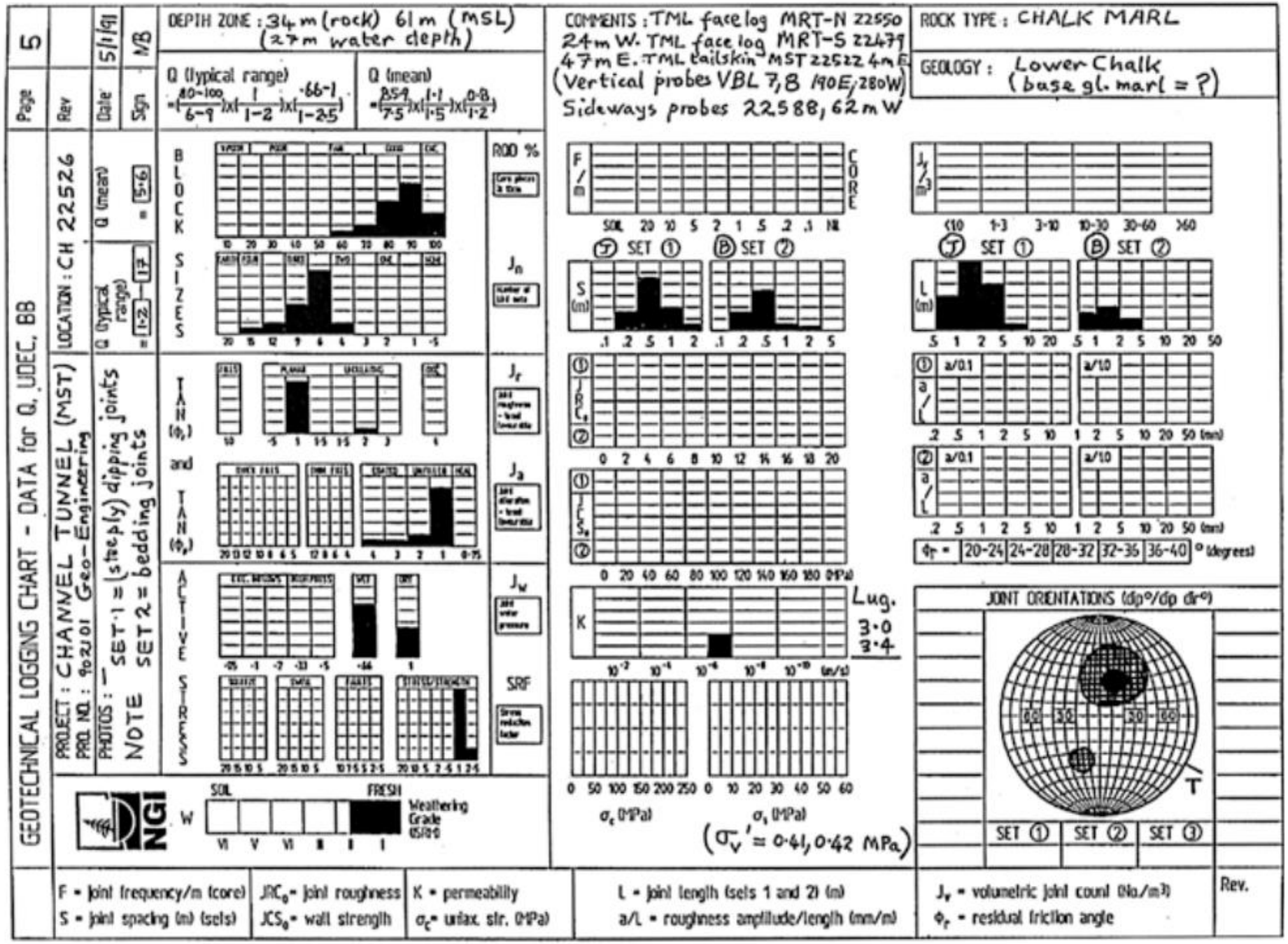
PHOTO-LOGGING OF PB1 TO PB8 CORE

Parameter	(mean)	Description	Sum/number of observations	Typical range
RQD	90.1	Excellent	6940/77	90–100
J_n	6.5	Two joint sets plus random	473/73	4–9
J_r	1.5	Rough, planar joints	93/63	1–2
J_a	1.5	Slight alteration	79/53	1–2
J_w	0.9	Slight water inflow	42/48	0.66–1.0
SRF	1.5	Slight stress problems	74/48	1–2

Data from	Most frequent	Next most frequent	Possible problem ground
Predicted conditions (Pre-construction data)	$Q = 11.1$ $\left(\frac{100}{9}\right)\times\left(\frac{1}{1}\right)\times\left(\frac{1}{1}\right)$	$Q = 7.4$ $\left(\frac{90}{4}\right)\times\left(\frac{2}{2}\right)\times\left(\frac{0.66}{2}\right)$	$Q = 1.7–3.3$ $\left(\frac{90}{9}\right)\times\left(\frac{1}{2}\right)\times\left(\frac{0.66}{1-2}\right)$
Encountered conditions (typical: km 20–24)	$Q = 13.2$ $\left(\frac{90}{5.8}\right)\times\left(\frac{1}{1}\right)\times\left(\frac{0.9}{1}\right)$	$Q = 2.6$ $\left(\frac{88}{6.6}\right)\times\left(\frac{1.4}{2.5}\right)\times\left(\frac{0.8}{2.3}\right)$	$Q = 1.8–4.3$ $\left(\frac{88}{6.6}\right)\times\left(\frac{1}{2.5}\right)\times\left(\frac{0.8}{1-2.3}\right)$
Encountered conditions (Poorest: km 20–24)	$Q = 10.0$ $\left(\frac{90}{6}\right)\times\left(\frac{1}{2}\right)\times\left(\frac{.66}{1}\right)$	$Q = 3.0$ $\left(\frac{100}{9}\right)\times\left(\frac{2}{3}\right)\times\left(\frac{1}{2.5}\right)$	$Q = 0.9–2.2$ $\left(\frac{90}{9}\right)\times\left(\frac{1}{3}\right)\times\left(\frac{0.66}{1-2.5}\right)$

Comparison of TML and the writer's Q-estimates at well-documented MST and MRT chainages between km 19.8 and km 27.2. (On a 10E-3 to 10E+3 log-scale these are close)

Chainage (± 50 m)	Author's estimates of Q		TML estimates of Q	
	Q (range)	Q (mean)	Q (range)	Q (mean)
1. 19,824 km	7.5–50	17.6	2.4–80	28.9
2. 19,925 km	0.9–25	7.5	1.7–40	11.4
3. 20,651 km	1.0–100	7.8	1.6–40	11.7
4. 21,026 km	1.7–50	9.4	4.4–13.2	7.4
5. 22,151 km	1.2–17	5.7	2.4–19	7.0
6. 22,526 km	1.2–17	5.6	3.0–6.7	4.9
7. 22,901 km	0.5–20	2.6	1.8–10.7	4.9
8. 23,276 km	0.5–25	3.7	1.1–8.2	4.0
9. 23,651 km	1.5–25	5.7	2.5–13.3	7.9
10. 27,025/167 km	9.9–133	12.4	3.0–80	16.2



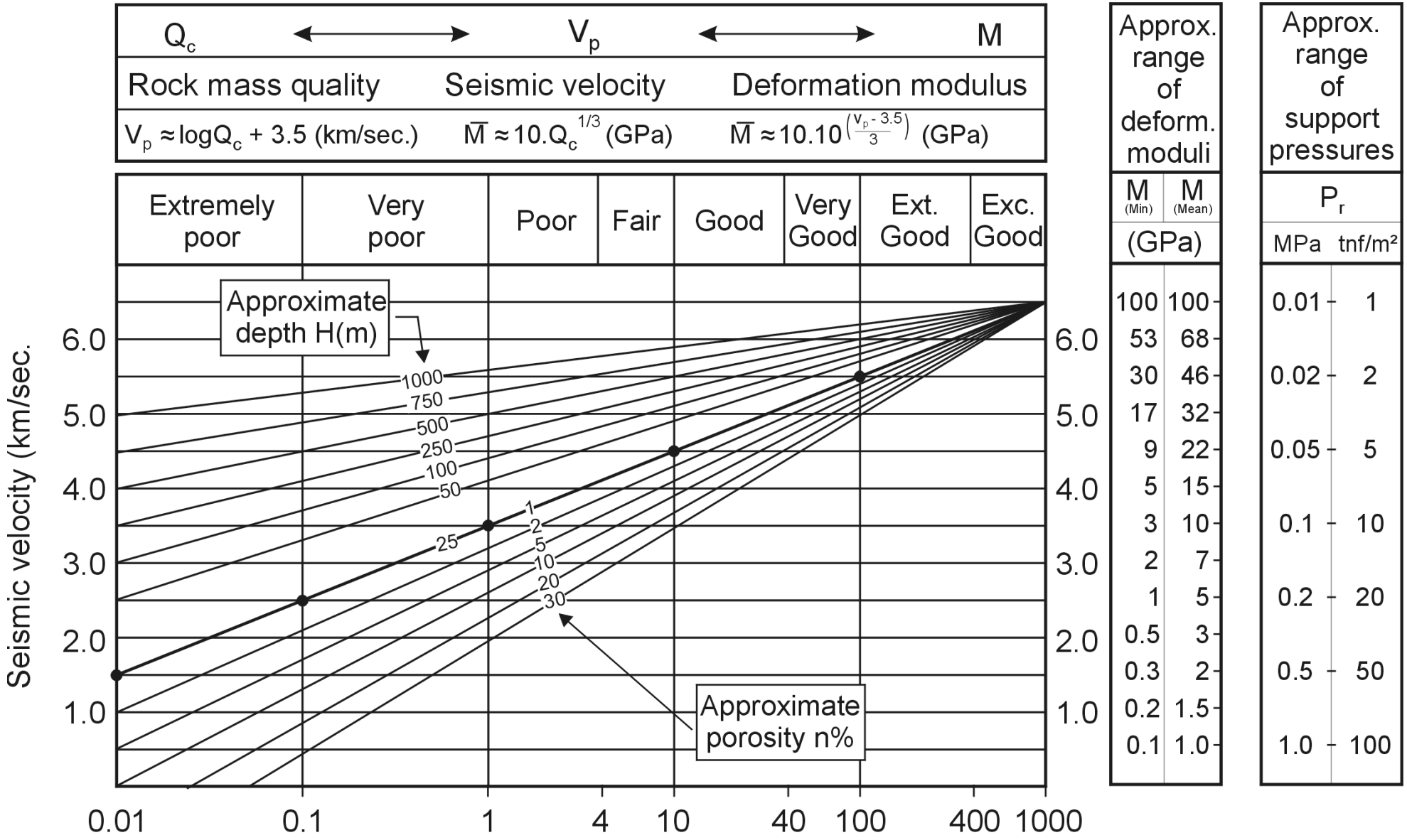
Example of the TML Q-parameter and joint statistics at a well-documented section from the MRT and MST on either side of ch. 22,500m.

Note that the most frequent ratio of Jn/Jr is 6/1, but 9/1 is also frequent. **Both suggest overbreak since Jn/Jr ≥ 6** (Barton, 2007).

See also joint spacing and length statistics.

SEISMIC Q-Vp CORRELATION CHART

- **Seismic correlation chart (NEXT SCREEN):** weighted mean value of Q = 8 from precedent study (PB1 to PB8 core, Terlingham and Beaumont tunnels). This value is close to the TML mean of Q = 9 for km 20–24 obtained from all the face logs in the MST, MRTN and MRTS.
- **Since $Q_{mean} \approx 8$, and σ_c (mean) ≈ 6 MPa, therefore $Q_{c\ mean} \approx 8 \times (6/100) \approx 0.5$.**
- This Q_c value intersects the reference diagonal line at V_P = 3.2 km/s. Correction for average porosity (*n* = 27.7%) results in a reduction of 1.6 km/s giving $V_P \approx 1.6$ km/s. However, tunnel depths of, for example, 40 m brings this value up to about **2.0 or 2.1 km/s. Note: with higher Q, get higher Qc, and higher Vp.**
- An uncertainty in the above correlations which potentially show excellent agreement with the offshore geophysics (**typical V_P = 2.0–2.6 km/s**) is the effect of water depth and effective stress.
- An undisturbed, fully *confined* modulus prediction of 3–5 GPa, and a **fully *disturbed* modulus prediction of 0.5–1 GPa (left column of seismic chart)** may well be in line with the assumed **near-tunnel values of about 0.8–1.4 GPa** that were derived by back-analysis of the TBM tunnel deformation measurements by Eves and Curtis, 1992.



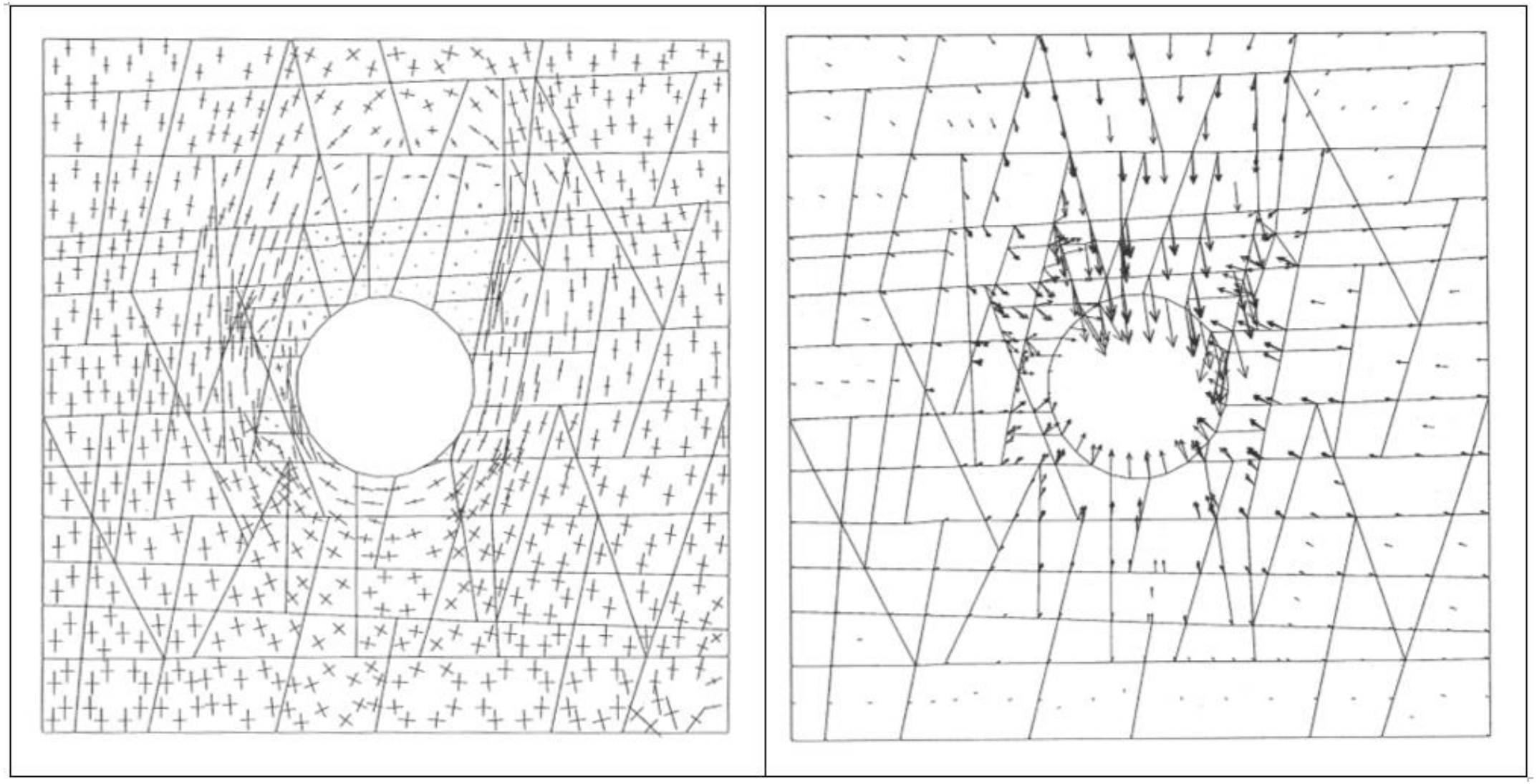
$$Q_c = \left[\frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}\right] \frac{\sigma_c}{100}$$

Empirically derived correlation chart for **V_P from/to the Q-value**, with correction for depth (+ve) and porosity (-ve) and uniaxial compressive strength. Barton, 1995, 2006.

(8, 0.5, 3.2, -1.6, but 40m depth, Vp 2.0-2.1)

OVERBREAK AND WATER INFILTRATION INEVITABLE

- Overbreak with blocks falling onto the trailing fingers of the TBM was inevitable in many locations in this sub-sea chainage. In general, remarkable consistency was shown between the “blind logging” of precedent sources of data for the chalk marl (Terlingham and Beaumont tunnels, PB-series sub-sea cores), and the extensive logging in the three TBM drives by the contractor TML. The latter data sets were subsequently provided to the author for comparison.



The tendency for troubling over-break and water ingress in the adverse chainages of the tunnel gain some support from this UDEC-BB model by NGI colleague of the time (1990) Axel Makurat. Low effective stresses in the horizontal direction, combined with the low JRC meant that predicted vertical deformation, and maximum joint shearing could be several millimeters. Note the poor arch action. This was a 50m deep sub-sea-bed simulation of the 5.3m diameter MST. (Pers.comm. Makurat, 1990).

CONCLUSIONS - REGARDING Q-APPLICATION IN FAULTED OR WEAK ROCK

1. The Q-system has been successfully used for designing and executing bolt, anchor and fibre-reinforced shotcrete reinforcement and support for a large machine-hall cavern in some of the most adverse rock mass conditions ever utilized for large caverns. Thirteen bedding plane faults with clay fillings, up to 1 and 2m thick in the worst case, crossed the cavern axis, and the locally relevant Q-values consequently varied on either side of 0.1 in the 'extremely poor' and 'very poor' categories, much lower than in the intervening jointed and bedded sandstones.

2. A novel 'downwards and forwards' (20°/20°) bolt and anchor installation (not the usual radial-perpendicular 0°/0°) was used. This effectively limited down-dip shearing reaction to excavation and benching-down. This, and additional reinforcement in the arch, was designed with a capacity increased by 25% due to the use of a dynamic design rule-of-thumb of 2 x SRF or $Q_{dynamic} = ½ Q_{static}$.

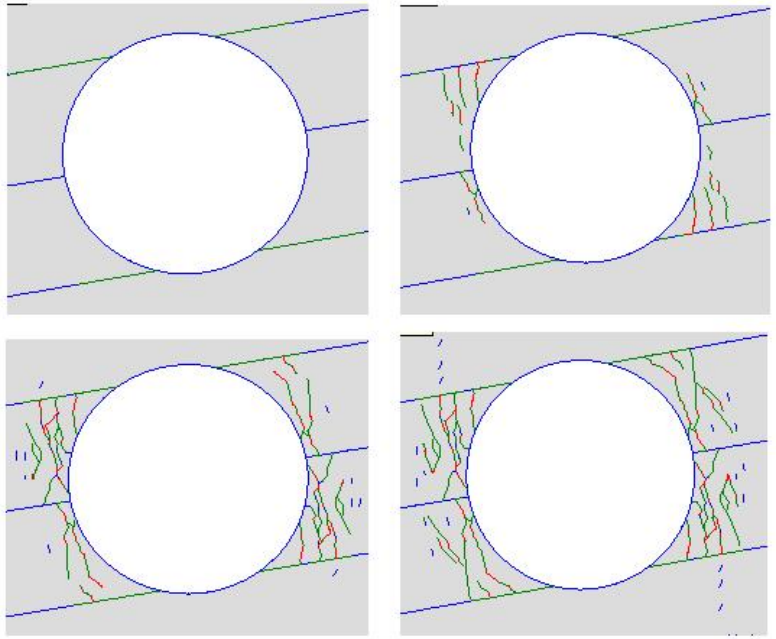
3. The cavern complex was tested severely some years later during the 30 seconds duration M7.7 Chi-Chi earthquake, which had an epicentre 12km from the upper reservoir of the pumped hydro project and was only 7km deep. A new 7m high waterfall across a river and thousands killed and homeless due to over-turned buildings can be contrasted with only the lights going out in the main cavern. Shotcrete cracking was not reported. Workers did feel the earthquake.

4. At the Channel Tunnel between England and France the Q-system was utilized by one of the world's largest contractor consortia TML. The chosen medium for the three TBM tunnels was the chalk marl with UCS in the typical range 4 to 7MPa. Perhaps surprisingly for the designers of the wedge-lock unbolted PC-element liner on the UK side, the chalk marl was significantly jointed with up to three sets of rather planar, non-dilatant joints.

5. The TBM operations were severely delayed in some early sub-sea chainages, especially km 20 to 24. This was because of weathering, more continuous jointing, higher permeability, and a strong tendency for over-break which hampered ring-building due to the need for wedge-lock closure. In addition, there was damage to electrics and electronics on all the TBM due to unexpected seawater drips. A Q-value mostly in the range 3 to 8 in this difficult 4km, with sufficient *connectivity* due to up to three sets of joints, readily explains the minor but troublesome seawater inflow, since Lugeon ≈ 1/Q, suggesting permeabilities of approx. k = 0.1 to 0.3x 10^{-7} m/s.

6. A large claim by TML for unforeseen conditions was countered by owner Eurotunnel, who claimed conditions could have been anticipated if local mapping of the chalk marl in existing tunnels had been performed. Q-parameter histogram logging was performed of precedent occurrences (Terlingham, Beaumont, foreshore at Shakespears cliffs), and later compared with the subsequently provided TML Q-logging. The likelihood of over-break and inflow could be anticipated, in both sets of similar Q-data.

7. The weakness of the chalk marl with UCS of 4 to 7MPa and its high porosity (approx. 28%) mean that combining the Q_{mean} of 8 from the writer's Q-logging in the Terlingham and Beaumont tunnels, plus foreshore logging, and that of eight marine borehole cores has been used to arrive at a predicted velocity of approx. 2.0km/s, at the lower end of the 2.0 to 2.6km/s measured range, and a disturbed (EDZ) modulus ranging from 0.5 to 1.0 GPa, roughly in line with (deformation) back-calculated values of 0.8 to 1.4km/s.



CRACKING IS ACTUALLY CAUSED BY EXCEEDING THE ***CRITICAL EXTENSION STRAIN:***

Cracking in tension, then shear:

(Not 'compression' failure). FRACOD model)

(Stacey, 1981 and Baotang Shen……Barton and Shen, 2017)

$\sigma_{critical\ tangential\ stress} \approx (0.4 \times UCS) \approx \sigma_t/\nu$

(derived from $\varepsilon_3 = [\ \sigma_3 - \nu.\sigma_1]/E$

$$\sigma_t/\nu \approx \boxed{\frac{UCS}{10\nu} = \frac{UCS}{10 \times 0.25} = 0.4 \times UCS}$$