Lessons learned from Q and from tunnel and cavern design and performance

ATHENIAN LECTURE 2014

HELLENIC SOCIETY FOR SOIL MECHANICS & GEOTECHNICAL ENGINEERING

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Content of Lecture

1. A VARIED CAREER IN TUNNELLING, LAUNCHED BY Q-SYSTEM
2. INTRODUCTION TO FUNCTION OF SOME Q-PARAMETERS
3. SINGLE-SHELL NMT USING Q, or DOUBLE-SHELL NATM ?
4. INTEGRATION OF Q (or Q_c) WITH PARAMETERS: V_p, E_m, Δ, L
5. NUMERICAL MODELLING DISCUSSION, c and ϕ from Q_c ?
6. CASE RECORD OF HIGH STRESS – NEGATIVE EFFECTS
7. CASE RECORD OF THE BENEFITS OF (REASONABLY) HIGH STRESS
8. CONCLUSIONS

An early question which accidentally launched the tunneling career of the lecturer 40 years ago:

Why Norwegian powerhouses, mostly with spans of 18 to 24m, showed *significant variation in deformations*?

Q-system development – *powerhouse deformation* question answered after ½ yr

Different rock qualities, different depths and stress levels, and different support methods each played a role in these *deformations*.

Six months of hard work with case record analysis and re-analysis resulted in the 'accidental' development of the Q-system.

It is now used more widely around the world than anyone would have guessed. Some do not like this!

SO WHAT IS THE 'Q-system' ?

Hellenic Society Soil Mech./Geotech.

engineers may not be familiar with 'Q'

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Q used here!

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')**

SUGAR LOAF MOUNTAIN, RIO DE JANEIRO

TOP END OF ROCK MASS QUALITY SCALE.

$Q \approx 100/0.5 \times 4/0.75 \times 1/1$

i.e. >1000

BRAZILIAN HYDROPOWER PROJECT COLLAPSE IN FAULT

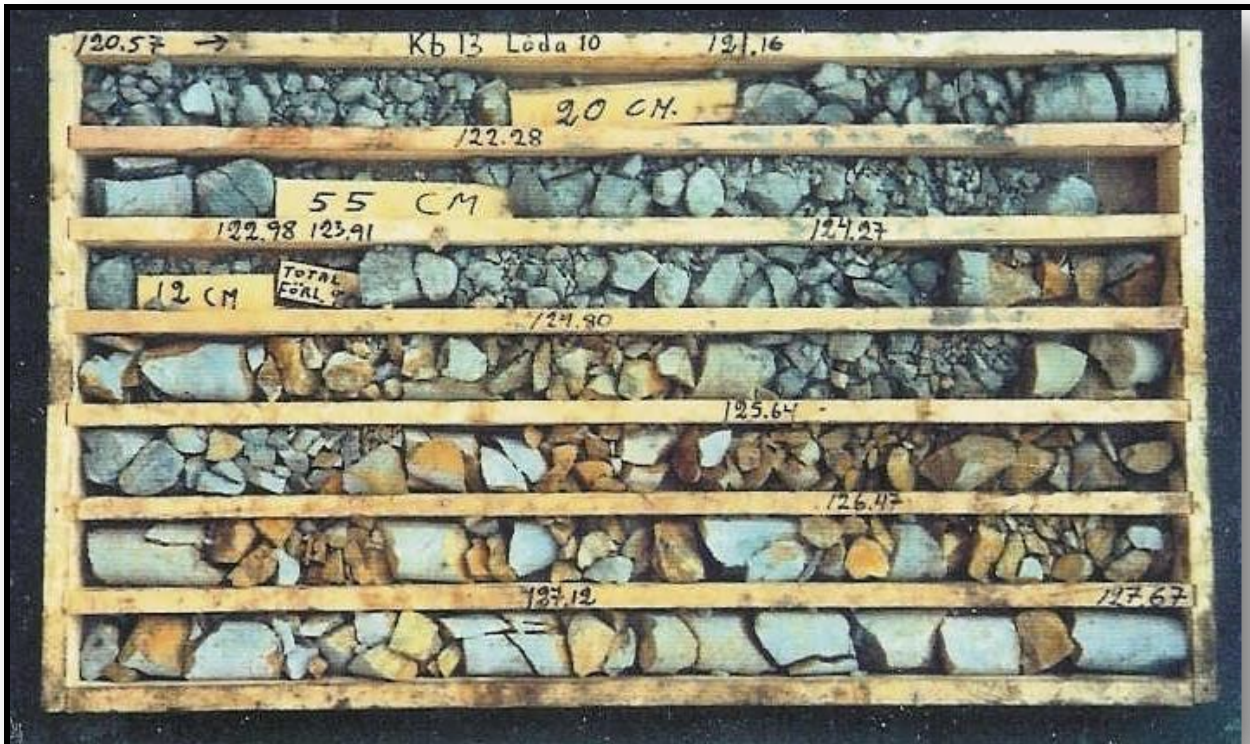
LOWEST END OF THE ROCK MASS QUALITY SCALE.

$Q \approx 10/20 \times 1/8 \times 0.5/20$

i.e. < 0.001

Strength contrast, modulus contrast, constructability contrast (15 years/1 year)

0.001→1000, or 5→95, or F7→F1 ???

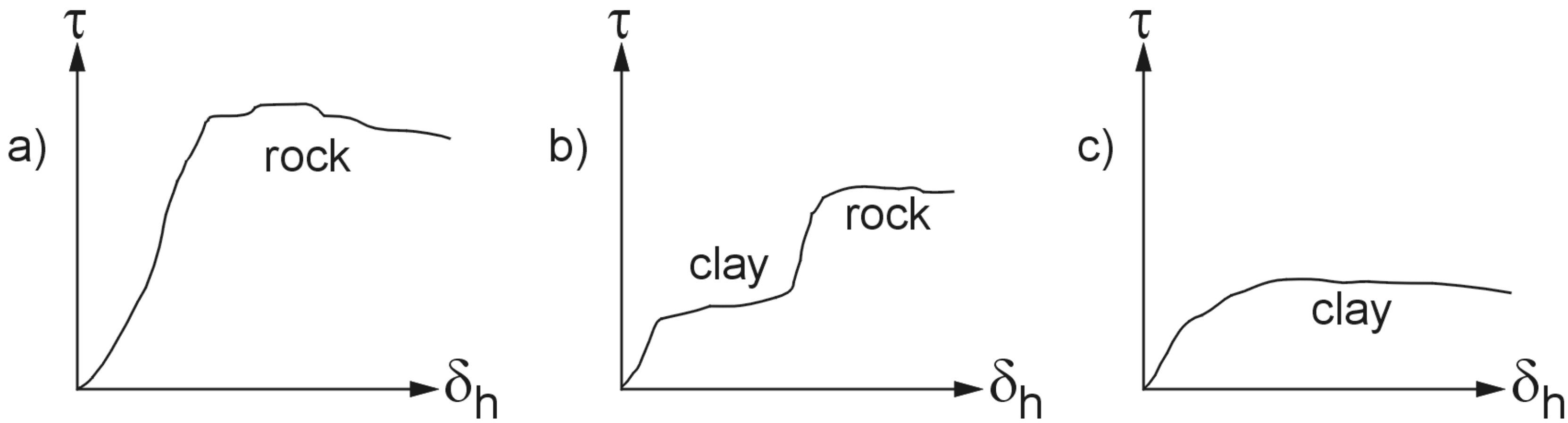


THE FIRST TWO PAIRS OF PARAMETERS HAVE DIRECT PHYSICAL MEANING:

RQD / Jn = relative block size

Jr / Ja = frictional strength (≈ μ)

Jw / SRF = effects of water, faulting, strength/stress ratio, squeezing or swelling (an 'active stress' term)

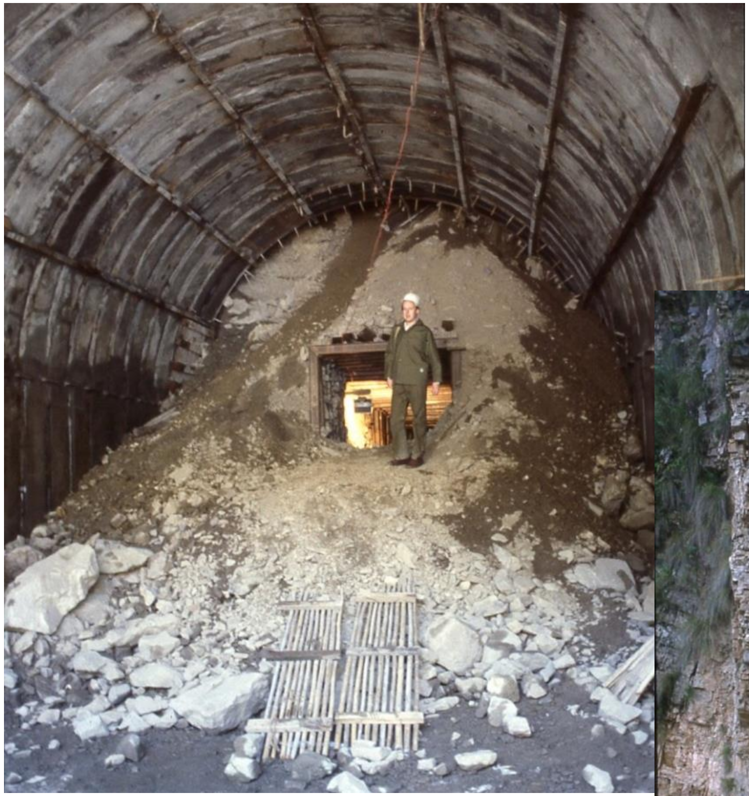


(a) Rock wall contact

(a) Rock wall contact	J_a/J_r =	0.75	1.0	2	3	4
		(thin coatings)				
		$\tan^{-1}(J_r/J_a)°$				
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°

Jr/Ja is like a 'friction coefficient'

FAULT ZONES ARE UNIQUE CHALLENGES FOR TUNNELLERS BECAUSE…….

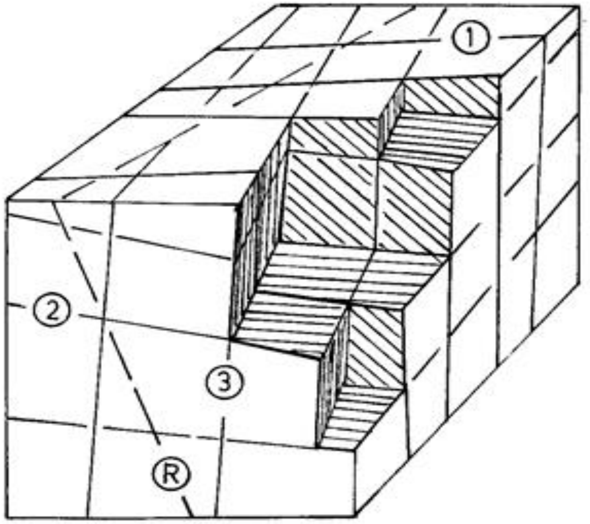


RQD, Jn, Jr, Ja, Jw, SRF……..all Q-parameters may be adverse

also TIME + COST

OTHER *COMBINATIONS OF Q-PARAMETERS* ALSO PROVIDE USEFUL GUIDANCE ABOUT TUNNEL BEHAVIOUR

Jn/Jr ……over-break



OVERBREAK IF Jn/Jr ≥ 6

Jn = number of sets

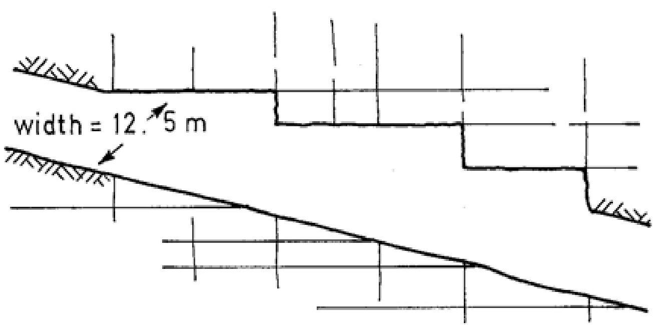
Jr = roughness

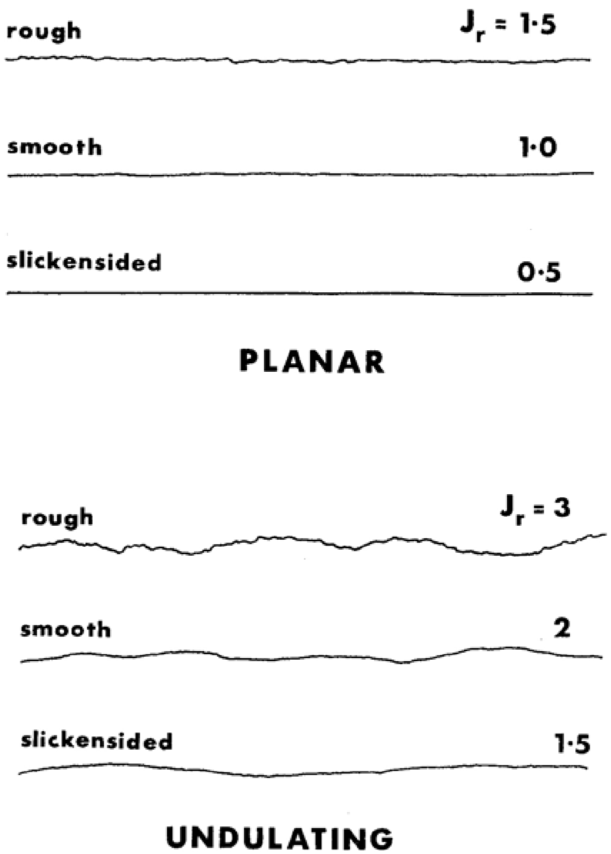
6/1.0	9/1.5
12/2	15/3

(DESPITE FOUR JOINT SETS, TOO MUCH ROUGHNESS AND DILATION)

In photos:

Jn/Jr = 9/1.5





OVER BREAK needing 4m of CONCRETE.

Reason: adverse Jn/Jr

CONCERNING THE PREDICTION OF OVERBREAK

RMR (Bieniawski) and GSI (= RMR-5) (Hoek) ARE PARAMETERS USED BY MANY IN ASSISTING TUNNEL DESIGN.

ONE SHOULD BE AWARE THAT THERE IS NO PARAMETER FOR *NUMBER OF JOINT SETS*, NOR FOR *ROCK STRESS* IN RMR, THEREFORE NOT IN GSI EITHER.

MANY POSSIBILITIES IN TUNNELLING

Among the possibilities:

NMT or NATM?

NMT or NATM?

1. ***SINGLE-SHELL*** **METHODS OF** ***SUPPORT (Sfr)*** **+** ***REINFORCEMENT (B)*** **ARE USED IN 'ALL' THE WORLD'S HYDROPOWER GENERATION CAVERNS, OIL STORAGE CAVERNS ETC.**

2. **BUT IN OTHER EXCAVATIONS (LIKE ROAD, RAIL, METRO TUNNELS) THERE IS A DECISION TO BE MADE: 'NATM' or 'NMT'?**

REISSECK II PUMPED STORAGE, AUSTRIA

***EVEN IN AUSTRIA*, SOMEONE DECIDED TO USE ‘SINGLE-SHELL’ S(mr) + B …..in this large machine-hall.**

(Similar decisions could/should be made about smaller tunnels?)

CROSSRAIL, LONDON. Stepney Green Station, 40m depth, London Clay.

Final lining (2013) = multi-layer S(fr) (i.e.'SCL')

Two examples of single-shell tunnels with sprayed membrane as final seal against water.

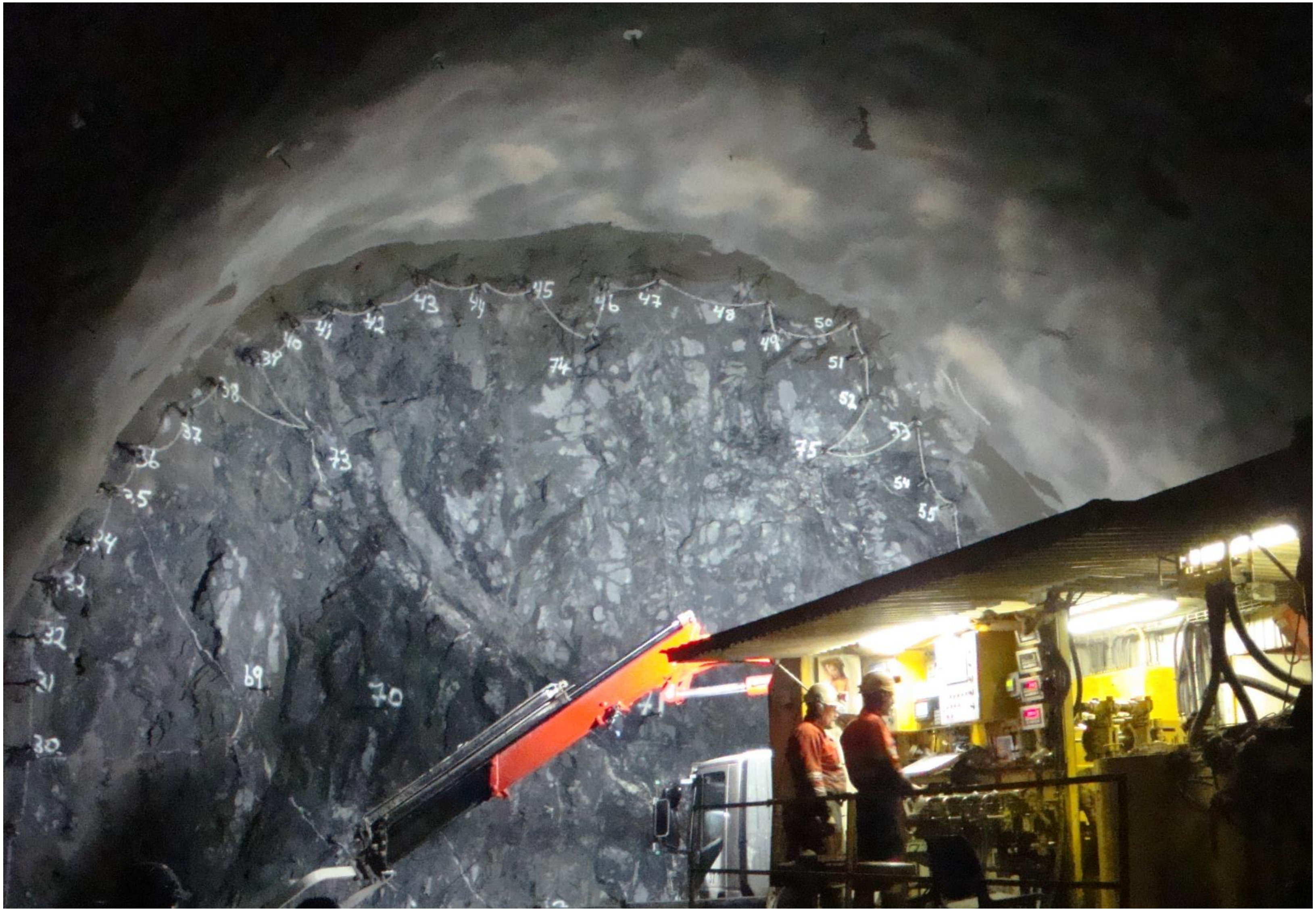
Lausanne Metro, Switzerland

Hinehead Tunnel, UK

These resemble appearance of NMT in Norway

IN NORWAY THE WATER-CONTROL METHOD MAY BE PRE-INJECTION

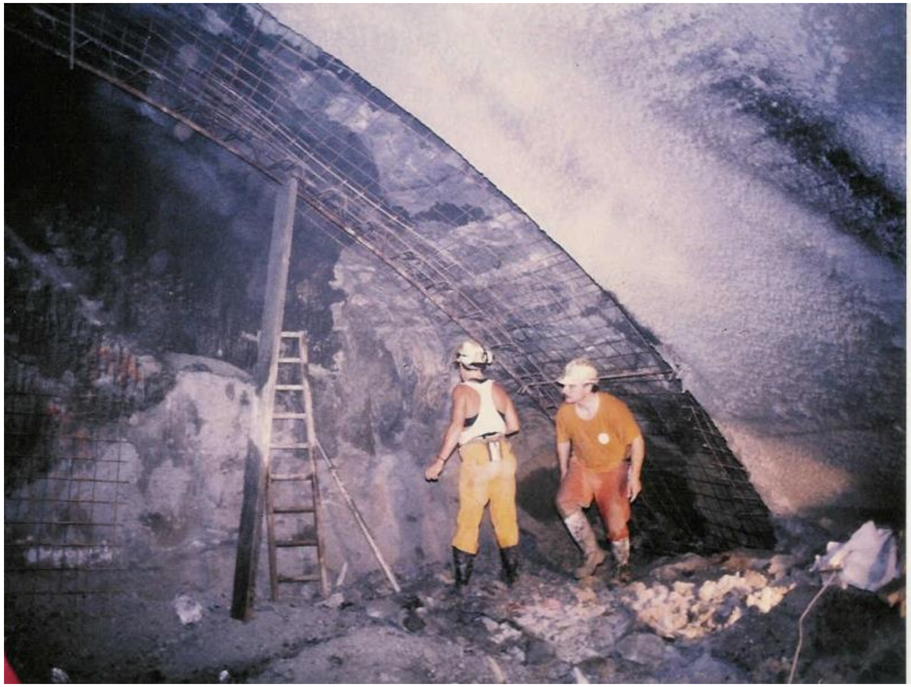
Q-BASED PERMANENT SUPPORT IS B + S(fr)



PRE-INJECTED SHALES / AND LIMESTONES ……B + 1st S(fr)

Q-based permanent support behind this.

Used in some low speed city tunnels



Temporary support phase of NATM: S(mr) + B? + lattice girders?

Eurotunnel sub-sea Cross-Over Cavern, Channel Tunnel Project.

Final CCA.

With RMR/GSI, unwanted overbreak is not 'seen'. But with 'double-shell' NATM, S(fr) or S(mr) volume + CCA volume: all increase considerably + time/cost.

Difficult (3D) membrane construction with overbreak from Hong Kong

≈ 15 km of membrane welds per 1 km of tunnel

What if leaks?

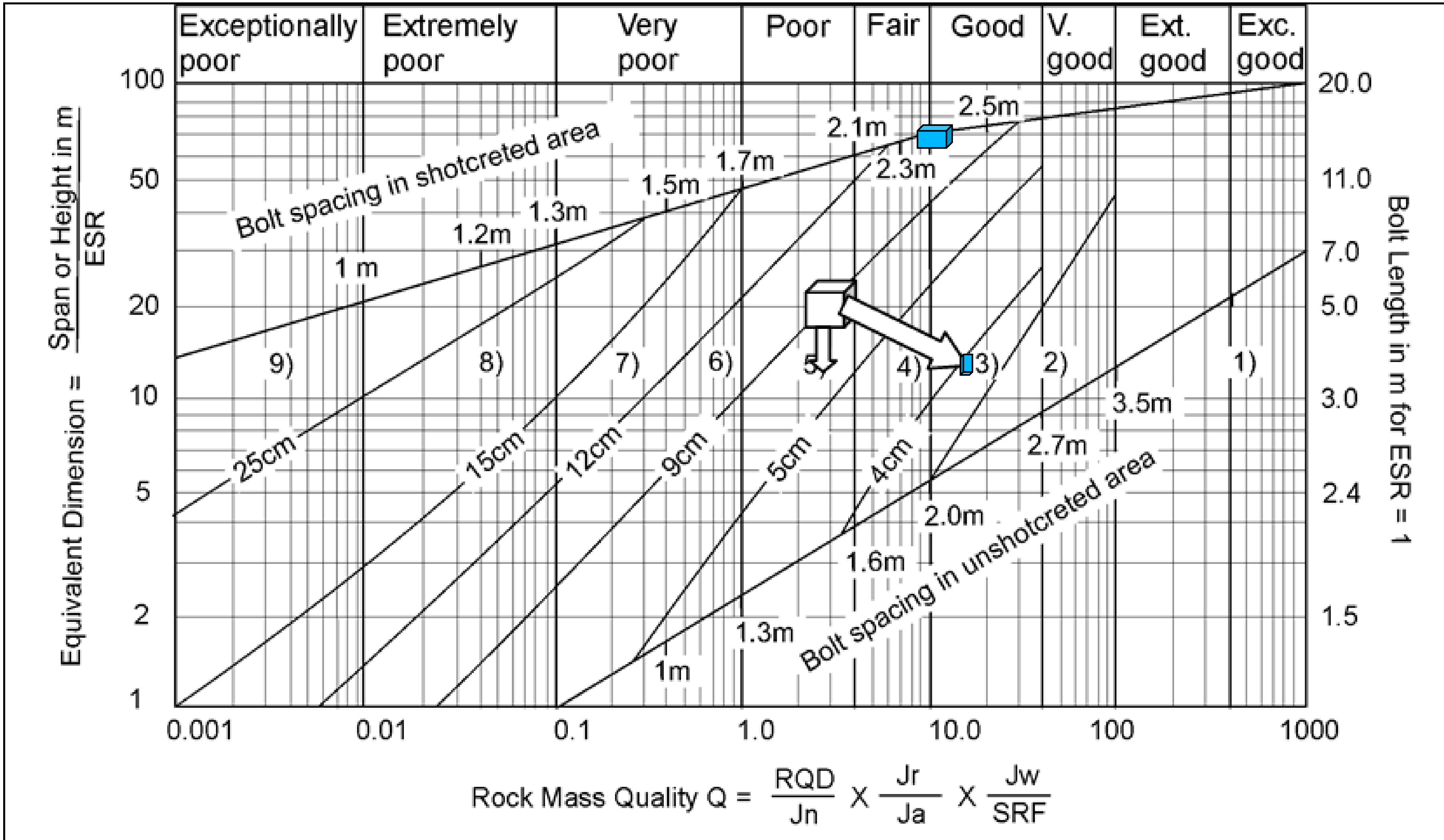
Impossible to locate.

NMT/NATM ?

JUST AS A CURIOSITY – NMT (Q) AND NATM CAN BE COMBINED !

THOSE WHO INSIST ON NATM – CAN USE Q FOR TEMPORARY SUPPORT SELECTION…5Q + 1.5 x ESR

(25 years use in HK road tunnels and metro tunnels)

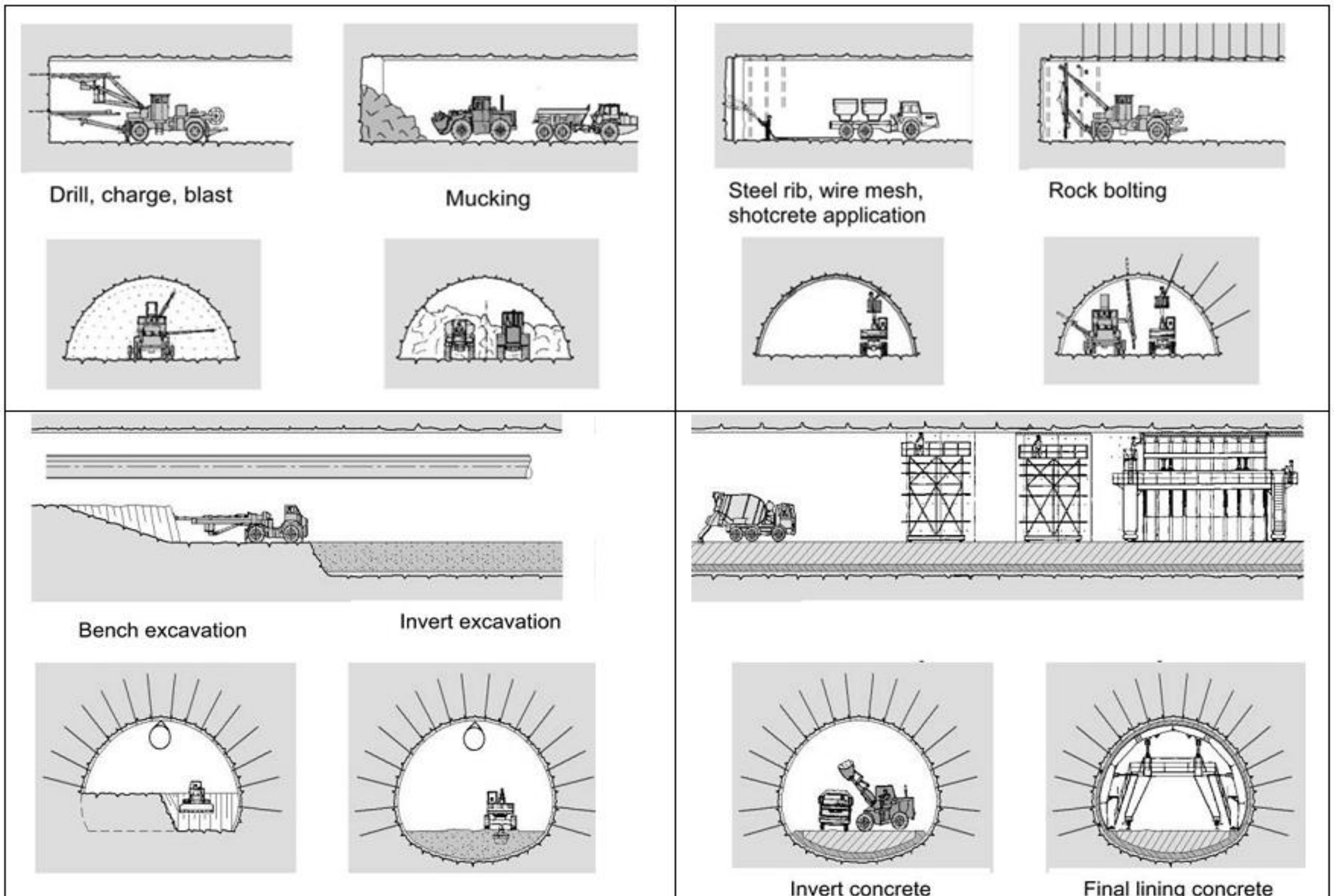


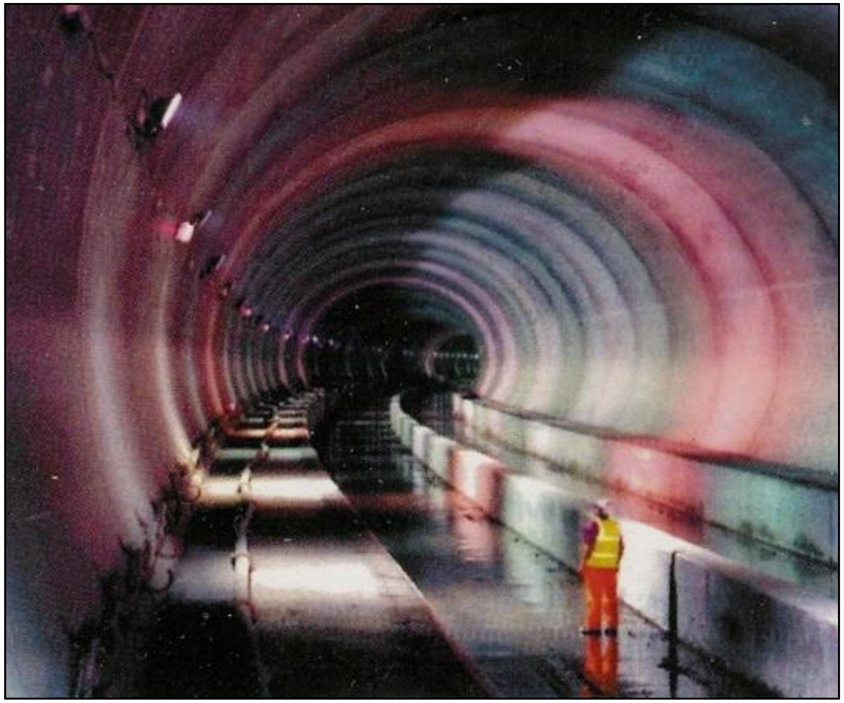
CONTRASTING THE TYPICAL COMPONENTS:

1. **'DOUBLE-SHELL' ('NATM')**
   (<u>Temporary</u>: Sfr/Smr, B, steel/lattice girders, <u>Permanent</u>: fleece, membrane, cast concrete CCA)………..*needs large work force*

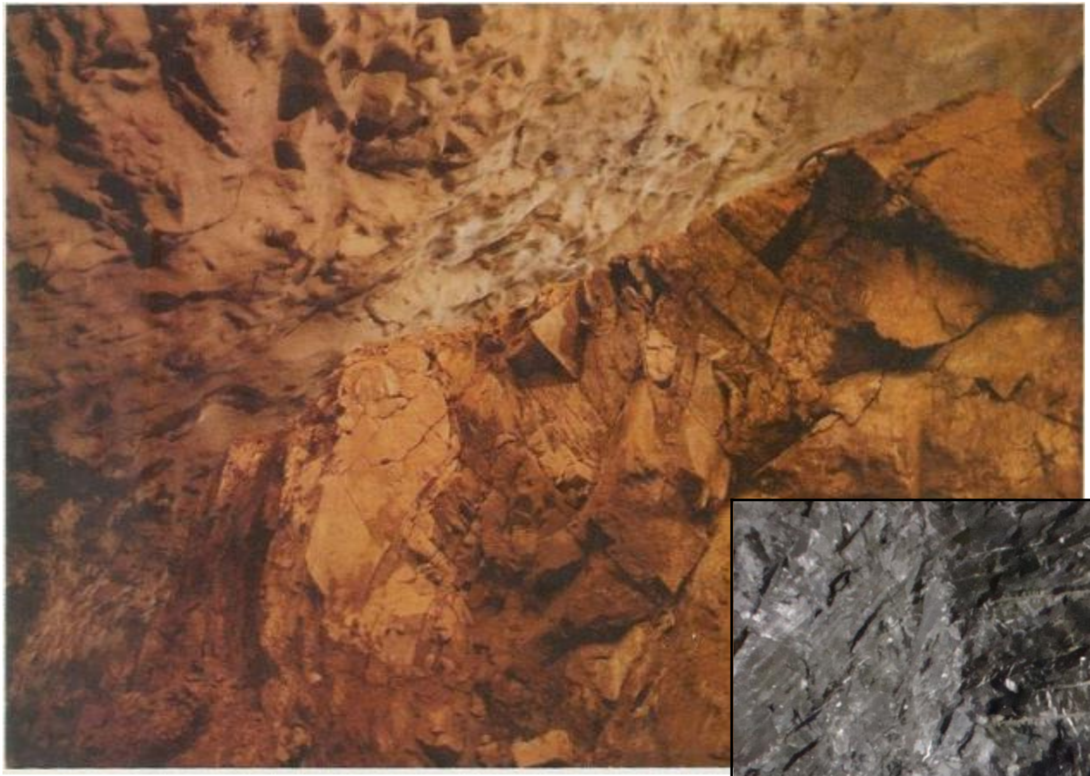
2. **'SINGLE-SHELL' (NMT)**
   (pre-grouting?) + <u>B + Sfr</u> + (RRS?)…………..
   needs small work force (x 1/10?)

Schematic construction sequence of a typical NATM tunnel, used in both softer and harder rock, from "Austrian Society for Geomechanics, 2010. NATM, 'The Austrian Practice of Conventional Tunnelling'. **This method has been observed in many countries when Q is 'poor', 'fair', 'good' i.e. Q = 1 to 40, where NMT would be suitable and much faster and cheaper.**



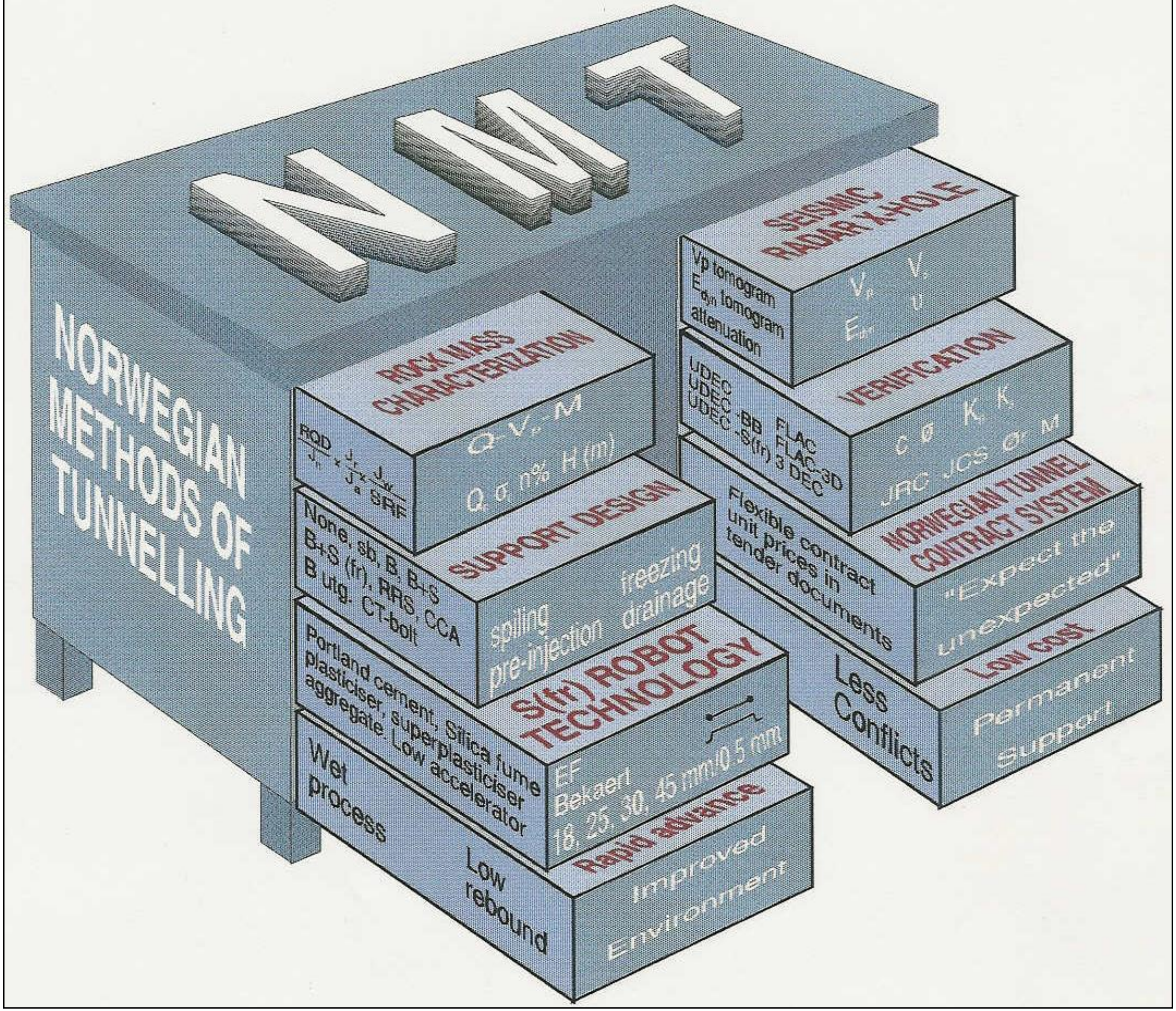


High-speed rail tunnel through jointed chalk in Southern England, had final *(year 2000)* costs of US$ 128M /3.2 km, or $ 40,000 per metre. This *was* three to four times higher than a typical NMT tunnel, with similar Q-value rock, using B+S(fr) as permanent rock support, and a PC-element + membrane liner, for a drained-but-dry solution.



Single-shell (NMT) cavern

Single-shell (NMT) tunnel


NMT
NORWEGIAN METHODS OF TUNNELLING
ROCK MASS CHARACTERIZATION
Q-Vp-M
Q σ n% H (m)
RQD/Jn x Jr/Ja x Jw/SRF
SUPPORT DESIGN
None, sb, B, B+S
B+S (fr), RRS, CCA
B utg. CT-bolt
spiling freezing
pre-injection drainage
S(fr) ROBOT TECHNOLOGY
Portland cement, Silica fume
plasticiser, superplasticiser
aggregate. Low accelerator
EF
Bekaert
18, 25, 30, 45 mm/0.5 mm
Wet process
Low rebound
Rapid advance
Improved Environment
SEISMIC RADAR X-HOLE
Vp tomogram
Edyn tomogram
attenuation
Vp Vs
Edyn υ
VERIFICATION
UDEC
UDEC -BB FLAC
UDEC -S(fr) FLAC-3D
3 DEC
c ø Kn Ks
JRC JCS ør M
NORWEGIAN TUNNEL CONTRACT SYSTEM
Flexible contract
unit prices in
tender documents
"Expect the unexpected"
Less Conflicts
Low cost
Permanent Support

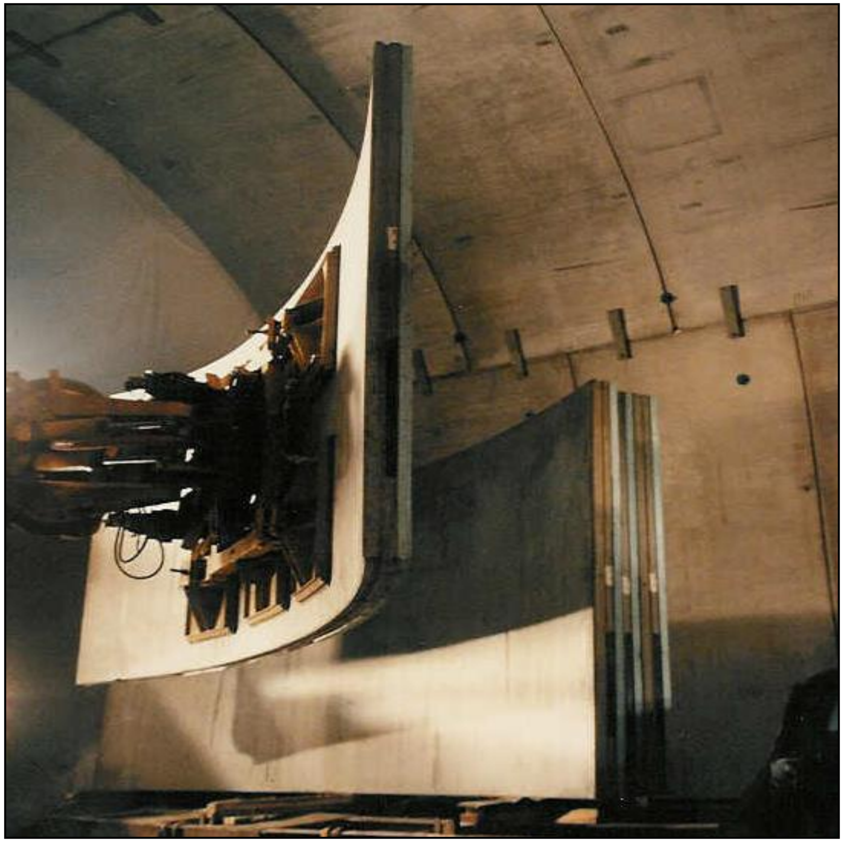
SOME DETAILS OF NMT

Design Preliminary design is based on field mapping, drill core logging and seismic interpretation.

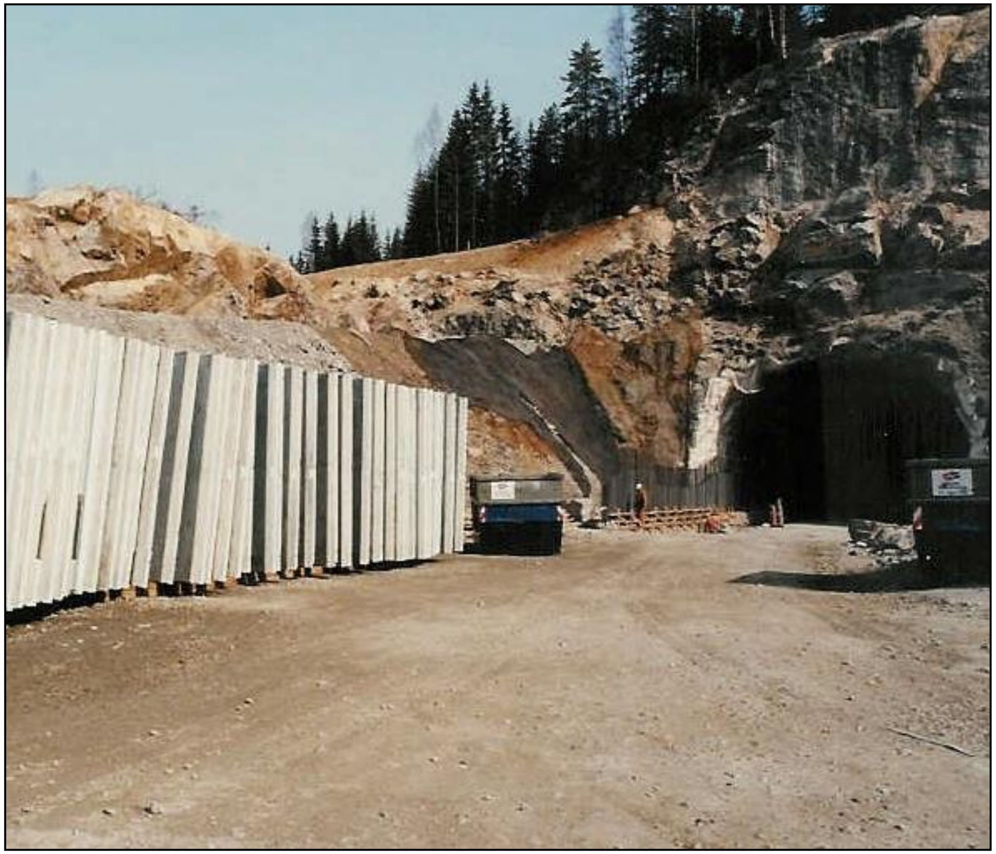
Final support is selected during tunnel construction based on tunnel logging and use of the Q-system support recommendations.

Support The permanent support usually consists of high quality wet process, fibre reinforced shotcrete and fully grouted, corrosion protected rock bolts.

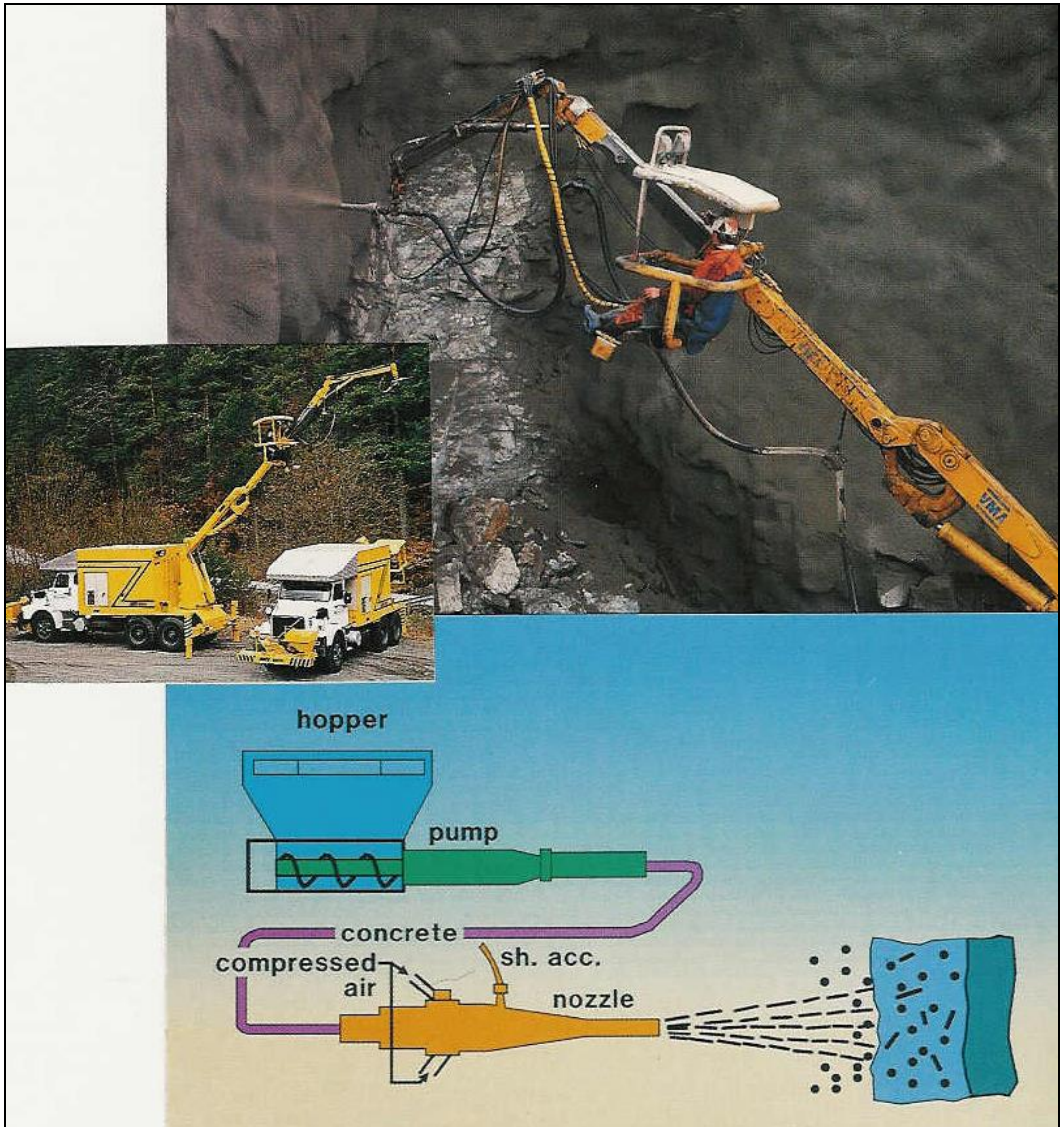
Contract The owner pays for technically correct support. Needed support is based on the agreed Q-value, and may vary frequently.

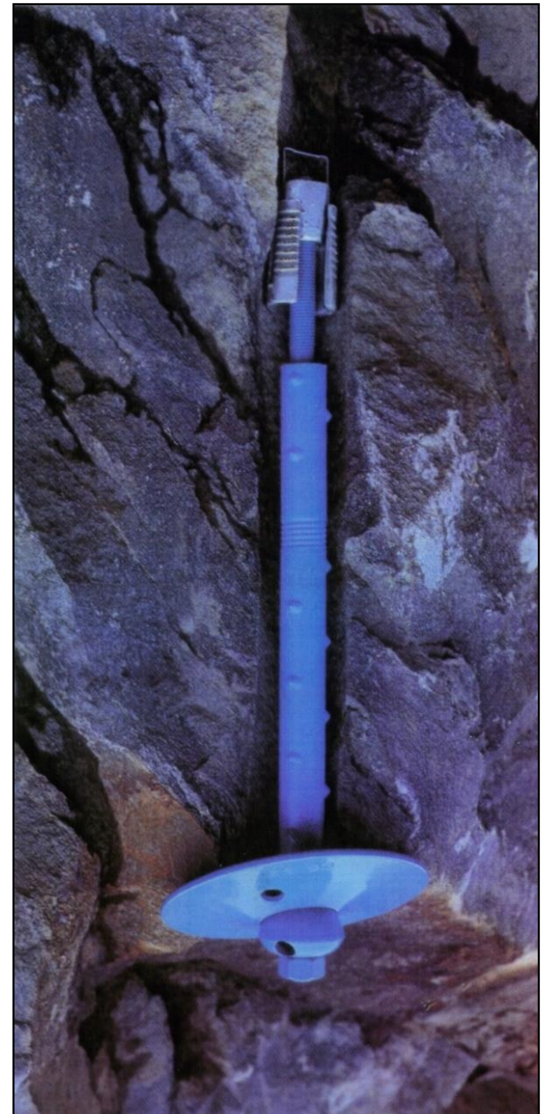


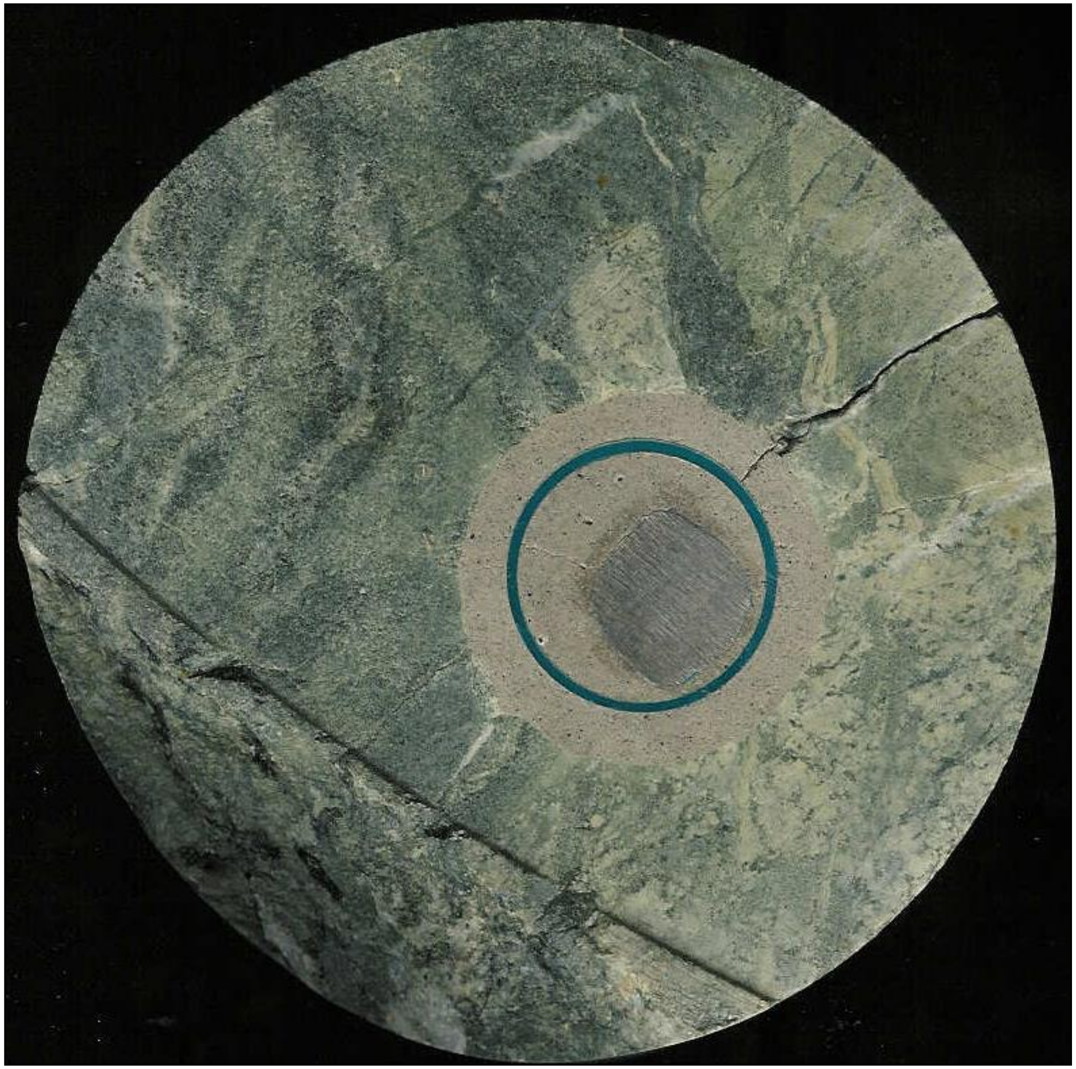
HIGH-SPEED (250 km/hr) TWIN-TRACK RAIL TUNNEL



Wet process S(fr) + CT bolts





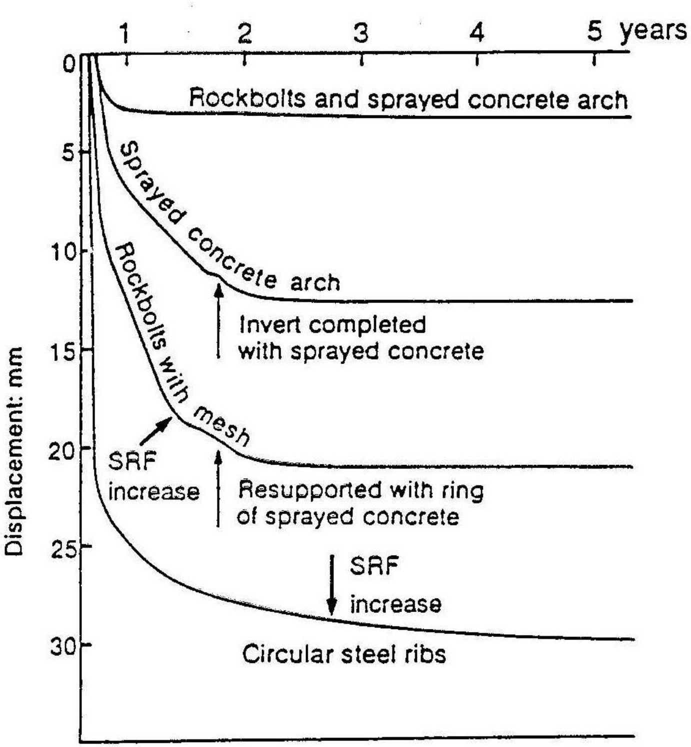


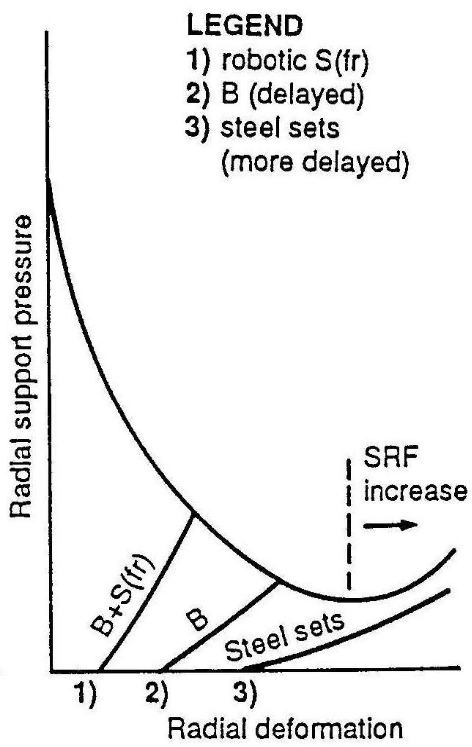
An over-cored CT bolt showing crack (joint) penetration to outer layer of grout – the usual *potential commencement of corrosion* for a conventional bolt near the face.

There remain four layers of corrosion protection even with the joint/crack.

WHAT IF BAD CONDITIONS IN SINGLE-SHELL (NMT) TUNNELS ?

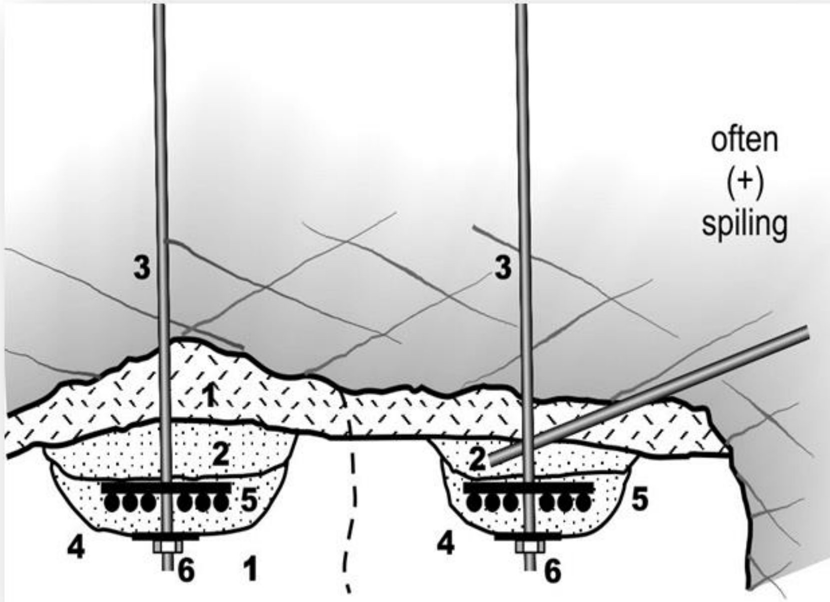
(NEVER USE STEEL ARCHES……because….)

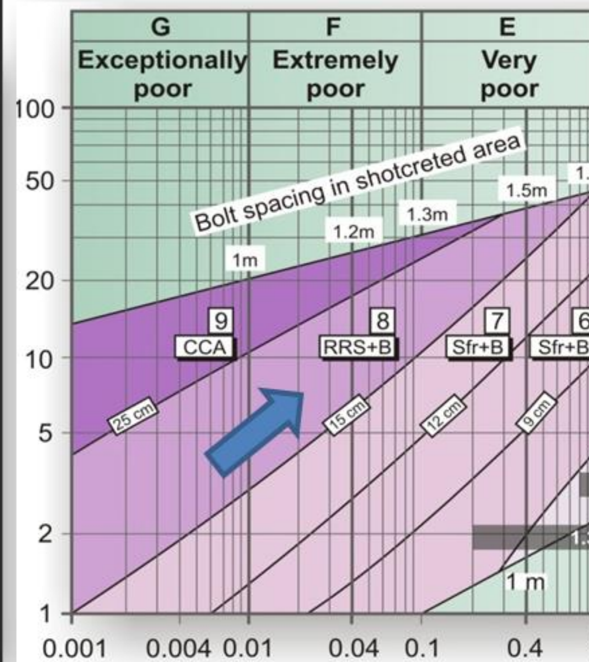


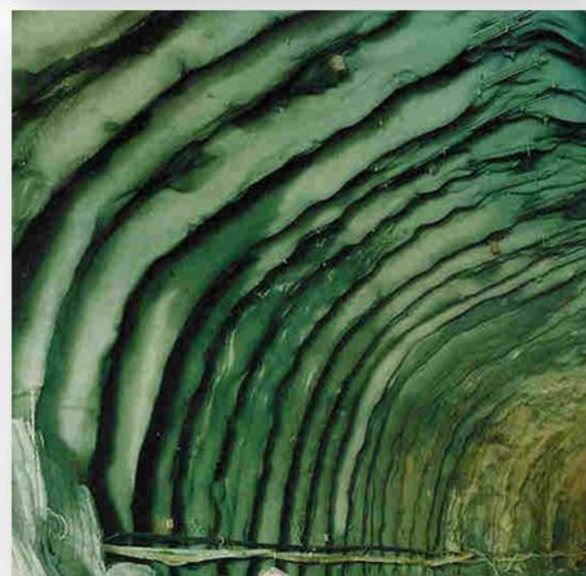


STEEL ARCHES or LATTICE GIRDERS

THE CONSEQUENCES OF LOOSENING ROCK – SRF ?







RRS is a flexible (until bolted) 'lattice' girder.

3D effect because of S(fr) arches.


AMV

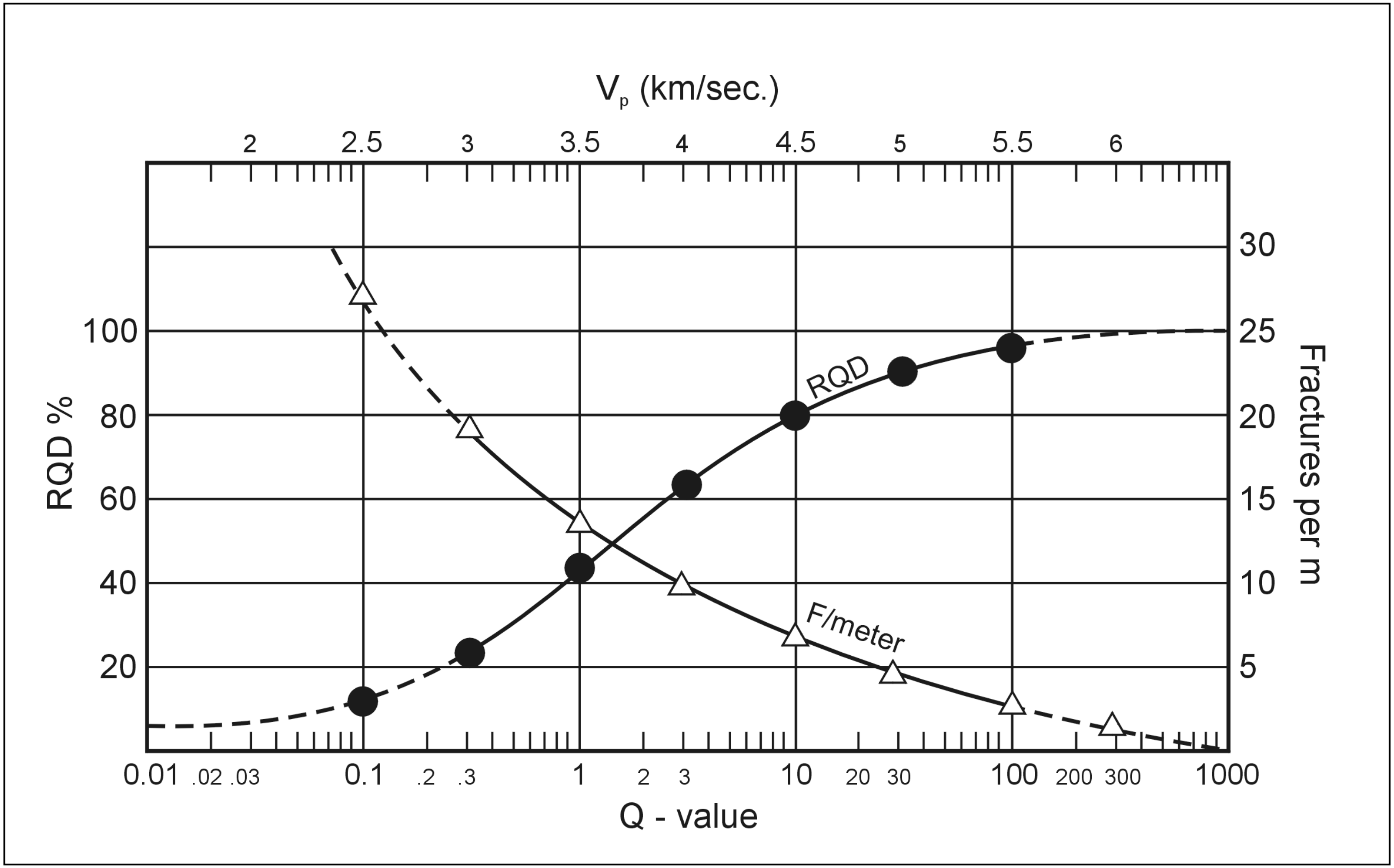
Integration of rock mass quality (Q) with

seismic velocity (Vp),
deformation modulus E_{mass},
deformation Δ,
Lugeon L?

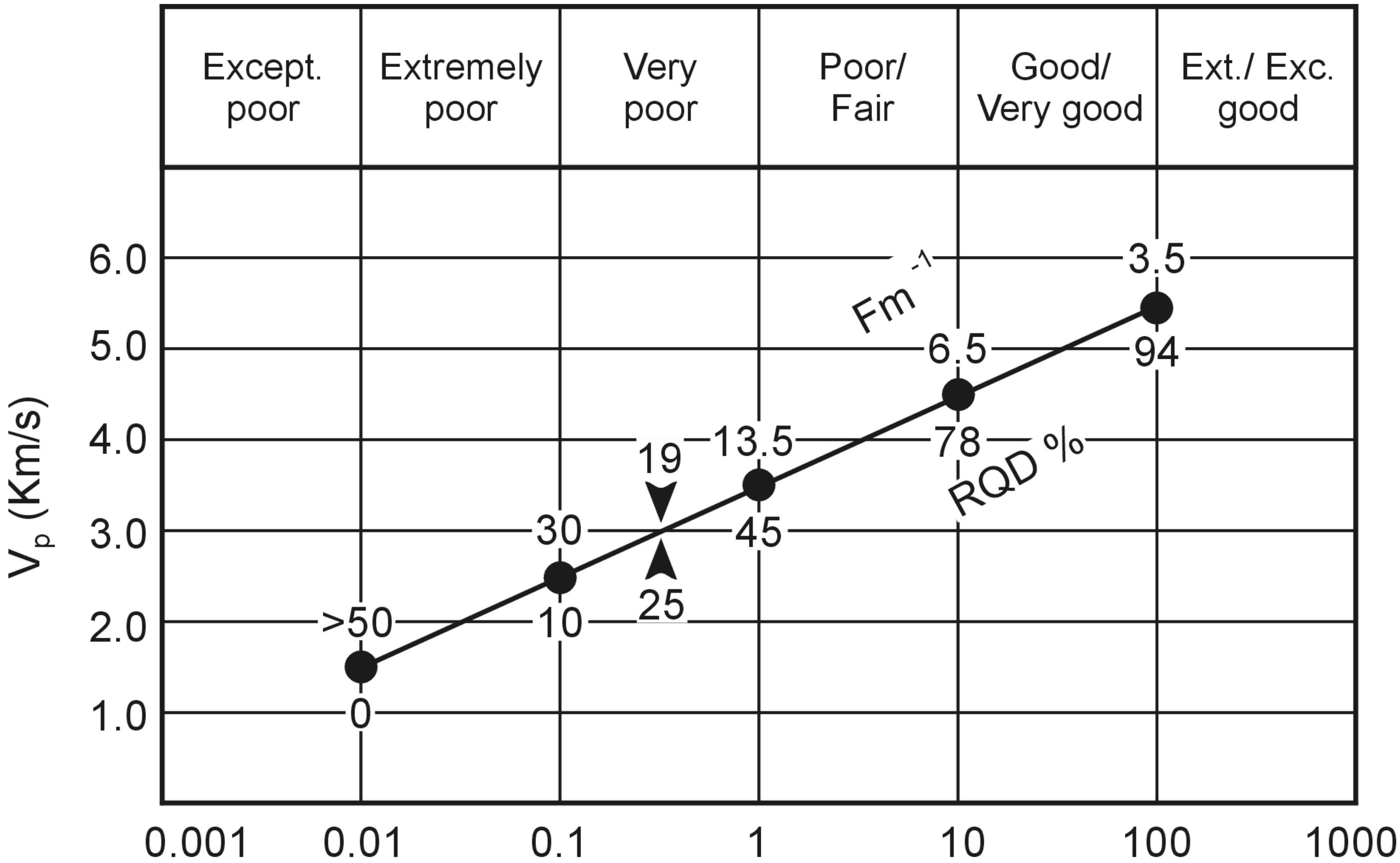
P-wave velocity Vp conversion to Q

(useful for interpolating between boreholes)

Sjøgren et al., 1979 data from 120 km of seismic profiles, and 2.8 km of oriented core from hard-rock Scandinavian sites with little weathering.



(Sjøgren et al. 1979, with Barton, 1995 addition of Q-scale)

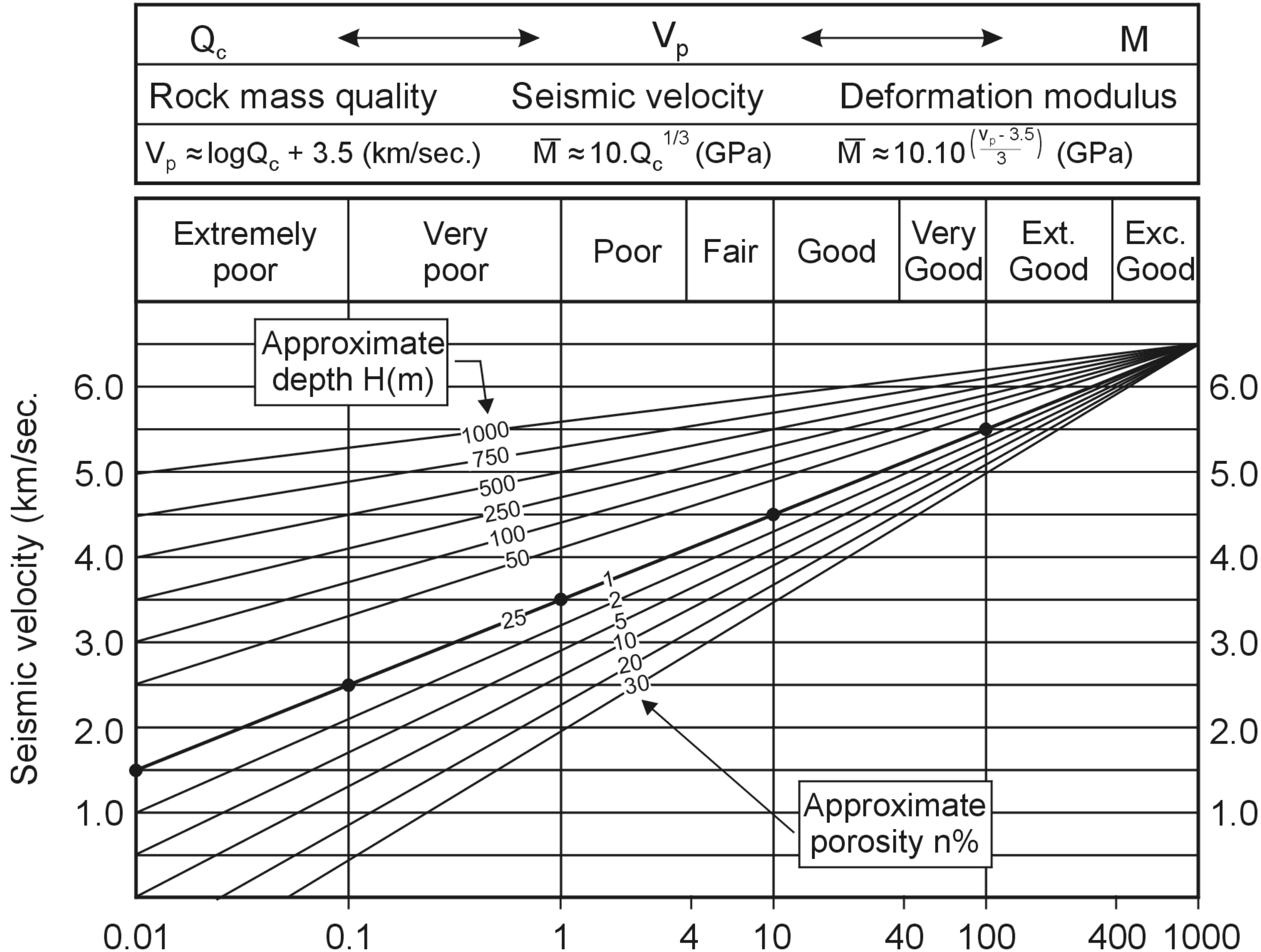


$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$$

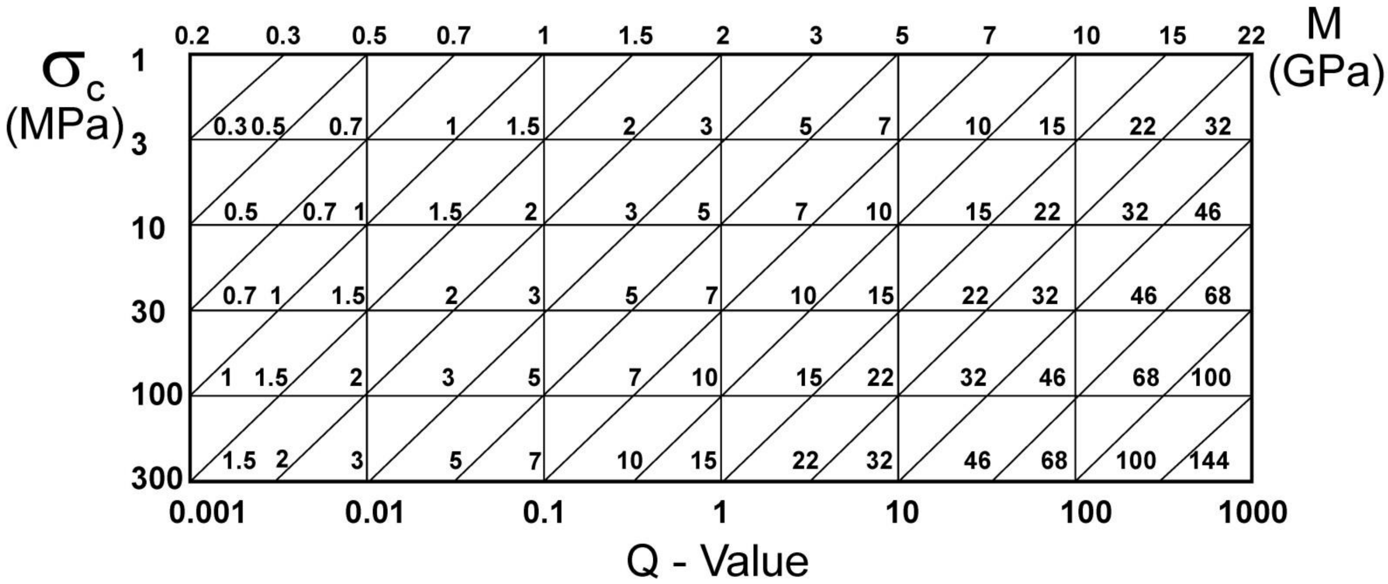
(As with all Sjøgren data: hard rock, near-surface)

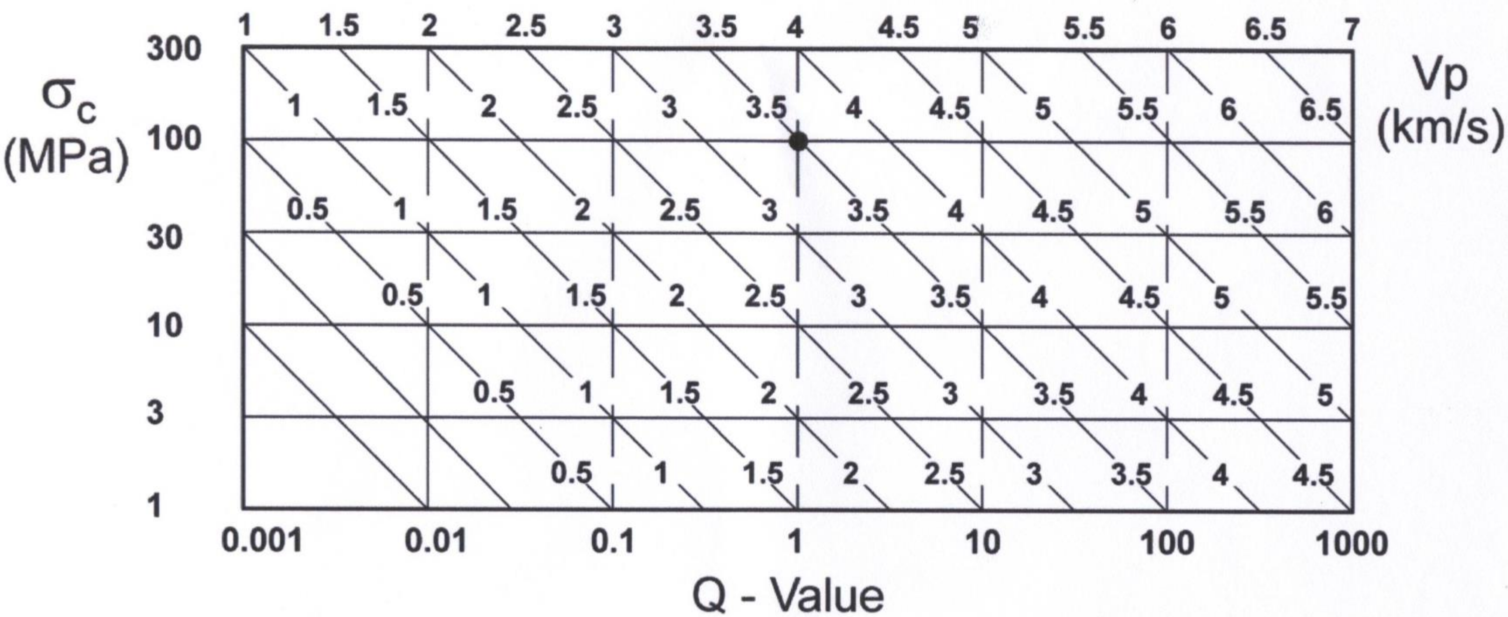
In the case of correlating Q-values to engineering/ geophysical parameters like V_p (P-wave velocity) and E_{mass} (deformation modulus) use of the term $Q_c = Q \times \sigma_c / 100$ *is better than Q alone!* **(σ_c in MPa)**

The six-order of magnitude Q range of 0.001 to 1000 (approx.) and *the larger (eight-orders-of-magnitude) range of Q_c* correlate fairly simply, to the huge - real world - range of rock mass properties.



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

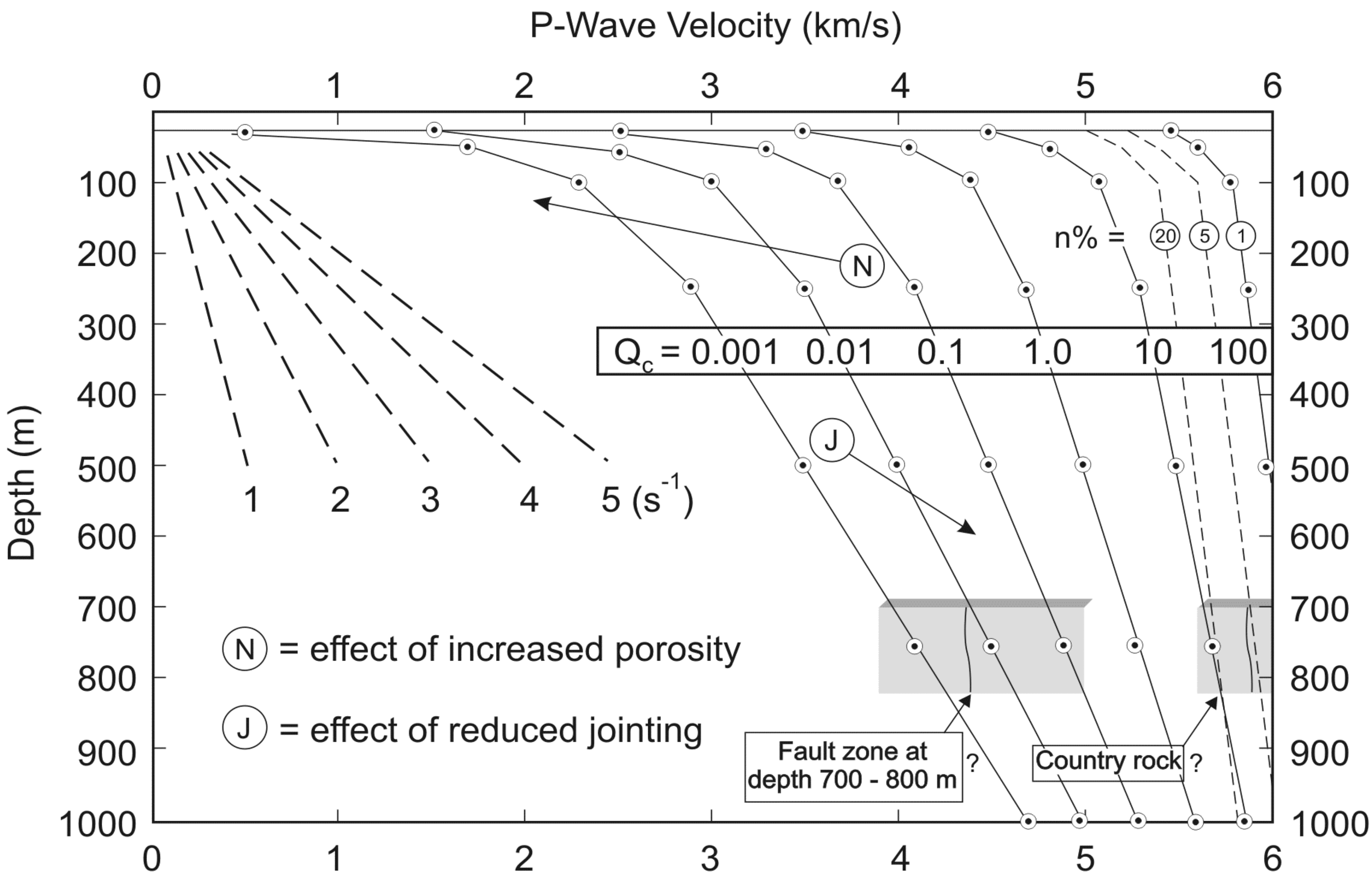




NOTE: NO CORRECTION FOR DEPTH (OR STRESS) – from central diagonal in previous figure – nominal depth 25m

DEPTH-DEPENDENT Qc 'iso-curves'. In practice 'Qc-jumping' is experienced, with both Vp and Qc increasing rapidly in the top 10 to 100 m, depending on weathering depth.

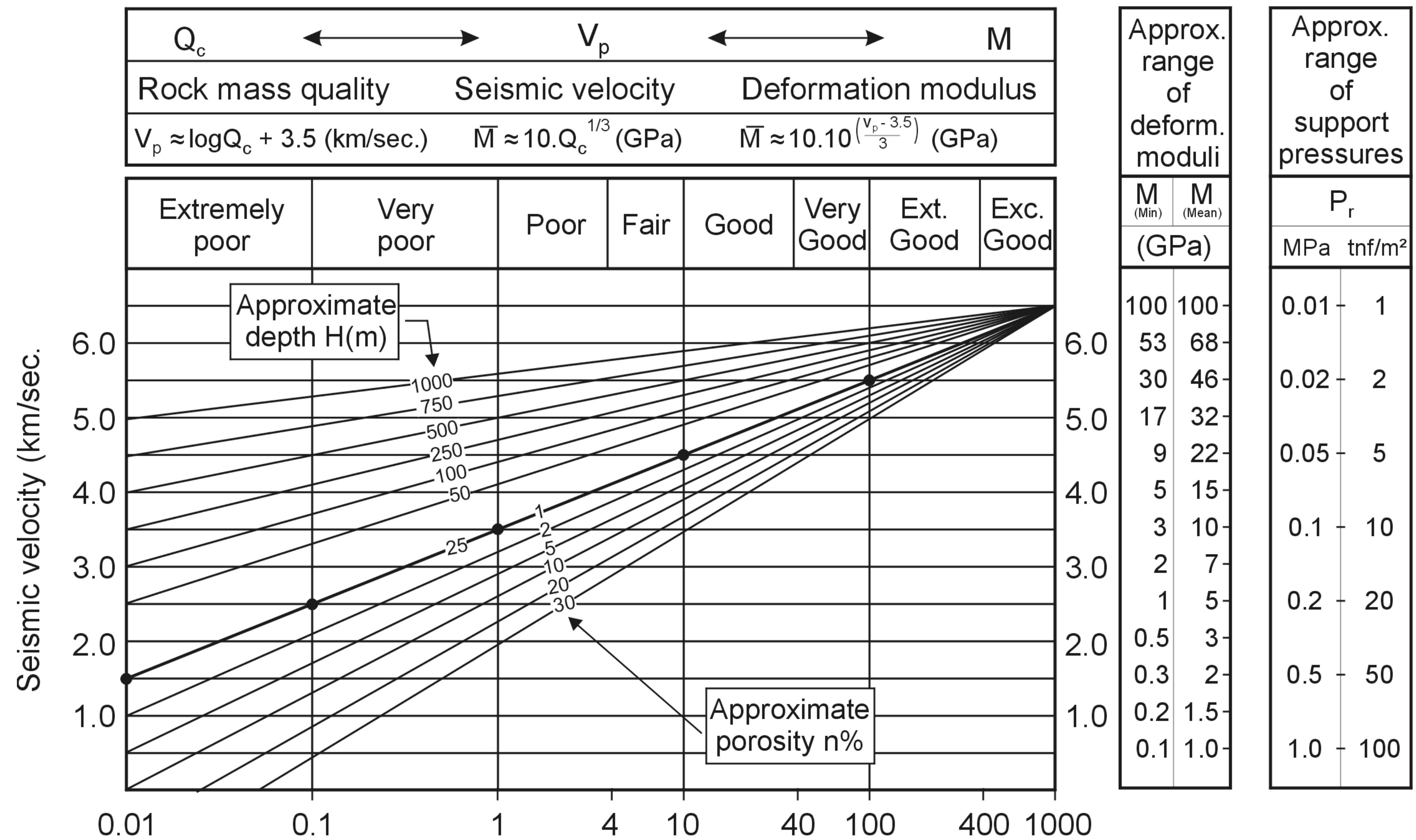
(Note km/s per km = s^{-1} units of velocity gradient).



CONVERSION OF Q_c TO DEFORMATION MODULUS

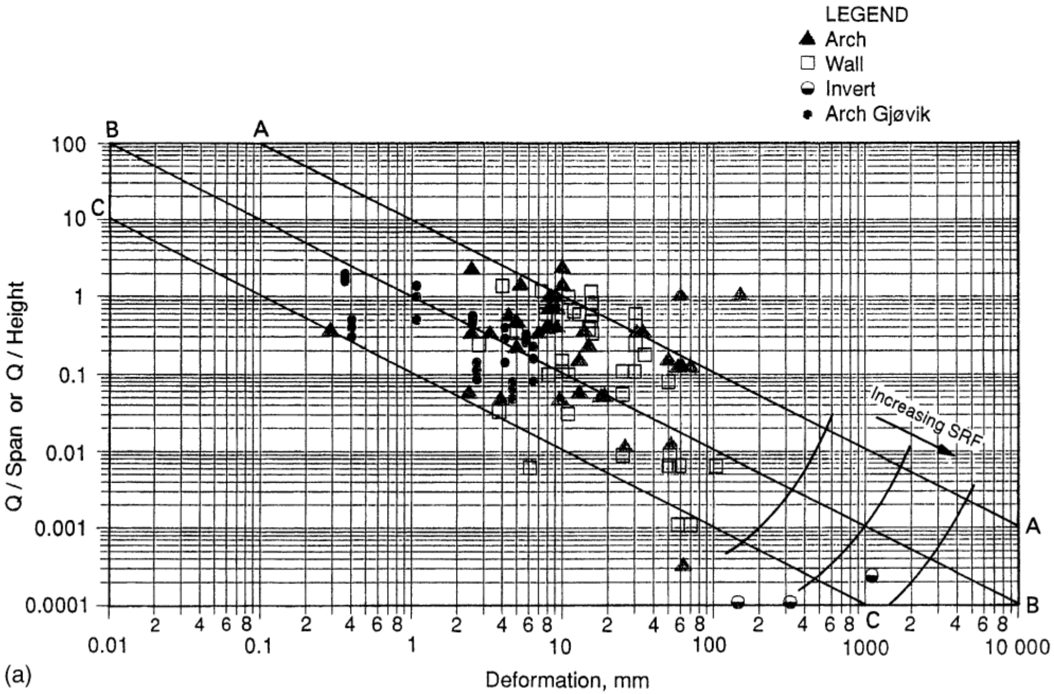
E_{mass} (or M)

MORE INTEGRATED MODEL: Q_c – V_p – M – Pr NOTE Pr ≈ 1/M



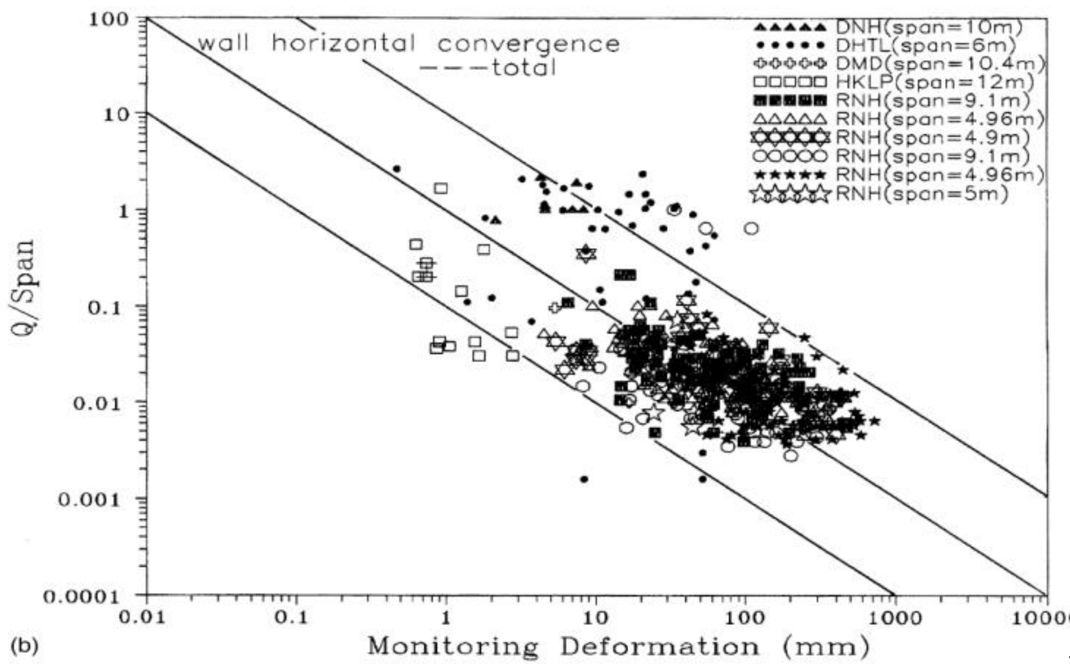
$$Q_c = \left[\frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}\right] \frac{\sigma_c}{100}$$

TUNNEL AND CAVERN DEFORMATION Δ IN RELATION TO Q and SPAN



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

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



$$\Delta_v = \frac{SPAN}{100Q}\sqrt{\frac{\sigma_v}{\sigma_c}}$$

$$\Delta_h = \frac{HEIGHT}{100Q}\sqrt{\frac{\sigma_h}{\sigma_c}}$$

$$k_o = \left(\frac{SPAN}{HEIGHT}\right)^2 \left(\frac{\Delta_h}{\Delta_v}\right)^2$$

Units:

SPAN, HEIGHT, Δ_v and Δ_h (mm)

Rock stresses and rock strengths (MPa).

(But *over-simplified central trend* is Δ (mm) ≈ SPAN(m)/Q from many hundreds of case records, many from Taiwan).

$$\Delta_v = \frac{20{,}000}{100 \times 3} \times (6/35)^{1/2} = 28 \text{ mm}$$

$$\Delta_h = \frac{50{,}000}{100 \times 3} \times (4/35)^{1/2} = 56 \text{ mm}$$

(SPAN = 20m, HEIGHT = 50m, Q = 3, σ_V=4 MPa, σ_h = 6 MPa, σ_C = 35 MPa). (In the middle of the range of MPBX measurements for the arch and walls).

Nathpa Jakri HEP powerhouse cavern India

Gjøvik cavern Norway

$$\Delta_v = \frac{60{,}000}{100 \times 10} \times (1/75)^{1/2} = 6.9 \text{ mm}$$

(SPAN = 60m, Q_{mean} = 10, σ_V = 1 MPa at 40 m depth, σ_C = 75 MPa) (Almost identical to that measured with nine MPBX, and almost identical to UDEC-BB modelling results).

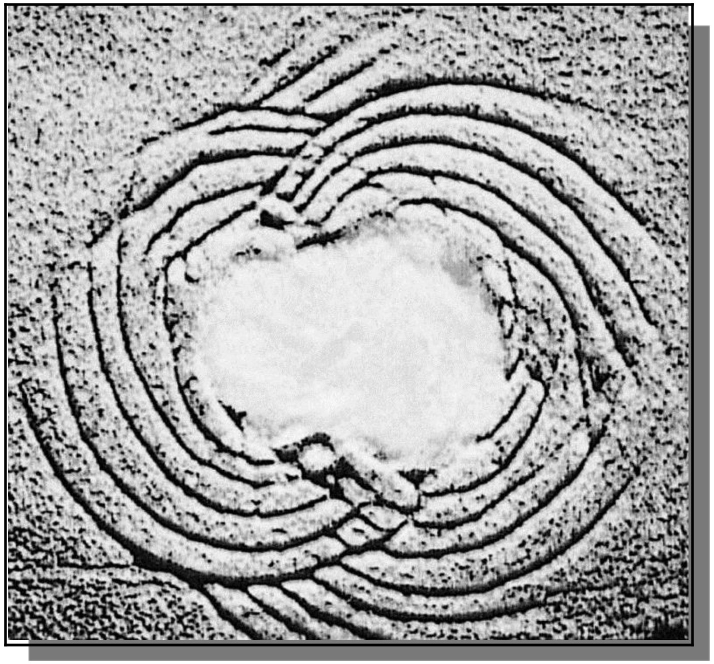
It is wise to check MODELLING RESULTS *with these formulæ* when doing numerical modelling (including UDEC), and obviously if doing continuum modelling.

THE EMPIRICAL FORMULÆ MIGHT BE CLOSER TO THE (FUTURE) MEASURED REALITY THAN THE NUMERICAL MODEL !

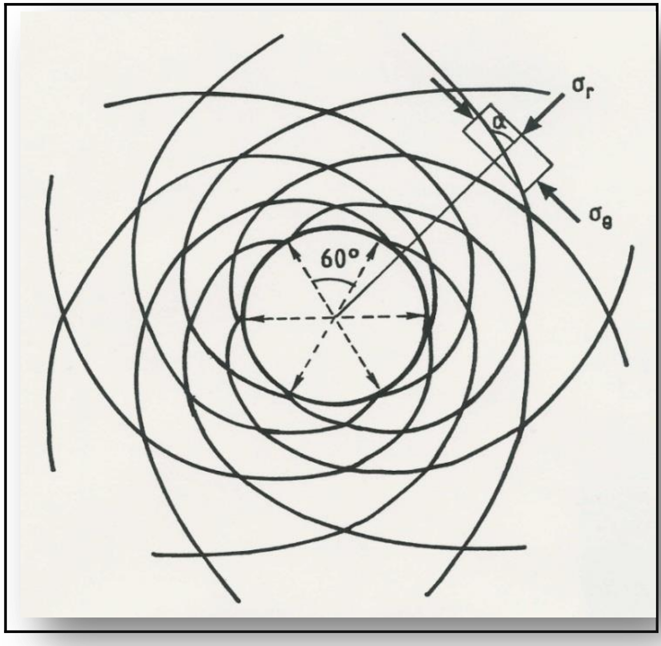
CONTINUUM (??)

or

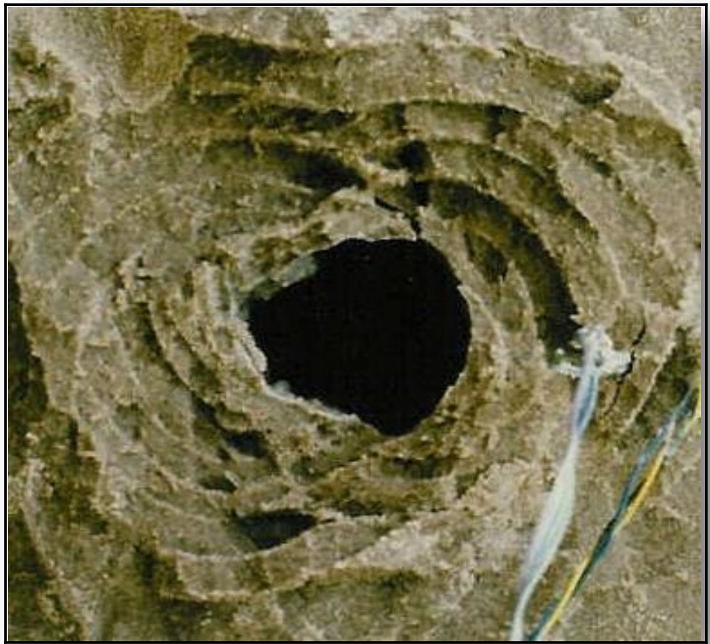
DISCONTINUUM MODELLING



Borehole stability studies at NGI

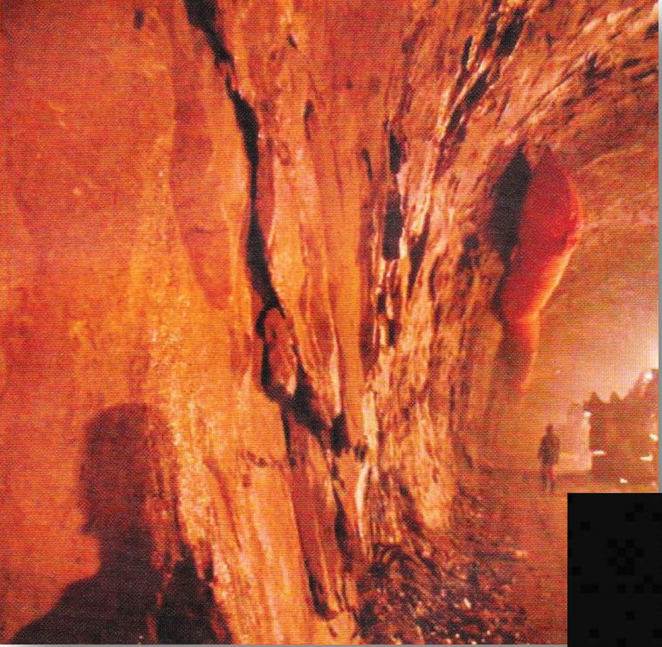


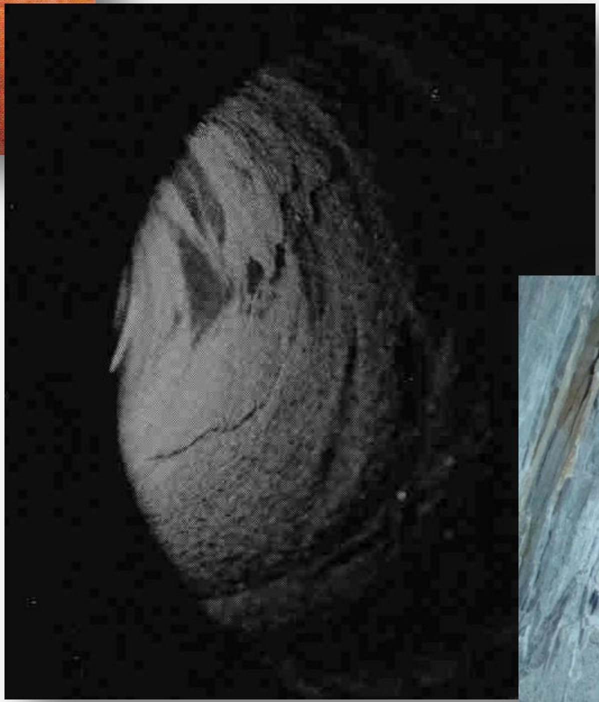
Continuum becomes a discontinuum!



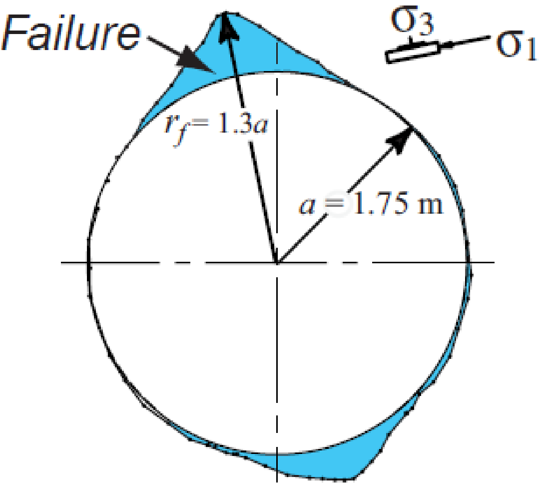
Drilling into
$\sigma_1 > \sigma_2 > \sigma_3$
loaded cubes
0.5 x 0.5 x 0.5 m
of model sandstone

Jinping II (D+B) – ISRM News Journal
Physical model – bored under stress (NGI)
Jinping II (TBM) – ISRM workshop (NB)





Log-spiral shear modes in weaker rock types



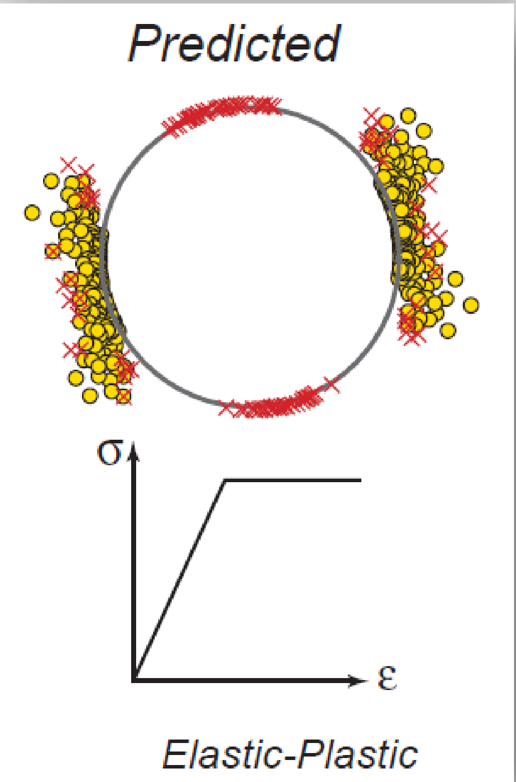
NEED for CHANGE

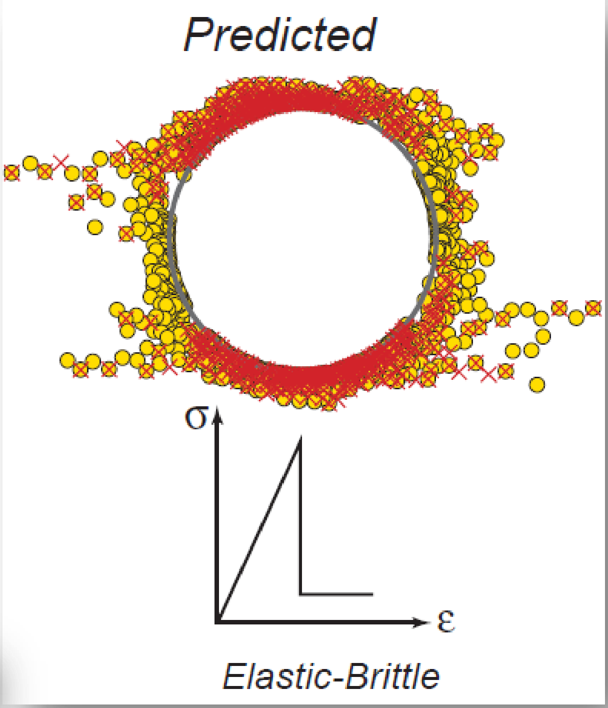
CONVENTIONAL continuum modelling methods are suspect.

Poor simulation with Mohr Coulomb or Hoek and Brown strength criteria.

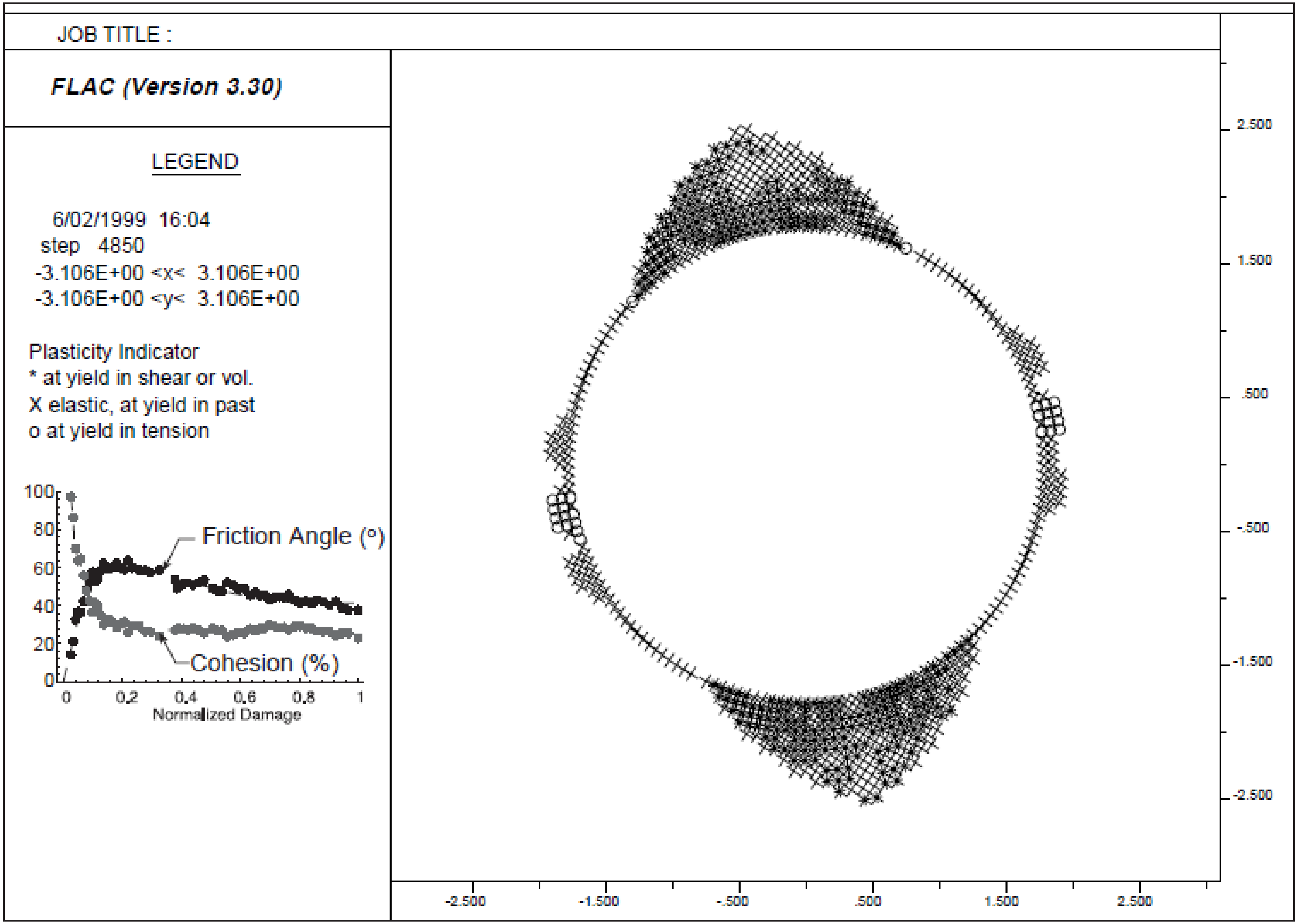
(Hajiabdolmajid, Martin and Kaiser, 2000 “Modelling brittle failure”, NARMS.)

So why performed by so many consultants?





× Shear failure ○ Tensile failure



Degrade cohesion, mobilize friction: excellent match.

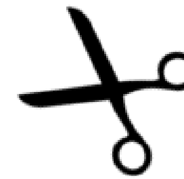
(**Hajiabdolmajid, Martin and Kaiser, 2000** "Modelling brittle failure", NARMS.)

NOW HAVE AN ALTERNATIVE *Q-BASED* WAY TO ESTIMATE ***'c' and 'φ'*** FOR ROCK MASSES!

(but still need to ***degrade c*** at small strain, and ***mobilize φ*** at larger strain)

CC and FC from $Qc = Q \times \sigma_c /100$:

$Qc = RQD/Jn \times Jr/Ja \times Jw /SRF \times \sigma_c /100)$



CC = cohesive strength (the component of the rock mass requiring shotcrete)

FC = frictional strength (the component of the rock mass requiring bolting).

Cut Q_c into two halves → 'c' and 'φ'

$$CC = \frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100}$$

$$FC = \tan^{-1}\left(\frac{Jr}{Ja} \times Jw\right)$$

	$$c' = \frac{\sigma_{ci}\left[(1+2a)s + (1-a)m_b\sigma'_{3n}\right]\left(s + m_b\sigma'_{3n}\right)^{a-1}}{(1+u)(2+a)\sqrt{1+\left(6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}\right)/((1+a)(2+a))}}$$
CC	$$"c" \approx \left(\frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100}\right)$$
	$$\phi' = \mathrm{asin}\left[\frac{6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}{2(1+a)(2+a)+6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}\right]$$
FC	$$"\phi" \approx \tan^{-1}\left(\frac{J_r}{J_a} \times \frac{J_w}{1}\right)$$

GSI-based algebra for 'c' and 'φ'

contrasted with

Q-based 'empiricism'

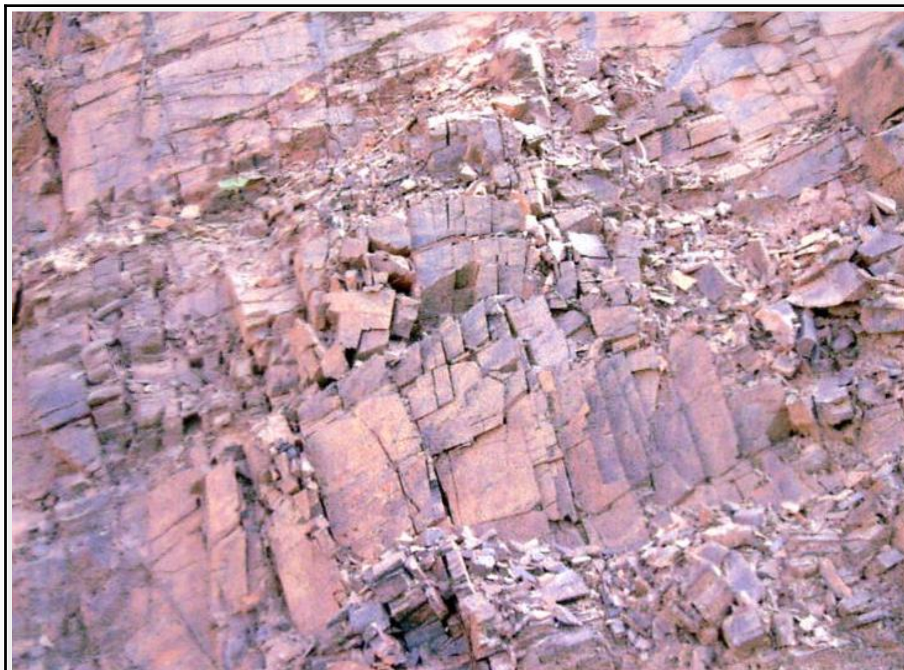
Note: shotcrete needed when low CC, bolting needed when low FC.

RQD	J_n	J_r	J_a	J_w	SRF	Q	σ_c	Q_c	*FC°*	*CC* MPa	V_p km/s	E_{mass} GPa
100	2	2	1	1	1	100	100	100	63°	50	5.5	46
90	9	1	1	1	1	10	100	10	45°	10	4.5	22
60	12	1.5	2	0.66	1	2.5	50	1.2	26°	2.5	3.6	10.7
30	15	1	4	0.66	2.5	0.13	33	0.04	9°	0.26	2.1	3.5

Four rock masses with successively reducing character: more joints, more weathering, lower UCS, more clay.

Low CC –shotcrete preferred

Low FC – bolting preferred



ROCK MASS PERMEABILITY AND Q

(Can there possibly be some relation?)

According to some simple theory in Barton, 2006 - the answer is ‘yes’!

A SERIES OF APPROXIMATIONS.

STRONG LACK-OF-FIT WOULD SUGGEST CLAY-FILLED JOINTS

Q-value	0.1	1	10	100
Lugeon	10	1	0.1	0.01
K ≈ m/sec	10^{-6}	10^{-7}	10^{-8}	10^{-9}
V_P km/s	2.5	3.5	4.5	5.5

TWO VERSIONS OF PERMEABILITY ESTIMATION

No clay present:

$L \approx 1/Q_c$

For hard, jointed, clay-free, rock masses)

(1 Lugeon ≈ 10^{-7} m/s ≈ 10^{-14} m^2 for water at 20ºC)

Q_c = RQD/Jn x **Jr/Ja** x Jw/SRF x σ_c/100

(standard equation, normalized by σ_c/100)

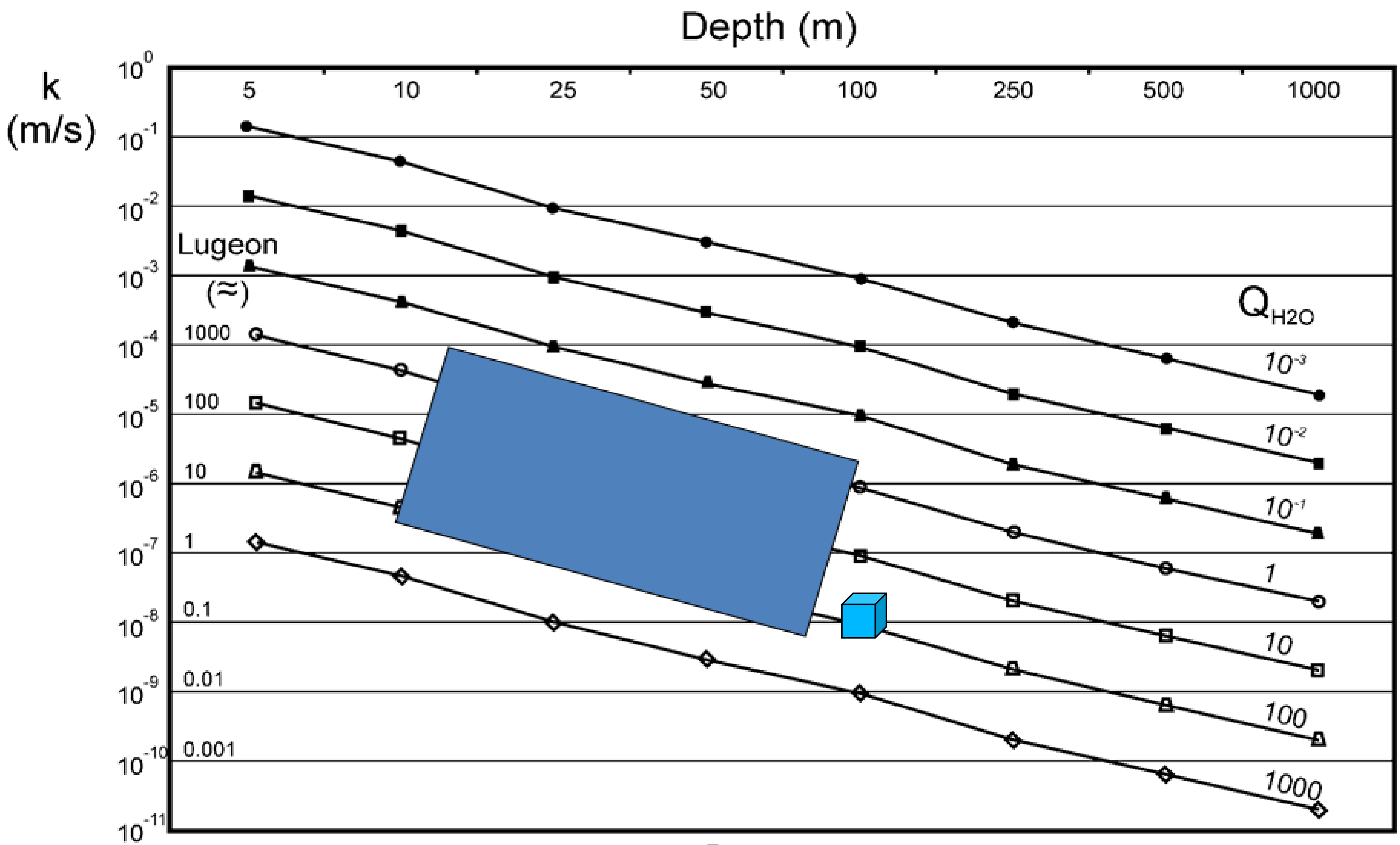
General case, with or without clay, with depth or stress allowance, *and consideration of* ***joint wall strength JCS***

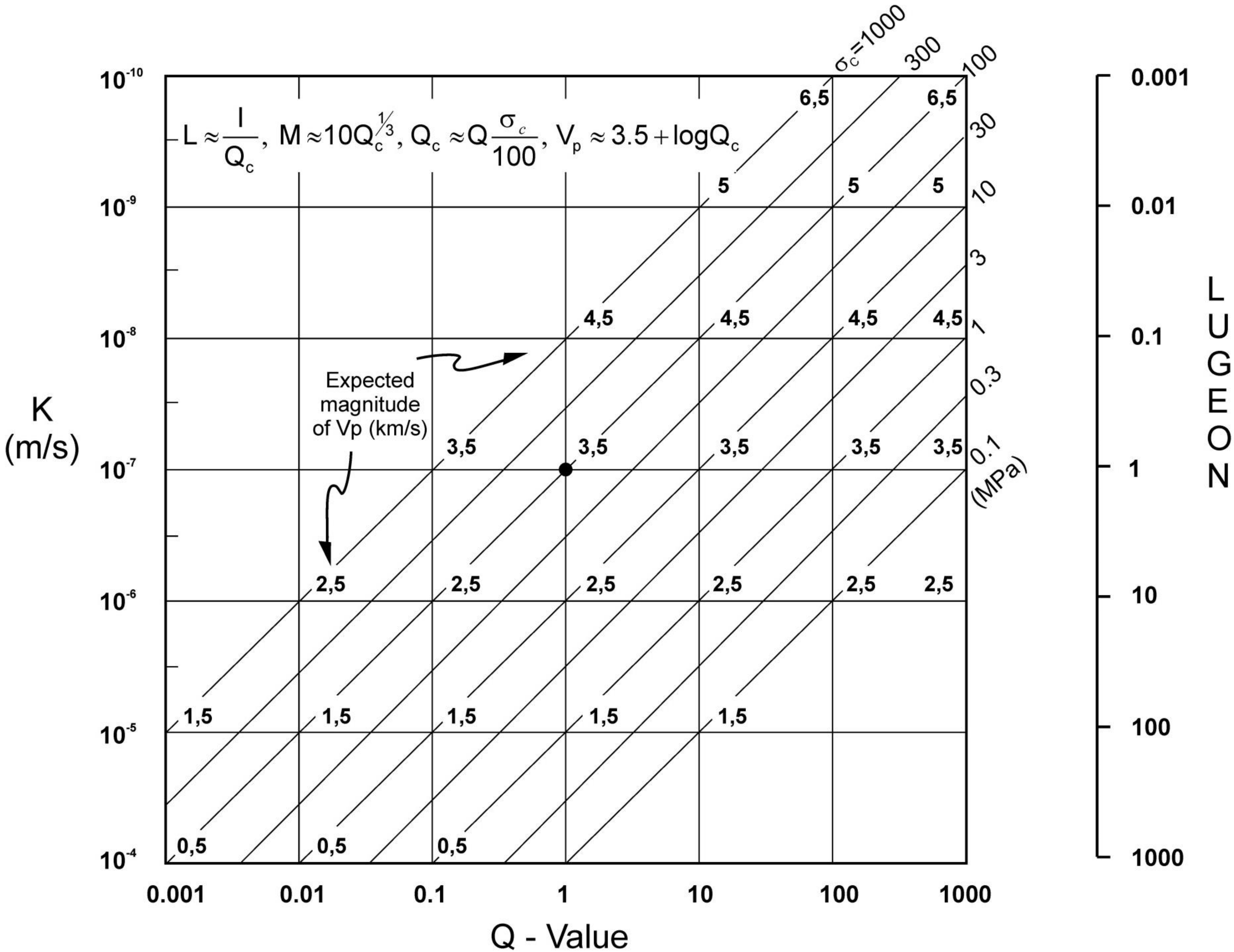
Q_{H2O} = RQD/Jn x *Ja/Jr* x Jw/SRF x 100/JCS

$K \approx 0.002 /(Q_{H20}\ D^{5/3})$ m/s

Clay-bearing, well-jointed rock at 100 m depth, with a low assumed JCS of 10 MPa due to low UCS of 15 MPa.
Regular Q-value = 50/9 x 1.5/4 x 0.66/1 = 1.4, i.e. 'poor'
$Q_{H_2o} = \frac{50}{9} \times \frac{4}{1.5} \times \frac{0.66}{1} \times \frac{100}{10} = 98$
The estimated result is **$K \approx 10^{-8}$ m/s** (at 100 m depth) (Quite low permeability due to clay coatings, and compressible joint walls, despite the well-jointed nature of this Q = 1.4 rock mass).

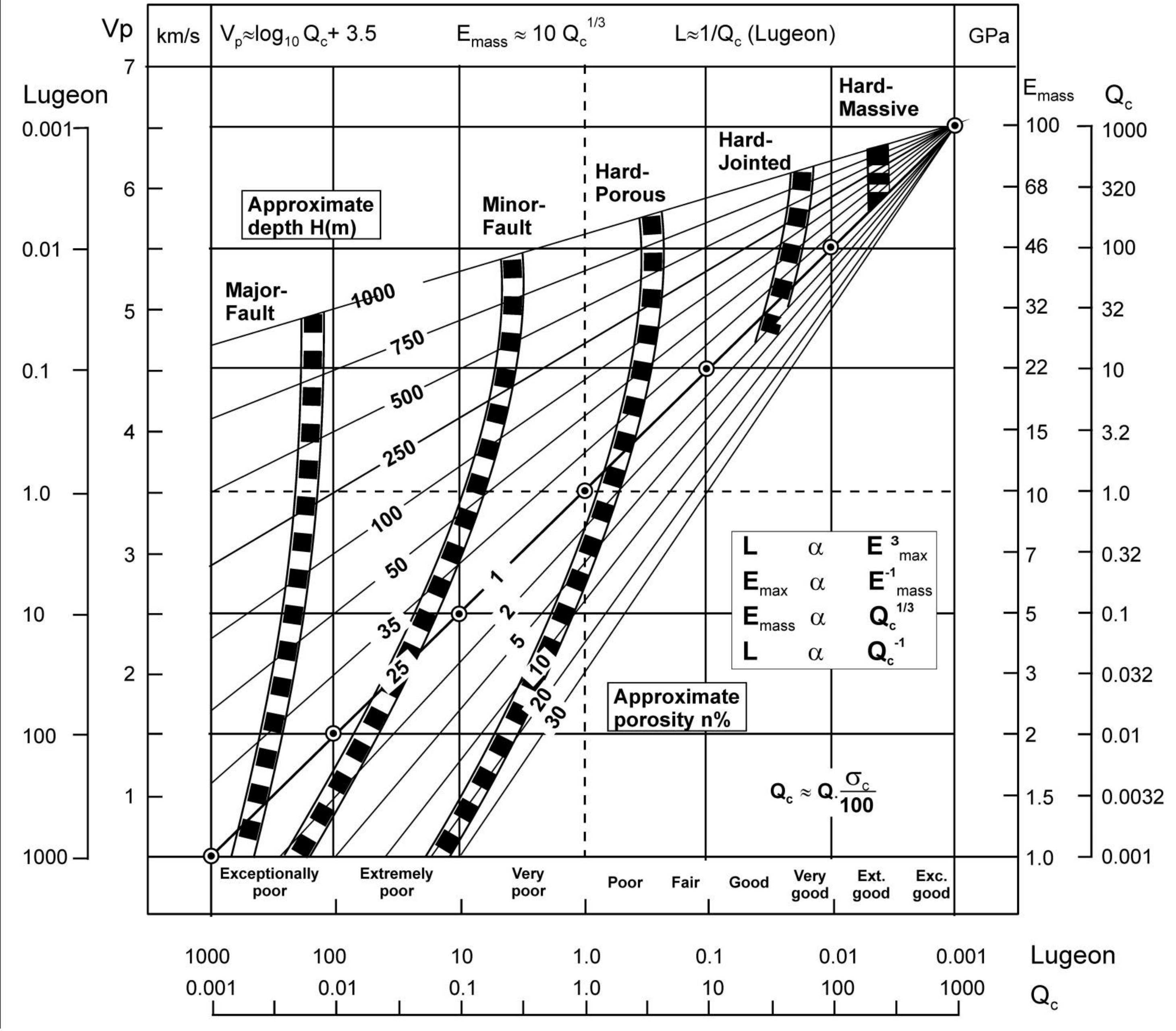
USUAL RANGE OF K at DAM SITES





$L \approx \frac{I}{Q_c}$, $M \approx 10 Q_c^{1/3}$, $Q_c \approx Q \frac{\sigma_c}{100}$, $V_p \approx 3.5 + \log Q_c$
Expected magnitude of Vp (km/s)
K (m/s)
Q - Value
LUGEON
σc=1000
300
100
30
10
3
1
0.3
0.1
(MPa)
6,5
5
4,5
3,5
2,5
1,5
0,5
10^-10
10^-9
10^-8
10^-7
10^-6
10^-5
10^-4
0.001
0.01
0.1
1
10
100
1000
0.001
0.01
0.1
1
10
100
1000

?



‘Type curves’ for rock masses, with tentative cross-integration of some key parameters

LESSONS LEARNED FROM TWO SPECIFIC PROJECTS

- ❑ ONE INVOLVING SHALLOW TUNNELLING UNDER HIGH STRESS
- ❑ ONE INVOLVING A SHALLOW CAVERN WITH *BENEFICIAL* STRESS

HYDROELECTRIC PROJECT TUNNELLING

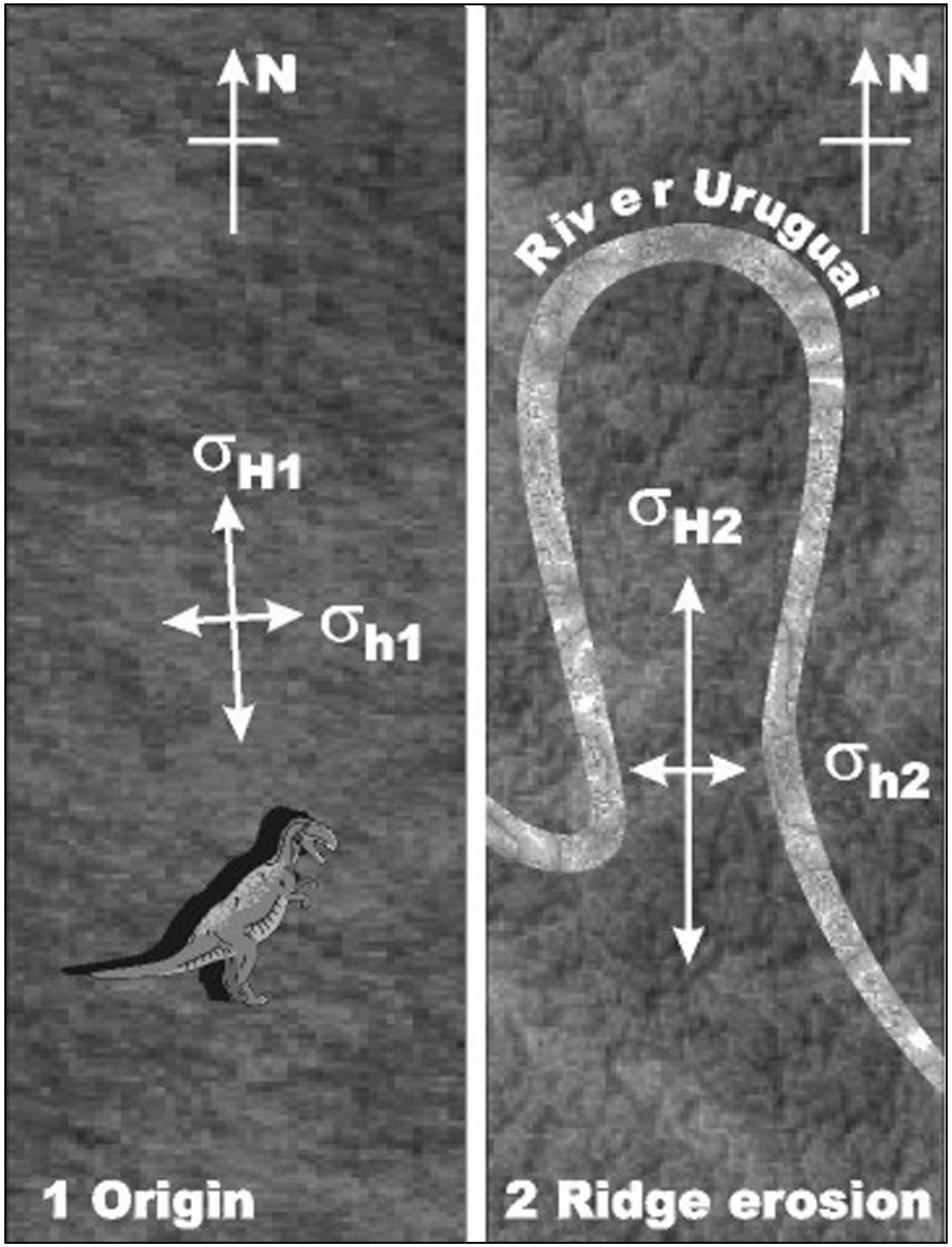
ITA HEP, BRAZIL

SUFFERED FROM VERY HIGH ROCK STRESS DESPITE SHALLOW TUNNELLING

Major horizontal stress = NNW.

Ridge formed of intrusive dyke has same orientation.

HEP ITA CONSTRUCTED ACROSS NARROWEST < 1km wide RIDGE

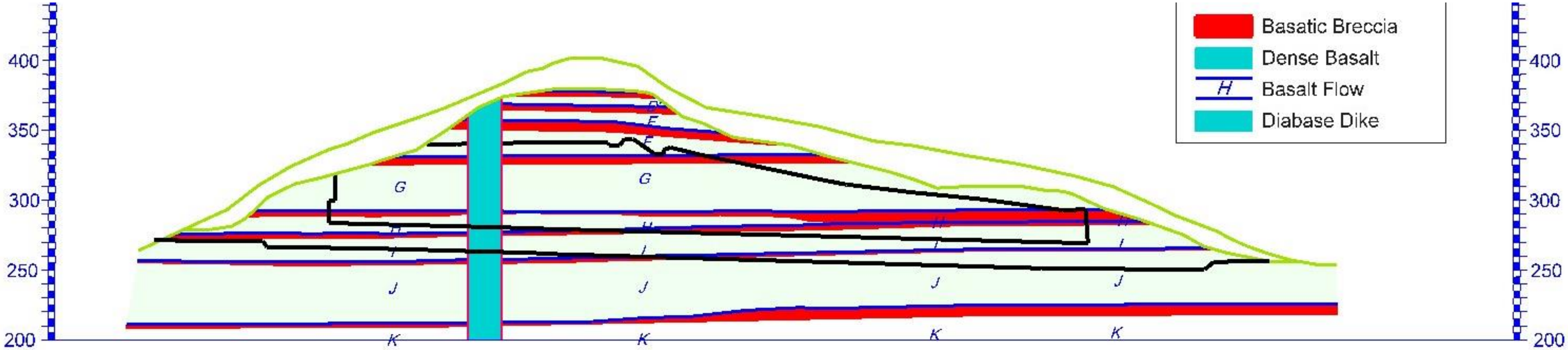


Regional stress of long ago, induces fracturing which *prejudices the meander direction* of the river.

The river eventually causes a *stress concentration in the ridge.*

$\sigma_{H2}/\sigma_{h2} >> \sigma_{H1}/\sigma_{h1}$

The 125 m-high Itá concrete-faced rockfill dam and 1450 MW powerplant in Brazil



AT THE SITE THERE ARE 4 SPECIFIC BASALT FLOWS

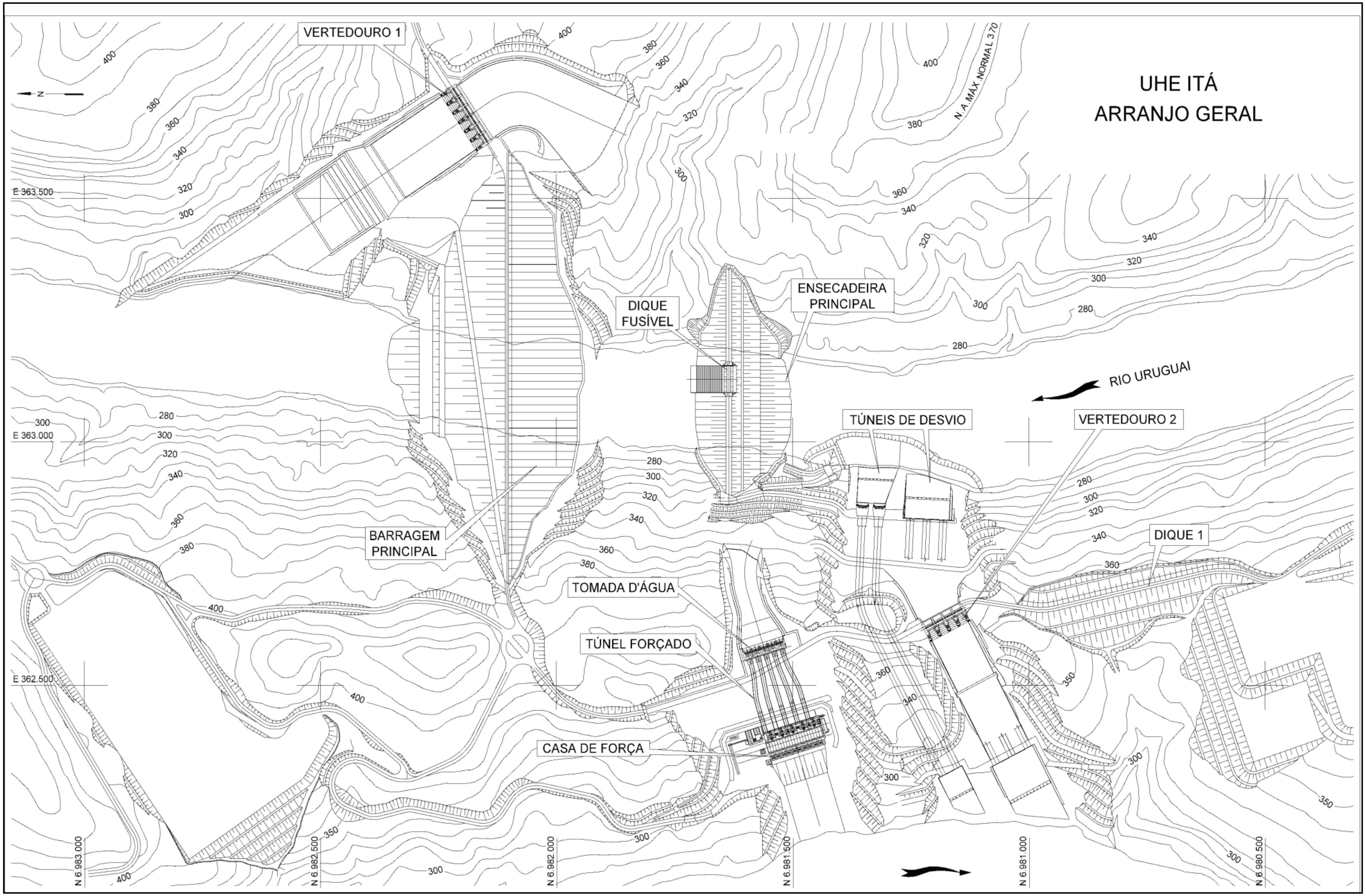
TWO ARE MASSIVE……………….HIGH Q-value………HIGH E $_{MODULUS}$
TWO ARE JOINTED……………..LOWER Q-value……LOWER E $_{MODULUS}$

(THE 'H' AND 'I' FLOWS ARE MASSIVE….and apparently attract higher stress)

Flows G and J (jointed flows) $$Q = \frac{70-90}{6-9} x \frac{1.5-2}{1-2} x \frac{.66}{1}$$ **Q = 5 to 13**

Flows H and I: (massive flows) $$Q = \frac{90-100}{3-6} x \frac{1.5-4}{.75-1} x \frac{1}{1}$$ **Q = 30 to 100**

Prior to assumption of significant stress difference **between the two pairs of flows, following preliminary Q-ranges of 5 to 13, and 30 to 100 respectively.**



LAYOUT OF THE ITA HYDRO-ELECTRIC PROJECT

STRESS-INDUCED PHENOMENA

1. Crack of 80 m length across spillway 'exit'.

2. Stress-induced fracturing 3 to 4 m deep in large diversion tunnels ('12 o'clock' and '6 o-clock').

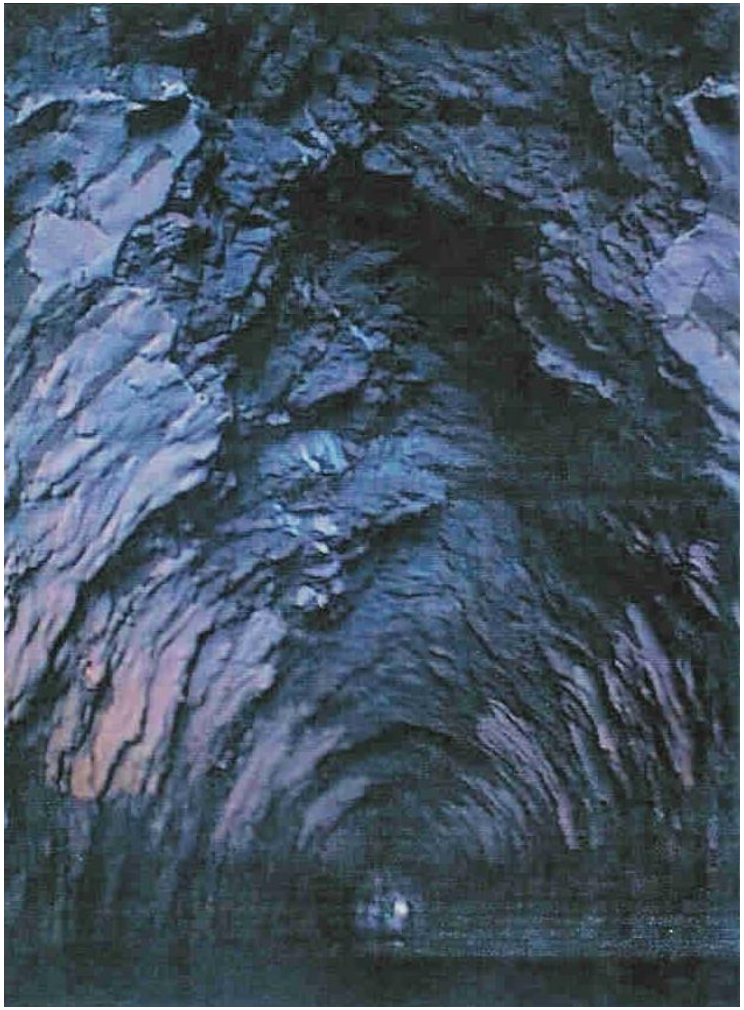
3. Long 'linear' cracks (hundreds of meters in total) along all the inclined pressure tunnels ('9 o-clock' and '3 o-clock').

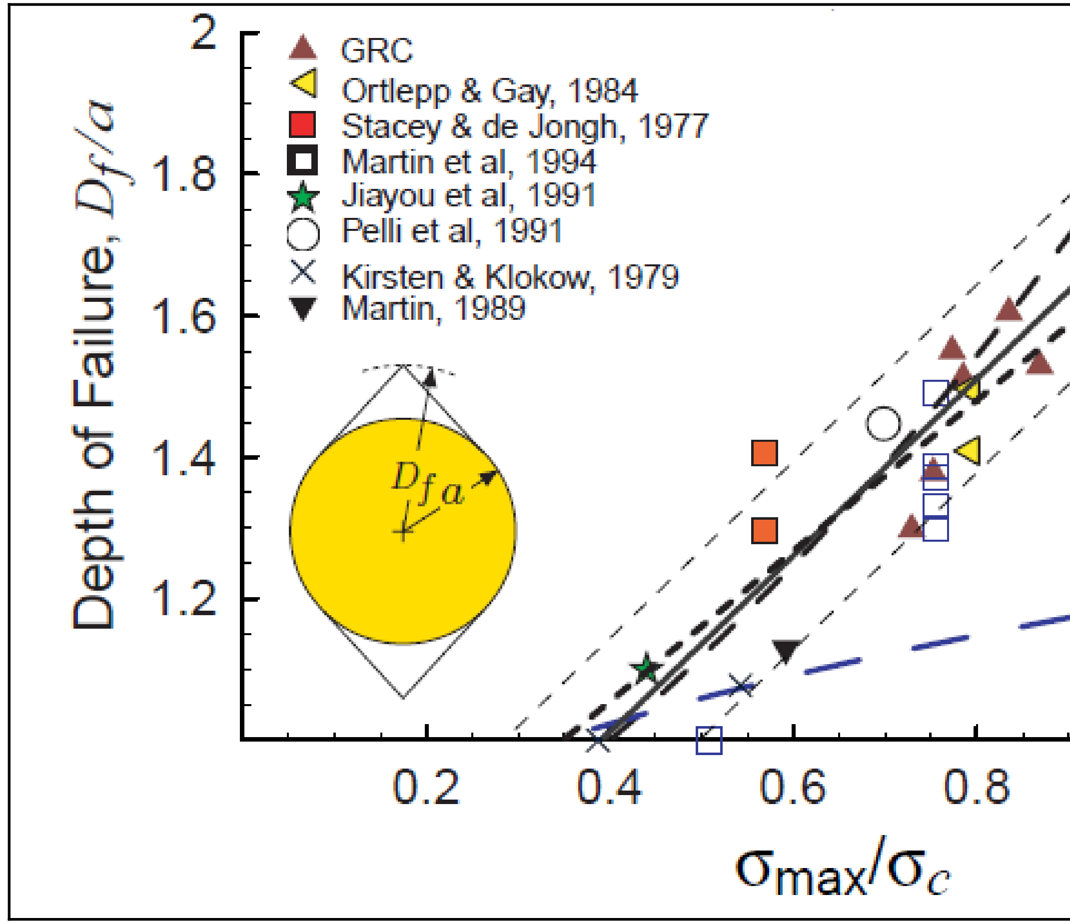
4. Extensive 'N-*S*' cracking, both vertical and horizontal, dividing the emergency spillway basalts into small blocks which eroded too fast.

STRESS-INDUCED PROBLEMS IN:

- River diversion tunnels: *two* main tunnels (1 and 2) 14,0m x 14,0m
- *Three* auxiliary tunnels (3, 4 and 5) 15,0m x 17,0m high, (operate during floods)
- Pressure tunnels 5@ 120m, 9 m diameter, 53º inclined (concrete lined: lower section: steel penstock)
- Emergency spillway

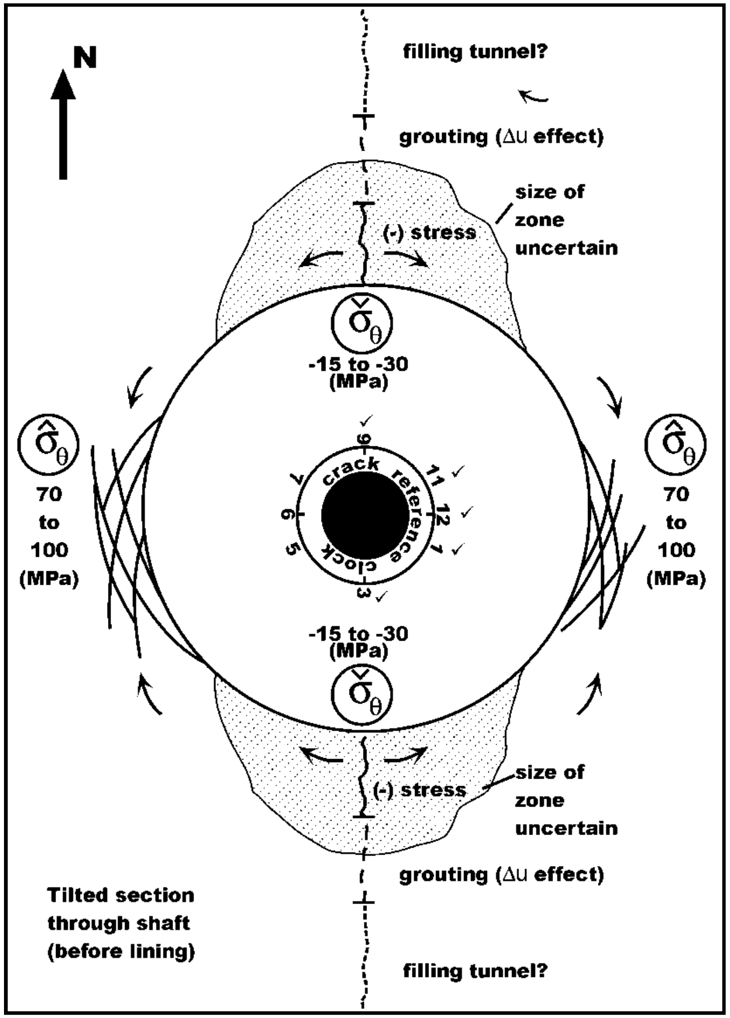
Ita HEP, Brazil $\sigma_H \approx 40$ MPa, $\sigma_V \approx 1.5$ MPa at 50m depth (!)
$\sigma_\varphi / \sigma_c \approx 115 / 200 \approx 0.6$ SRF ≈ 25 to 35
depth of failure/ 'radius' $(D_f / a) \approx (3+7m)/7m \approx 1.4$
(see $\sigma_{max} / \sigma_c \approx 0.6$) next figure

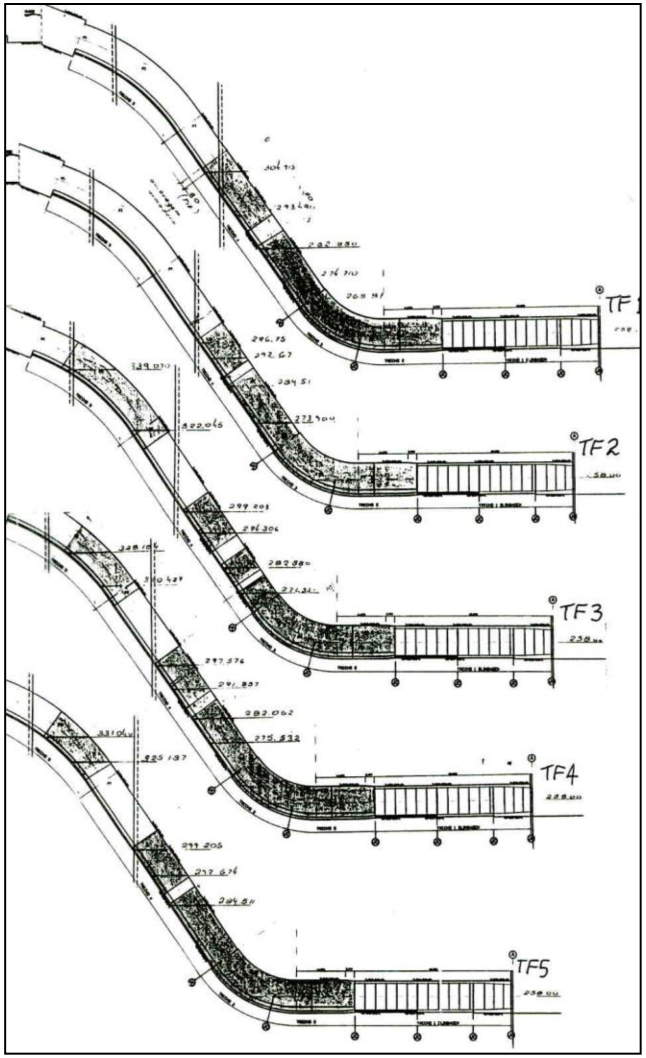




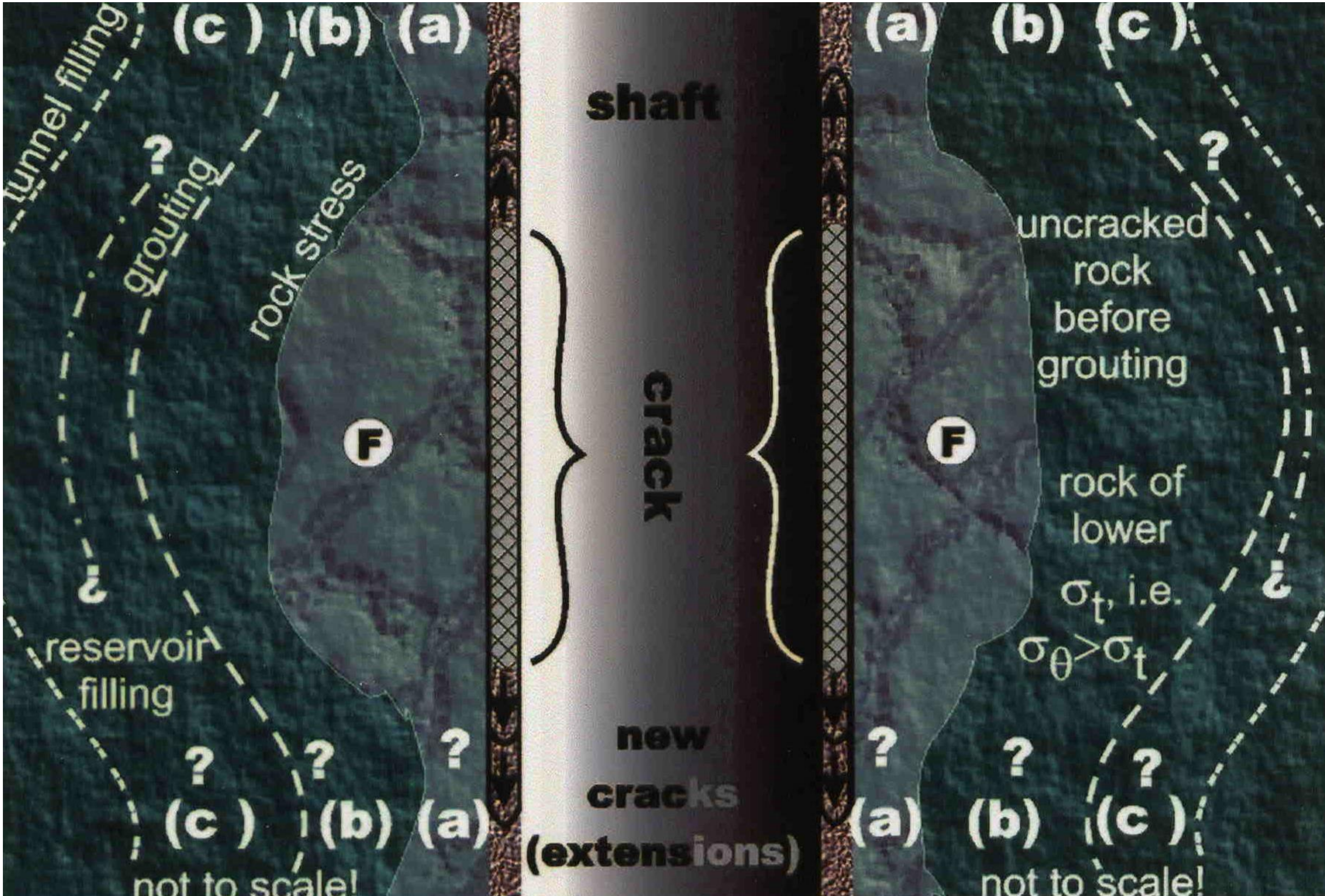
THE REALITY OF STRESS-INDUCED FAILURES IN THE ARCHES (AND INVERTS) OF THE DIVERSION TUNNELS. MANY THOUSAND m^3 OF ROCK FAILED…and were then eroded by the water – most in the invert.

THE PRESSURE TUNNELS….5 of them, inclined at 53˚





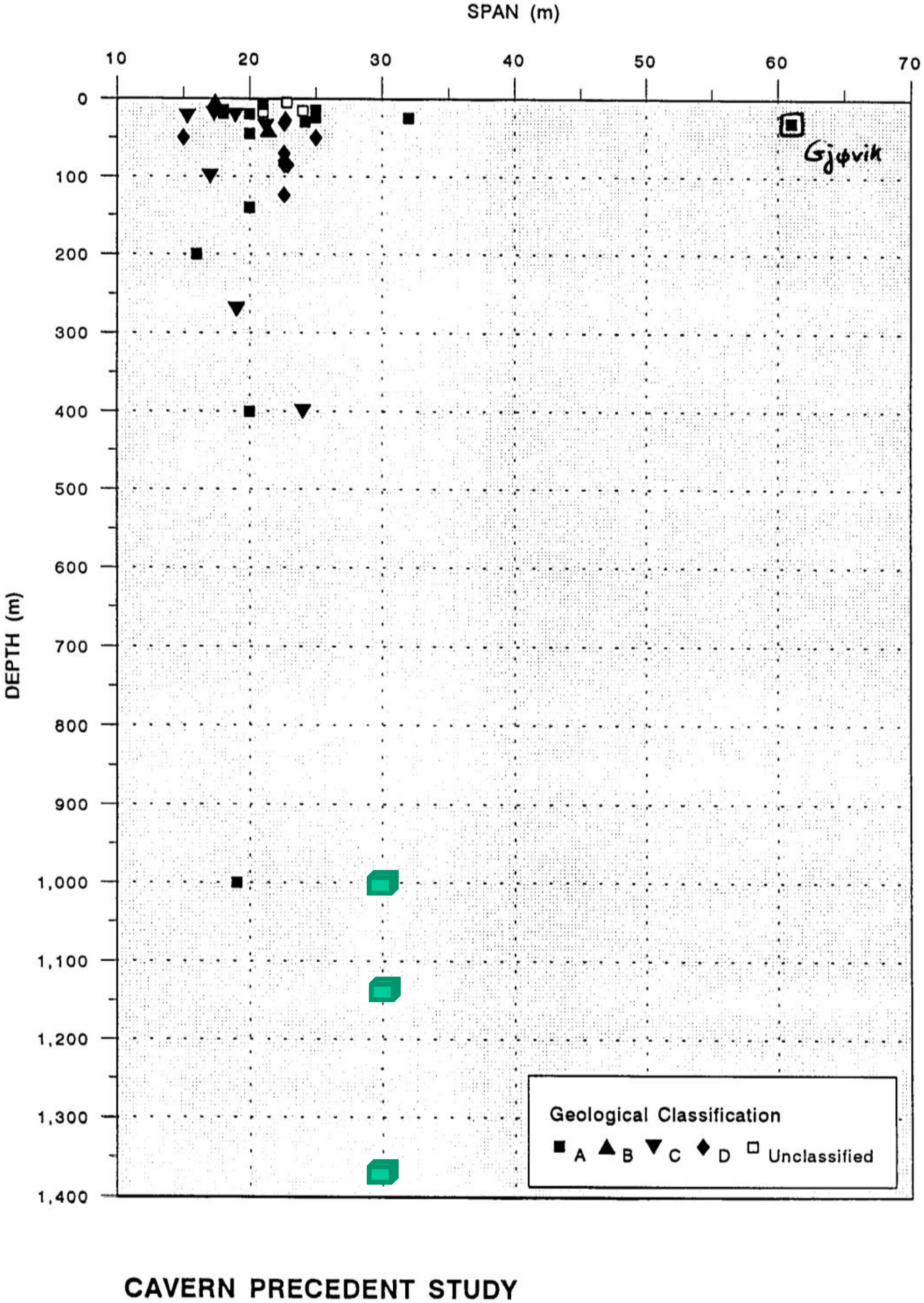
EACH SHAFT WAS CRACKED (SHADED)



shaft
crack
new cracks (extensions)
(a) (b) (c)
tunnel filling
grouting
rock stress
reservoir filling
F
uncracked rock before grouting
rock of lower σ_t, i.e. $\sigma_\theta > \sigma_t$
not to scale!
not to scale!

GJØVIK OLYMPIC CAVERN

INCREASE OF LARGEST CAVERN SPAN BY ALMOST 2 x



Gjøvik Olympic cavern represented a big jump.......in span and confidence!

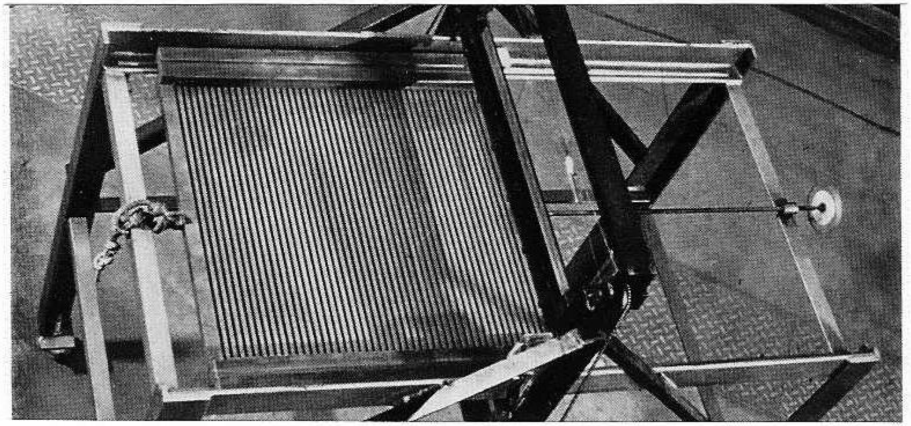
(Figure from Sharp, 1996: UK Nirex study)

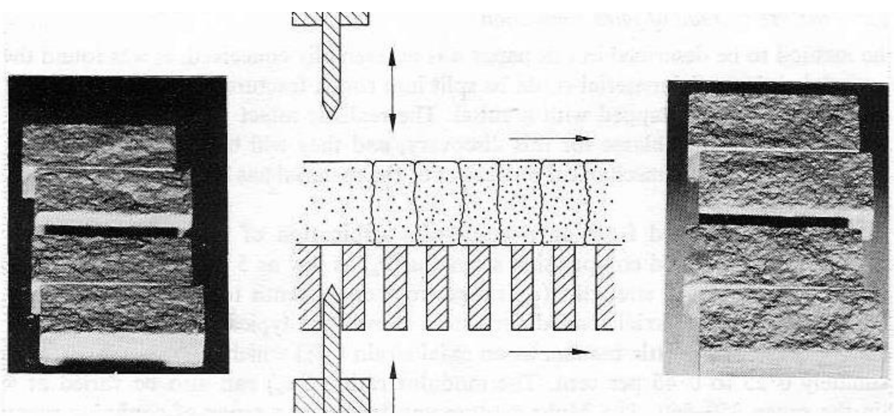
BLUE: Lærdal Tunnel (three, lorry-turning and *'wake-up-driver'* caverns in 24.5 km long tunnel)

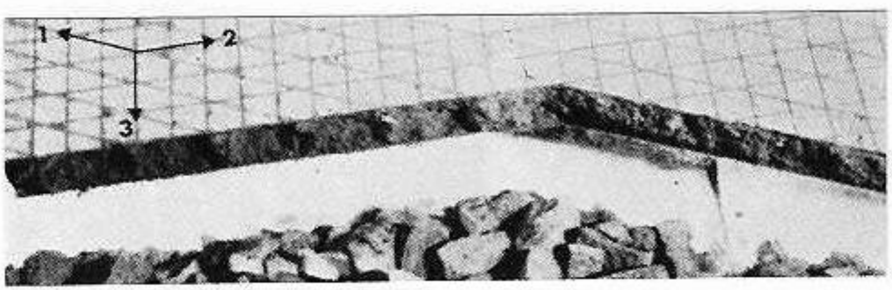
LÆRDAL TUNNEL lorry-turning caverns (three of them) 30 m span, depths 1,000 to 1,400 m (Photo G.Lotsberg)

EARLIER STUDIES FOR UNDERGROUND NUCLEAR POWER PLANTS with 50 m spans in 1970's. These were fore-runners for future 'Gjøvik'.

PHYSICAL (2D) MODELS of ROCK CAVERNS, *PERFORMED SOME YEARS BEFORE UDEC-BB FLEXIBILITY*

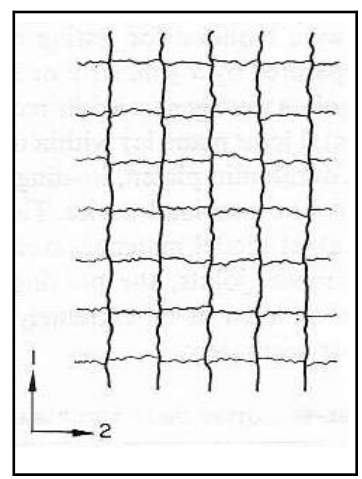


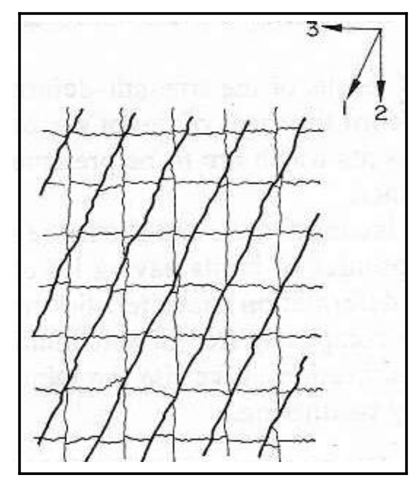


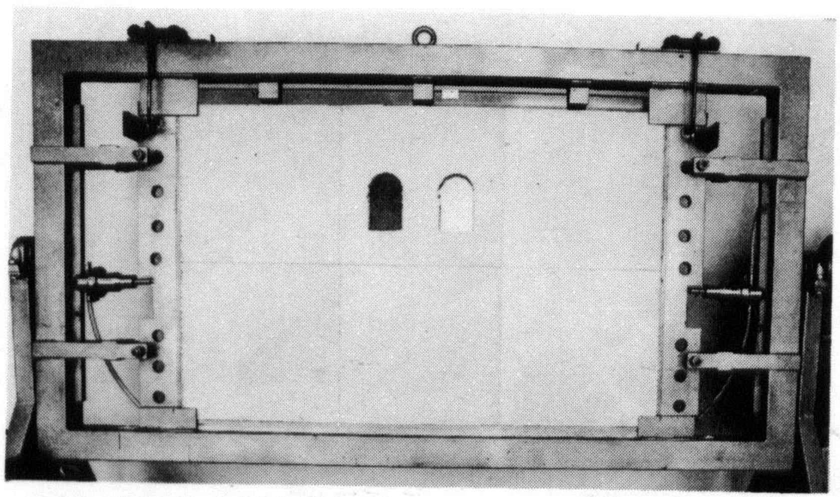
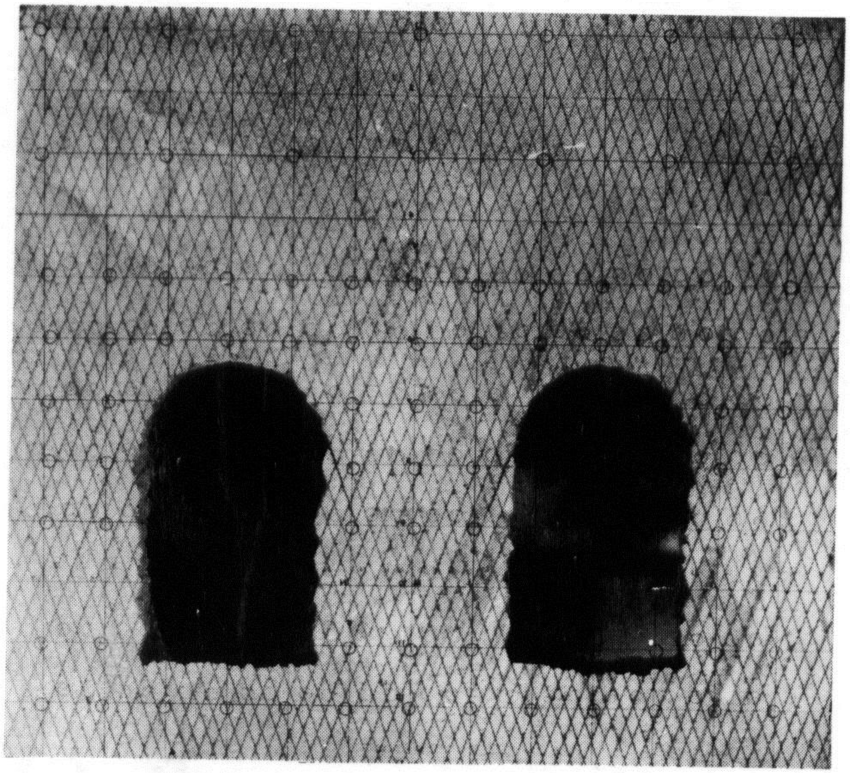
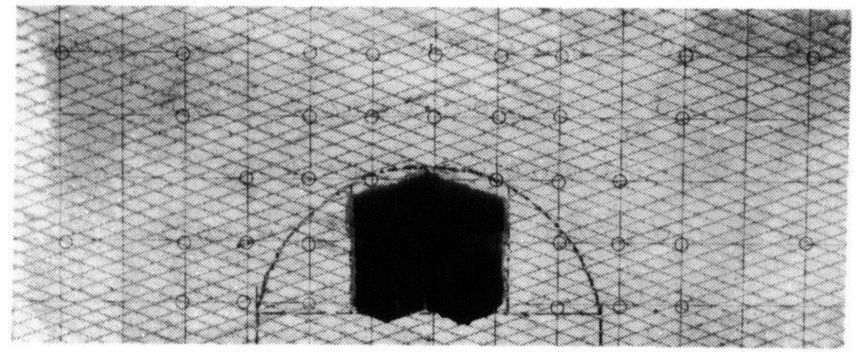
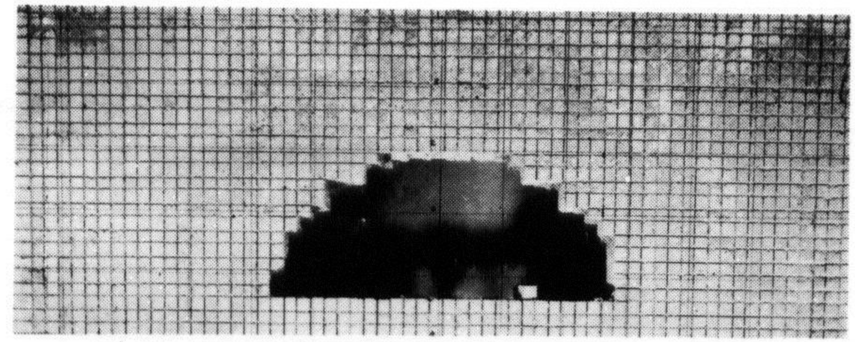
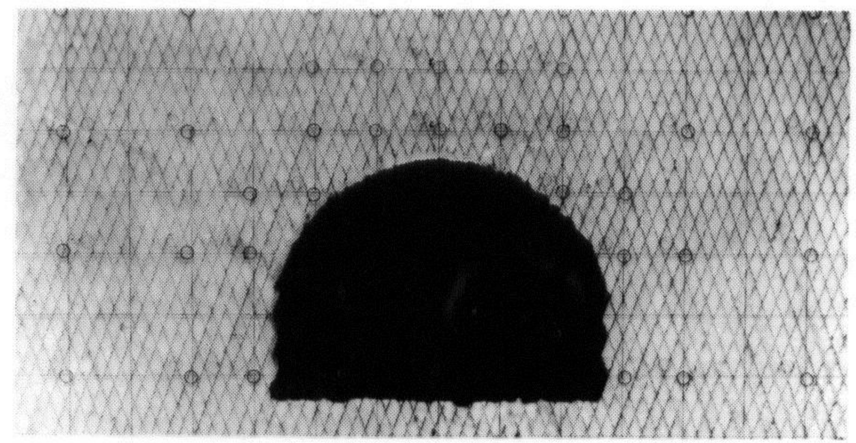


"Jointed rock-mass" (1968-1969 photos) Barton, 1971

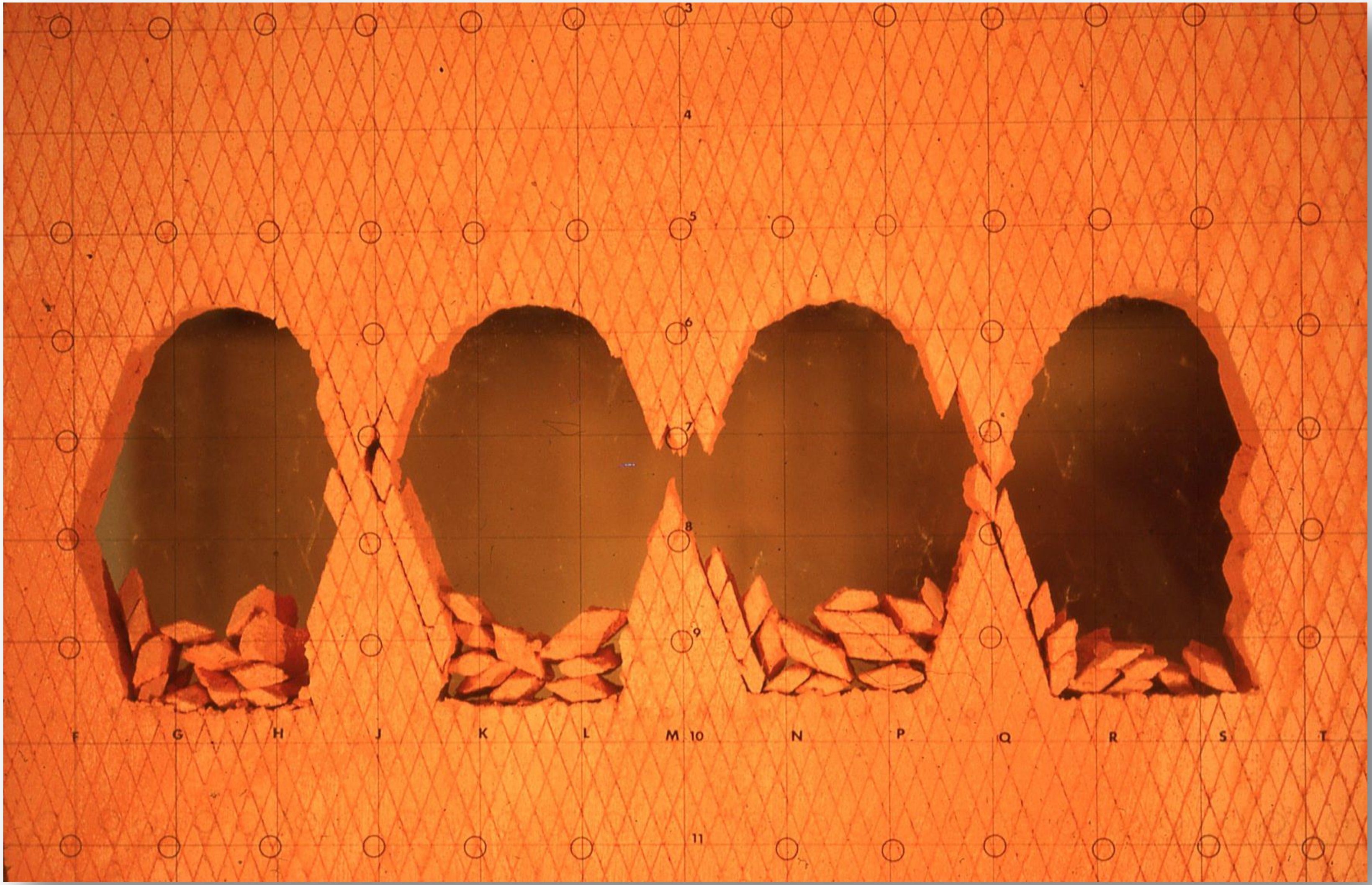
Tension-fracture models for slope and cavern investigations (pre-UDEC)

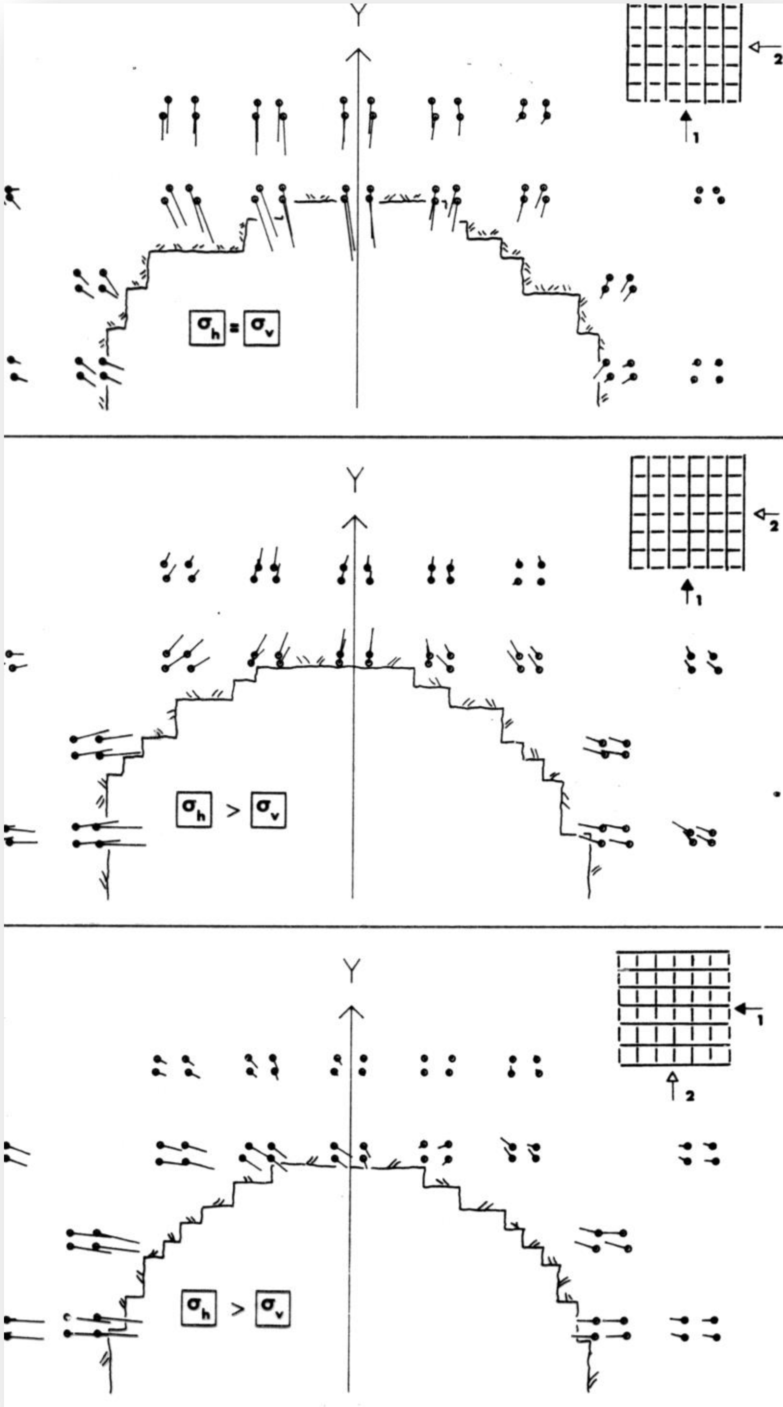




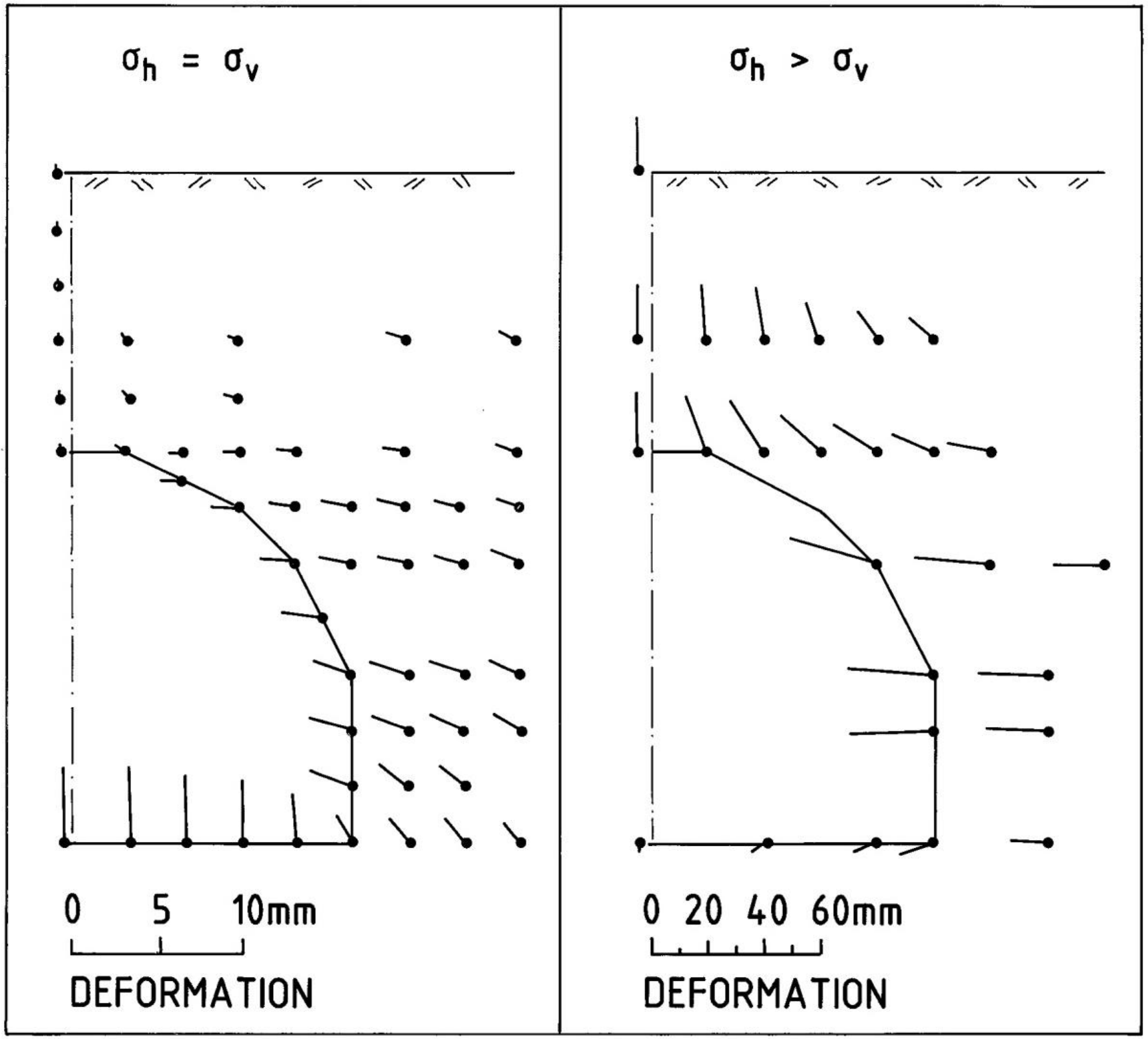
Post-seismic loading result (0.2 to 0.5 g)





Physical and FEM modelling (Barton and Hansteen, 1979) suggested possible 'heave' resulting from large-cavern construction near the surface…….

……….depended on joint pattern and horizontal stress level in the physical models.



FEM continuum modelling of large caverns had also suggested the possibility of 'heave' if σ_H was large enough (H.Hansteen),

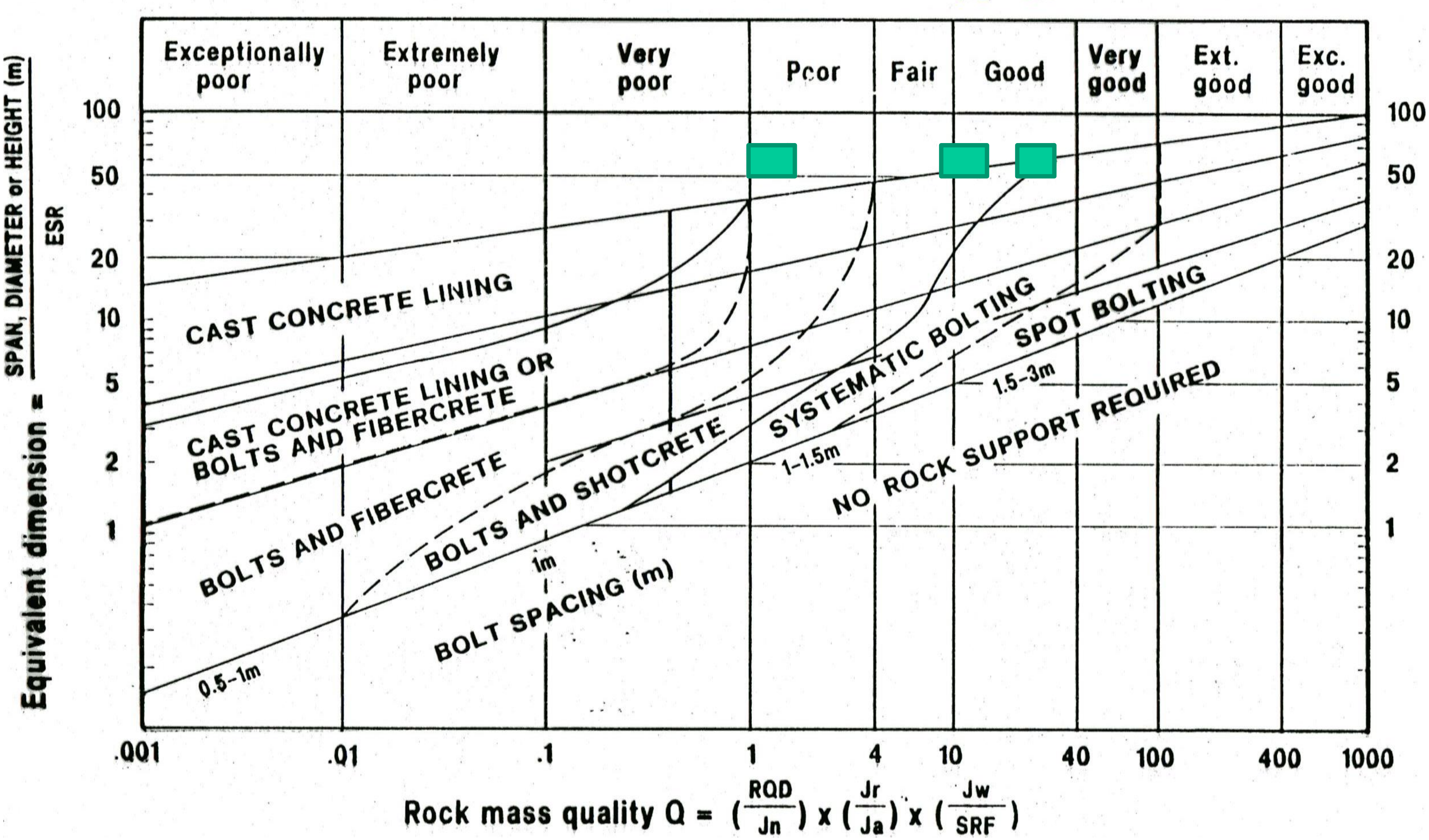
But we did not know what to expect for the 62 m span jointed structure

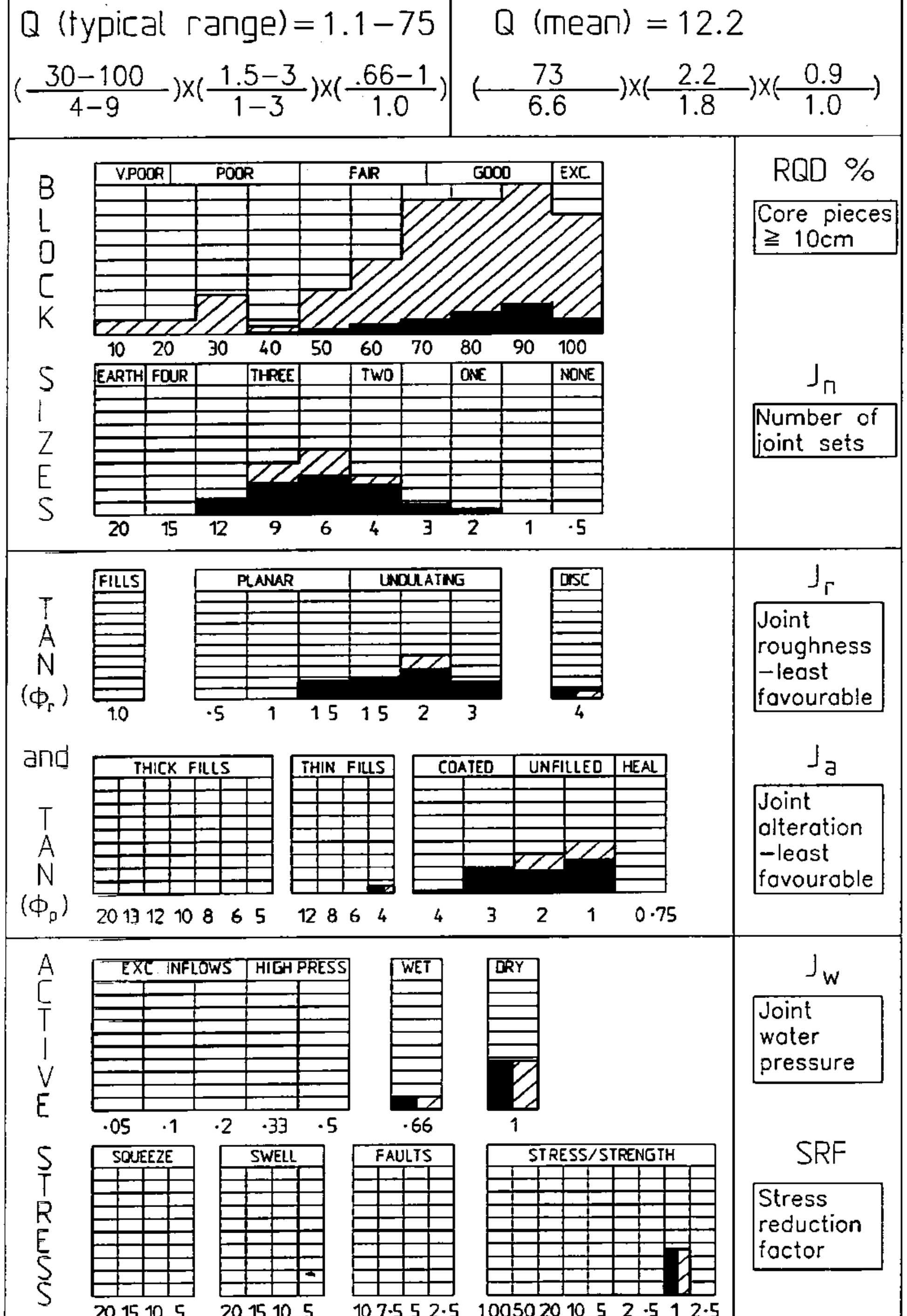
(10 years later)

Gjøvik cavern : represented an 'extension' of 1974 Q-system data base.

(Q_{min}, Q_{mean}, and Q_{max} values of 1, 12, 30 logged in the cavern arch)

RQD = 60-90%, UCS = 90 MPa was typical.



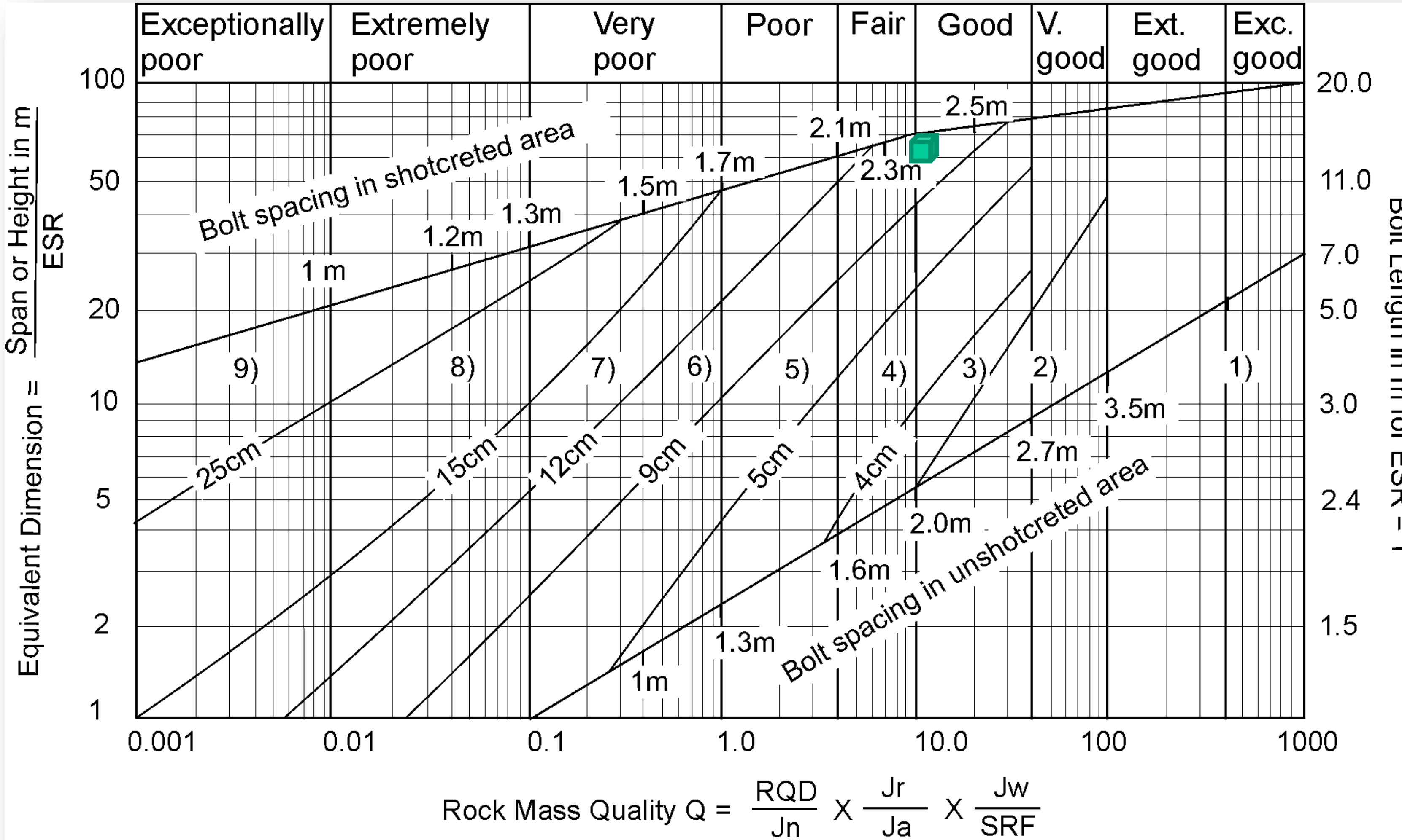


Q-characterization using pre-construction data

Core logging = //////////////

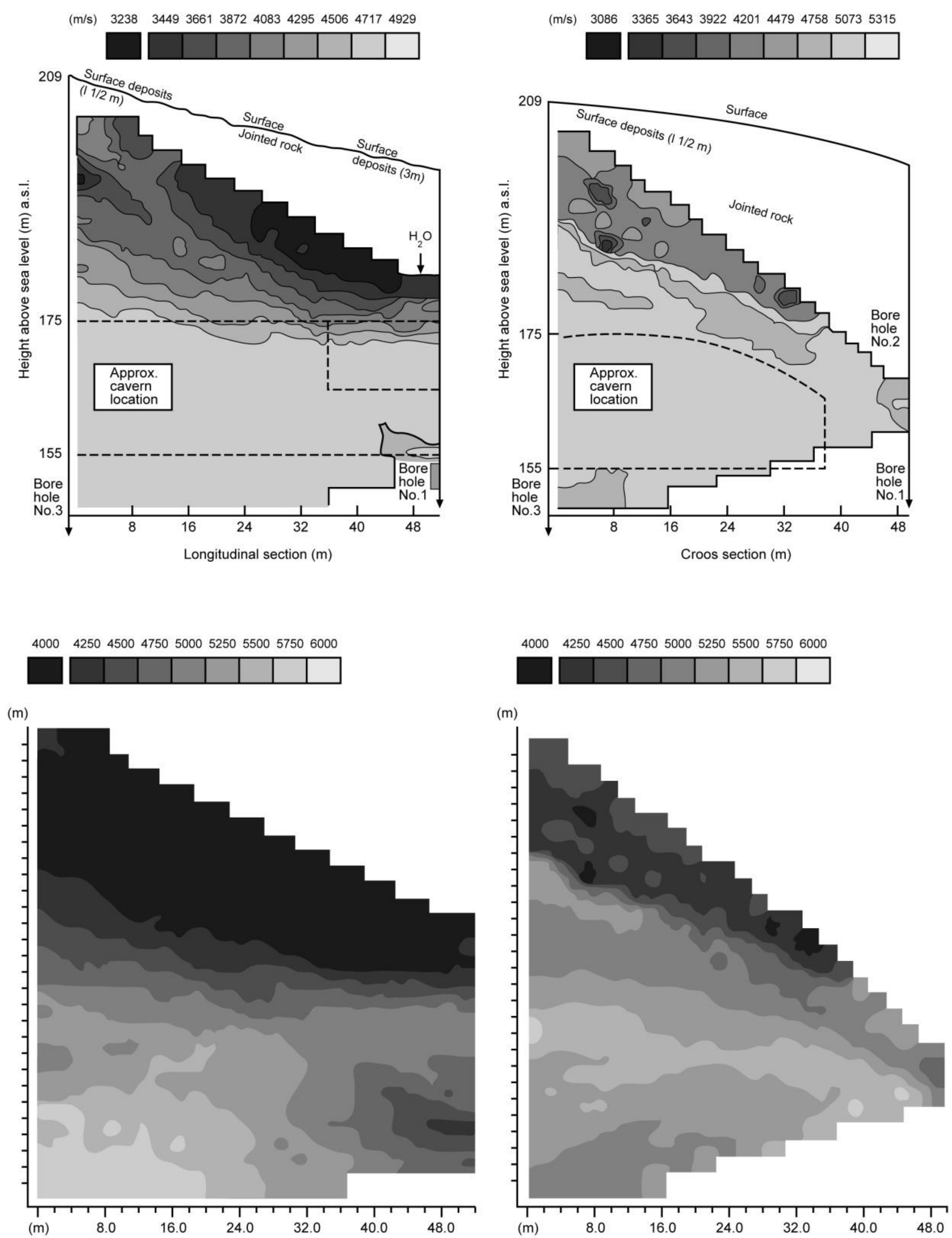
Existing nearby cavern = █

The boreholes used for core recovery were also permeability tested (K mostly ≈ 10^{-7} to 10^{-8} m/s), ≈ consistent with L ≈ $1/Q_c$



REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting, sb
3) Systematic bolting, B
4) Systematic bolting (and unreinforced shotcrete, 4-10cm, B(+S)
5) Fiber reinforced shotcrete and bolting, 5-9cm, Sfr+B
6) Fiber reinforced shotcrete and bolting, 9 - 12cm, Sfr+B
7) Fiber reinforced shotcrete and bolting, 12 - 15cm, Sfr+B
8) Fiber reinforced shotcrete > 15cm, reinforced ribs of shotcrete and bolting, Sfr, RRS+B
9) Cast concrete lining,CCA

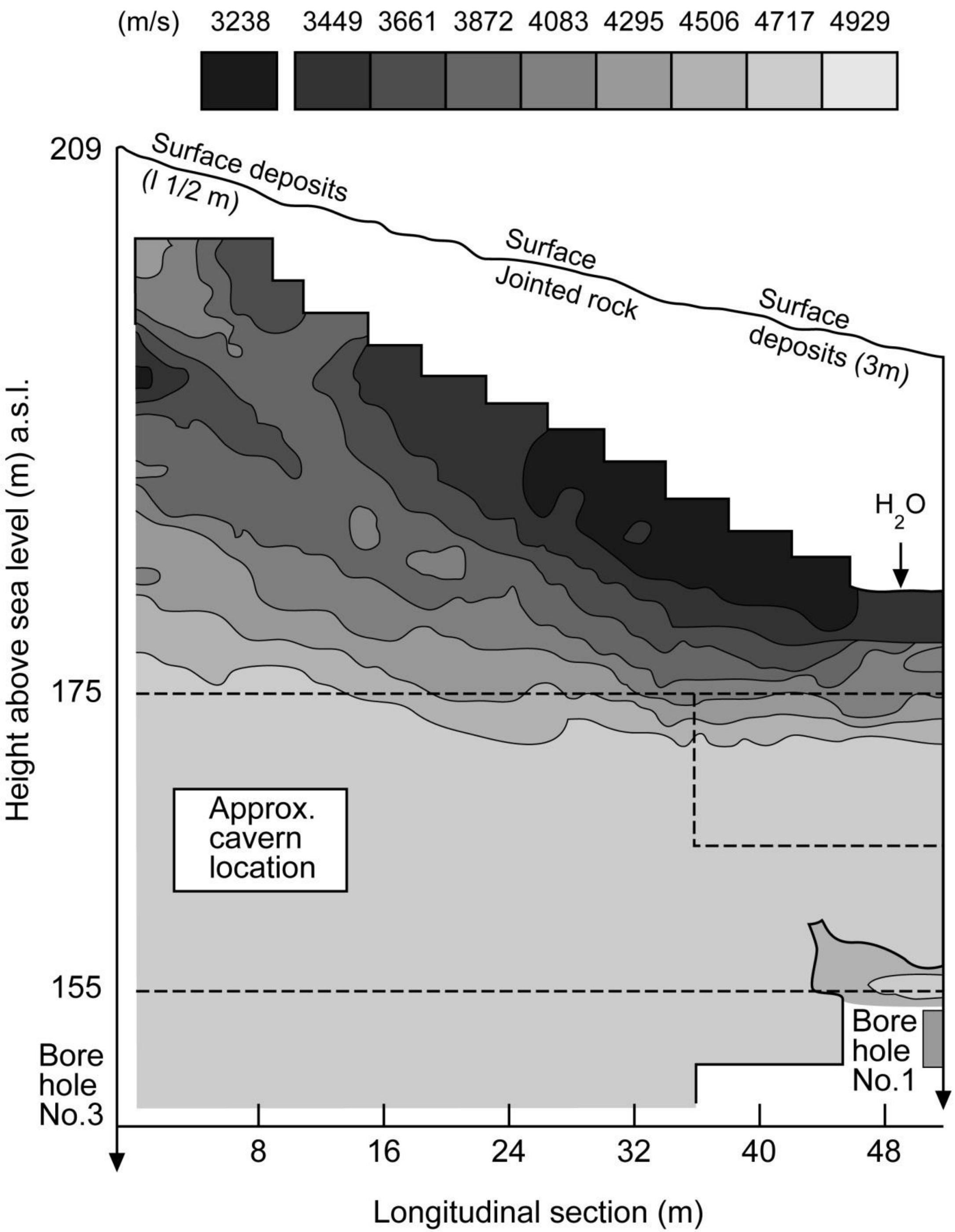


Cross-hole seismic tomography at Gjøvik showed the expected increase in velocity with depth........

but it was more than expected due to stress-gradient effects....

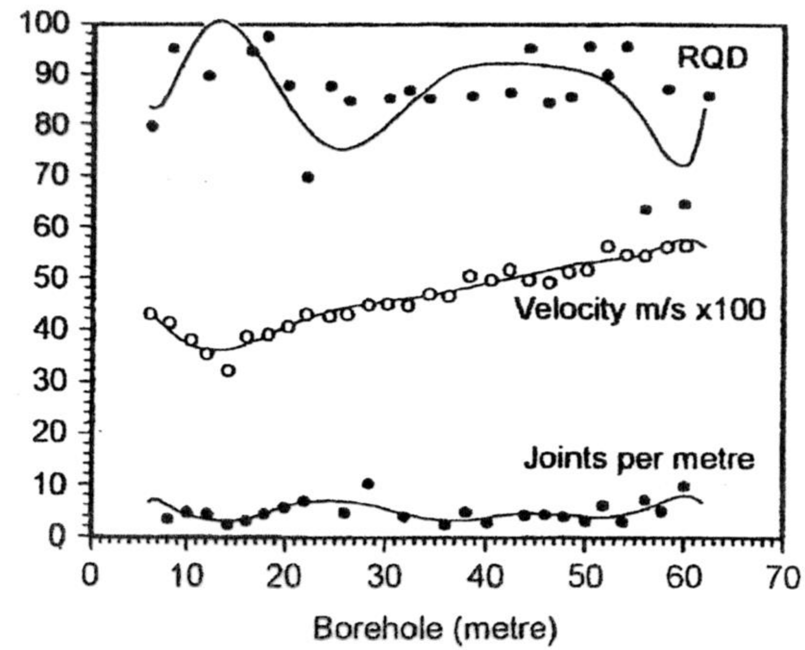
the quality was not 'as-good-as-the-velocity'

(Barton, 2006)



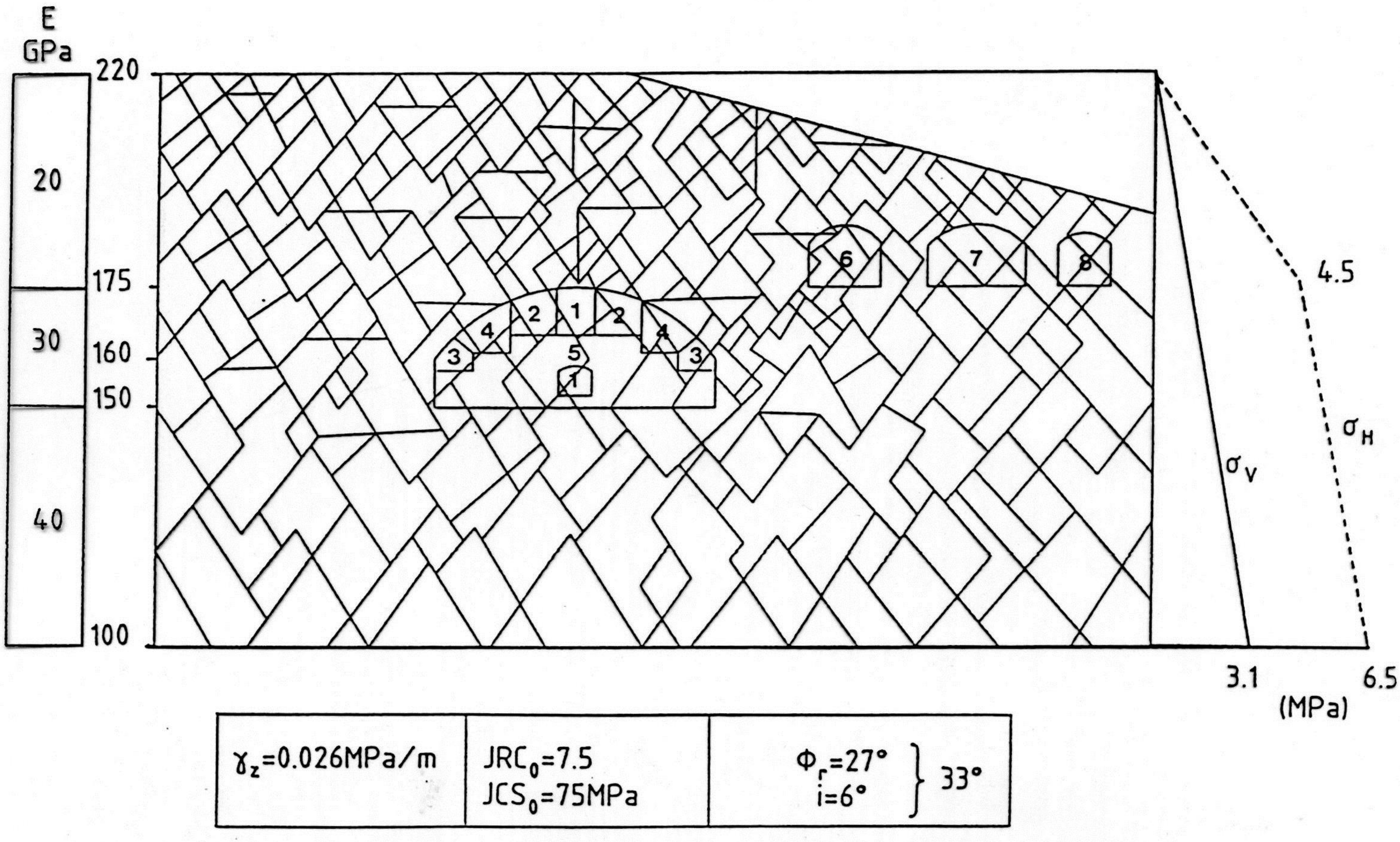
Despite no improvements to rock quality with depth (below top 5 m), velocity continued to rise.

(Barton et al. 1994)

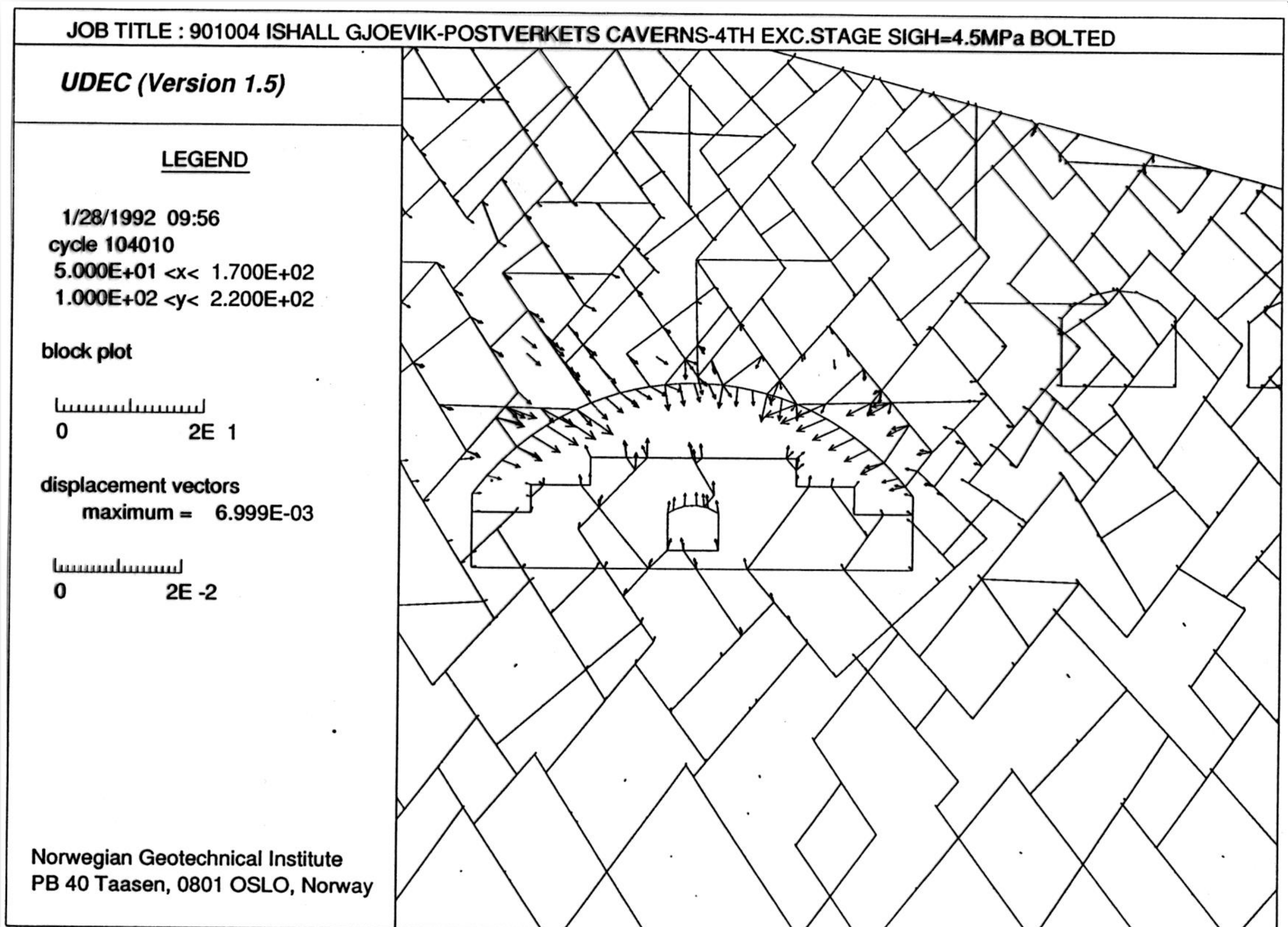


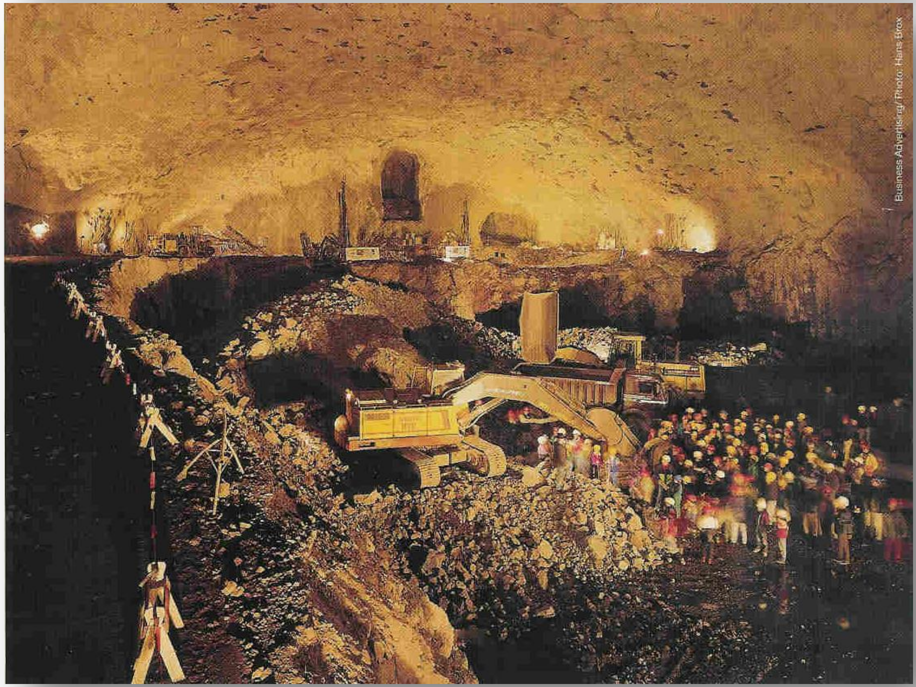
GJØVIK CAVERN JOINT-GEOMETRY ASSUMPTIONS
Input data, boundary stresses

Barton, N., By, T.L., Chryssanthakis, P., Tunbridge, L., Kristiansen, J., Løset, F., Bhasin, R.K., Westerdahl, H. & Vik, G. 1994. Predicted and measured performance of the 62m span Norwegian Olympic Ice Hockey Cavern at Gjøvik. *Int. J. Rock Mech, Min. Sci. & Geomech. Abstr.* 31:6: 617-641. Pergamon.



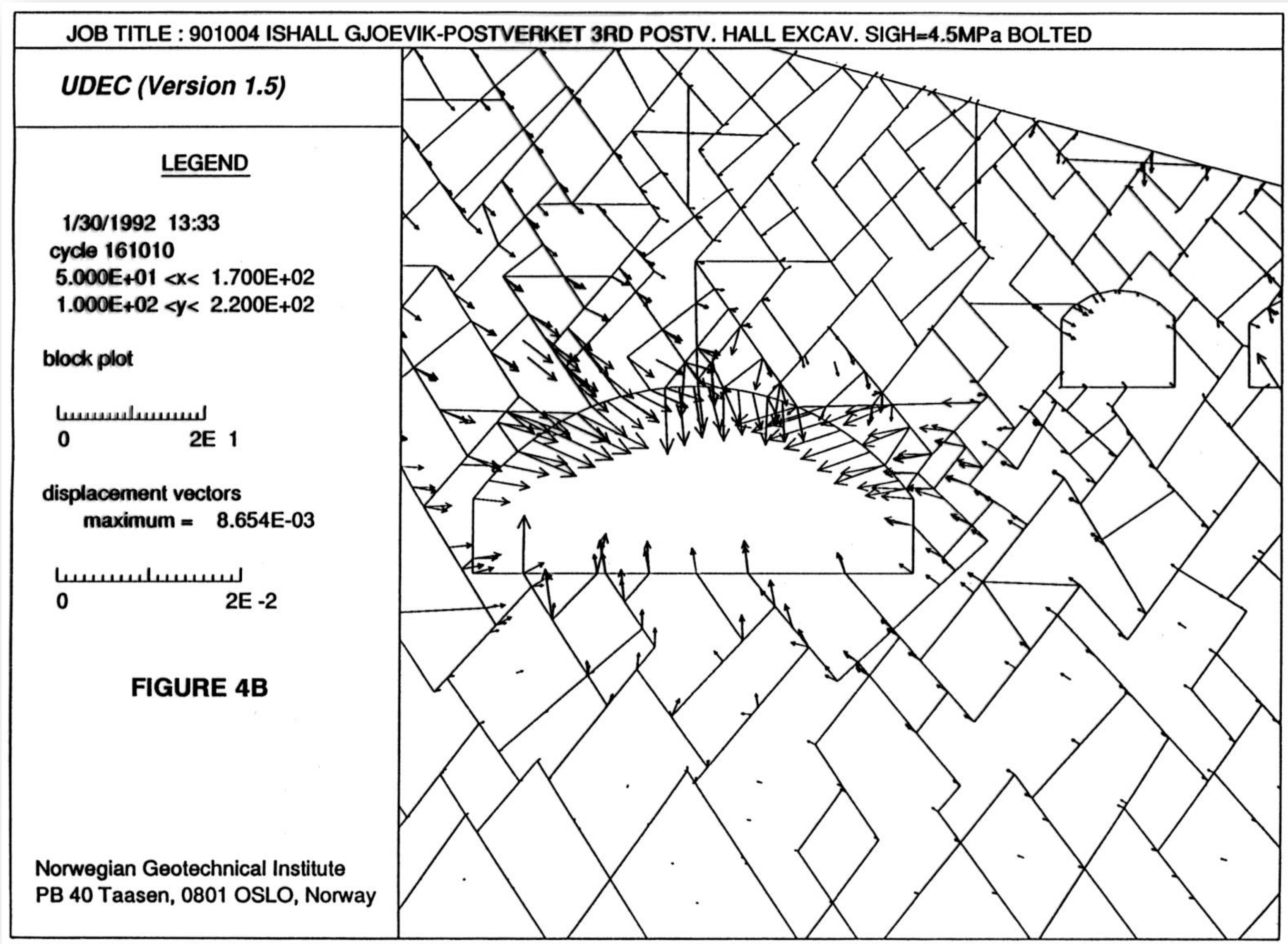
TOP HEADING TOO WIDE TO OBSERVE FROM ONE LOCATION



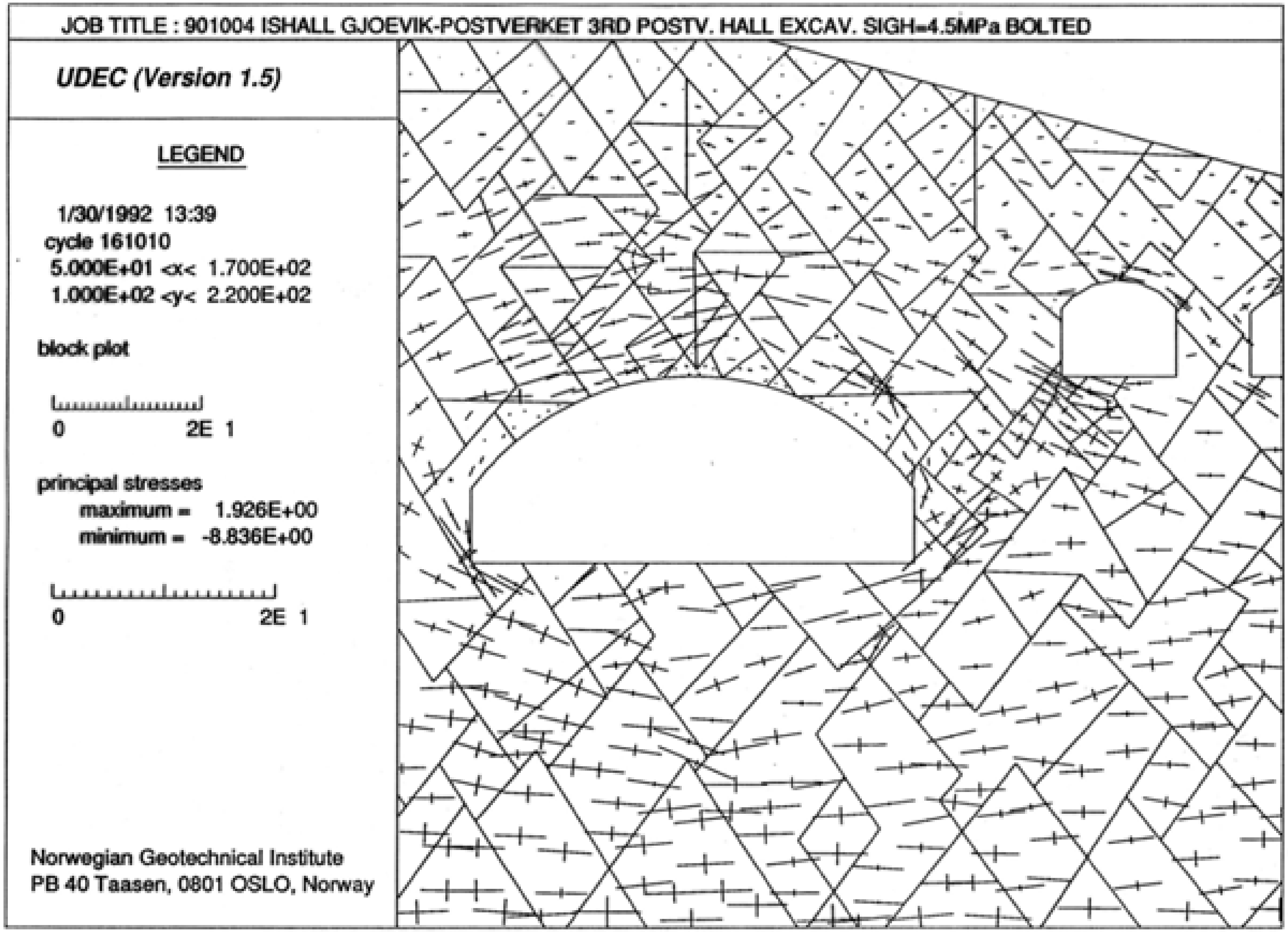

Business Advertising/Photo: Hans Brox

The final modelled 7 to 9 mm (***downwards directed***) deformations matched the subsequently measured MPBX results almost perfectly.

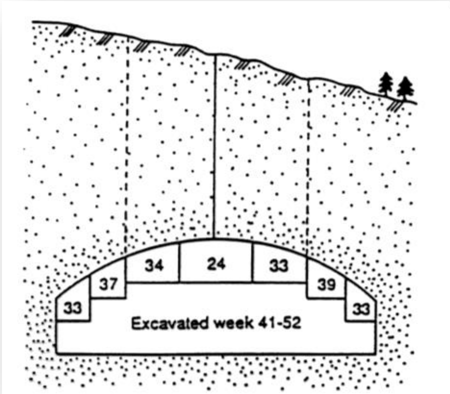
(UDEC-BB modelling by Chryssanthakis, NGI)

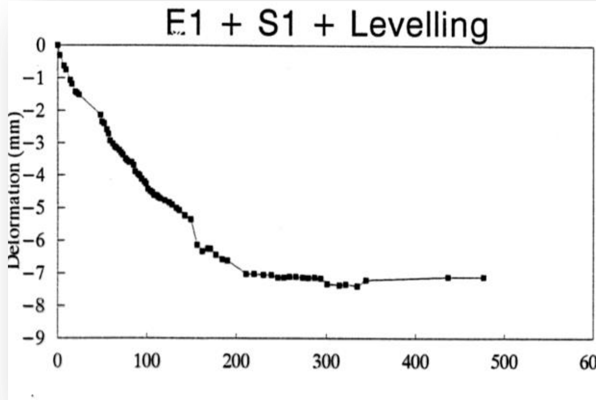


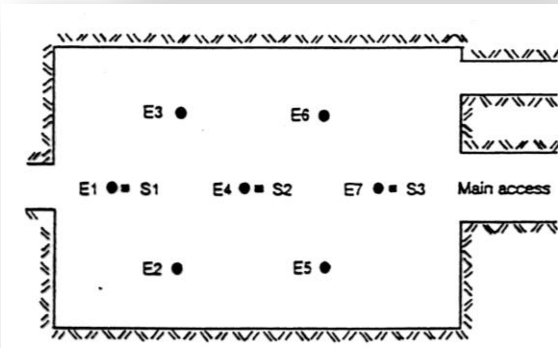
Stress arching calculations …..were affected by the subsequent near-by caverns for the Post Service

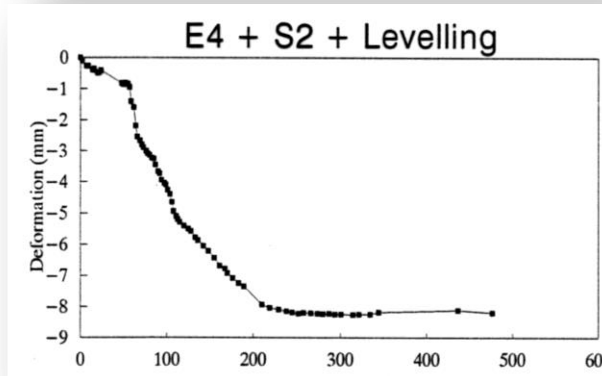


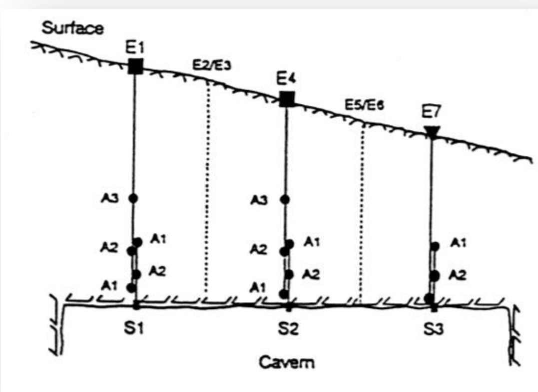
DEFORMATION RECORDS FROM MPBX AND LEVELLING

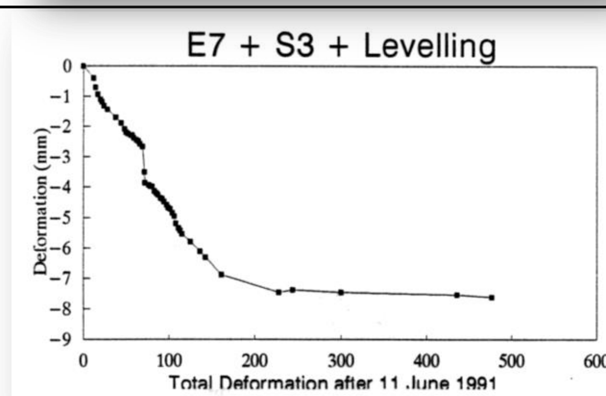












Δ = 7 to 8 mm was typical.

Construction period: **week 24 to week 50**, following arrival of access tunnels (top and bottom).

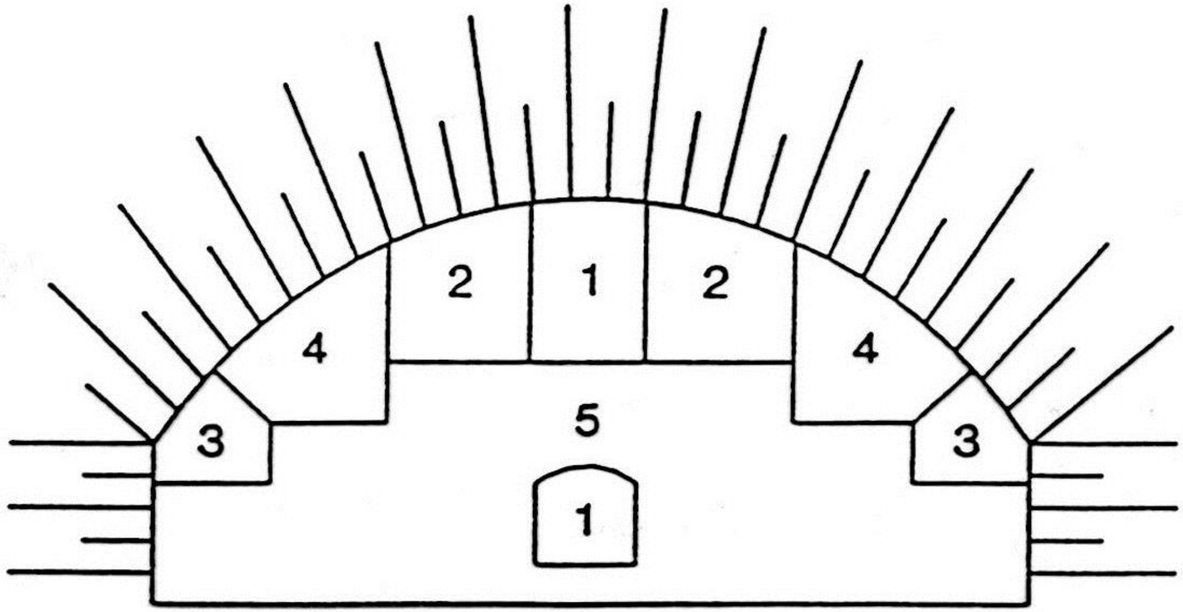
B x H x L
= 62 x 24 x 90
= 140,000 m3

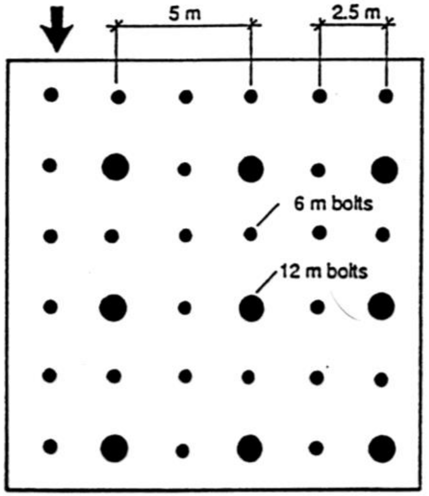
SUPPORT/ REINFORCEMENT

S(fr) 100mm + B c/c 2.5 m + A c/c 5.0 m

(mean S (fr) thickness from numerous control borings = 98 mm)

(bolts and twin-strand anchors:
L = 6 m *bolts*, and L = 12 m *anchors*)





CONCLUSIONS

1. SINGLE-SHELL NMT or DOUBLE-SHELL NATM?
2. BOTH ARE VALID METHODS BUT THERE ARE SIGNIFICANT COST and TIME DIFFERENCES
3. Q SEEMS TO BENEFIT FROM THE ‘LOG’ SCALE
4. NUMERICAL MODELLING WITH (GSI) c and ϕ NEEDS REVISED PHILOSOPHY…degrade/mobilize
5. CASE RECORDS TEACH MANY LESSONS!
6. IS THERE TOO MUCH OPTIMISM ABOUT ROCK CONDITIONS GENERALLY?


THE END
9 MAI 2009