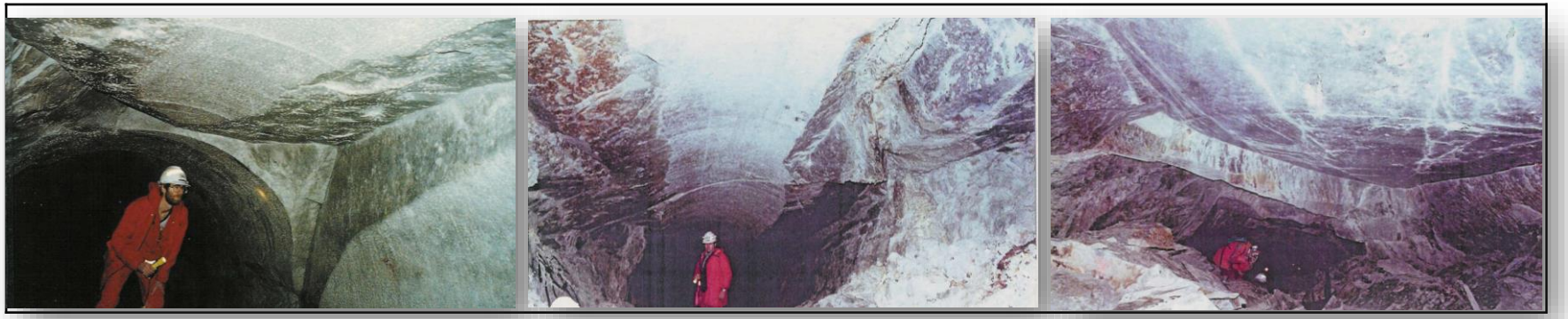
FROM EMPIRICISM, THROUGH THEORY, TO PROBLEM SOLVING IN ROCK ENGINEERING

Nick Barton

6th Leopold Müller Lecture,

Beijing ISRM Congress, 2011



Beaumont TBM Tunnel, 1880 : wedge-failure, stress-failure, tidal influence. Three TBM photos separated by 150 m.

***"The deformation resistance of the material bridges takes effect at much smaller deformations than the joint friction: this joint friction makes partly up for lost strength"*. (Müller, 1966).**

Our rock masses 45 years later continue to rely on joint friction, despite today's 'downloadable' ***continuum wishful thinking – and the <u>assumed relevance</u> of :***

$c + \sigma'_n \tan \varphi$ *(or its non-linear versions)*

Why not 'c <u>then</u> σ'_n tan φ' (degrade cohesion, mobilize friction) (as in parts of Canada, Sweden, India, Norway)?

JINPING I (305m DAM)

CHALLENGES IN AN OVER-STEEPENED CANYON
(solved by designers CHIDI)

Just up this valley is a *much smaller feature also* deserving our rock mechanics attention!

WHY THE 'OVER-BREAK' / POTENTIAL INSTABILITY?

Because of adverse Jn, Jr, Ja (JRC, JCS, ϕr), Jw, SRF

……and dip/dip direction/gravity/density……

HOW DID THESE PARAMETERS MATERIALISE, and HOW HAVE THEY BEEN USED? SOME TOPICS to be DISCUSSED:

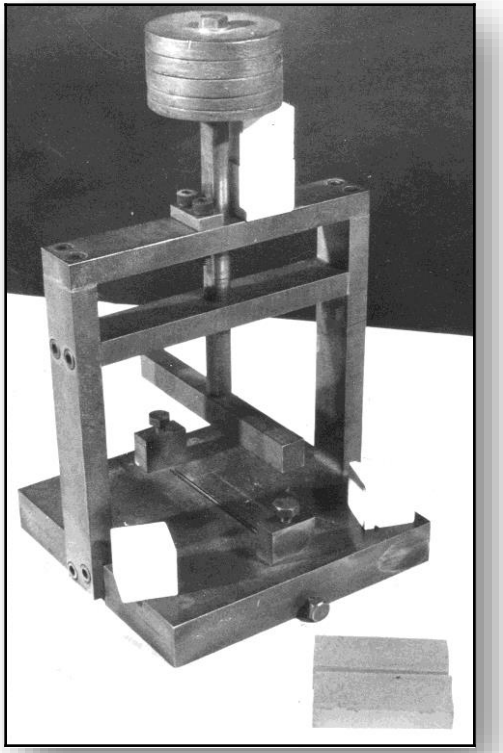
1. SINGLE TENSION FRACTURES → 40,000 BLOCKS
2. SHEAR STRENGTH CRITERIA ('20'→JRC, UCS→JCS)
3. INFLUENCE OF BLOCK SIZE
4. SLOPES, CAVERNS AND BOREHOLES WITHMODELS
5. UDEC-BB VALIDATION FROM A SPECIAL CAVERN
6. CHALLENGING QUESTION FROM A CLIENT→ Q
7. Q, Q_{TBM}, BB APPLICATIONS, SELECTED CASES
8. Q, S(mr), S(fr), B(c/c), ✂ Q_c (=Q xσ_c /100)→CC, FC (c, ϕ)
9. DEGRADE CC, MOBILIZE FC, <u>not c + σ_n (tan ϕ)</u>
10. OTHER TOPICS – SEE PAPER!

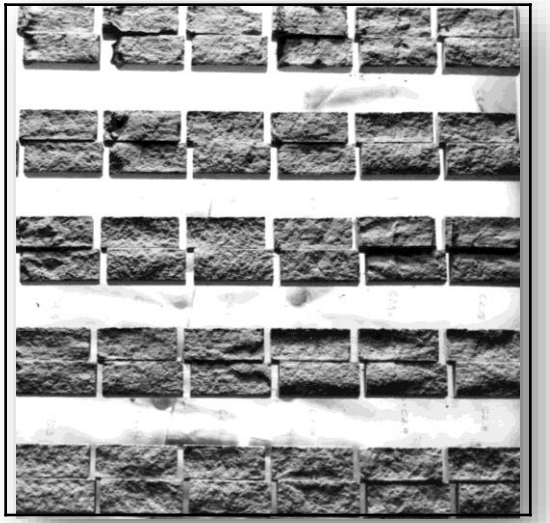
AN IDEALIZED, DISCONTINUOUS START, IN 1966

(Imperial College)

Includes early L-B,L

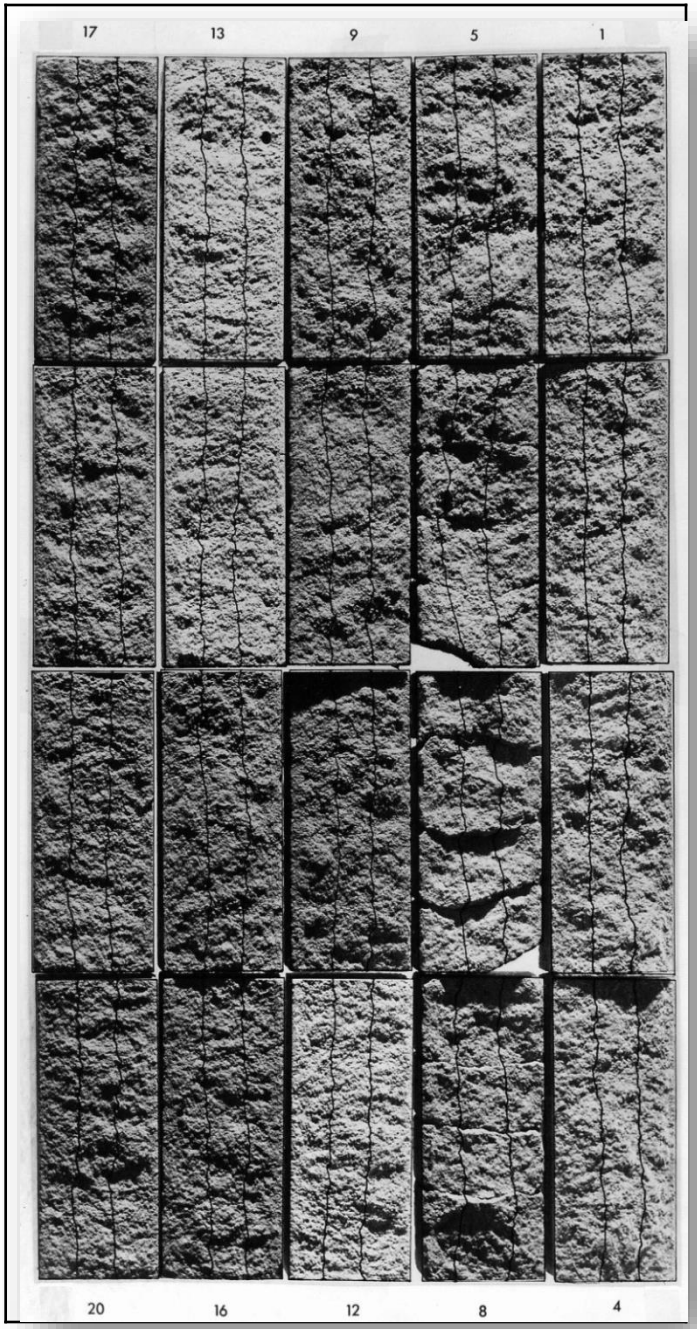
(lucky-breaks, lessons)

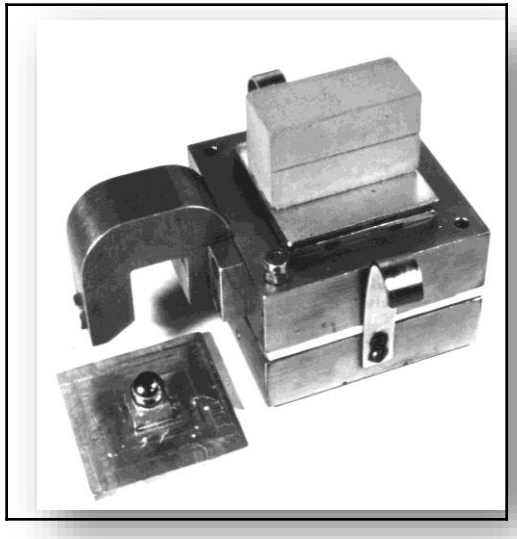


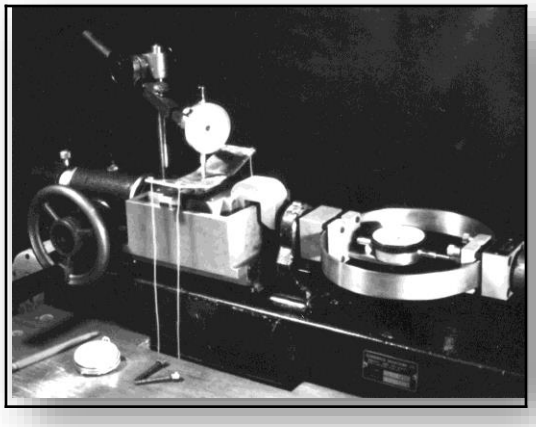


DST on 200 artificial tension fractures in a variety of brittle model materials

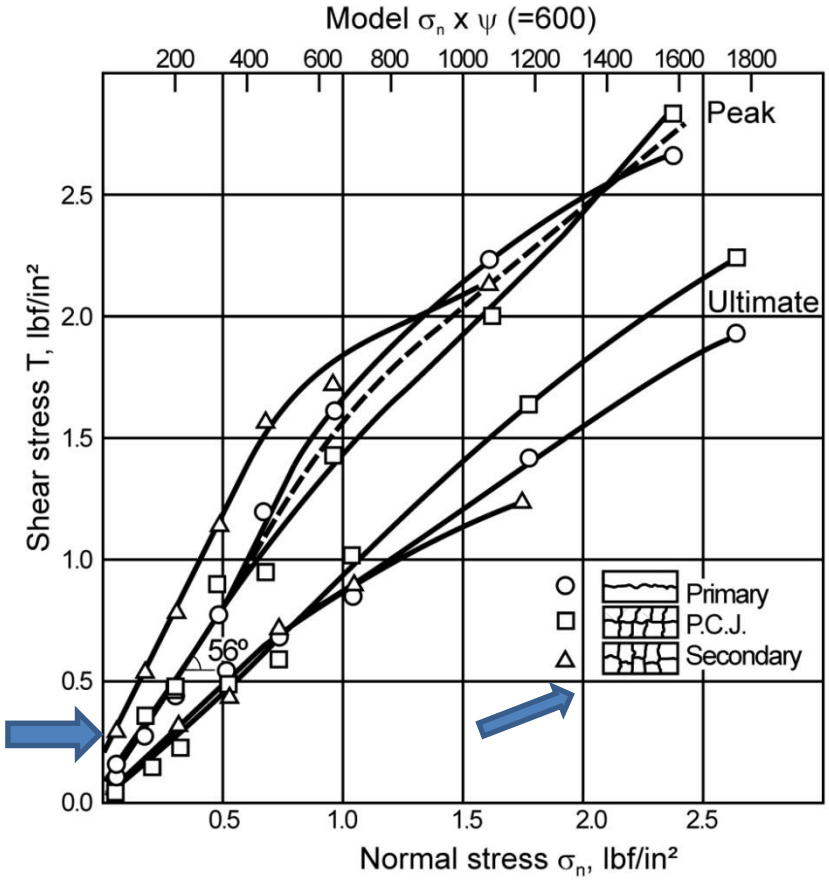
(Barton, 1971)



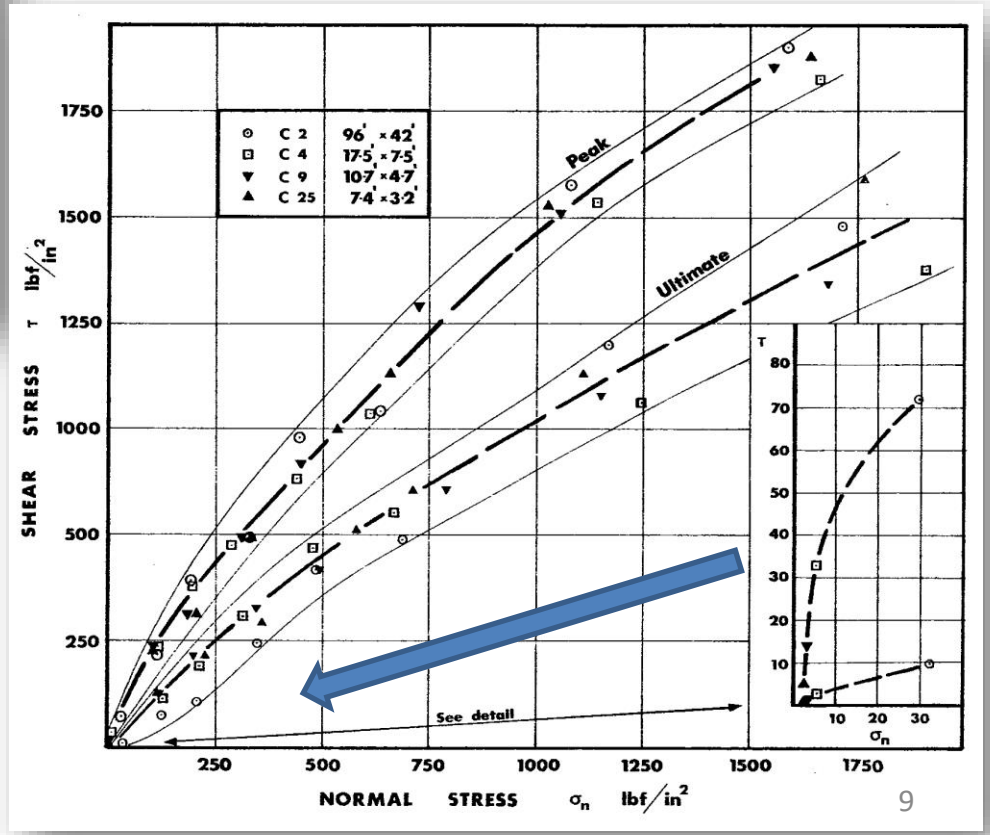




LESSON #1

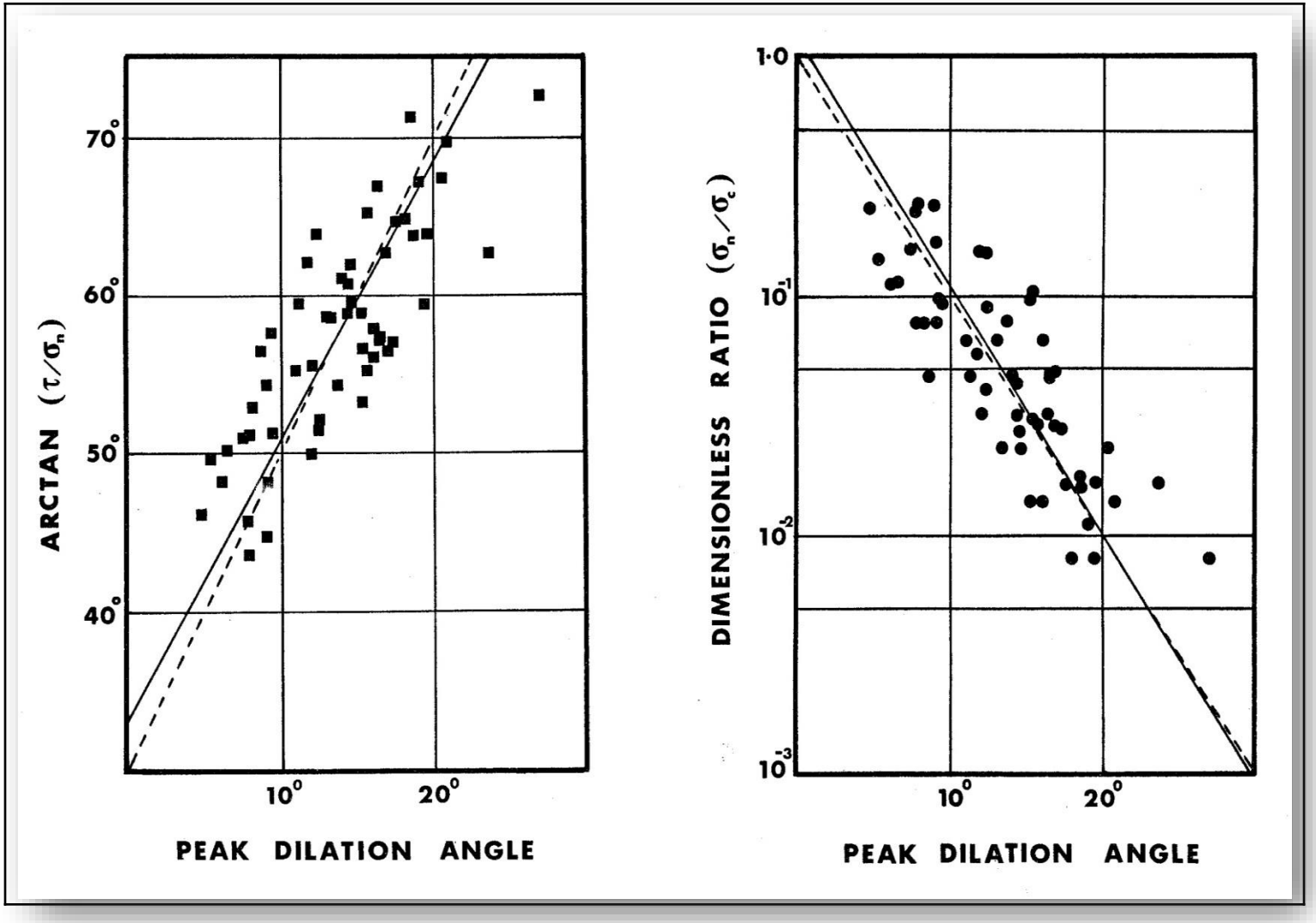


LACK OF ACTUAL COHESION UNLESS STEPPED ("secondary") FRACTURES ARE TESTED

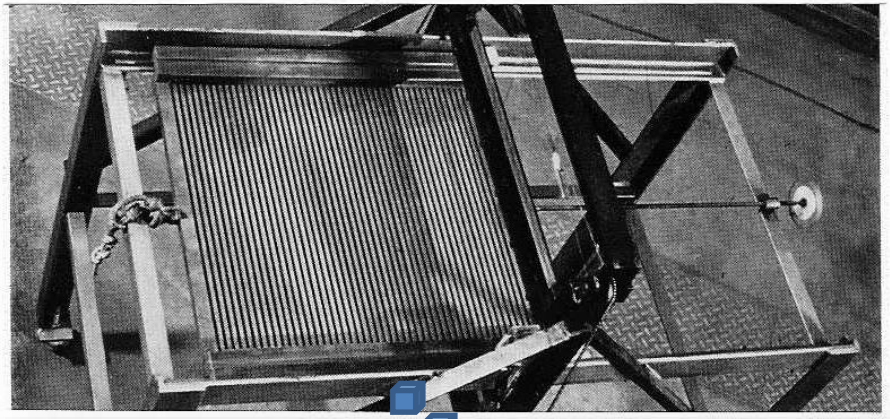


Lucky break #1 (------ = no decimal places)

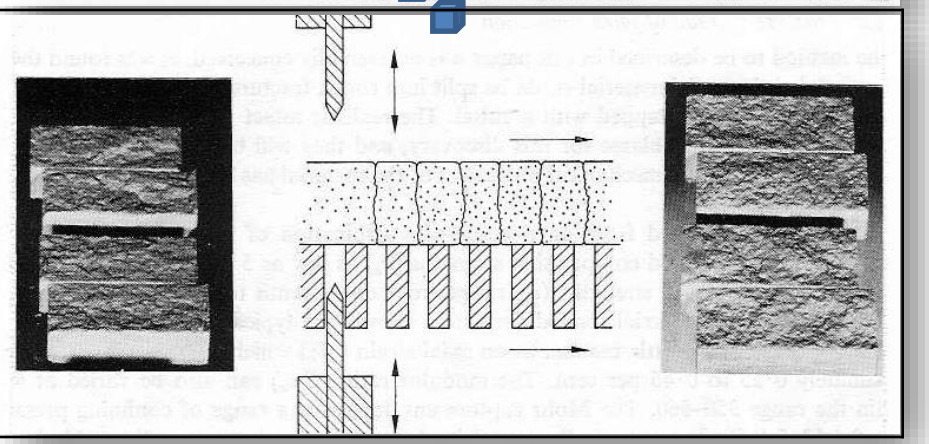
$$\tau = \sigma_n \,.\, \tan\left[\, 20.\, \log\left(UCS/\sigma_n \right) + 30^o \,\right]$$



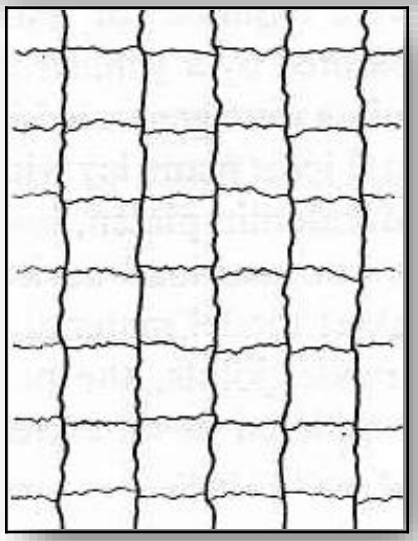
2D JOINTED "ROCK-MASS"

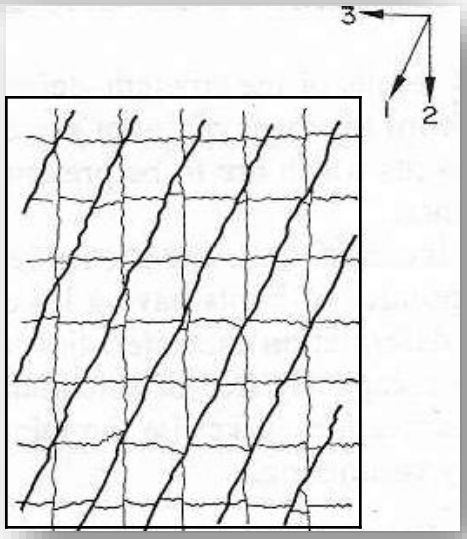


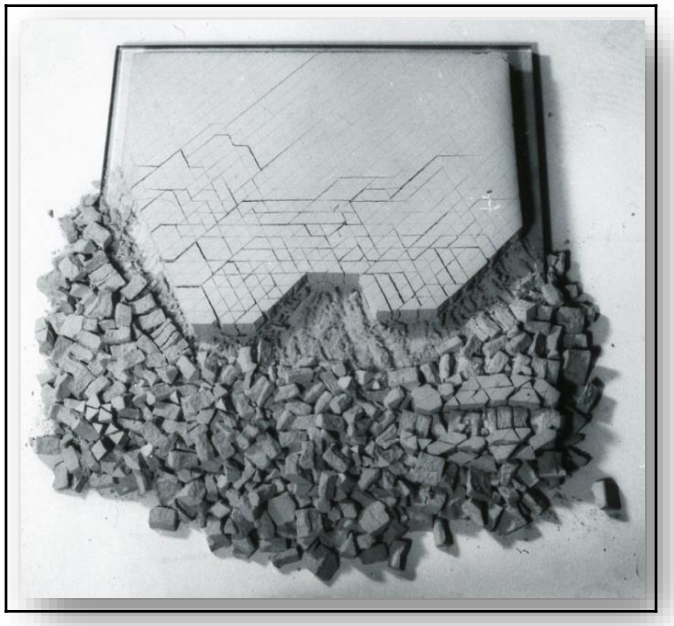
Tension-fracture models for *rock slopes* (at Imperial College) 1968-1969.

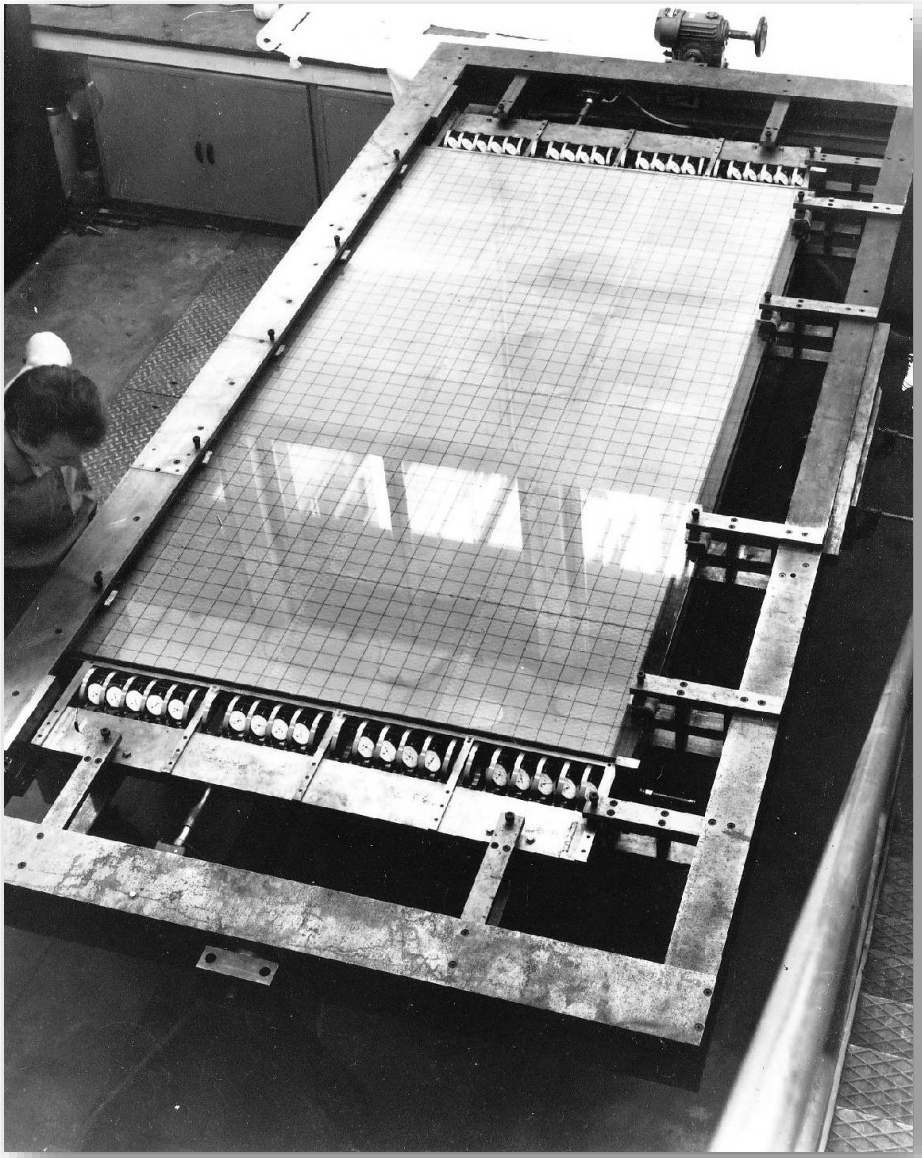


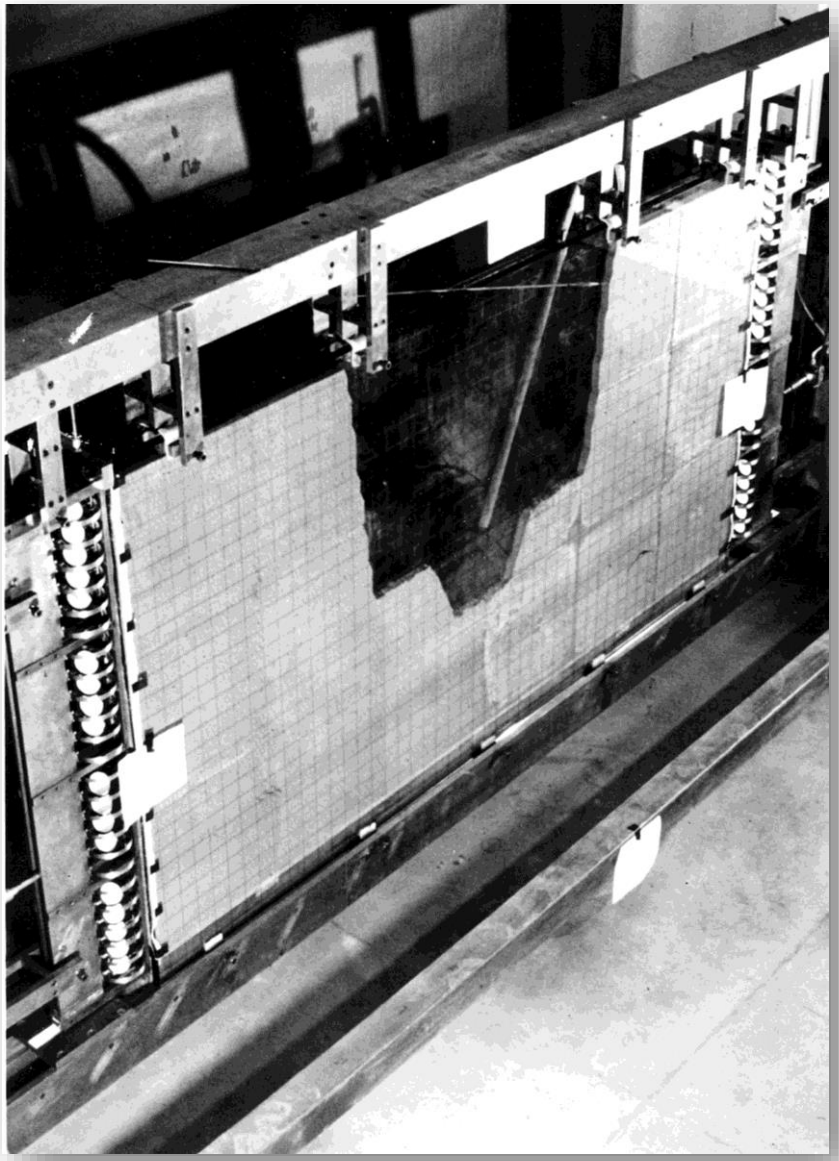
'Nuclear power' *cavern* investigations (at NGI) 1977-1978.







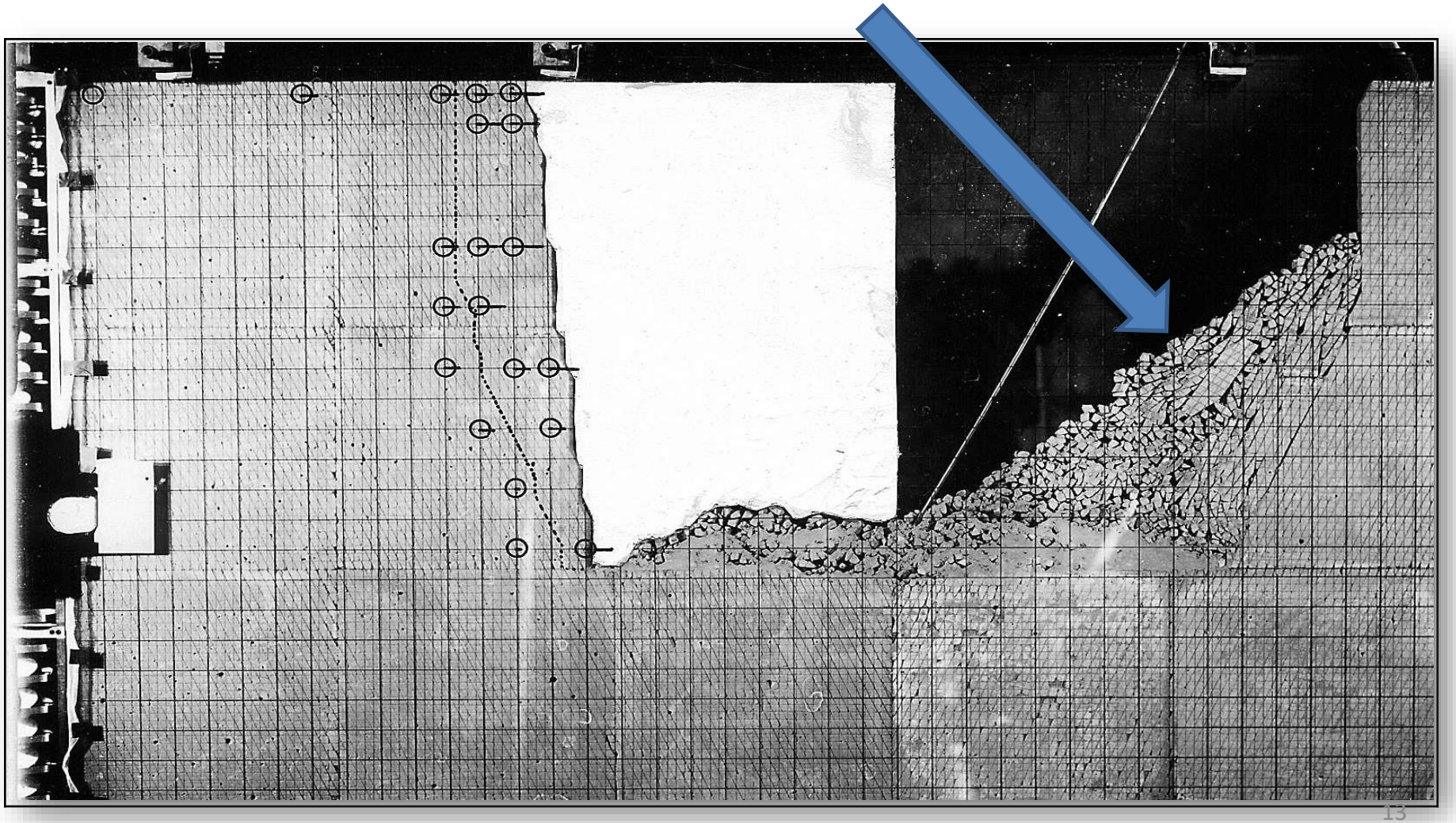


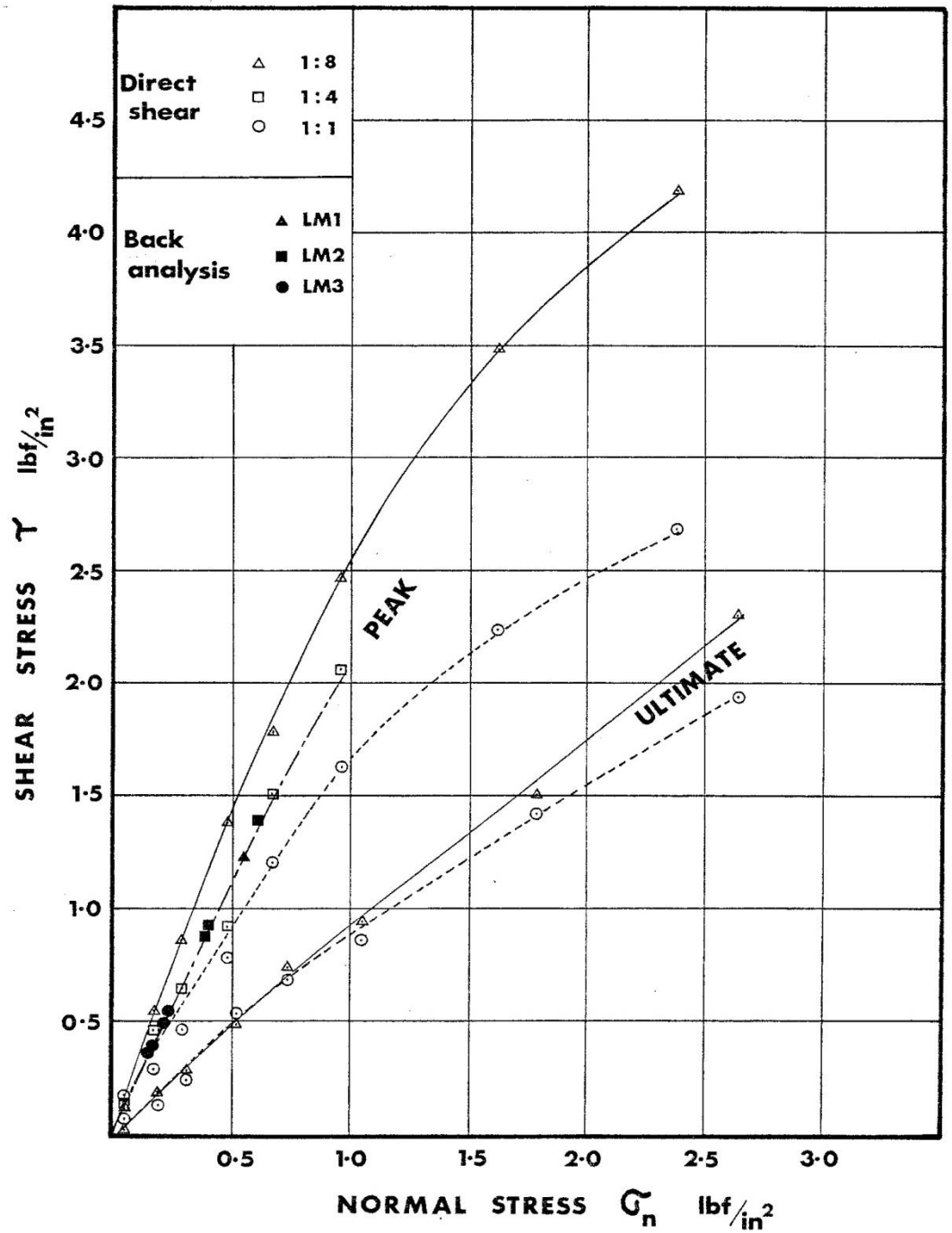


A Ph.D. study of *(unexpectedly stable)* 'steep excavated rock slopes' with **40,000 blocks, and three 'joint' sets**.

See some large blocks with ***'over-closed'*** fractures.

Note also that the ***normal stress*** on the critical (dipping) joints mostly ***reduces*** as a result of excavating steep slopes.





OVER-CLOSED (O.C.) SHEAR TESTS

LESSON #2

EXPLAINED WHY THESE STEEP SLOPES WERE *SO* STABLE.

DST WITH SHEARING *under normal stress of 1 x* $\boldsymbol{\sigma_n}$ *, after prior normal load application* of:

1 x $\boldsymbol{\sigma_n}$ (=conventional)

4 x $\boldsymbol{\sigma_n}$ **(= O.C) ,** 8 x $\boldsymbol{\sigma_n}$ **(= O.C)**

Why are we not doing ***O.C.*** DST in Rock Mechanics?

Rock bridges of course.

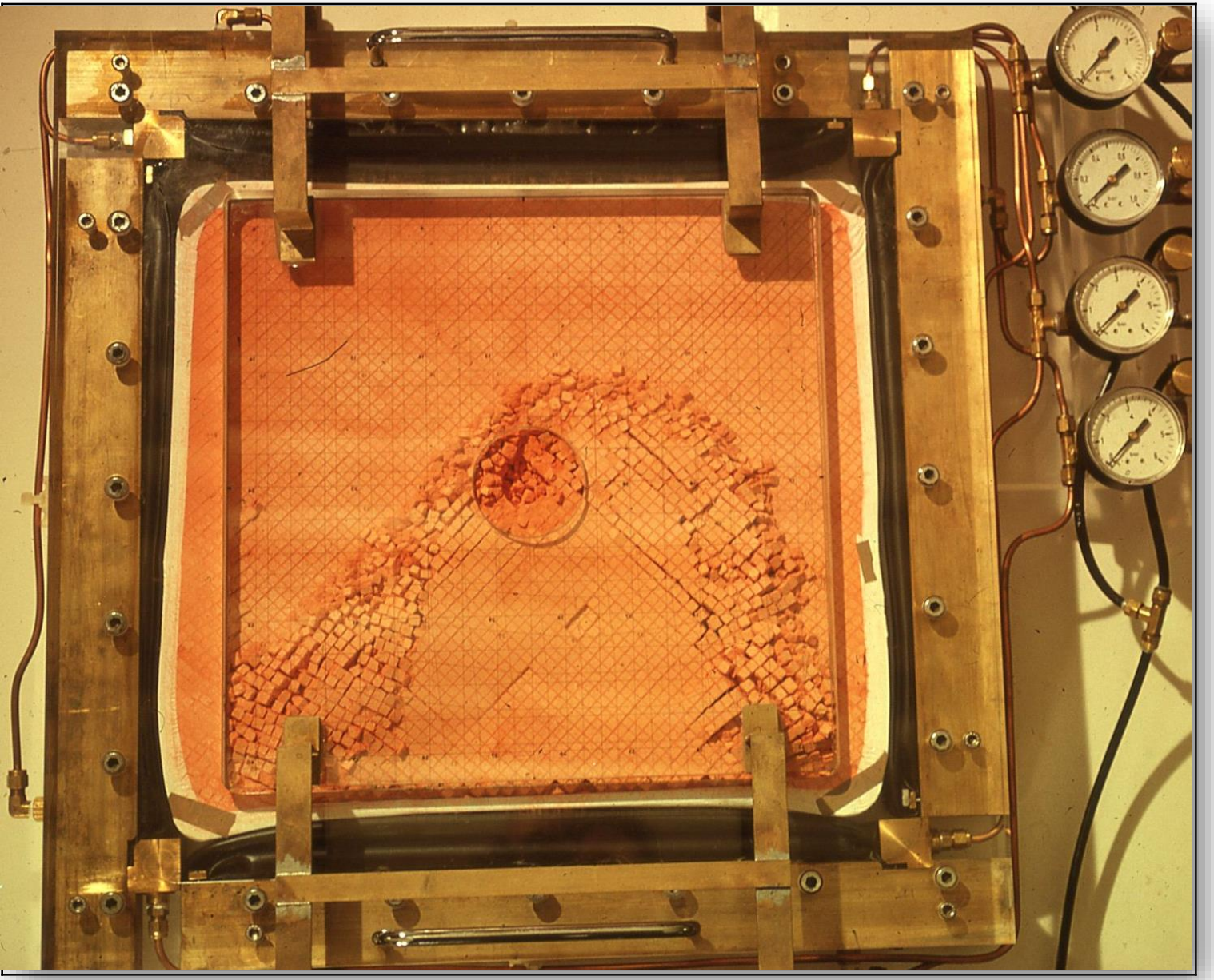
Perhaps also *over-closed* where jointed ???

Opinion/Hypothesis

The multiple influences of block size are not yet taken care of very well in our numerical modelling?

Physical models that follow here:

- Artificial, but some useful lessons
- They are physical, not conceptual



BIAXIAL LOADING

Scale-effect investigations

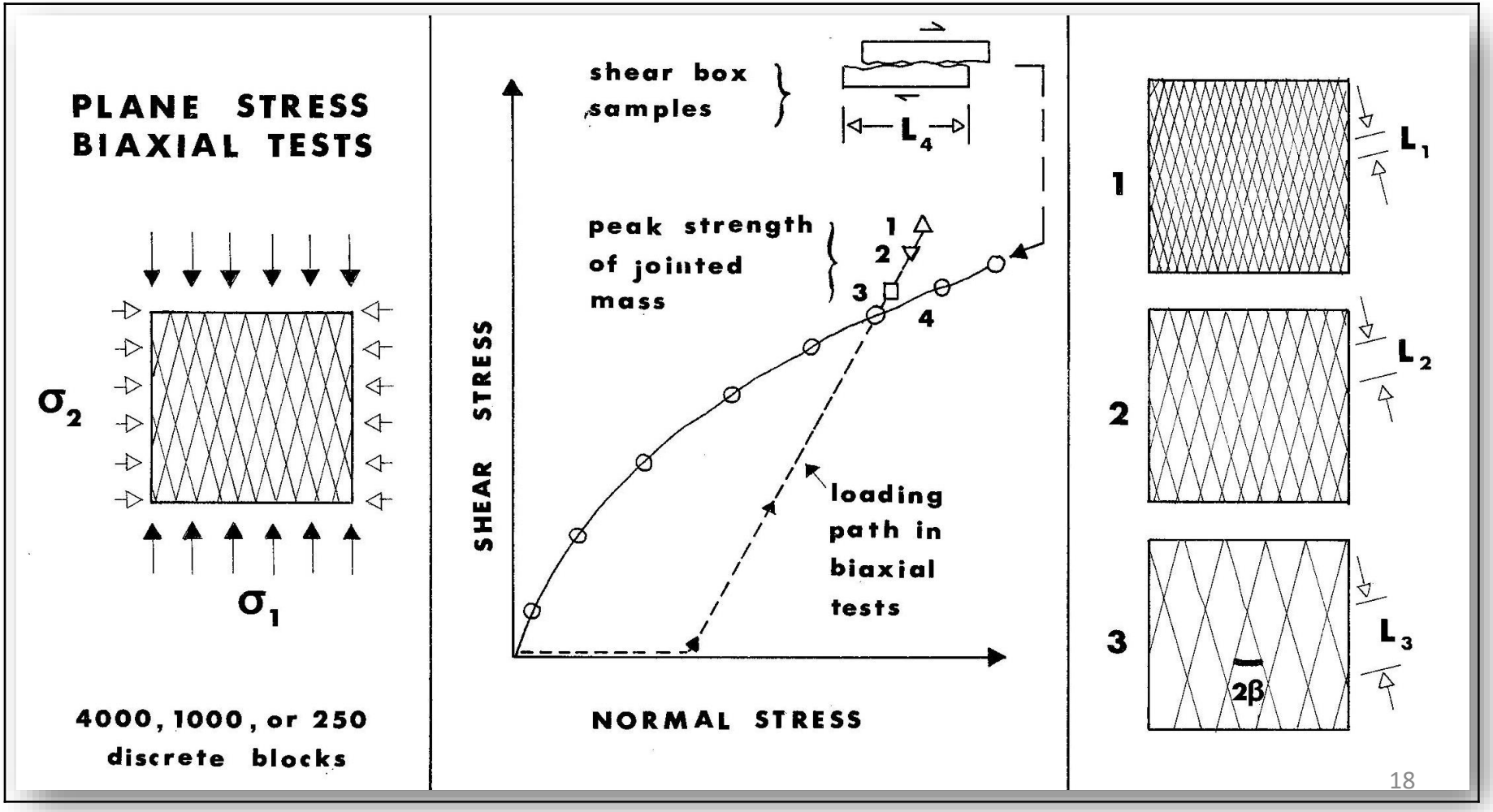
250, 1000, or 4000 blocks.

Lesson #3

"Always" got rotational failures with small blocks

Lesson #4 2D 'ROCKMASS' : SHORTEST BLOCK-SIDES: HIGHEST STRENGTH .....(DST samples = L 4)

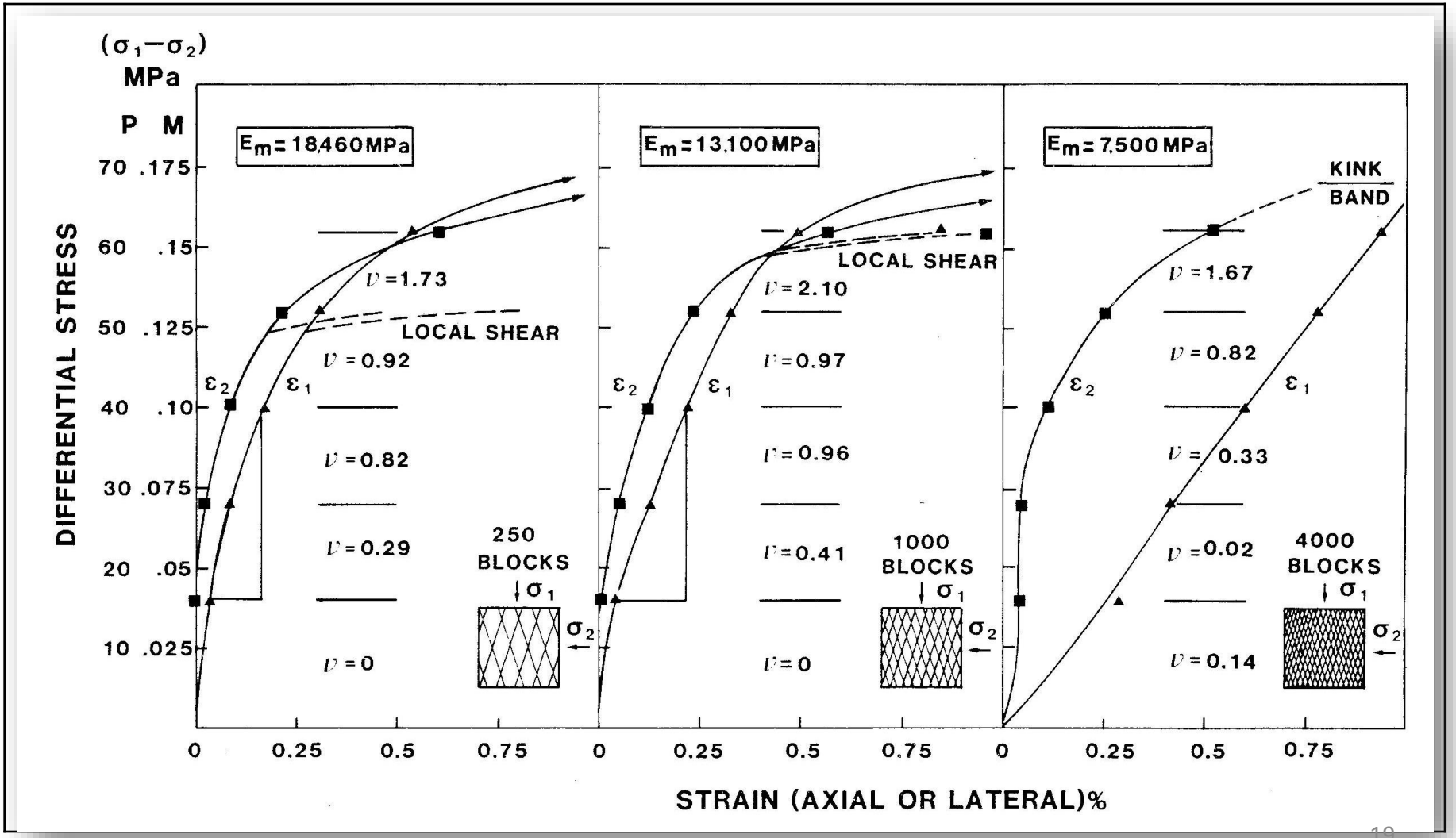
(L4 > L3 > L2 > L1) (Barton and Hansteen, 1979)



‘STRESS-STRAIN’ BEHAVIOUR : 250, 1000 or 4000 BLOCKS.

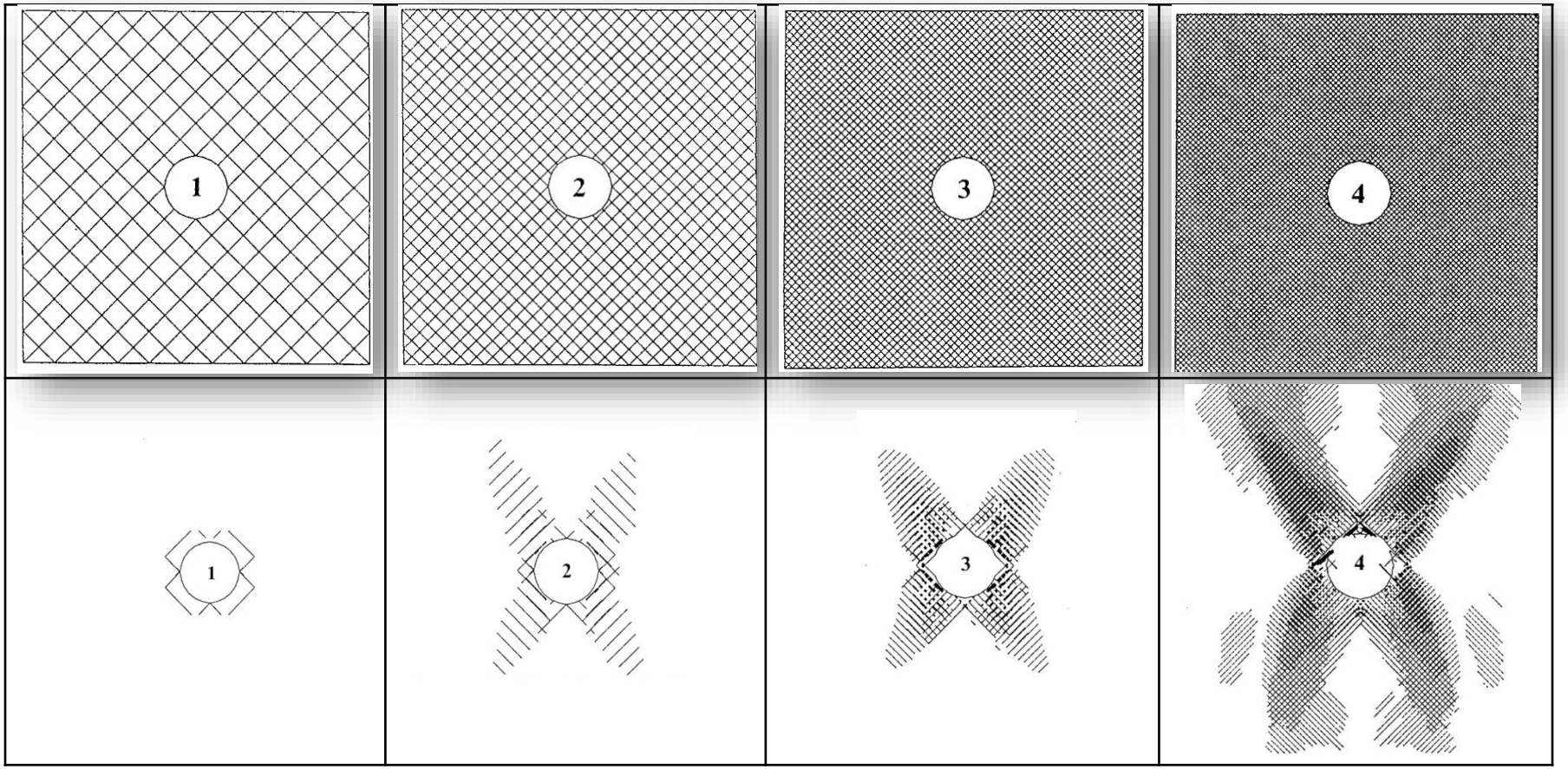
Lesson #5 *LINEARITY* OF 4000 blocks model

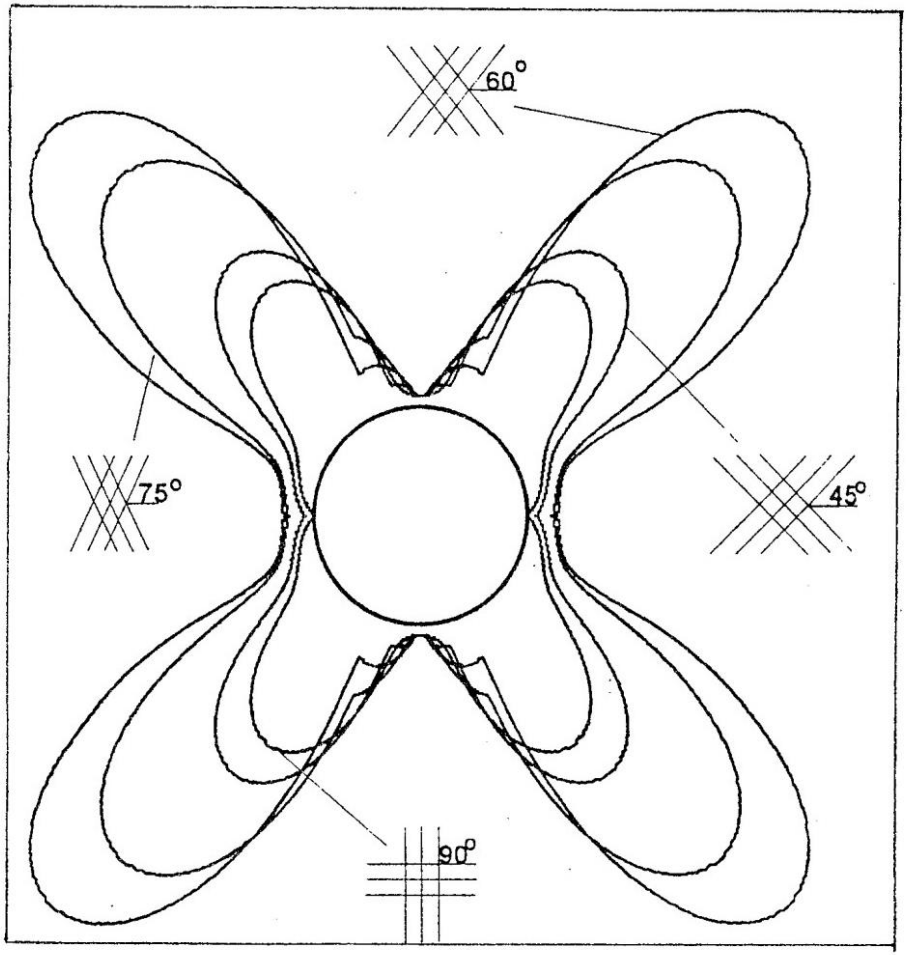
(partially-linear? : 1000 blocks) ***Lesson #6*****.....*****high “Poisson” ratio)**



SUCCESSIVE HALVING OF THE BLOCK SIZE – HAS DRAMATIC ROTATIONAL (degree-of-freedom) EFFECTS, ALSO WITH *UDEC-MC* .

Shen, B. & Barton, N. 1997. The disturbed zone around tunnels in jointed rock masses.





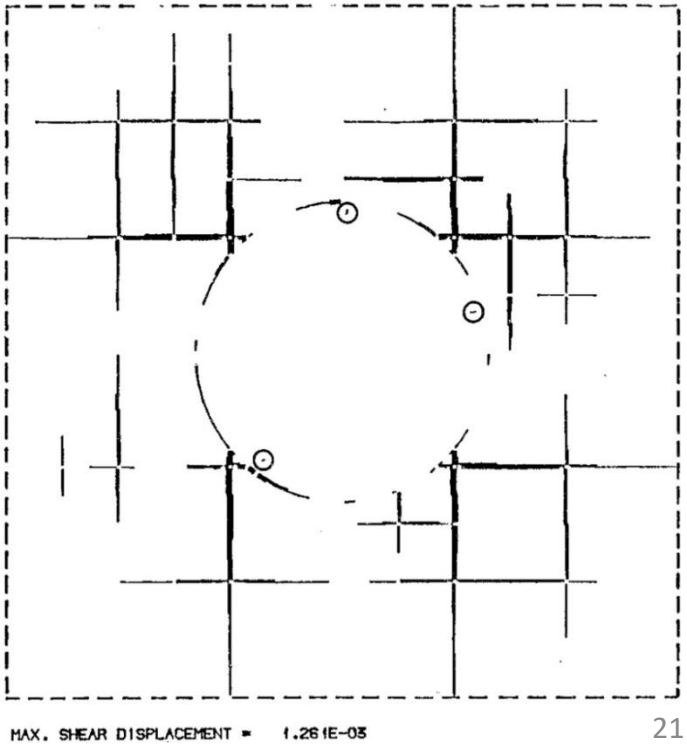
ANALYTICAL MOHR-COULOMB JOINT SHEAR-LOCATIONS

(Shen and Barton, 1997).

The first UDEC-BB model !

Sector-related shear despite isotropic stress.

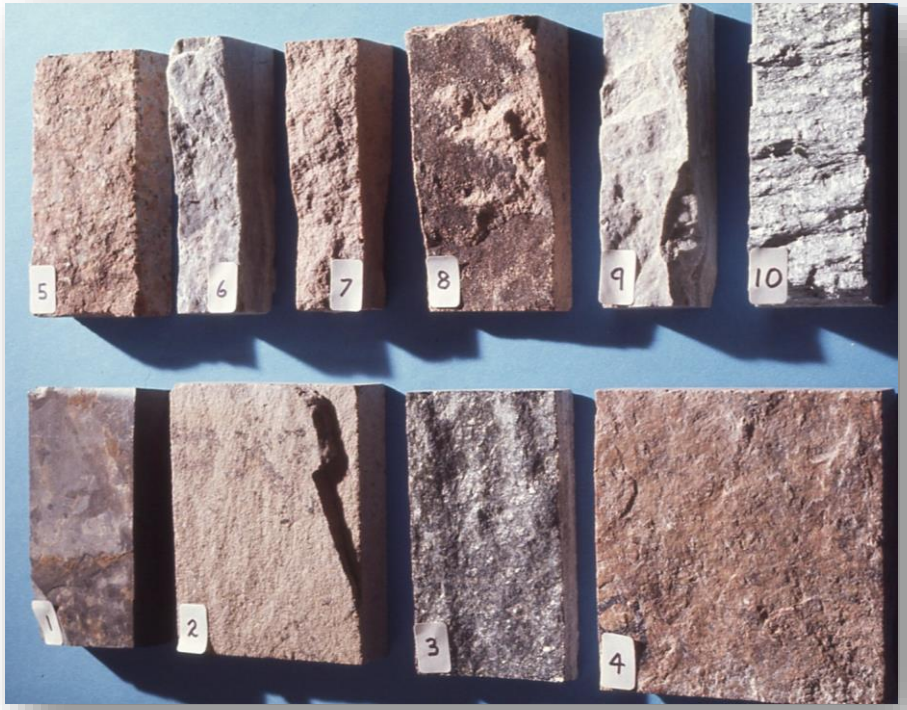
(Mark Christiansson, Itasca/NGI, 1985)

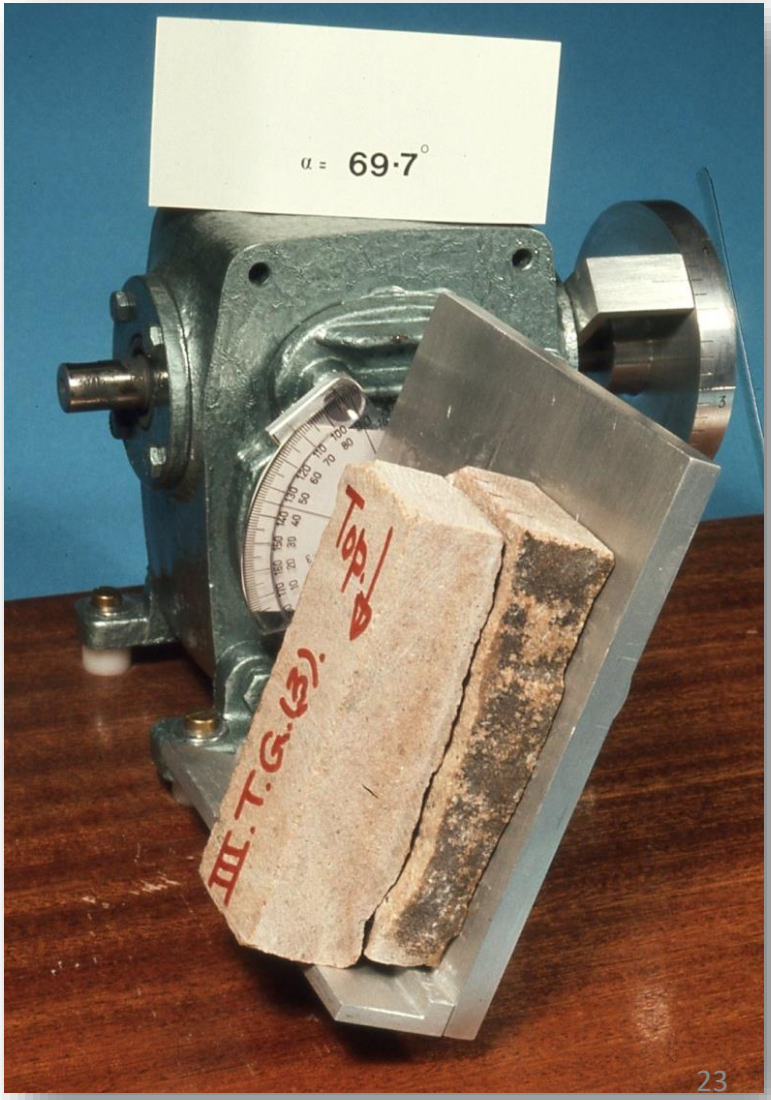


RESEARCH WITH ROCK JOINTS PROVIDES A SOLUTION TO NON-LINEAR SHEAR STRENGTH

130 joint samples. Roughness measurement and ***tilt test***.

(Barton and Choubey, 1977)



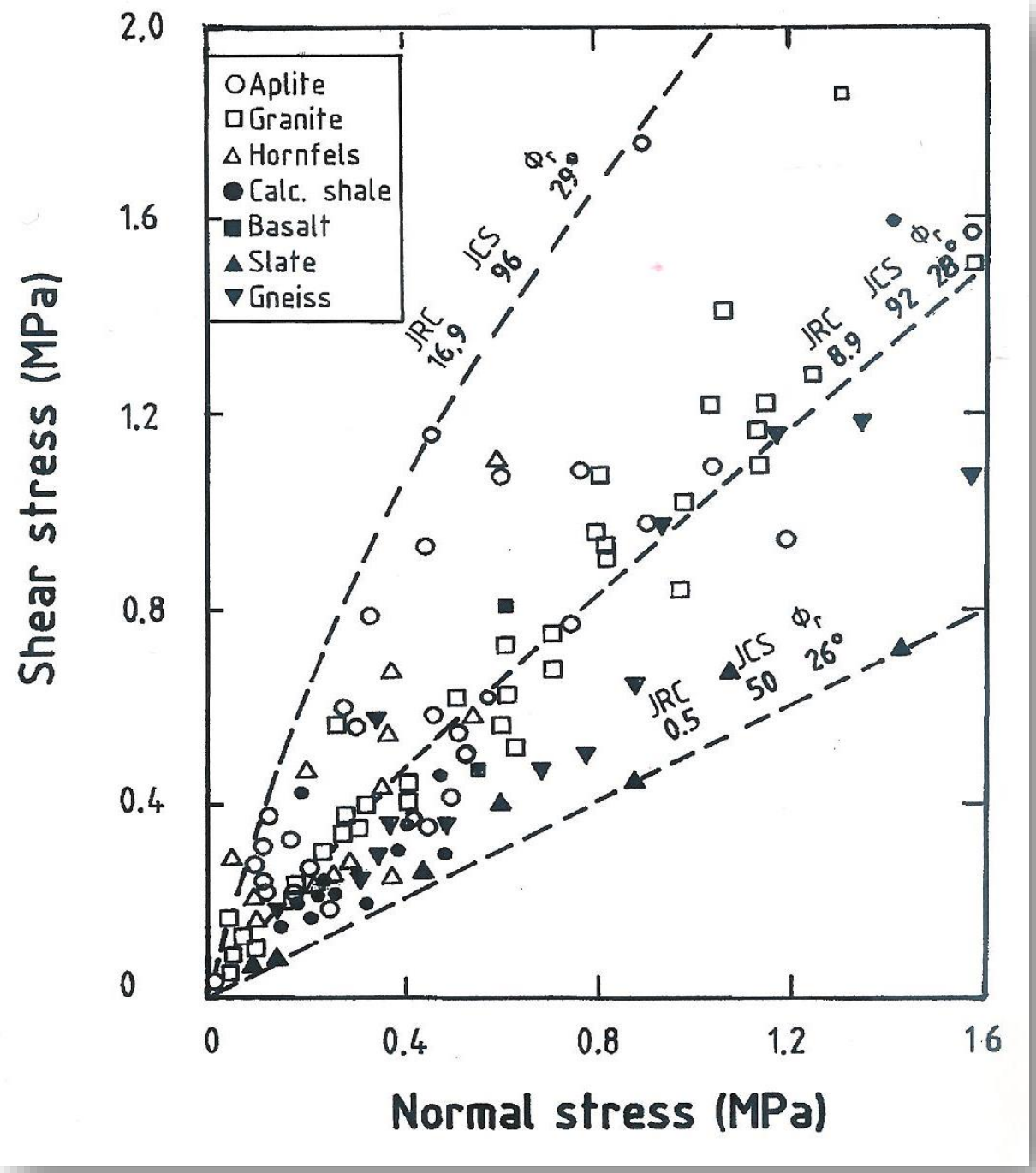


TILT TEST 'THEORY'

σ_n (MPa)	arctan (τ/σ_n)	arctan $(\tau/\sigma_n)°$	
		JRC=5	JRC=10
100 †	$>\phi_r$	>30°	>30°
10	ϕ_r + JRC	35°	40°
1	ϕ_r + 2 JRC	40°	50°
0.1	ϕ_r + 3 JRC	45°	60°
0.01 ‡	ϕ_r + 4 JRC	50°	70°
0.001 ‡	ϕ_r + 5 JRC	55°	80°

$$JRC = \frac{\alpha^\circ - \phi_r^\circ}{\log\left[\frac{JCS}{\sigma_n}\right]}$$

(Barton and Bandis, 1990)



130 rock-joint samples

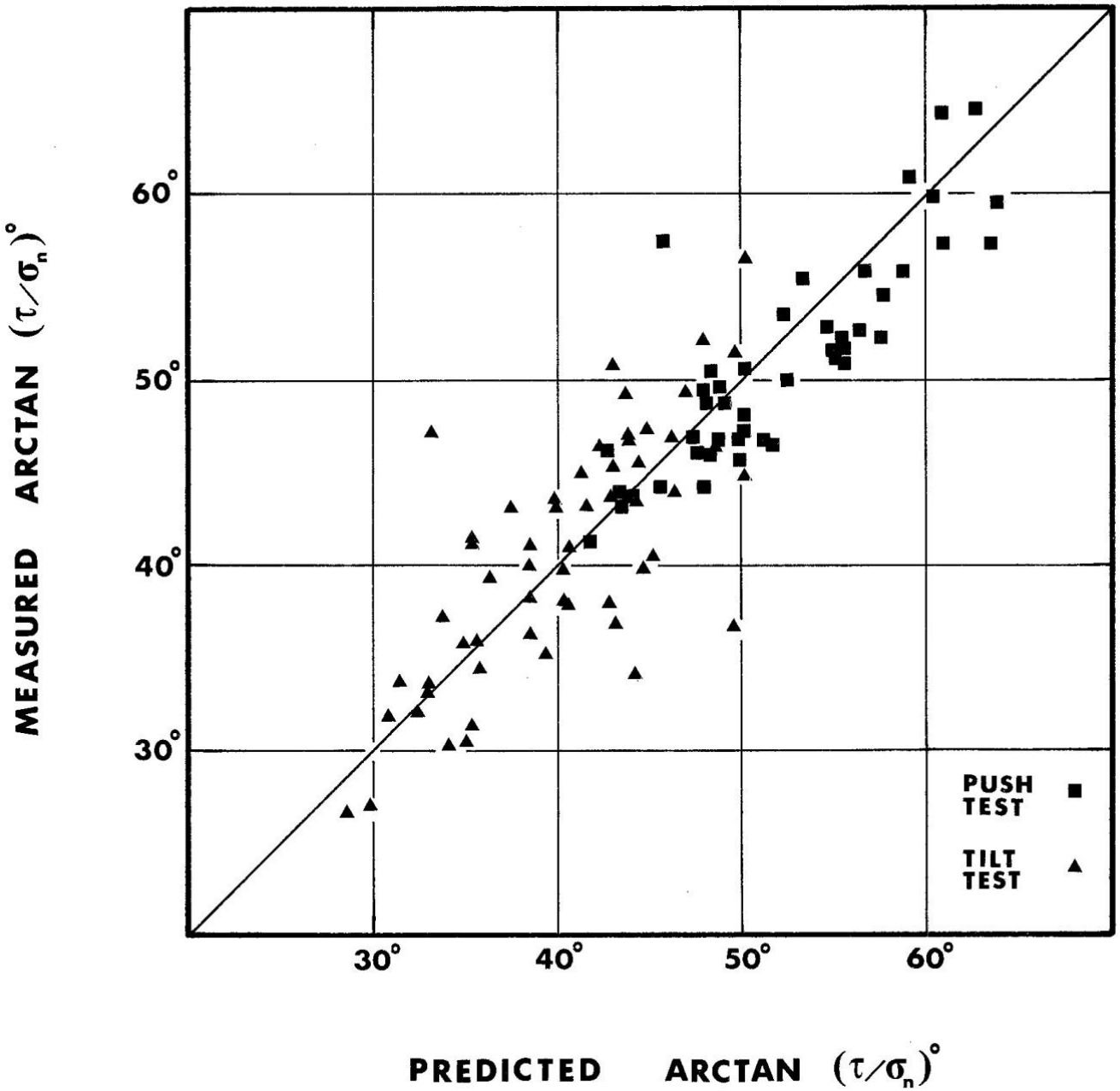
(Barton and Choubey 1977)

Three curved **peak shear strength** envelopes shown:

1. Maximum strength with JRC = 16.9

2. Mean parameters
JRC=8.9,
JCS=92MPa
ϕ_r=28º

3. Minimum strength with $\varphi_r = 26^o$



COMPARING DST-MEASURED PEAK SHEAR STRENGTH *with tilt and push test predictions*

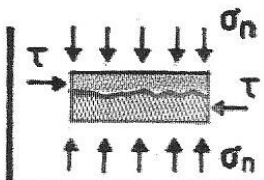
σ_n range involved :

0.001 MPa (tilt)
1.0 MPa (DST)

= x 1000

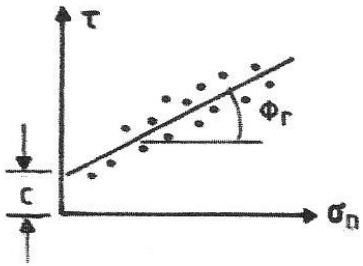
THREE SHEAR STRENGTH CRITERIA FOR ROCK JOINTS

Typical test method



(1) $\tau = \sigma_n \tan \Phi_r + c$

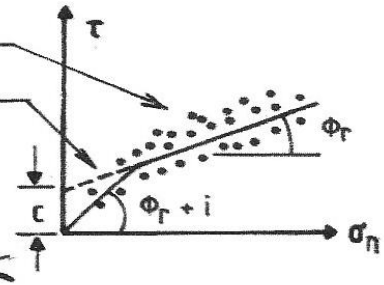
Mohr – Coulomb



(2) $\tau = \sigma_n \tan \Phi_r + c$

$\tau = \sigma_n \tan (\Phi_r + i)$

Patton

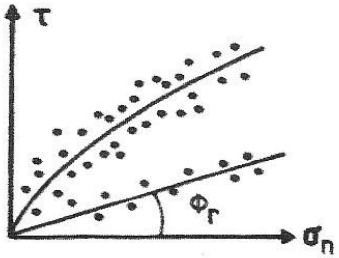


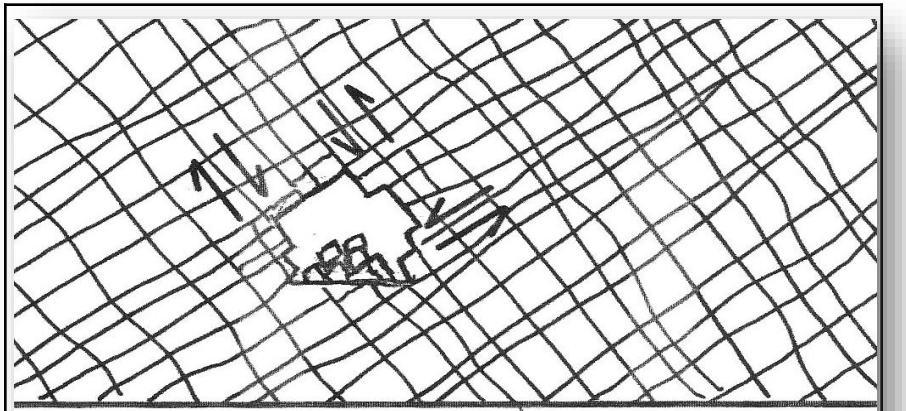
(3) $\tau = \sigma_n \tan \left[JRC \log \left(\frac{JCS}{\sigma_n} \right) + \Phi_r \right]$

JRC = joint roughness coefficient

JCS = joint wall compression strength

Φ_r = residual friction angle





BARTON-BANDIS (scaled)

$$\tau = \sigma_n . \tan\left[JRC_n . \log\left(\frac{JCS_n}{\sigma_n}\right) + \Phi_r\right]$$

Lucky break #3

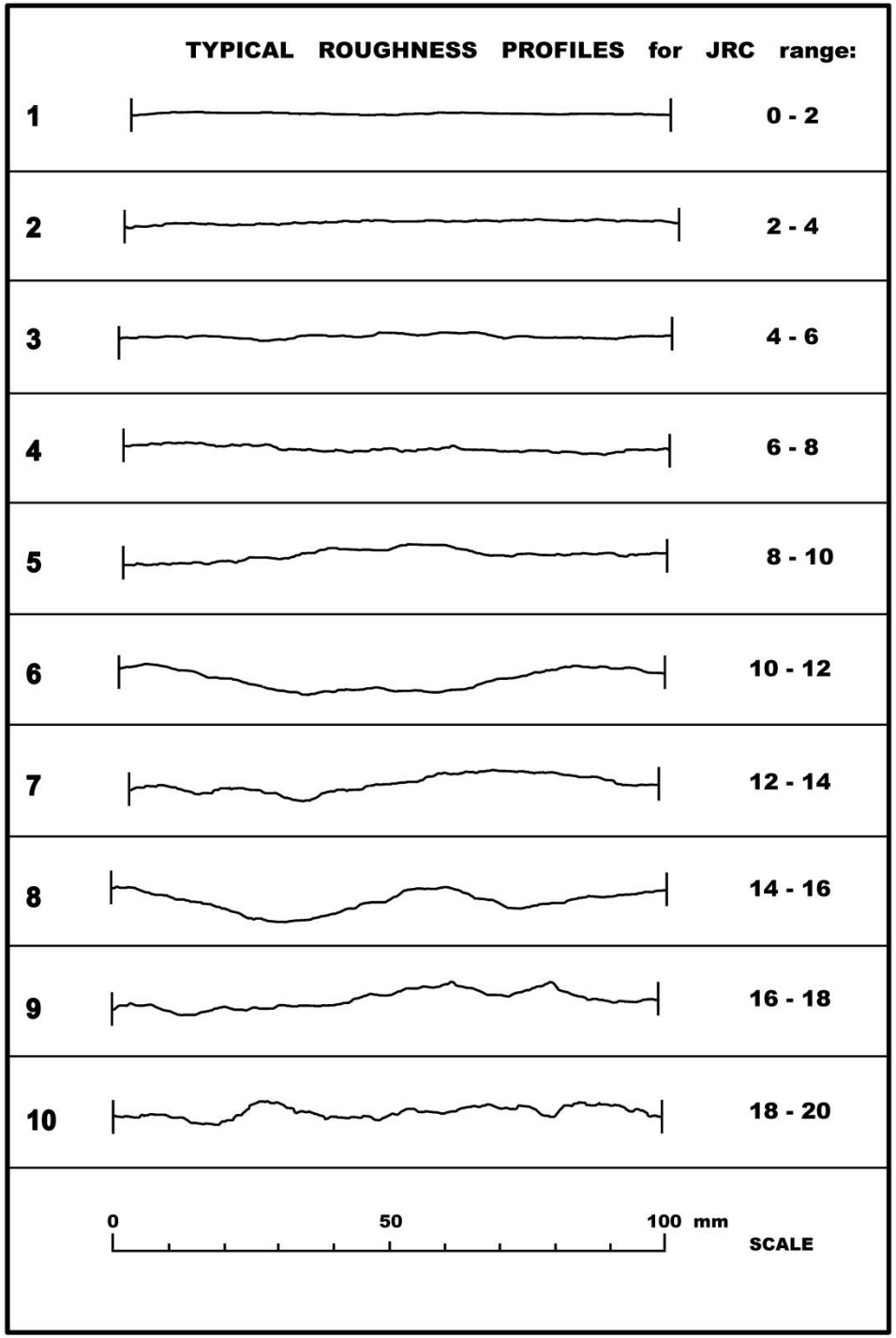
Note: the original tension fracture-based equation (1971) was:

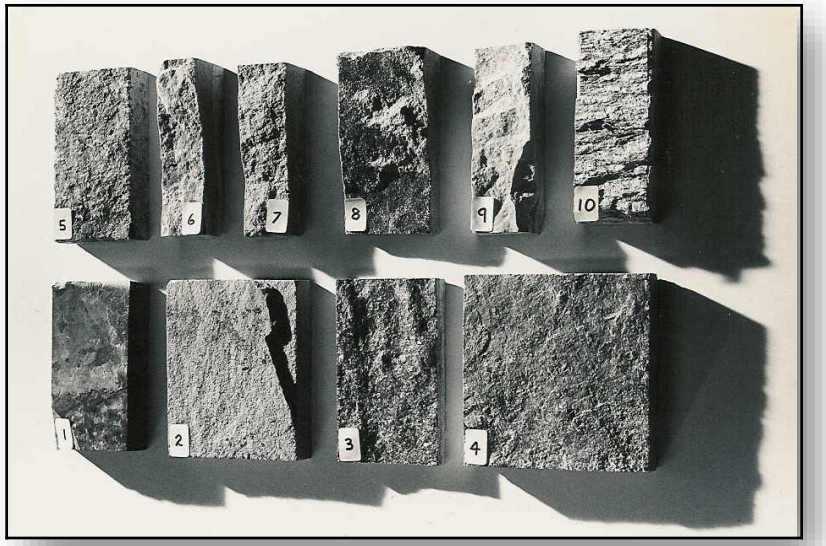
$$\tau = \sigma n . \tan [20. \log(UCS/ \sigma n) + 30^o]$$

JRC JCS φb (now φr)

TO THOSE WHO HAVE PERFORMED PH.D.'s AND ARE SELLING SOFTWARE – PLEASE NOTE IT IS φr since 1977 !!

VISUAL MATCHING OF ROUGHNESS – for JRC HAS OBVIOUS LIMITATIONS (Barton and Choubey, 1977)

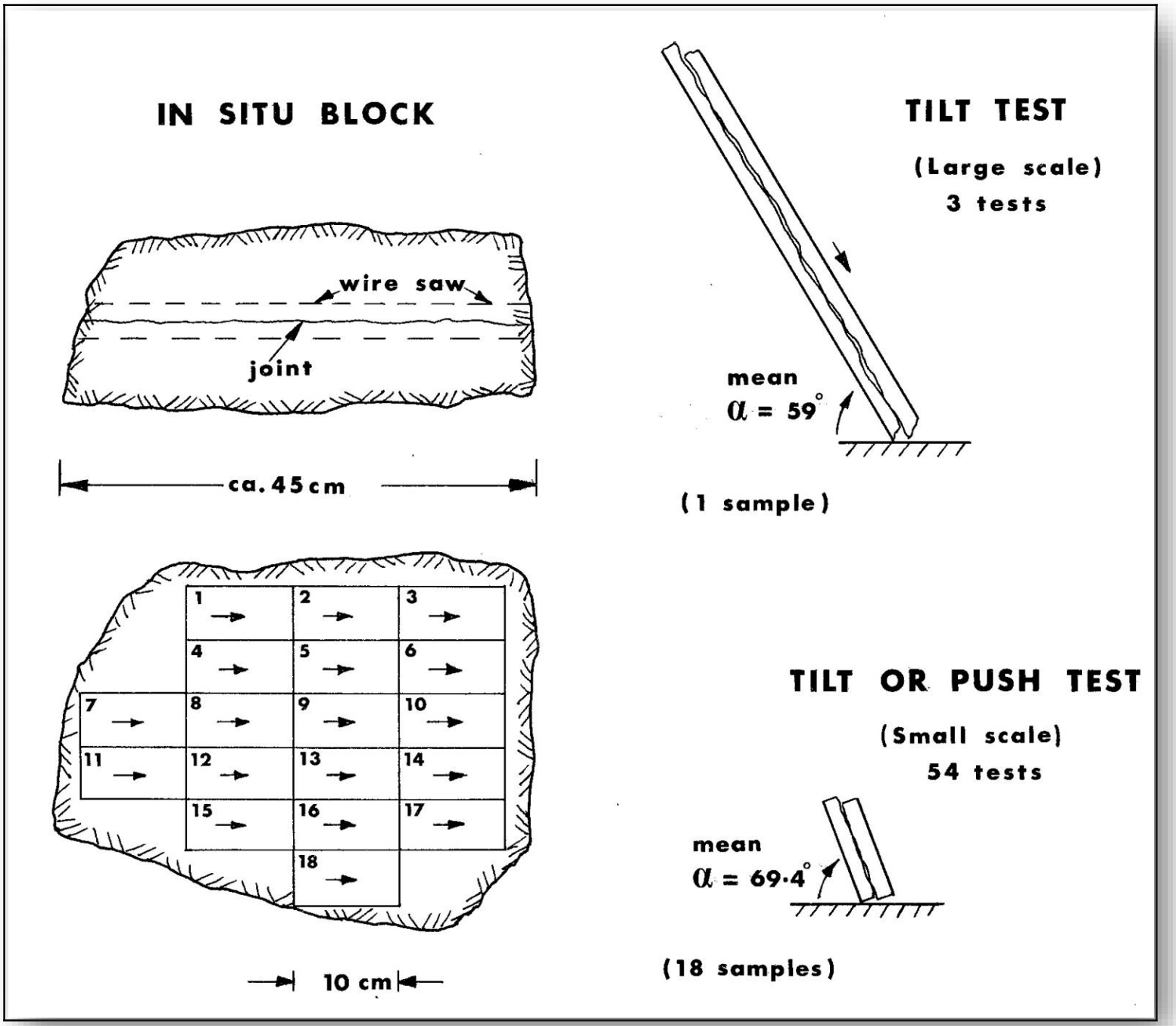




JCS < UCS

JCS > UCS (?)

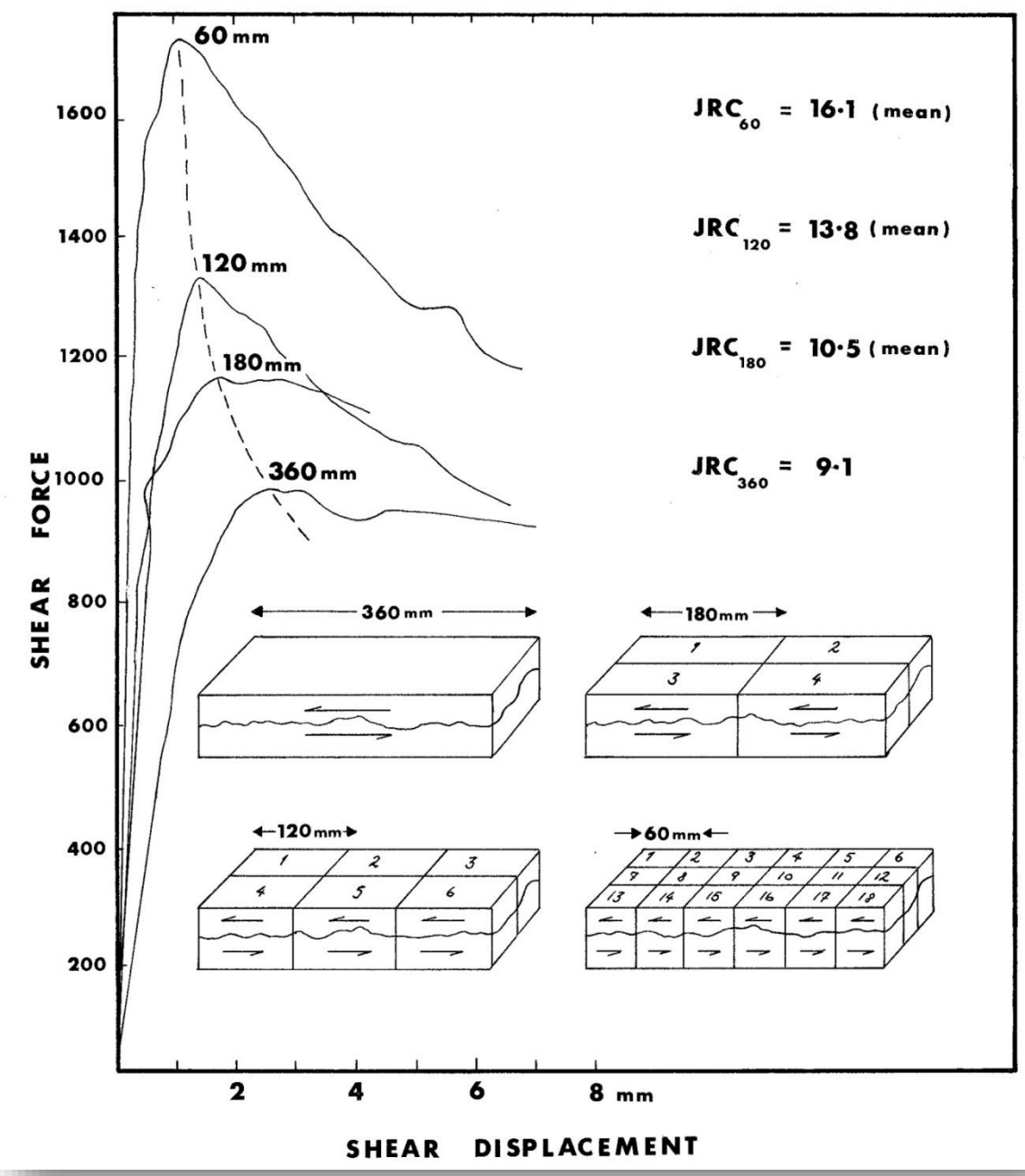
SCALE EFFECTS FOR INDIVIDUAL JOINTS



Tilt tests repeated at different scale - there is *almost no damage*.

Note:
$JRC_1 < JRC_2$

(Barton and Choubey, 1977)

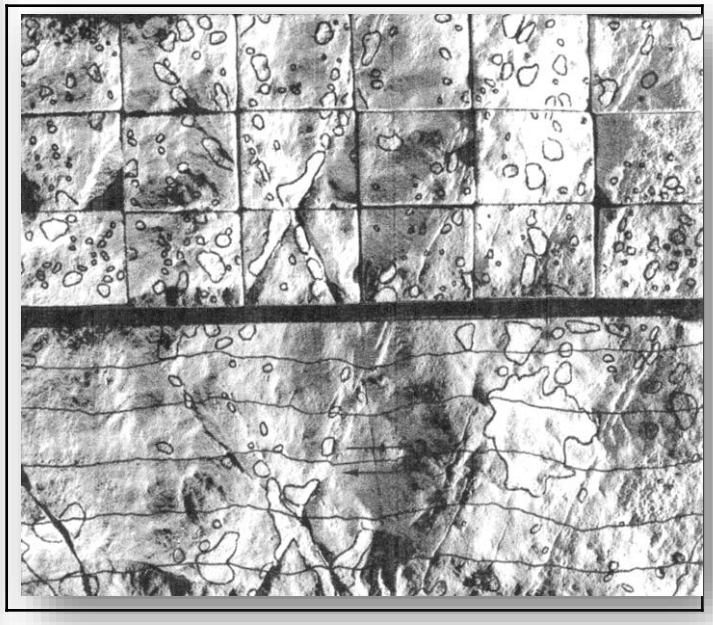


Bandis 1980 Ph.D.

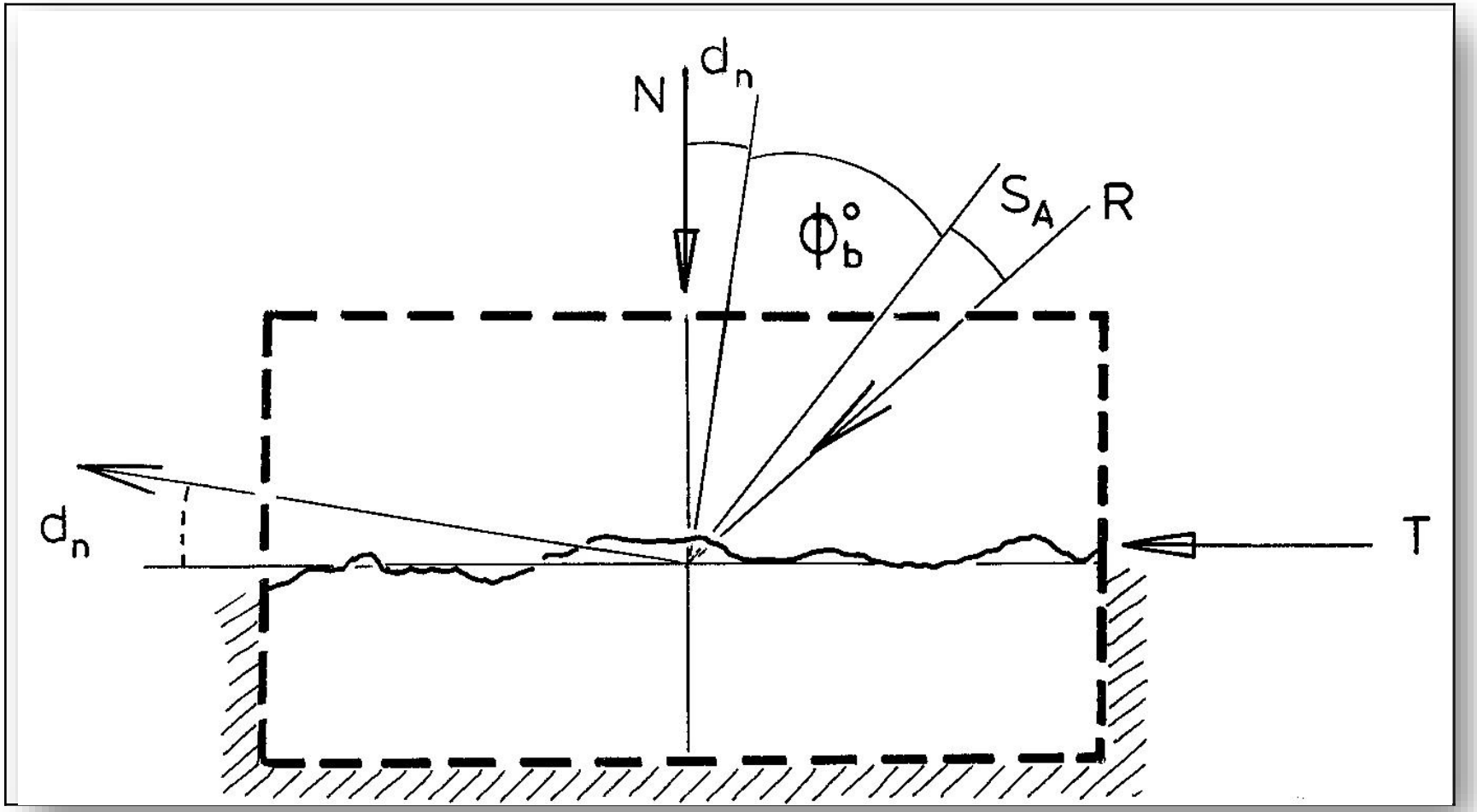
(Lucky break #4!)

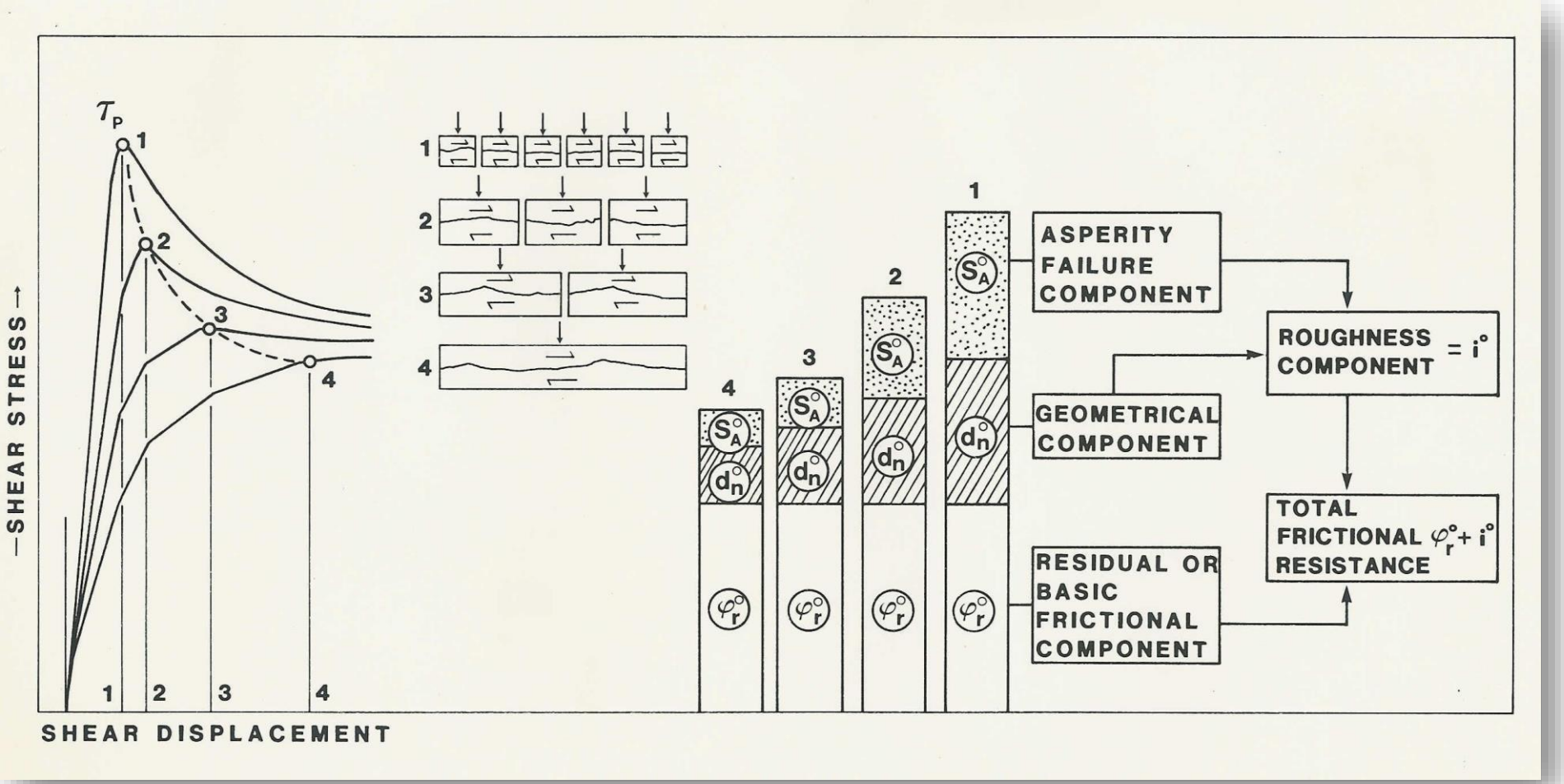
Ahead-of-their-time scale-effect investigations.

One set of many joint replica tests.



The angular components of ***peak shear strength***, with asperity strength (S_A), and peak dilation angle (d_n), each included. (Barton, 1971)



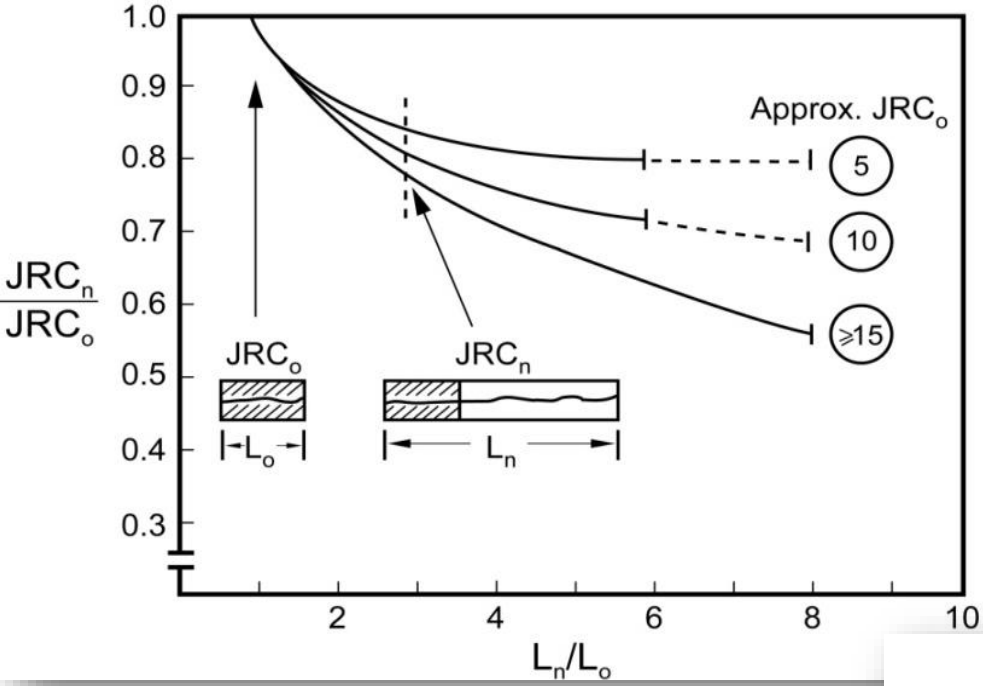


The asperity component S_A (Barton, 1971 and Bandis, 1980) means that ***JRC*** (or φ_r) cannot be back-calculated *by subtracting dilation (d_n) from peak strength*, as done by some! Φ_r or Φ_b would then be *dangerously too high* (and/or JRC would be incorrect).

SCALE-EFFECTS

REDUCTION OF of JRC and JCS with block-size

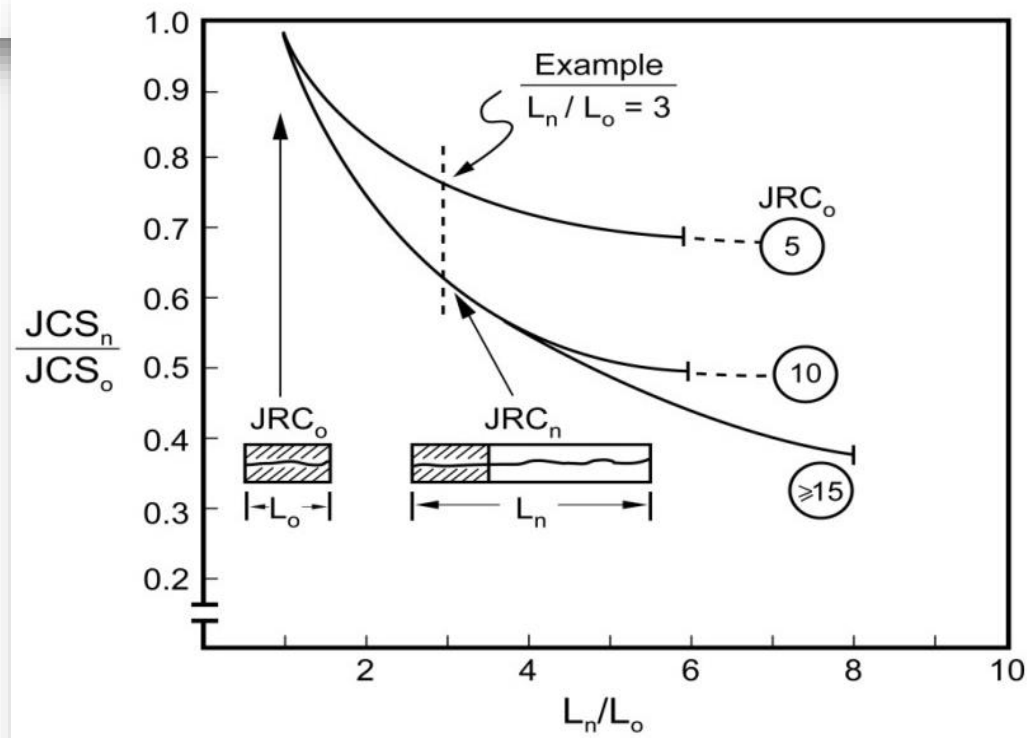
Ln > L0



$$JRC_n \approx JRC_o \left[\frac{L_n}{L_o} \right]^{-0.02\,JRC_o}$$

$$JCS_h \approx JCS_o \left[\frac{L_n}{L_o} \right]^{-0.03\,JRC_o}$$

(Bandis, Dearman, Barton, 1981)



Well-jointed wedge.

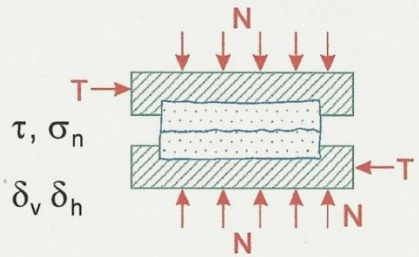
Remains in place because of the *higher shear strength* of the *smaller component blocks* ?

$$\tau = \sigma_n \tan\left[JRC_n \log\left(\frac{JCS_n}{\sigma_n}\right) + \phi_r\right]$$

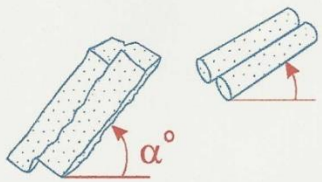
In this case larger block(s)

(and fundamentally lower shear strength too)

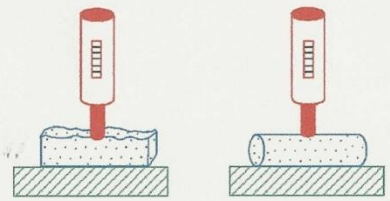
SHEAR BOX AND INDEX TESTING OF ROCK JOINTS



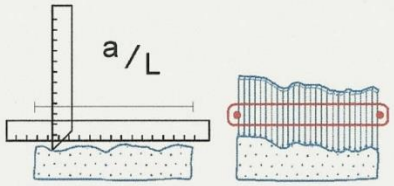
SHEAR BOX



TILT TEST



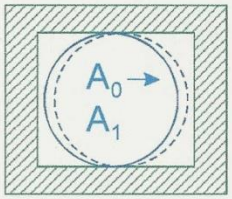
SCHMIDT HAMMER

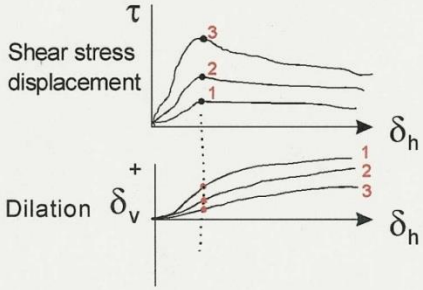


ROUGHNESS PROFILES

$\sigma_n = \frac{N}{A_1}$

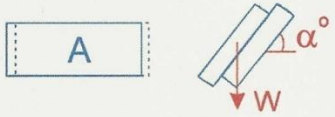
$\tau_n = \frac{T}{A_1}$





$$JRC_o = \frac{\alpha^o - \phi_r}{\log_{10} \frac{JCS}{\sigma_{no}}}$$

$$\sigma_{no} = W/A \cdot \cos \alpha^o$$



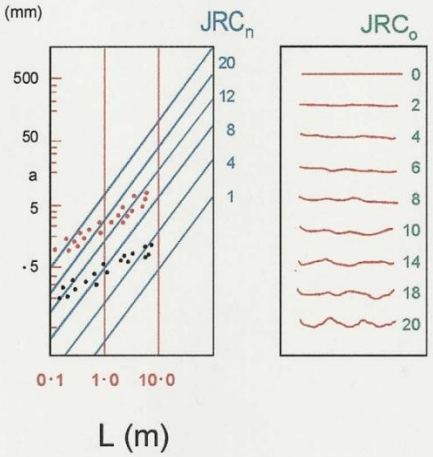
$$\phi_r = (\phi_b - 20^o) + 20 \; r_5/R_5$$

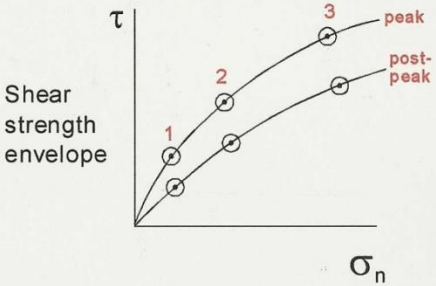
r	R
30	40
~~25~~	~~33~~
31	41
32	~~35~~
32	40
~~24~~	42
31	~~34~~
~~29~~	41
~~23~~	~~36~~
~~26~~	~~37~~

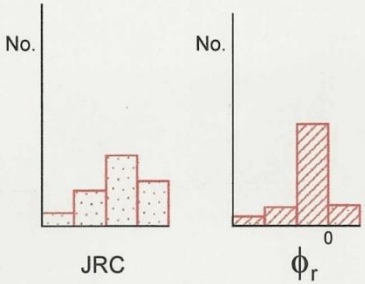
$\bar{r}_5 \rightarrow JCS_o$

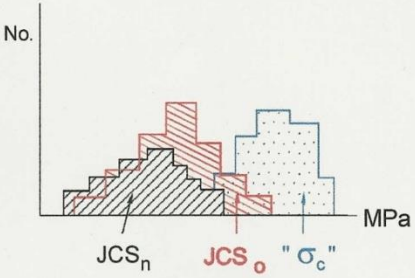
$\bar{R}_5 \rightarrow$ "σ_c"

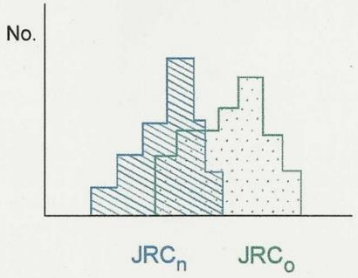
$\bar{r}_5 = 31{\cdot}1 \quad \bar{R}_5 = 40{\cdot}8$











g:\desktop\coreldrw\542004\RockJoints.cdr

Note use of two cores for (unweathered) φ_b

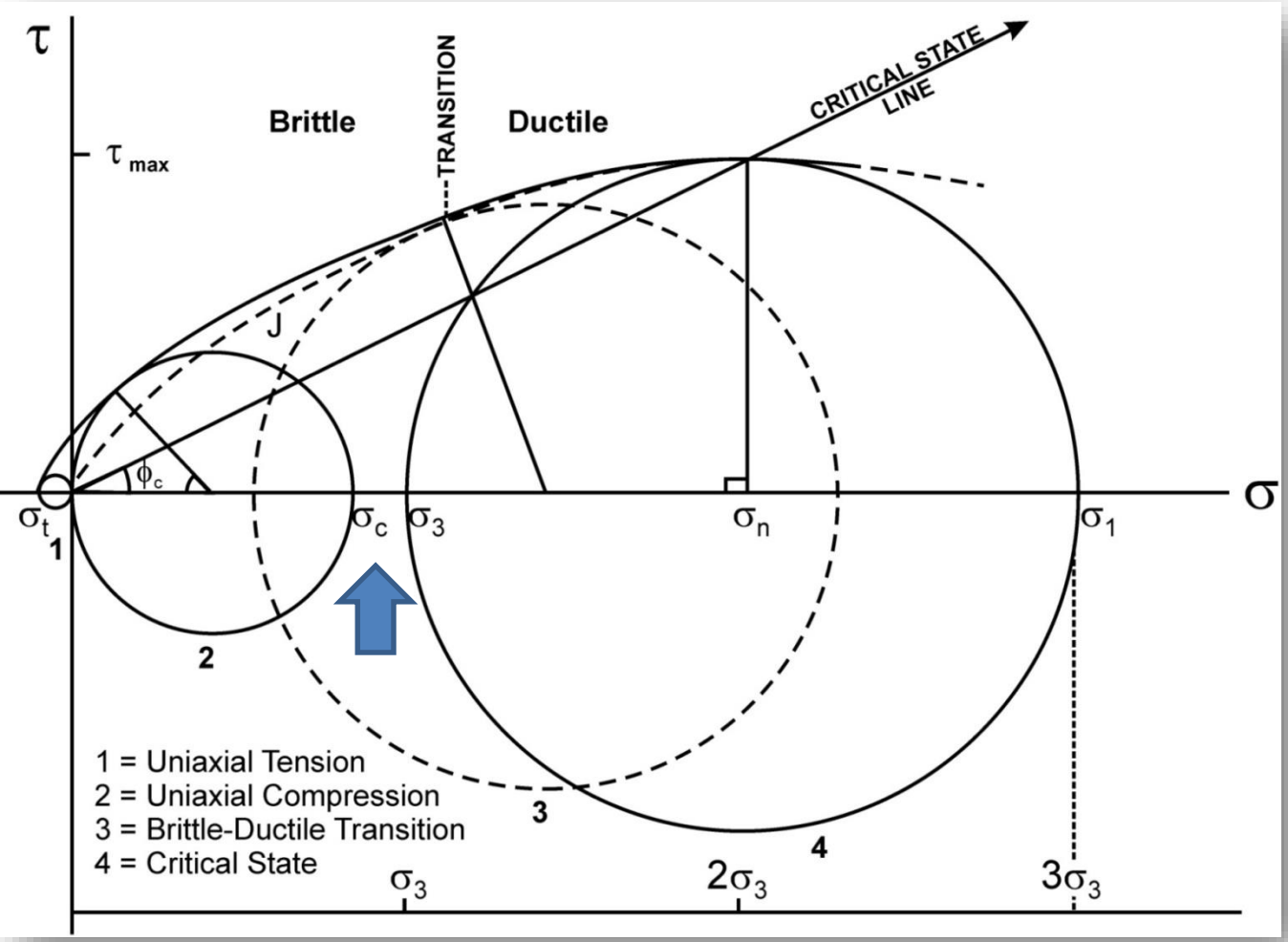
Three cores cause wedging and false (high) values

$$\varphi_r = (\varphi_b - 20^o) + 20\ r_5/R_5$$

SHEAR STRENGTH of INTACT ROCK

NEW CRITERION BASED ON OLD (1976) CONCEPT

(Lucky-break #5)



Critical state concept **recently used by Singh et al., 2011 as basis for** ***improved strength criterion*** **for intact rock.**

The simple *correct-curvature* formulation, *indicates how much deviation from Mohr-Coulomb* is necessary to match the strong curvature up to the critical state.

($\sigma_1 = 3\sigma_3$ and figure Barton, 1976, 2006).

They found that

σ_3 (critical) ≈ σ_c

INTO THE FIELD !!

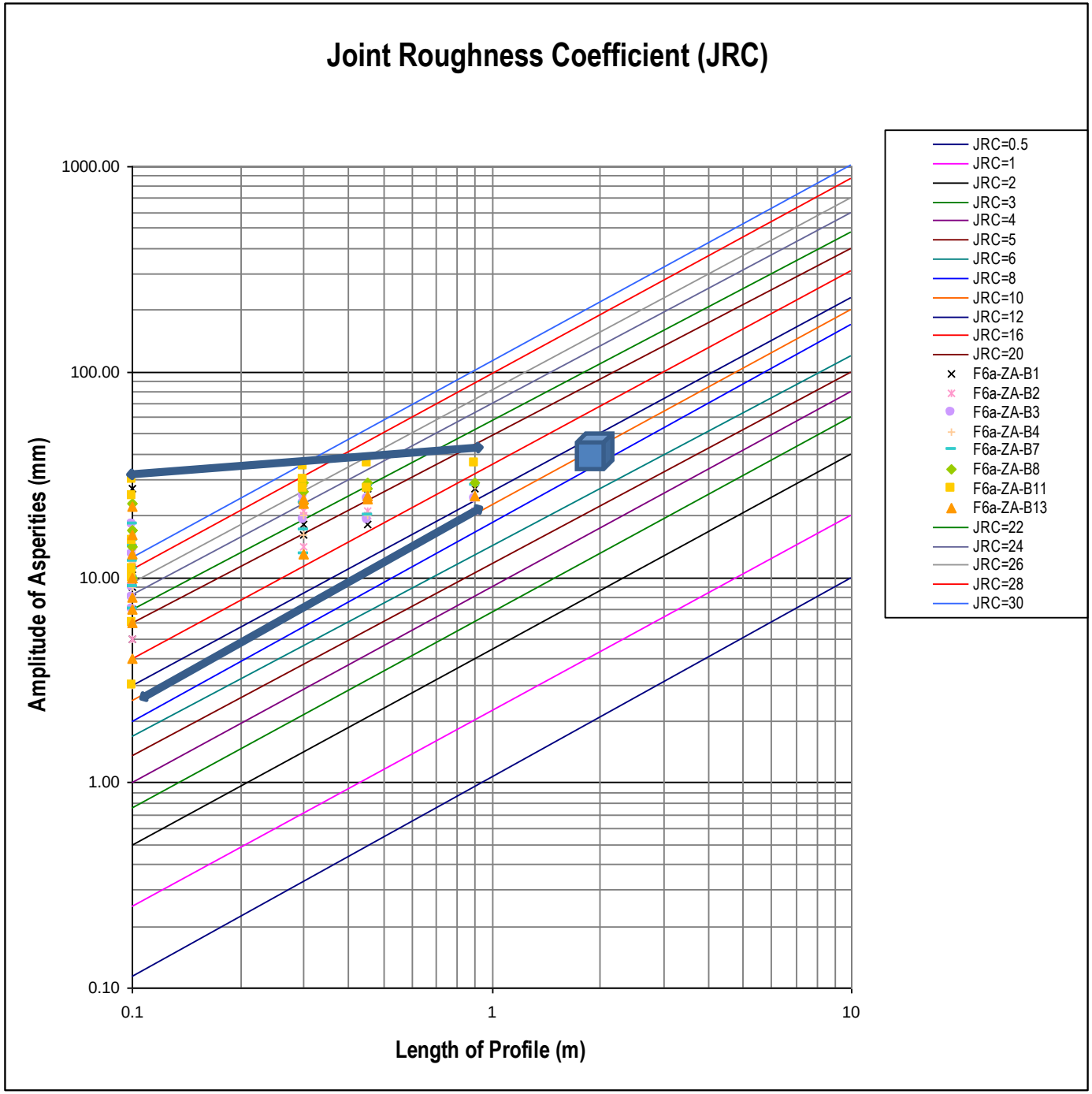
CHARACTERIZATION OF JOINTING, DEFORMABILITY, AT MAJOR DAM SITES

- IRAN: KARUN IV 230 m, BAKHTIARY 325 m
- TURKEY: DERINER DAM 249 m
- CHINA: BAIHETAN DAM 277 m (12.6 MW)

“You need to hire a *rock-climbing-engineering-geology* group to characterise the major joint planes that define the two major wedges that your company are worried about”

Some kilos lighter, and not telling his wife the reason, Iranian colleague M.Zargari is profiling major-joint MJ-67,

Karun IV Dam, Iran



For the very rough bedding plane, used **"a/L"** method

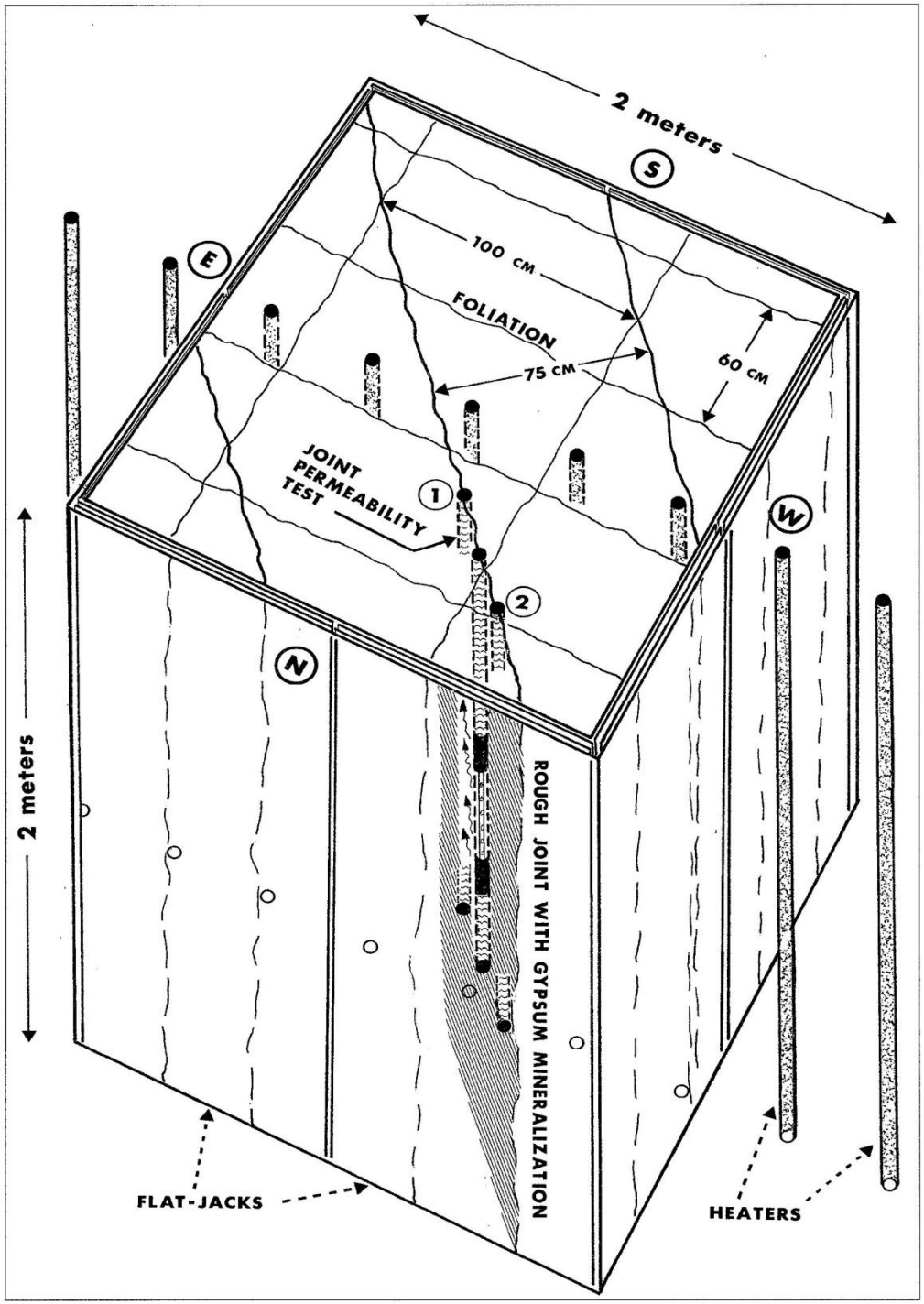
Mean

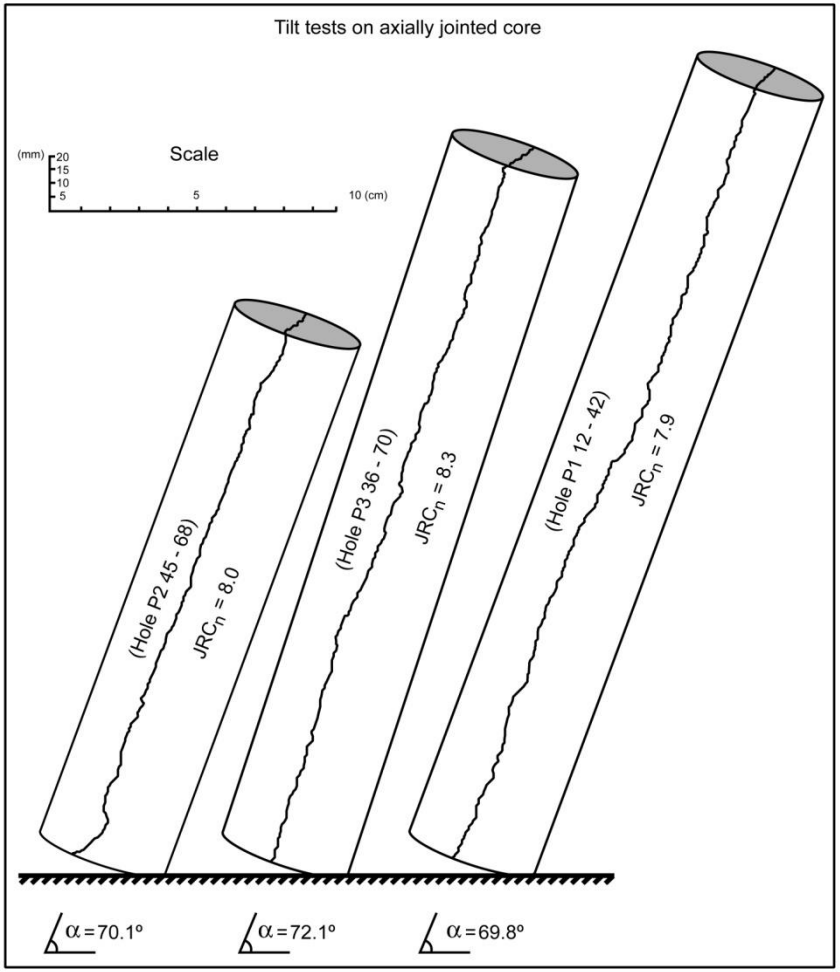
$JRC_n = 11$

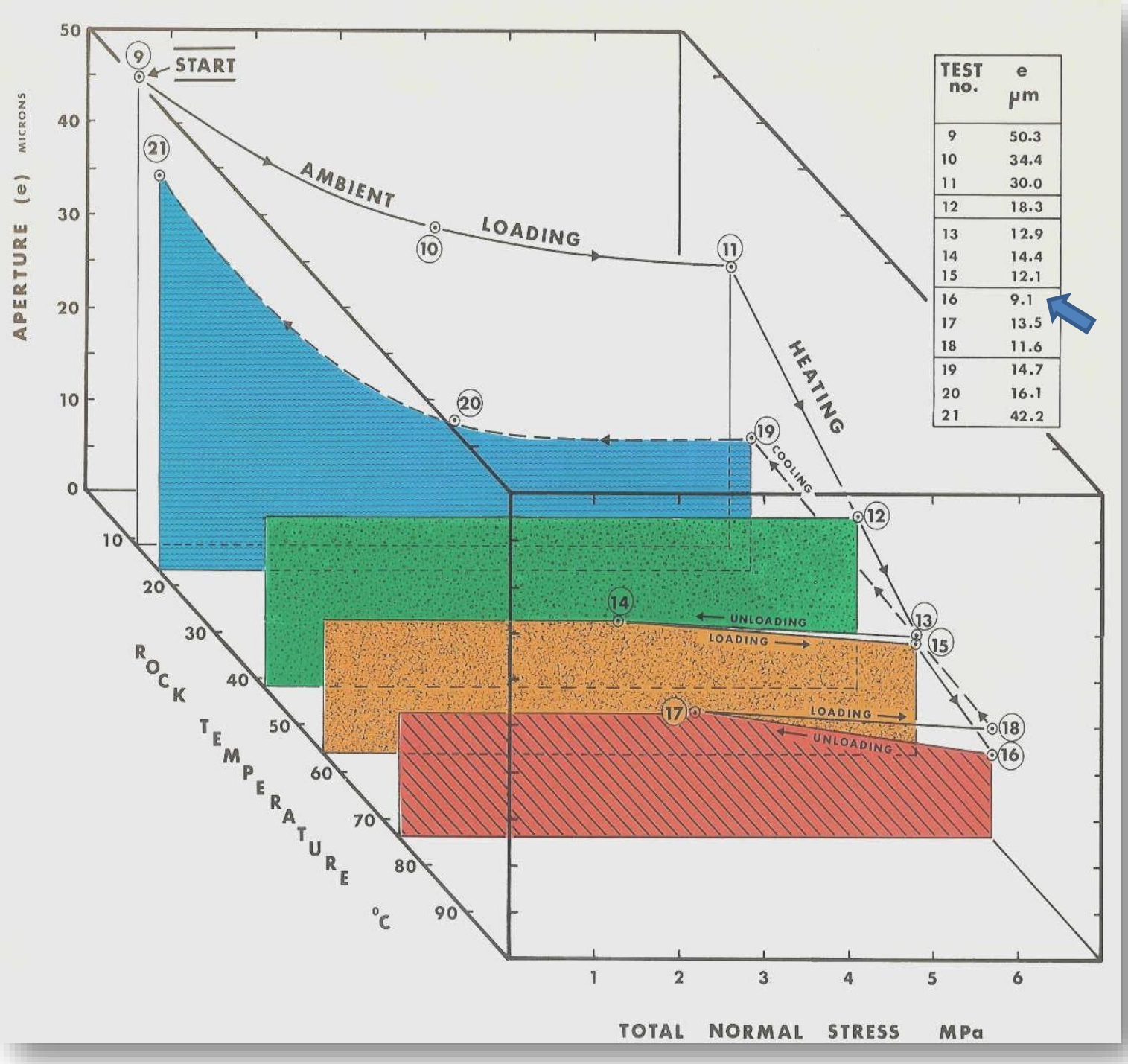
(for 2m block size)

Index characterization of joints at nuclear waste projects:

- BWIP (basalt waste isolation project) USA
- **HTM block test in crystalline rock (Terra Tek)**
- Yucca Mountain/Nevada Test Site/G-Tunnel
- AECL/URL Manitoba (by others)
- UK Nirex Sellafield (NGI/Atkins)
- SKB: Stripa, Åspö, Simpevarp, Forsmark, (NB/NGI)


2 meters
S
E
100 CM
FOLIATION
75 CM
60 CM
JOINT PERMEABILITY TEST
1
2
W
N
2 meters
ROUGH JOINT WITH GYPSUM MINERALIZATION
FLAT-JACKS
HEATERS


Tilt tests on axially jointed core
Scale
(mm)
20
15
10
5
5
10 (cm)
(Hole P2 45 - 68)
JRCn = 8.0
(Hole P3 36 - 70)
JRCn = 8.3
(Hole P1 12 - 42)
JRCn = 7.9
α = 70.1°
α = 72.1°
α = 69.8°



TEST no.	e µm
9	50.3
10	34.4
11	30.0
12	18.3
13	12.9
14	14.4
15	12.1
16	9.1
17	13.5
18	11.6
19	14.7
20	16.1
21	42.2

HTM BLOCK TEST 1980,1981

TERRA TEK

(Lesson #X)

Conducting aperture (**e**) **reduction with temperature alone**

Barton, N. 2007. Thermal over-closure of joints and rock masses and implications for HLW repositories. Proc. of 11th ISRM Congress, Lisbon.

Thermal over-closure so far ignored in numerical modelling and nuclear waste studies??

COLUMNAR JOINTING CONCERNS AT A MAJOR DAM SITE IN CHINA

(Baihetan)

THE STORY FROM INSIDE

RECORDING OF ROUGHNESS FOR JRC ESTIMATION, 70 to 140 m into the canyon walls.

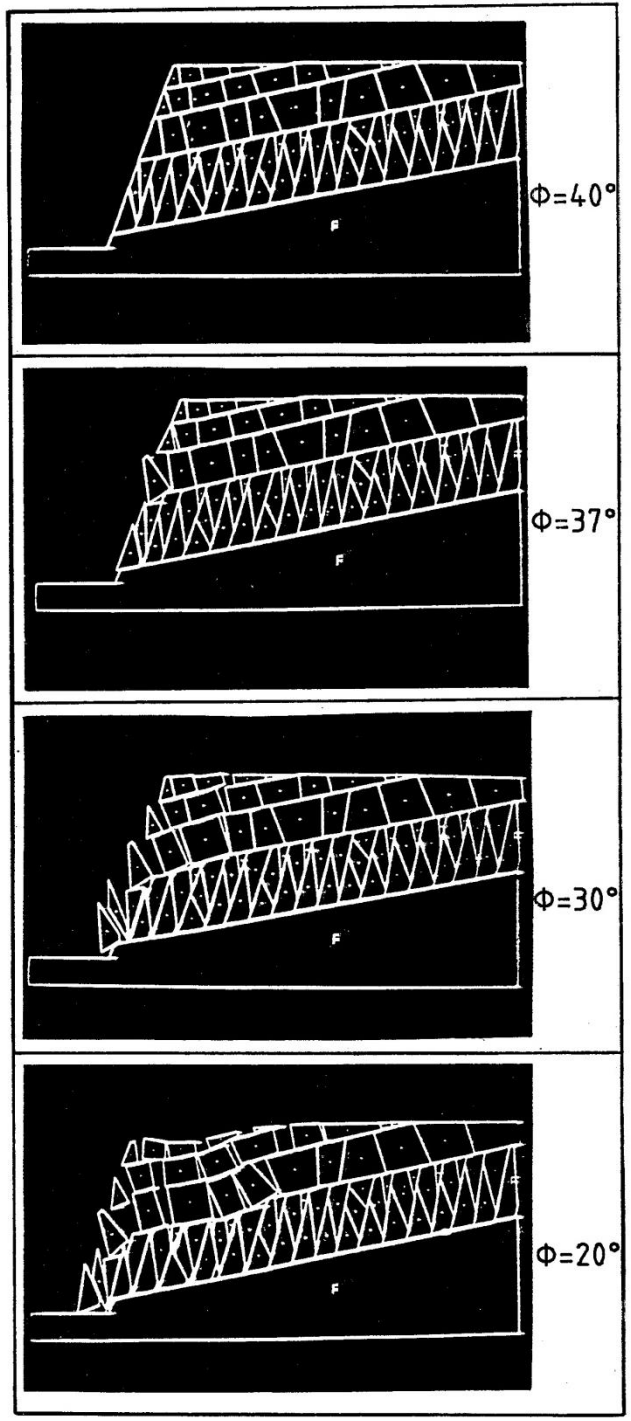
(May need stripping to this competent-rock depth)

RECORDING OF SCHMIDT-HAMMER REBOUND FOR JCS ESTIMATION

APPLICATION OF JRC AND JCS to ROCK MASS DEFORMABILITY UNDERSTANDING

and

MODELLING



μDEC with rigid blocks 1975

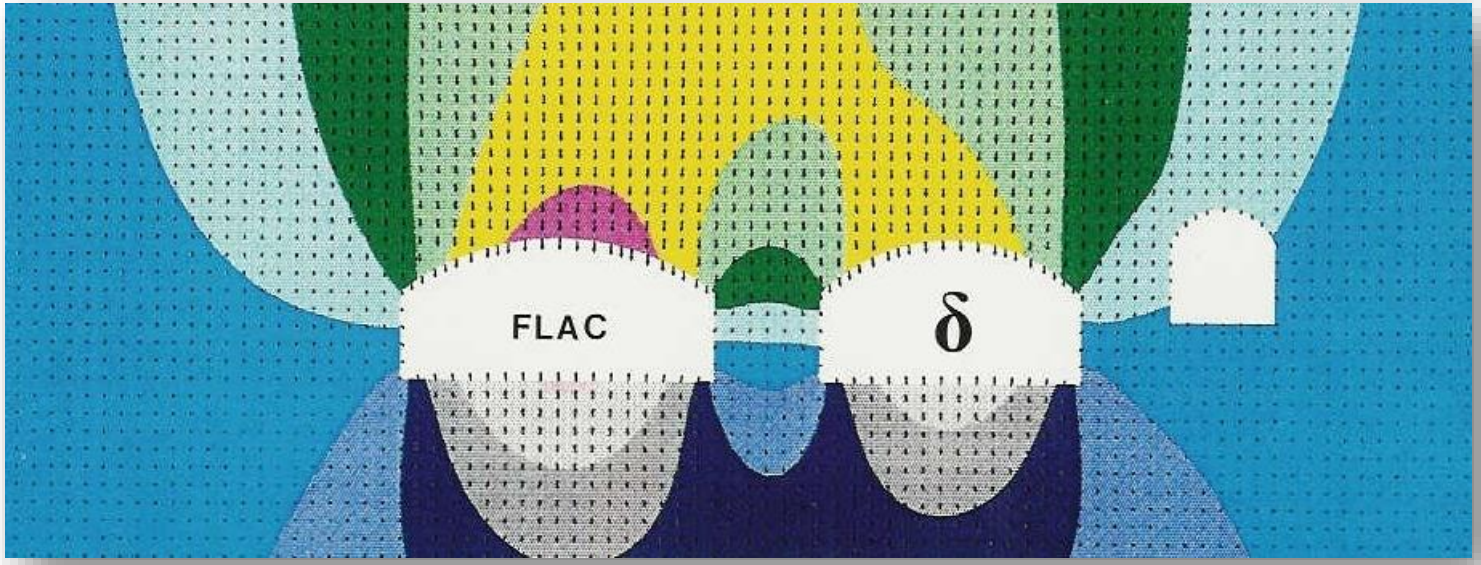
(*Cundall, Voegele and Fairhurst, 1977*)

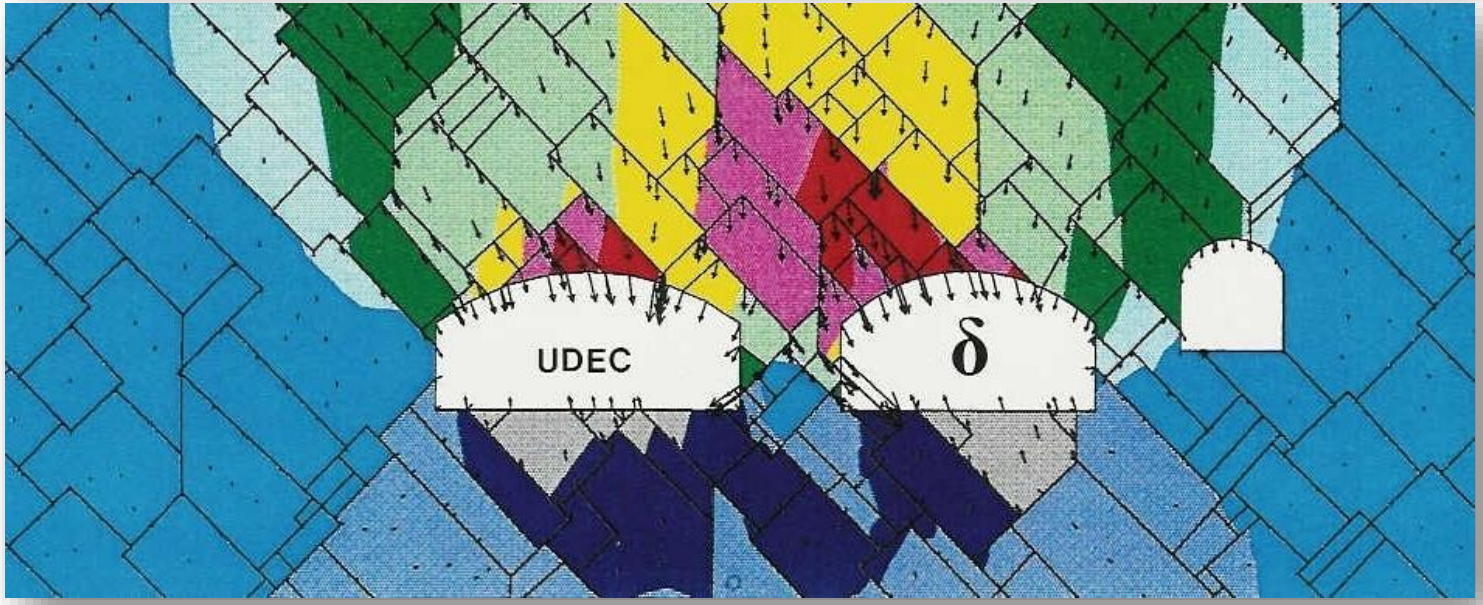
Soon to be followed (in 1980) by UDEC, then UDEC-BB (in 1985)

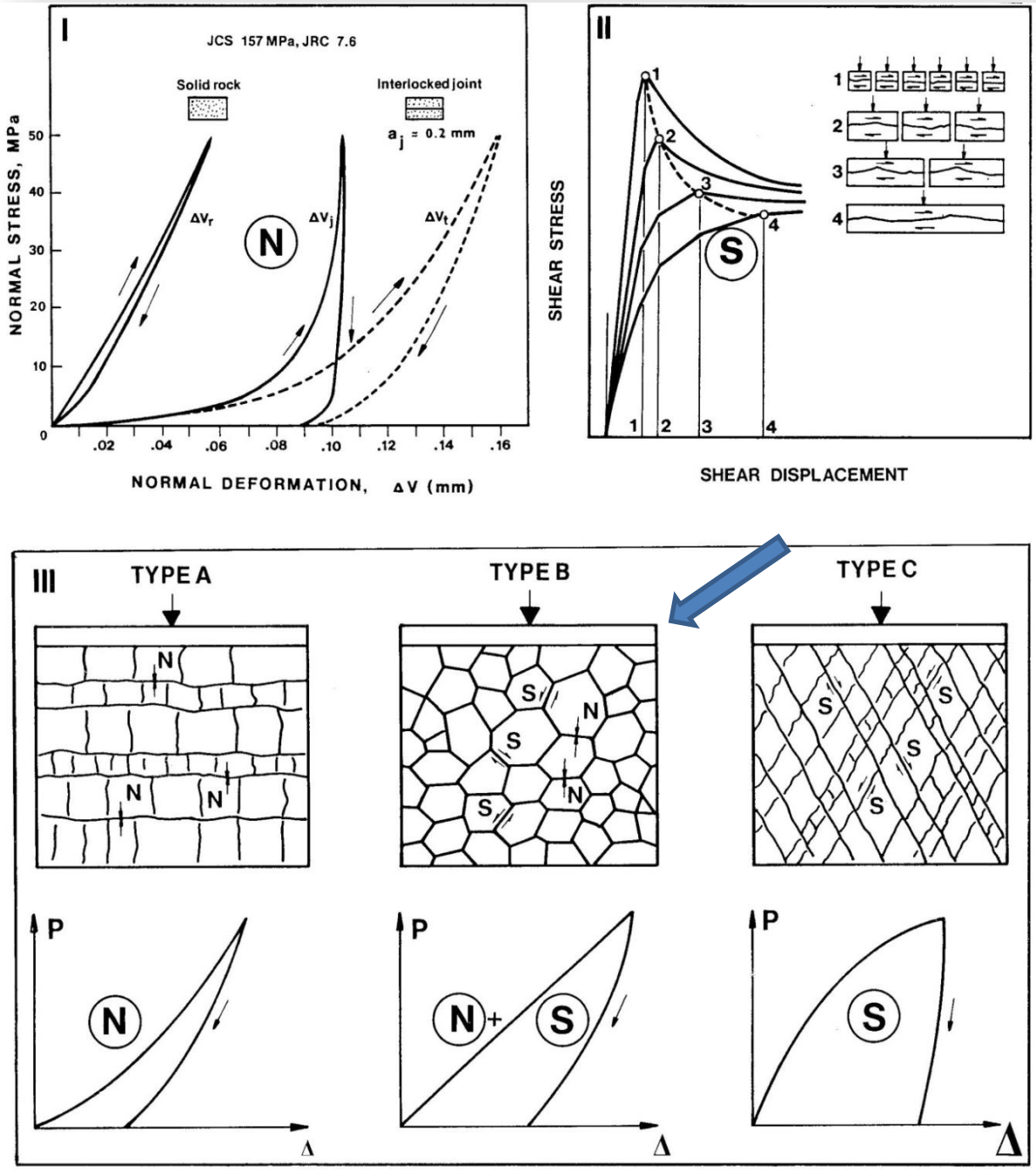
(see next example and contrast to continuum model)

Cundall and Cundall……but the choice is clear!

(NGI modelling by Lisa Backer)





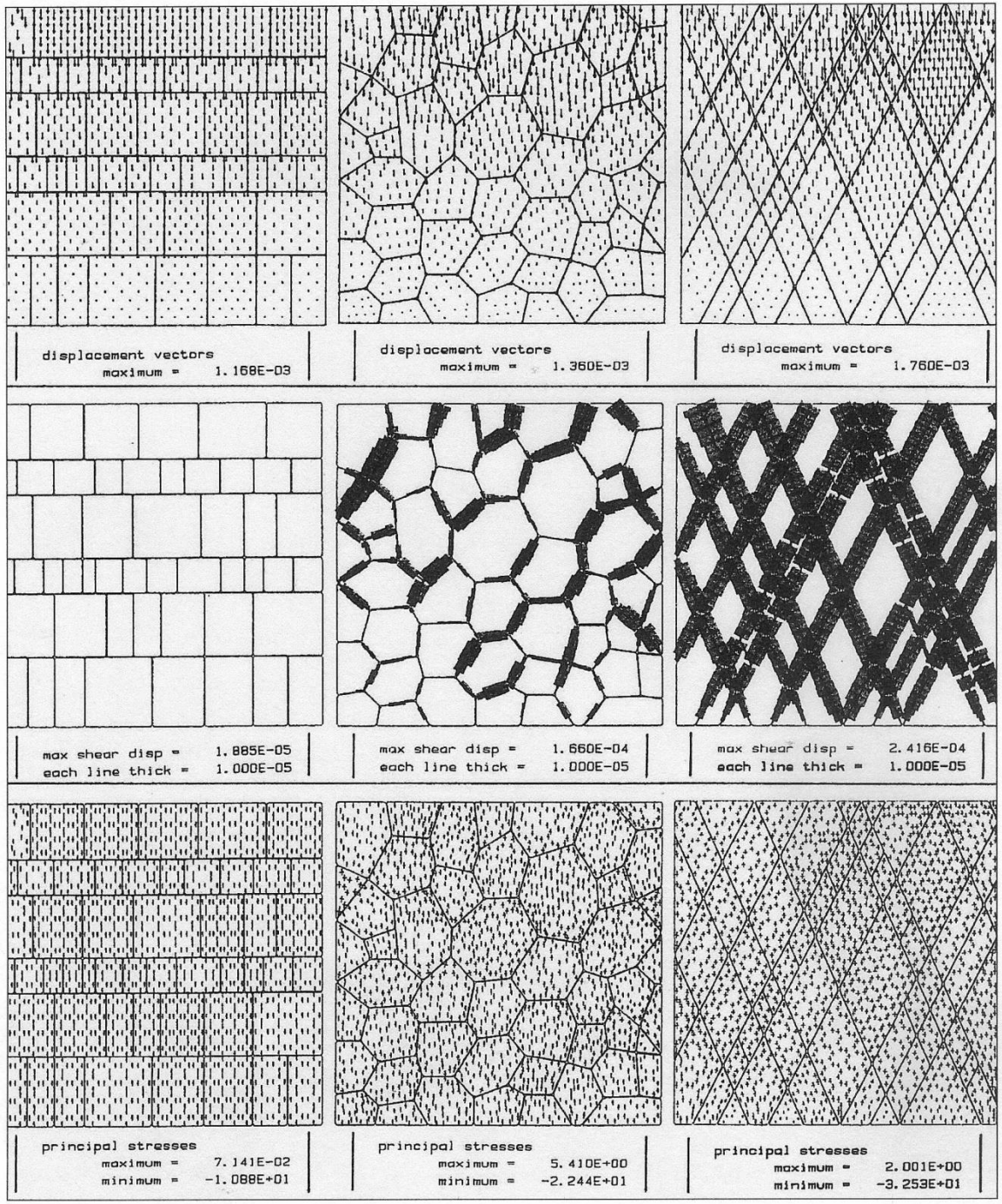


Stress-closure and scale-effect shear tests. Bandis et al. 1981 and 1983

The N, S components in potential rock mass load-deformation mechanisms.

(Barton, 1986)

There is plate-load / block-test evidence for these three P-Δ type-curves.



UDEC-BB simulations

(Chryssanthakis, NGI)

EMPHASISES WHY *DISCONTINUUM ANALYSES GIVE MORE EXCITEMENT/INTEREST/ VALUE/REALISM than analyses without joints!*

Q-SYSTEM (Lucky-break #7)

- SINCE 1974 Q HAS ACTUALLY BECOME "A SYSTEM", SINCE THERE ARE NOW SEVERAL COMPONENTS.
- (About 1500 case records in total)
- Q rockmass classification, **Q-histograms (L-B # 8)**
- Q for helping to select **NMT support (sb, B, Sfr, RRS)**
- Q_c for correlating with **V_P and E_{MASS}**
- Q_{TBM} for TBM prognosis
- Q_{SLOPE} for selecting safe slope angles (in progress)
- **Qc split into CC and FC** (***if*** 'continuum' modelling)

EXAMPLE OF SLOPE ANGLE MATCHED TO GEOTECHNICAL PROPERTIES (*or to local Q-slope: 0.1, 1.0, 10*). (Panama expansion project, 2011. PCA photo)

Why/how was Q developed?

Because of a question to NGI in 1973:

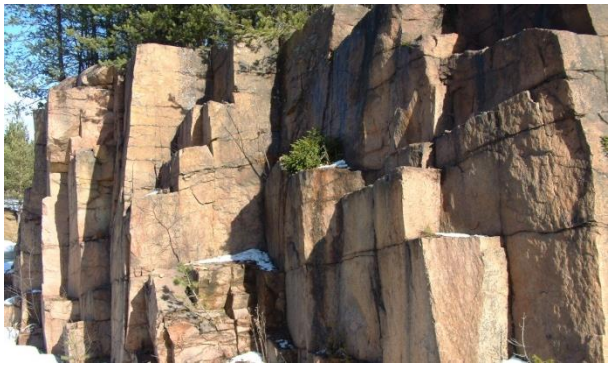
"Why are (our)Norwegian underground power houses showing such a variety of deformations"?

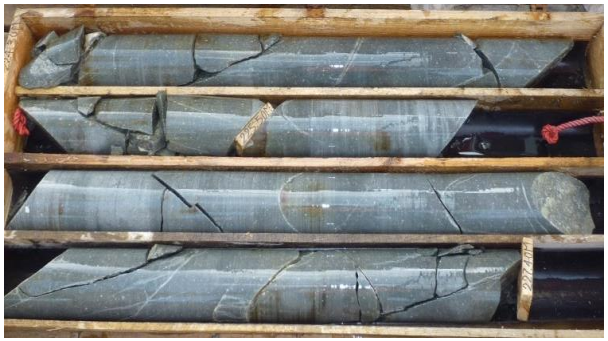
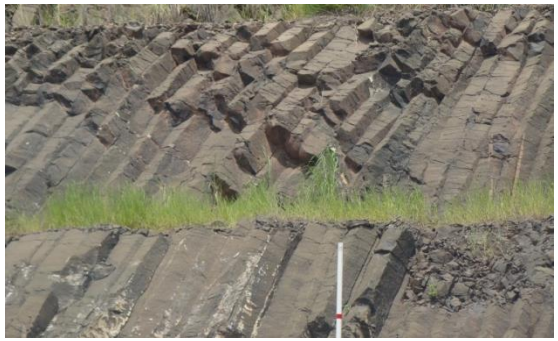
(from Norwegian State Power Board/ STATKRAFT)

Question passed to NB. Answer given after 6 months of *Q-system development!*

VARIABLES: Rock mass quality, support type/quantity, span/height, depth, stress.

212 case records used. B, S(mr), B+S(mr), CCA.

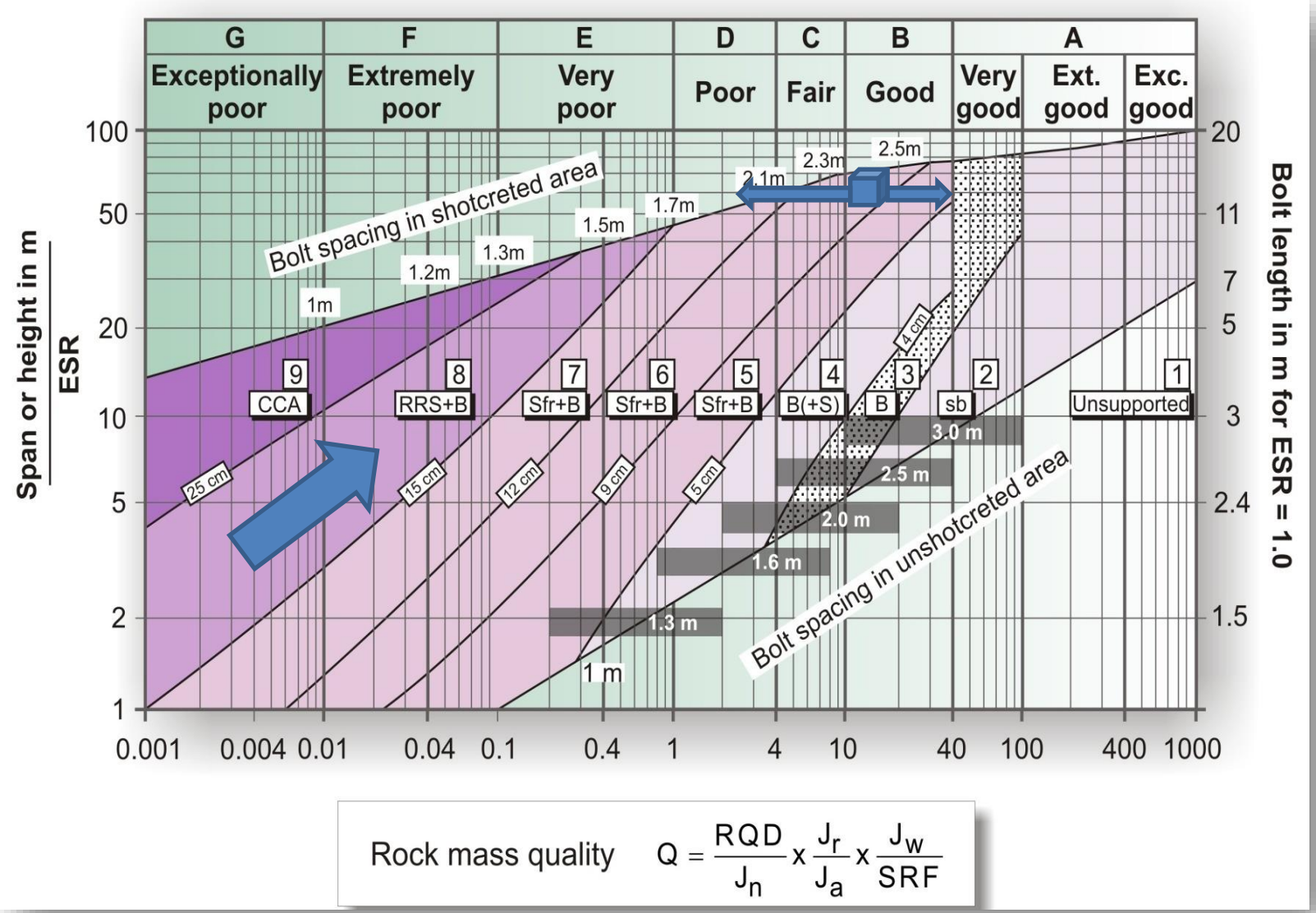


VARIABLE WORLD NEEDS *BROAD-REACH CLASSIFICATION* METHOD

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

0.001→1000, or 5→95, or F6→F1 ???



Rock mass quality $Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$

Grimstad and Barton, 1993

(Norwegian conference),

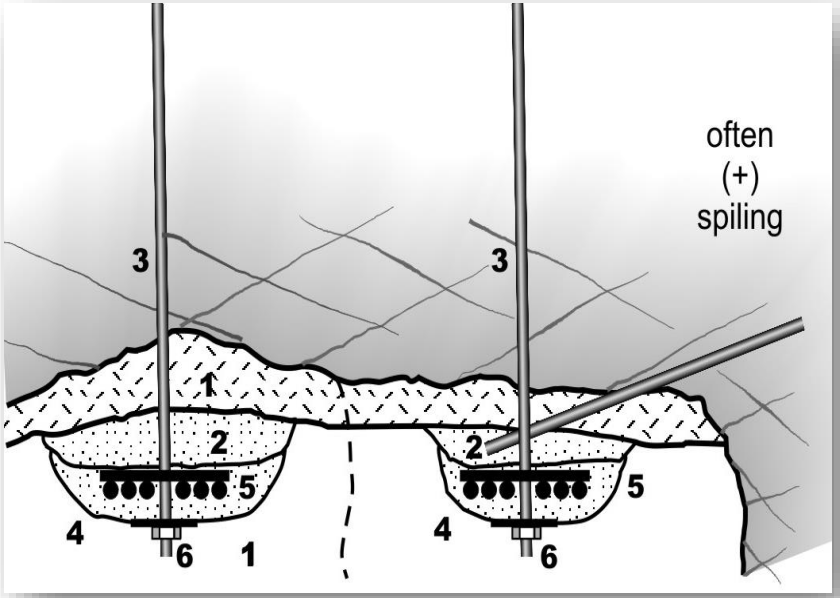
Barton and Grimstad, 1994

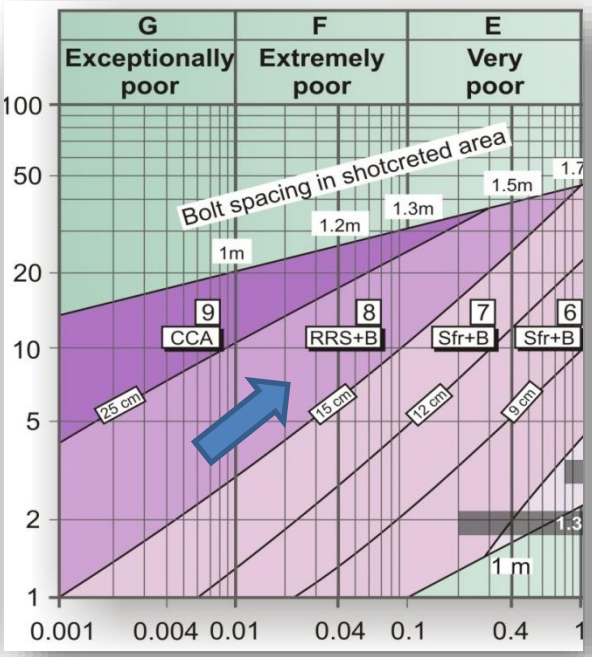
(Austrian conference).

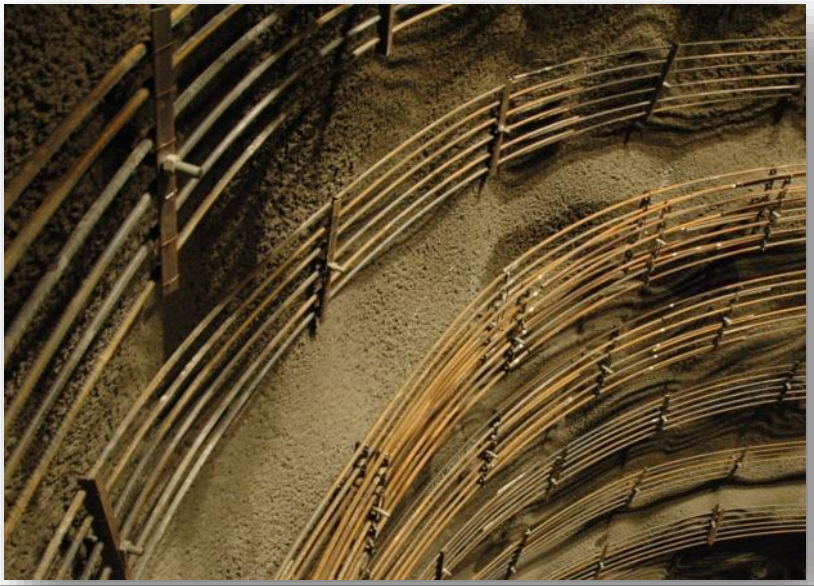
The Q-system is most strongly associated with '<u>single shell'</u> NMT solutions :

(B+Sfr + water control) in *mostly* better rock, cost about 1/5 x 'double-shell' NATM, e.g. 20,000 US $/m compared to 100,000 US $/m

(Costs from many countries).







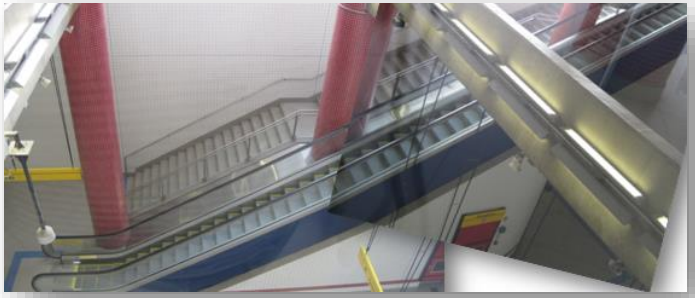
RRS is a flexible (until bolted) 'lattice' girder.

3D effect because of S(fr) arches.

QUESTION:

SHALLOW METRO or DEEP METRO?

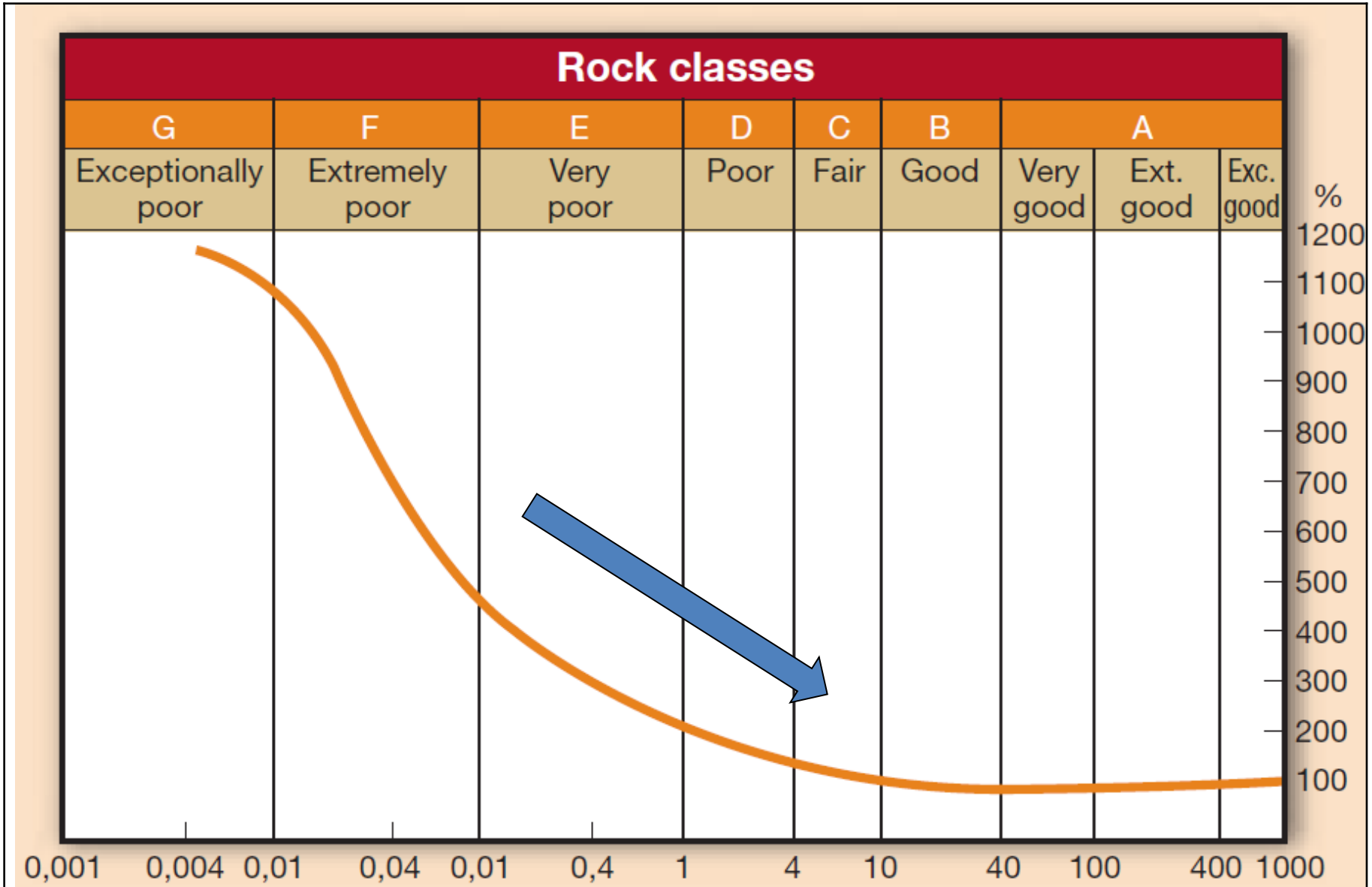
- MIXED-FACE OR ROCK?
- **5 m PER WEEK OR 20 m PER WEEK?**
- COST DIFFERENCE MAY BE 5:1 (at least)



RELATIVE ***COST*** FOR TUNNEL EXCAVATION AND SUPPORT

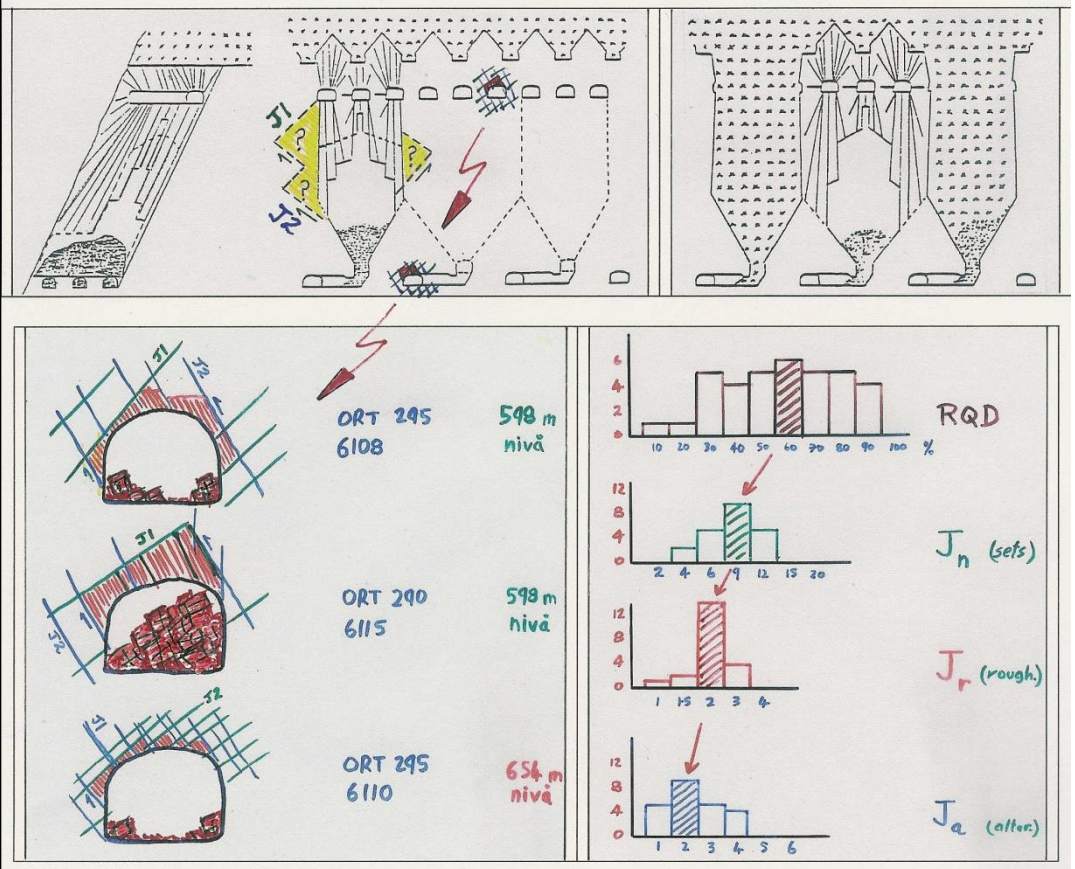
(Barton, Roald, Buen, 2001)

……potential benefits of pre-grouting, especially if Q ≈ 0.1

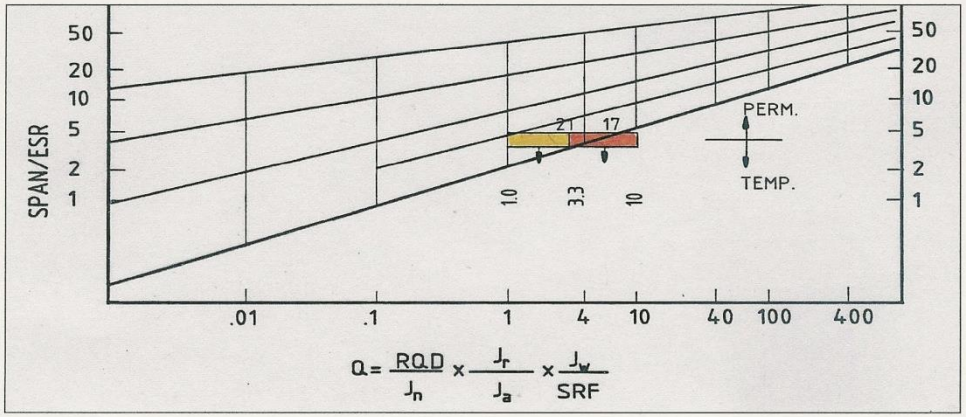


Q-HISTOGRAM METHOD of logging

(brief examples)



$Q_{most\ frequent}$ = 60/9 x 2/2 x 1/2 = 3.3	$Q_{typ.\ range}$ = (50-80)/(6-12) x (2-3)/(2-4) x 1/2 = 1.0 → 10
SPAN = 6m ESR ≥ 1.6	*'17' = S 2-3 cm, '21' = B 1m + S 2-3 cm*



Kiruna LKAB 'Oscar' caving project.

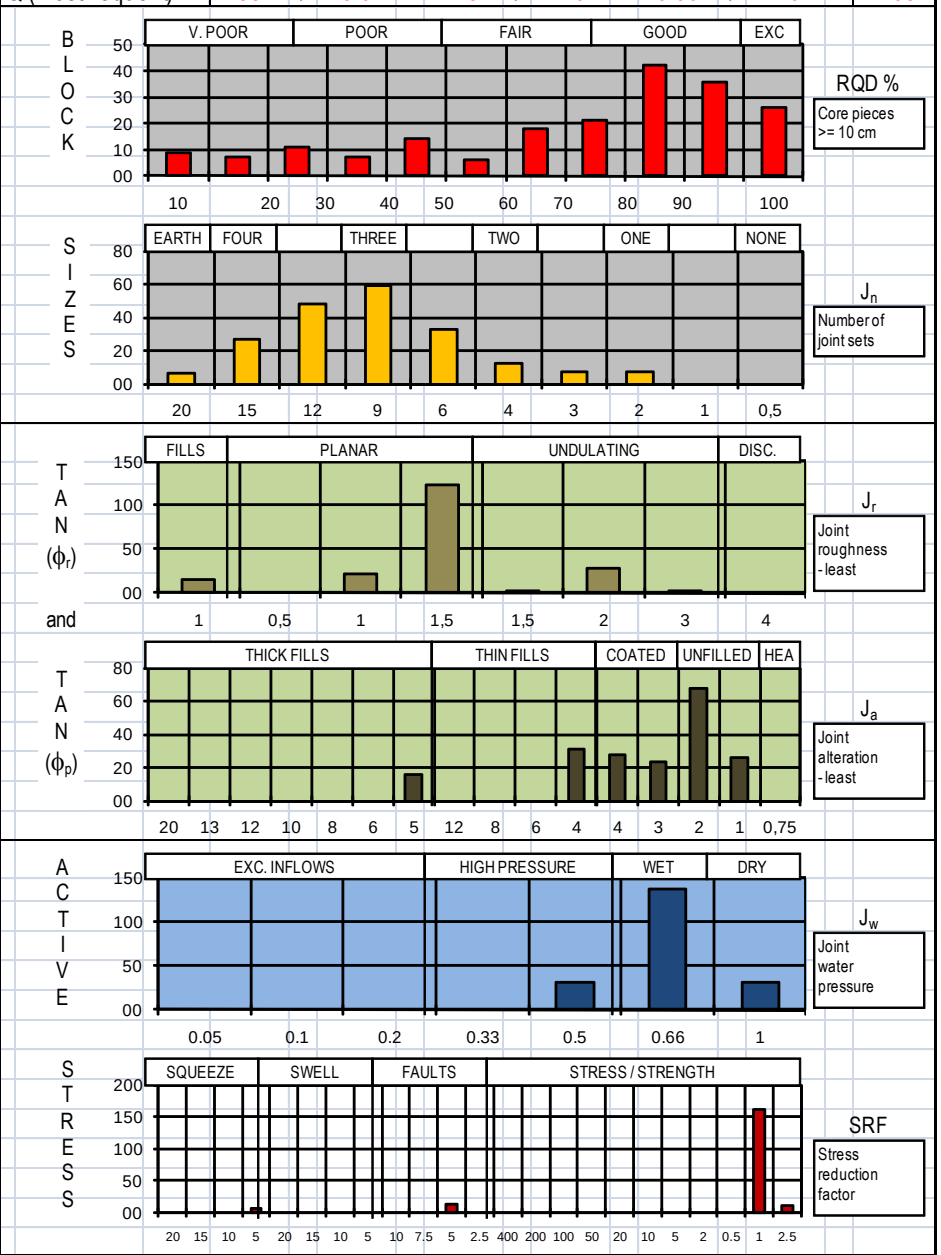
Some details of rock mass Q-characterization, and a pre-1993 Q-system temporary support assessment.

Barton, 1987 NGI contract report.).

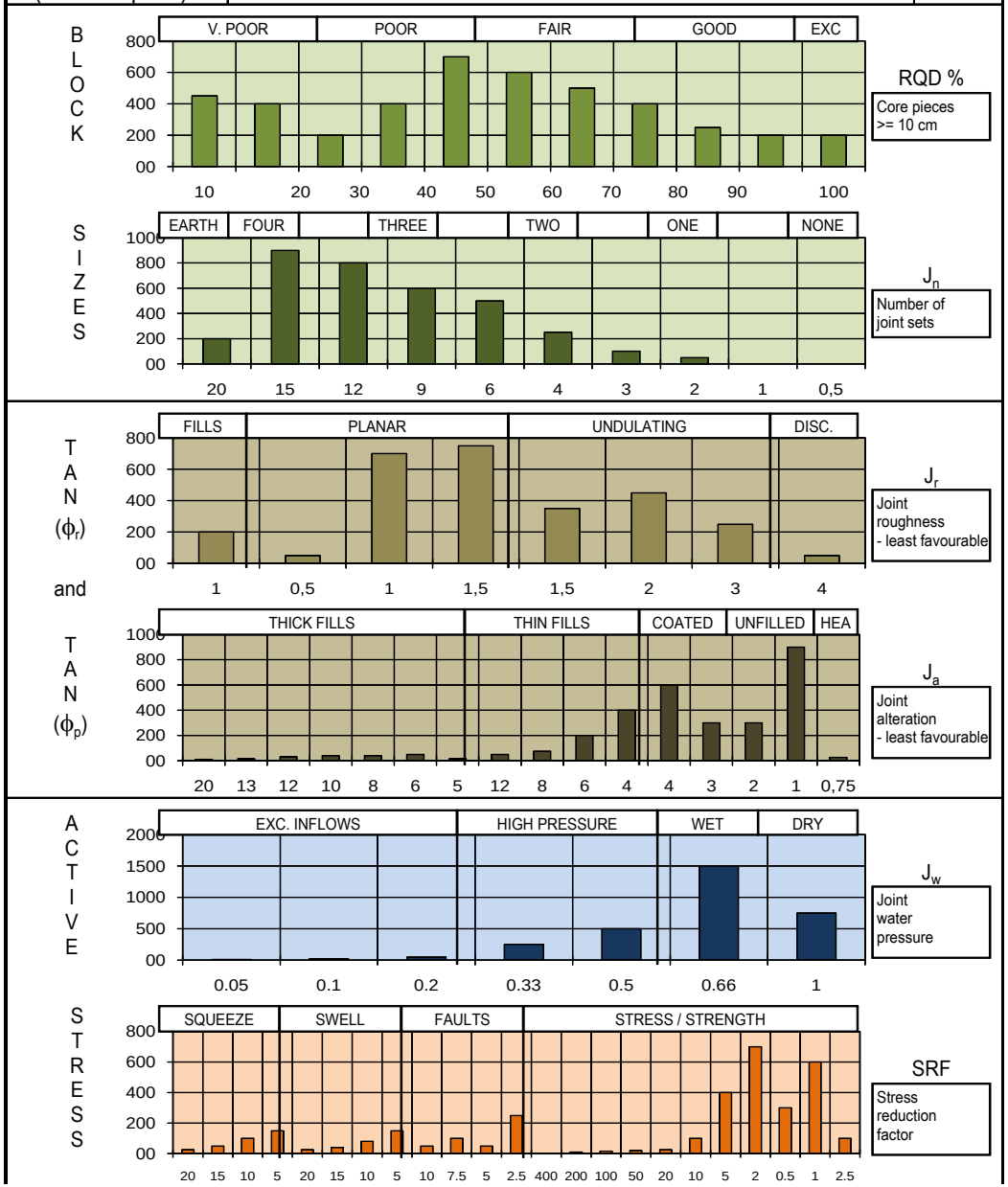
Note the early use of Q-parameter histogram-logging used by NB in the last 25 years.

THE RESULT OF Q-HISTOGRAM LOGGING OF SIX CORES AT A PLANNED METRO PROJECT IN HONG KONG. DEVIATE HOLES as here!

Q - VALUES:	(RQD	/	Jn)	*	(Jr	/	Ja)	*	(Jw	/	SRF)	=	Q
Q (typical min)=	10	/	20.0	*	1.0	/	5.0	*	0.50	/	5.0	=	0.010
Q (typical max)=	100	/	2.0	*	2.0	/	1.0	*	1.00	/	1.0	=	100.0
Q (mean value)=	71	/	9.6	*	1.5	/	2.8	*	0.69	/	1.5	=	1.78
Q (most frequent)=	85	/	9.0	*	1.5	/	2.0	*	0.66	/	1.0	=	4.68



Q - VALUES:	(RQD	/	Jn)	*	(Jr	/	Ja)	*	(Jw	/	SRF)	=	**Q**
Q (typical min)=	10	/	20.0	*	1.0	/	12.0	*	0.20	/	20.0	=	**0.000**
Q (typical max)=	100	/	3.0	*	3.0	/	1.0	*	1.00	/	0.5	=	**200.0**
Q (mean value)=	50	/	10.9	*	1.6	/	3.5	*	0.68	/	5.4	=	**0.26**
Q (most frequent)=	45	/	15.0	*	1.5	/	1.0	*	0.66	/	2.0	=	**1.49**

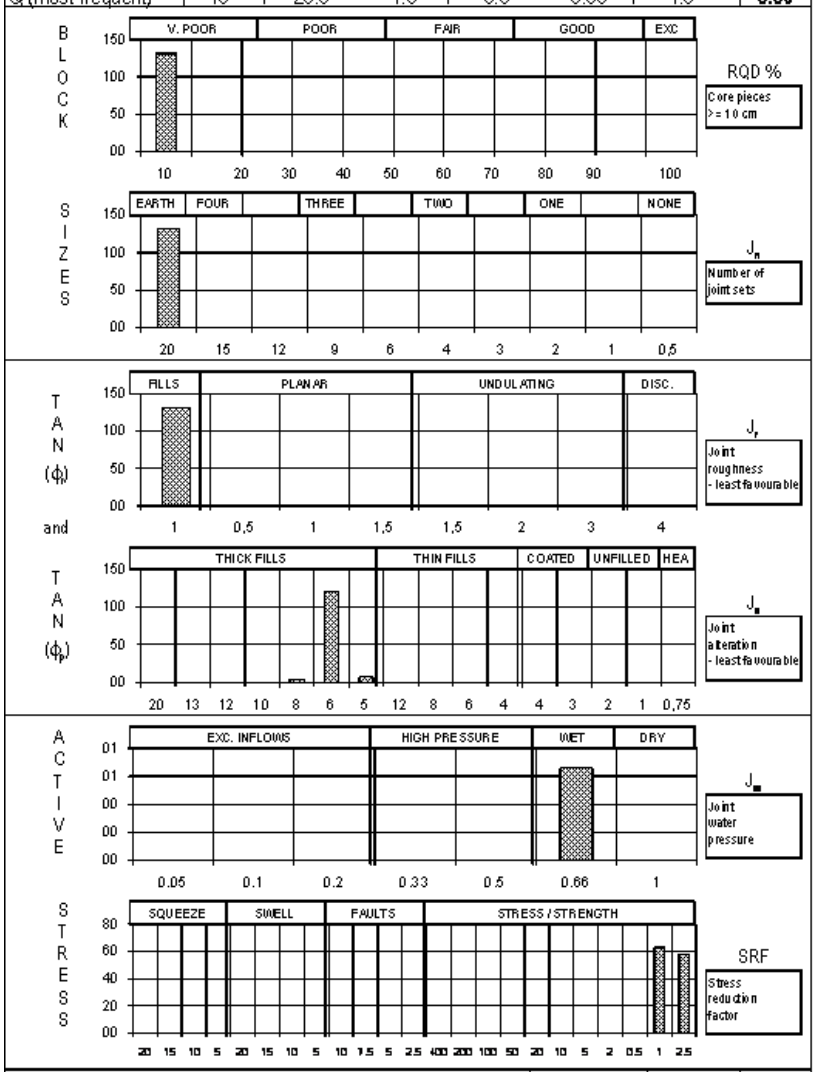


J & K rail-link, Kashmir.
Here 12m/2 years.

How do the Q-parameter histograms change, as depth is increased in the same rock type?

CHARACTER OF SAPROLITE AND SOIL

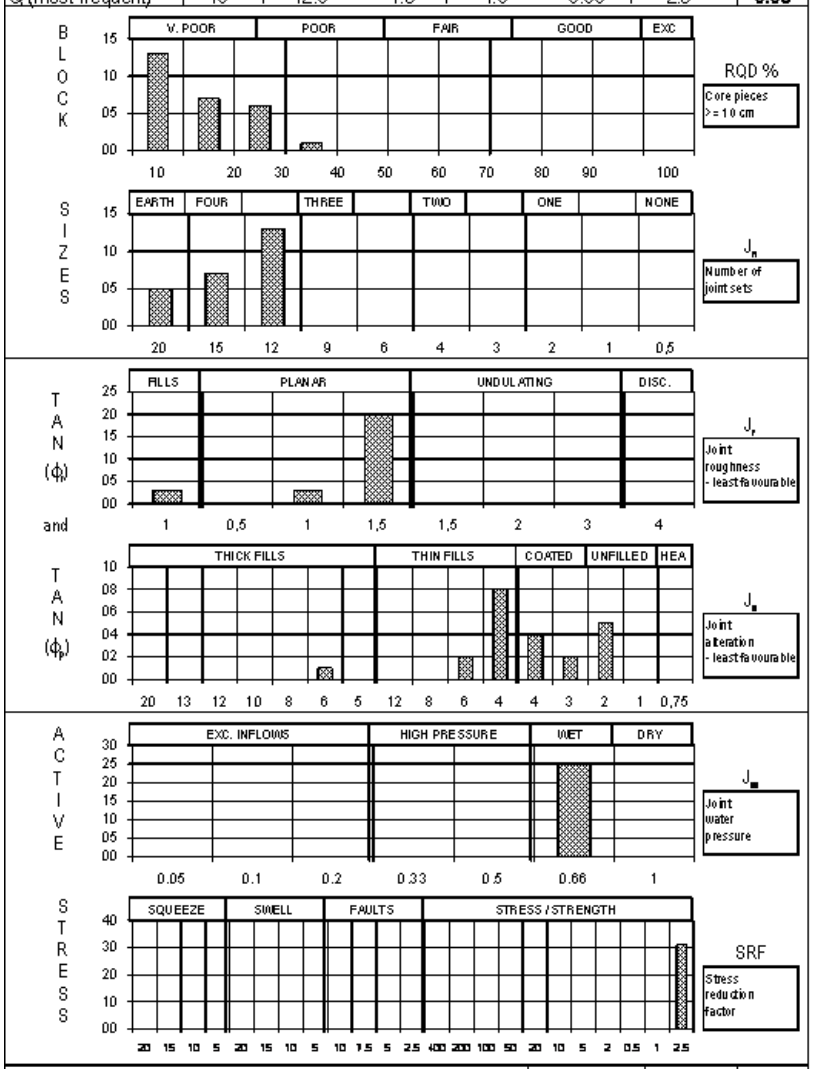
Q - VALUES:	(RQD	/	Jn)	*	(Jr	/	Ja)	*	(Jw	/	SRF)	=	Q
Q (typical min)=	10	/	20.0	*	1.0	/	8.0	*	0.66	/	2.5	=	**0.017**
Q (typical max)=	10	/	20.0	*	1.0	/	5.0	*	0.66	/	1.0	=	**0.1**
Q (mean value)=	10	/	20.0	*	1.0	/	6.0	*	0.66	/	1.7	=	**0.03**
Q (most frequent)=	10	/	20.0	*	1.0	/	6.0	*	0.66	/	1.0	=	**0.06**



PANAMA-COLÓN AUTOPISTA CORE LOGGING	Rev.	Report No.	Figure No.
		NB&A 1	G1
Q-histograms for the various sections of saprolite and soil	Borehole No.: various	Drawn by nb	Date #######
logged in the upper sections of SM-839(1), 842(1), 805(1),	Depth zone (m) mostly 0 to 10m	Checked nrb	
834(1), 807(1), 208(1), 827(1), 829(1), 8 832(1), 830(1) and 833(1).		Approved	

LOGGED CHARACTER OF NEAR-SURFACE SANDSTONES

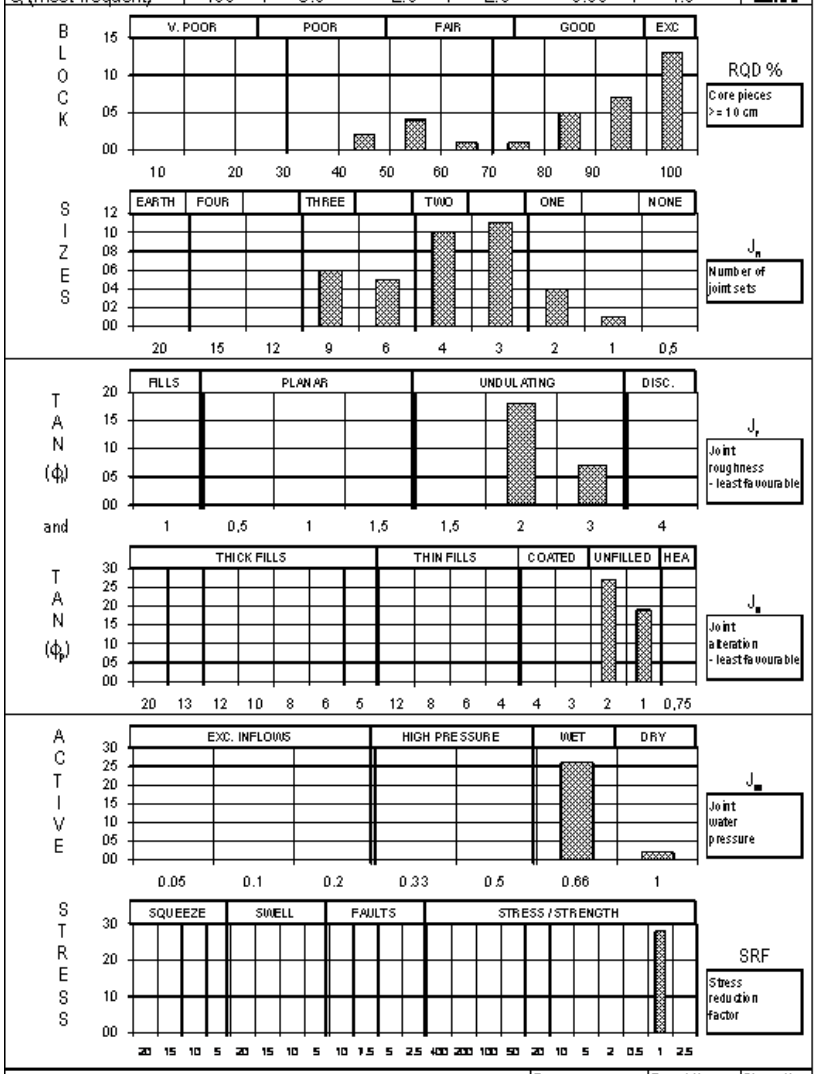
Q - VALUES:	(RQD	/	Jn)	*	(Jr	/	Ja)	*	(Jw	/	SRF)	=	Q
Q (typical min)=	10	/	20.0	*	1.0	/	4.0	*	0.66	/	2.5	=	**0.033**
Q (typical max)=	25	/	12.0	*	1.5	/	2.0	*	0.66	/	2.5	=	**0.4**
Q (mean value)=	16	/	14.4	*	1.4	/	3.7	*	0.66	/	2.5	=	**0.11**
Q (most frequent)=	10	/	12.0	*	1.5	/	4.0	*	0.66	/	2.5	=	**0.08**



PANAMA-COLÓN AUTOPISTA CORE LOGGING	Rev.	Report No.	Figure No.
		NB&A 1	G2
The nine locations with sandstones split	Borehole No.: various	Drawn by nb	Date 15/04/07
into near-surface and deeper characteristics.	Depth zone (m) various from 0 to 10m	Checked	
This figure G2 gives near-surface characteristics		Approved nrb	

LOGGED CHARACTER OF DEEPER SANDSTONES

Q - VALUES:	(RQD	/	Jn)	*	(Jr	/	Ja)	*	(Jw	/	SRF)	=	Q
Q (typical min)=	55	/	9.0	*	2.0	/	2.0	*	0.66	/	1.0	=	**4.033**
Q (typical max)=	100	/	2.0	*	3.0	/	1.0	*	0.66	/	1.0	=	**99.0**
Q (mean value)=	86	/	4.5	*	2.3	/	1.6	*	0.68	/	1.0	=	**18.86**
Q (most frequent)=	100	/	3.0	*	2.0	/	2.0	*	0.66	/	1.0	=	**22.00**



PANAMA-COLÓN CORE LOGGING	Rev.	Report No. NB&A 1	Figure No. G3
The nine locations with sandstones split	Borehole No.: various	Drawn by nb	Date #######
into near-surface and deeper characteristics.	Depth zone (m) mostly > 10m	Checked nrb	
This figure G3 gives the deeper characteristics		Approved	

“Q-system linkages to parameters useful for design are *based on sound, simple empiricism, that works because it reflects practice, and that can be used because it can be remembered*. It does not require black-box software evaluation”.

VELOCITY-MODULUS-PERMEABILITY-Q-VALUE CHALLENGES,

AT BAKHTIARY DAM SITE, IRAN

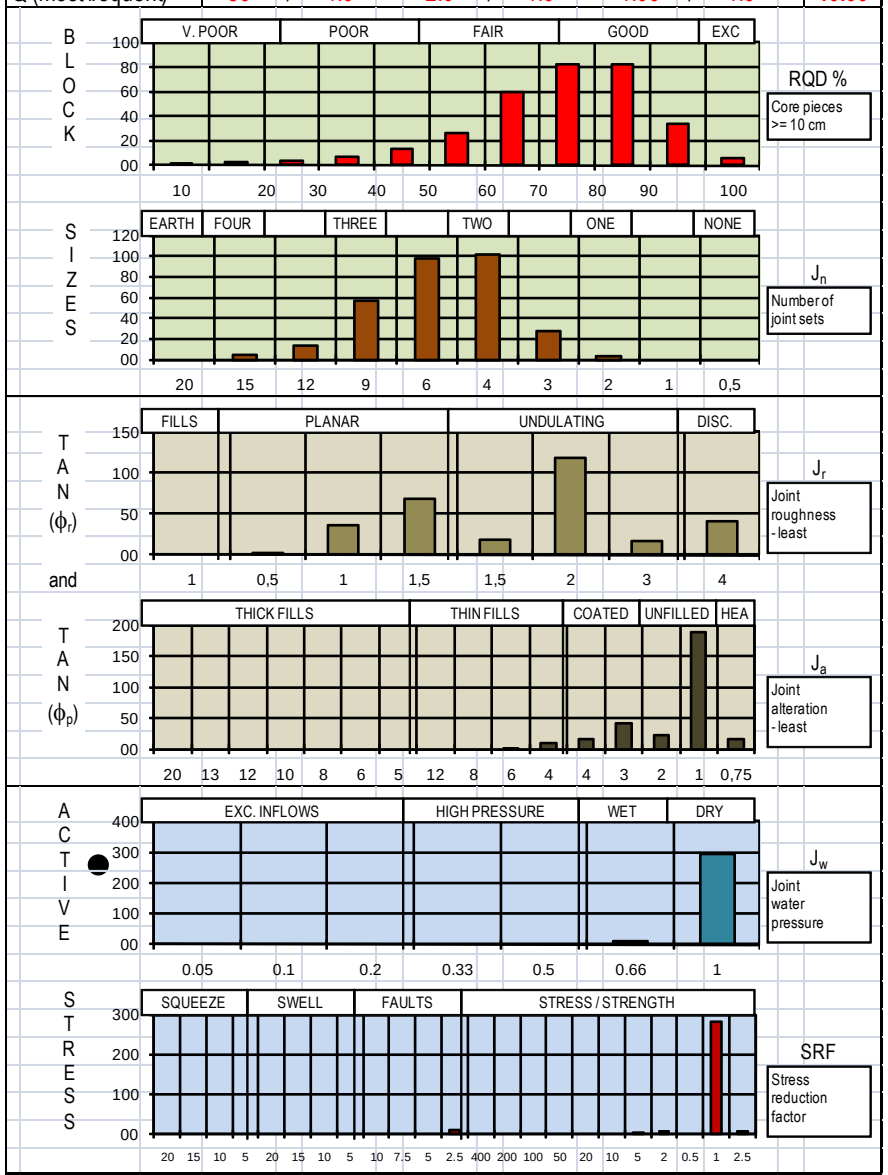
(PROJECTED 325 m)

How to characterize voids?

Velocity-modulus-permeability-Q-value correlation difficulties.

Upper diversion tunnel: top heading

Q - VALUES:	(RQD	/	Jn)	*	(Jr	/	Ja)	*	(Jw	/	SRF)	=	Q
Q (typical min)=	10	/	15.0	*	0.5	/	6.0	*	0.66	/	5.0	=	0.007
Q (typical max)=	100	/	2.0	*	4.0	/	0.8	*	1.00	/	1.0	=	266.7
Q (mean value)=	73	/	6.0	*	2.0	/	1.6	*	0.99	/	1.1	=	13.74
Q (most frequent)=	80	/	4.0	*	2.0	/	1.0	*	1.00	/	1.0	=	40.00



BAKHTIARY DAM HEPP UPPER DIVERSION TUN	Rev.	Report No.	Figure No.
		NB&A #3	6

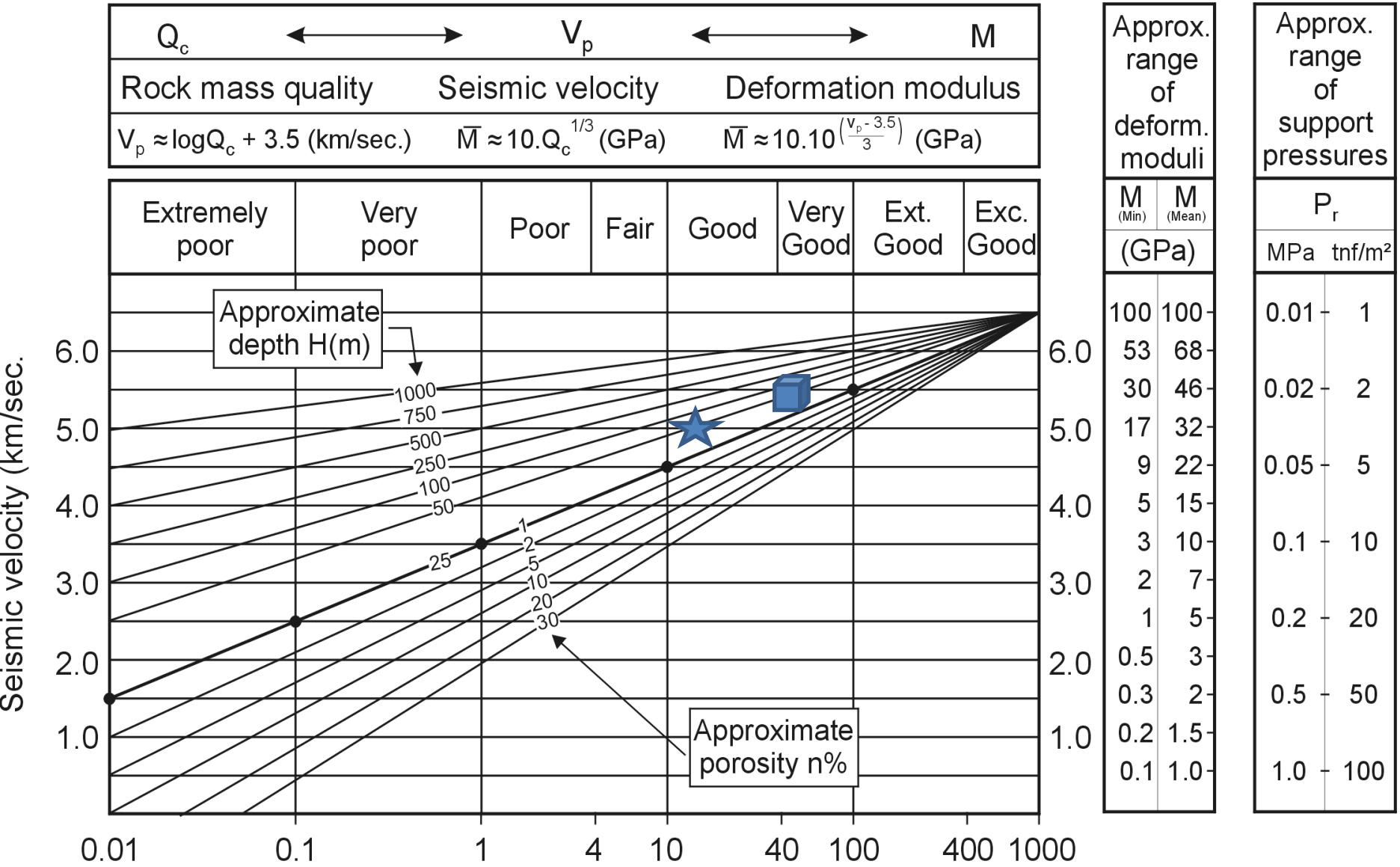
In diversion tunnel

$Q_{m.f.} = 40$

$Q_{mean} = 14$

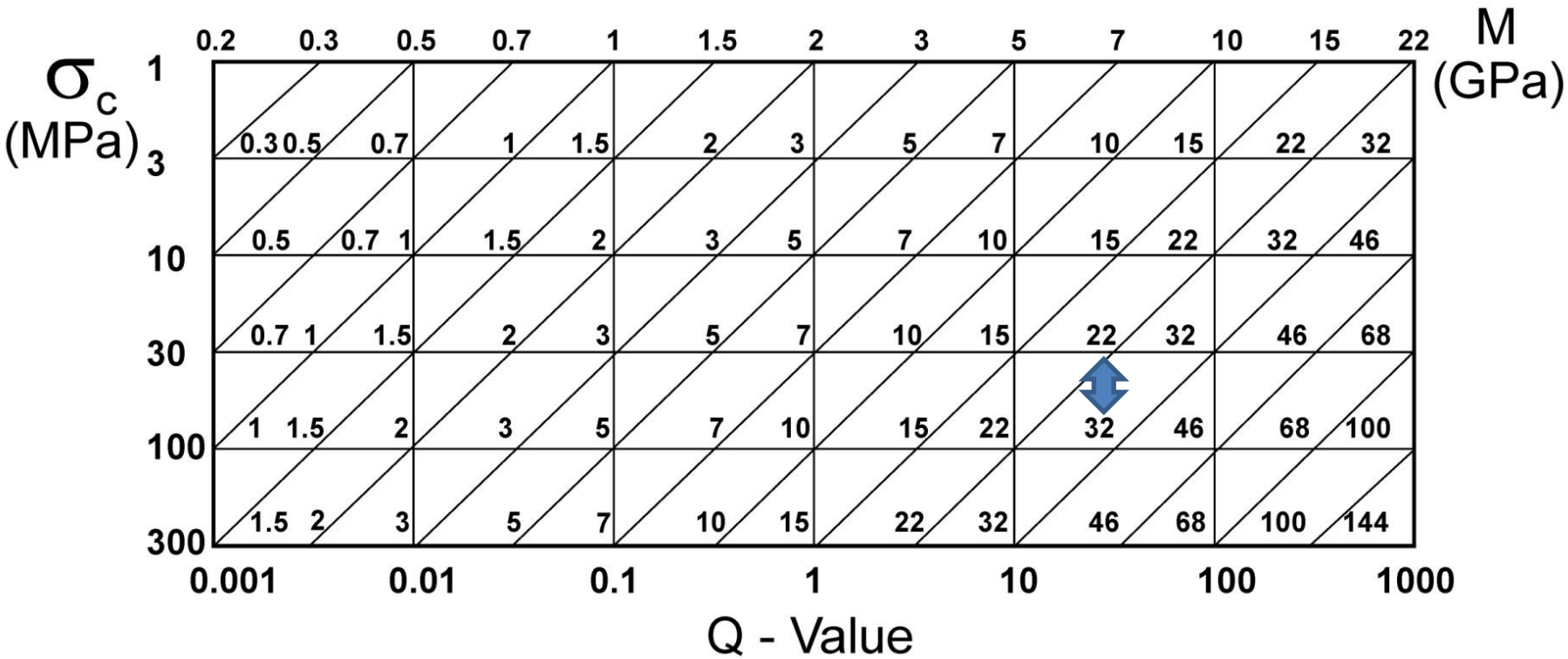
Next steps:

1. Convert Q to Qc
   (What UCS?)
2. Convert to V_p
3. Convert to E_{mass}



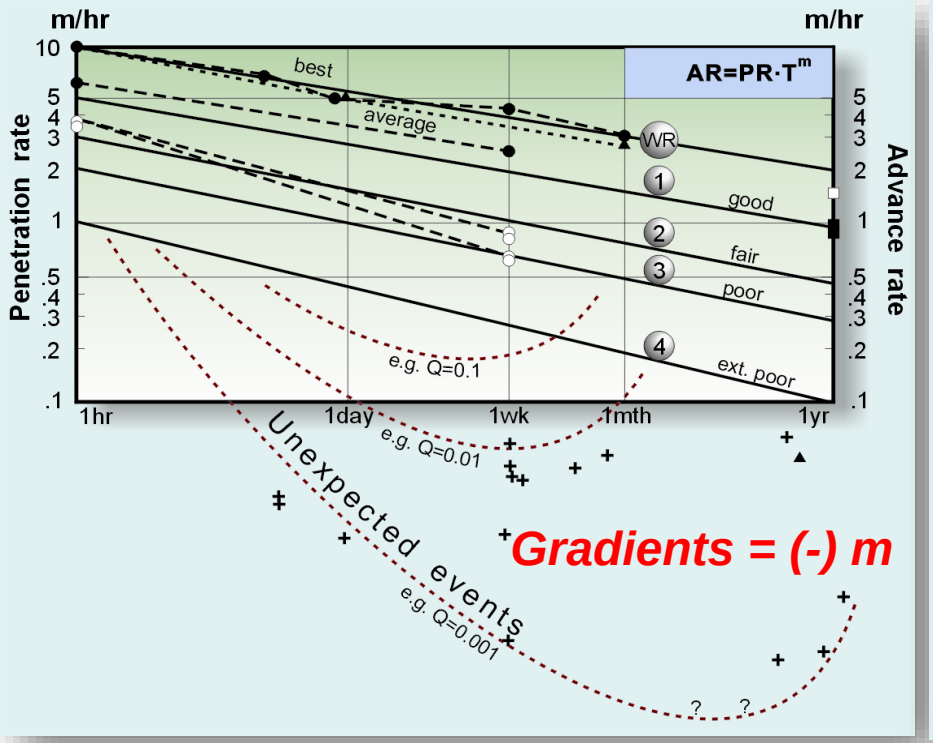
$$Q_c = \left[\frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \right] \frac{\sigma_c}{100}$$

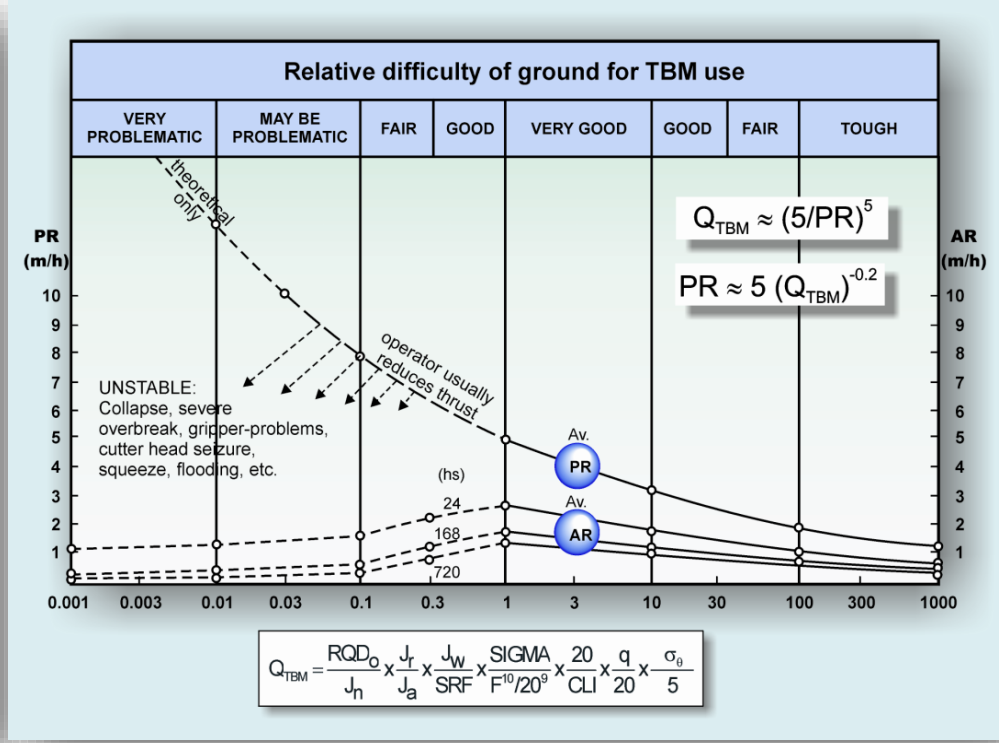
Extracting UCS from Qc (near-surface moduli only)



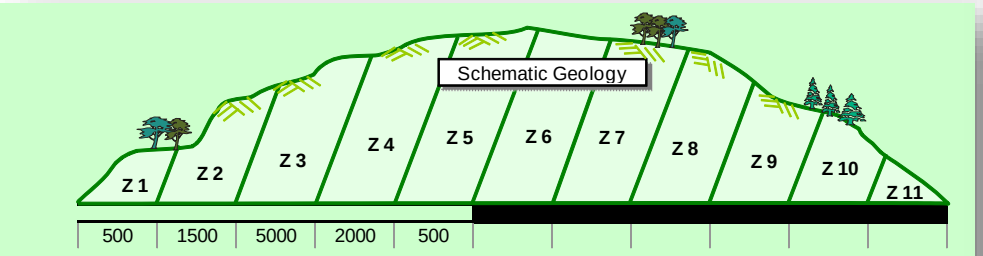
Few pages only!

Q_{TBM}





The Q_{TBM} method was also developed via case records (145 cases, 1000 km of TBM)

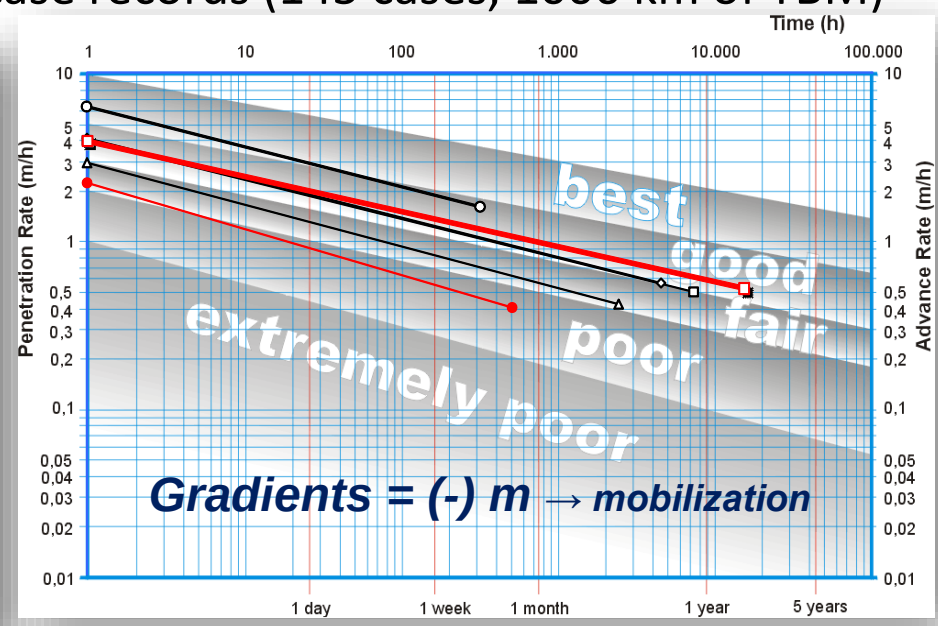


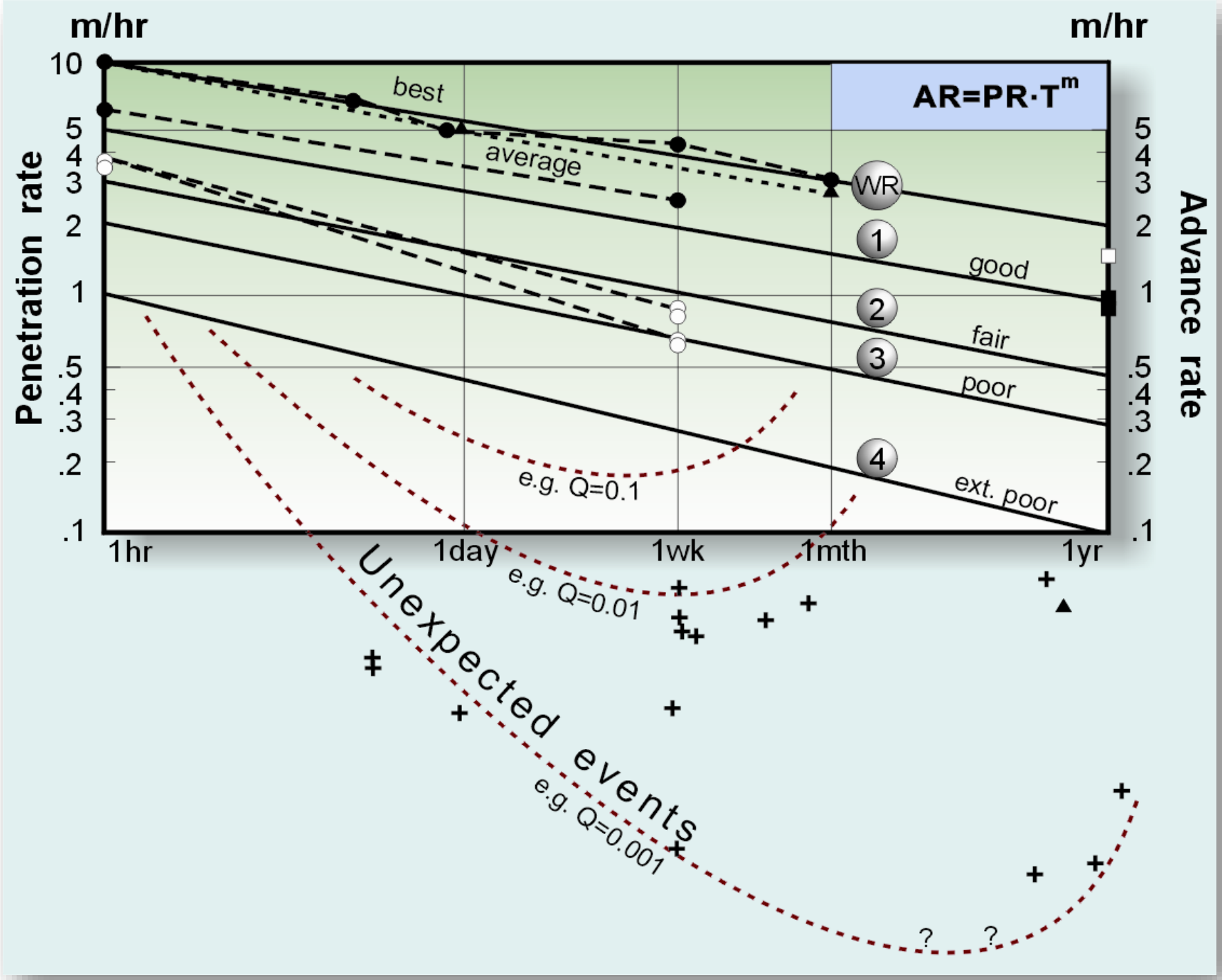
5	LITHOLOGY	Class 1 granitic gneiss	ZONE LENGTH	500

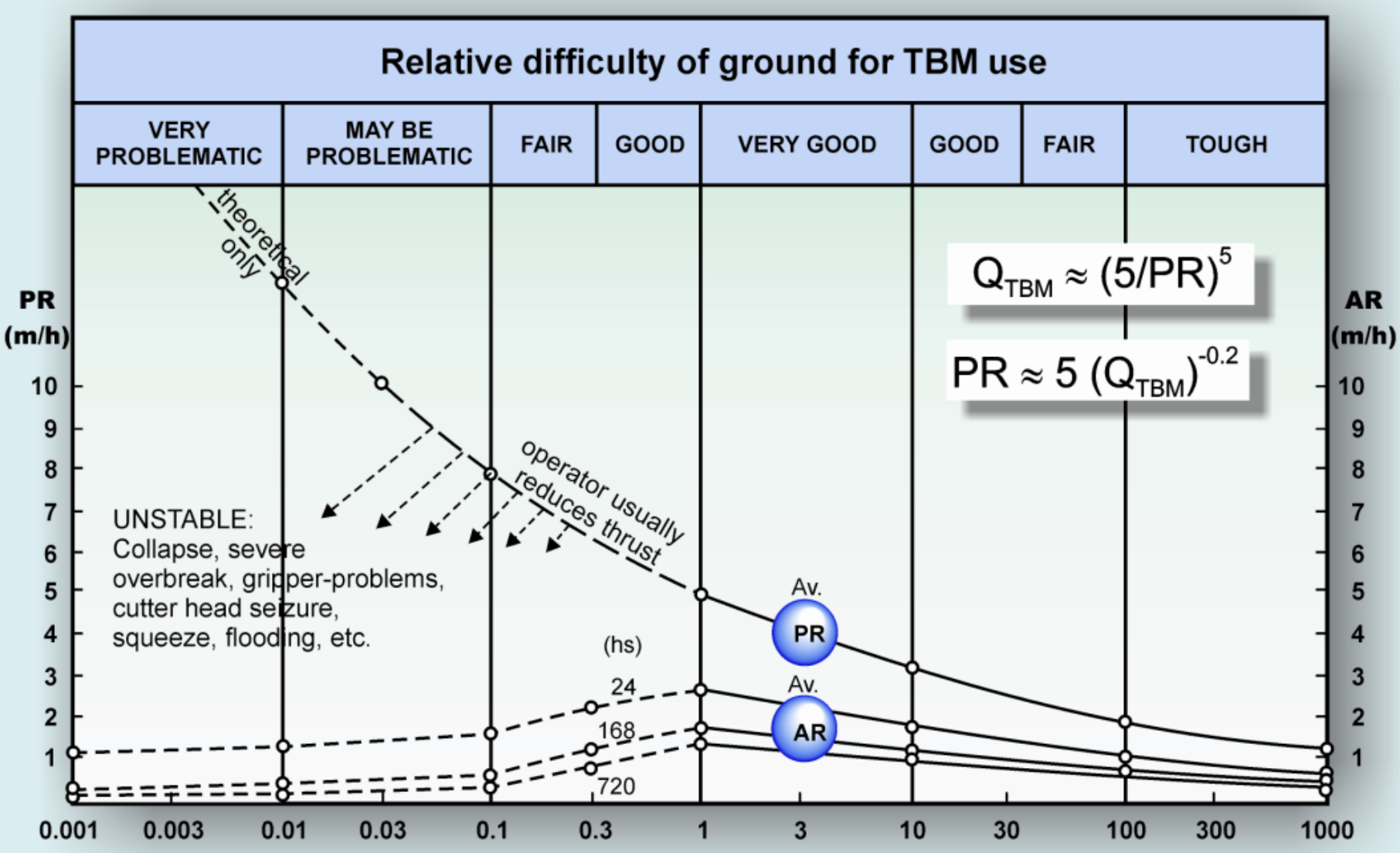
INPUT DATA

RQD	J_n	J_r	J_a	J_w	SRF	$-m_1$	RQD_0	γ (g/cm³)	V_P (km/s)
100.0	2.0	3.0	1.0	1.00	1.0	-0.19	100.0	2.8	

β°	σ_c (MPa)	I_{50} (MPa)	F (tf)	CLI	q %	σ_θ (MPa)	D (m)	n %
	250.0		32.0	5.0	35.0	8.0	10.0	1.0



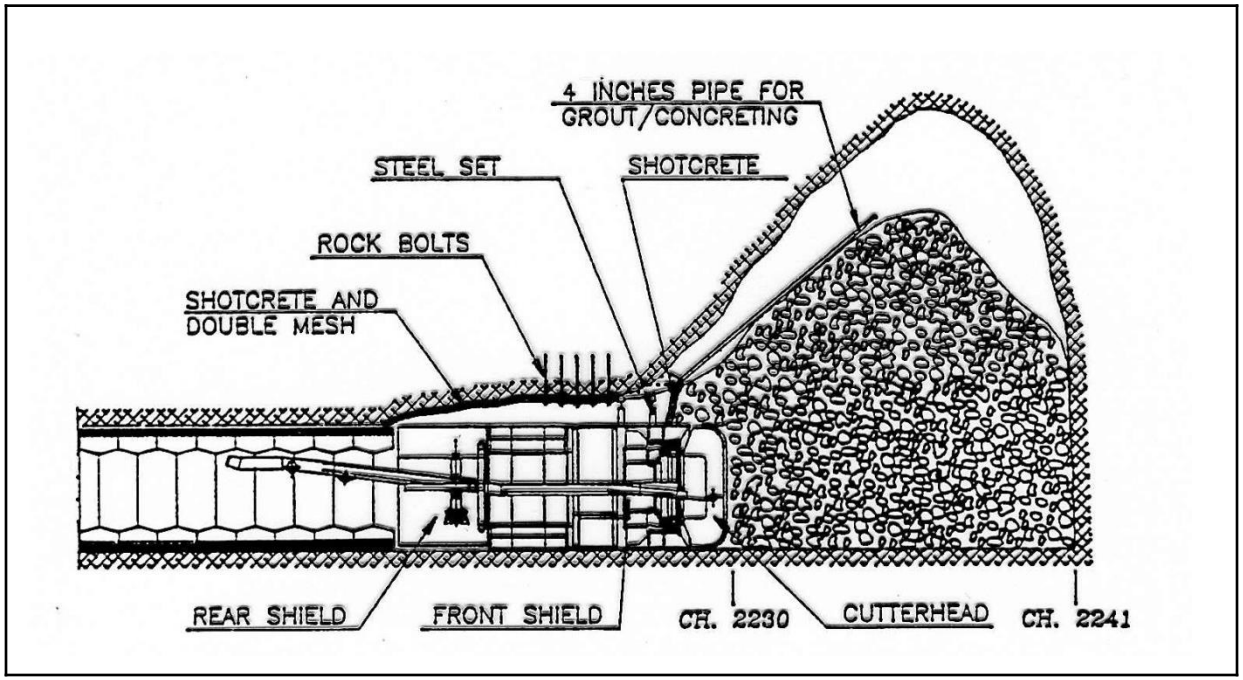

m/hr
m/hr
Penetration rate
Advance rate
AR=PR·T^m
best
average
WR
1
2
3
4
good
fair
poor
ext. poor
e.g. Q=0.1
e.g. Q=0.01
e.g. Q=0.001
Unexpected events
1hr
1day
1wk
1mth
1yr
10
5
4
3
2
1
.5
.4
.3
.2
.1



$$Q_{TBM} = \frac{RQD_o}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \times \frac{SIGMA}{F^{10}/20^9} \times \frac{20}{CLI} \times \frac{q}{20} \times \frac{\sigma_\theta}{5}$$

WHY TBM DELAYS IN FAULT ZONES ?

"Theo-empirical" reasons. Lack of belief gets paid for!



'THEO – EMPIRICAL' REASONS WHY FAULT ZONES ARE SO DIFFICULT FOR TBM

We need three basic equations:

1. ***AR = PR x U*** **(all TBM must follow this)**

2. ***$U = T^m$*** **(due to the decelerating advance rate with time)**

3. ***T = L / AR*** **(obviously time for length L must be proportional to 1/AR)**

Therefore we have the following:

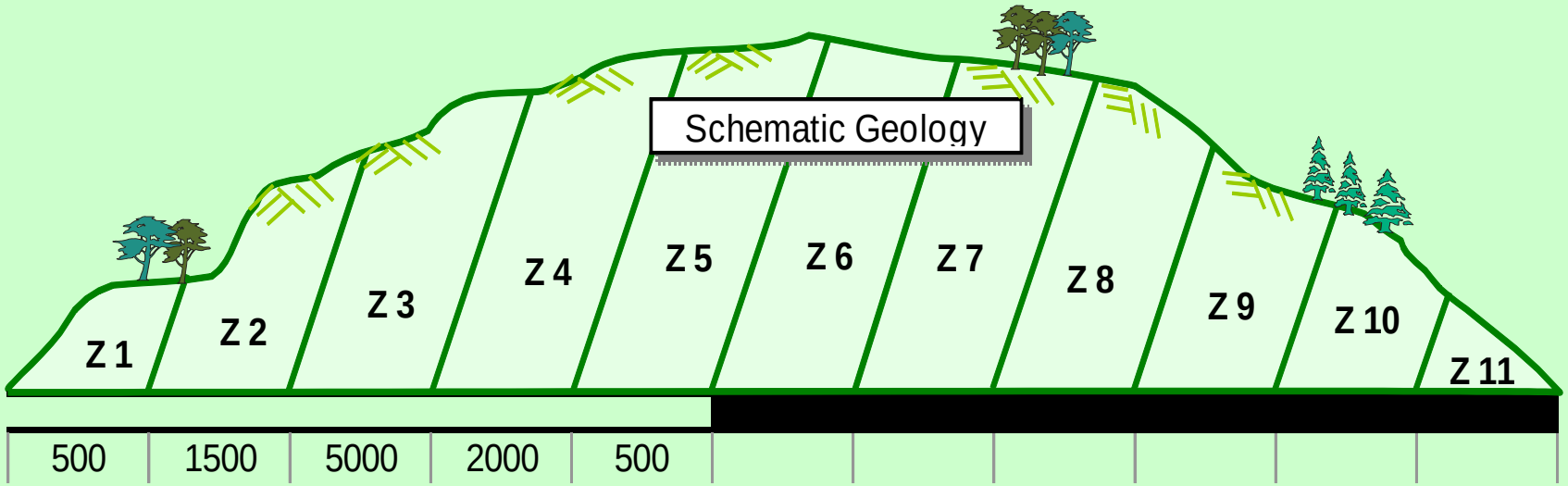
4. ***$T = (L / PR)^{(1 / 1+m)}$*** (from #1, #2 and #3)

5. This is VERY important for TBM……since m is strongly related to Q-values …..in FAULT ZONES.

6. It is important because very ***negative (-)m*** values make (1/(1+m) TOO BIG

BUT…Q CAN BE IMPROVED BY PRE-GROUTING !
(IMPROVE –m.....to less negative value)

Rock mass quality $$Q = \left(\frac{RQD}{J_n}\right) x \left(\frac{J_r}{J_a}\right) x \left(\frac{J_w}{SRF}\right)$$



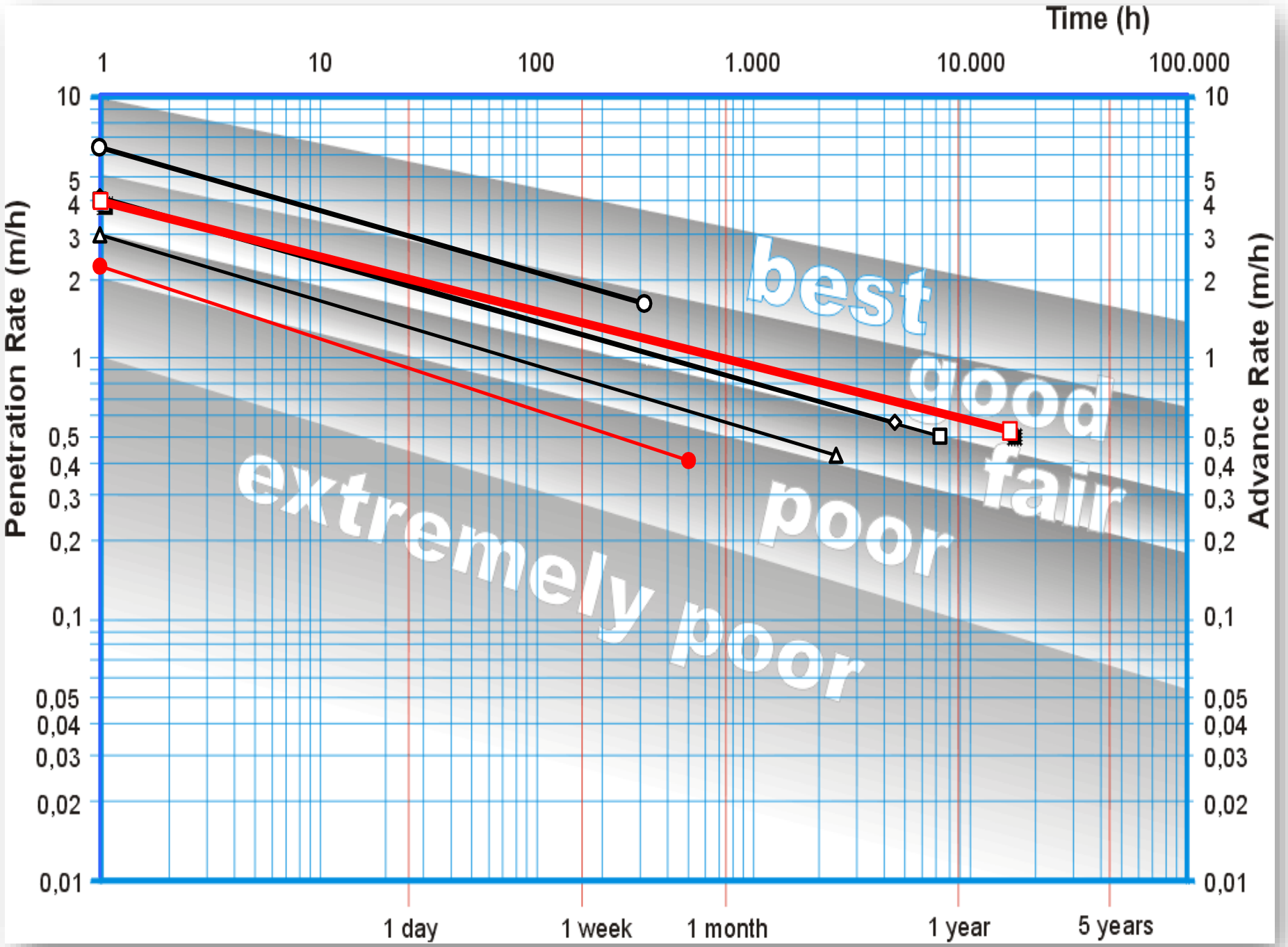
5	LITHOLOGY	*Class 1 granitic gneiss*	ZONE LENGTH	*500*

INPUT DATA

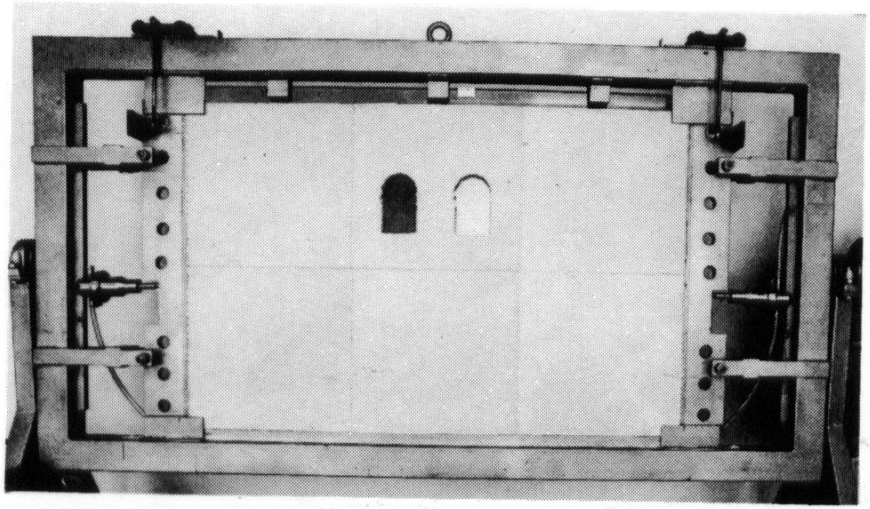
RQD	J_n	J_r	J_a	J_w	SRF	$-m_1$	RQD_0	γ (g/cm³)
100.0	2.0	3.0	1.0	1.00	1.0	-0.19	100.0	2.8

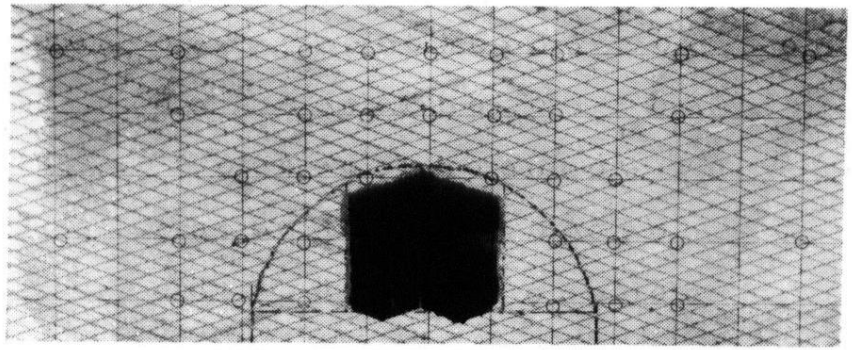
V_P (km/s)

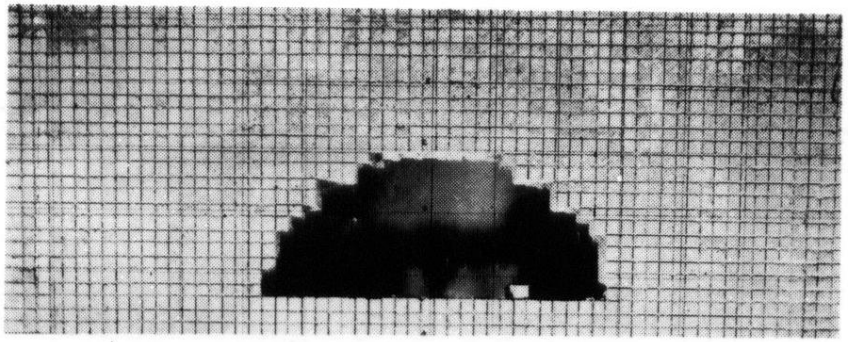
β^o	σ_c (MPa)	I_{50} (MPa)	F (tf)	CLI	q %	σ_θ (MPa)	D (m)	n %
	250.0		32.0	5.0	35.0	8.0	10.0	1.0

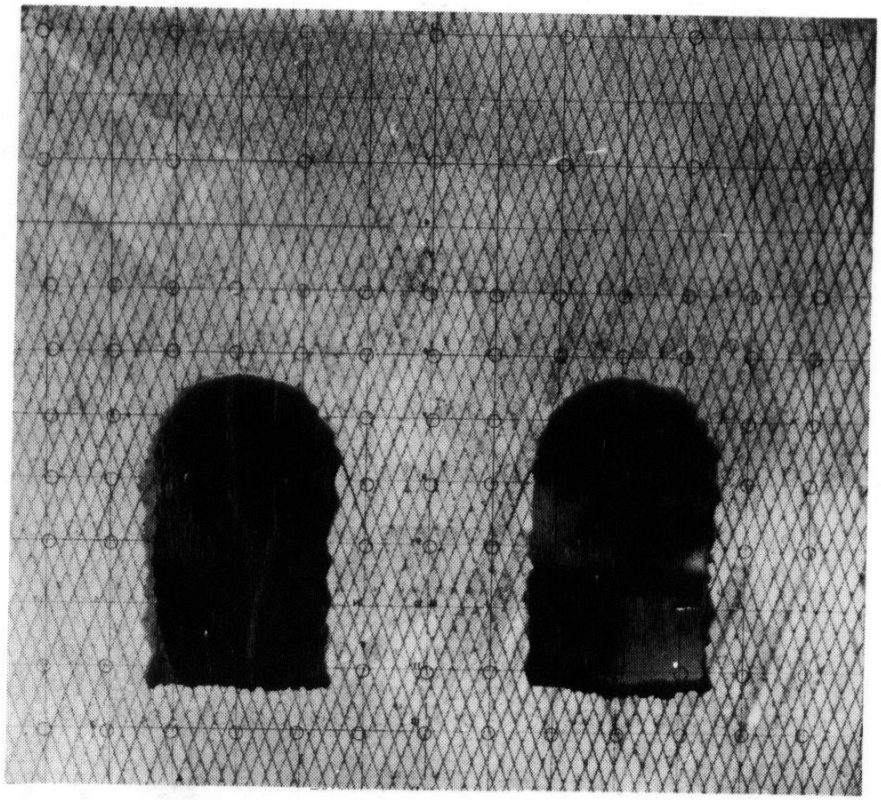

Time (h)
1
10
100
1.000
10.000
100.000
Penetration Rate (m/h)
Advance Rate (m/h)
10
5
4
3
2
1
0,5
0,4
0,3
0,2
0,1
0,05
0,04
0,03
0,02
0,01
best
good
fair
poor
extremely poor
1 day
1 week
1 month
1 year
5 years

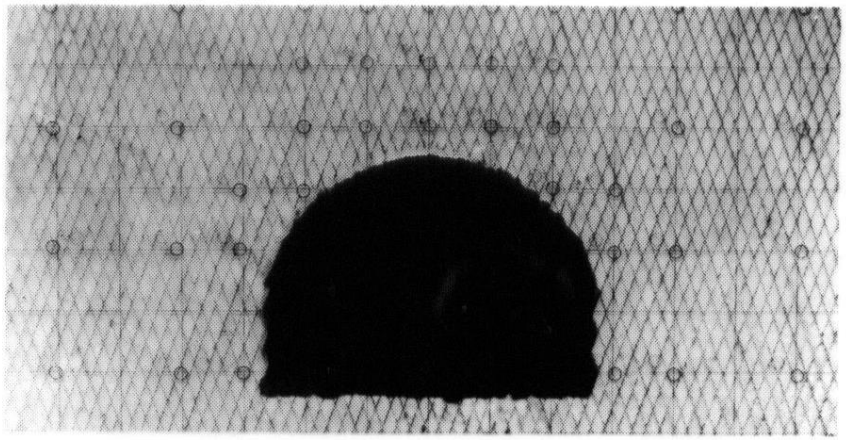
PHYSICAL (2D) MODELS of ROCK CAVERNS, AS FORE-RUNNER TO UDEC-BB FLEXIBILITY



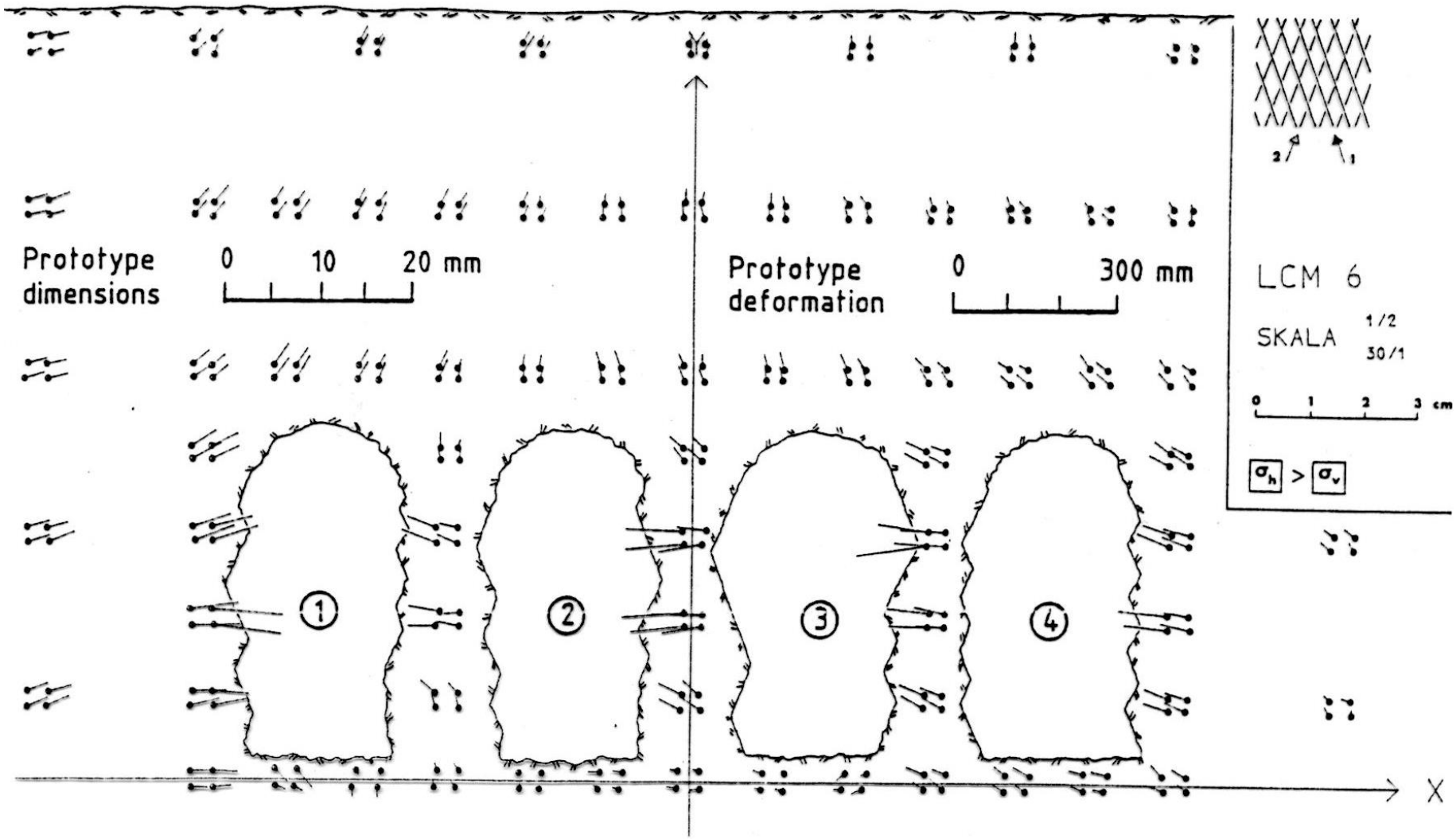




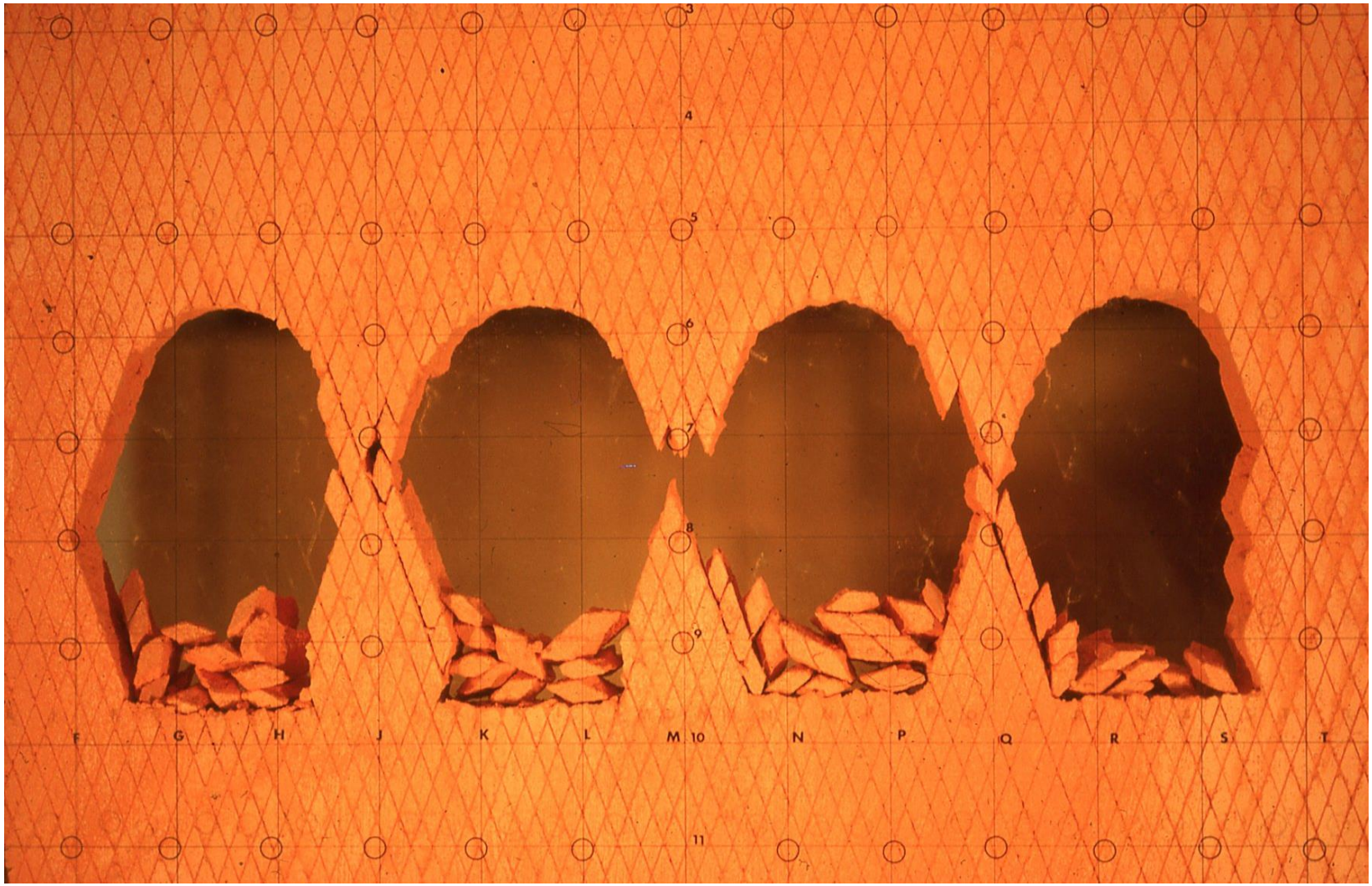


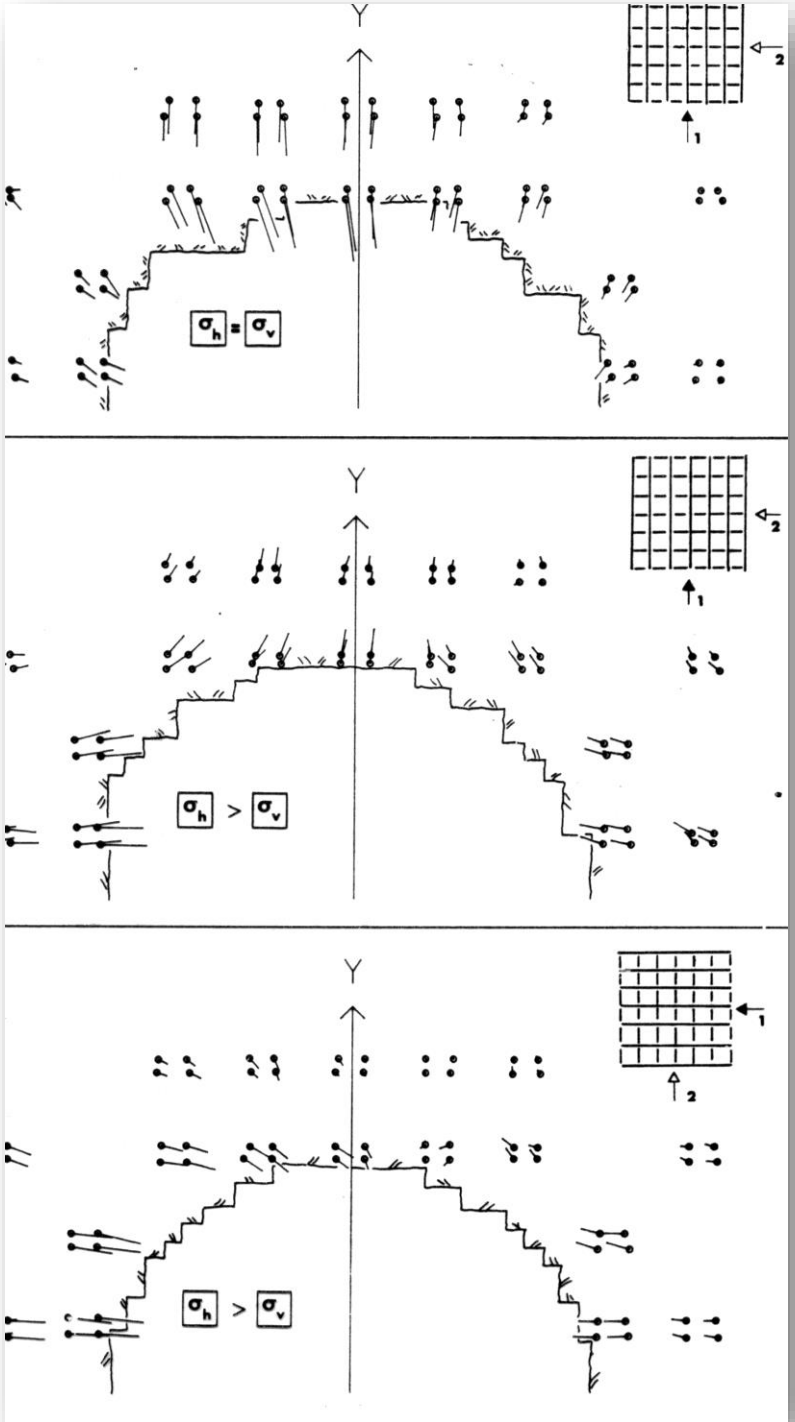


Fracture over-closure from previous stress state: new excavations do not reverse the deformation direction.



Post-seismic loading result (0.2 to 0.5 g)





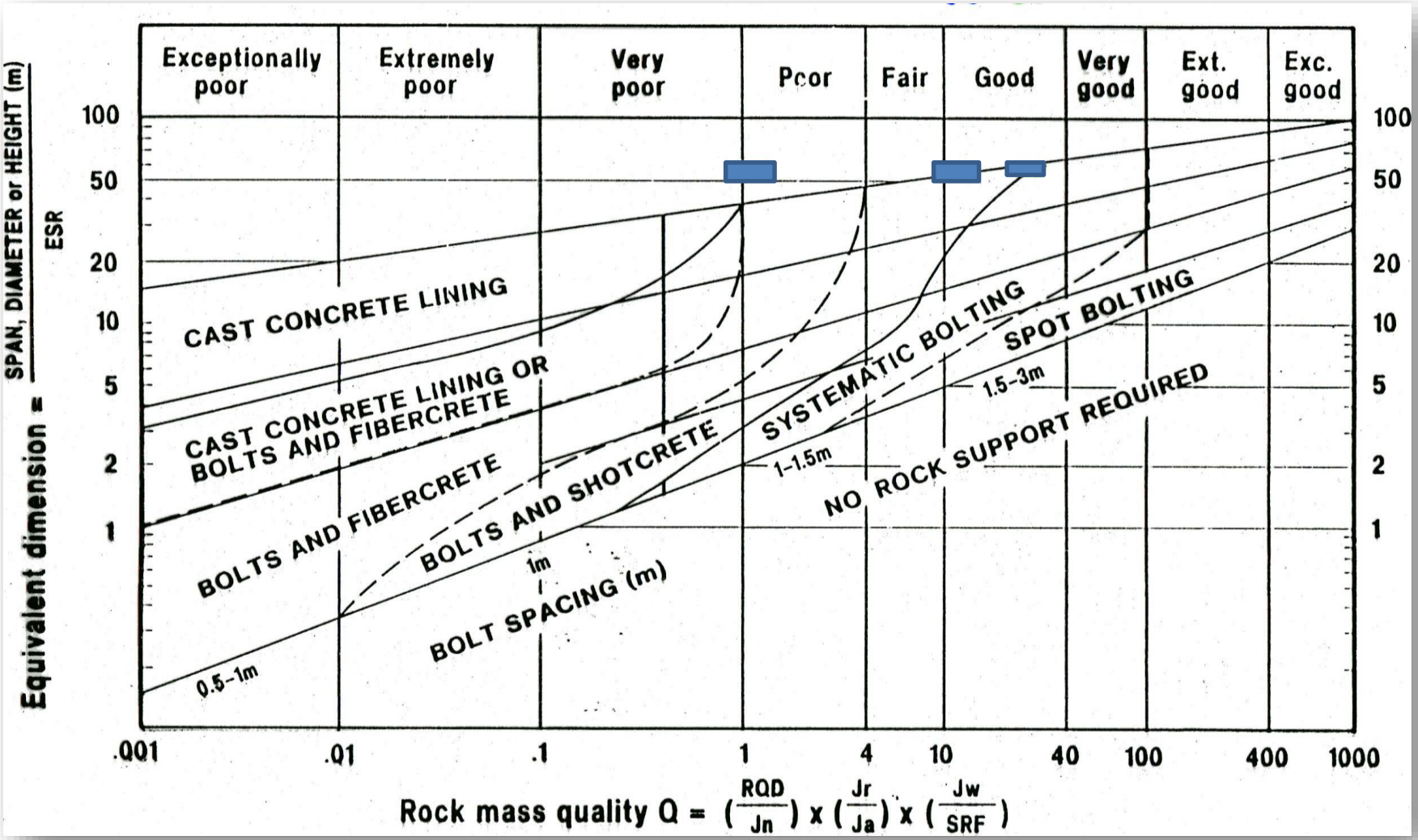
Physical and FEM modelling (Barton and Hansteen, 1979) suggested possible 'heave' resulting from large-cavern construction near the surface……..

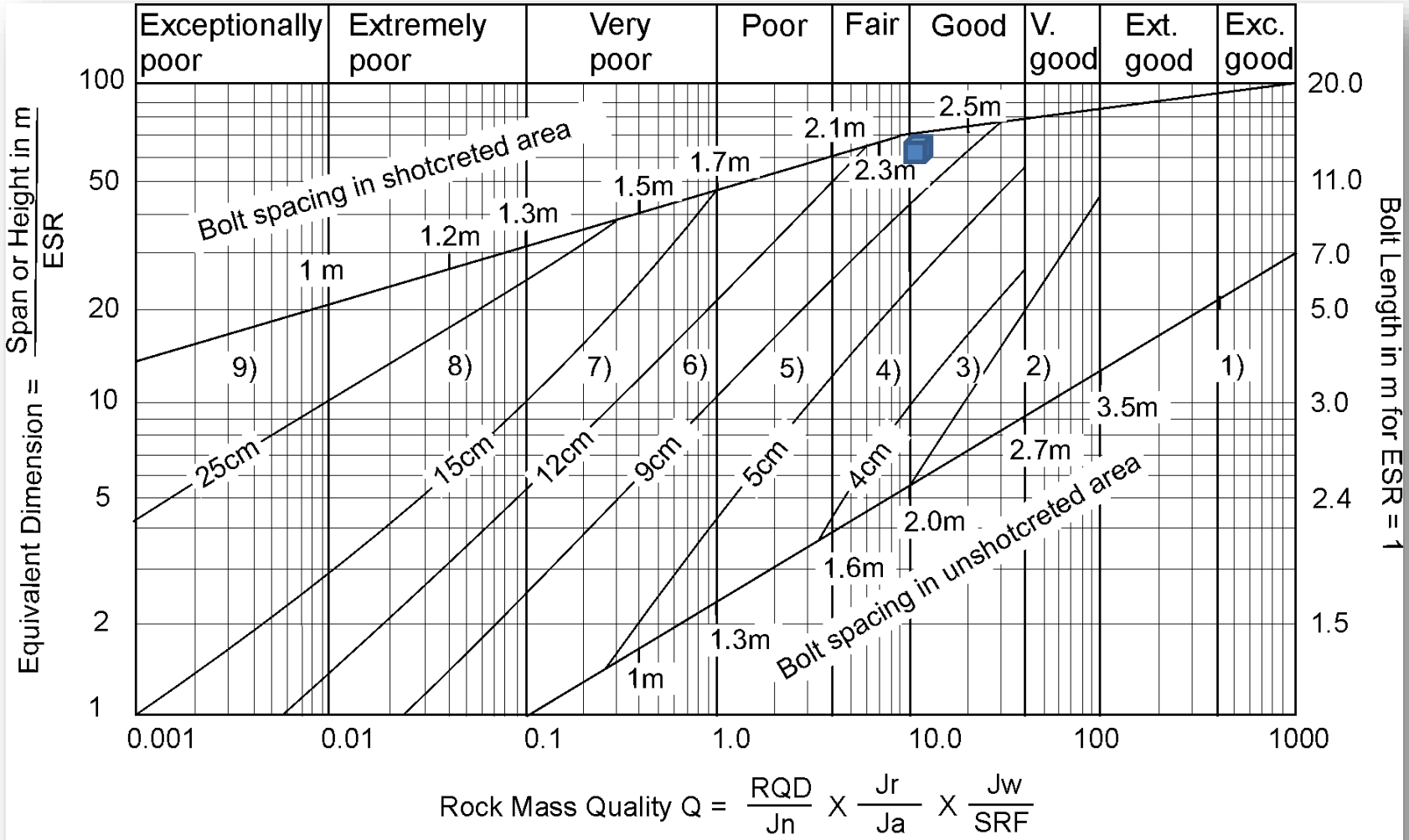
…….depended on joint pattern and horizontal stress level in the physical models.

GJØVIK CAVERN

INCREASE OF LARGEST CAVERN SPAN BY ALMOST 2 x

Gjøvik cavern : an 'extension' of 1974 Q-system data base.
(Q_{min}, Q_{mean}, and Q_{max} values of **1**, **12**, **30** logged in the cavern)
RQD = 60-90%, UCS = 90 MPa was typical.



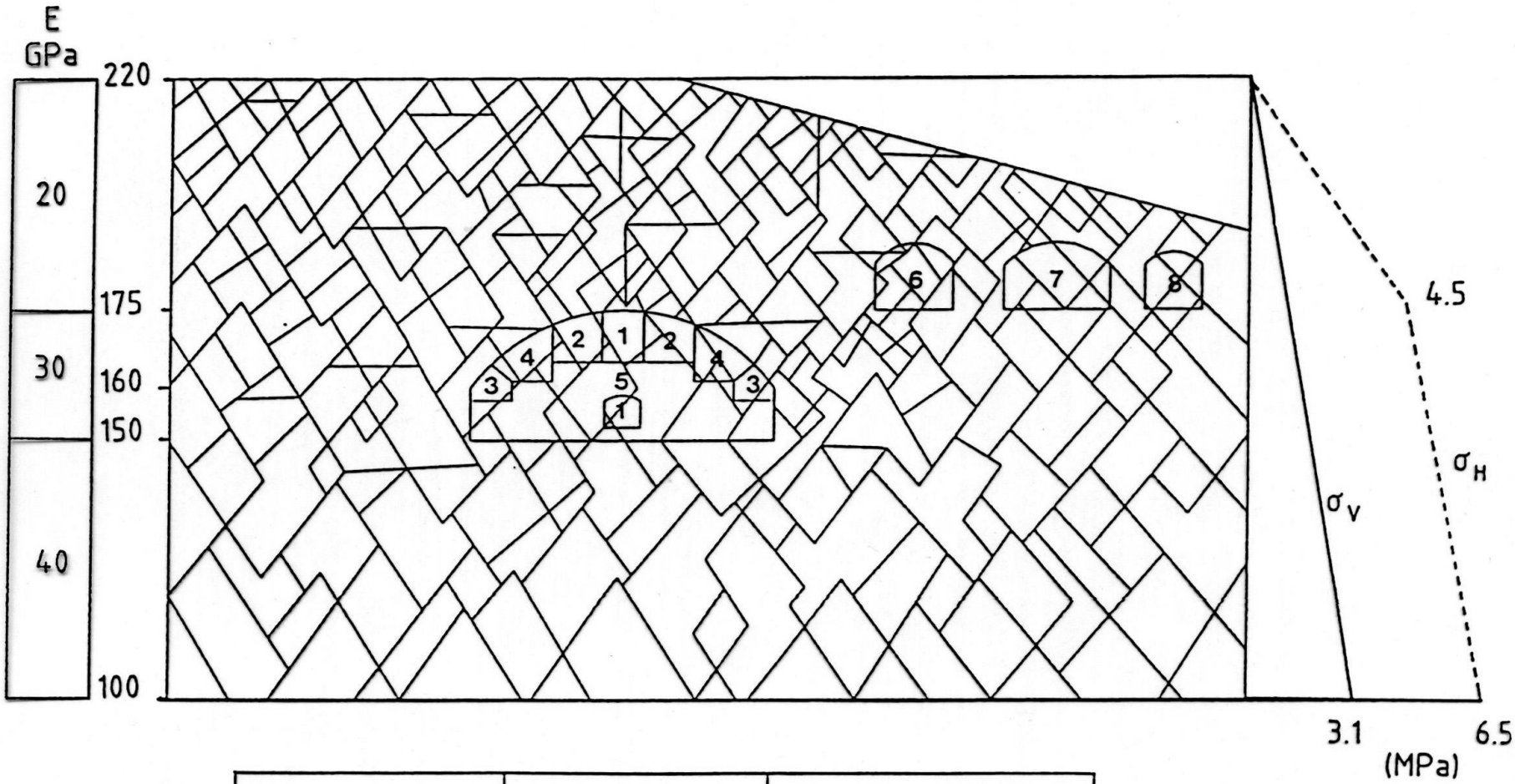


REINFORCEMENT CATEGORIES

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

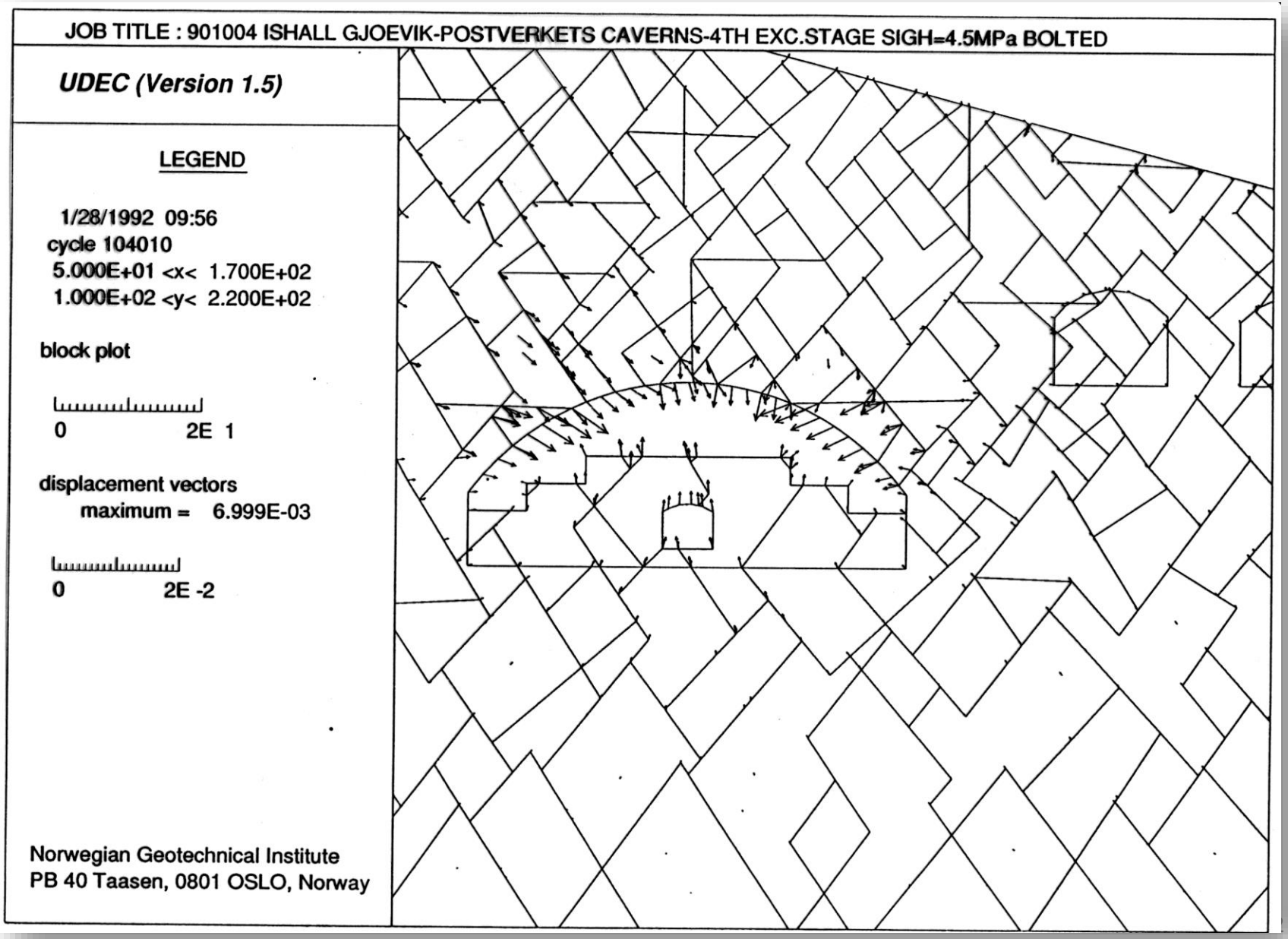
GJØVIK CAVERN JOINT-GEOMETRY ASSUMPTIONS
input data, boundary stresses

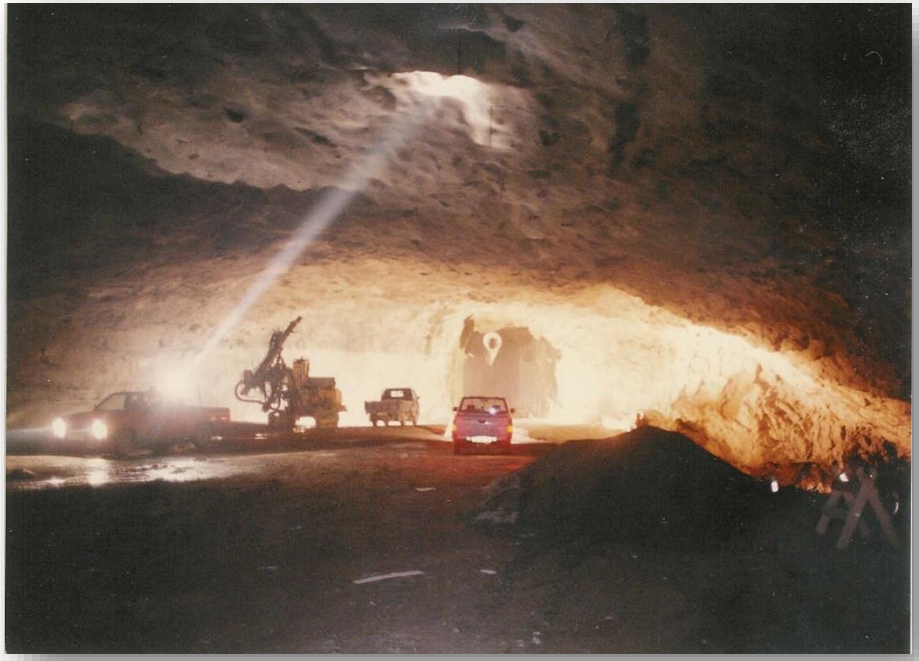
Barton, N., By, T.L., Chryssanthakis, P., Tunbridge, L., Kristiansen, J., Løset, F., Bhasin, R.K., Westerdahl, H. & Vik, G. 1994. Predicted and measured performance of the 62m span Norwegian Olympic Ice Hockey Cavern at Gjøvik. Int. J. Rock Mech, Min. Sci. & Geomech. Abstr. 31:6: 617-641. Pergamon.

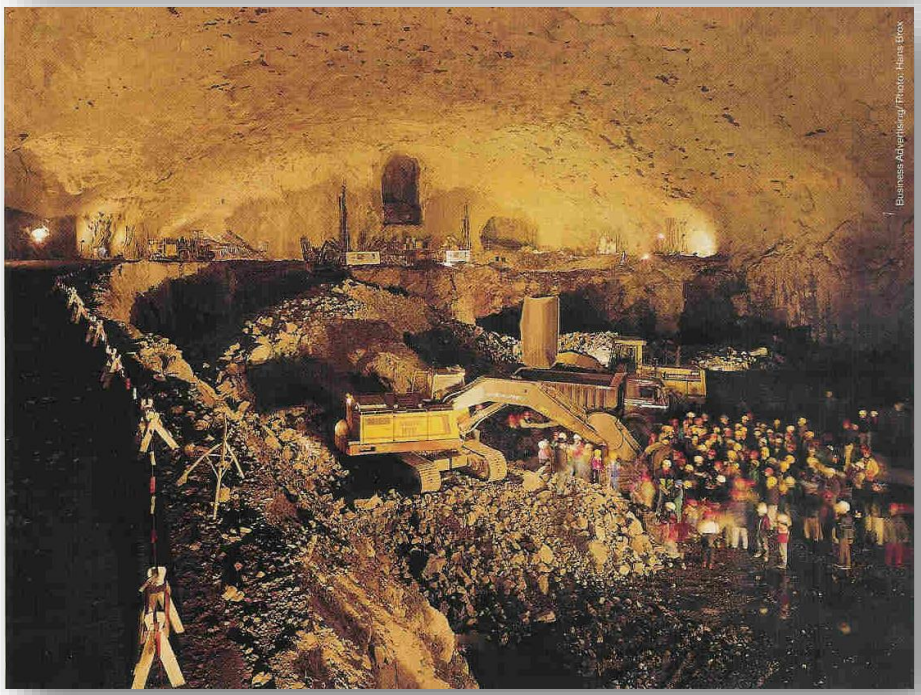


γ_z=0.026MPa/m	JRC_0=7.5 JCS_0=75MPa	Φ_r=27° i=6° } 33°

TOP HEADING TOO WIDE TO OBSERVE FROM ONE LOCATION

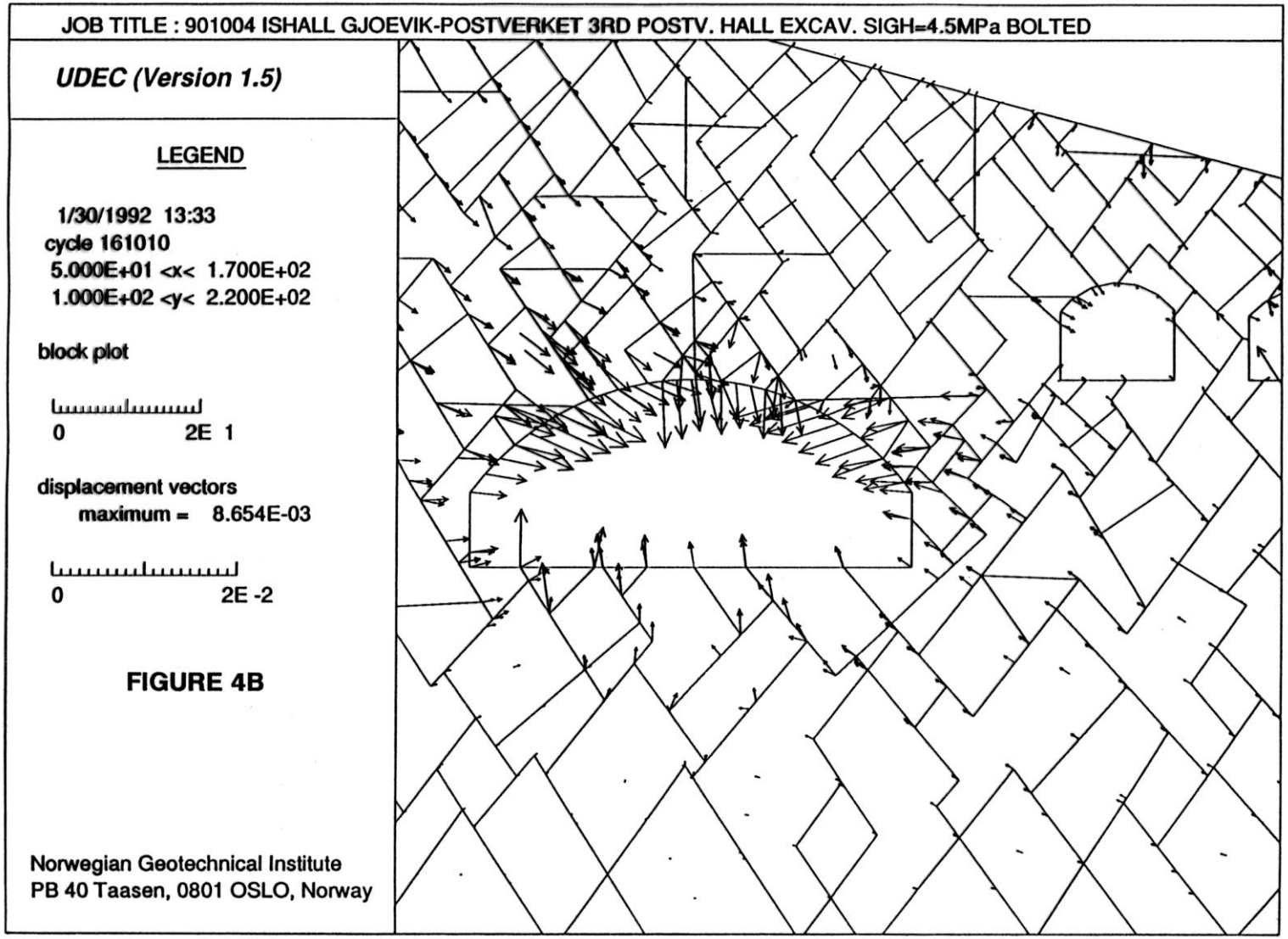




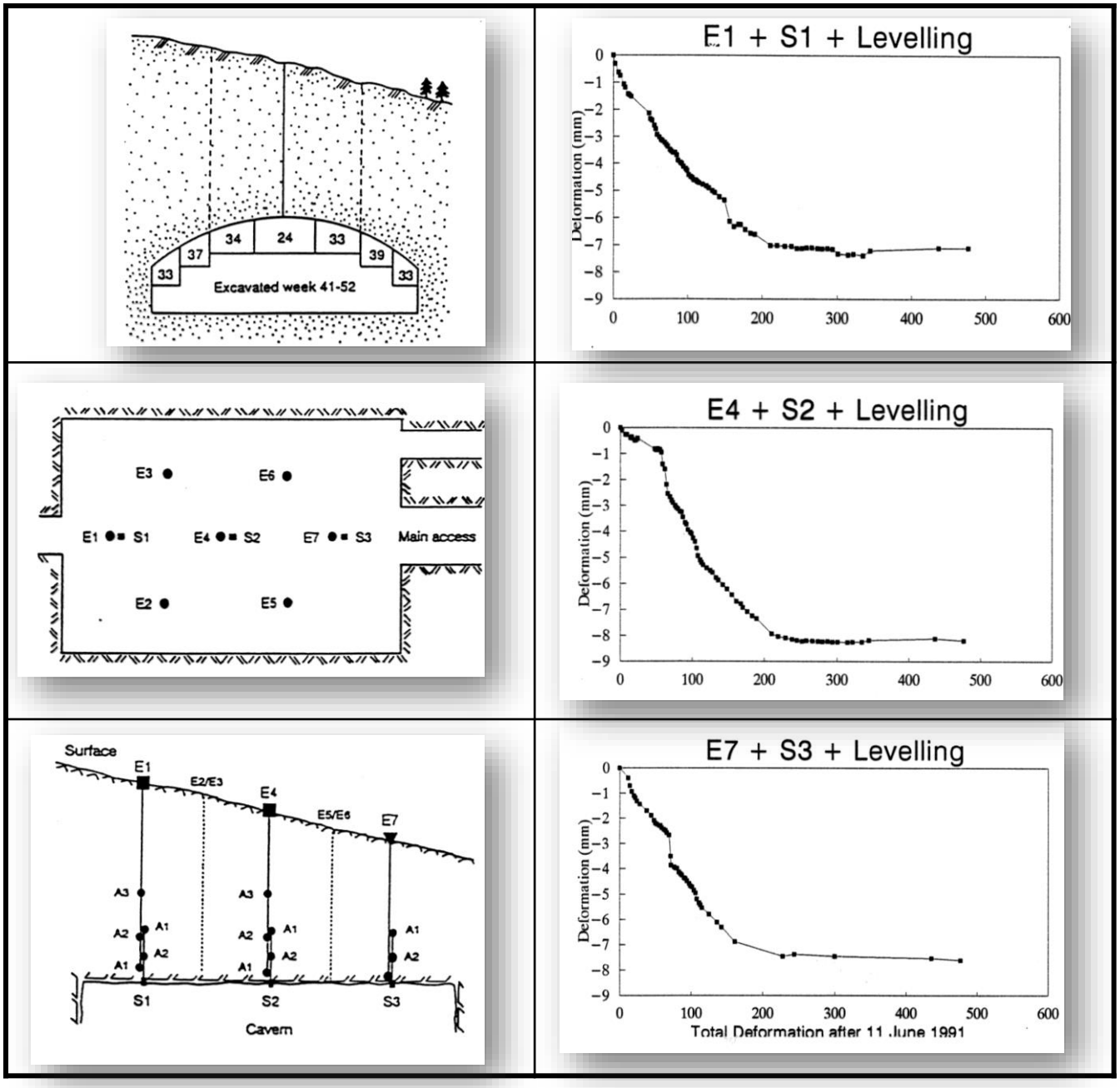

Business Advertising/Photo: Hans Brox

The final modelled 7 to 9 mm (downwards directed) deformations matched the unknown (to be measured) result almost perfectly.

(UDEC-BB modelling by Chryssanthakis, NGI)



DEFORMATION RECORDS FROM MPBX AND LEVELLING

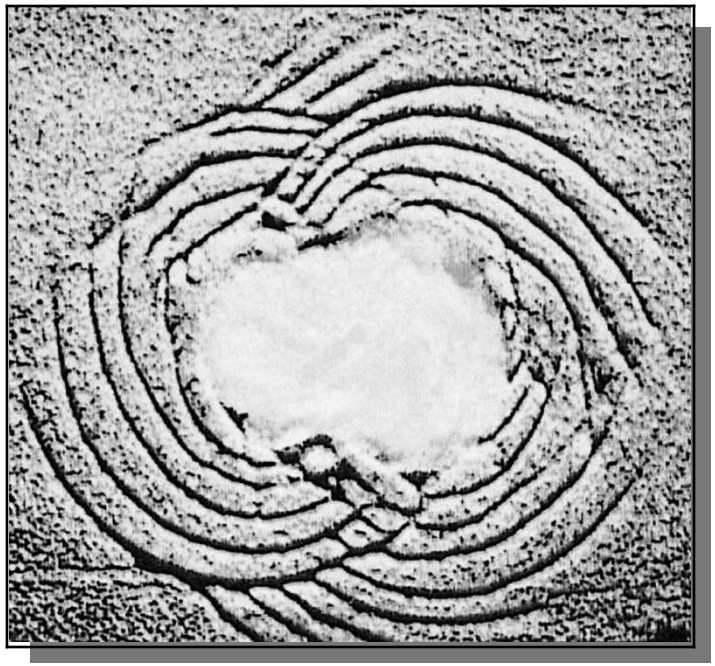


Δ = 7 to 8 mm was typical.

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

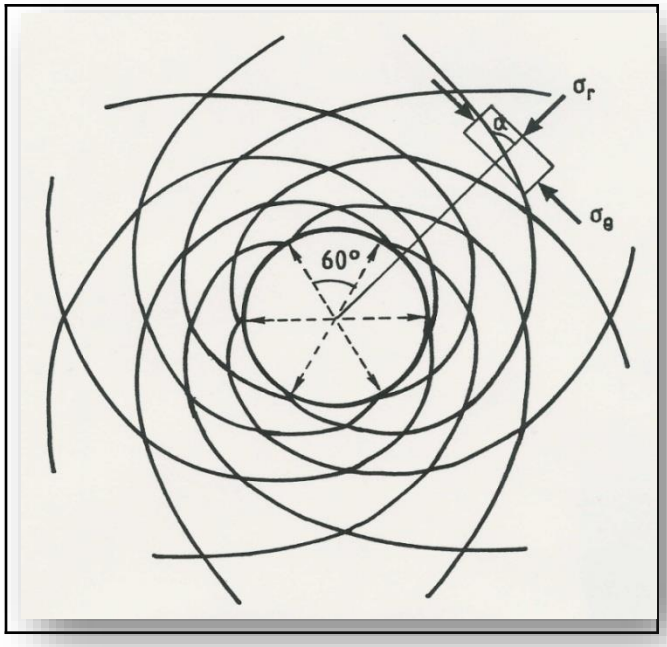
B x H x L
= 62 x 24 x 90
= 140,000 m3

CONTINUUM (??) MODELLING



Borehole stability studies at NGI.

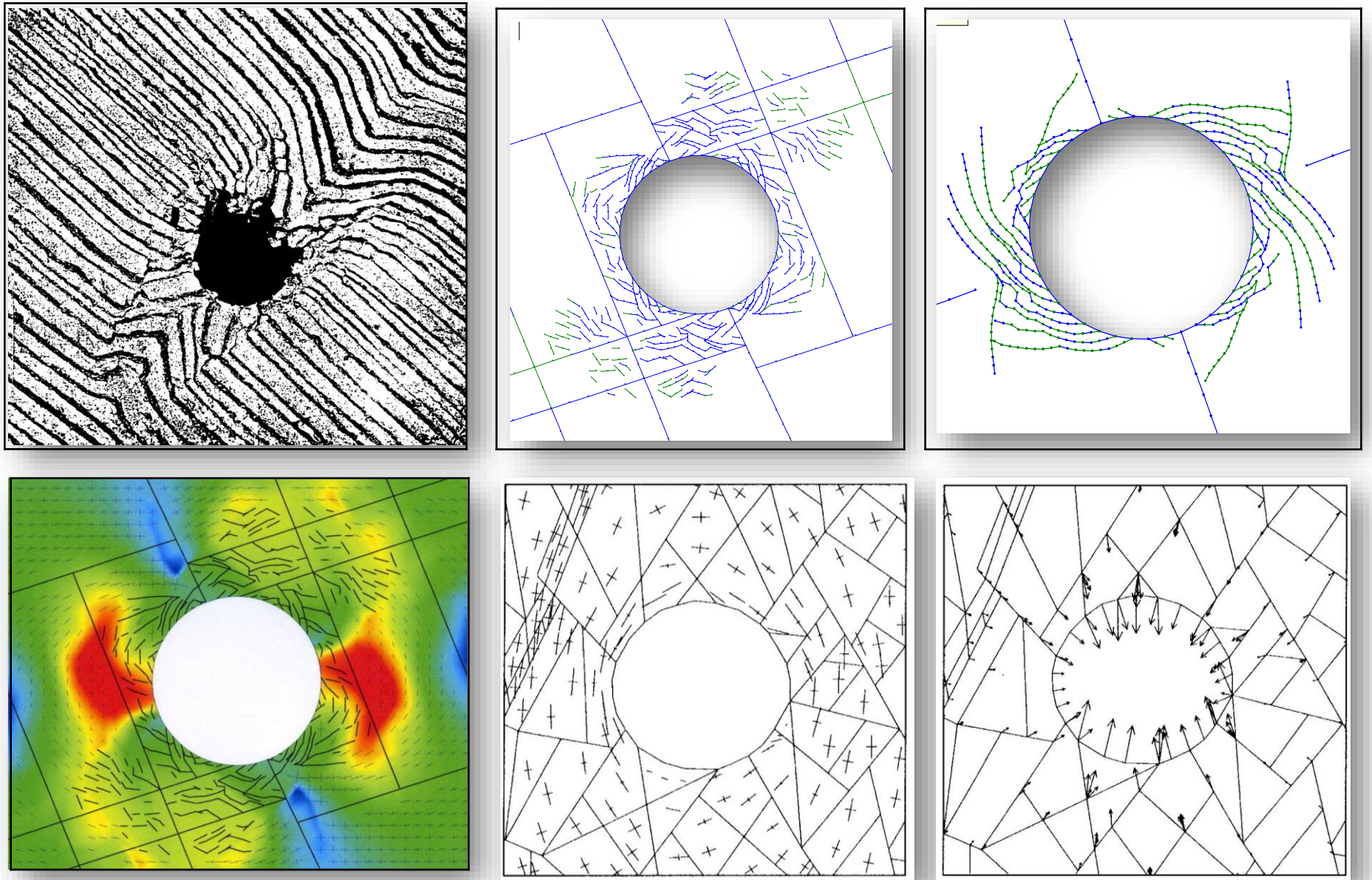
(Joint Industry Project). Addis et al., SPE, 1990.



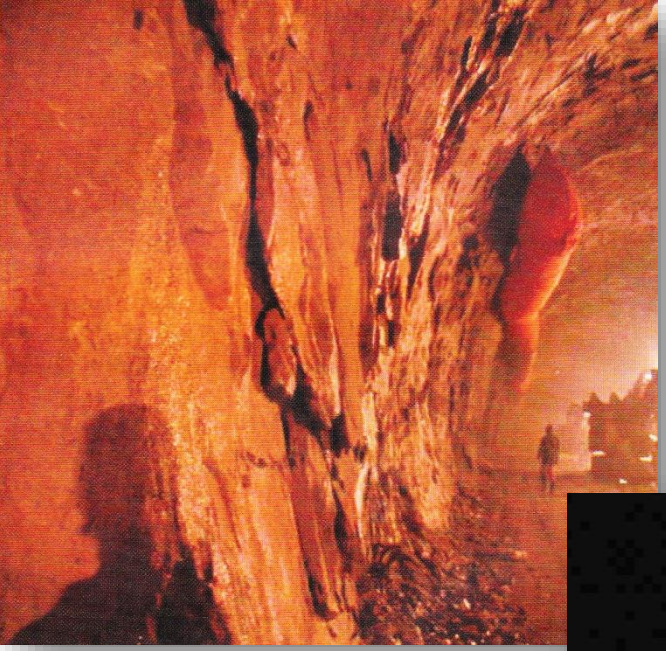
Drilling into $\sigma_1 > \sigma_2 > \sigma_3$ loaded cubes

0.5 x 0.5 x 0.5 m

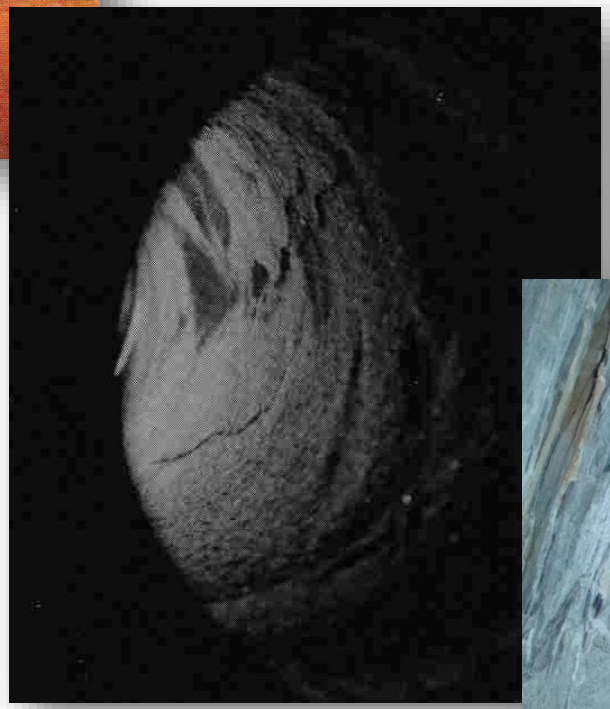
of model sandstone



Physical model: layered, by Bandis 1987, three **FRACOD** models by Baotang Shen, 2004, two **UDEC-BB** models, Hansteen, NGI, 1991.

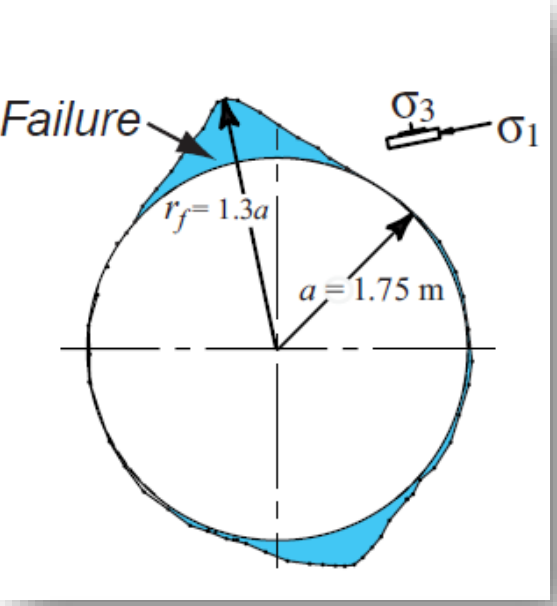


Jinping II (D+B) – ISRM News Journal
Physical model – bored under stress (NGI)
Jinping II (TBM) – ISRM workshop (NB)



Log-spiral shear modes

NEED for CHANGE

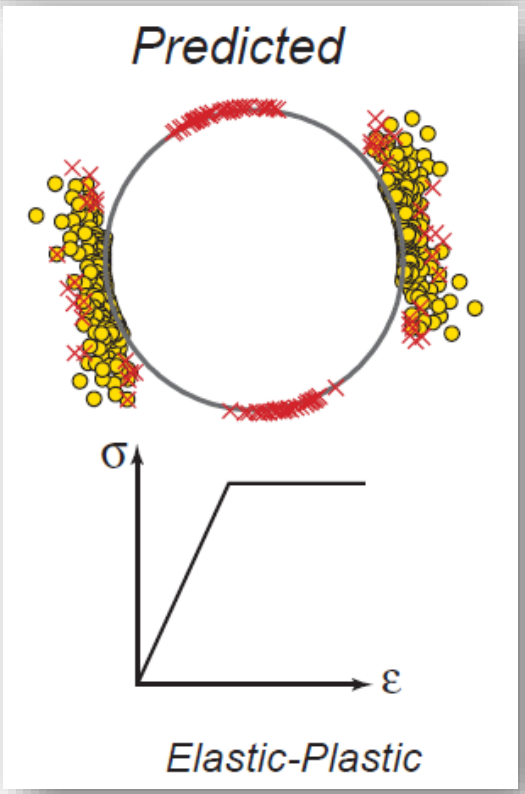


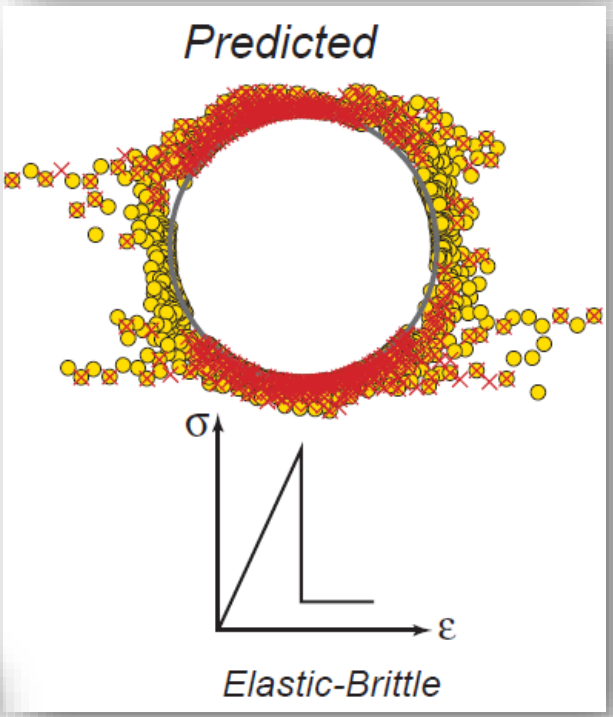
CONVENTIONAL continuum modelling methods.

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

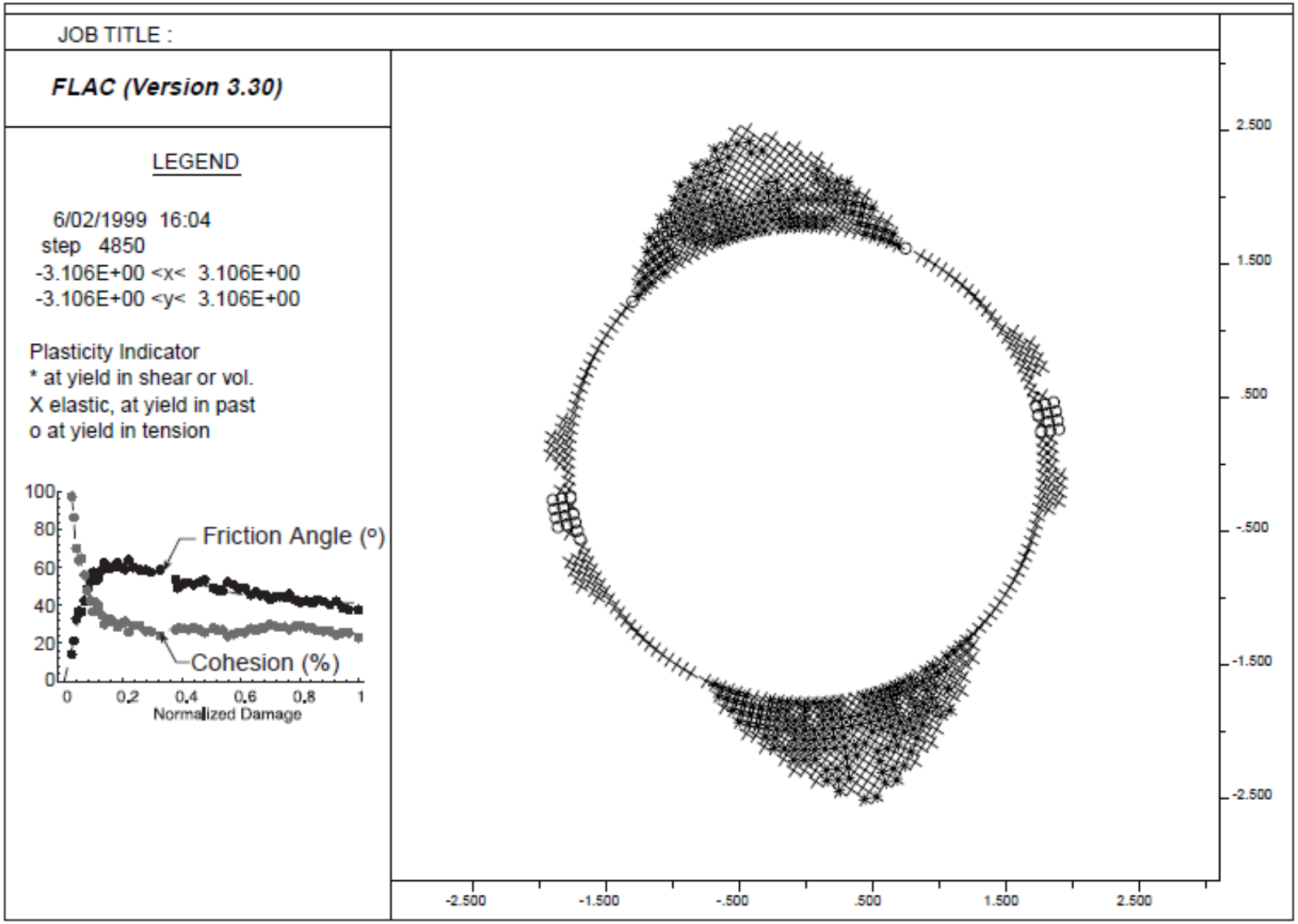
(Hajiabdolmajid, Martin and Kaiser, 2000 "Modelling brittle failure", NARMS.)

So why performed by so many consultants?





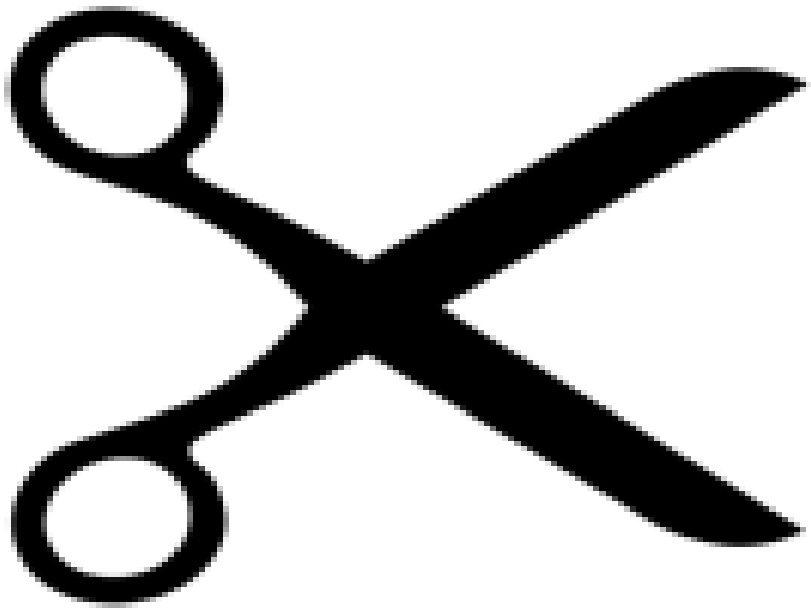
× Shear failure ○ Tensile failure



Degrade cohesion, mobilize friction: excellent match.

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

WHY SCISSORS ?



CC and FC from Qc:

Qc = Q x UCS/100)

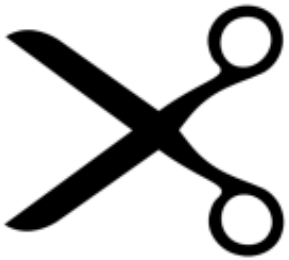
= cohesive strength ***(the component of the rock mass requiring shotcrete)***

x frictional strength ***(the component of the rock mass requiring bolting).***

Cut Q_c into two halves →'c' and 'φ' !?

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

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



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

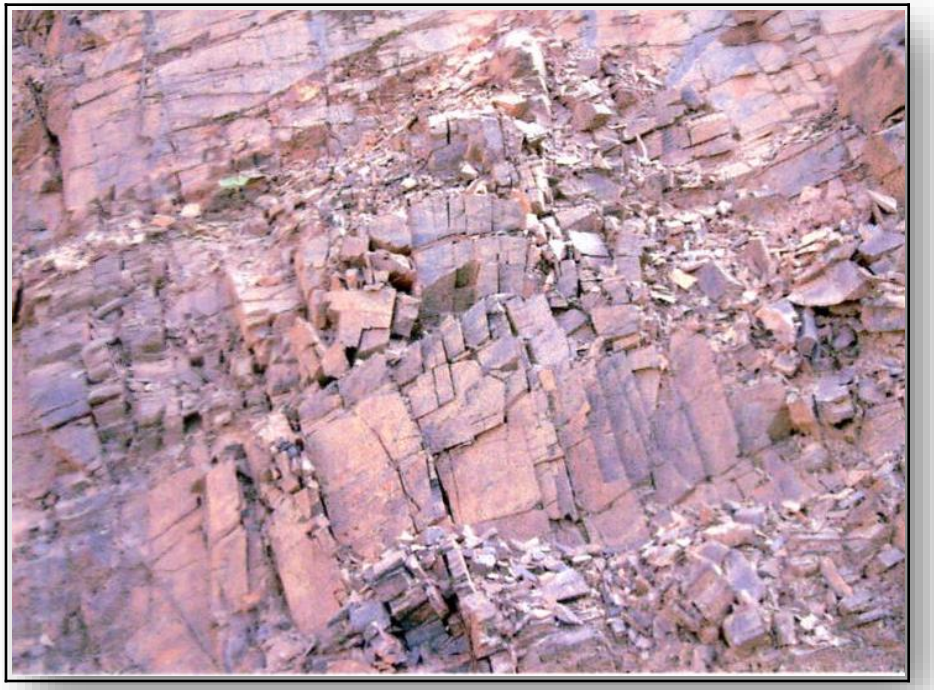
Q-based 'empiricism'

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

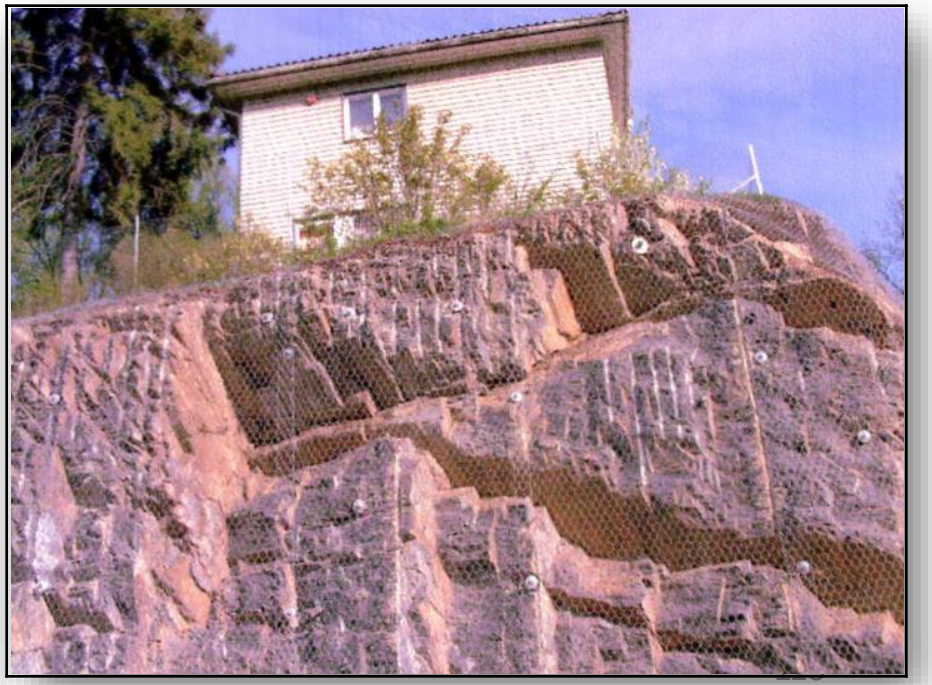
RQD	J_n	J_r	J_a	J_w	SRF	Q	σ_c	Q_c	*FC°*	*CC* MPa	V_p km/s	E_{mass} GPa
100	2	2	1	1	1	100	100	100	63°	50	5.5	46
90	9	1	1	1	1	10	100	10	45°	10	4.5	22
60	12	1.5	2	0.66	1	2.5	50	1.2	26°	2.5	3.6	10.7
30	15	1	4	0.66	2.5	0.13	33	0.04	9°	0.26	2.1	3.5

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

Low CC –shotcrete preferred



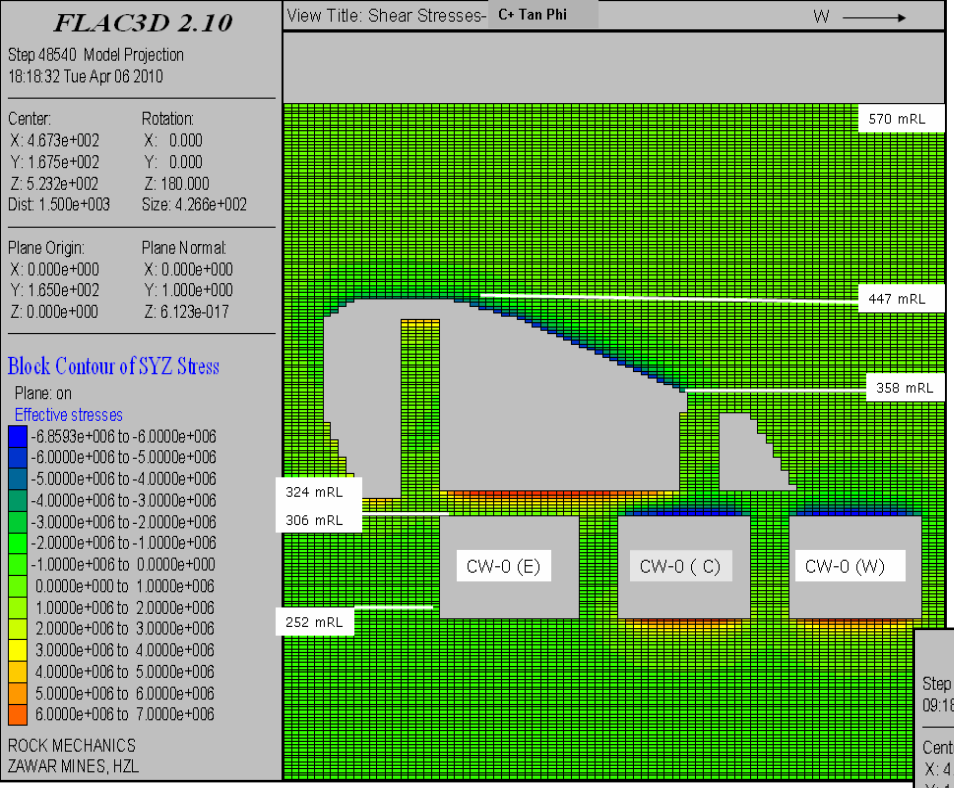
Low FC – bolting preferred

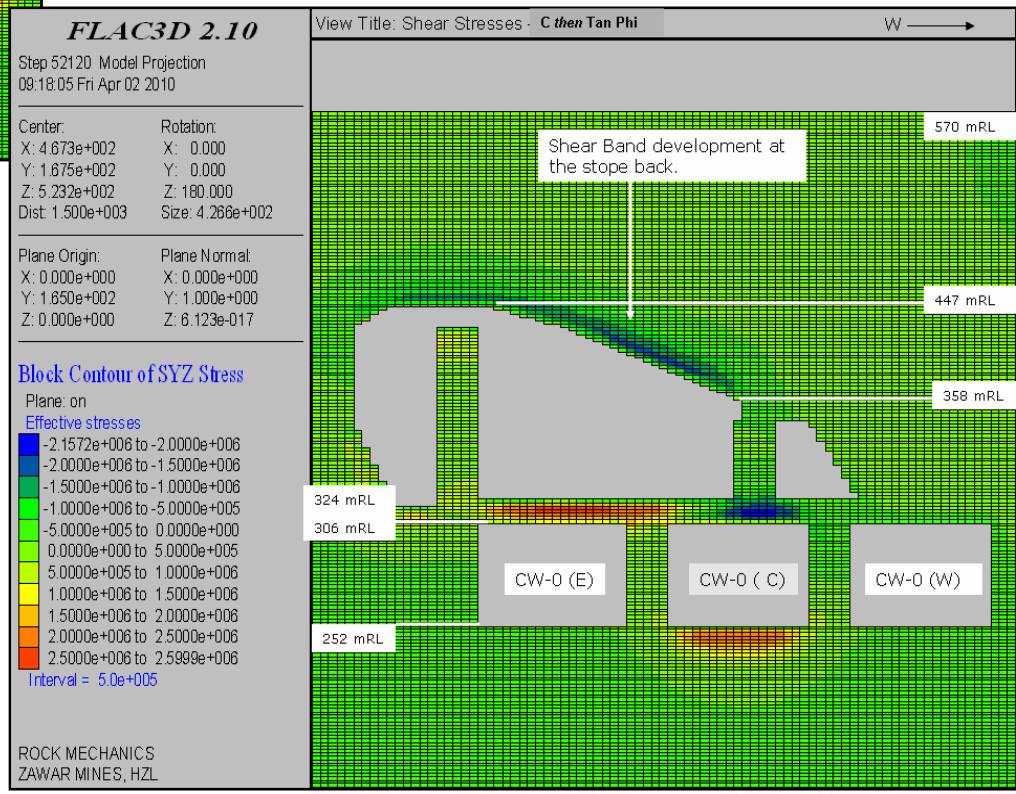


FLAC 3D

‘c+ tan ϕ’ (left)
‘c *then* tan ϕ’ (below)

(Barton and Suneet Pandey, 2011)





‘New’ approaches:

- ❑ **c *then* tan ϕ (not new, but rare!)**
- ❑ **Comparing modelled and measured displacements with pre-installed *MPBX*.**
- ❑ **Back-calculating Q from empirical Δ equations, as well as logged Q.**

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

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

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

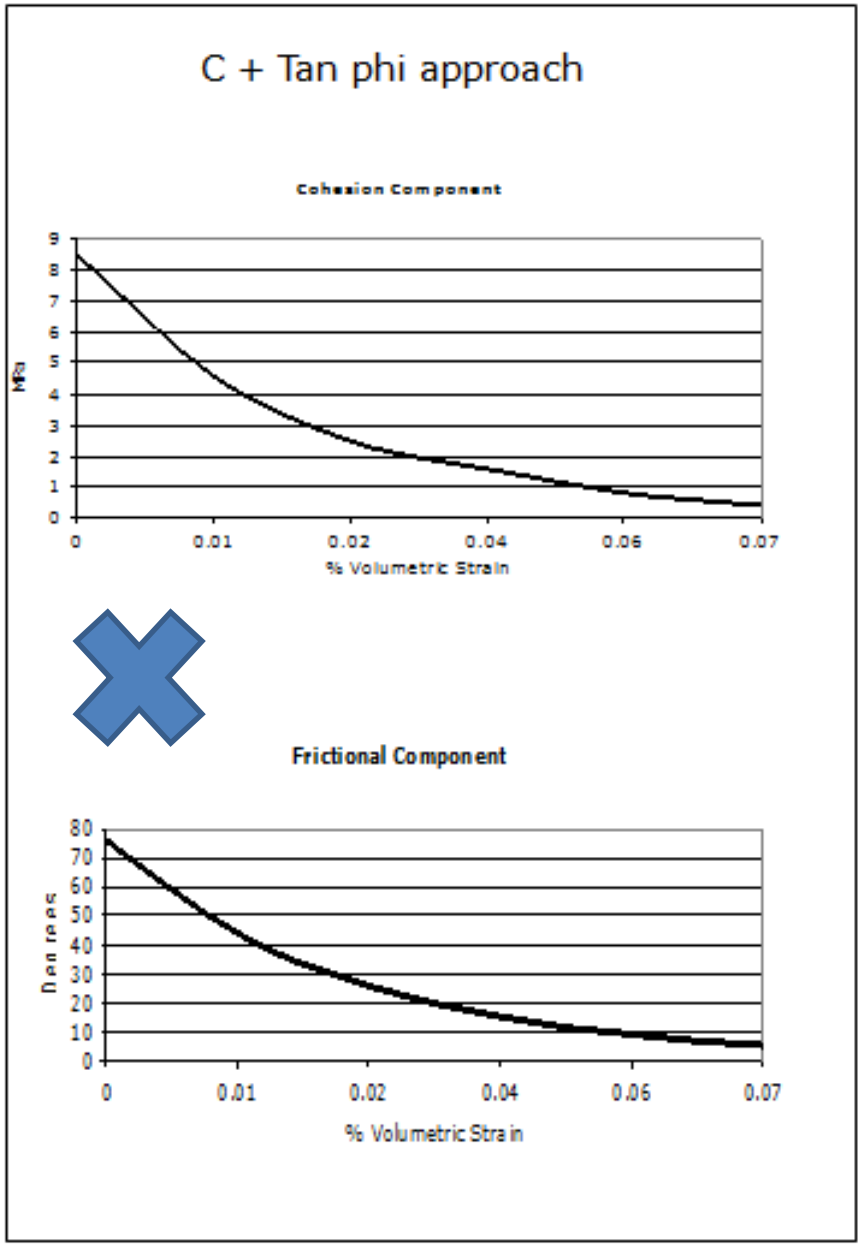
Units:

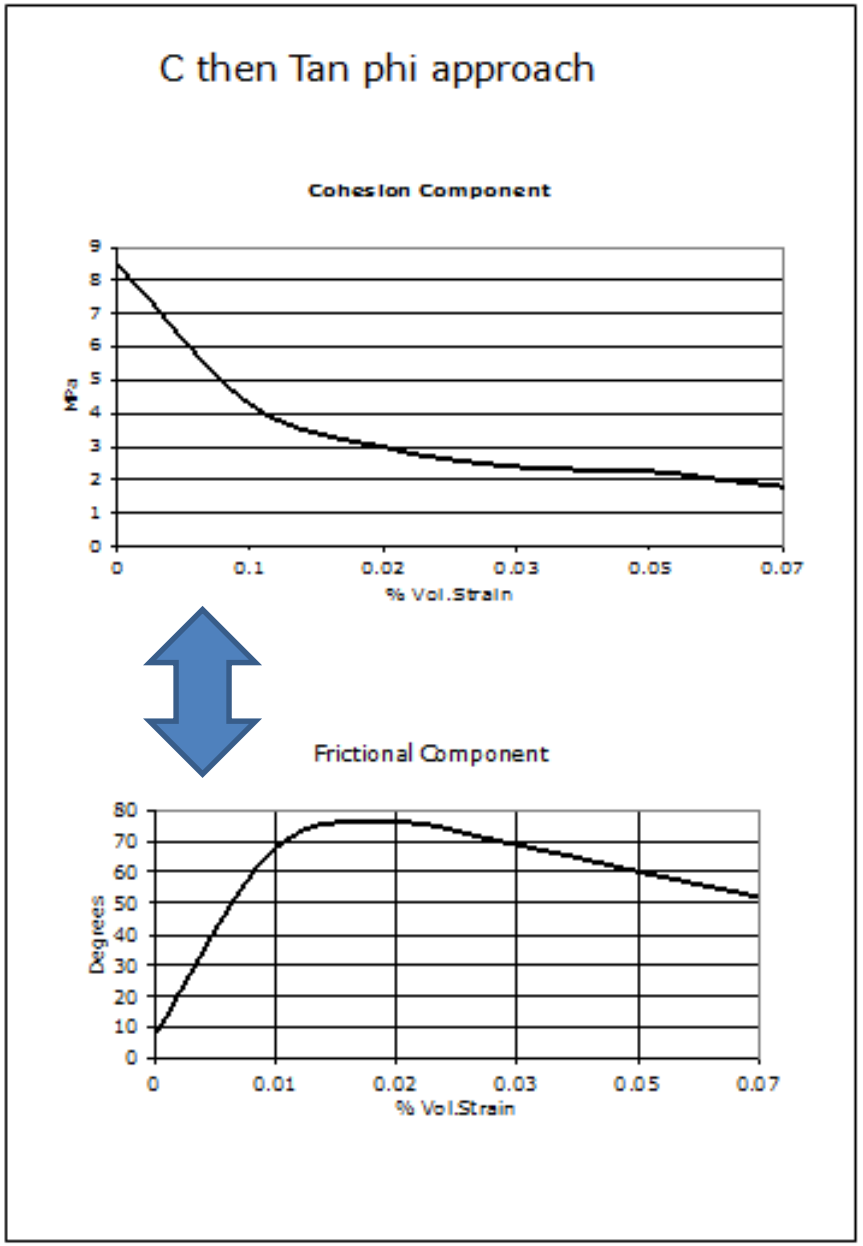
SPAN, HEIGHT, Δ_v and Δ_h (millimetres)

Rock stresses and rock strengths (MPa).

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

‘C *then* tan phi’ (as used in Barton and Pandey, 2011)





Please notice the nice dry NMT tunnel (= single-shell) in pre-injected shales @ < 20,000 US $/meter.