LESSONS LEARNED USING EMPIRICAL METHODS APPLIED IN MINING

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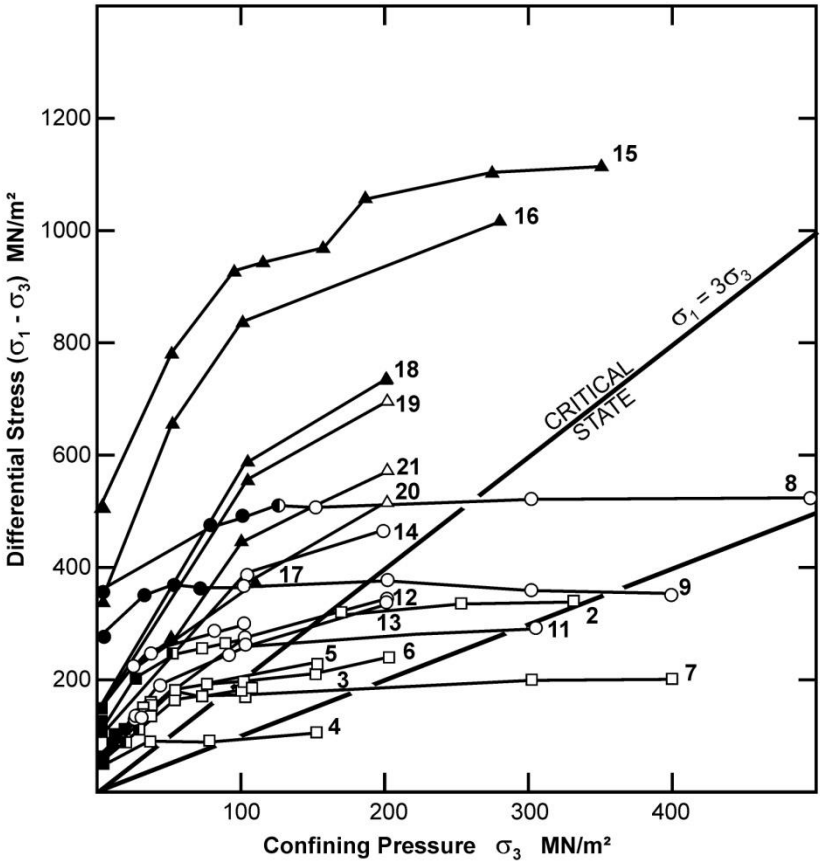
EMPIRICAL DESIGN METHODS IN MINING,

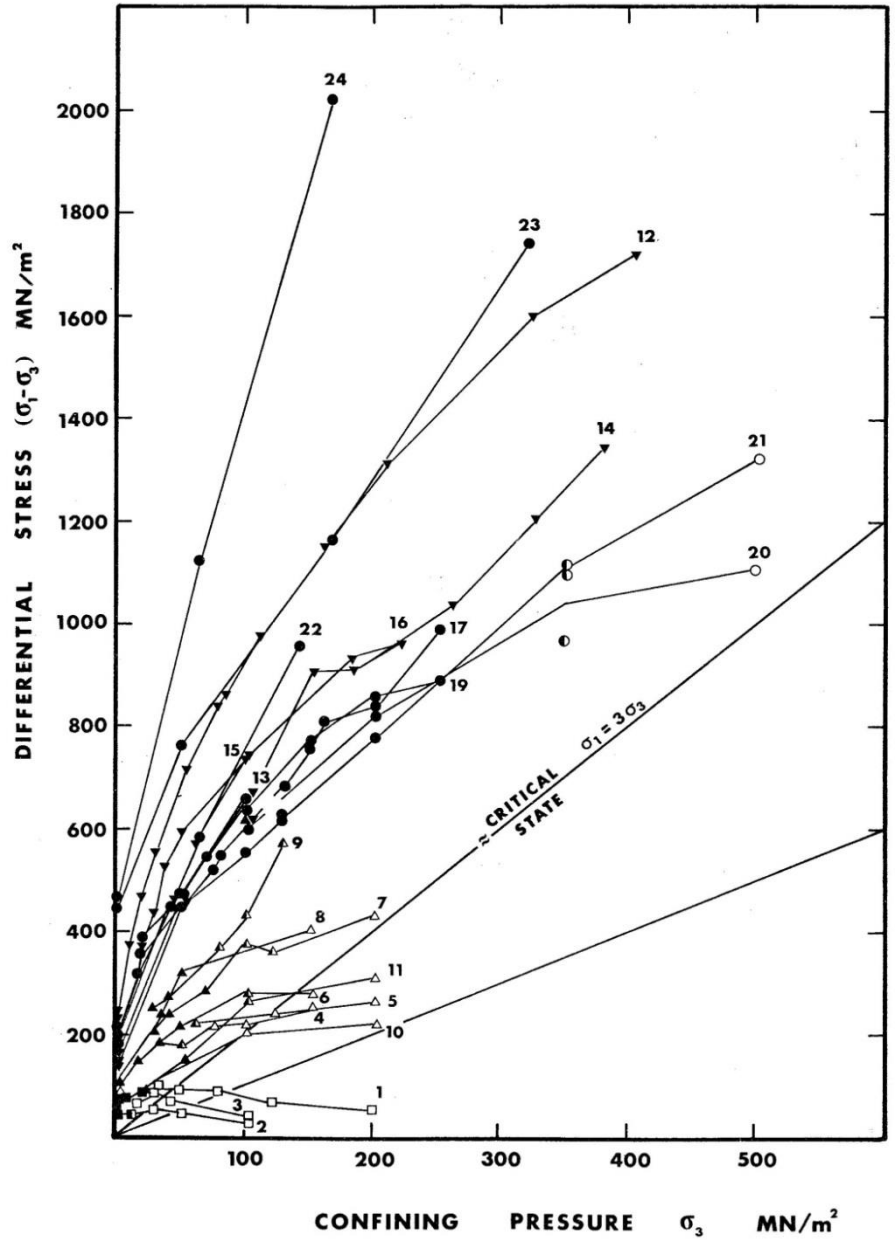
LIMA, PERU. June 2014

Content of Lecture

1. INTACT ROCK – CRITICAL STATE – A NEW CRITERION
2. ROCK JOINTS – NO COHESION – TESTING PROBLEM
3. SCALE EFFECTS, SCALED JRC, JCS, a/L-method
4. ROCK MASSES – displacement-and-process dependent
5. CANNOT ADD ‘c’ and ‘$\sigma_n \tan \varphi$’ / MODELLING OPTIONS
6. DEPTH DEPENDENT MODULI, VELOCITY, WITHOUT Q
7. SHEAR STRENGTH OF ROCK DUMPS, R and S
8. ZONATION WITH Q AND Q-PARAMETERS, OVER-BREAK
9. Q-CHART UPDATE, RRS, TEMPORARY SUPPORT, SRF
10. Q’ AND STOPE CHARACTERIZATION, ESR EQUIVALENT

1. INTACT ROCK with improved NON-LINEAR CRITERION



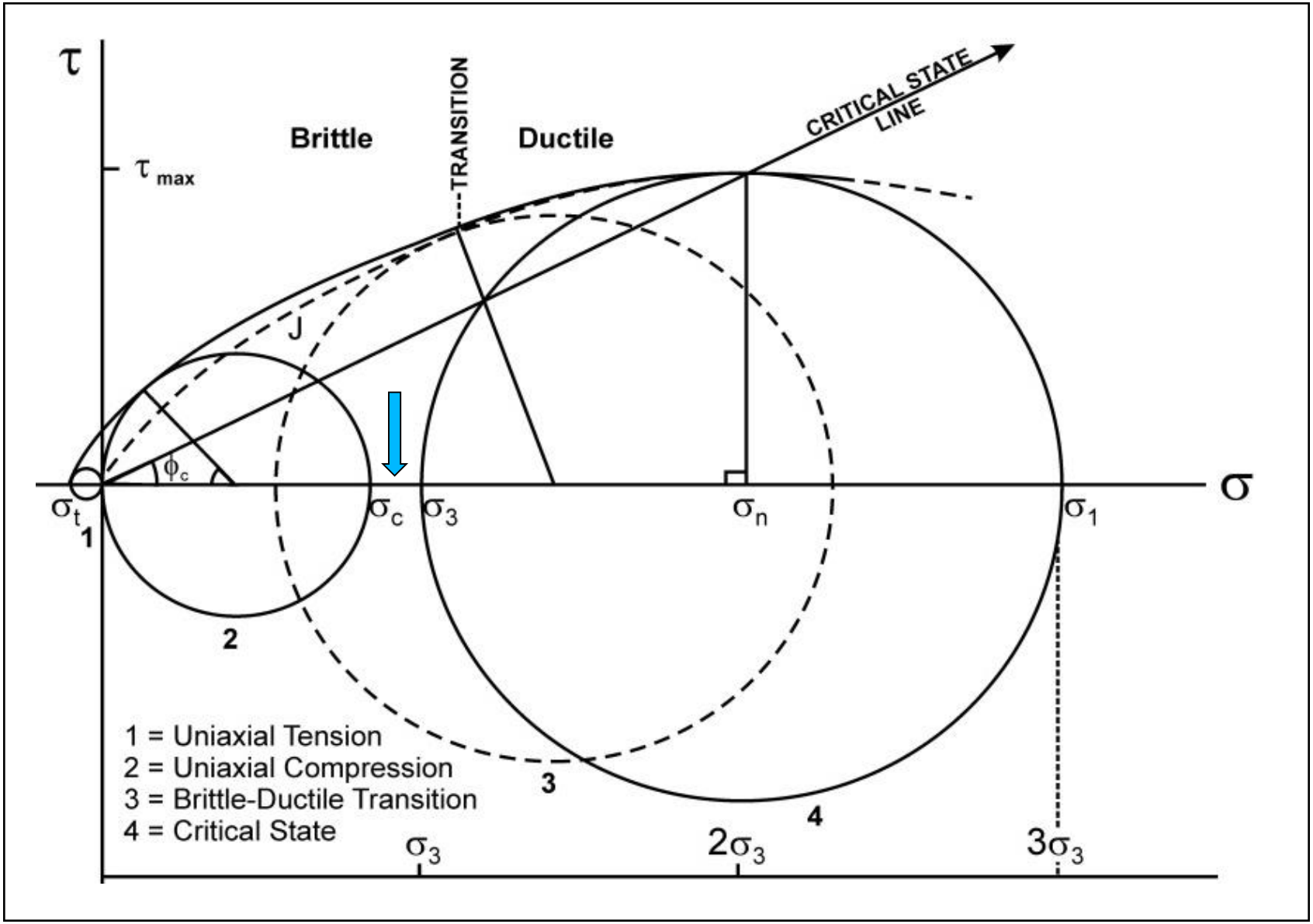


Some high stress triaxial data (Mogi 1966). Silicates and carbonates.

(σ_1 max = $3\sigma_{3\ crit}$) Barton, 1976

MOST FREQUENTLY: $\sigma_{1\ MAX} = 3\sigma_{3\ CRITICAL}$ (Barton, 1976)

σ_3 (CRITICAL) ≈ σ_c (Singh-Singh, 2011)



The ‘critical state’ concept (Barton, 1976) has been applied to better define the curvature of the *shear strength envelopes* for intact rock.

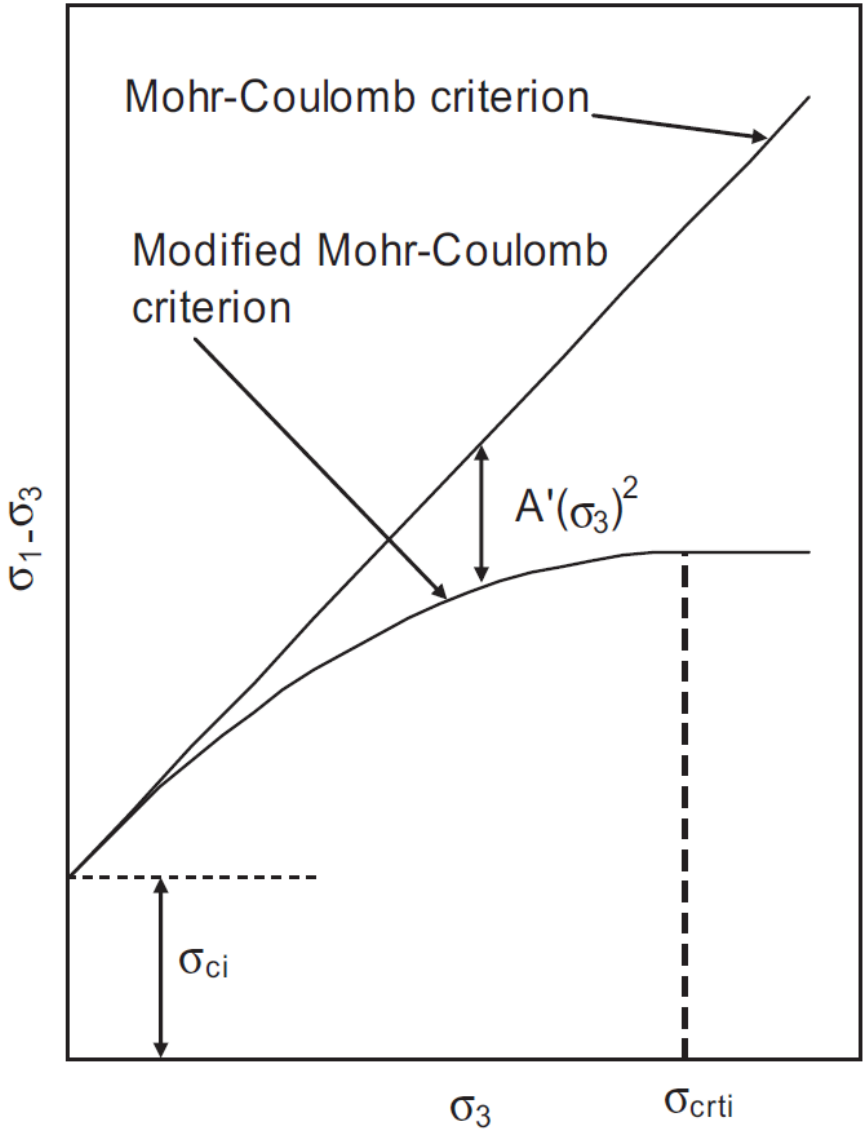
A few triaxial tests at *low confining pressures* provide all the data needed for extrapolation to high levels of confinement.

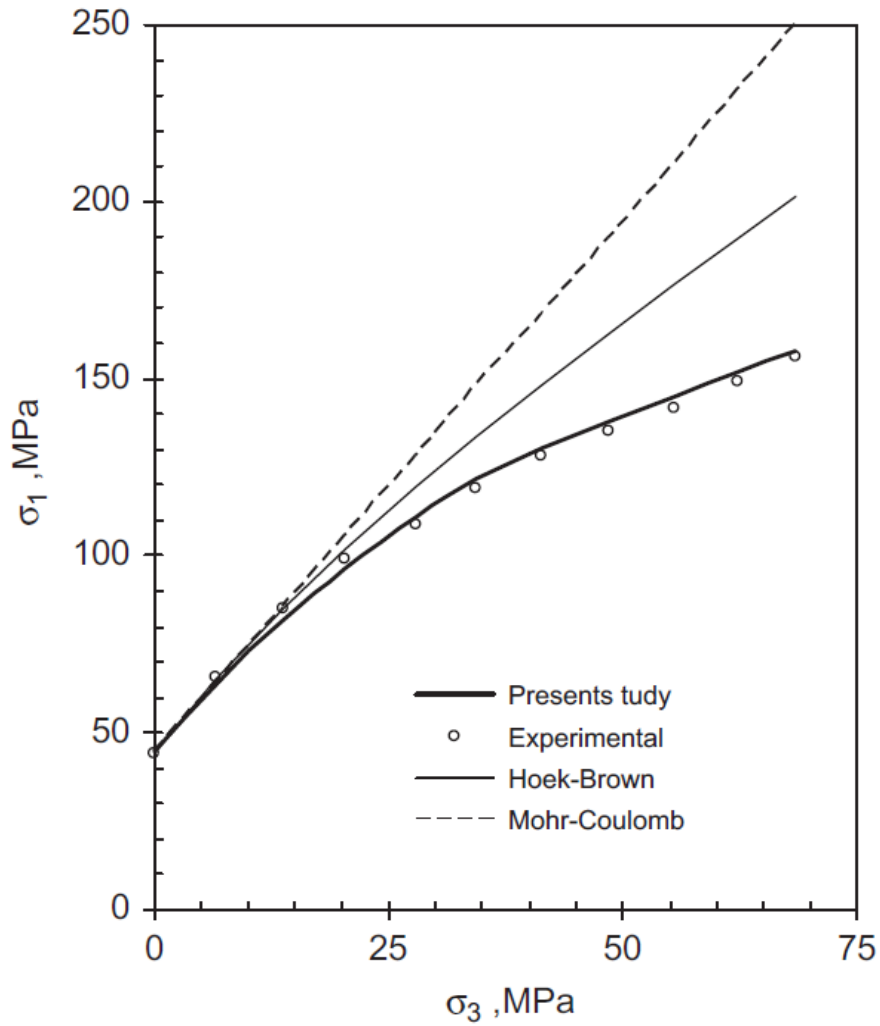
The elegant Singh et al., 2011 criterion gives the *correct deviation from linear Mohr Coulomb*

(= greater curvature than Hoek Brown, for which triaxial tests over a wider range of stress are needed)

From Singh et al., 2011

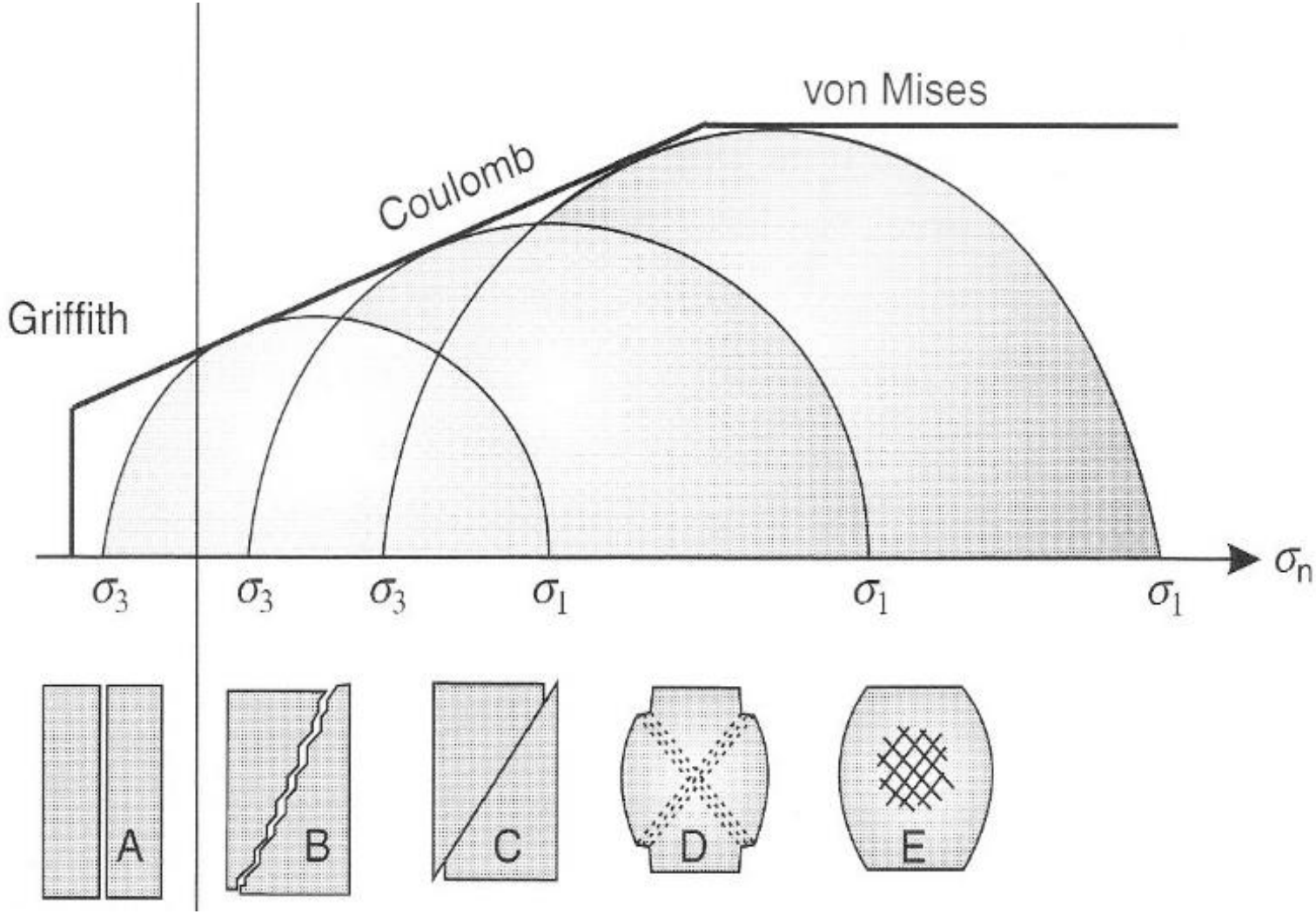
1. Objective
2. Example





The Singh-Singh criterion is a 'continuous' alternative to these classical criteria.

(From Gudmundsson, 2011)

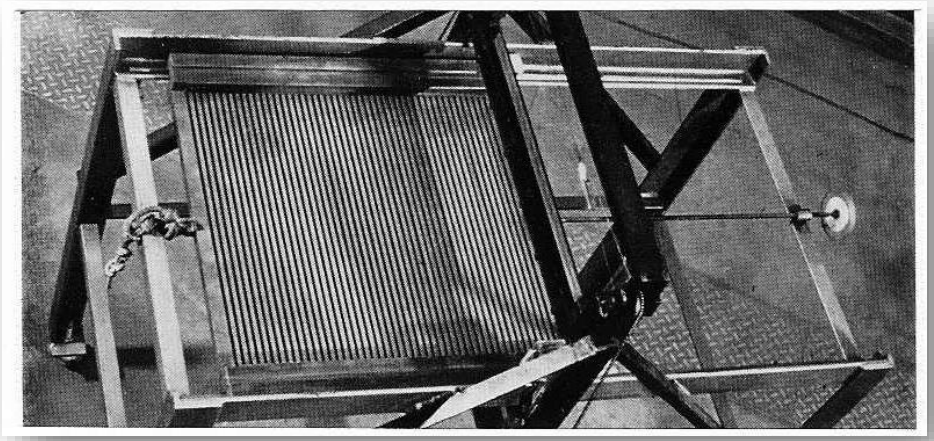


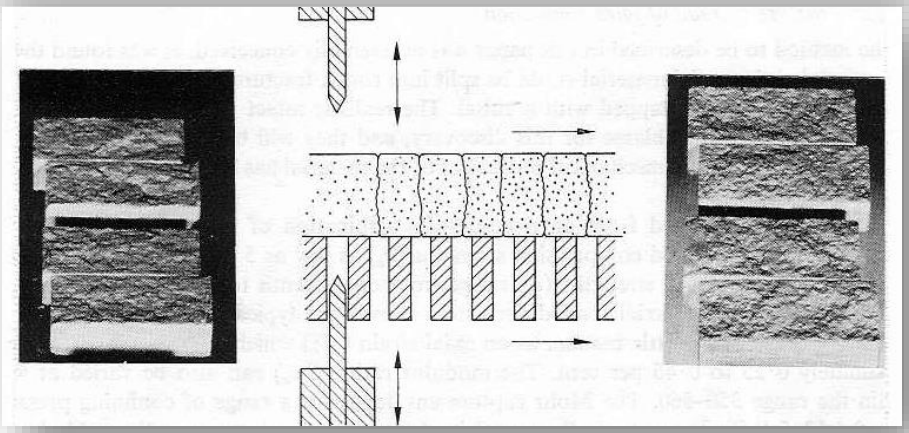
2. ROCK JOINTS

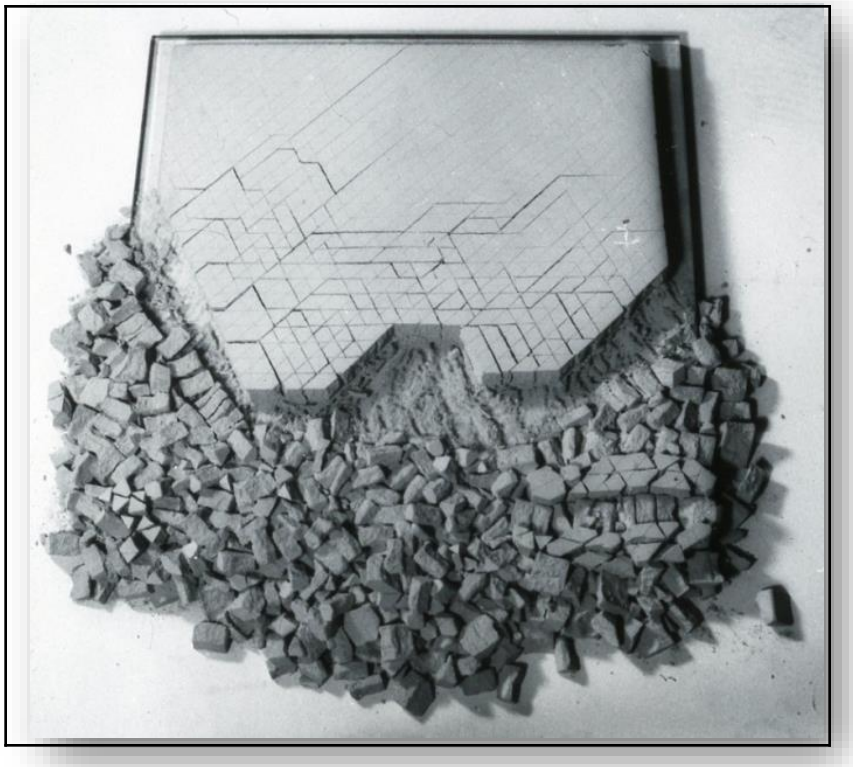
FIRST SOME BACK-GROUND.

WHY ROCK JOINTS DO NOT HAVE REAL COHESION !

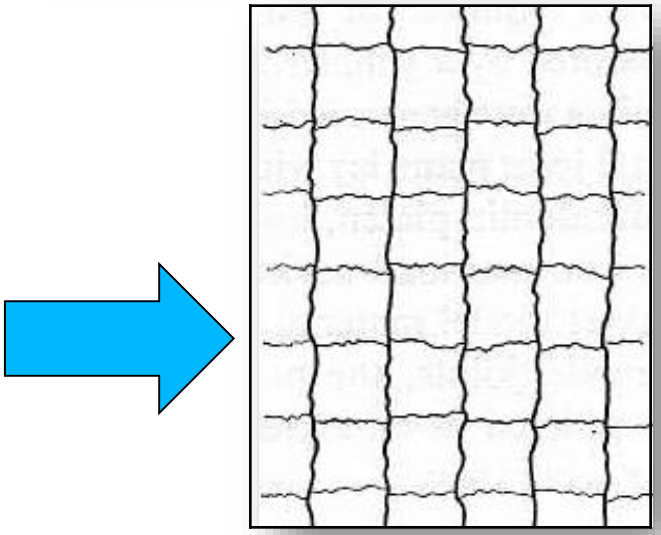
(Yet ‘everybody’ quotes ‘c’ and ‘ϕ’ ??)

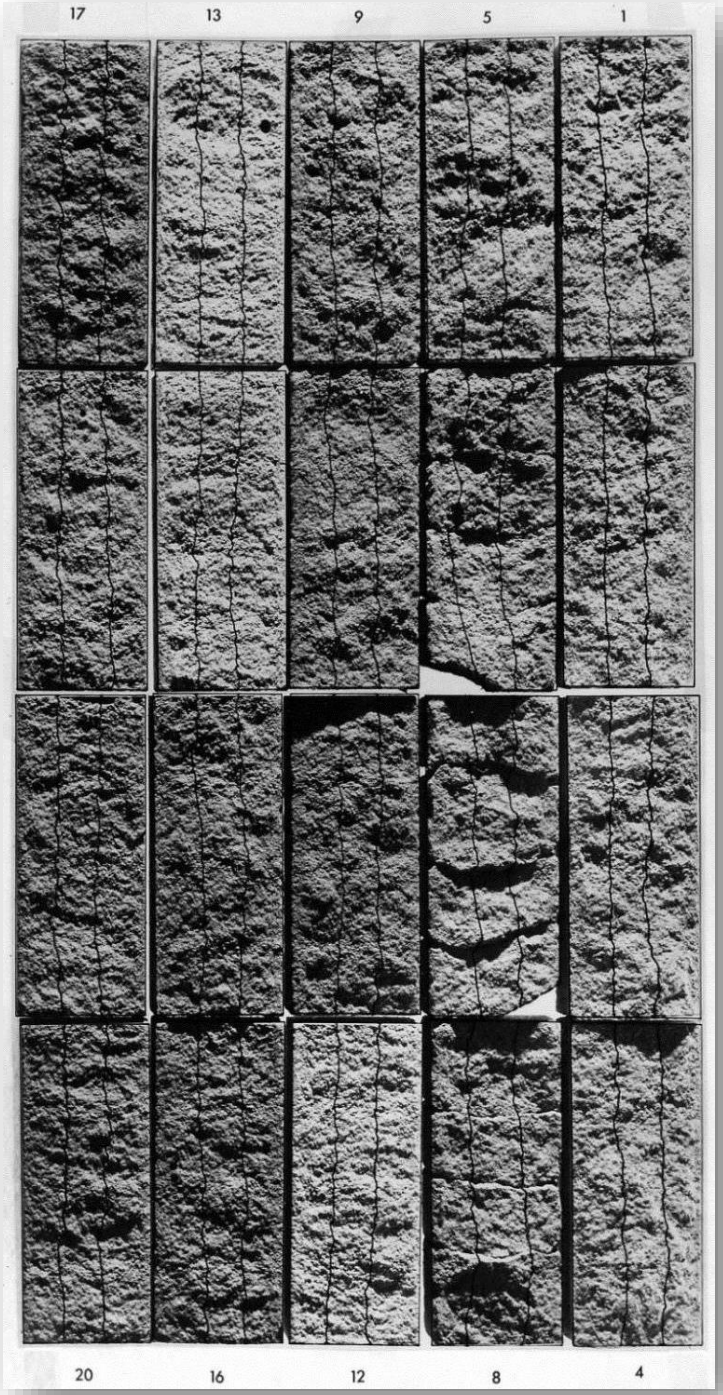




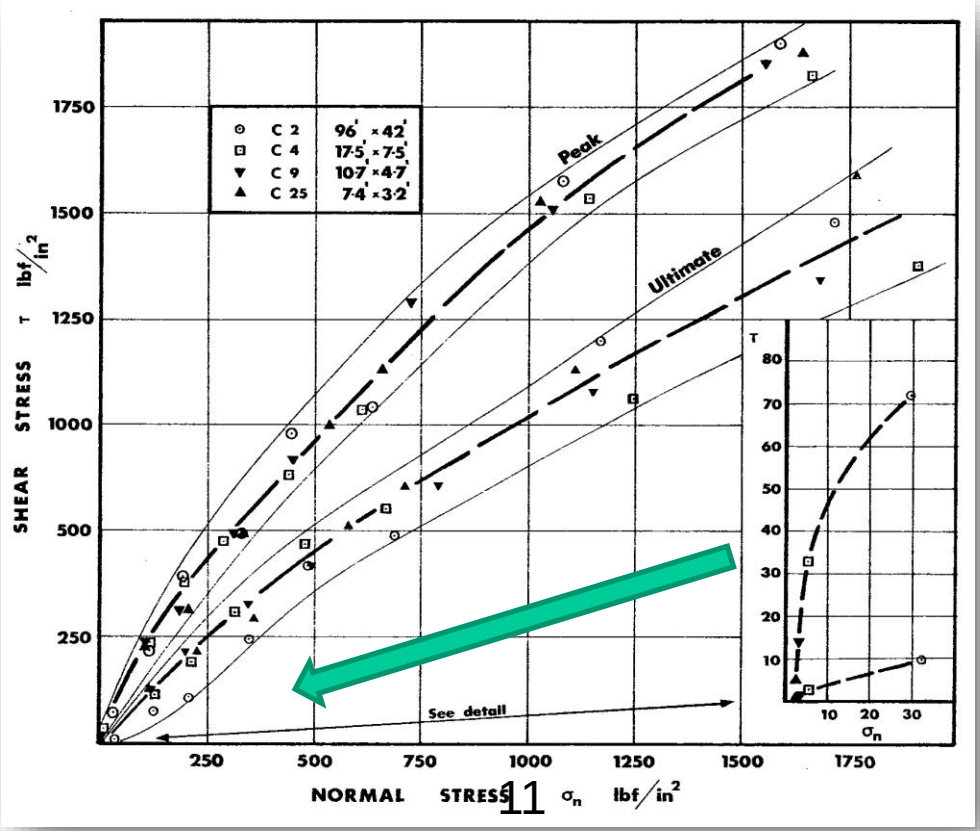


The 2nd (stepped) set of tension fractures has actual cohesion. The primary set does not.





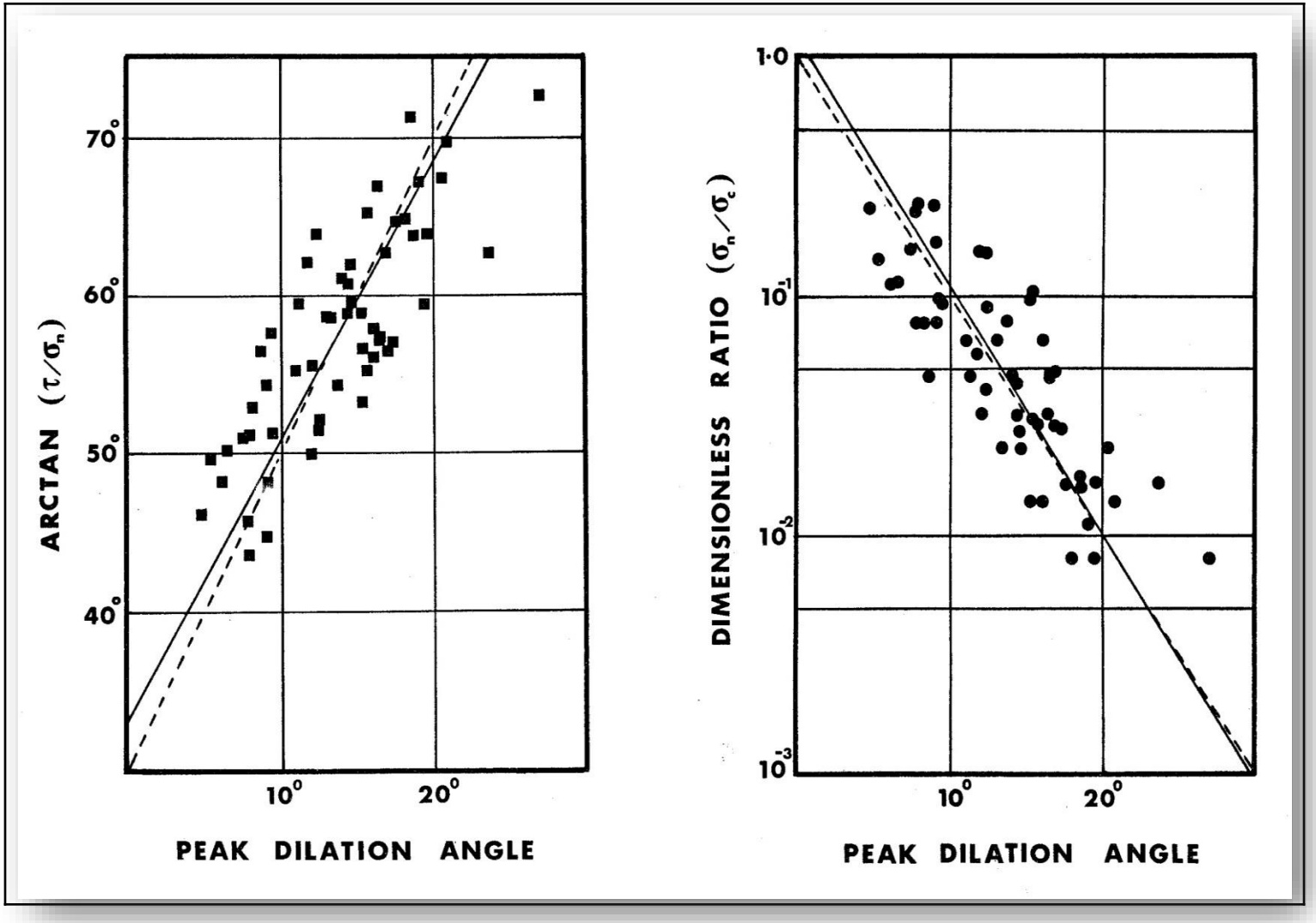
THESE ROUGH TENSION FRACTURES HAVE ***NO ACTUAL COHESION*** UNLESS STEPPED ("secondary") FRACTURES ARE TESTED

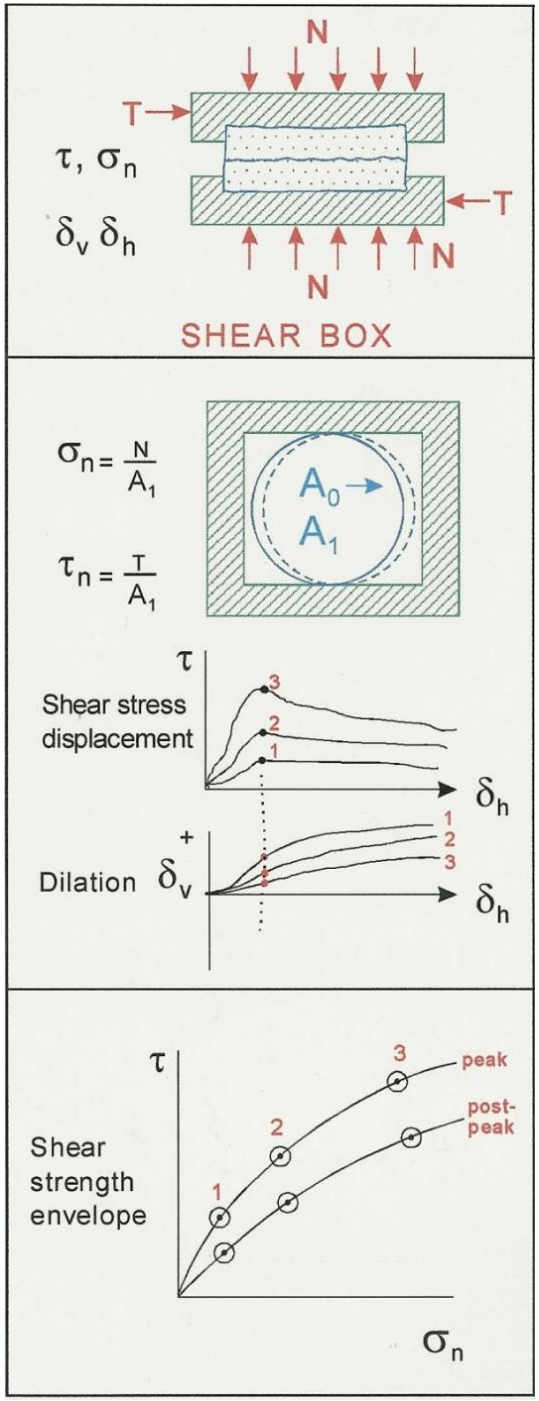


Peak strength criterion for the tension fractures

(------ = no decimal places, from Barton, 1971)

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





SOME YEARS LATER

DIRECT SHEAR TESTS ON REAL ROCK JOINTS (130 samples) SUGGESTED VARIABILITY

'20' (*became variable JRC*)

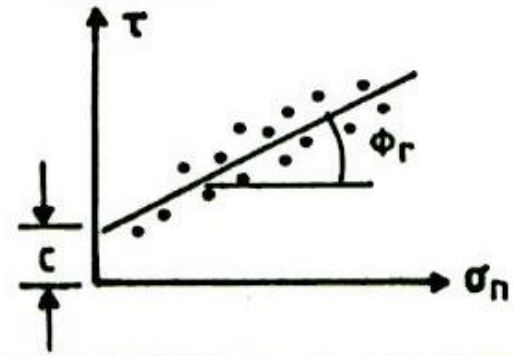
'UCS' (*became variable JCS*)

ϕ_b (no weathering) Barton, 1973, 1976

ϕ_r (weathered joints) Barton & Choubey, 1977

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

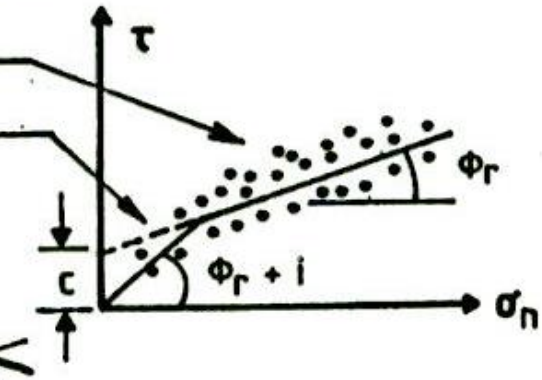
Mohr - Coulomb



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

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

Patton

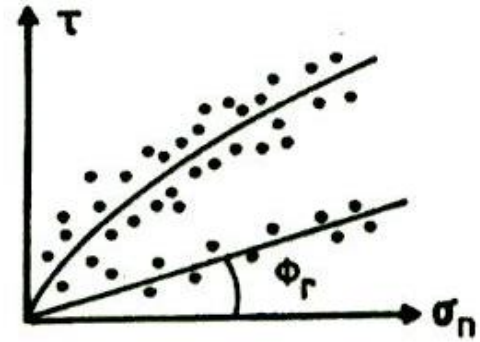


(3) $$\tau = \sigma_n \tan \left[JRC \log \left(\frac{JCS}{\sigma_n} \right) + \Phi_r \right]$$

JRC = joint roughness coefficient

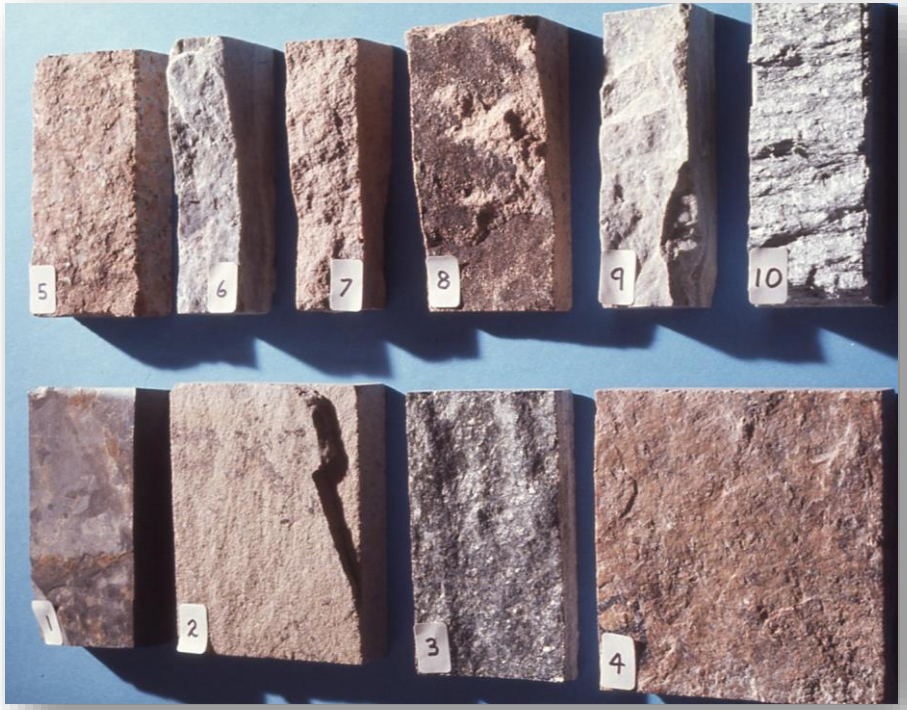
JCS = joint wall compression strength

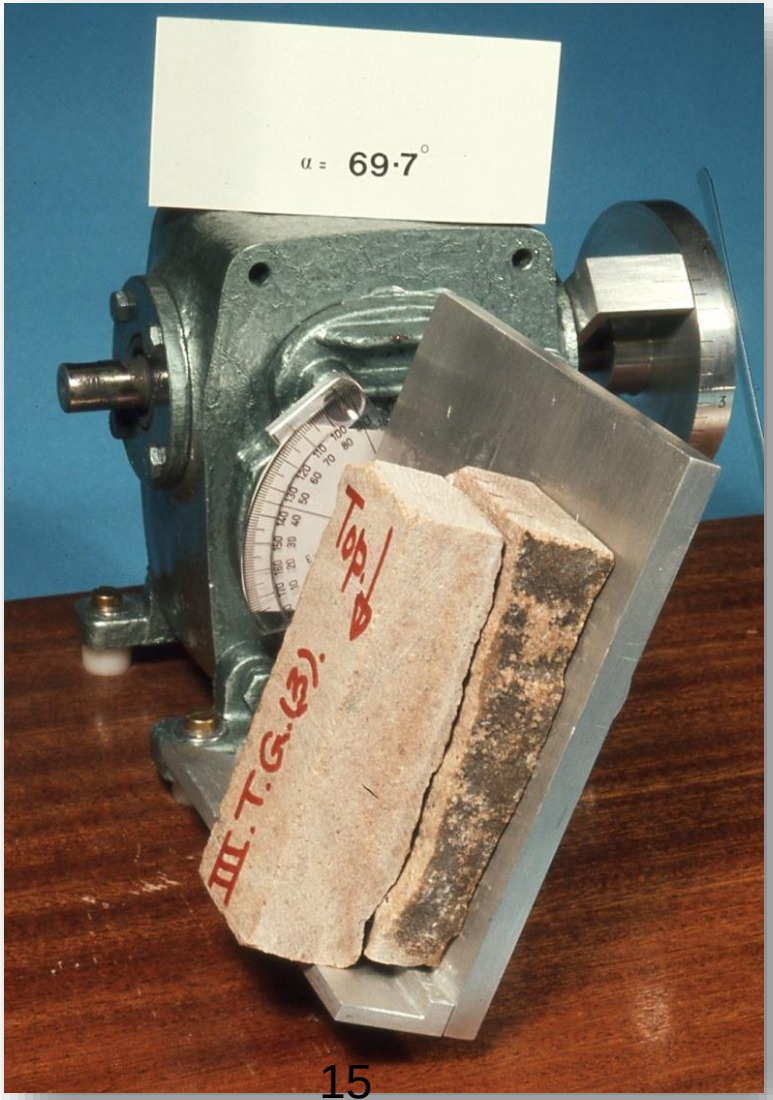
Φ_r = residual friction angle



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

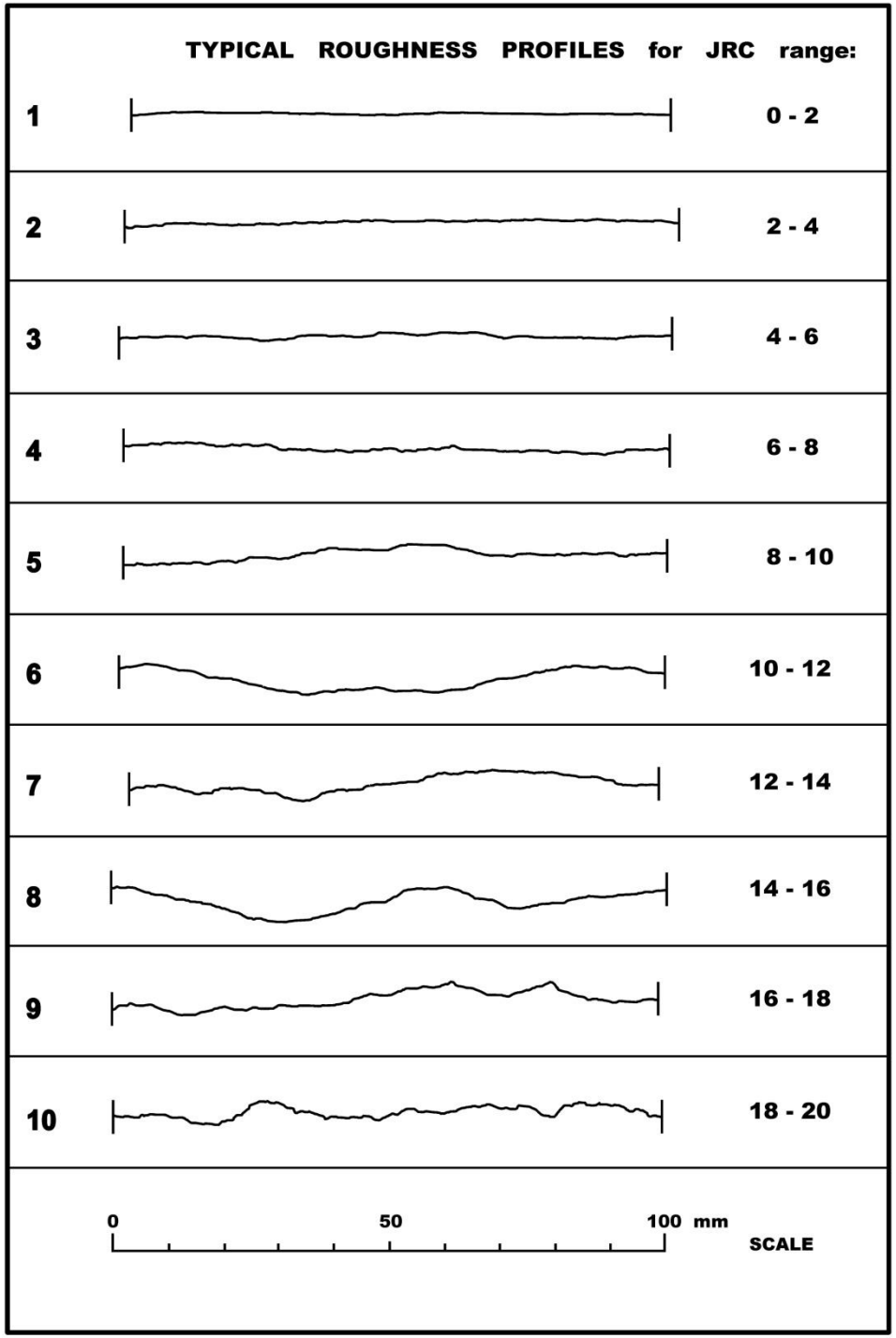
(Barton and Choubey, 1977)

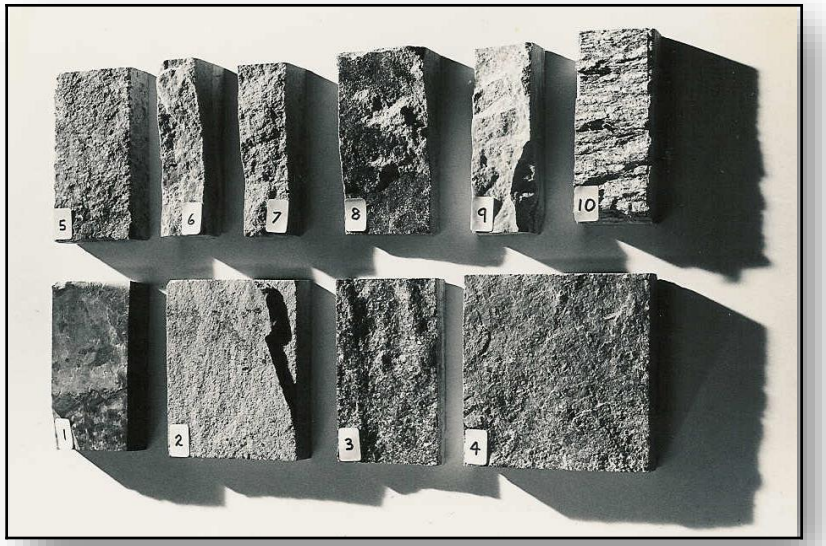


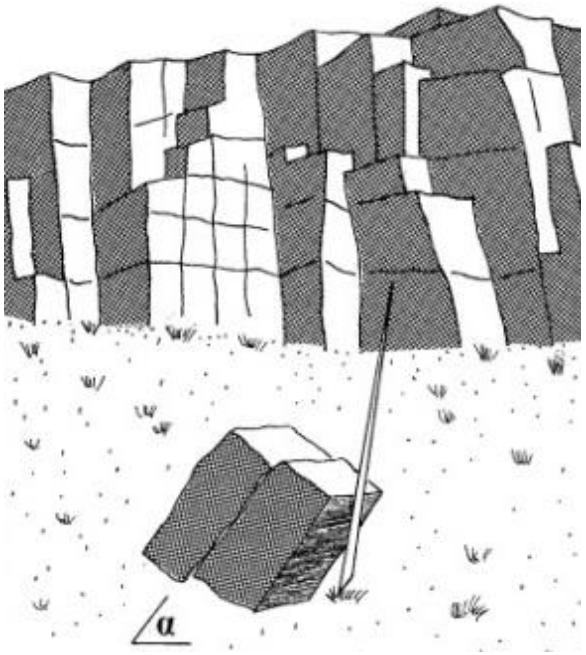


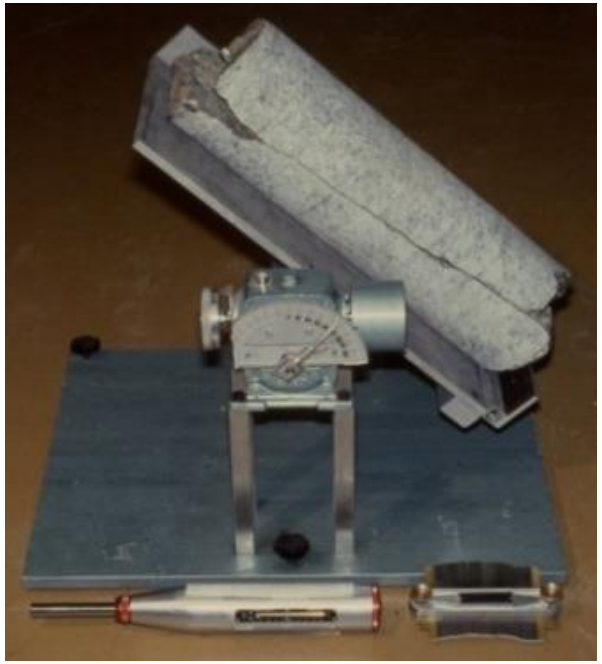
VISUAL MATCHING OF ROUGHNESS – for JRC: SUBJECTIVE OF COURSE

(Barton and Choubey, 1977)

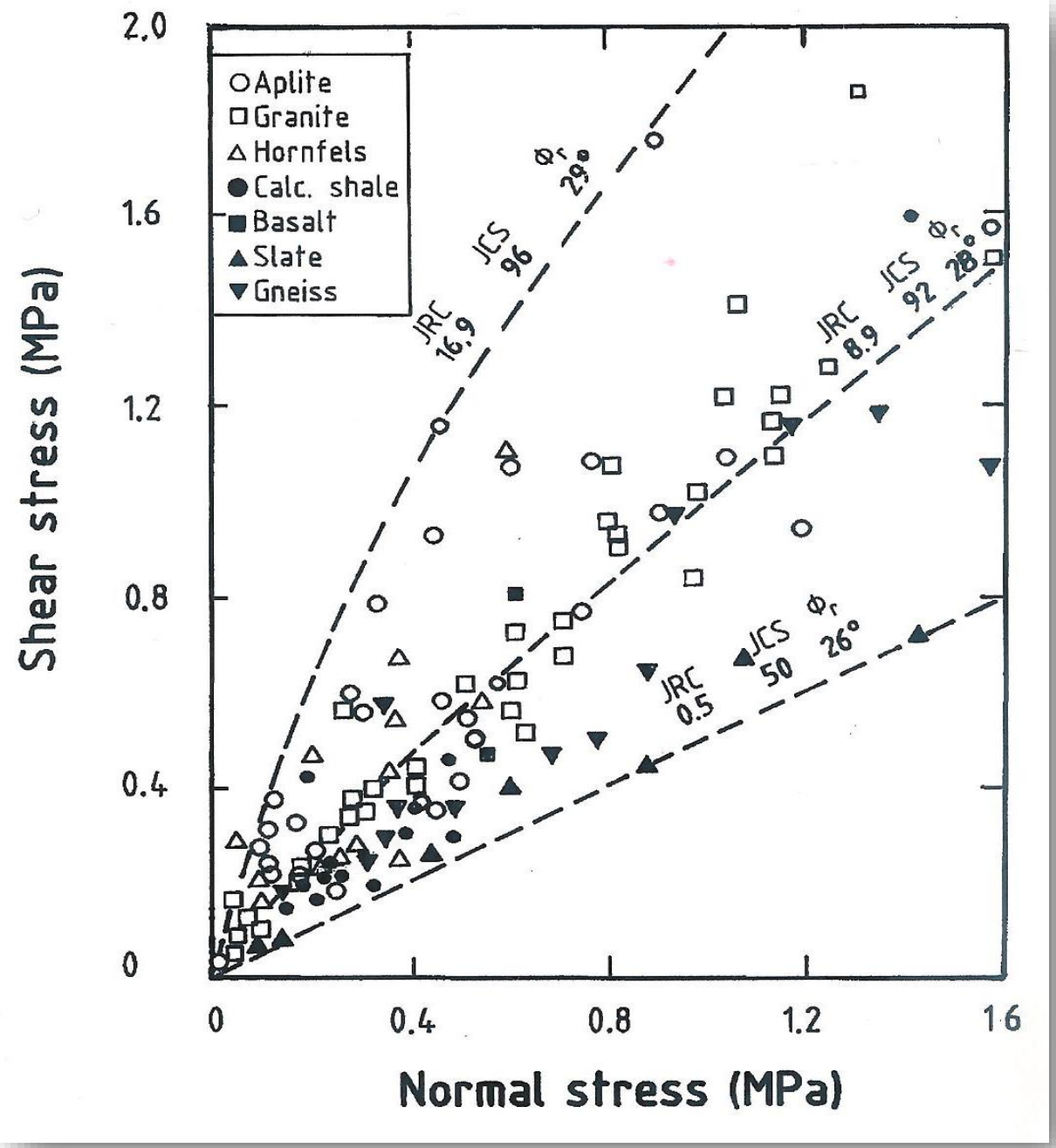








For those who don’t trust in profile matching for JRC – do tilt tests as NB & VC, 1977 recommended



130 rock-joint samples

(Barton and Choubey 1977)

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

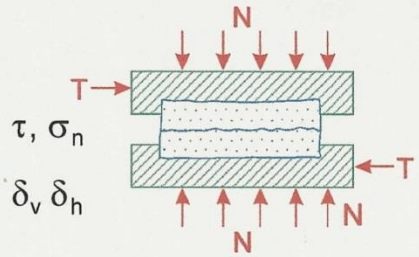
1. Maximum strength with JRC = 16.9

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

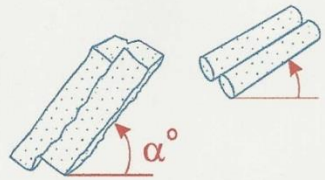
3. Minimum strength with $\varphi_r = 26^o$

EXAMPLES of ROUGHNESS CONTRAST, JRC CONTRAST.

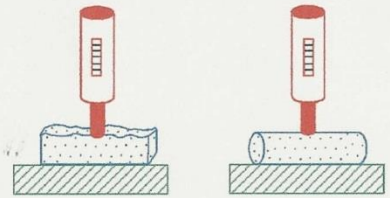
SHEAR BOX AND INDEX TESTING OF ROCK JOINTS



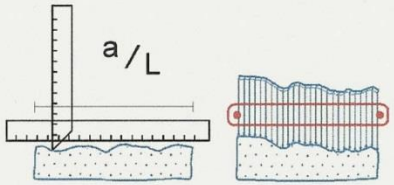
SHEAR BOX



TILT TEST



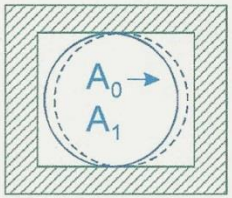
SCHMIDT HAMMER

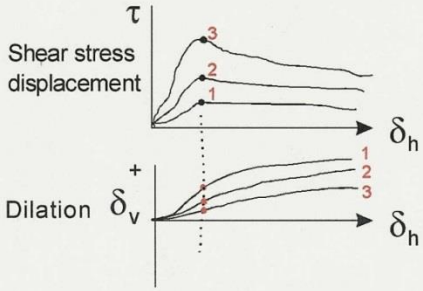


ROUGHNESS PROFILES

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

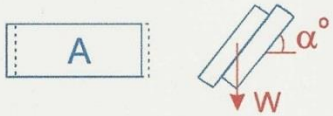
$\tau_n = \frac{T}{A_1}$





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

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



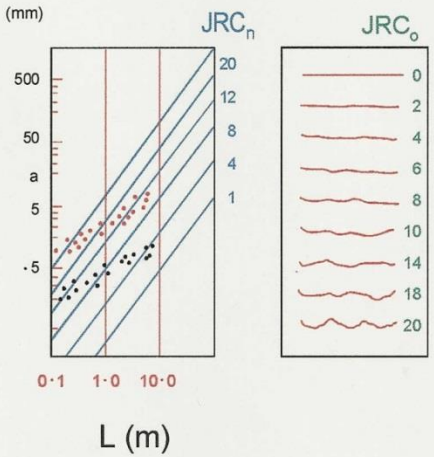
$$\phi_r = (\phi_b - 20^\circ) + 20\ r_5/R_5$$

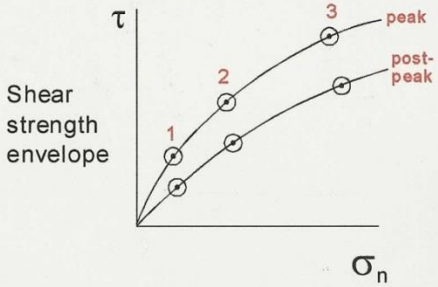
r	R
30	40
~~25~~	~~33~~
31	41
32	~~35~~
32	40
~~24~~	42
31	~~34~~
~~29~~	41
~~23~~	~~36~~
~~26~~	~~37~~

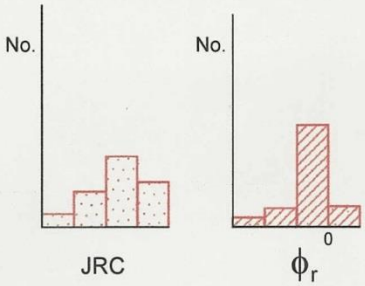
$\bar{r}_5 \rightarrow JCS_o$

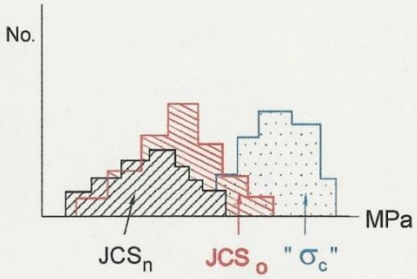
$\bar{R}_5 \rightarrow$ "σ_c"

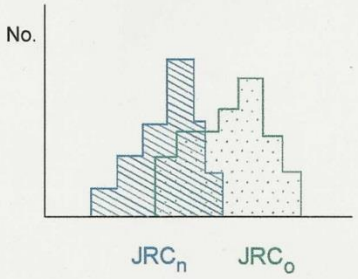
$\bar{r}_5 = 31{\cdot}1 \quad \bar{R}_5 = 40{\cdot}8$





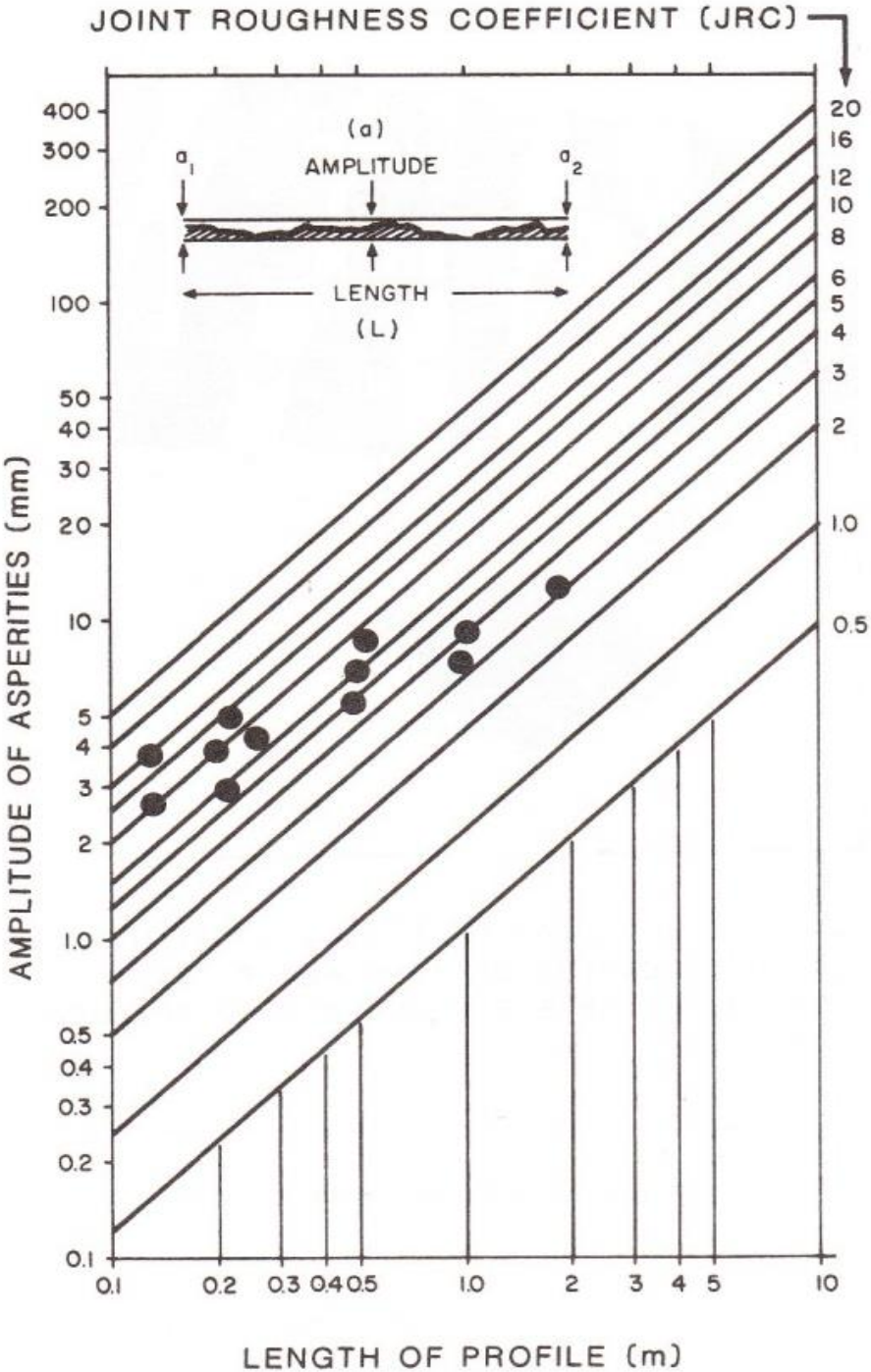






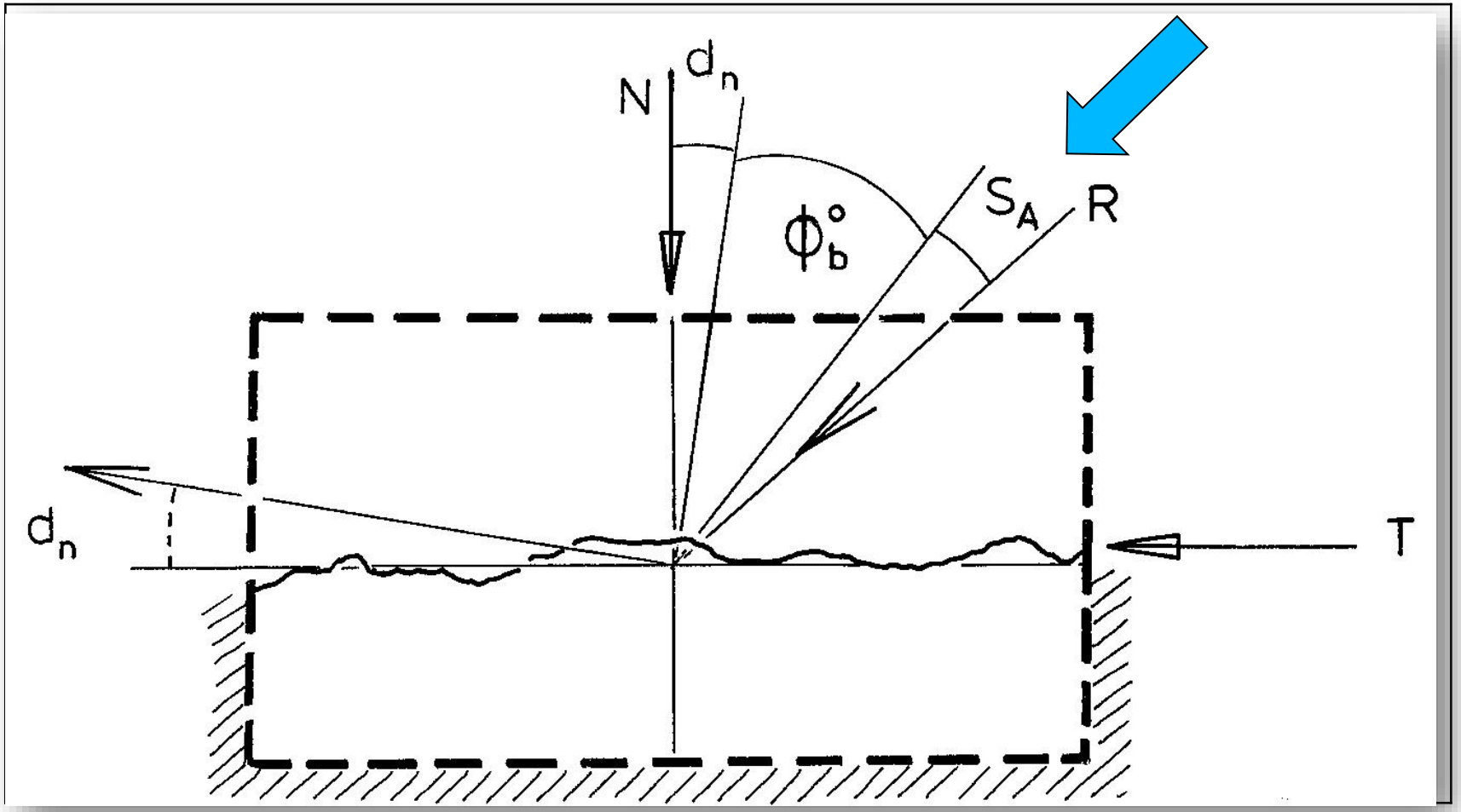
g:\desktop\coreldrw\542004\RockJoints.cdr

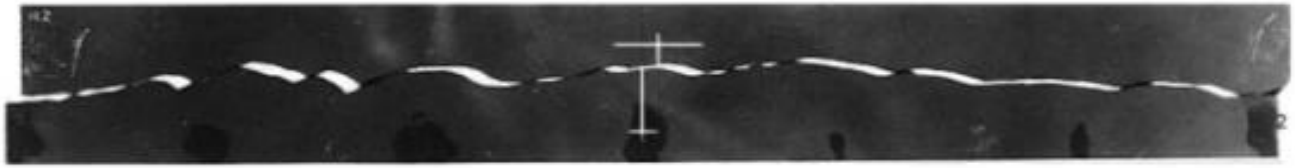
The ‘a/L’ method

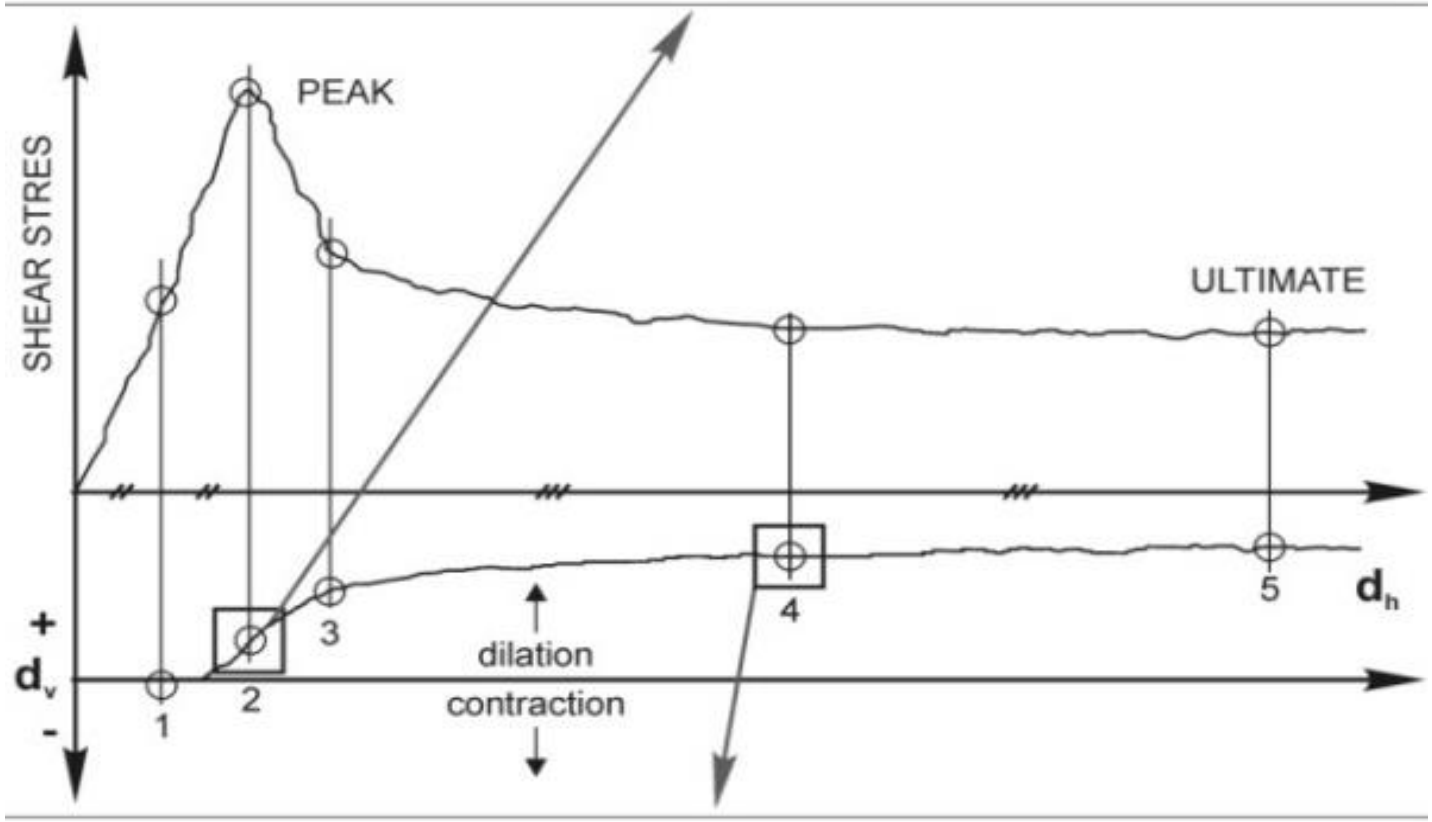


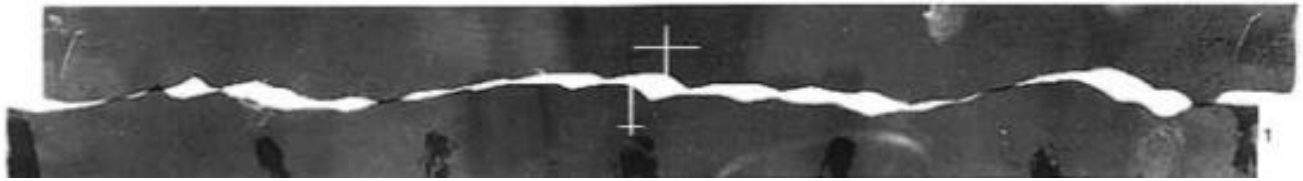
3. AN OVER-LOOKED COMPONENT OF SHEAR STRENGTH causing MISLEADING ERRORS

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

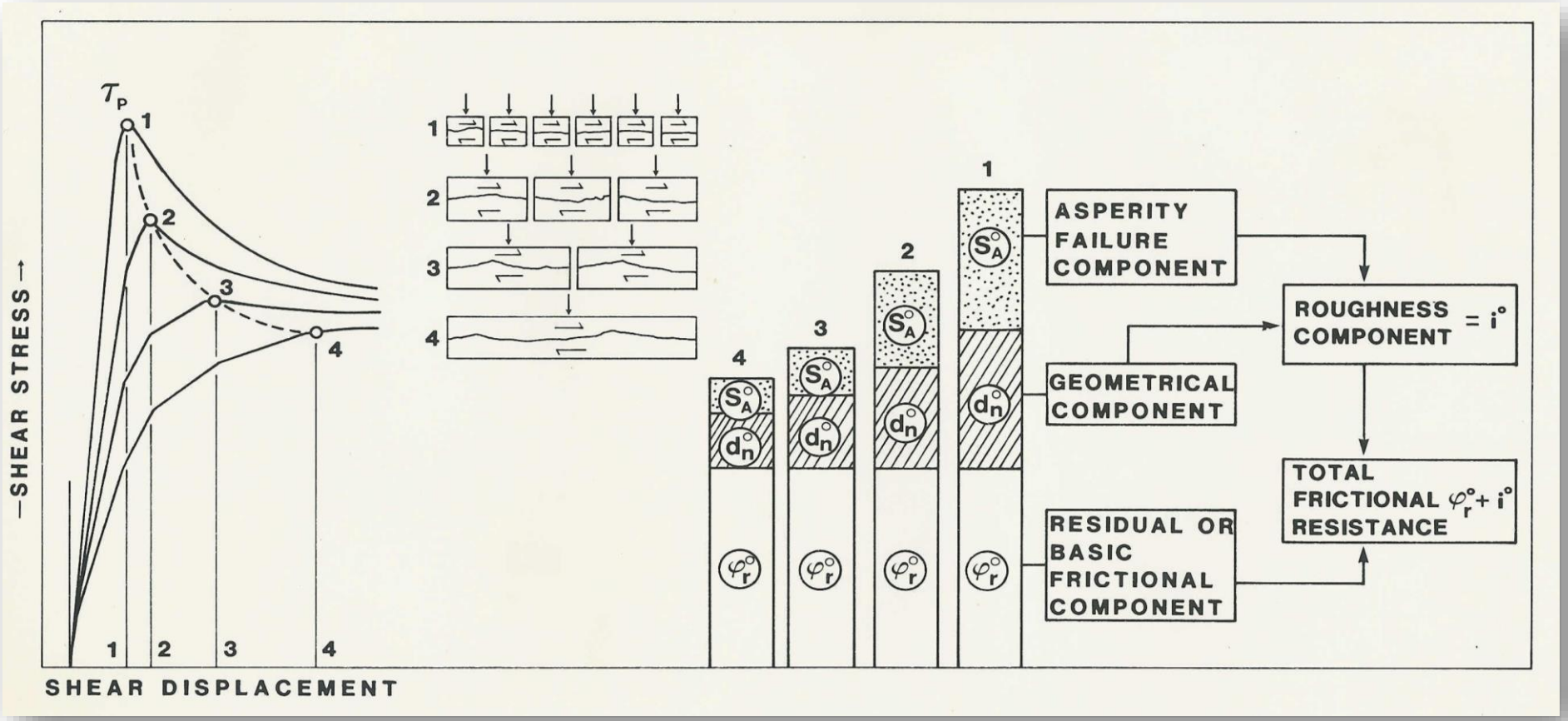








Asperity strength S_A due to highly stressed, failing asperities

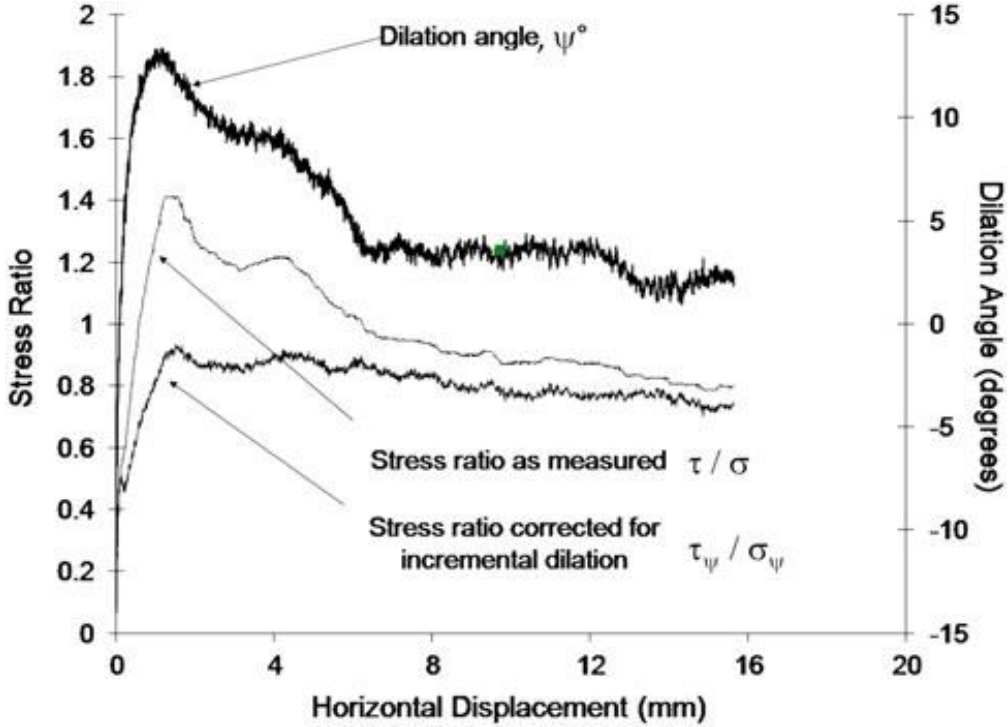


THE ASPERITY COMPONENT S_A

Bandis, 1980)

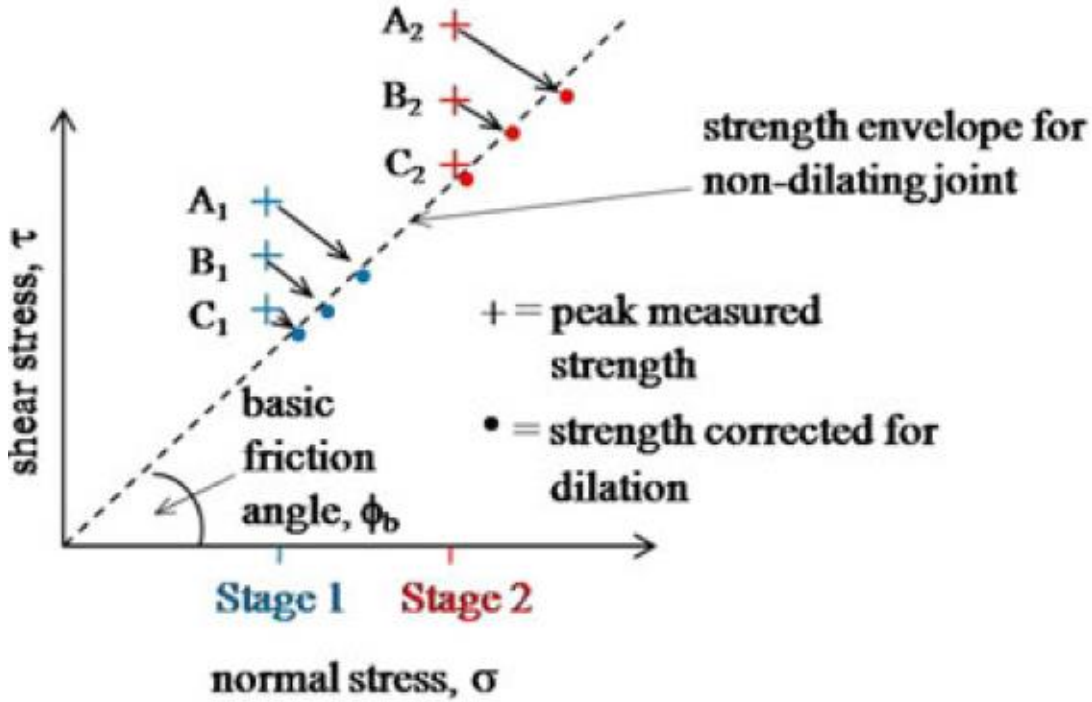
THEREFORE SUBTRACTING dn FROM PEAK STRENGTH DOES NOT GIVE ϕr (nor ϕb).

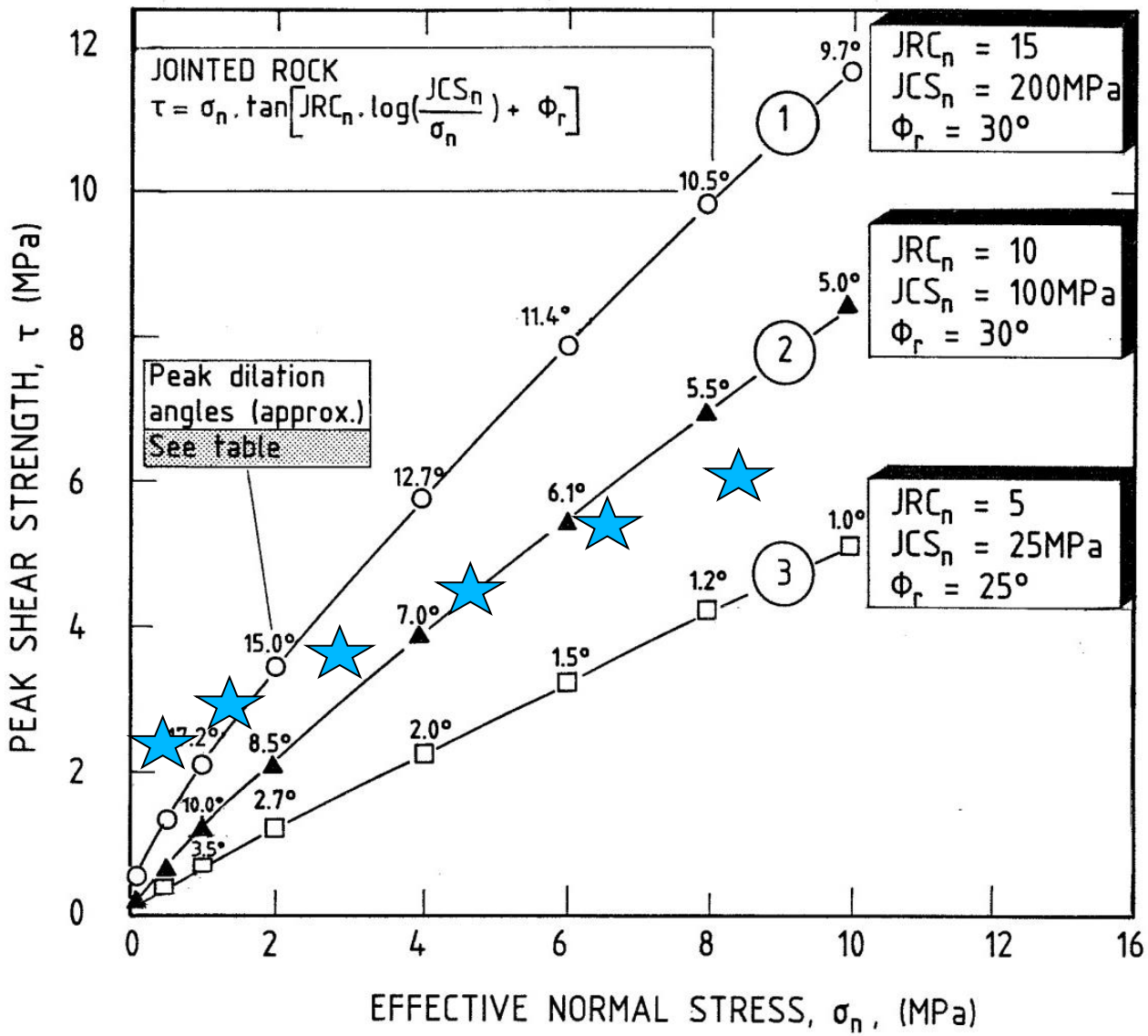
Misleading method, erroneous citations



Hencher error:

$\tan^{-1}(\tau/\sigma_n) - d_n \neq \phi_b$



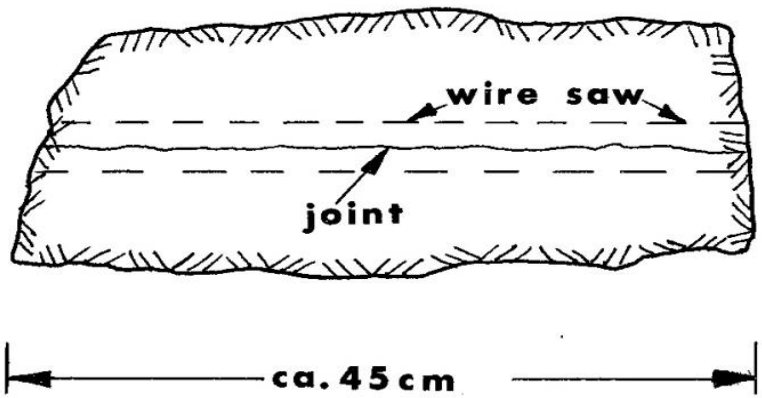


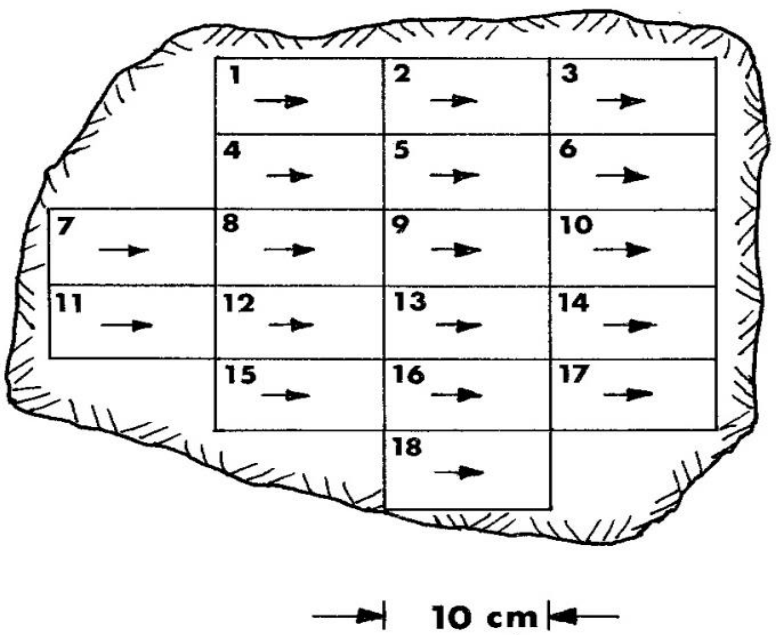
ISRM, TEST LABS recommendation of pre-loaded ***multi-stage shear testing*** ***(of same sample)*** if insufficient rock joint samples.

Results in 'rotation' of the strength envelope…… therefore artificial 'cohesion'

4. SCALE EFFECTS concerning ROCK JOINTS

IN SITU BLOCK

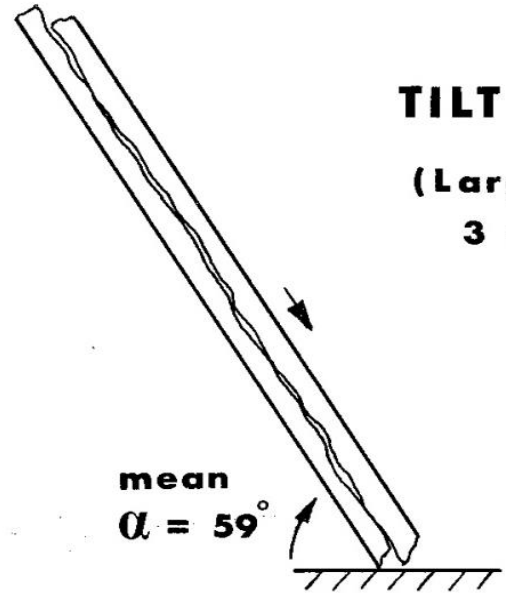




TILT TEST

(Large scale)

3 tests

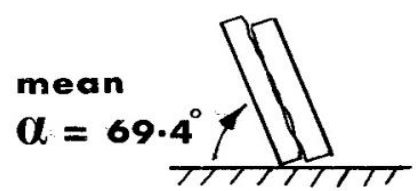


(1 sample)

TILT OR PUSH TEST

(Small scale)

54 tests



(18 samples)

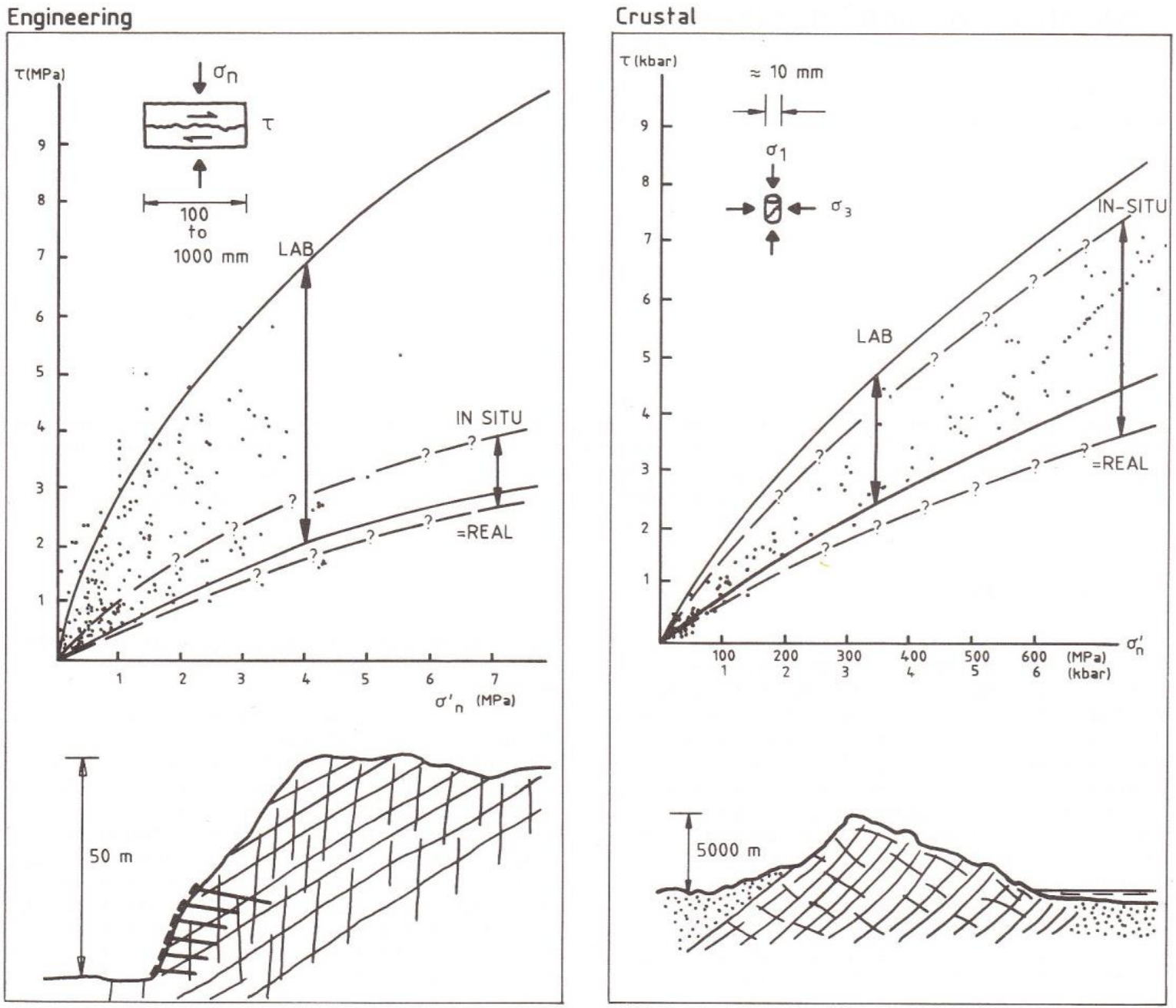
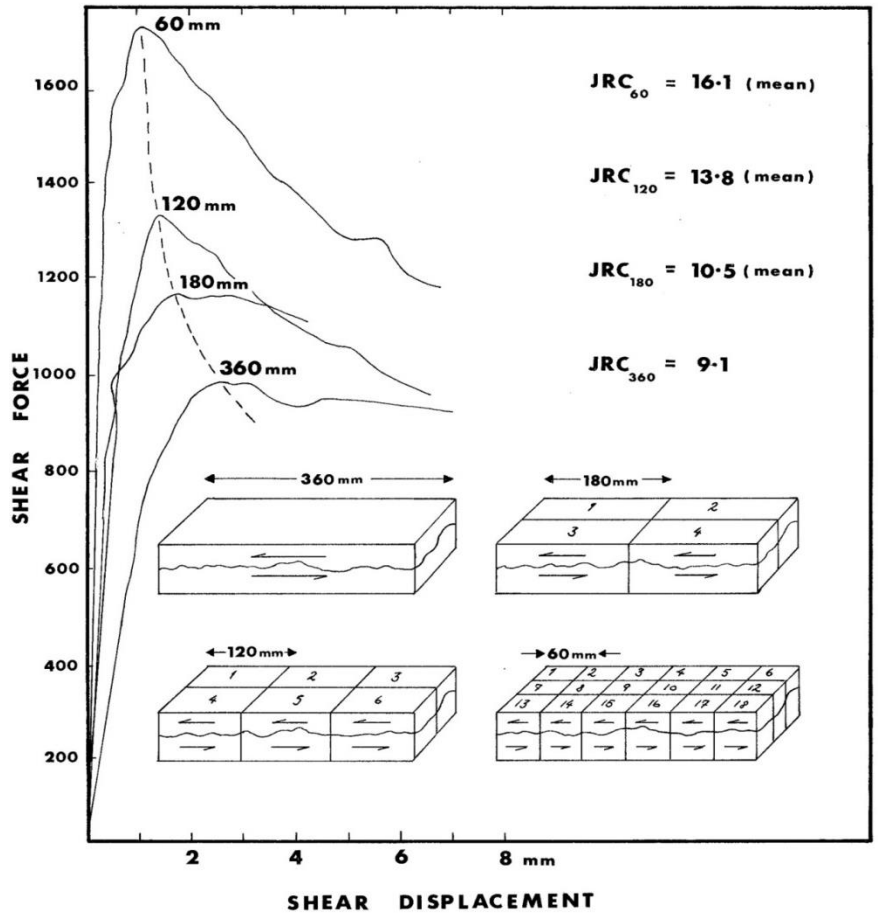
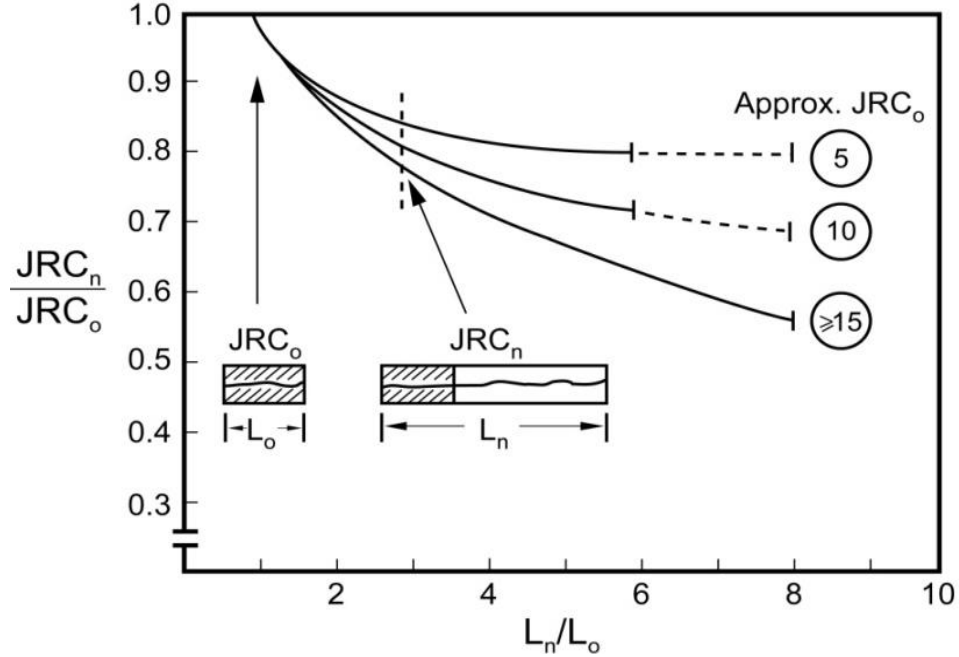


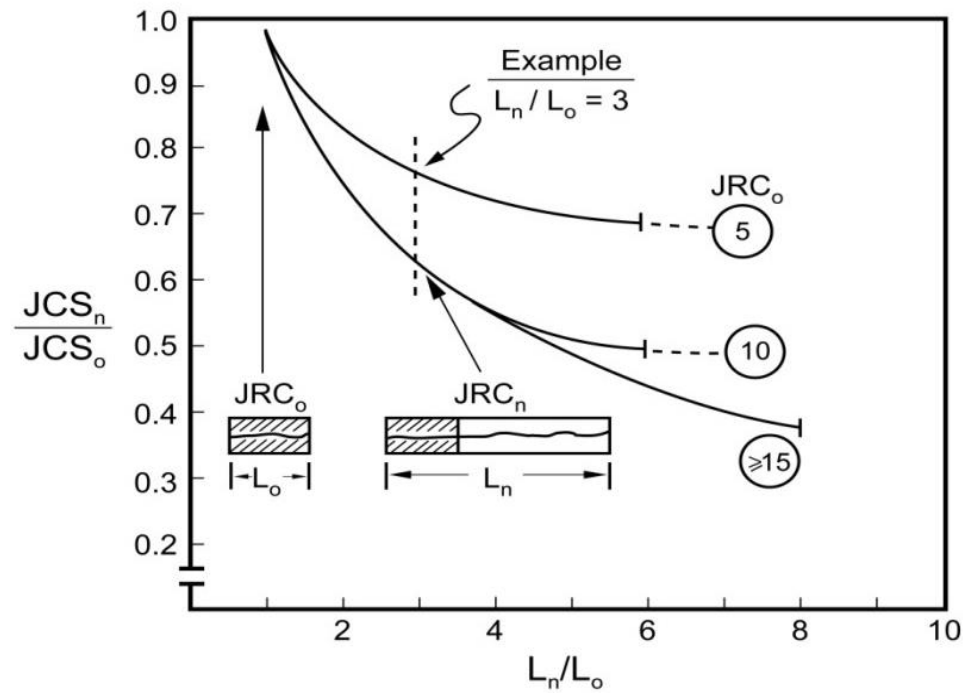
Figure 1. Different magnitudes of scale effect are expected under high and low stress. Adapted from Barton (1976). Additional data from Byerlee (1978).

Barton, 1990.

(Loen scale-effects workshop)



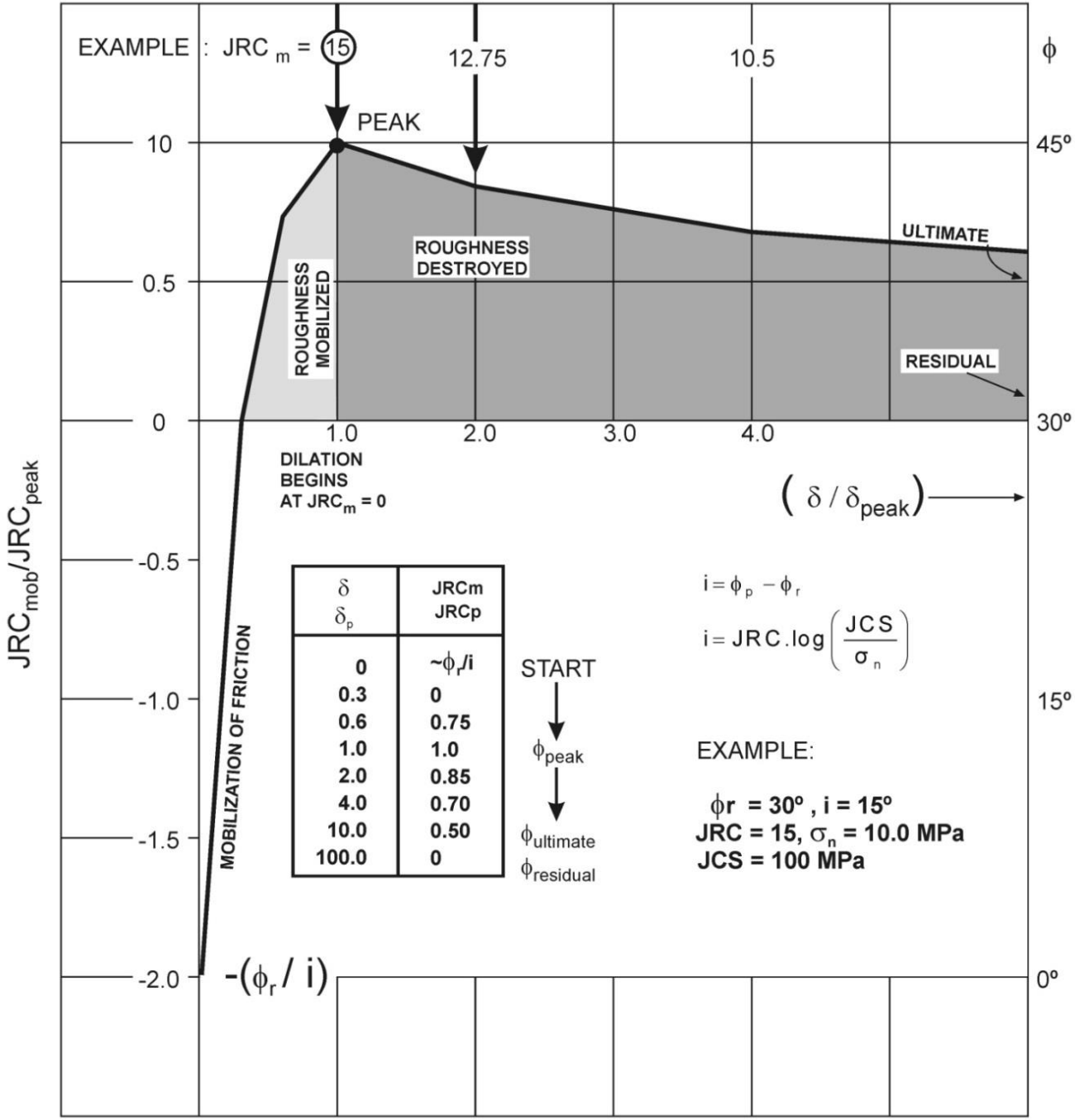




(Bandis et al. 1981)

$$JRC_n \approx JRC_o \, [\, L_n/L_o]^{-0.02\,JRCo}$$

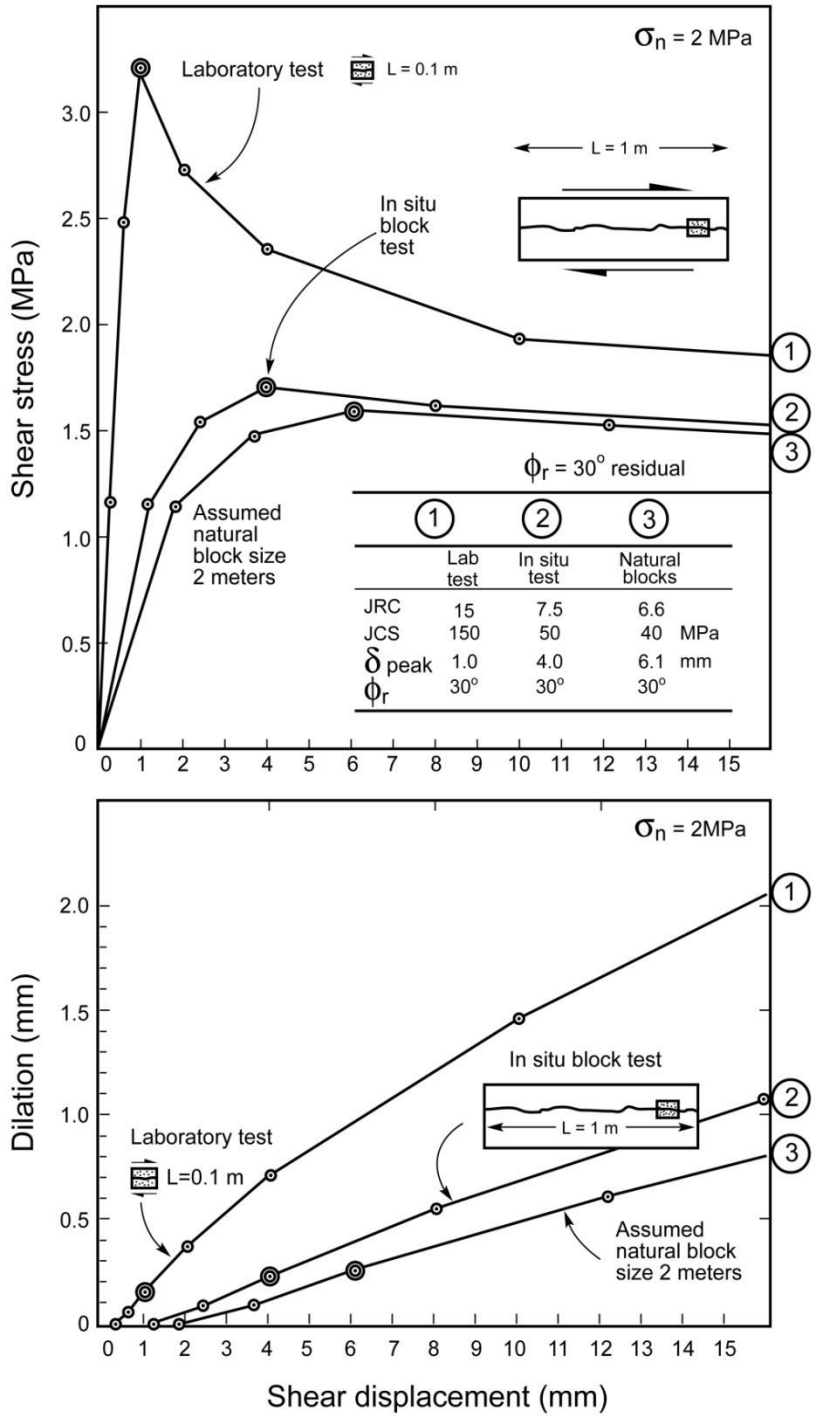
$$JCS_n \approx JCS_o \, [\, L_n/L_o]^{-0.03\,JRCo}$$



The JRCmobilized concept

(Barton, 1982)

ENABLES ***STRENGTH-DISPLACEMENT-DILATION MODELLING***

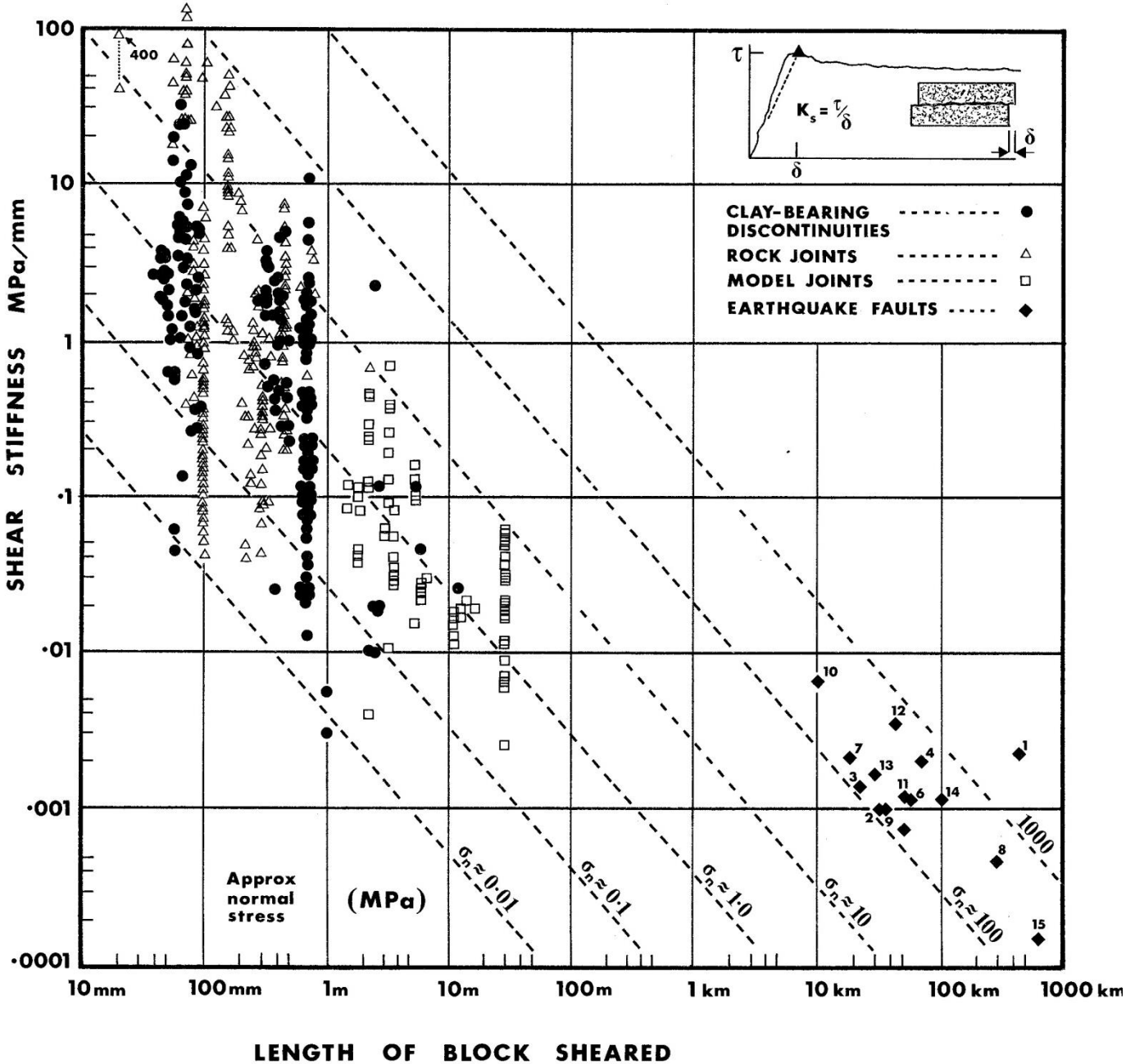


Shear stress-displacement and dilation-displacement modelling (Barton, 1982), with scale effect from Bandis et al.1981.

Note scale effect on shear stiffness (Ks), which is *strongly scale-and-stress-dependent*.

LOOK OUT FOR 'Kn = Ks' in some numerical models!!

(Usually Ks ≤ 1/50 x Kn)



Shear stiffness suffers a double scale effect

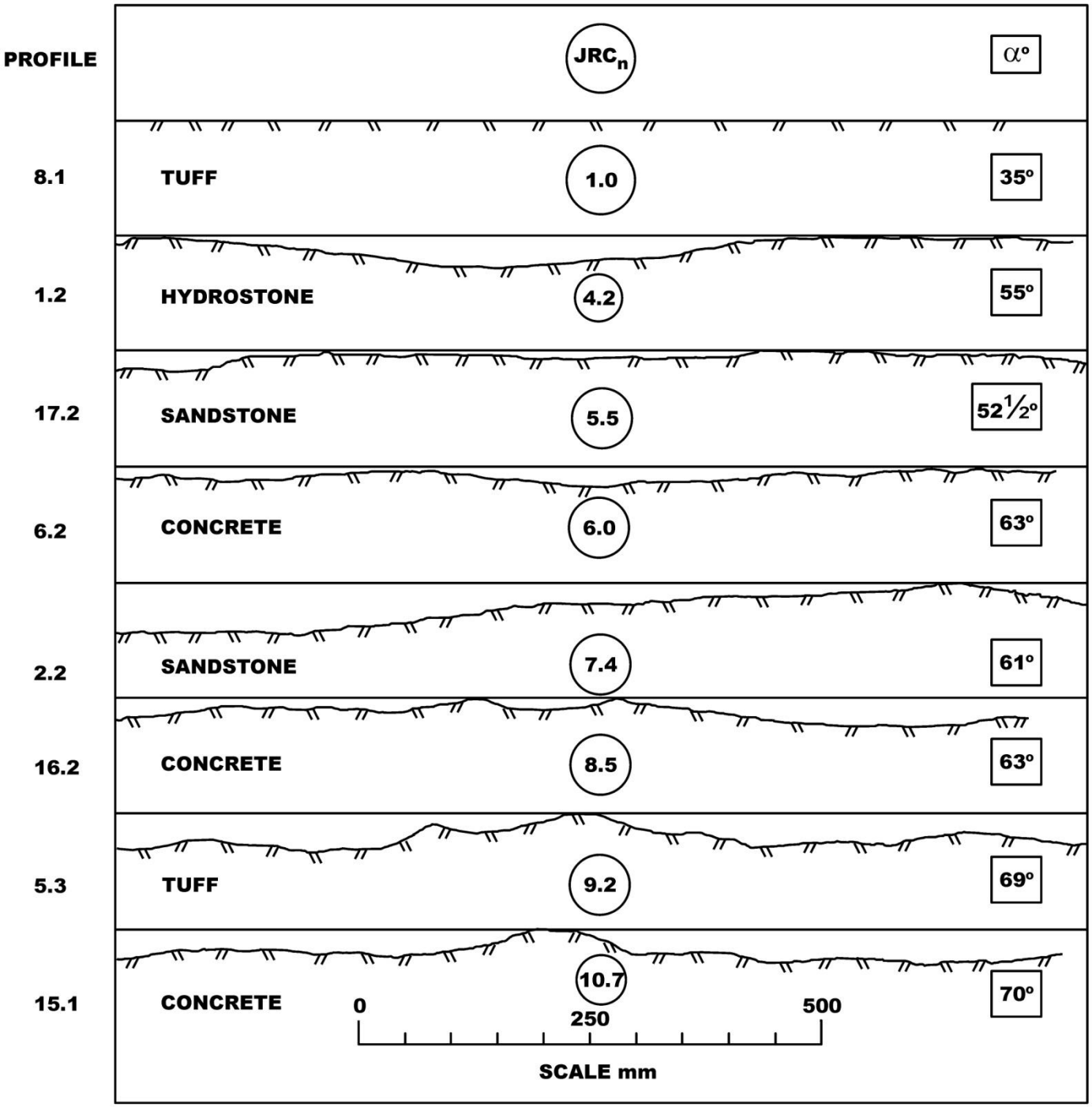
Well-jointed wedge.

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

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

Larger block(s)

(failure at much shallower angle of dip)



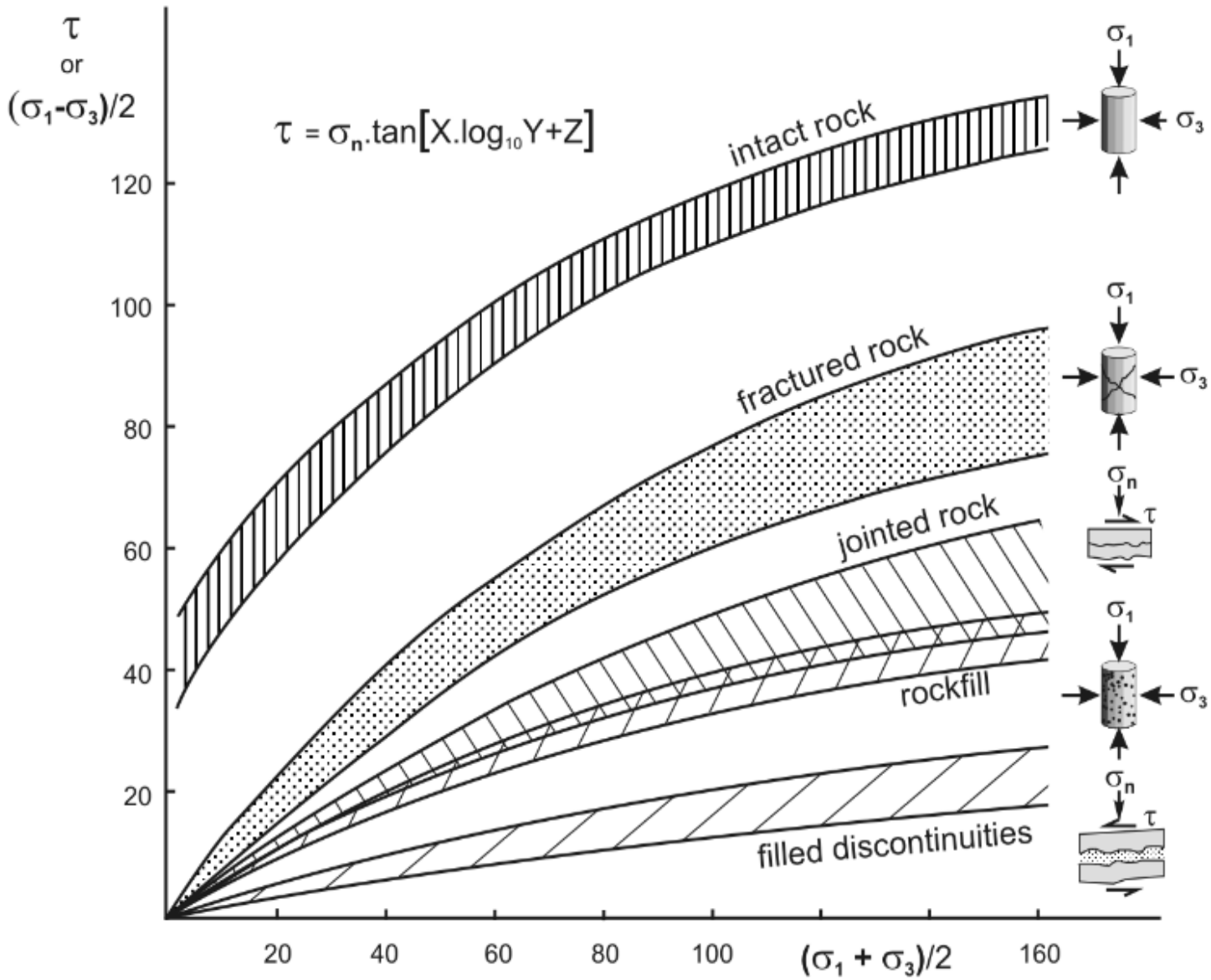
Very large-scale tilt tests (see angle α) with roughness profiles

(Bakhtar and Barton,1984)

5. ROCK MASS SHEAR STRENGTH

SHEAR FAILURE IS *PROCESS-AND-DISPLACEMENT* DEPENDENT

SEVERAL COMPONENTS OF SHEAR STRENGTH (Barton 1976, 2006)



x	y	z
50	$\left(\frac{\sigma_1 - \sigma_3}{\sigma_n}\right)$	ϕ_c
20	$\left(\frac{\sigma_1 - \sigma_3}{\sigma_n}\right)$	ϕ_b
JRC or	$\left(\frac{\sigma_1 - \sigma_3}{\sigma_n}\right)$ or $\left(\frac{JCS}{\sigma_n}\right)$	ϕ_b or ϕ_r
R	$\left(\frac{S}{\sigma_n}\right)$	ϕ_b
$[J_r / J_a]$ (from Q-system)		

THE REALITIES OF ROCK MASS SHEAR STRENGTH

PROCESS-AND-STRAIN-DEPENDENT FAILURE (1 to 4).

1. INTACT BRIDGES FAIL, BLOCK CRUSHING, AT SMALL 'STRAIN'

2. NEWLY CREATED FAILURE SURFACES WITH high JRC, JCS, $\varphi_r = \varphi_b$, SHEAR NEXT AT SMALL 'STRAIN'

3. SURROUNDING NATURAL JOINTS with lower JRC, JCS, φ_r, SHEAR NEXT AT LARGER 'STRAIN'

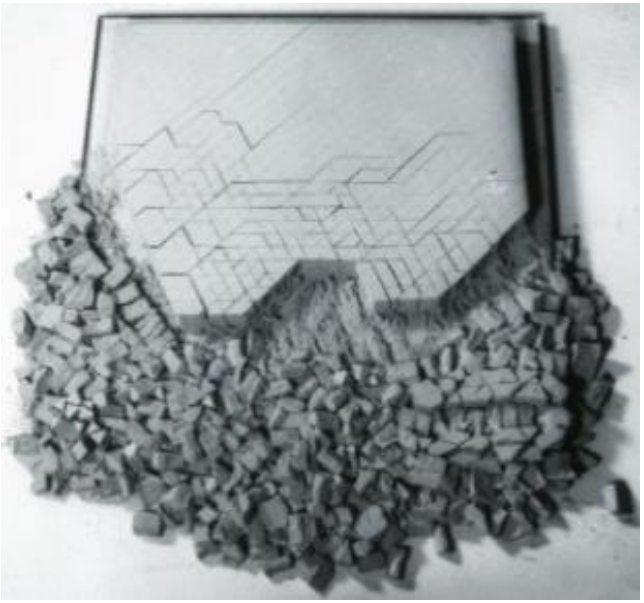
4. DISCONTINUITIES WITH CLAY, FAULTS, MOBILIZE AT STILL GREATER 'STRAIN'

ALL THE ABOVE ARE BEYOND Q, BEYOND GSI / H-B

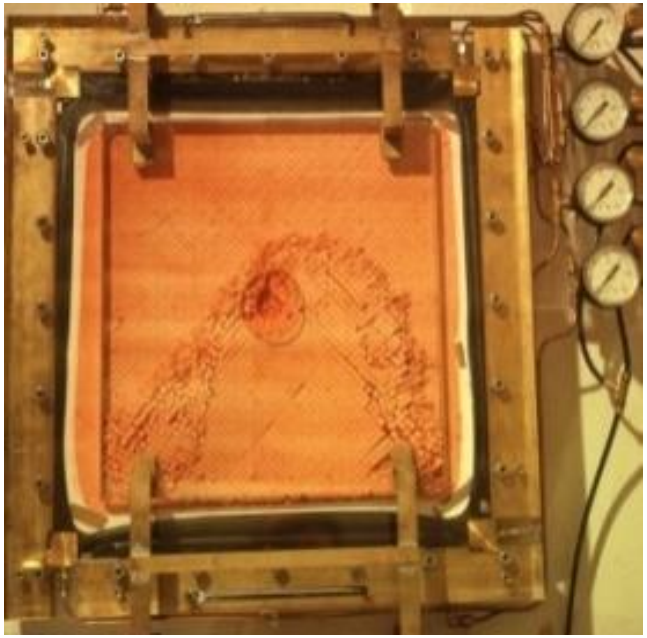
THEREFORE NOT SO ‘SIMPLE’ AS

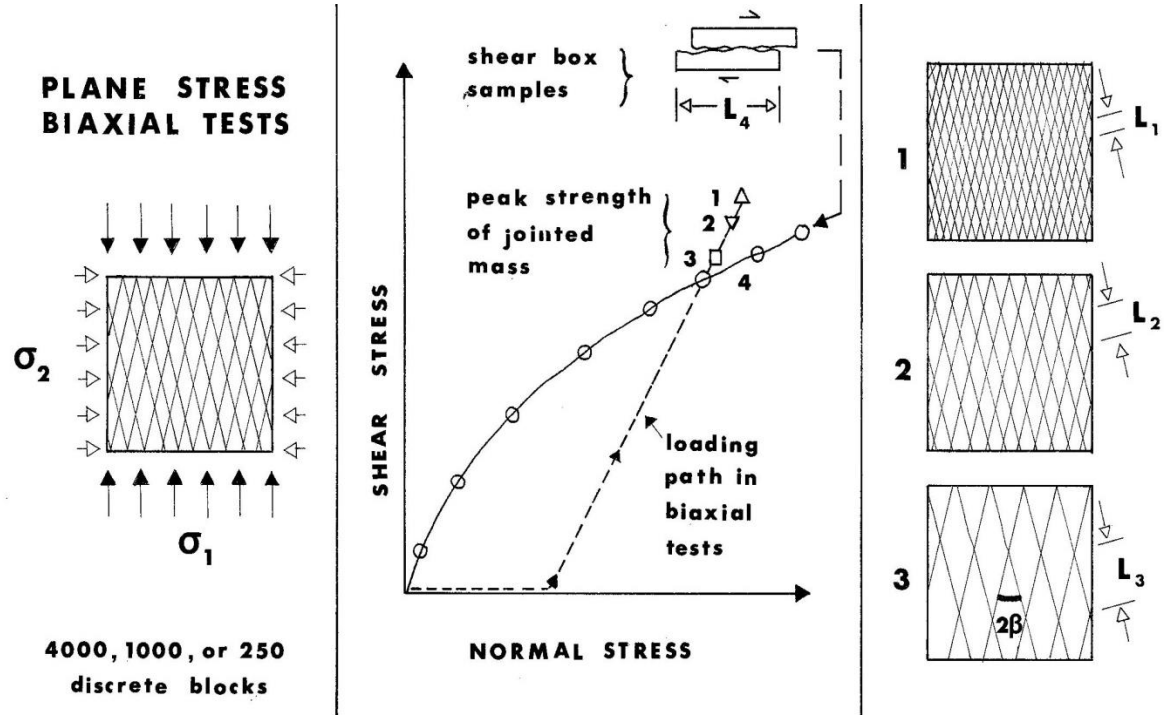
‘c’ + σ_n tan ‘φ’

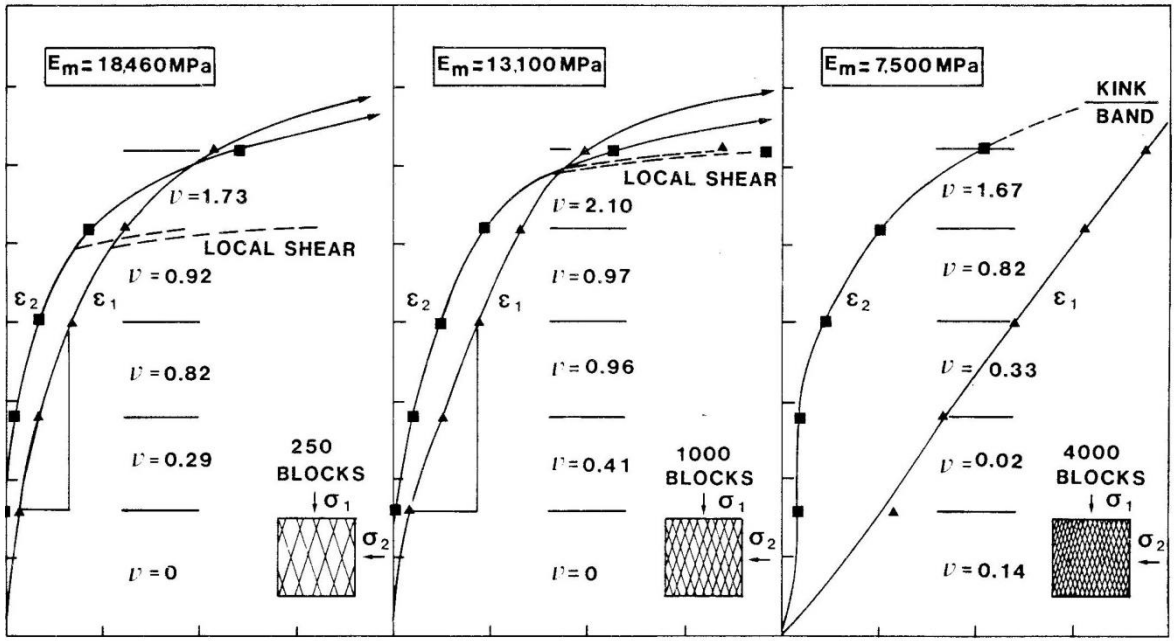
(whether linear or non-linear)

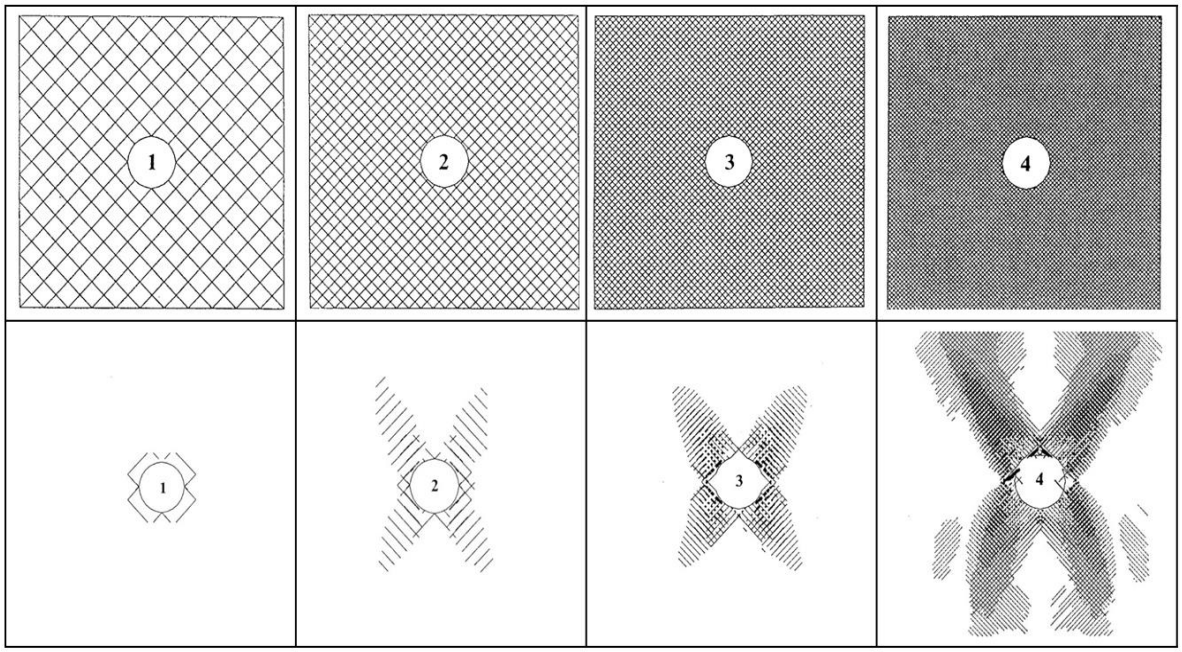


SHEAR STRENGTH OF 2D 'ROCK MASSES' APPEARS TO BE BLOCK-SIZE DEPENDENT. SMALL BLOCKS GIVE HIGHER STRENGTH, EVEN THOUGH (WEAK) FRACTURE ZONES ARE OFTEN ASSOCIATED WITH LOWER STRENGTH: DUE TO LOW Jr, HIGH Ja (i.e. CLAY-FILLINGS)









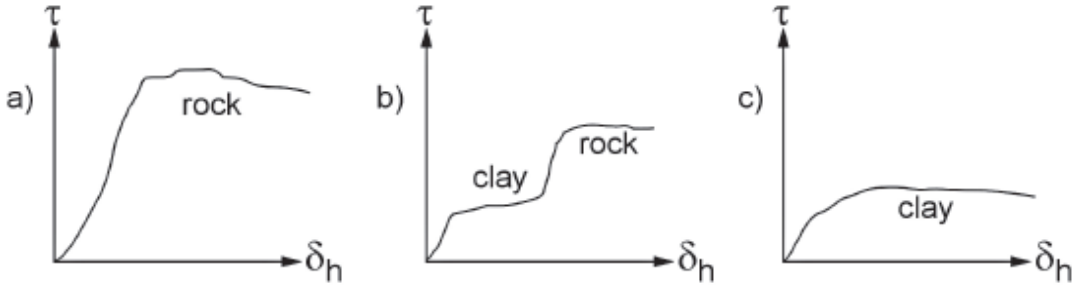
2D PHYSICAL MODELS (LIKE UDEC-BB) SHOW VARIOUS 'NON-CONTINUUM' CHARACTERISTICS:

INCREASING 'POISSON RATIO' (>> 0.4999)

BLOCK ROTATIONS

TRANSLATIONAL SHEARING

LINEARIZED STRESS-'STRAIN' IF SMALL BLOCKS



(a) Rock wall contact

(thin coatings)

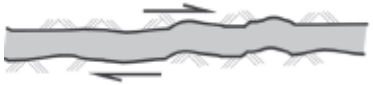
	Ja/Jr =	0.75	1.0	2	3	4
		$\tan^{-1}(Jr/Ja)°$				
A. Discontinuous joints	4	79°	76°	63°	53°	45°
B. Rough, undulating	3	76°	72°	56°	45°	37°
C. Smooth, undulating	2	69°	63°	45°	34°	27°
D. Slickensided, undulating	1.5	63°	56°	37°	27°	21°
E. Rough, planar	1.5	63°	56°	37°	27°	21°
F. Smooth, planar	1.0	53°	45°	27°	18°	14°
G. Slickensided, planar	0.5	34°	27°	14°	9.5°	7.1°

(b) Rock wall contact when sheared

(thin fillings)

	Ja/Jr =	4	6	8	12
		$\tan^{-1}(Jr/Ja)°$			
A. Discontinuous joints	4	45°	34°	27°	18°
B. Rough, undulating	3	37°	27°	21°	14°
C. Smooth, undulating	2	27°	18°	14°	9.5°
D. Slickensided, undulating	1.5	21°	14°	11°	7.1°
E. Rough, planar	1.5	21°	14°	11°	7.1°
F. Smooth, planar	1.0	14°	9.5°	7.1°	4.7°
G. Slickensided, planar	0.5	7°	4.7°	3.6°	2.4°

(c) No rock wall contact when sheared



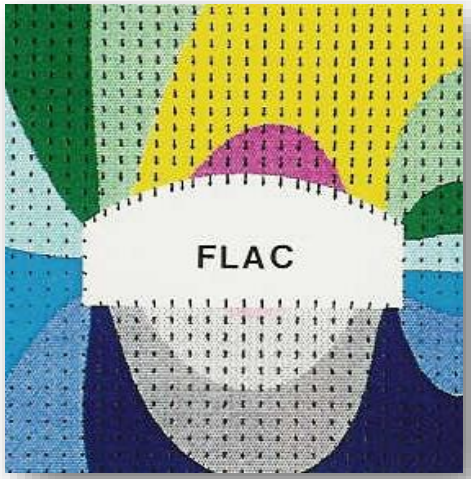
(thick fillings)

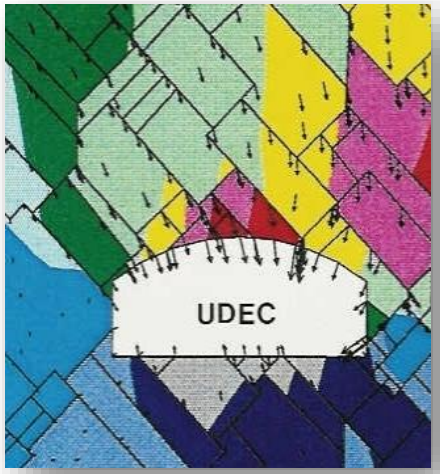
	Ja/Jr =	5	6	8	12	13	16	20
		$\tan^{-1}(Jr/Ja)°$						
Nominal roughness of discontinuity rock walls	1.0	11.3°	9.5°	7.1°	4.8°	4.4°	3.6°	2.9°

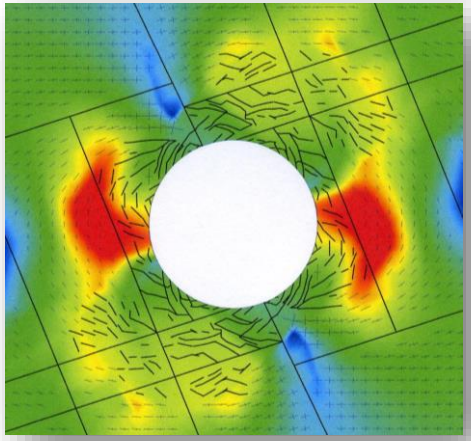
THE MUCH REDUCED SHEAR STRENGTH OF CLAY-FILLED DISCONTINUITIES

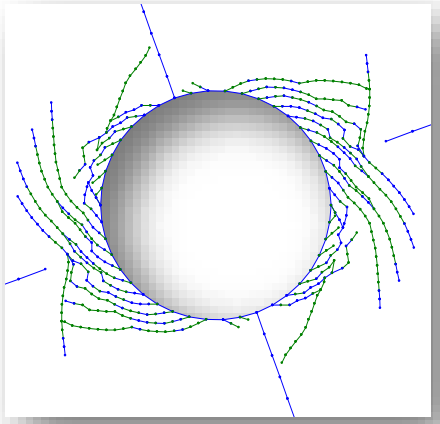
is the most serious (FOURTH) component

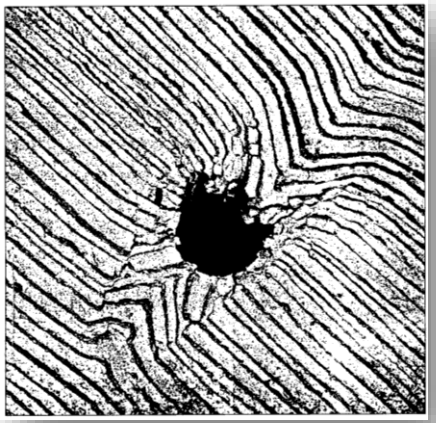
6. NUMERICAL MODELLING CHALLENGES…CONTINUUM ??









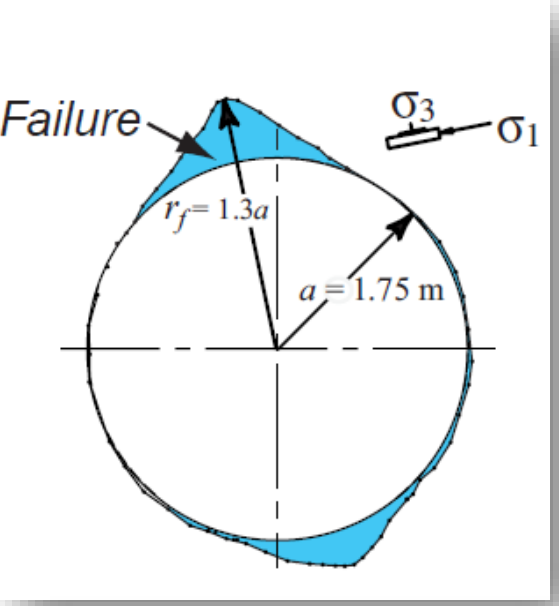


DO WE LEARN MUCH FROM CONTINUUM ANALYSES ?

IN MINING THE SCALE OF THE PROBLEM FIRST *DEMANDS* A CONTINUUM APPROACH

–but prefer

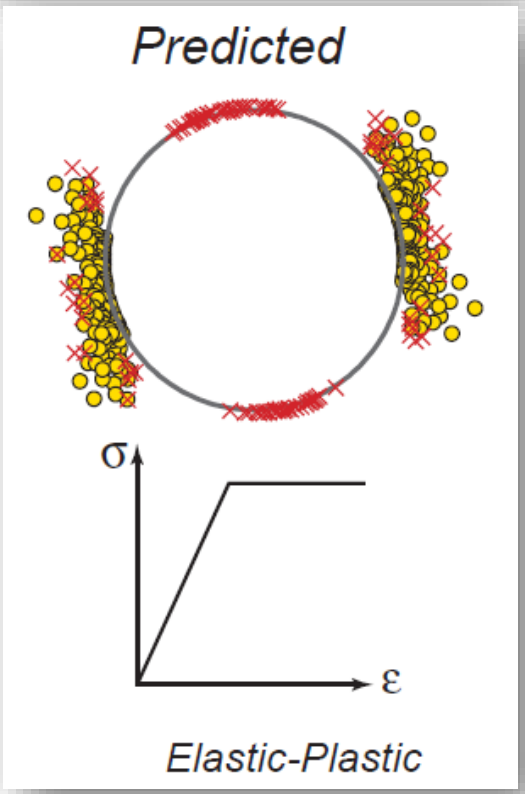
***'c then tan φ'* not M-C, not H-B.**



REASON:

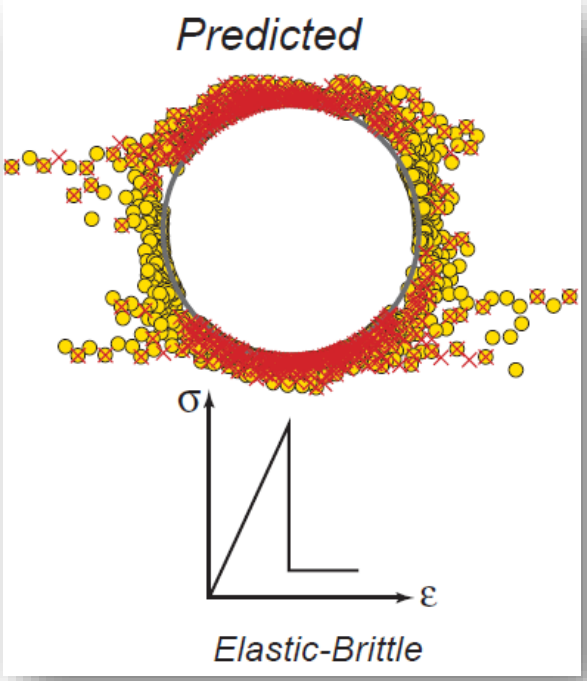
CONVENTIONAL continuum modelling methods.

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

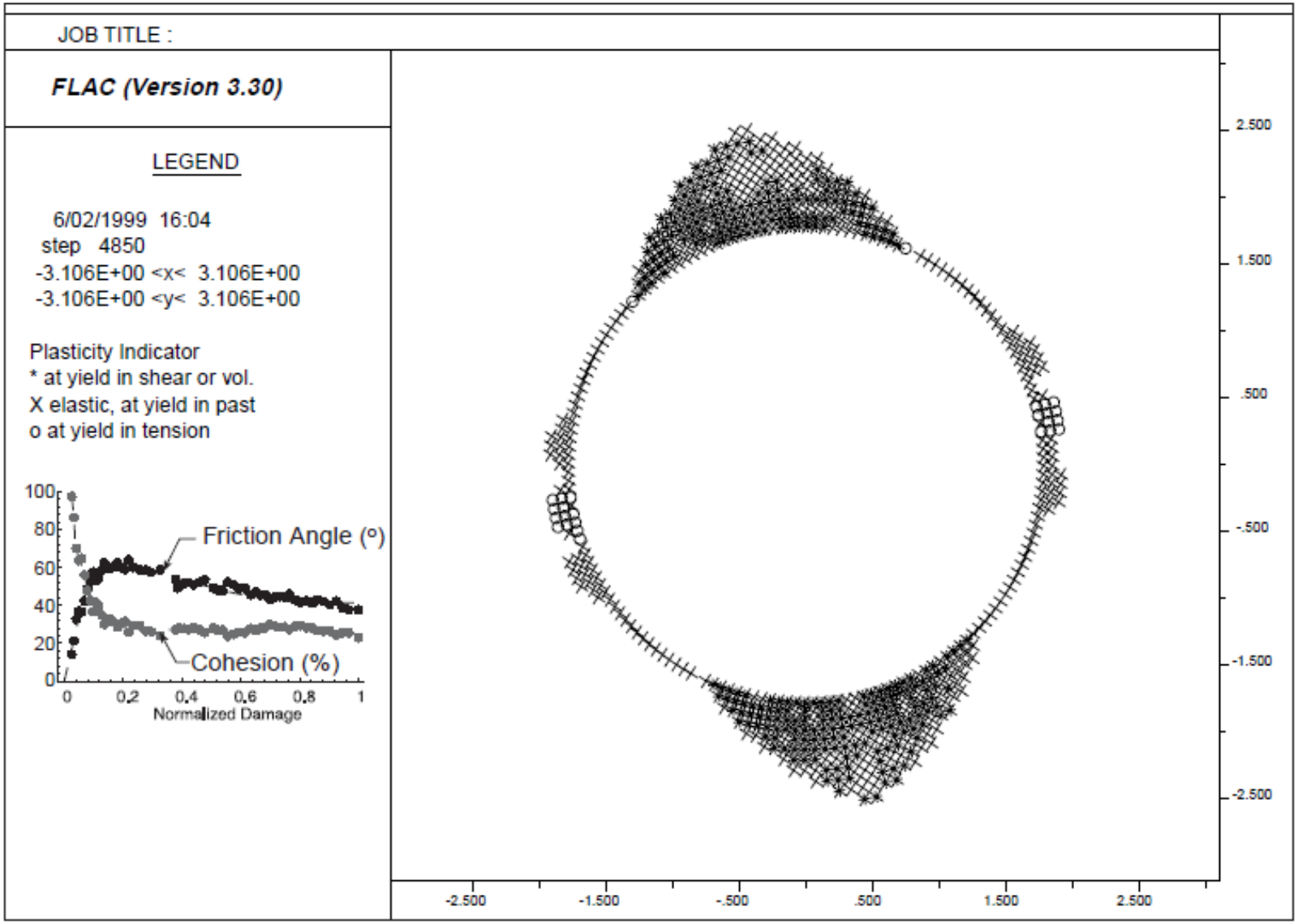


(Hajiabdolmajid, Martin and Kaiser, 2000 "Modelling brittle failure", NARMS)

So why performed by so many consultants?



× Shear failure ○ Tensile failure



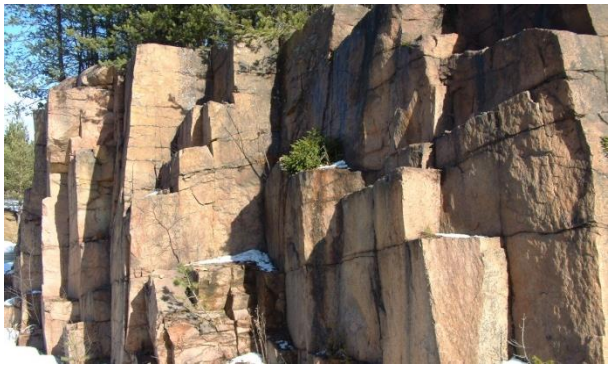
Degrade cohesion, mobilize friction: excellent match.

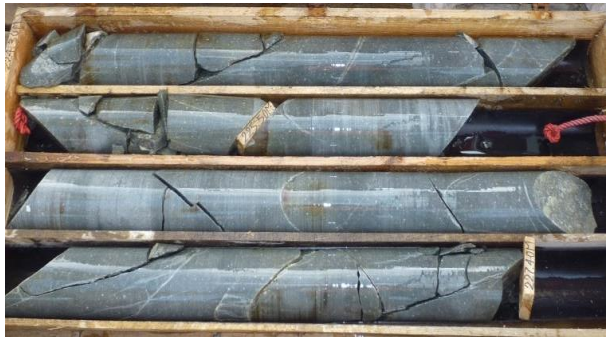
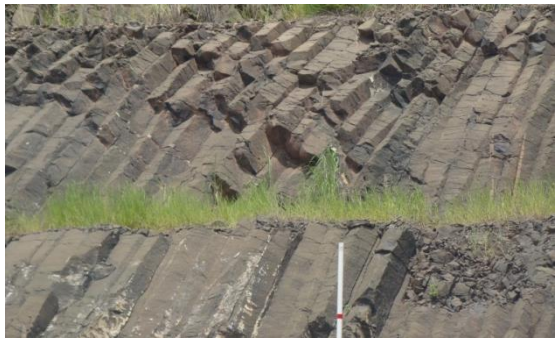
(Hajiabdolmajid, Martin and Kaiser, 2000 "Modelling brittle failure", NARMS.)

WHY NOT USE ANOTHER COMMON SOURCE OF MINE-SITE INPUT DATA …….Q ?

(which includes fundamentals like ***stress and number of joint sets !***)

(UNLIKE OTHER METHODS)



Q HAS AN ADVANTAGE: IT'S VERY LARGE NUMERICAL RANGE

(HERE 'ONLY' 10 DOWN TO 0.001 ..... ARGUABLY BETTER THAN 65-5)

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

0.001→1000, or 5→95 ?

CC and FC from Qc = Q x σ_c /100 :

Qc = RQD/Jn x Jr/Ja x Jw /SRF x σ_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'

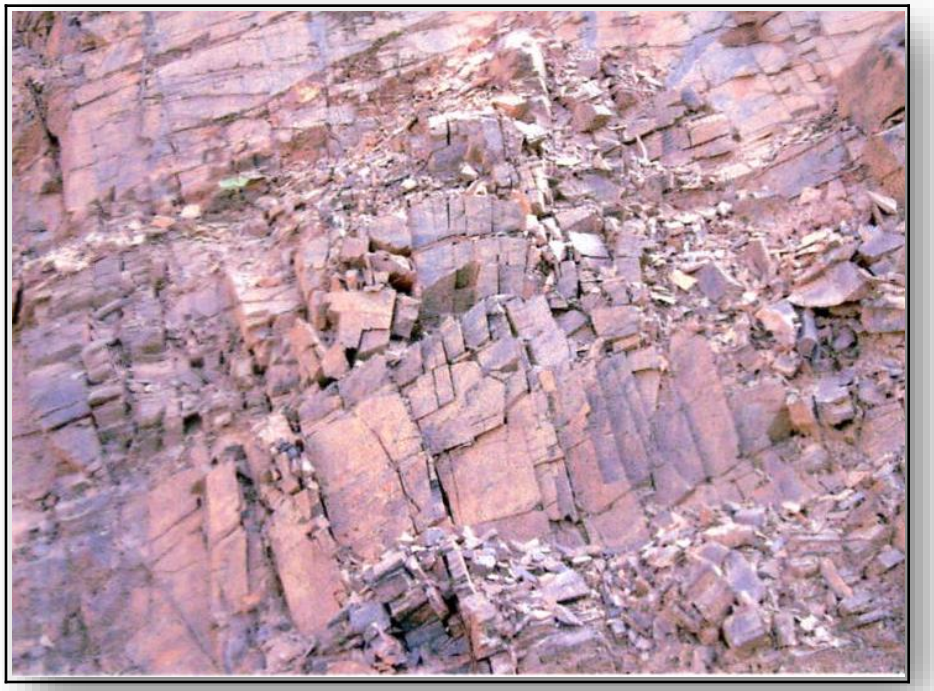
$$CC = \frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100}$$

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

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

$$E_m\,(GPa) = \left(1-\frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \times 10^{(GSI-10)/40}$$

$$\sigma'_{cm} = \sigma_{ci} \times \frac{(m_b+4s-a(m_b-8s))\,(m_b/4+s)^{a-1}}{2(1+a)\,(2+a)}$$

$$\varphi' = a\sin\left[\frac{6am_b\,(s+m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a)+6am_b\,(s+m_b\sigma'_{3n})^{a-1}}\right]$$

$$c' = \frac{\sigma_{ci}[(1+2a)s+(1-a)m_b\sigma'_{3n}]\,(s+m_b\sigma'_{3n})^{a-1}}{(1+u)\,(2+a)\sqrt{1+(6am_b(s+m_b\sigma'_{3n})^{a-1})/((1+a)\,(2+a))}}$$

where

$$\sigma_{3n} = \sigma'_{3\,\max}/\sigma_{ci}\ \ (+\mathrm{GSI}+a+s+m_b \text{ relations})$$

$$E_m \approx 10 \times Q_c^{1/3}$$

$$\sigma_{cm} \approx 5\gamma\, Q_c^{1/3}$$

$$\varphi \approx \tan^{-1}\left(\frac{J_r}{J_a} \times \frac{J_w}{1}\right)$$

$$c \approx \left(\frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100}\right)$$

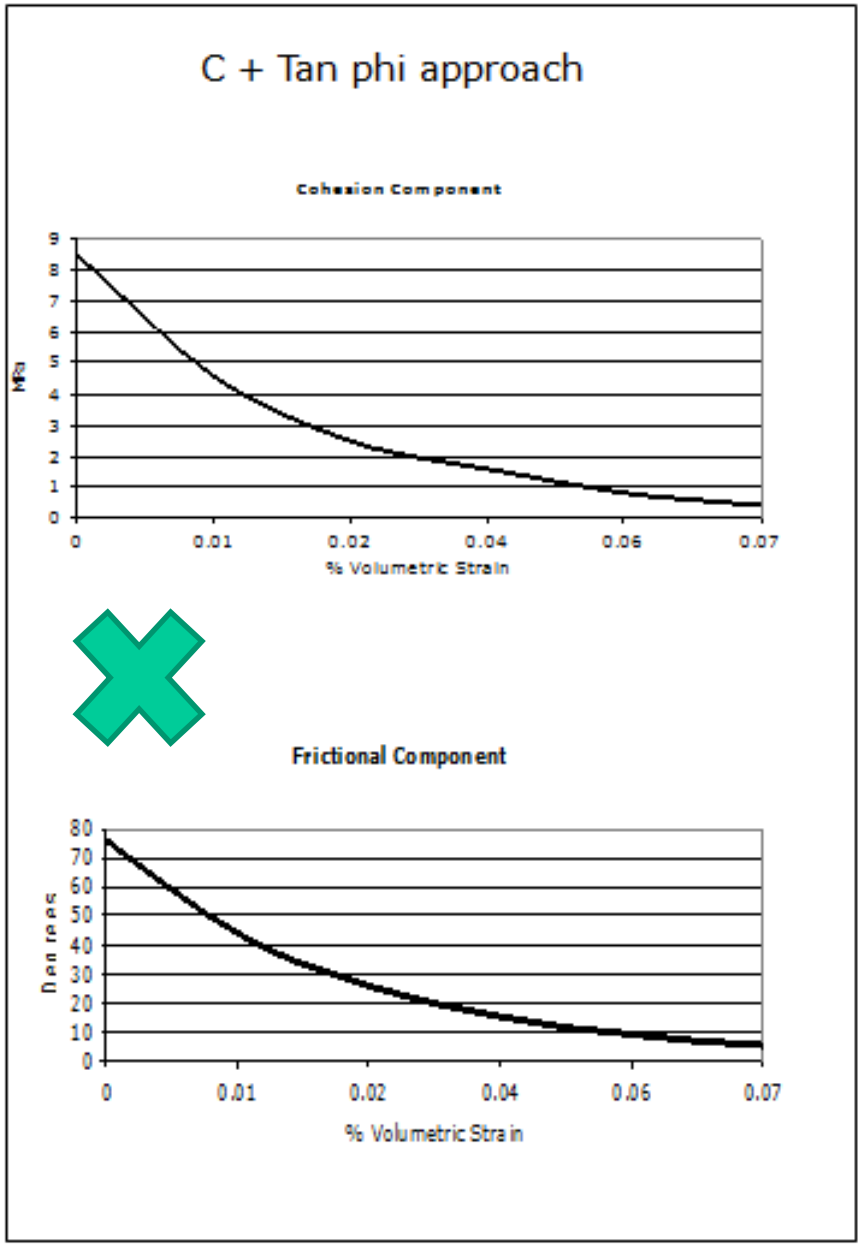
SO FOR THOSE WHO ARE SUSPICIOUS OF BLACK-BOX EQUATIONS – THERE ARE TRANSPARENT ALTERNATIVES.

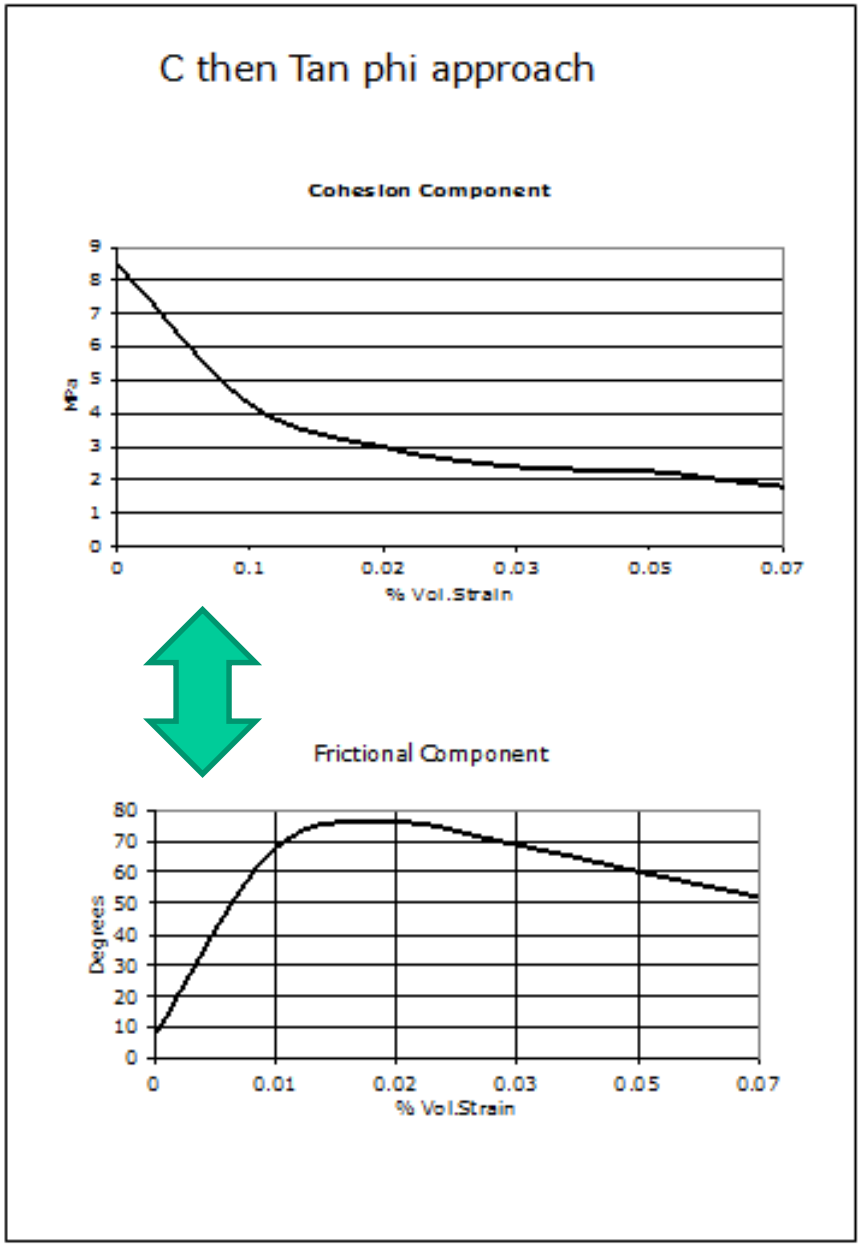
$$c' = \frac{\sigma_{ci}\left[(1+2a)s+(1-a)m_b\sigma'_{3n}\right]\left(s+m_b\sigma'_{3n}\right)^{a-1}}{(1+u)(2+a)\sqrt{1+\left(6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}\right)\Big/\left((1+a)(2+a)\right)}}$$

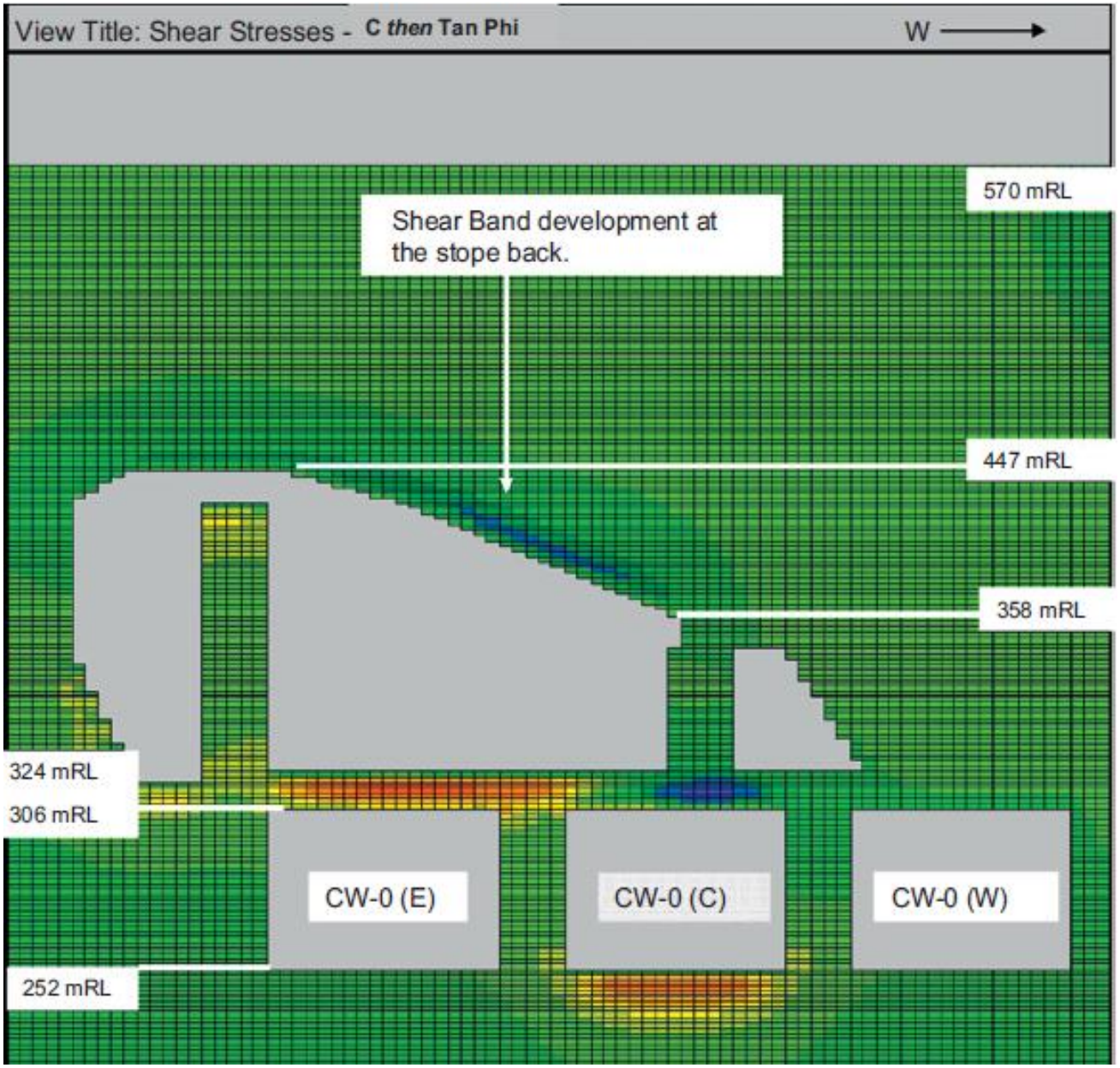
$$\phi' = a\sin\left[\frac{6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}{2(1+a)(2+a)+6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}\right]$$

'The non-linear Hoek-Brown criterion, based previously on valid empirical data from numerous tests on *intact rock*, has been algebraically adjusted, in order to apparently 'work' for rock masses. This has led to many artificial 'plastic zones', also for stable tunnels. Do these equations look as if they belong in rock engineering approximation'?

‘c *then* σ_n tan phi’ (as in Barton and Pandey, 2011)



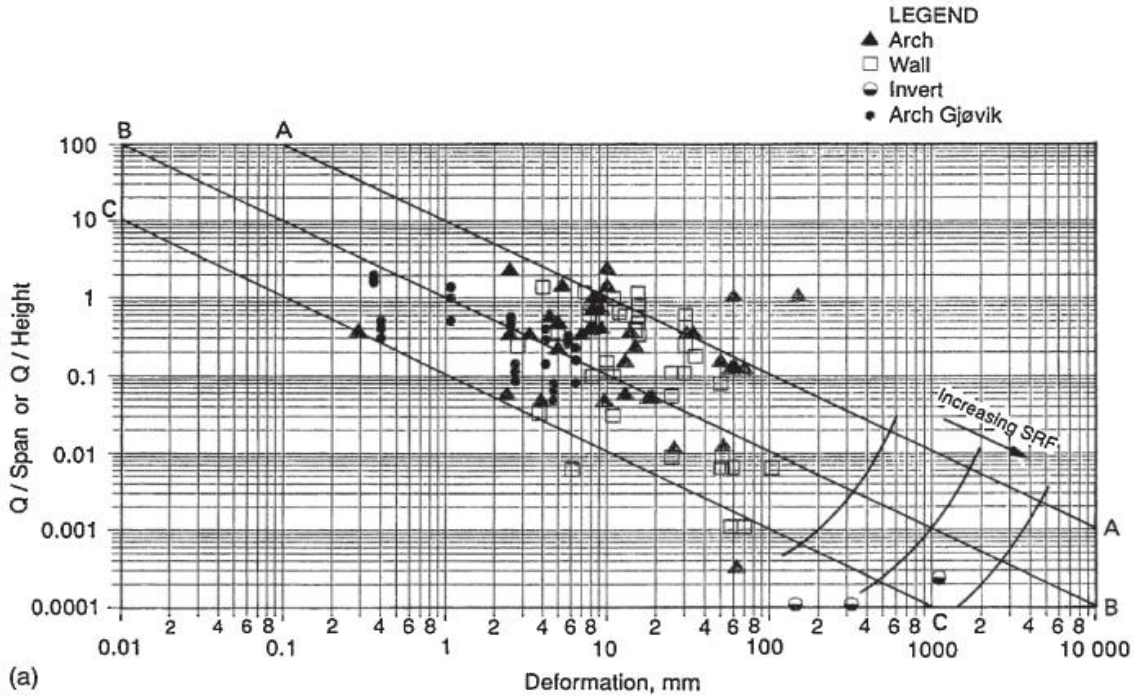


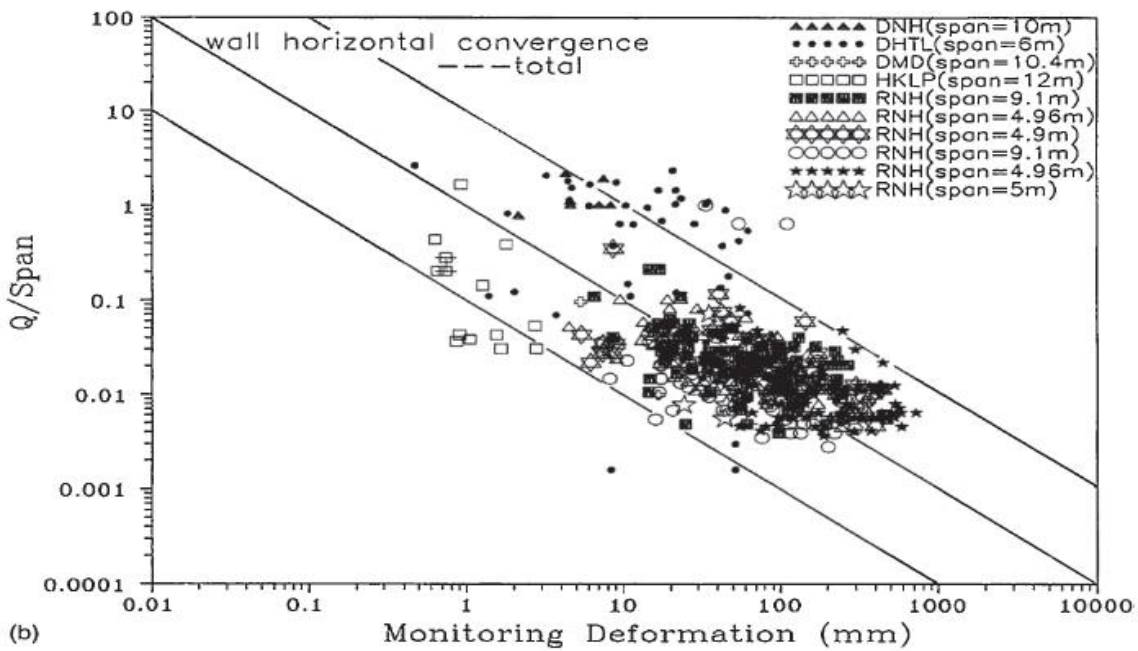


Modelling as ***'c then tan φ'*** causes subtle changes in results

ONE CAN, AND SHOULD, CHECK FOR THE REALISM OF MODELLED DEFORMATION.

REALITY MAY BE DIFFERENT FROM WHAT ONE ASSUMES IS CORRECT





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

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

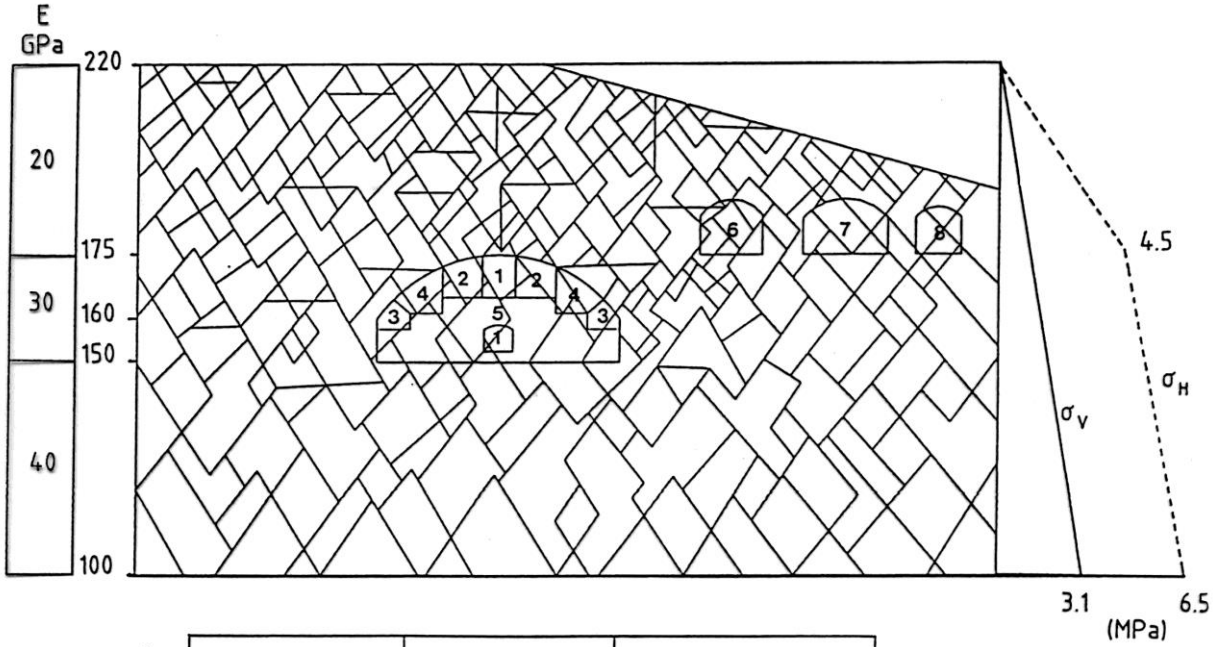
$$\Delta_h = \frac{\text{HEIGHT}}{100\,Q}\sqrt{\frac{\sigma_h}{\sigma_c}}$$

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

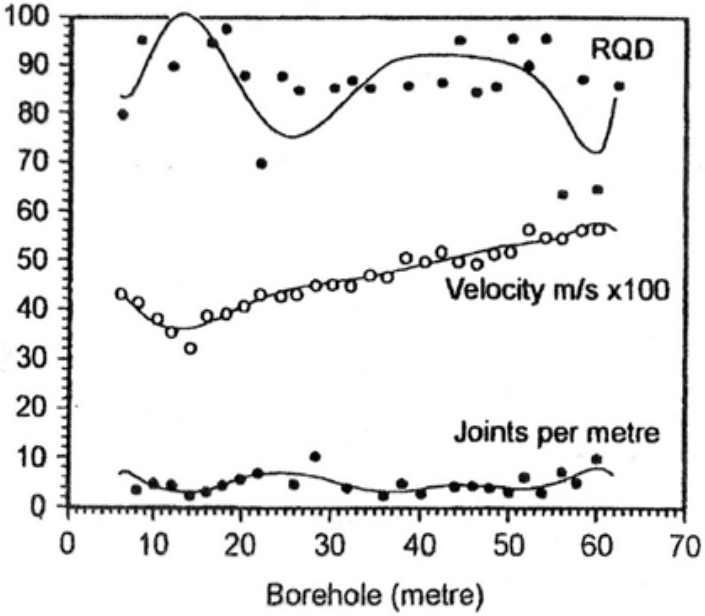
7. DEPTH DEPENDENT DEFORMATION MODULUS

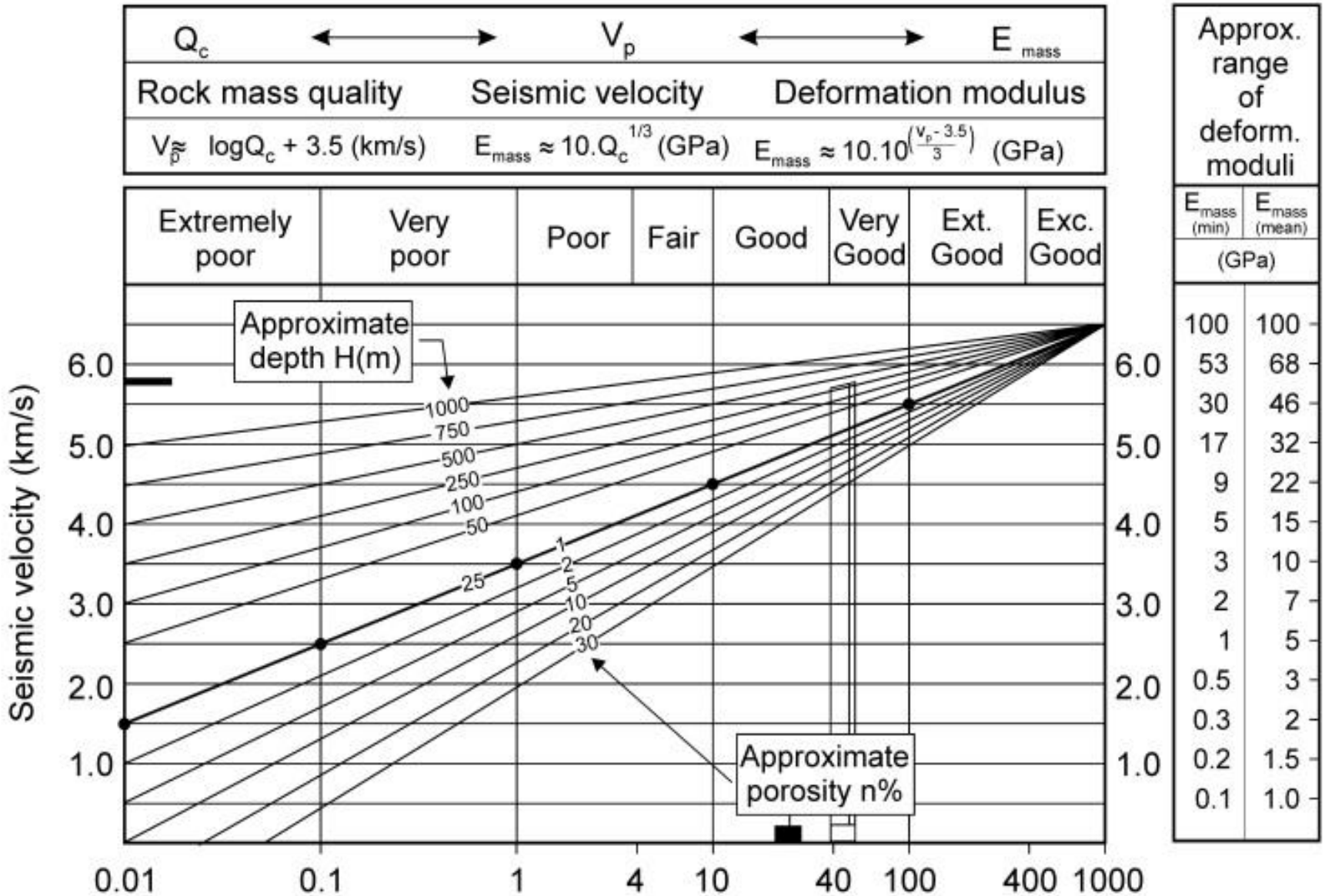
(this is needed because velocity also increases with depth)

V_P INCREASES DUE TO STRESS



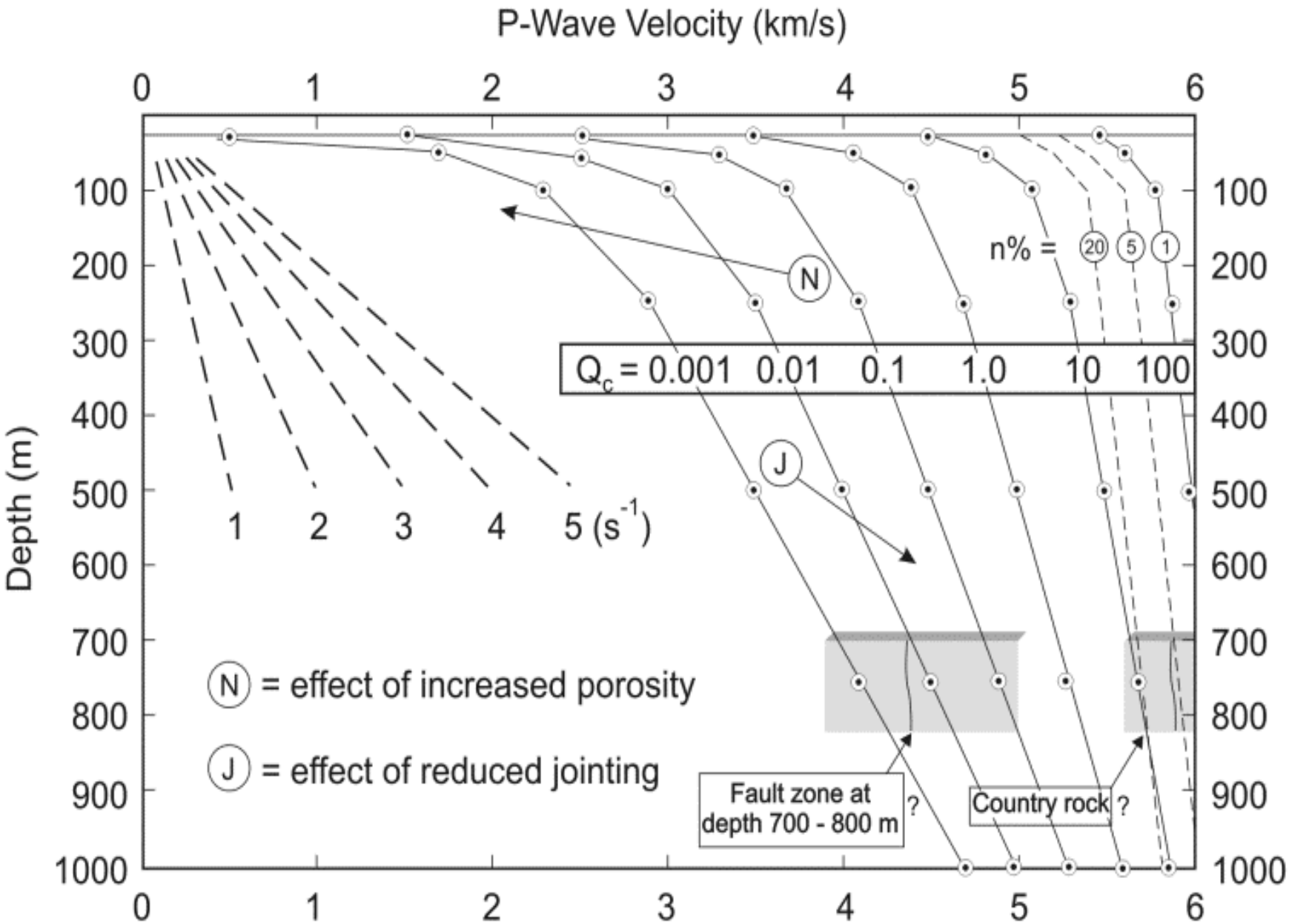
γ_z=0.026MPa/m	JRC_0=7.5 JCS_0=75MPa	Φ_r=27° i=6° } 33°





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

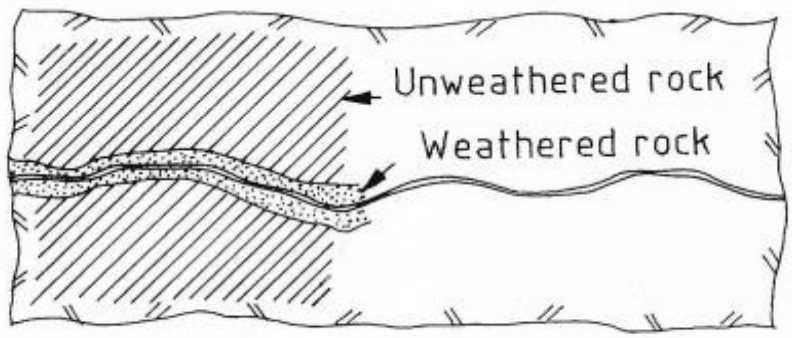
From Rock Quality, Seismic Velocity, Attenuation and Anisotropy. Barton, 2006



$$E_{mass} \approx 10^{(V_p - 2.5 + \log \sigma_c)/3}$$

V_p (km/s)	2.5	3.5	4.5	5.5	6.5
σ_c (MPa)	10	100	100	150	150
E_{mass} (GPa)	2.2	10.0	21.5	53.1	114

8. THE SHEAR STRENGTH OF ROCK DUMPS / ROCKFILL



ROCKJOINT

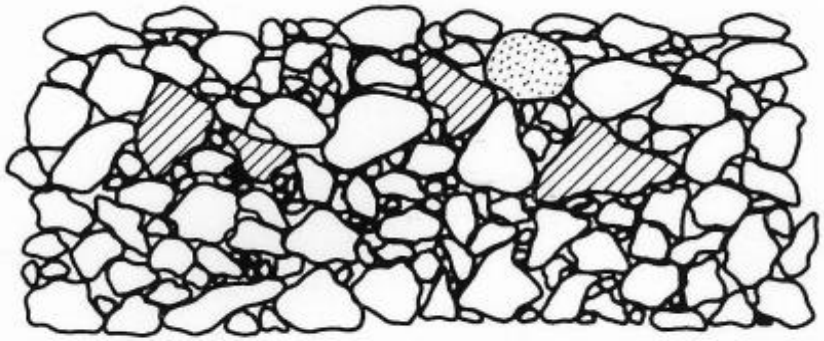
JCS (or σ_c)

JRC

Φ_r (or Φ_b)

weathered

unweathered

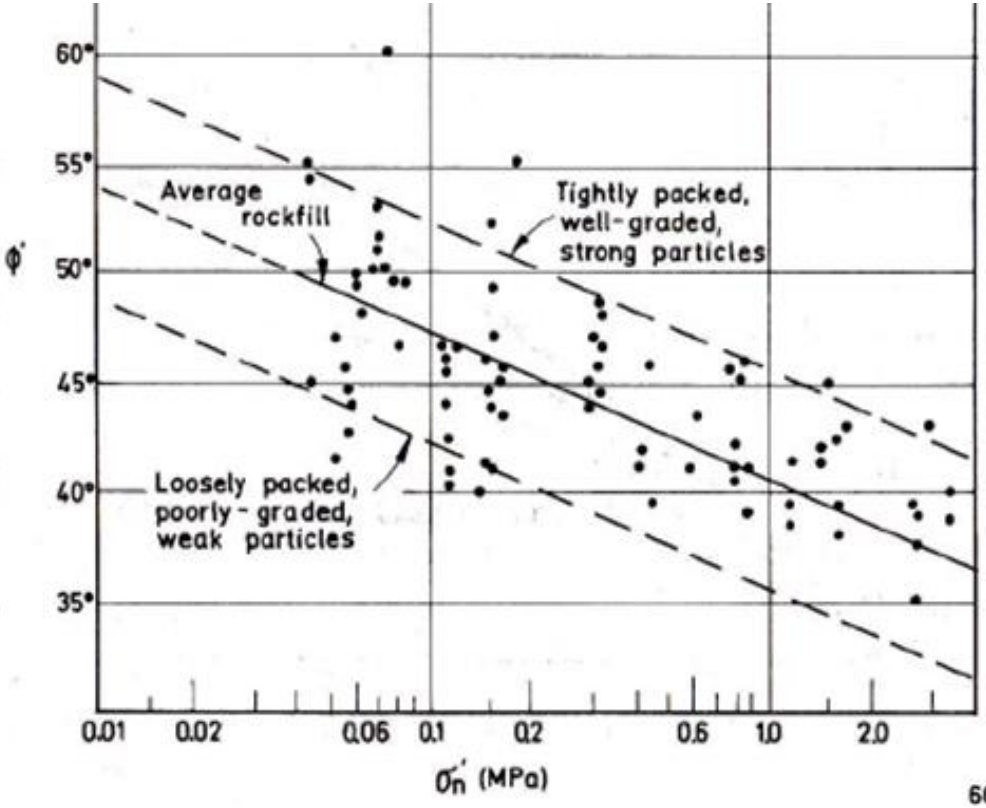


CRUSHED ROCK

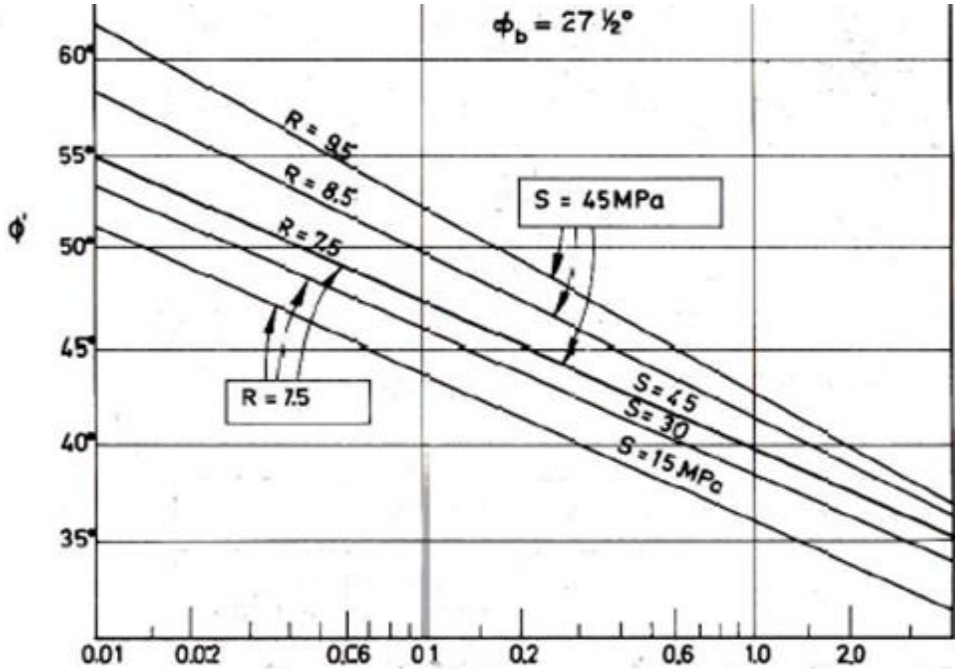
S (strength)

R (roughness)

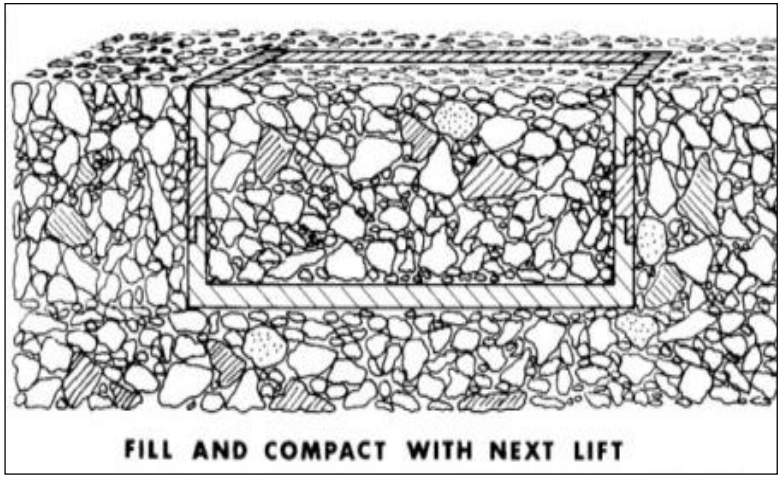
Φ_r (friction)

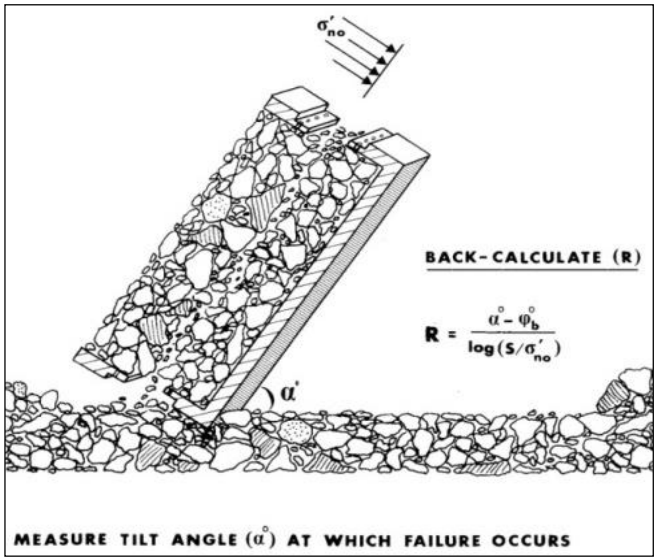


CAN MATCH WITH 'R' and 'S'

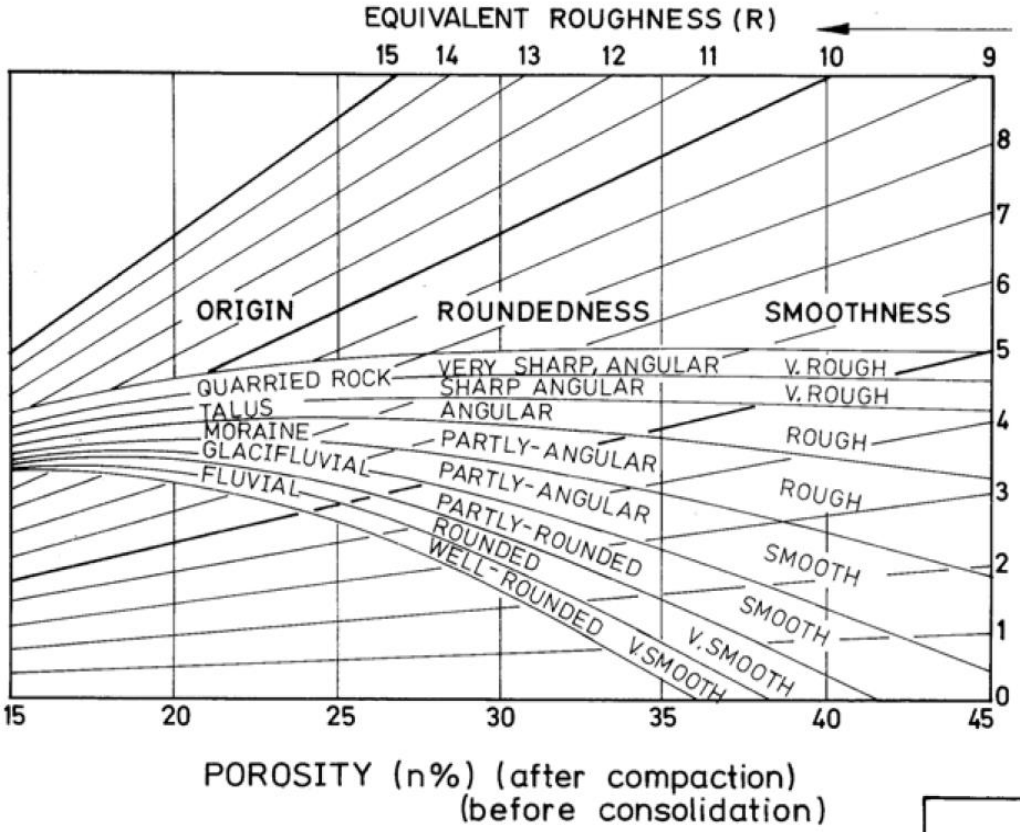


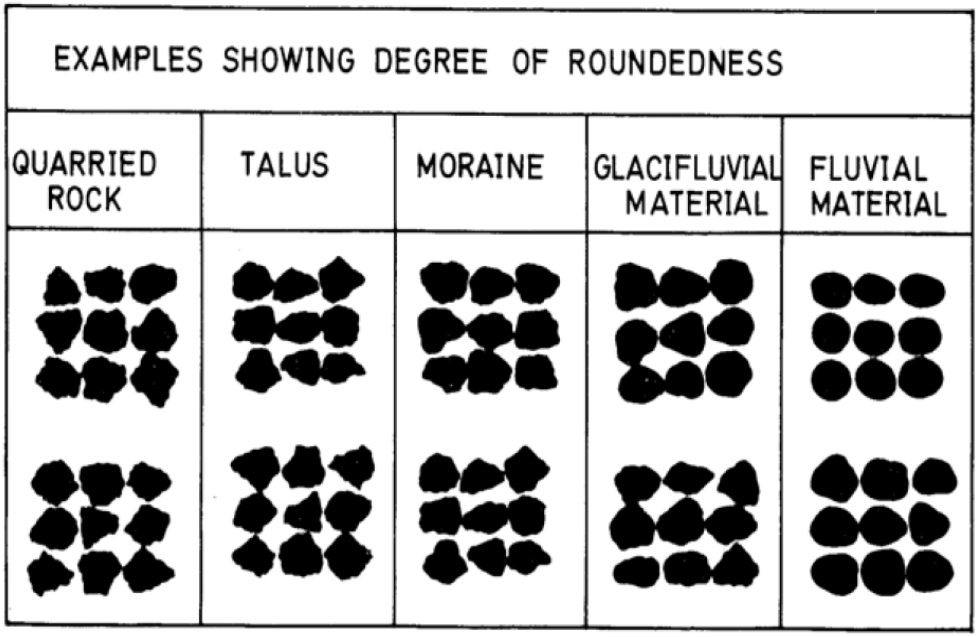
5m x 2m x 2m TILT TESTS ON COMPACTED FILL for back-calculation of R

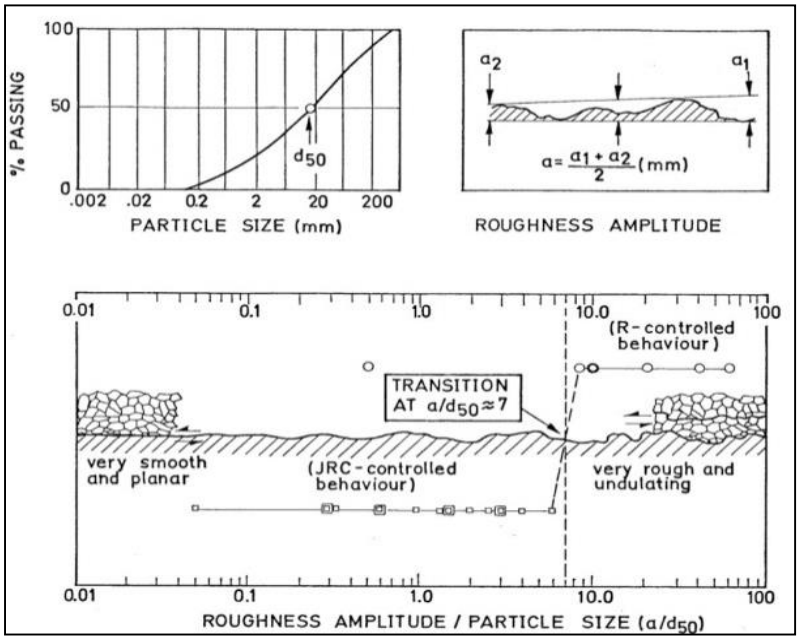




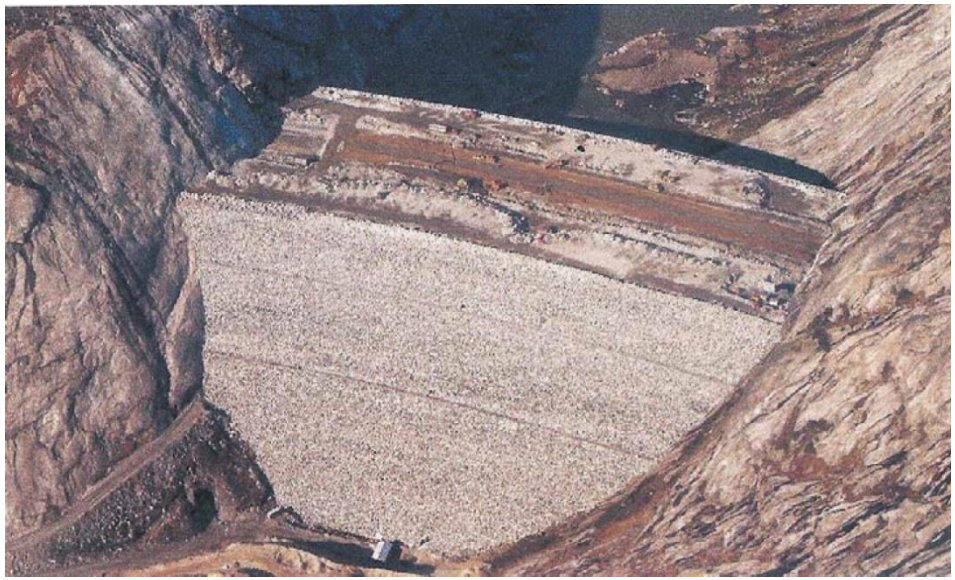
Estimating R from origin, roundedness, smoothness and POROSITY







INTERFACE STRENGTH AN ISSUE? ('Dam' photos: Linero, NGI)

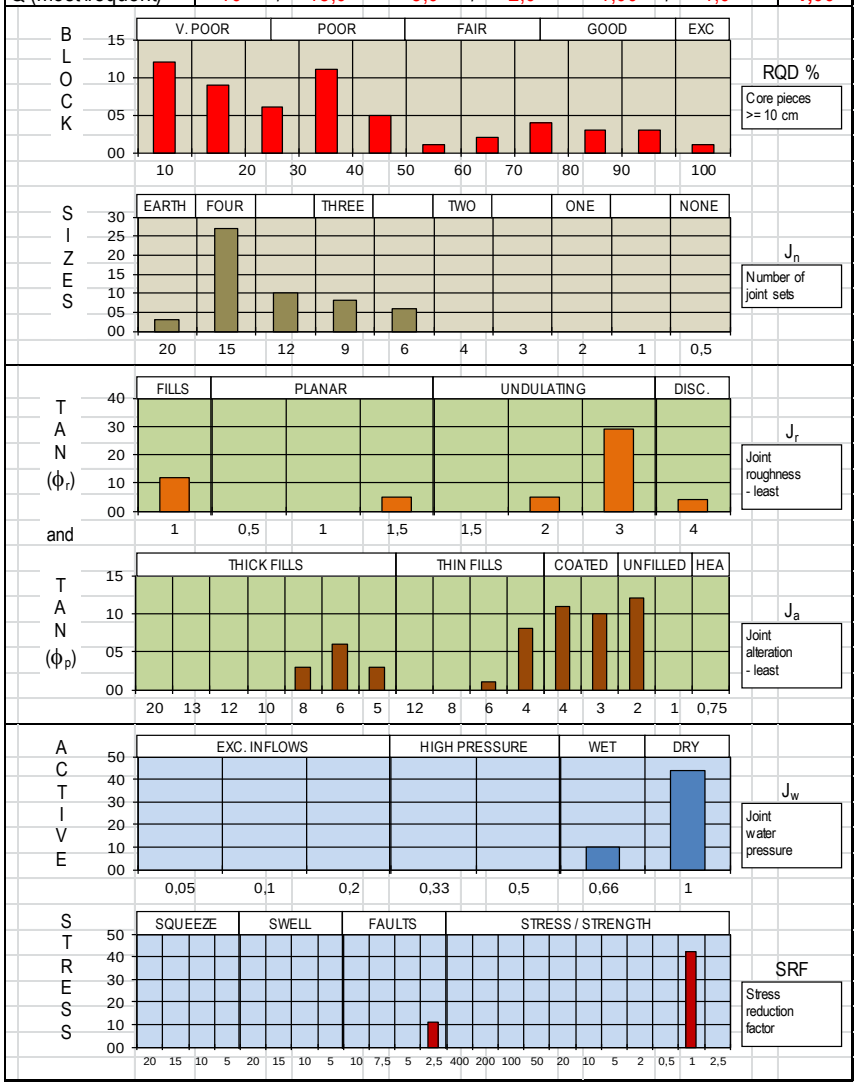


9. MINE ZONATION WITH Q

(SOME OF THE Q-PARAMETERS HAVE IMPORTANT SEPARATE ROLES)

Q IS ONLY PART OF A ROCK MASS DESCRIPTION EXERCISE

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



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

Key	Location:	Depth / chainage:	Date:
			Page:

Numbers for domains, core boxes, tunnel lengths (under-line, or specify)	Q (typical range) = ____	Q (mean) = ____	Q (most freq.) = ____
	(____)x(____)x(____)	(____)x(____)x(____)	(____)x(____)x(____)

BLOCK SIZES

Very Poor	Poor	Fair	Good	Exc.
0 10 20 30 40	50 60 70 80	90	100	100

RQD %

Core pieces ≧ 10 cm

Earth	Four		Three		Two		One		None
20	15	12	9	6	4	3	2	1	0.5

J_n

Number of joint sets

1=

2=

3=

4=

5=

6=

7=

8=

9=

FRICTION

Fills	Planar			Undulating			Disc.
1.0	0.5	1	1.5	1.5	2	3	4

J_r

Joint roughness - least favourable

Thick Fills							Thin Fills				Coated		Unfilled		Heal
20	13	12	10	8	6	5	12	8	6	4	4	3	2	1	0.75

J_a

Joint alteration - least favourable

Photos

or Sketch

ACTIVE STRESS

Exc. Inflows			High Press		Wet	Dry
0.05	0.1	0.2	0.33	0.5	0.66	1

$\frac{RQD}{J_a} =$ ____

$\frac{J_r}{J_a} =$ ____

(most freq.)

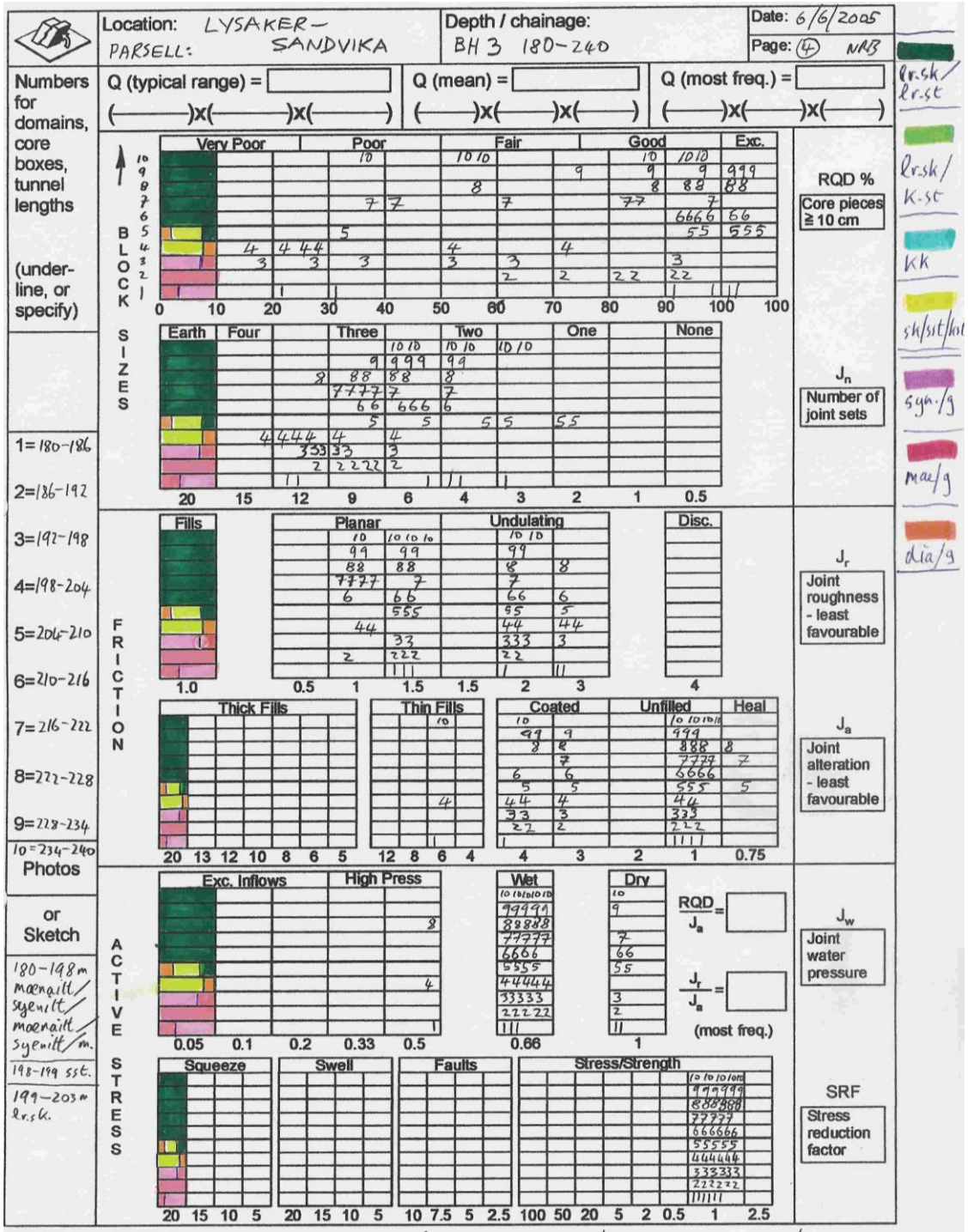
J_w

Joint water pressure

Squeeze				Swell				Faults				Stress/Strength							
20	15	10	5	20	15	10	5	10	7.5	5	2.5	100	50	20	5	2	0.5	1	2.5

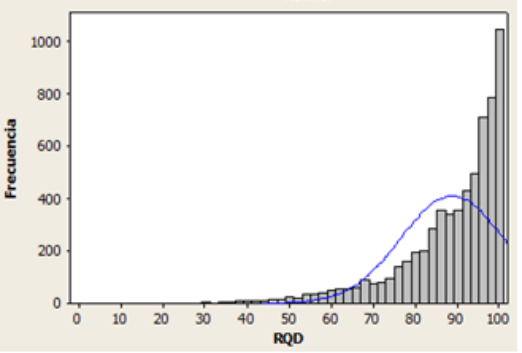
SRF

Stress reduction factor

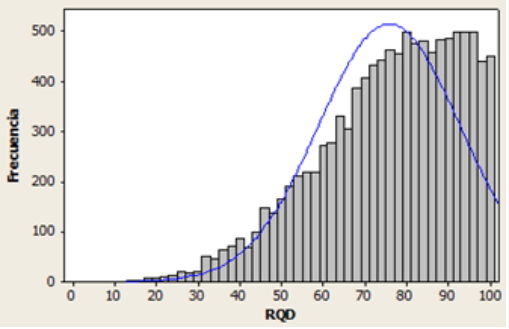


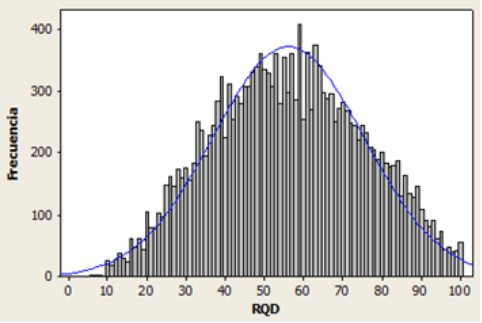
SUGGESTION FOR COLLECTING Q-PARAMETER DATA. FIRST CHARACTERIZE CORE. LATER CLASSIFY MINE OPENINGS.

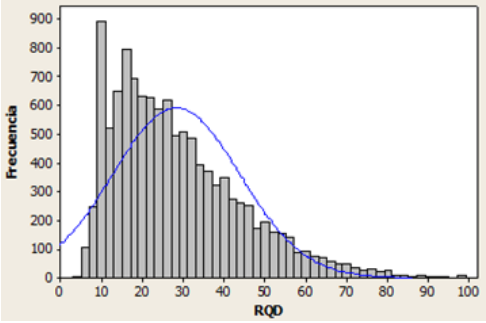
Q-classes 2, 3, *4* and 5, with respective Q-ranges 40-10, 10-4, *4-1,* 1-0.1.

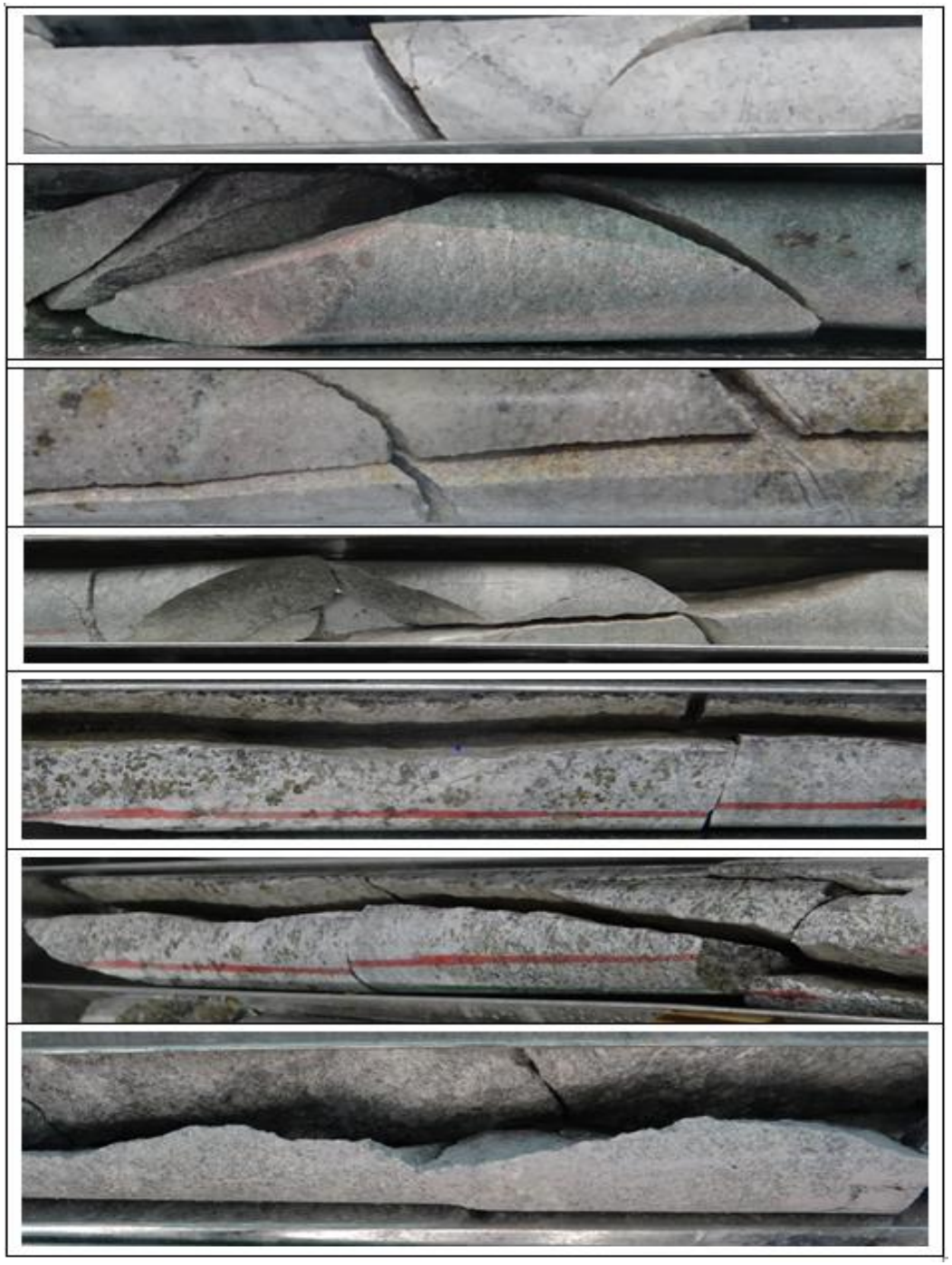


Demonstrates central role played by RQD in commonly experienced rock mass conditions. (> 40 km of core)



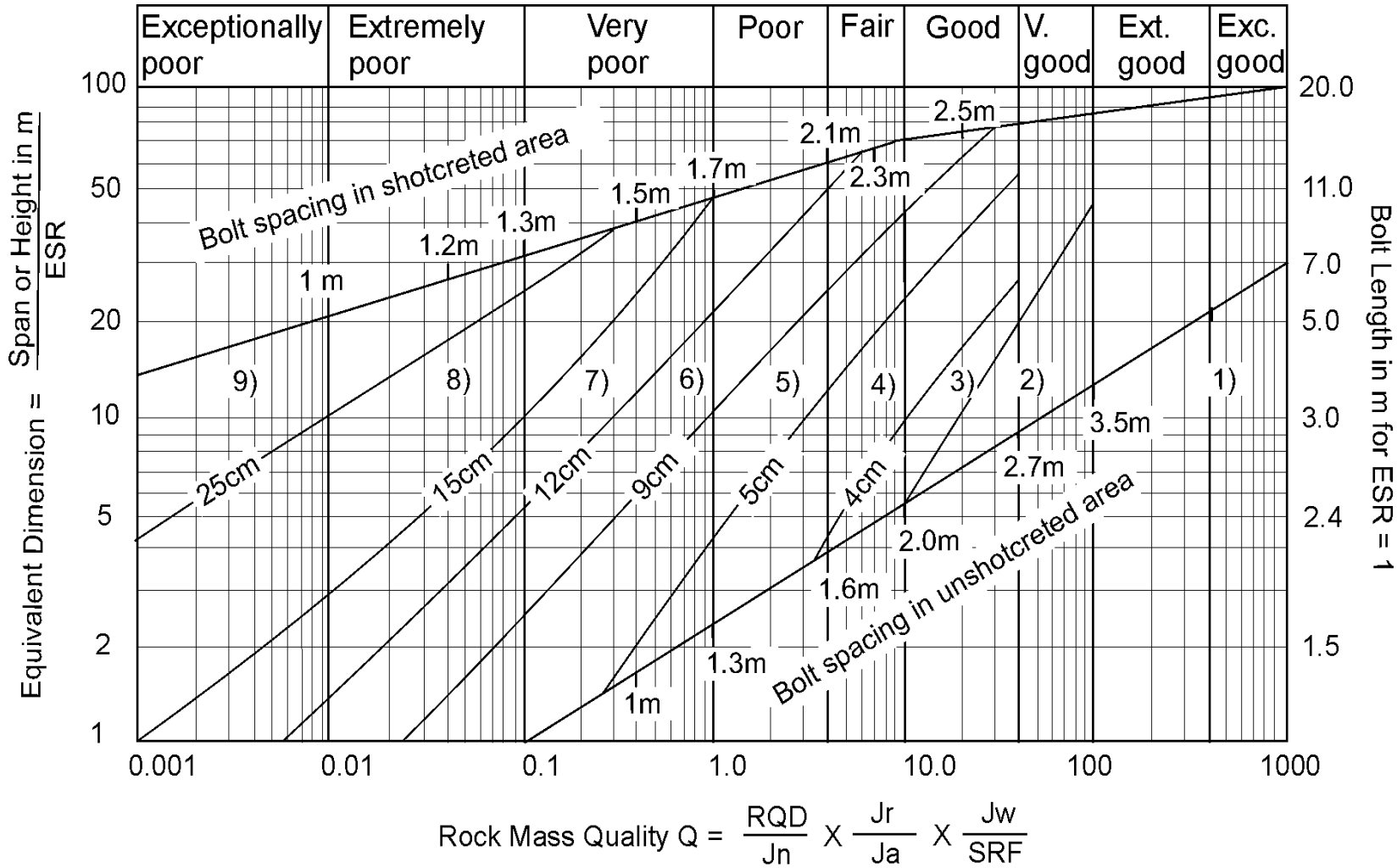


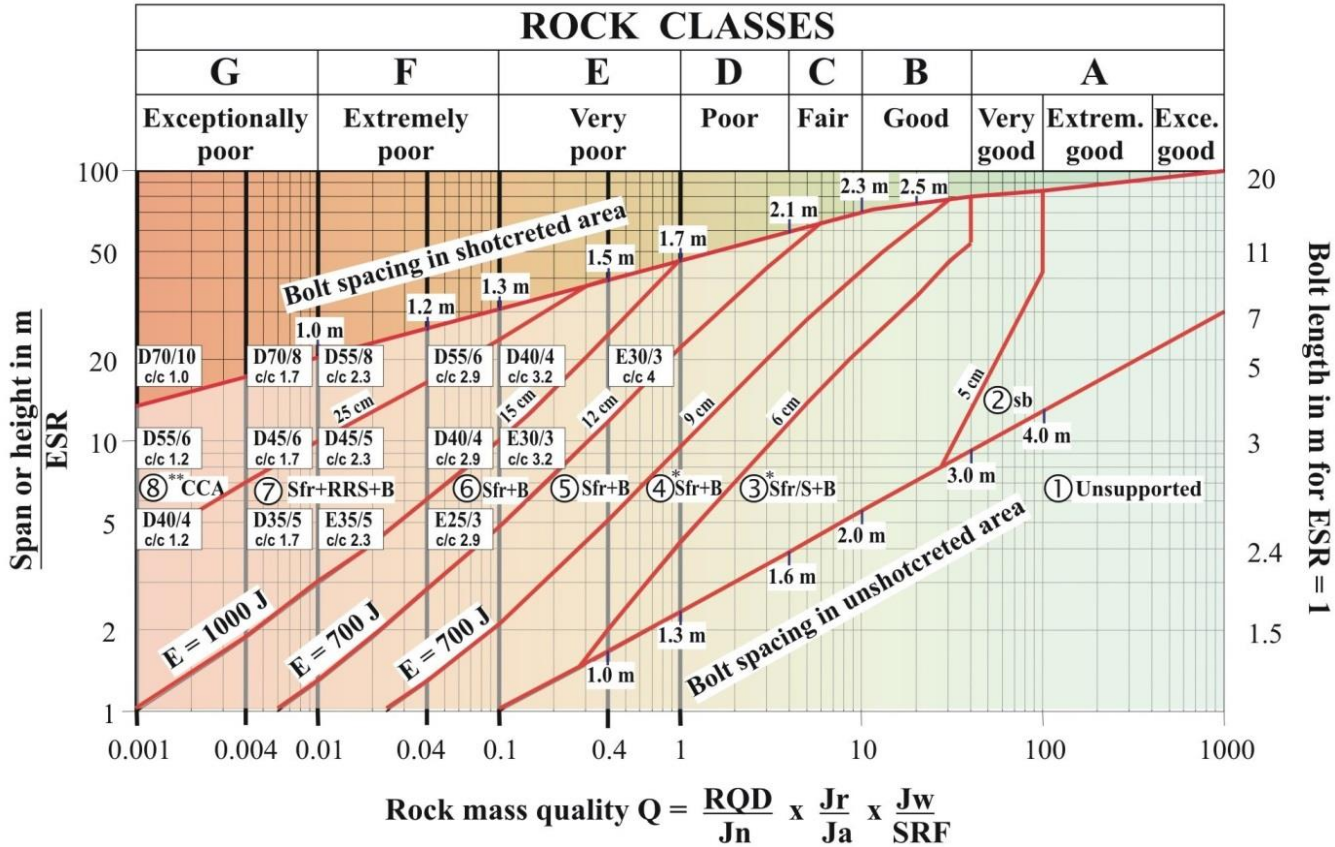


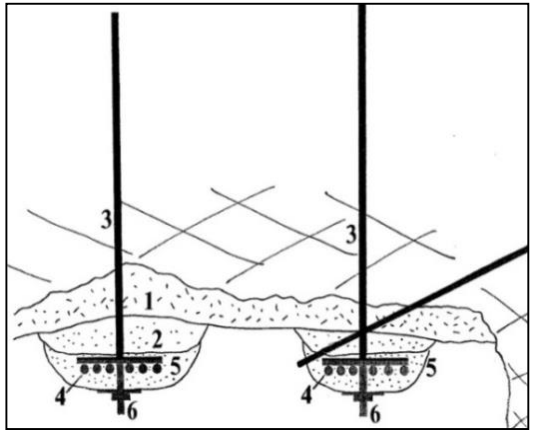


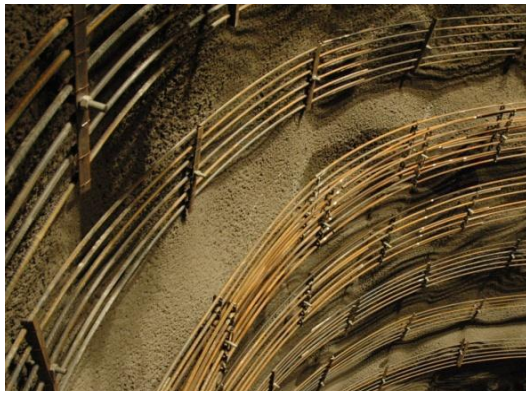
Photos of core with the following Jr values: Jr = 1.0 or 1.5, Jr = 1.5, Jr = 1.5, Jr = 1.5, Jr = 2, Jr = 2.5, Jr = 3.5

10. AN UPDATE ON Q-SUPPORT CHARTS *for SINGLE-SHELL NMT*









REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting, **sb**
3) Systematic bolting, and unreinforced or fibre reinforced shotcrete, 5-6 cm), **Sfr/B+S**
4) Fibre reinforced shotcrete and bolting, 6-9 cm, **Sfr+B**
5) Fibre reinforced shotcrete and bolting, 9-12 cm, **Sfr (E700) +B**
6) Fibre reinforced shotcrete and bolting, 12-15 cm, **Sfr (E700) +B**
7) Fibre reinforced shotcrete > 15 cm + reinforced ribs of shotcrete and bolting, **Sfr (E1000) +RRS+B**
8) Cast concrete lining, **CCA** or **Sfr (E1000) +RRS+B**

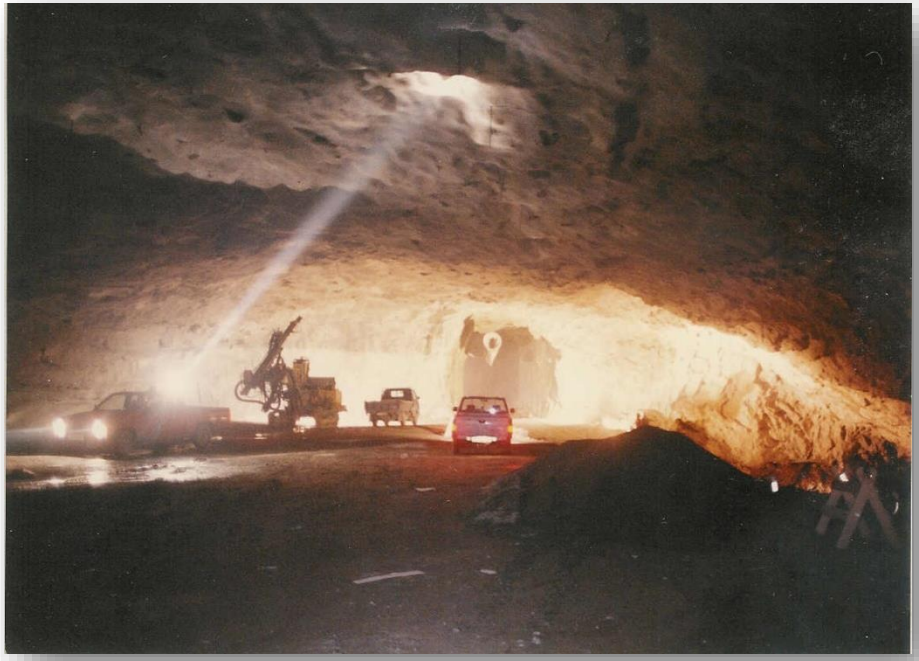
The bolts are 20 or 25 mm in diameter

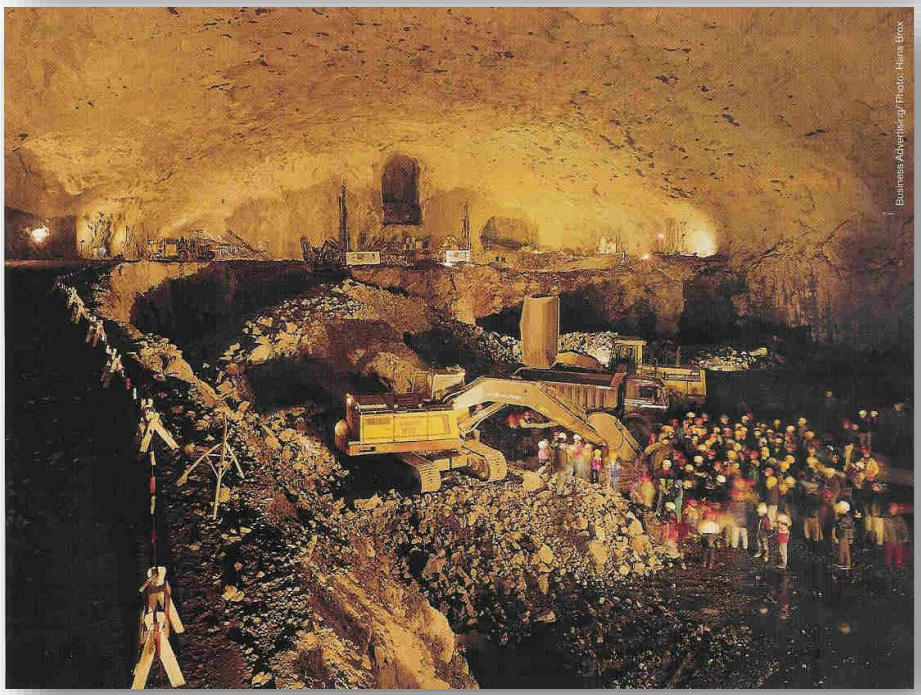
E) Energy absorbtion in fibre reinforced shotcrete at 25 mm bending during plate testing

D45/6 c/c 1.7 = RRS with totally 6 reinforcement bars in double layer in 45 cm thick ribs with centre to centre (c/c) spacing 1.7 m. Each box corresponds to Q-values on the left hand side of the box

*) Up to 10 cm in large spans
) Or **Sfr+RRS+B

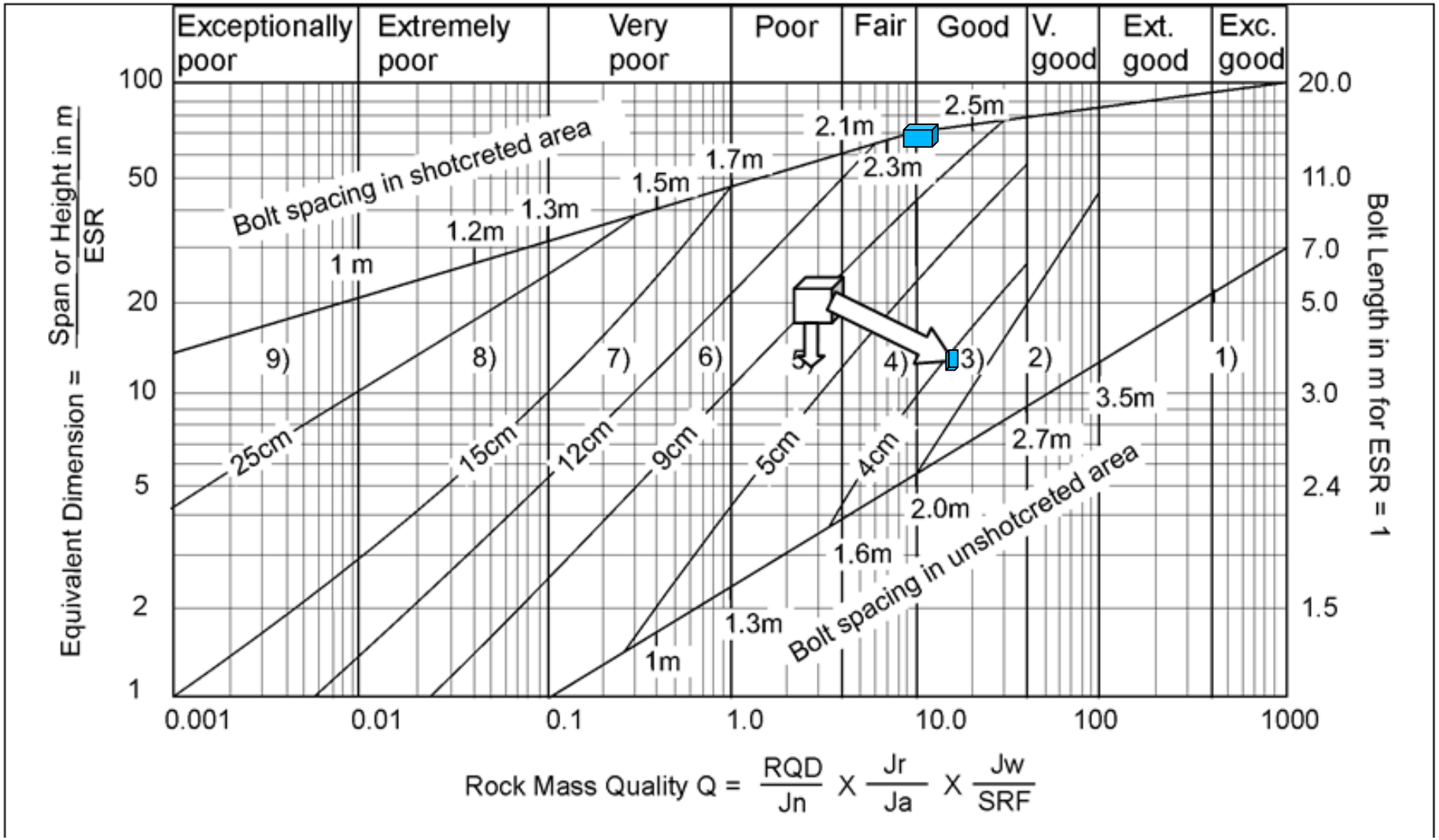
LÆRDAL TUNNEL lorry-turning caverns (three of them) 30 m span, depths 1,000 to 1,400 m (Photo G.Lotsberg)

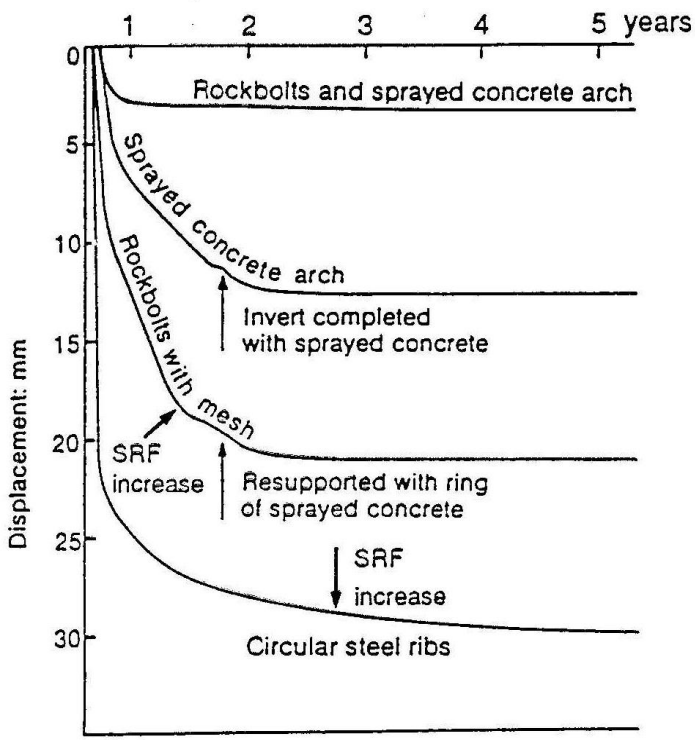


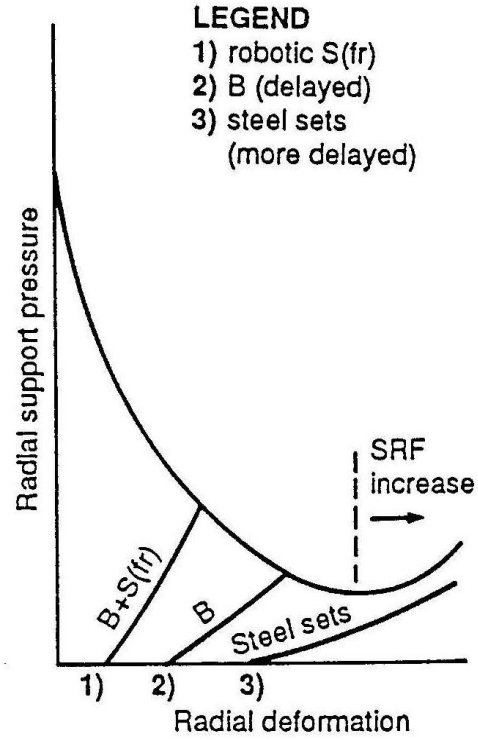

Business Advertising/Photo: Hans Brox

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)







STEEL ARCHES or LATTICE GIRDERS

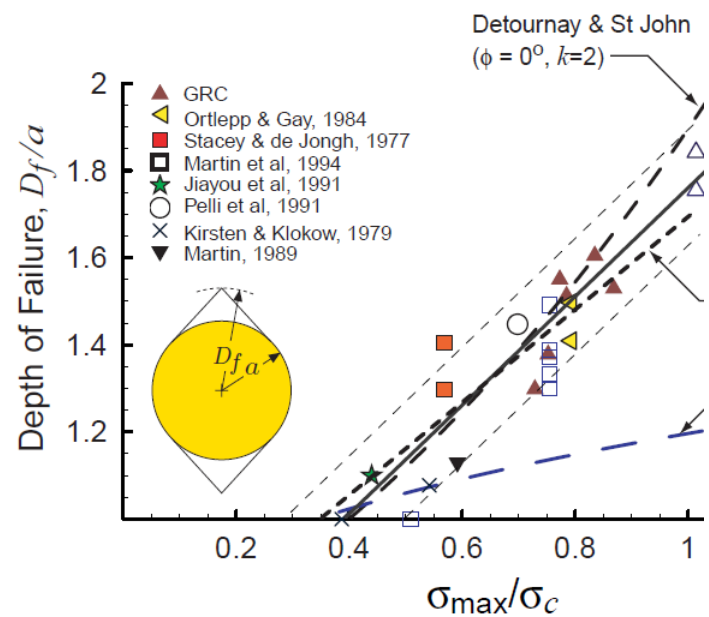
THE CONSEQUENCES OF LOOSENING ROCK – SRF ?

	b) Competent rock, rock stress problems	σ_c/σ_1	σ_θ/σ_c	SRF
H	Low stress, near surface, open joints.	> 200	< 0.01	2.5
J	Medium stress, favourable stress condition.	200-10	0.01-0.3	1
K	High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability.	10-5	0.3-0.4	0.5-2
L	Moderate slabbing after > 1 hour in **massive** rock.	5-3	0.5-0.65	5-50
M	Slabbing and rock burst after a few minutes in **massive** rock.	3-2	0.65-1	50-200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in **massive** rock.	< 2	> 1	200-400

IN CASE OF HIGH STRESS IN MASSIVE ROCK USE THESE HIGH SRF NUMBERS (from 1994 update).

IN CASE OF HIGH STRESS IN JOINTED ROCK USE THE ORIGINAL SRF NUMBERS (only L, M, and N were added in 1994)

Name	Rock type	Overburden in m	σ_1 MPa	σ_3 MPa	σ_c MPa	Maxσ_θ MPa	σ_c/σ_1	σ_θ/σ_c
Strynefjellet	Banded gneiss	230-600 •	20.4	3.5	47-127	56	4.3	0.4-1.2
Høyanger I	Granitic gneiss	650-800 •	33.4	8.1	100-177	92	3-5.3	0.5-0.9
Høyanger II	Banded gneiss	900-1100 •	29	14	55-126	73	1.9-4.3	0.6-1.3
Kobbskaret	Granite	200-600 *•	26	11.5	90	67	3.5	0.7
Svartisen I	Granite	700 •	21.4	12.1	181	52	8.4	0.3
Svartisen II	Mica gneiss	500 ▽	10.9	8.1	27	25	2.5	0.9
Tafjord	Gneiss, amphib.	500-1200 *▽	24.8	6.6	82-185	68	3.3-7.4	0.4-0.8
Fjærland	Granitic gneiss	600-1200 •	25.7	6.5	110	71	4.2	0.7
Frudalen	Granitic gneiss	900-1200 •▽	30?	20?	70-150	ca. 70	2,3-6.0	0.4-1,0
Tosen	Silicate gneiss	400-600 •	20?	10?	110-200	ca. 50	5.5-10	0.3-0.5
Fodnes	Gabbro, diorite	650-1100 *▽	30?	15?	100-150	ca. 75	3.3-5.0	0.5-0.8
Amla	Gabbro diorite	100-400 •	20?	5-10?	100-150	ca. 50	5-7,5	0.3-0.5
Lærdalstunn.	Banded gneiss	800-1400 •	40?	22?	100-150	ca. 100	2,5-3,8	0,7-1,0
Stetind	Granite	300-500? *▽	9,3	3,8	90	24	10	0,3
Pehuenche, Chile	Andesite ✣	400-1200 ▽	35?	15?	100-150	ca. 75	2.9-4,3	0.5-0.8
Ertan, China	Gabbro, diorite	300 -400 *•	40?	15?	105-160	ca. 90	2.7-4.5	0.6-0.9

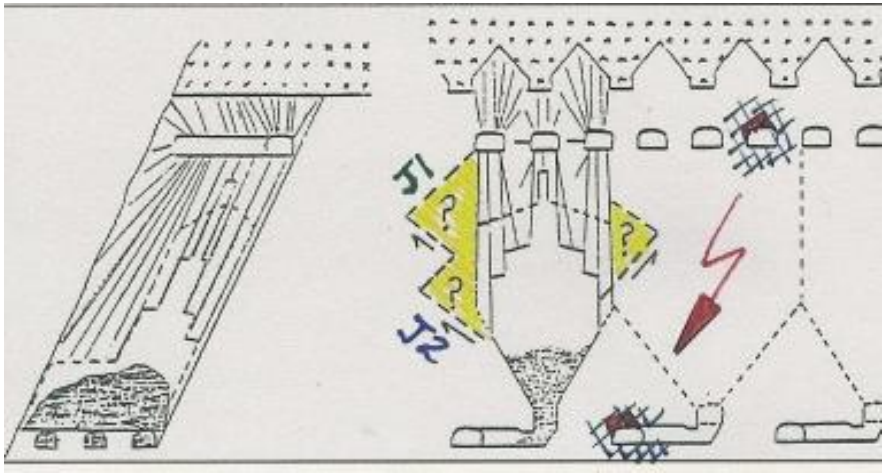


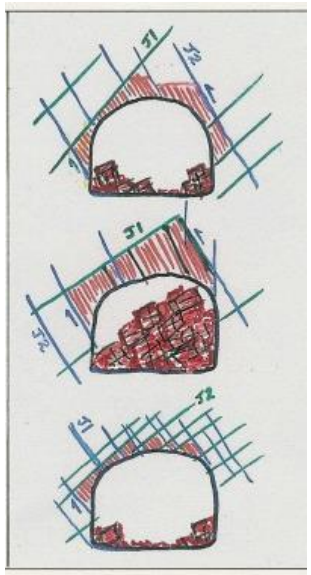
Q-manual after 40 years
(1974-2014)

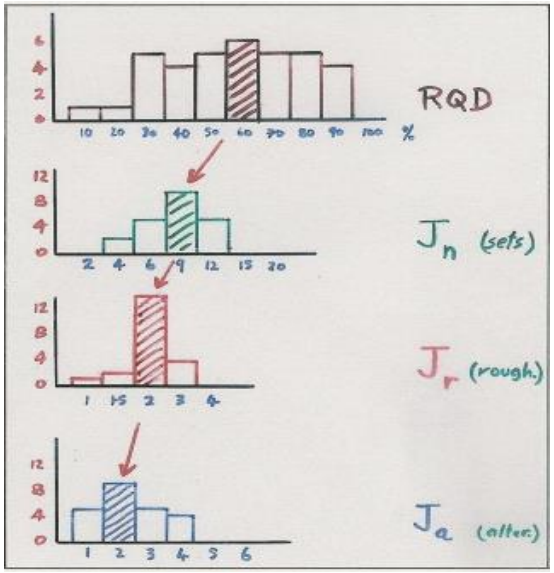
Tunnel and cavern support selection in Norway, based on rock mass classification with the Q-system. **Nick Barton and Eystein Grimstad 36 p.**

(see www.nickbarton.com p.3 of downloads)

11. OVER-BREAK, SMALL-SCALE BLOCK-CAVING





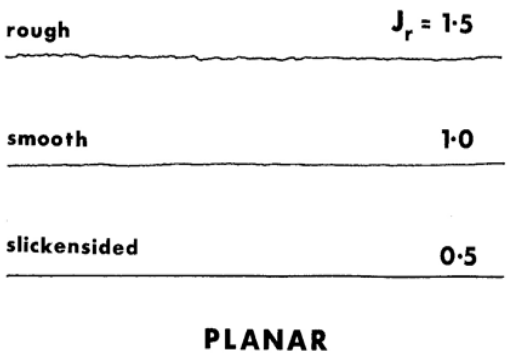


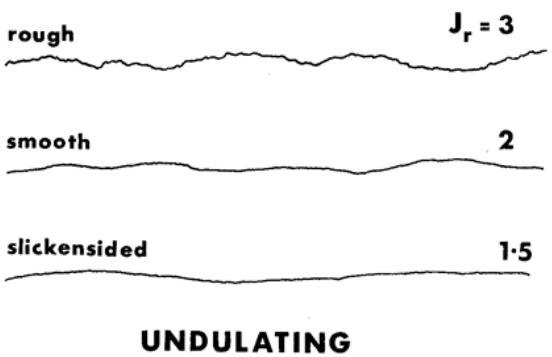
Joint sets J1 and J2 had adverse *Jr/Ja* ratios in some cases (see outliers in the histograms).

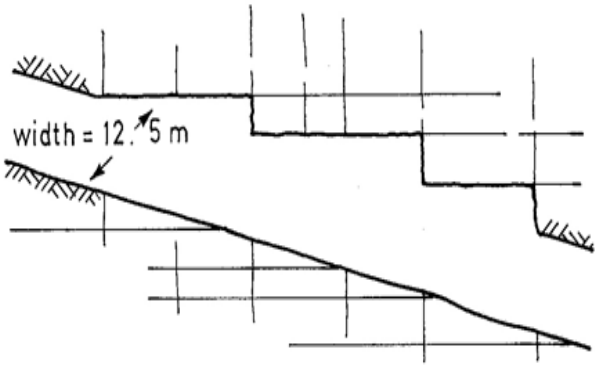
The adverse ratios of *Jn/Jr* were of most importance.

<u>Jn/Jr ≥ 6 meant over-break</u>

THE DUAL ROLES OF Jn and Jr







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

OVER-BREAK IN CIVIL AND MINING EXCAVATIONS

Jn/Jr ≥ 6…….9/1.5……..9/3

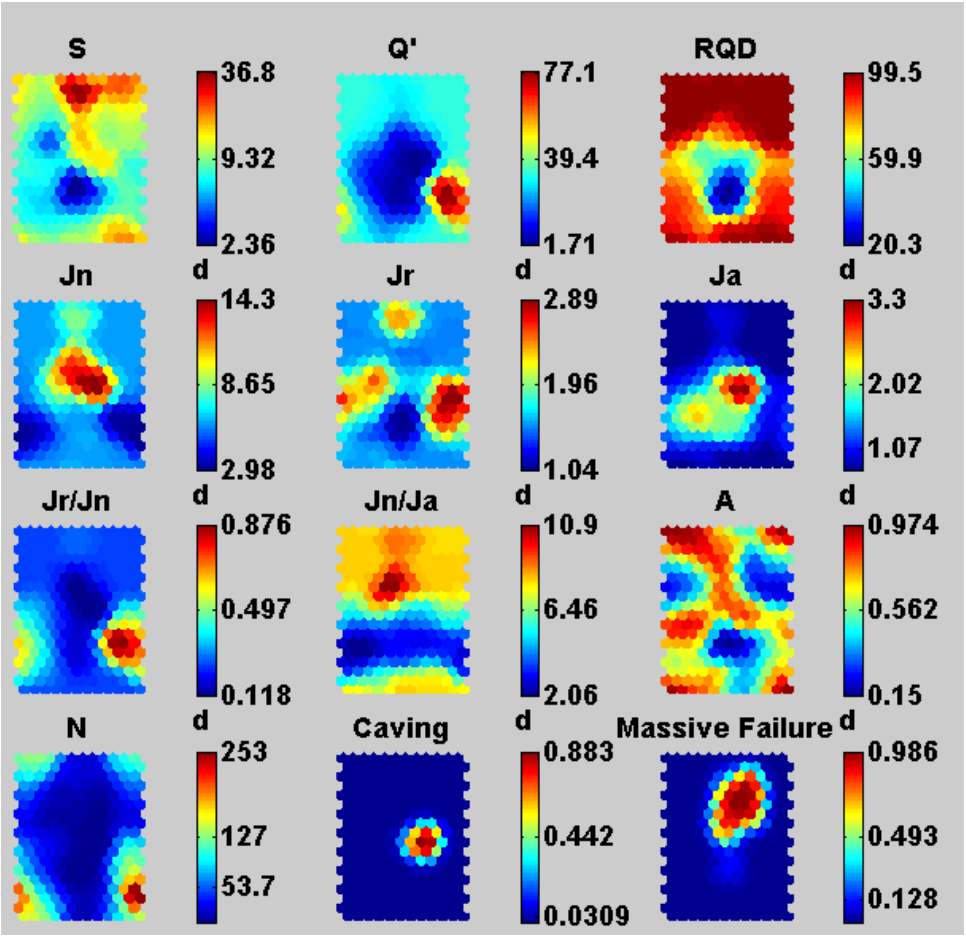
OVER BREAK needing 4m of CONCRETE.

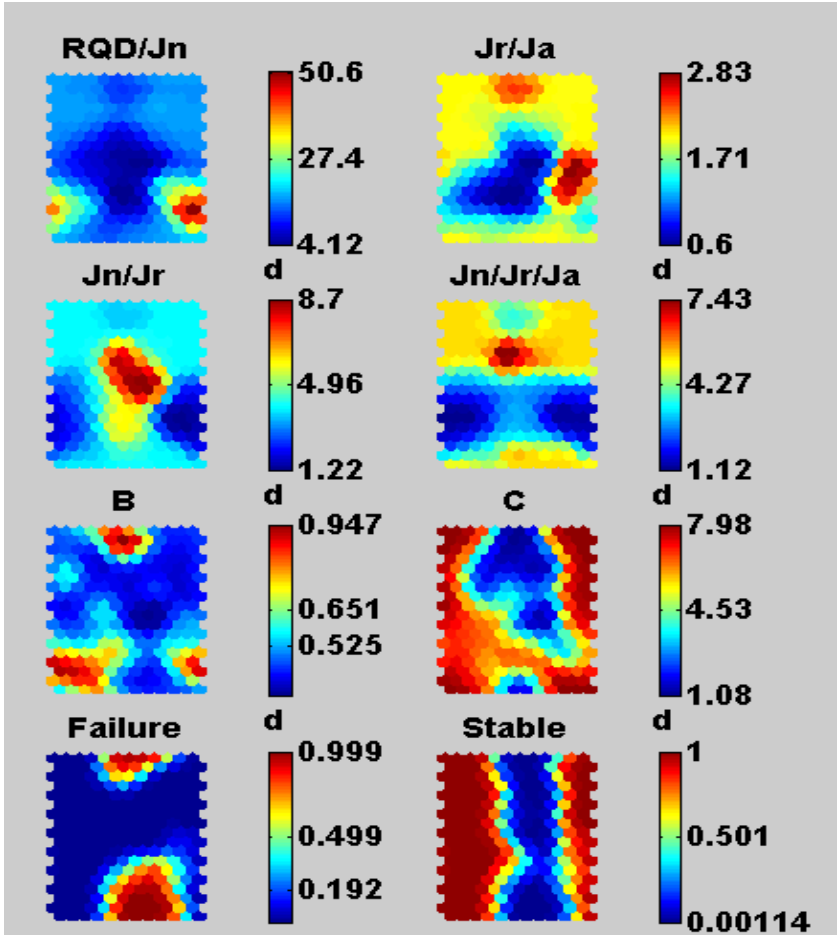
Reason: adverse Jn/Jr

Q-parameter ratio TENDENCIES NEEDED FOR CAVING

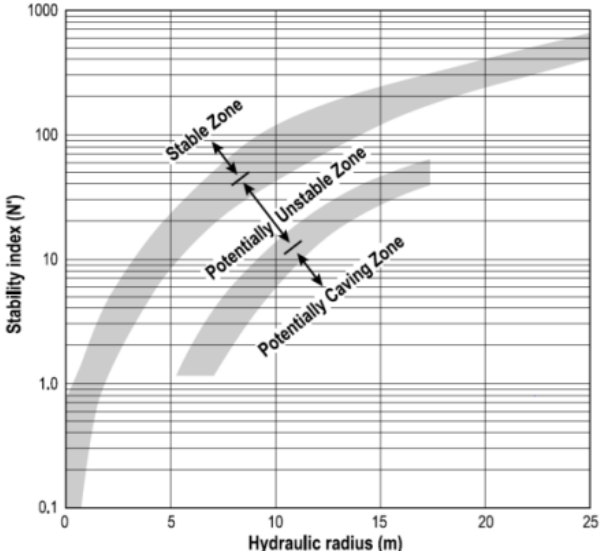
Frazer, CSIRO (priv. comm. 2006)

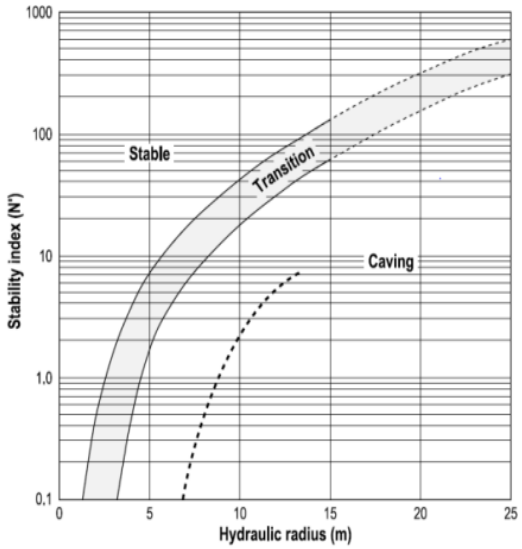
(see Jn, Ja, Jn/Jr ……. for goodness-of-fit)

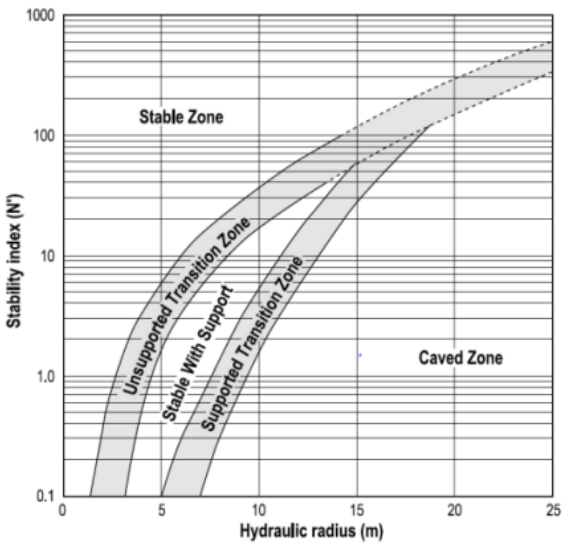


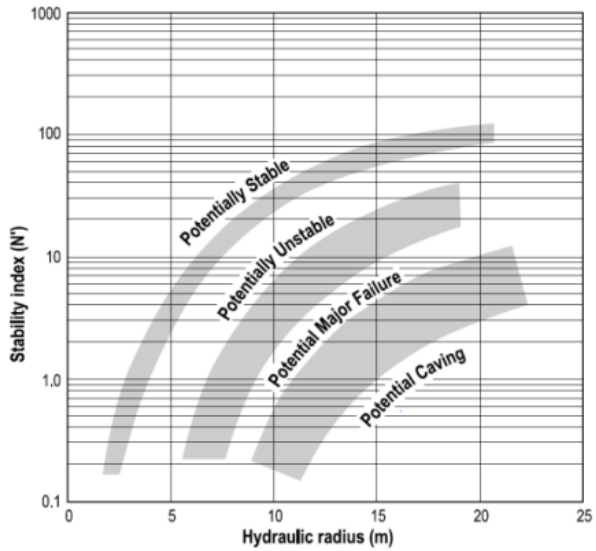


12. STABILITY GRAPH Q', DILUTION, ESR-EQUIVALENCE









Stability index (N') versus the stope hydraulic radius. Various authors, from top: Mathews et al., 1981, Potvin, 1988, Nickson, 1992 and Stewart and Forsyth, 1993. The specific graphic source of these figures was Potvin and Hadjigeorgiou 2001, as quoted by Capes, 2009.

Q’ = RQD/Jn x Jr/Ja

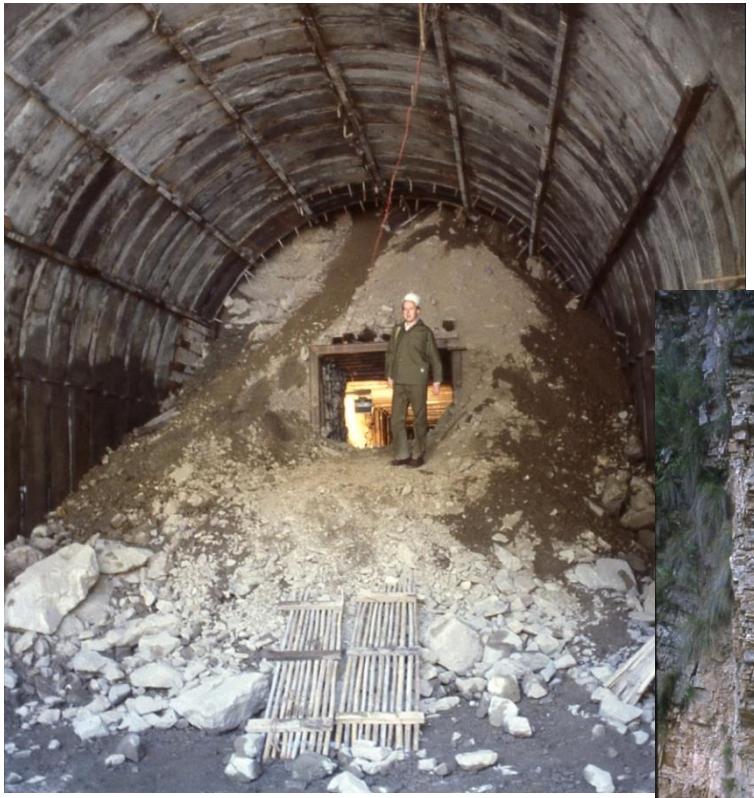
The ‘removed’ SRF (≈ 1/A) needed the stress/strength term A in the

N’ = Q’ x A x B x C equation.

What about very wet sub-river, sub-valley, sub-lake stoping operations? Jw could be useful?

What about proximity to faults? SRF was needed there too.

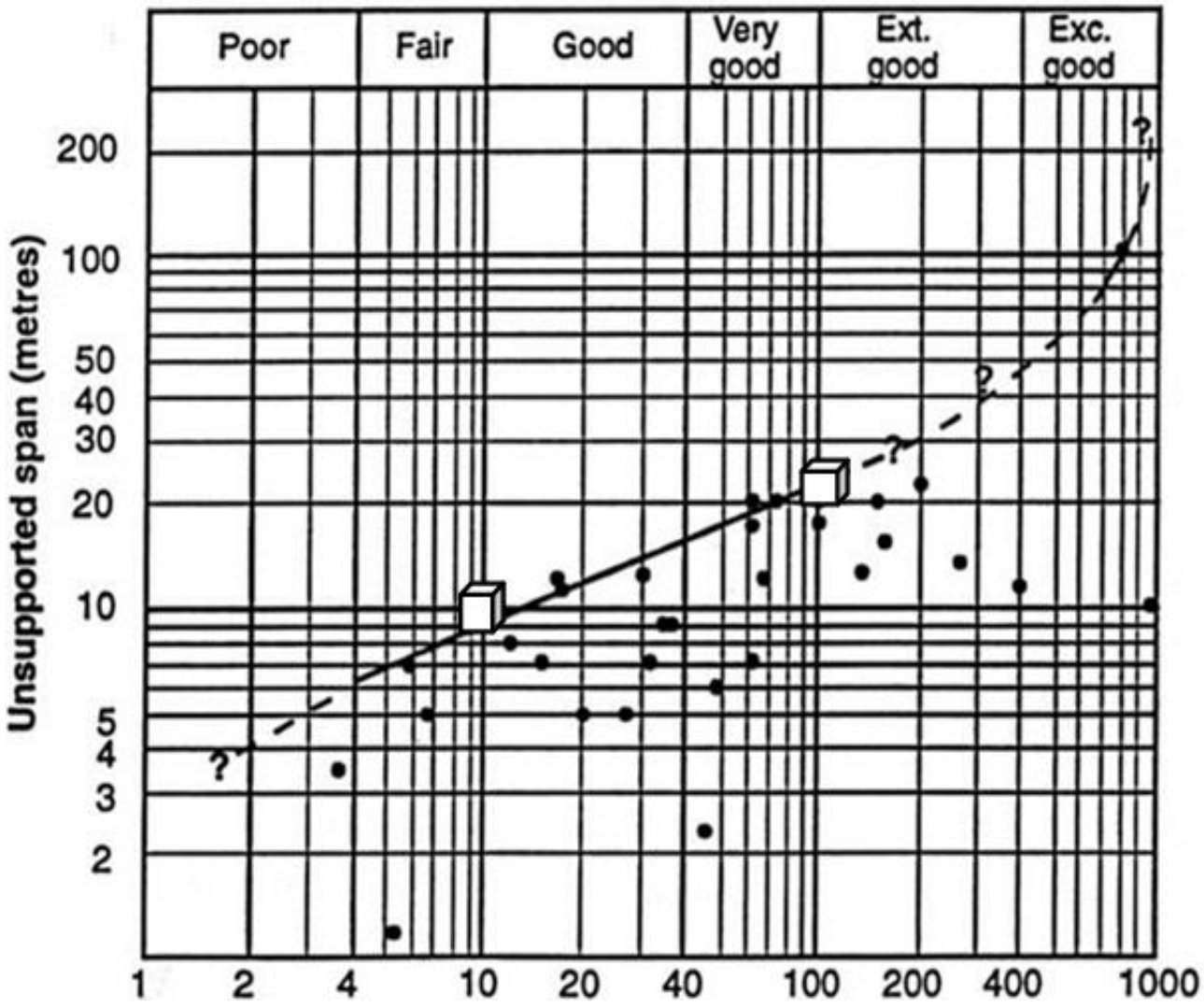
FAULT ZONES ARE UNIQUE CHALLENGES *ALSO* FOR MINING BECAUSE…….



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

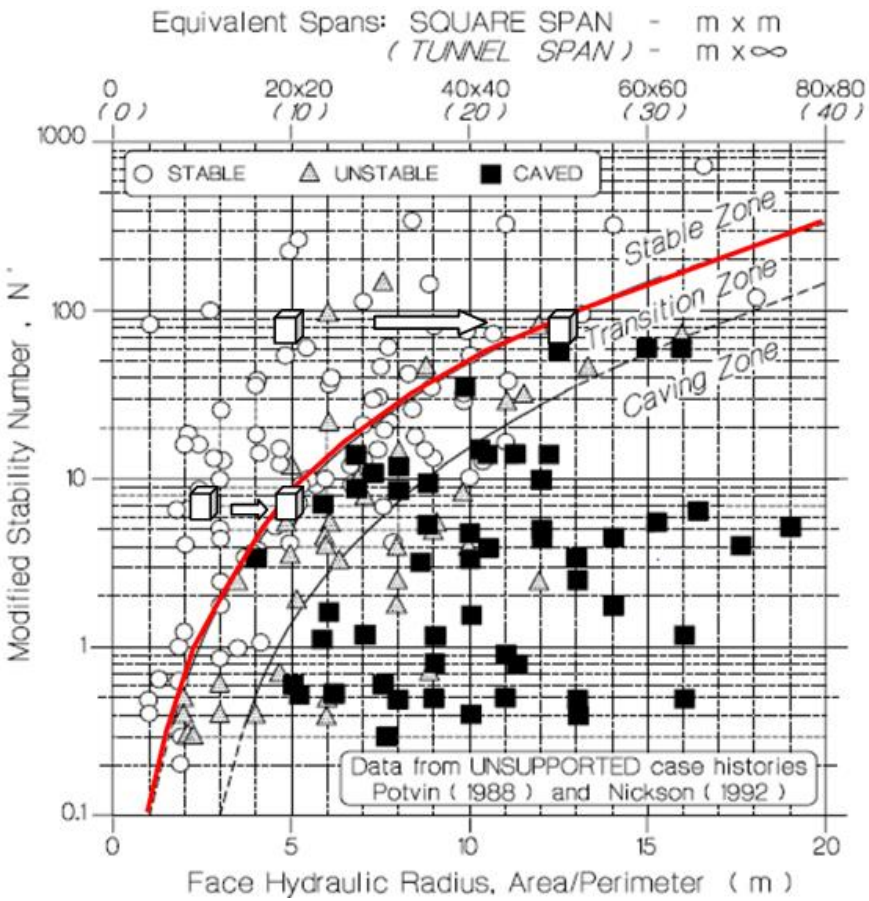
Permanently unsupported excavations

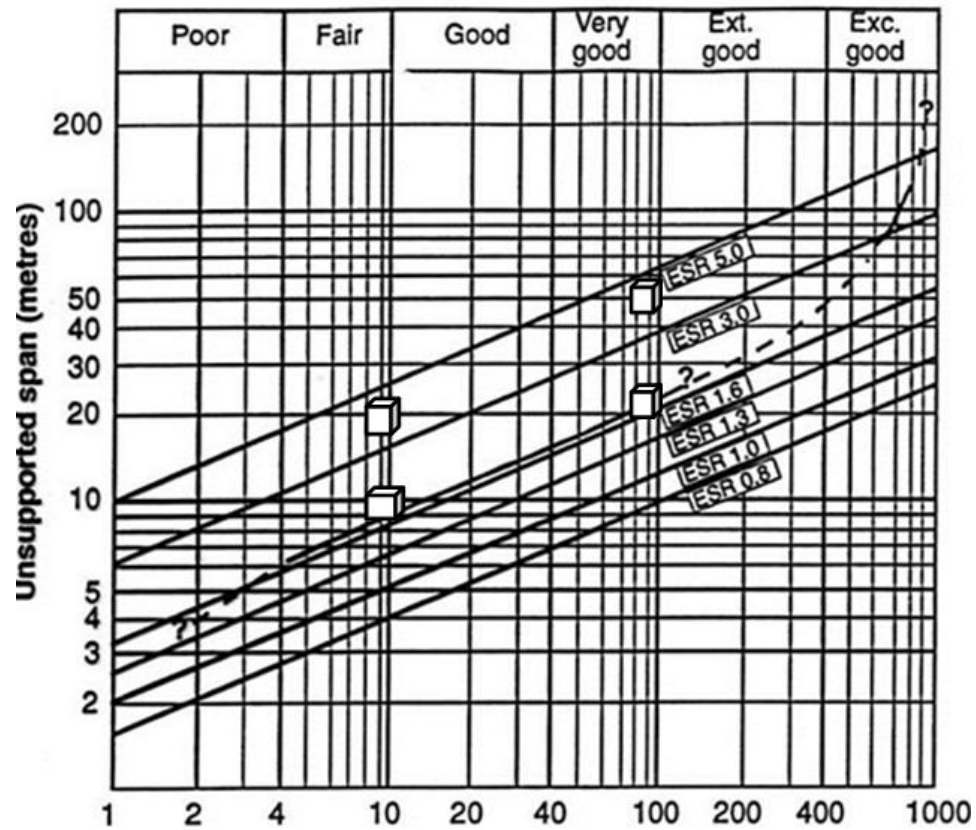
(Barton, 1976)



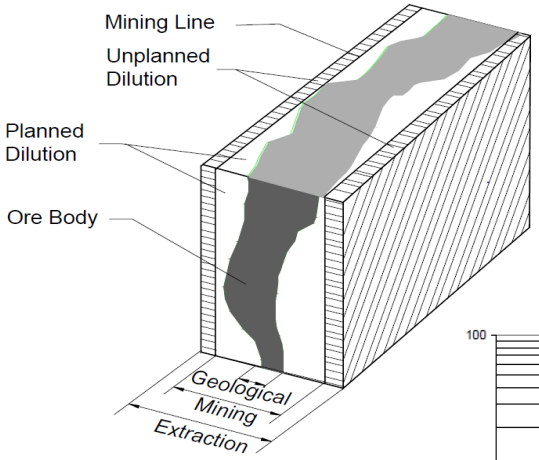
An attempt to relate civil (ESR) to mining

(Left: combined Potvin, 1988 and Nickson, 1992 data: Capes)

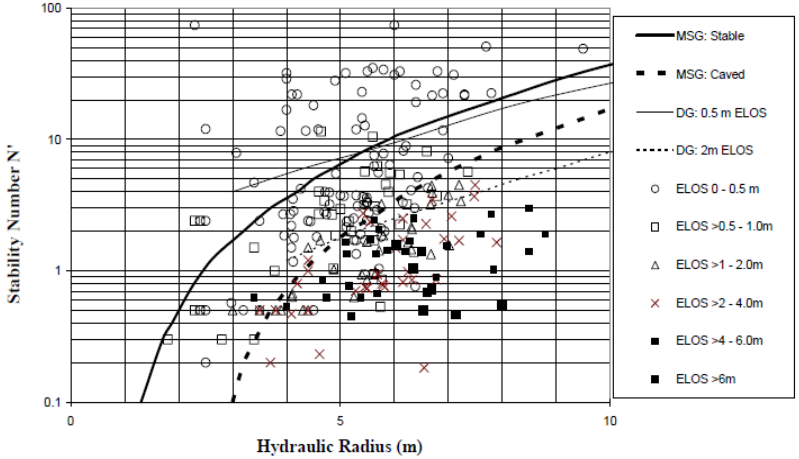




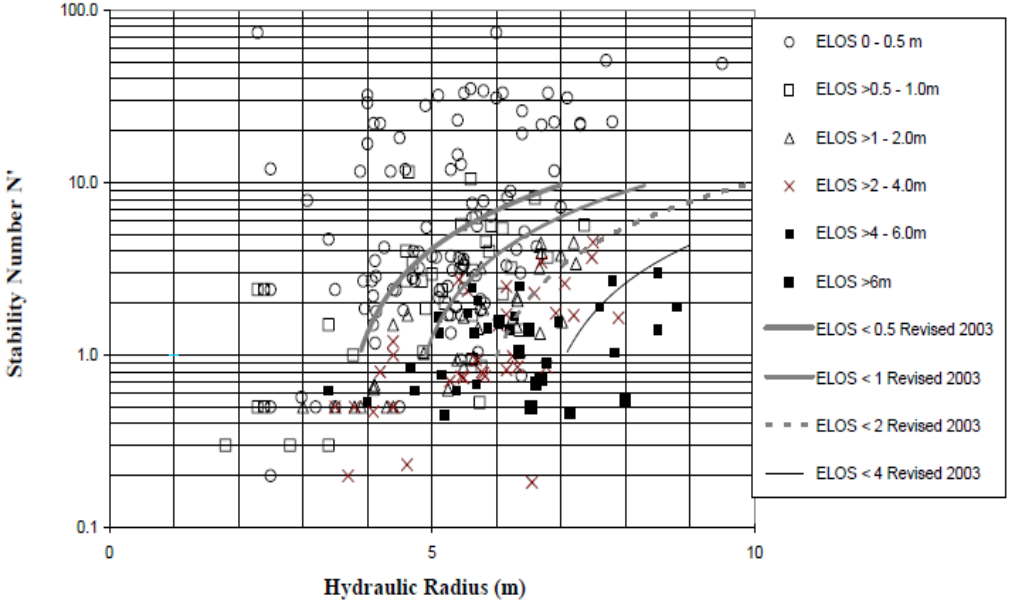
The 'cubes' showing 10 m increasing to 20 m, and 20 m increasing to 50 m span (approx.) in order to reach the red envelope.



A simple definition of (average) dilution, beyond the planned and inevitable dilution. The figure is based on Scoble and Moss, 1995, but was reproduced from Capes, 2009.



The 255 cases assembled by Capes, 2009, showing: a) the stable and caved Modified Stability Graph curves, and b) ELOS Dilution Graph curve.

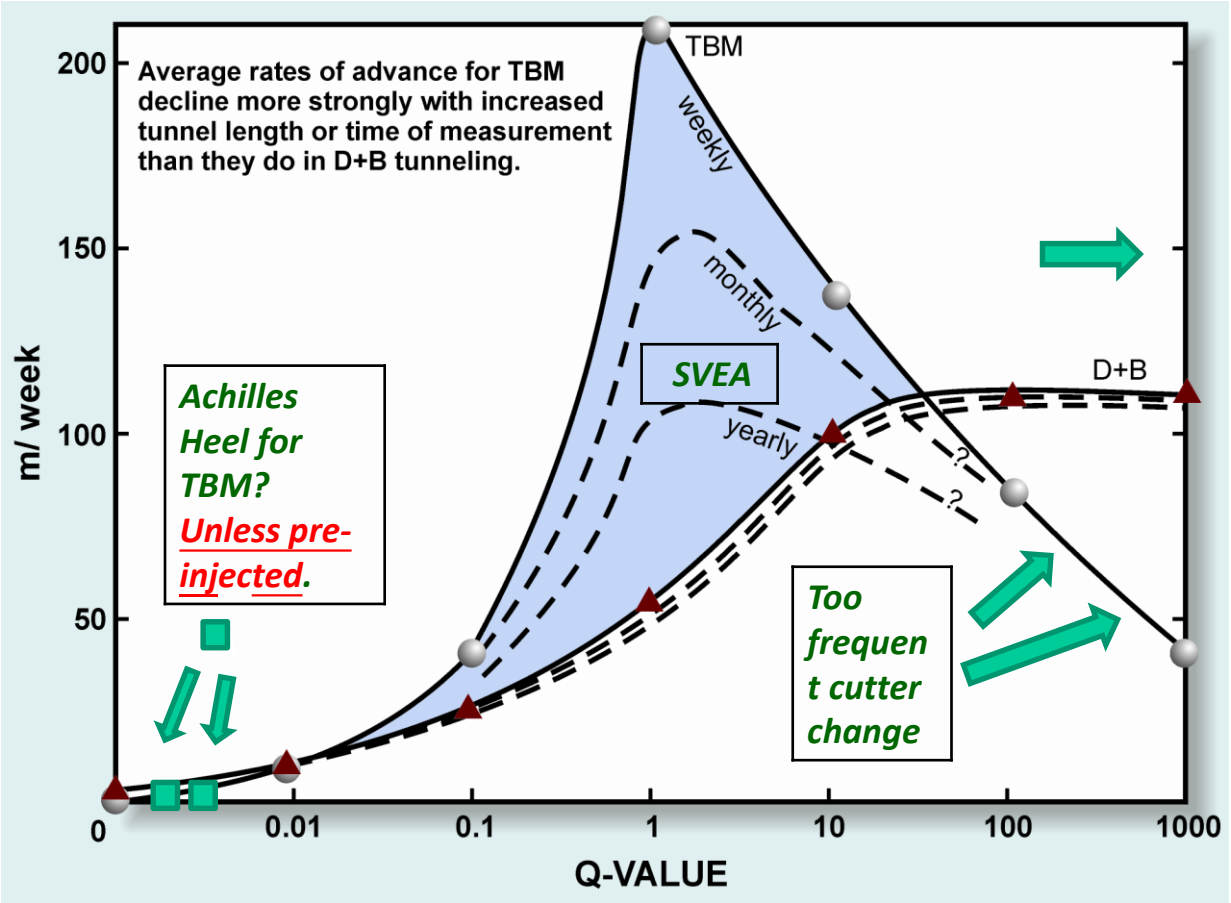


A ‘MINING’ WORLD RECORD TO END WITH

(Mine access tunnel)

A 'MINING' CASE RECORD – IN FACT A WORLD RECORD

(*CENTRAL* Q-VALUES *AND* Q_{TBM} VALUES BEST FOR TBM.
TAIL-DISTRIBUTIONS (of Q) ARE 'faster' WITH D+B !)



Record for drill-and-blast:

150m/BEST week (SVEA)

Whole project 104 m/week average, 5.8 km

SINGLE-SHELL NMT – Q-BASED SUPPORT SELECTION