SHEAR STRENGTH of ROCK, ROCK JOINTS and ROCK MASSES: PROBLEMS and SOME SOLUTIONS

Nick Barton, Oslo, Norway

www.nickbarton.com

EUROCK 2014, VIGO, SPAIN

WHY SHEAR STRENGTH AND FRICTION (or lack of these) ARE IMPORTANT?

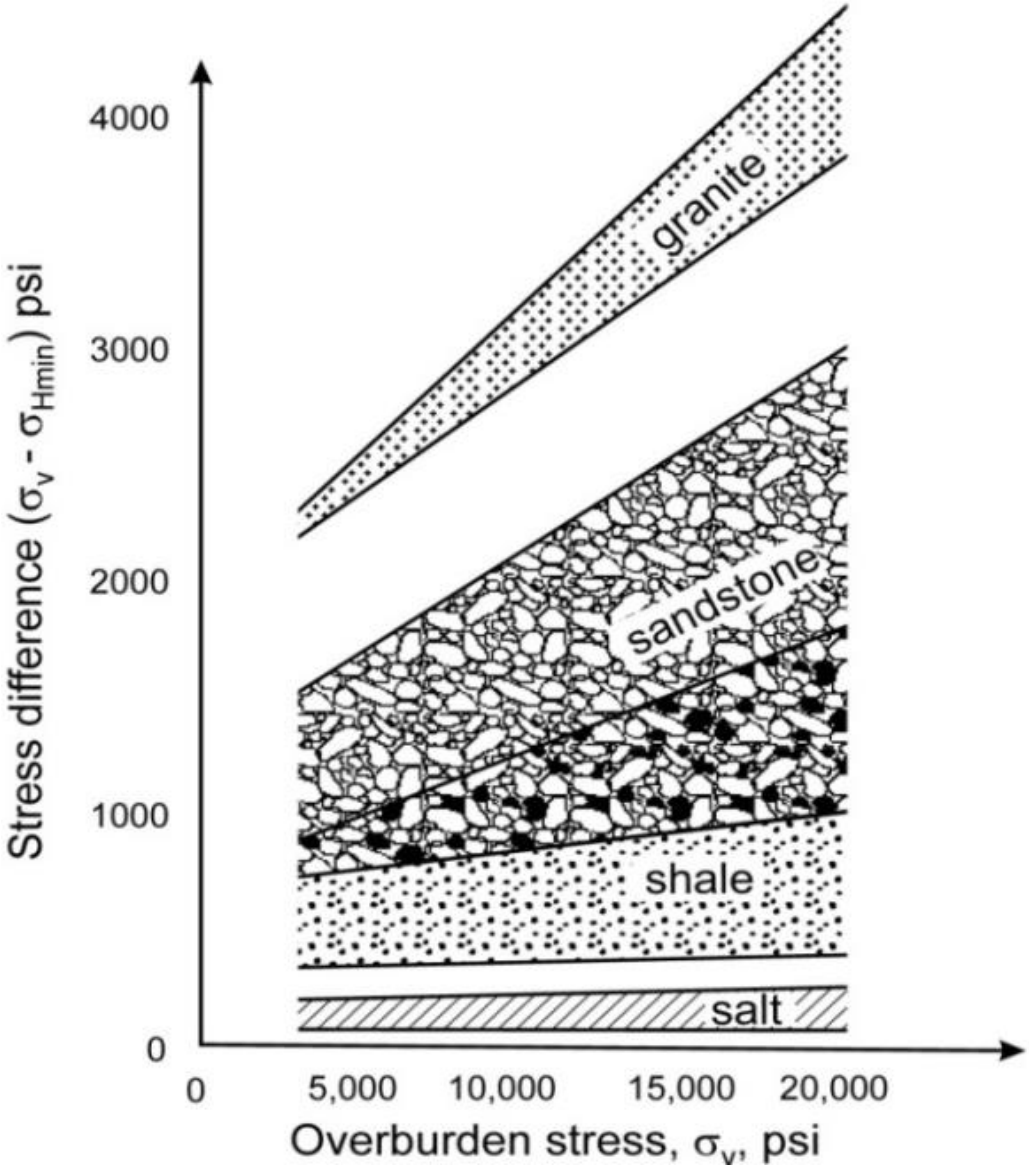
- Walking upright (or falling over)
- Mountains and valleys (or featureless terrain, no HEP)
- Cliffs, pebble beaches, sand dunes (or flooding)
- Petroleum traps: salt, shale, clay-cores in faults (or petroleum loss through seeps)?
- Gas-shale and oil-shale production (or rapid decline)
- Dam stability (or failure)
- Road cuttings (or road closures)
- Open pit and bench stability (or failures and equipment-damage)
- Tunnel and cavern stability (or collapses)
- Fault stability (or earthquakes)

Content of Lecture

1. INTACT ROCK – CRITICAL STATE – A NEW CRITERION
2. ROCK JOINTS – NO COHESION – TESTING PROBLEM
3. SCALE EFFECTS for ROCK JOINTS
4. STRESS TRANSFORMATION ERRORS: NO DILATION
5. ROCK MASSES – displacement-and-process dependent
6. CANNOT ADD ‘c’ and ‘σ_n tan φ’ – NOT COINCIDENT
7. A SIMPLE (-minded) ALTERNATIVE TO H-B/GSI: SPLIT Q_C
8. CONCLUSIONS

1. INTACT ROCK

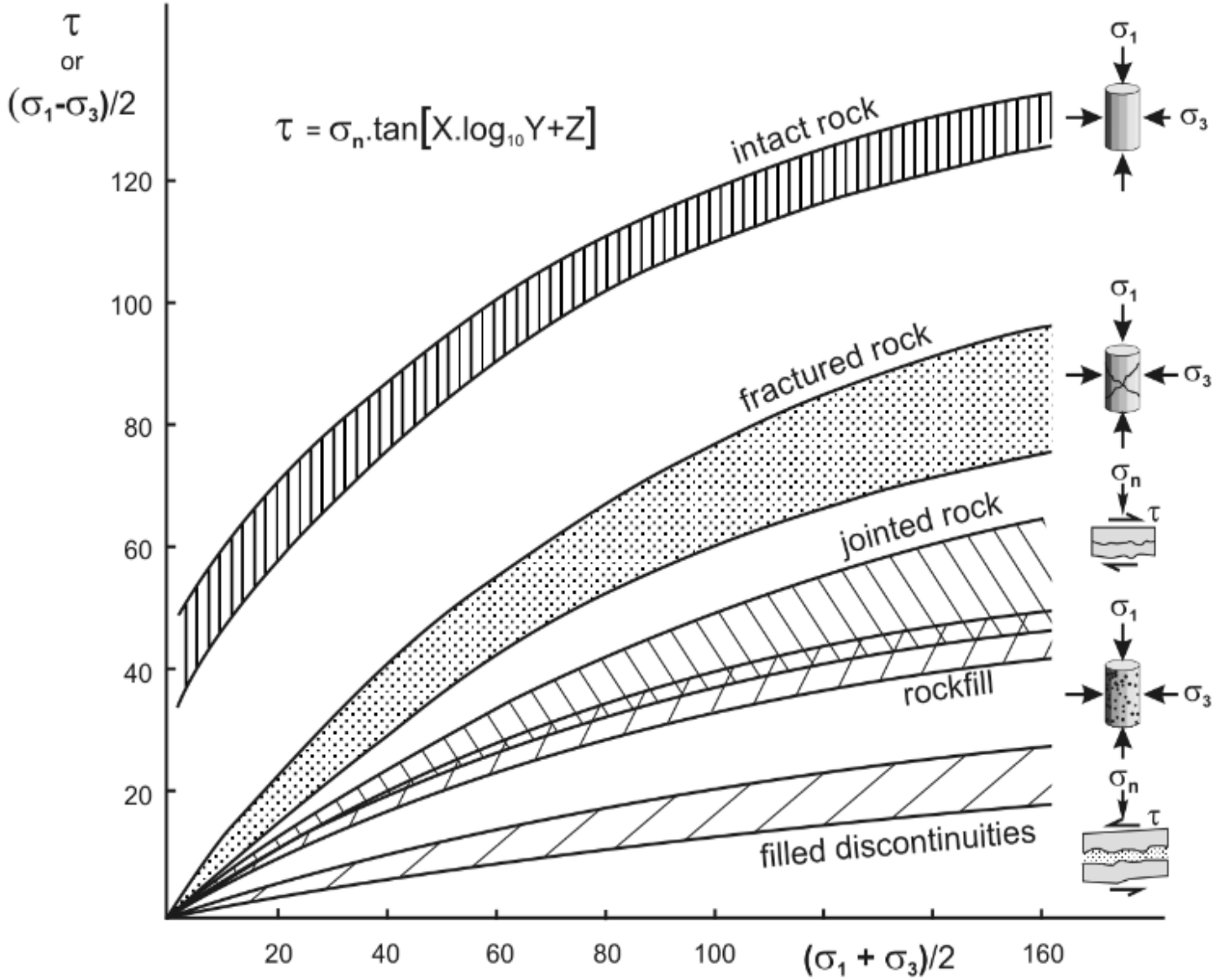
LINEARITY or NON-LINEARITY ?



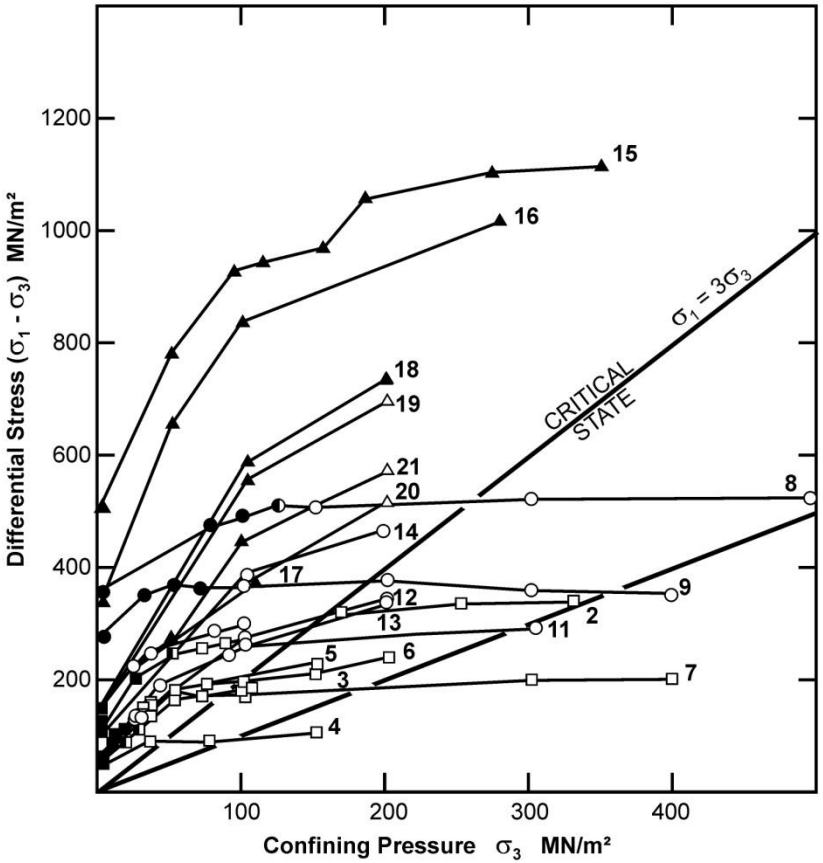
Numerous courses with petroleum and geomechanics participants, suggest linearity is still the sacred belief.

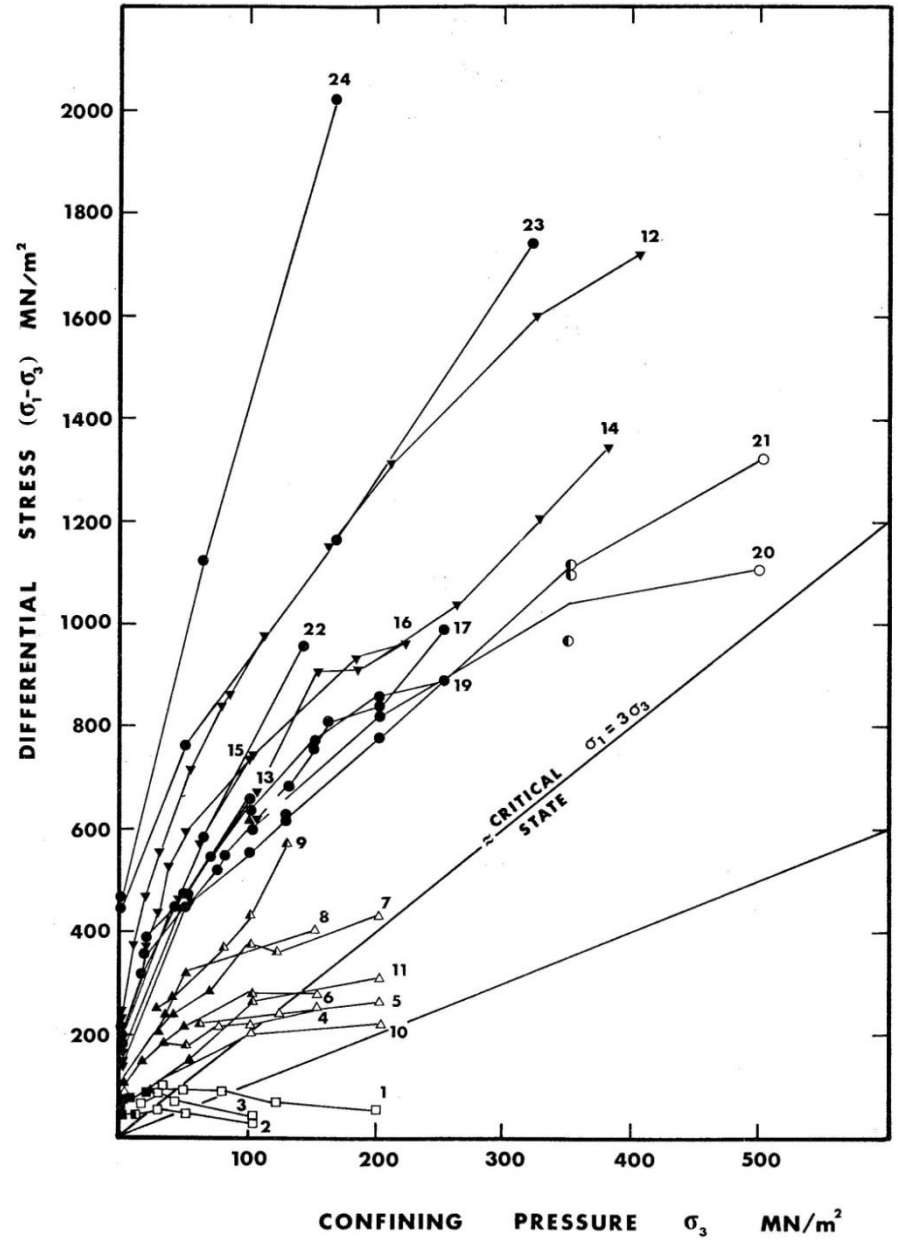
(Swolfs, 1977)

THE REALITY – IF A LARGE RANGE OF STRESS IS INVOLVED (e.g. AS IN MINING AND PETROLEUM ENGINEERING) (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)		

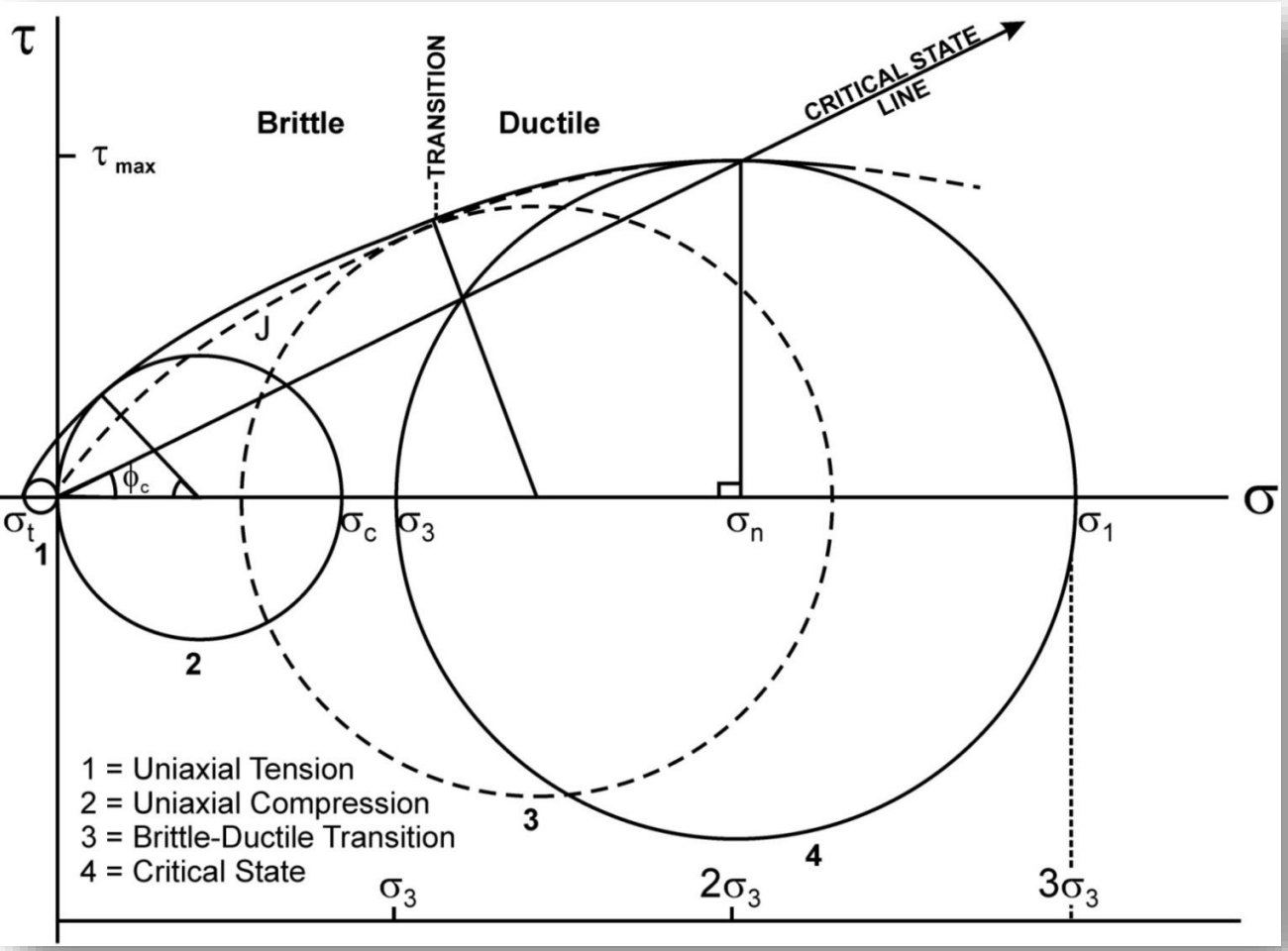




Some high stress triaxial data which influenced opinions

(Mogi 1966). Silicates and carbonates

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



Critical state concept recently used as basis for *improved strength criterion* for intact rock.

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

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

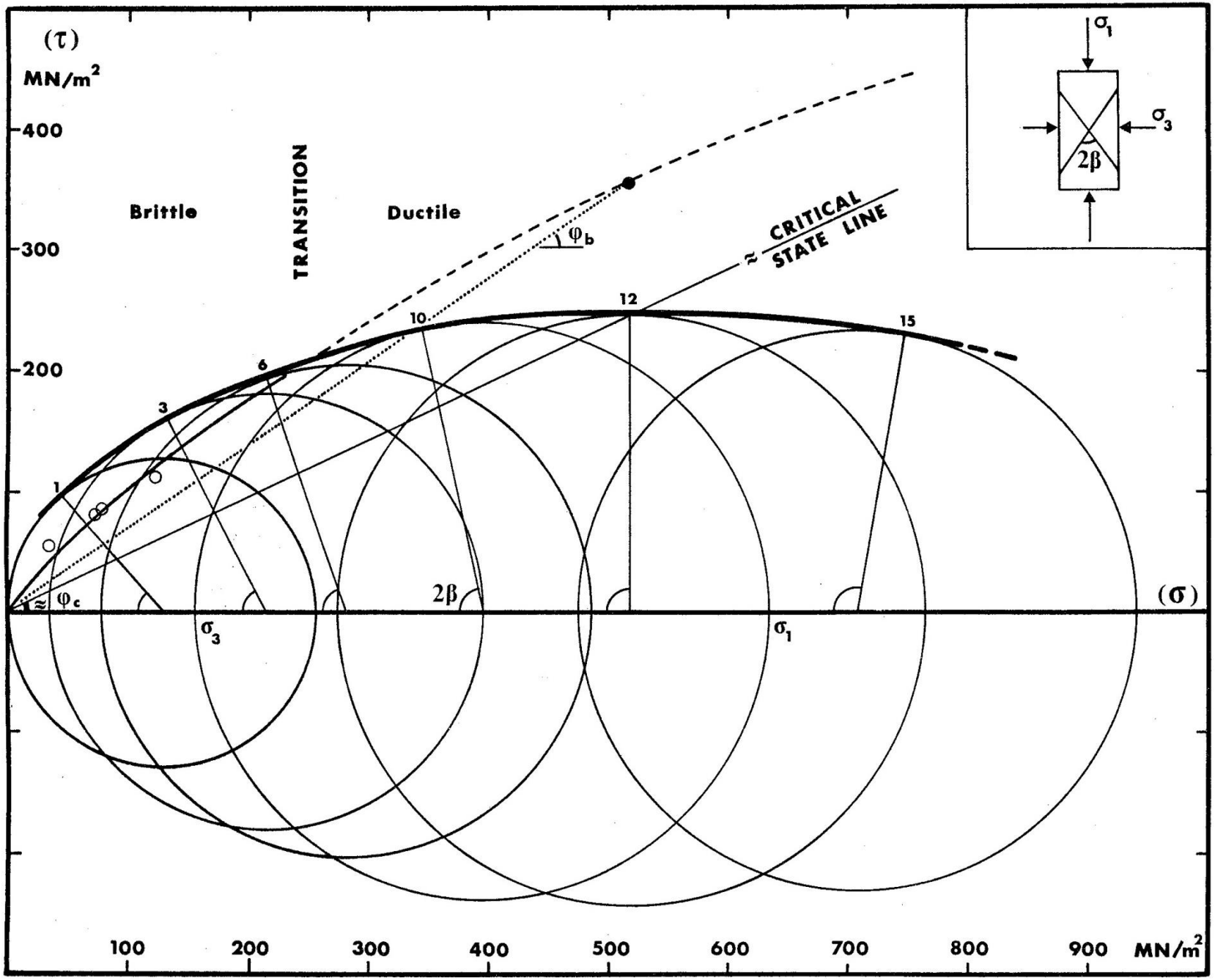
Note proximity of σ_c and $\sigma_{3\ critical}$.

The ‘critical state’ concept was 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.

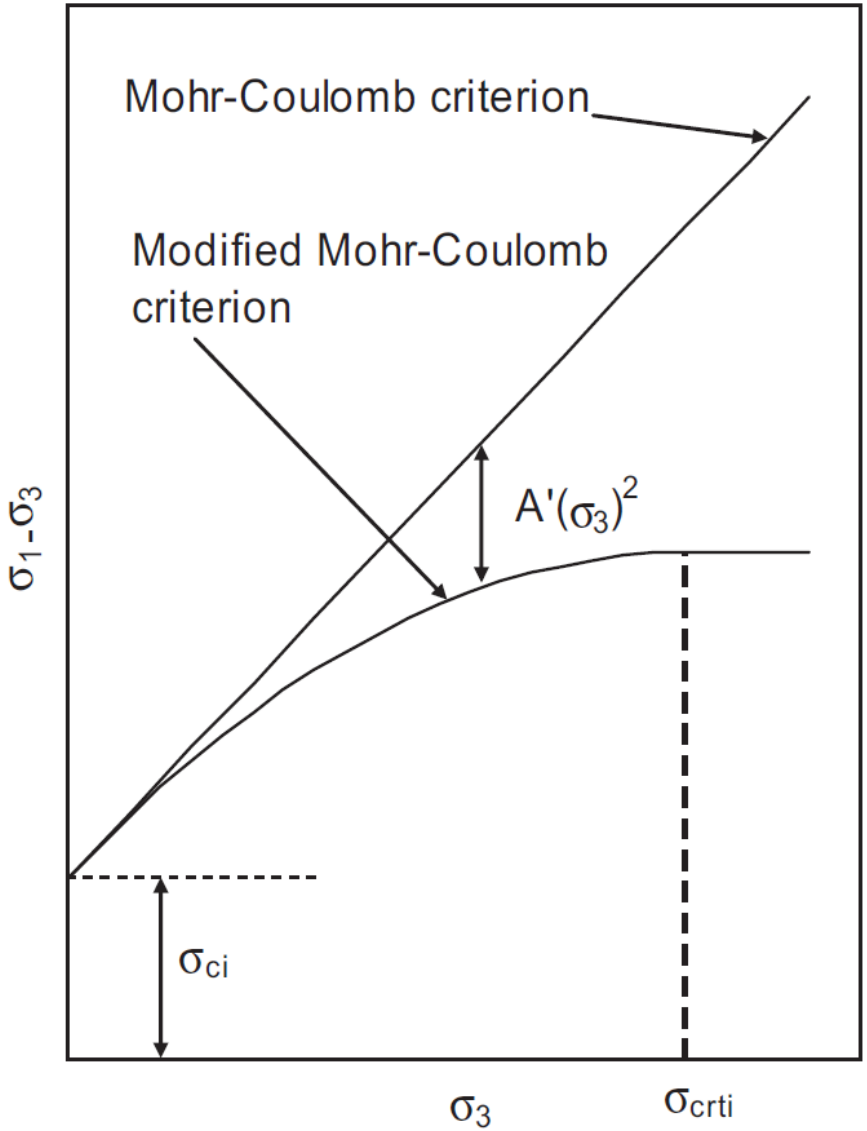
The elegant Singh et al., 2011 criterion gives the *correct deviation from linear Mohr Coulomb*

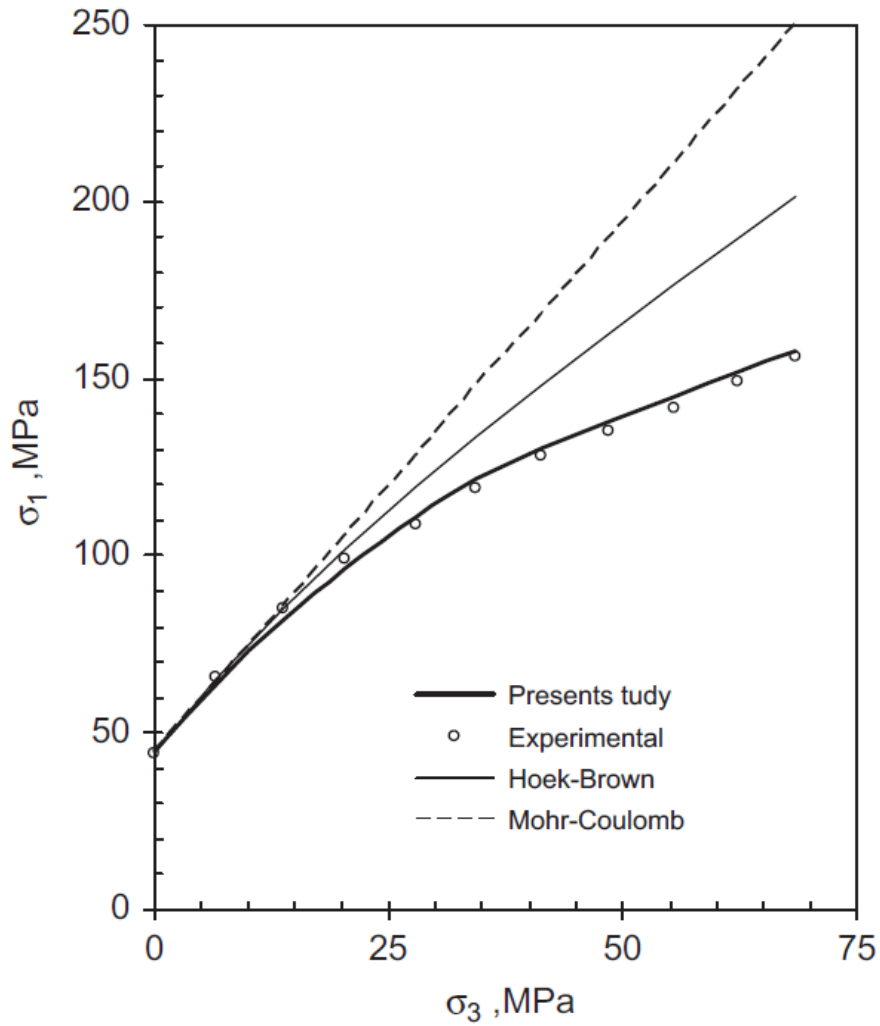
(= greater curvature than Hoek Brown, for which triaxial tests over a wider range of stress are needed)


(τ)
MN/m²
400
300
200
Brittle
TRANSITION
Ductile
φb
≈ CRITICAL STATE LINE
1
3
6
10
12
15
≈ φc
2β
σ3
σ1
(σ)
100
200
300
400
500
600
700
800
900
MN/m²

From Singh et al., 2011

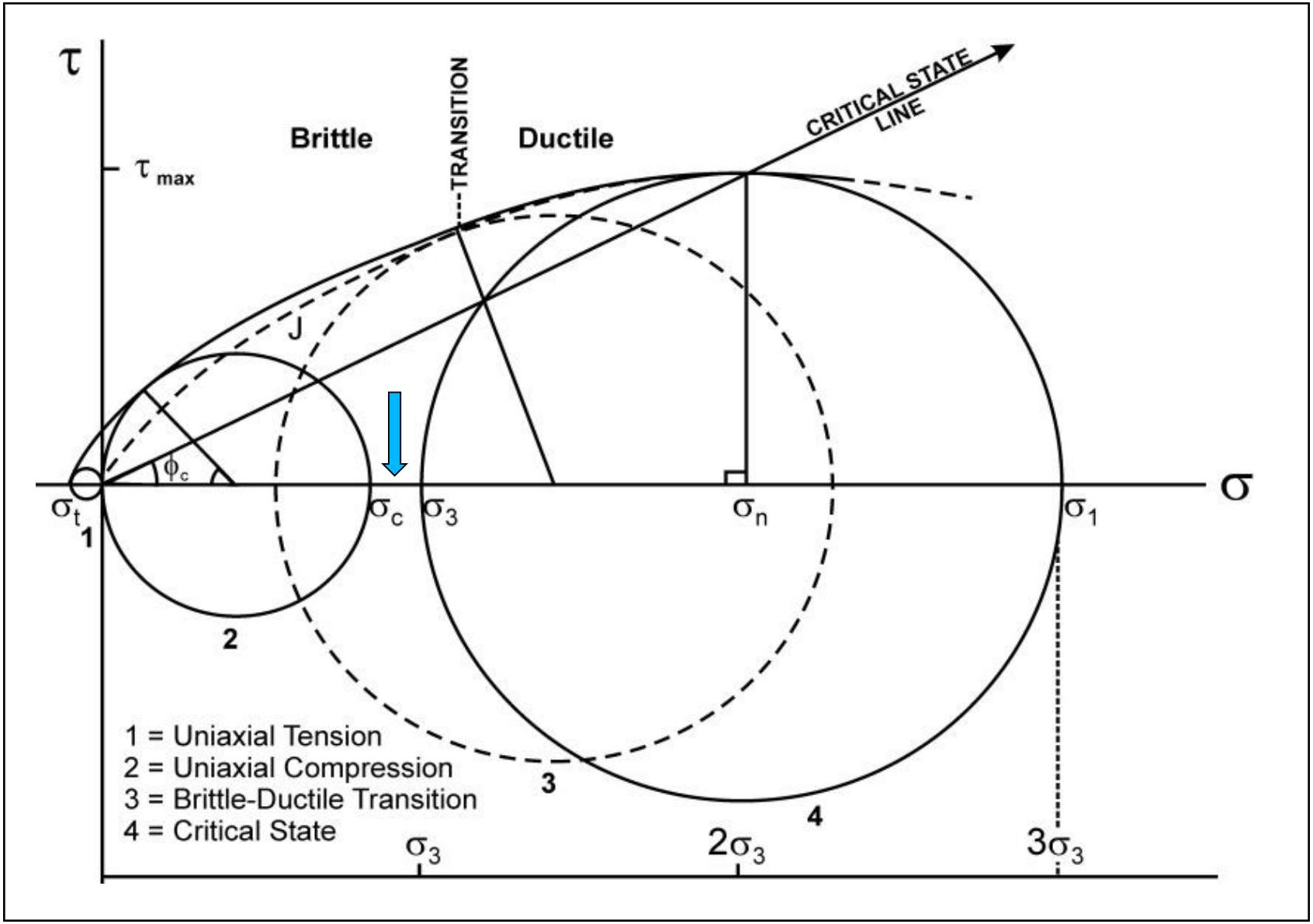
1. Objective
2. Example





MOST FREQUENTLY:

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



Contents lists available at ScienceDirect

International Journal of
Rock Mechanics & Mining Sciences

journal homepage: www.elsevier.com/locate/ijrmms

Modified Mohr–Coulomb criterion for non-linear triaxial and polyaxial strength of intact rocks

Mahendra Singh *, Anil Raj, Bhawani Singh

Department of Civil Engineering, IIT Roorkee, Roorkee-247667, India

ARTICLE INFO

Article history:
Received 10 February 2010
Received in revised form
4 January 2011
Accepted 9 February 2011

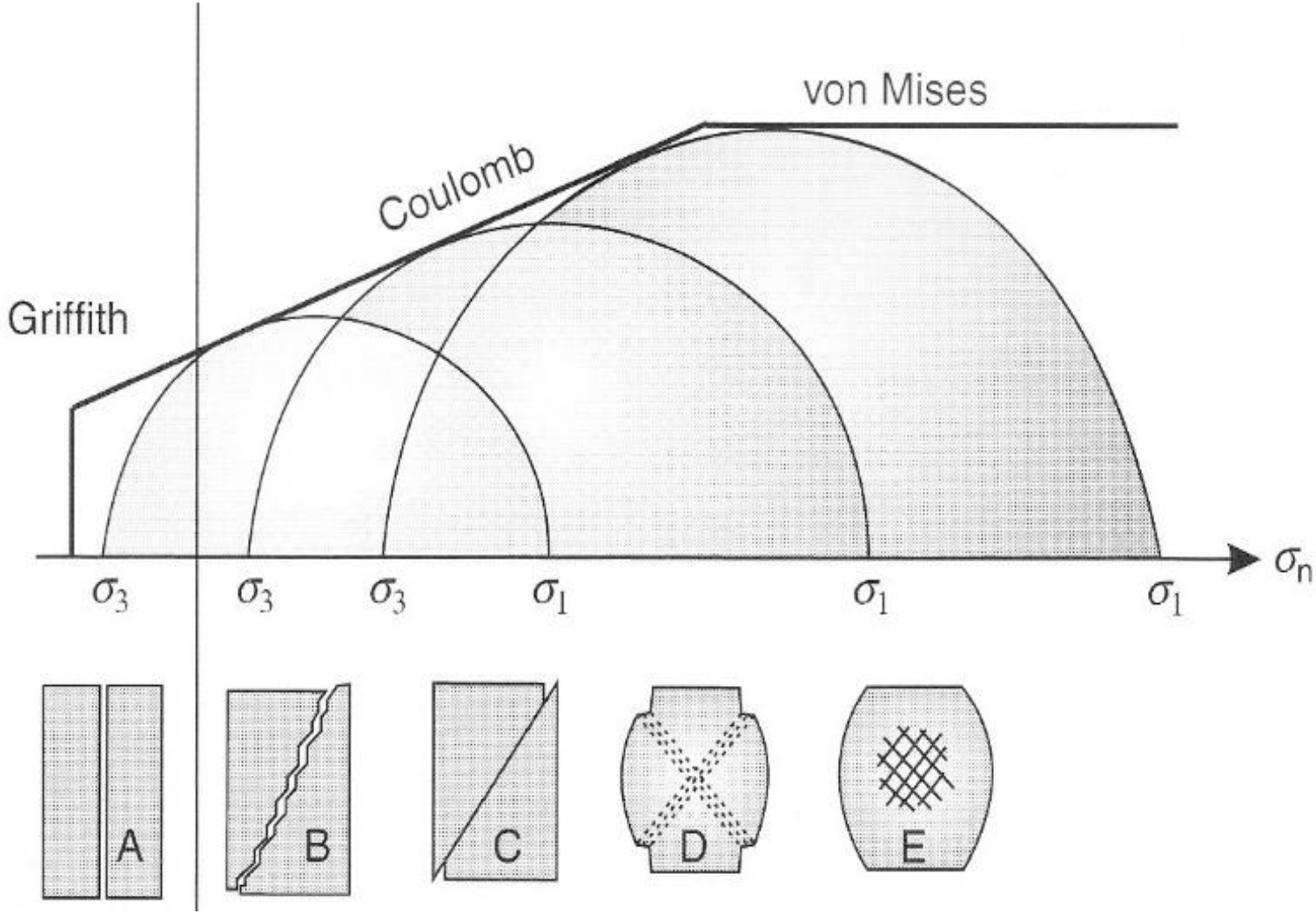
Keywords:
Intact rock
Triaxial
Polyaxial
Strength
Criterion

ABSTRACT

Triaxial or polyaxial strength of rocks is required while analysing many civil and mining engineering structures in rocks. Mohr–Coulomb criterion is the most widely used strength criterion in rock engineering problems. In its present form the criterion suffers from two major limitations. Firstly, it represents the strength of rock as a linear function of confining pressure. Secondly, the effect of intermediate principal stress is not considered by this criterion. In the present study, this criterion is modified to take into account the non-linearity and effect of intermediate principal stress on strength behaviour. Barton's [1] critical state concept for rocks has been employed for this purpose. The applicability of the proposed simple non-linear triaxial and polyaxial strength criteria has been verified by applying them to experimental results for the intact isotropic rock material available from literature and comparing the prediction with the other popular criteria in vogue. The agreement has been found to be excellent. The applicability of the concept to jointed rocks will be discussed in separate publication.

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

(From Gudmundsson, 2011)



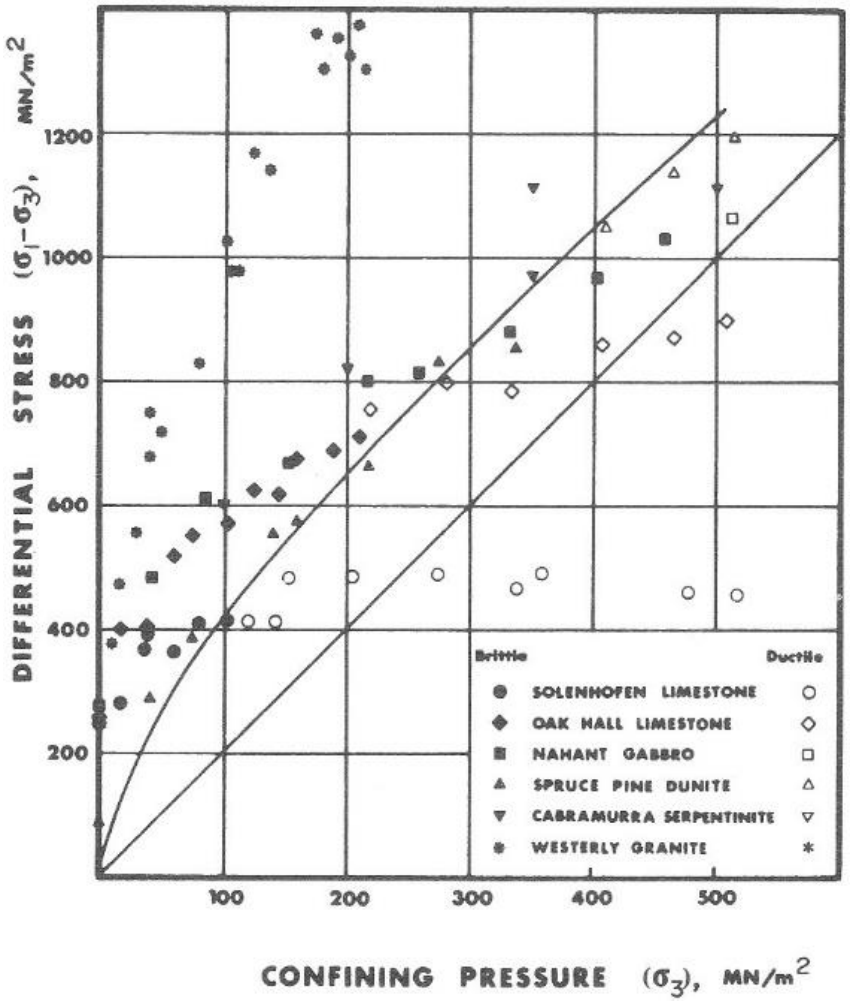
A SIMPLE-MINDED, TWO-PARAMETER ALTERNATIVE

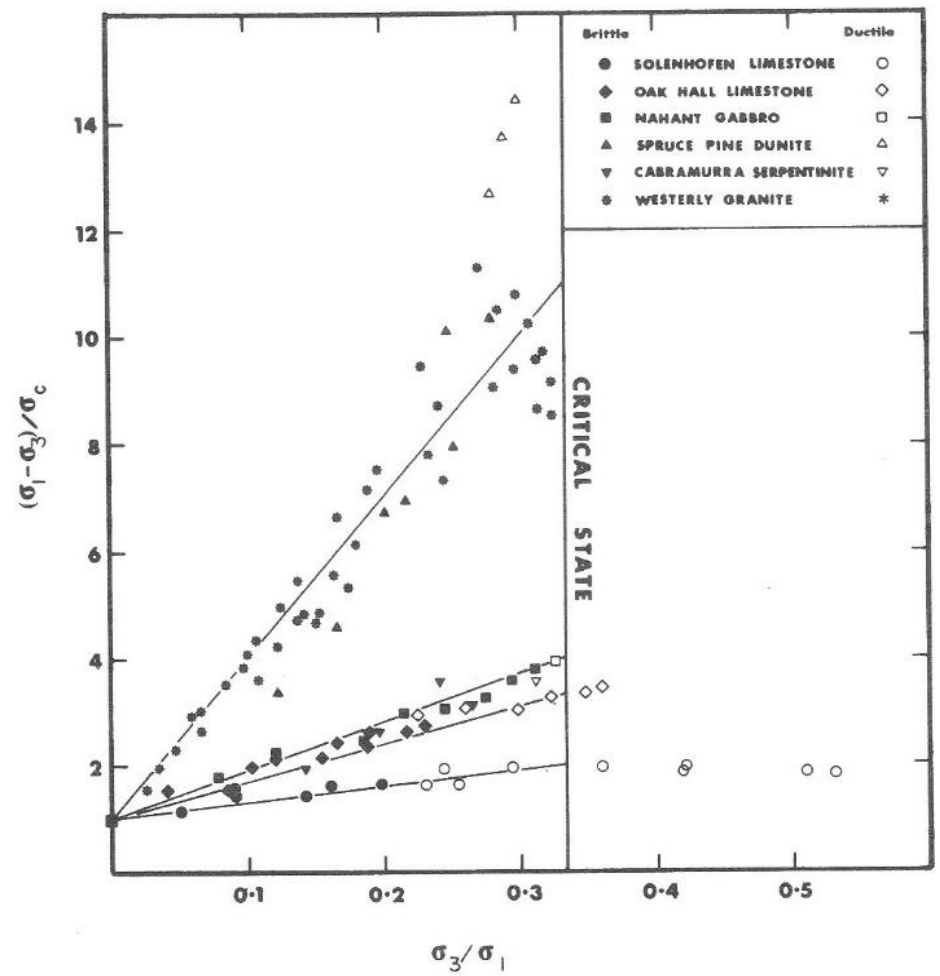
(Need only gradient **M** and **UCS**)

$(\sigma_1 - \sigma_3)/\sigma_c = M\ \sigma_3/\sigma_1 + 1.0$ (Barton, 1976)

M-values: Solenhofen limestone = 3, Oak Hall limestone = 7, Nahant gabbro = 9, Westerly granite = 30

Triaxial data from Byerlee, 1968.

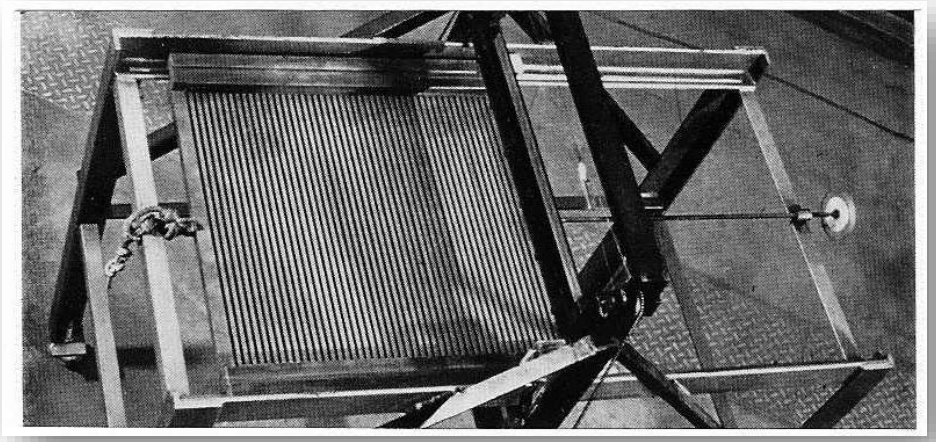


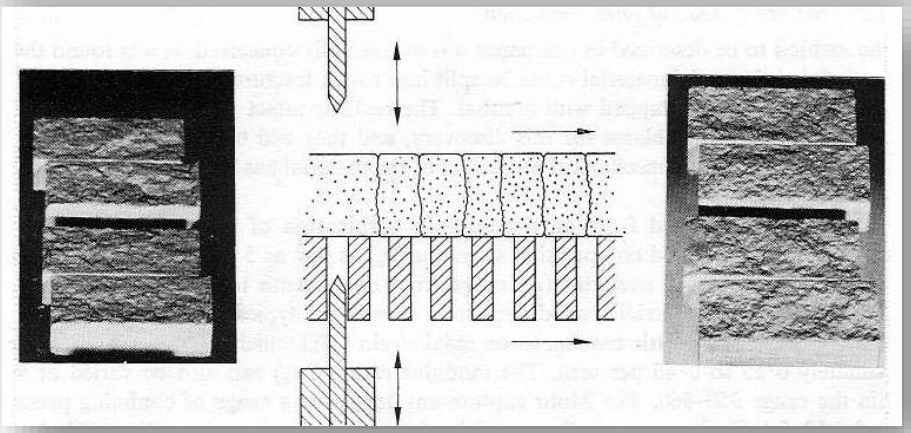


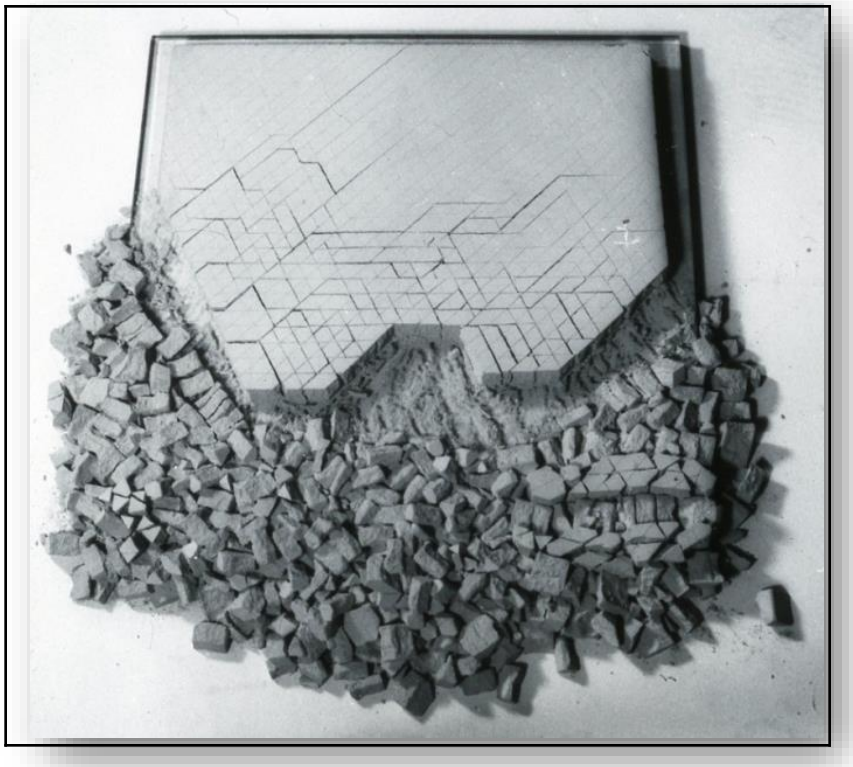
2. ROCK JOINTS

DO ROCK JOINTS HAVE REAL COHESION?

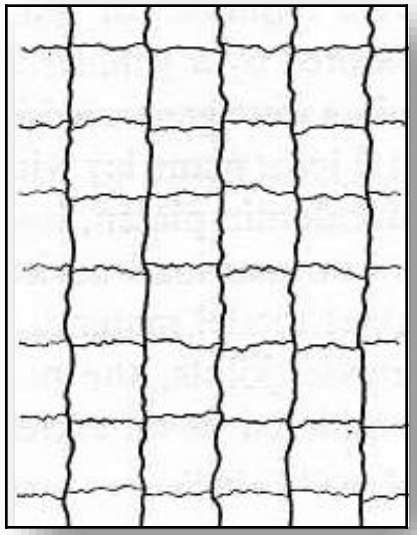
(‘everybody’ quotes ‘c’ and ‘ϕ’ ??)

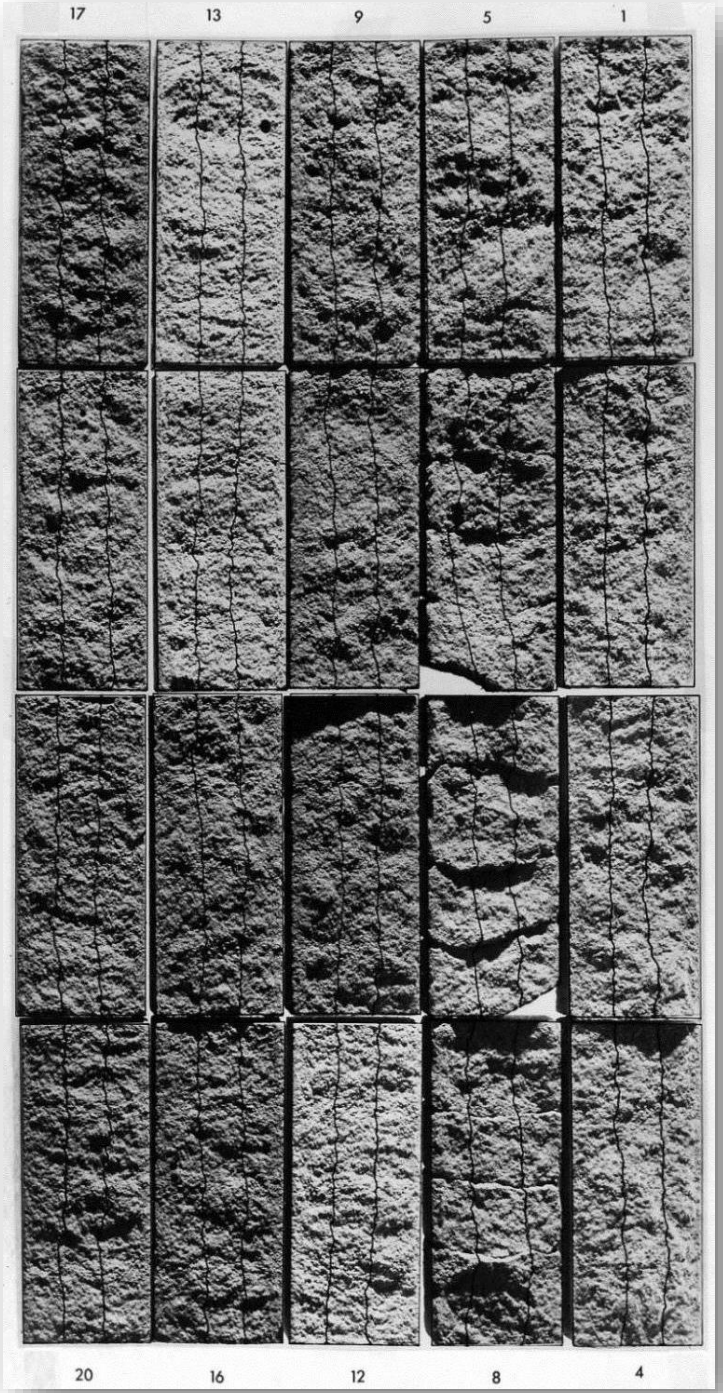




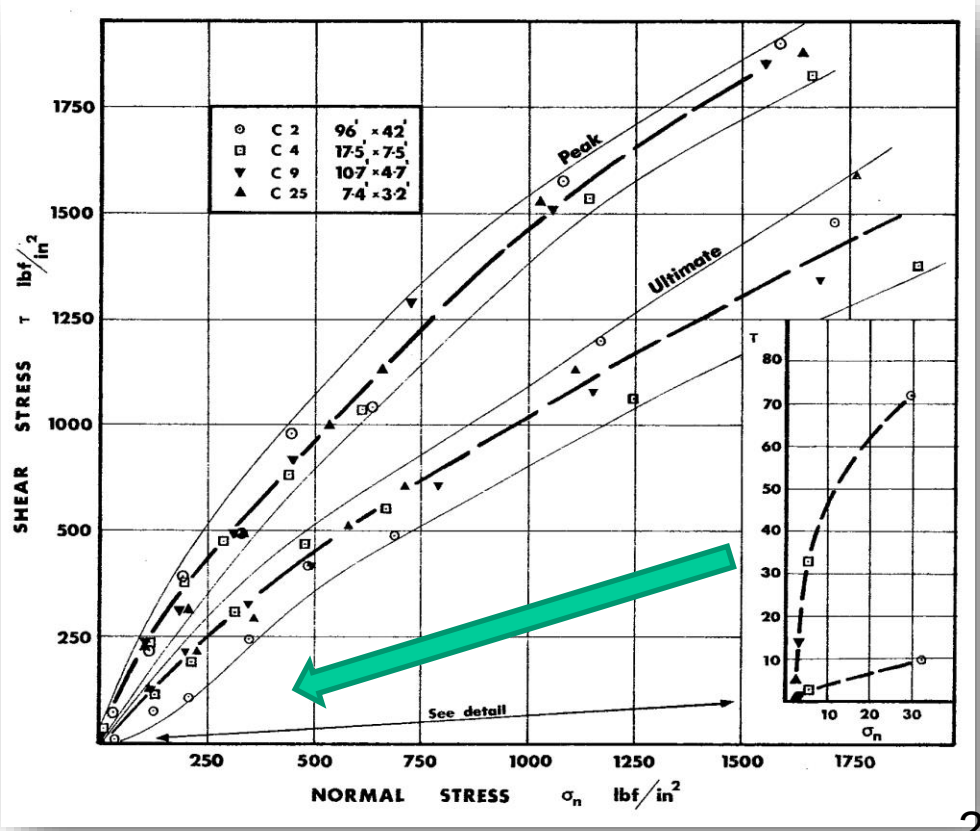


Only the 2nd (stepped) set of tension fractures has actual cohesion (NB, 1971)





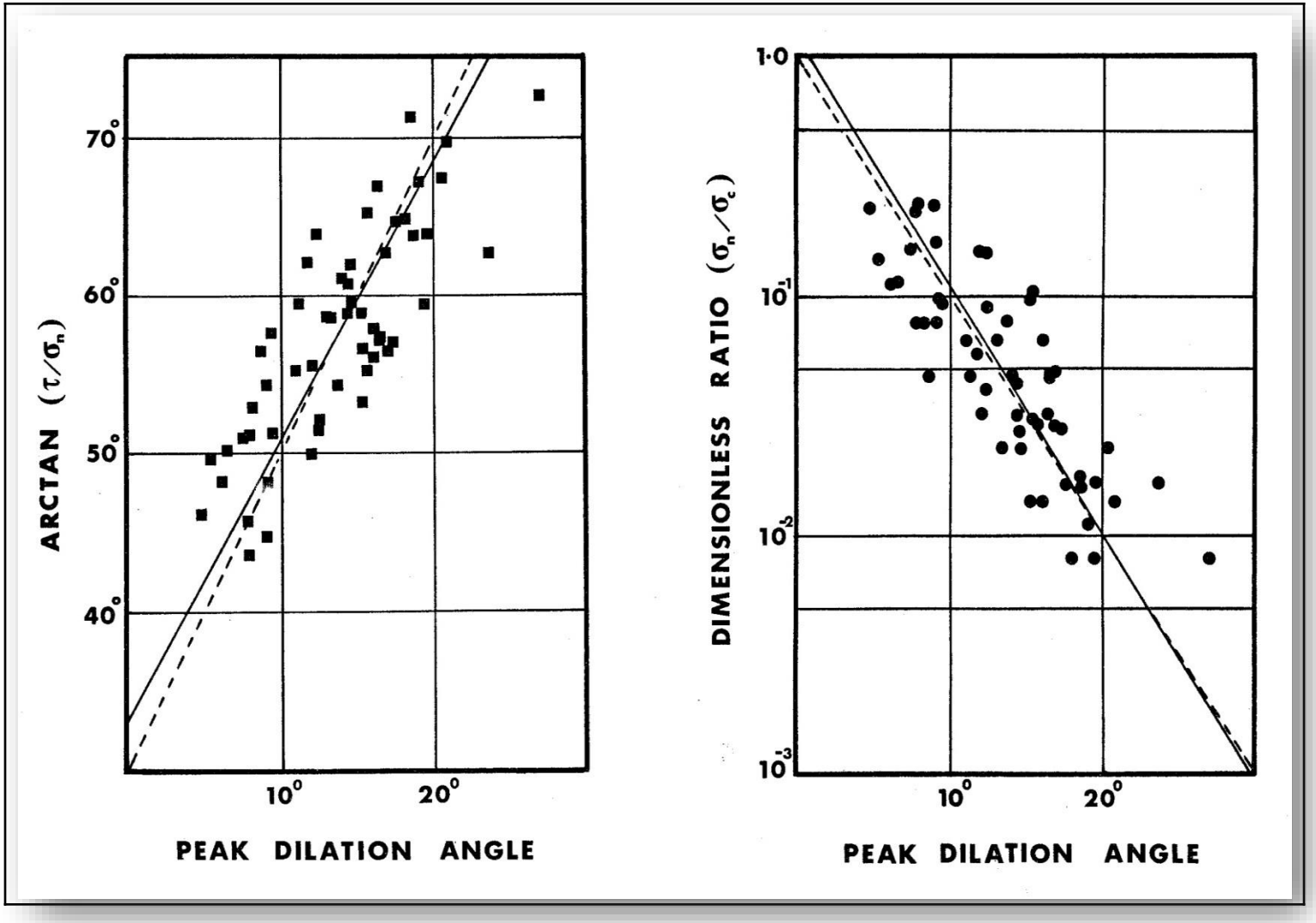
EVEN THESE ROUGH TENSION FRACTURES HAVE ***NO ACTUAL COHESION*** UNLESS STEPPED ("secondary") FRACTURES ARE TESTED

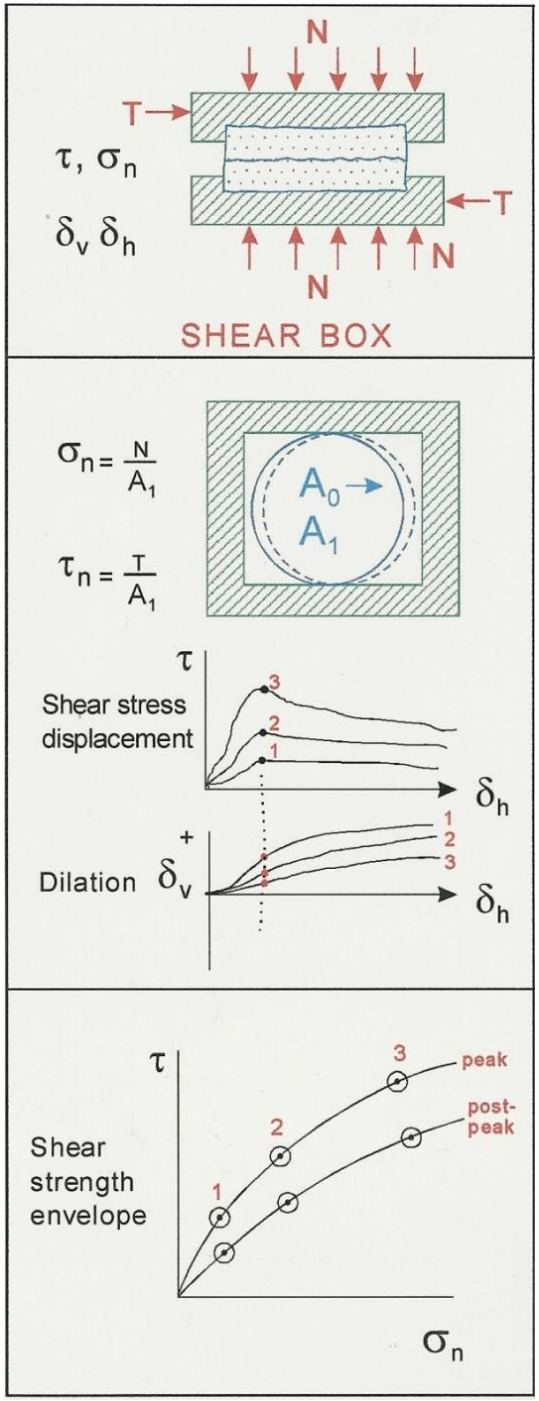


Peak strength criterion for the tension fractures

(------ = 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) CONFIRMED BARTON, 1973 SUGGESTION of VARIABILITY

'20' (*became variable JRC*)

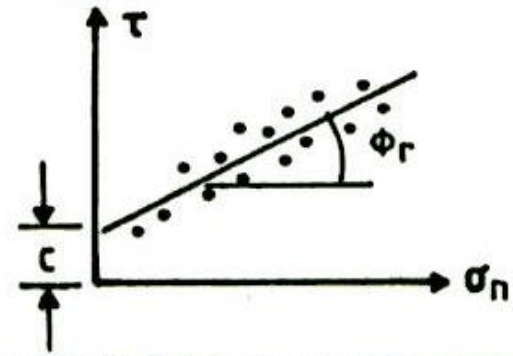
'UCS' (*became variable JCS*)

ϕ_b (no weathering) Barton, 1973, 1976

ϕ_r (weathered joints) Barton & Choubey, 1977

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

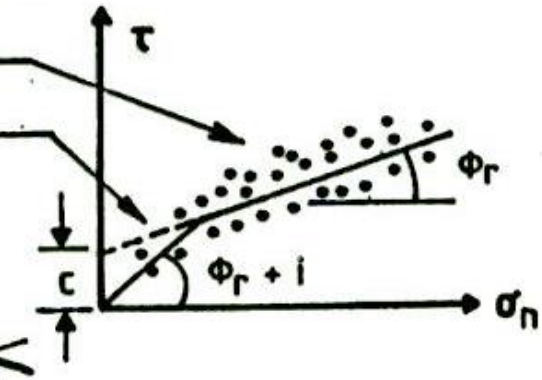
Mohr - Coulomb



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

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

Patton

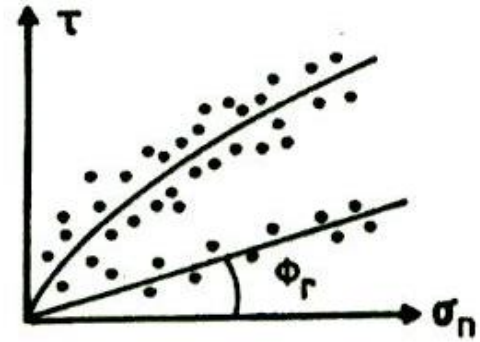


(3) $$\tau = \sigma_n \tan \left[JRC \log \left(\frac{JCS}{\sigma_n} \right) + \Phi_r \right]$$

JRC = joint roughness coefficient

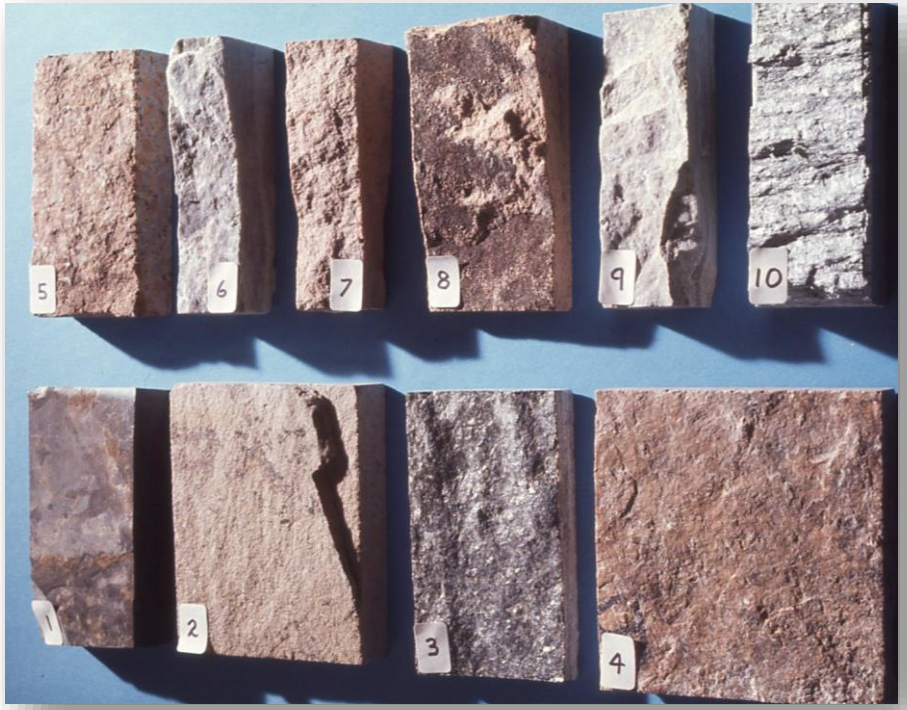
JCS = joint wall compression strength

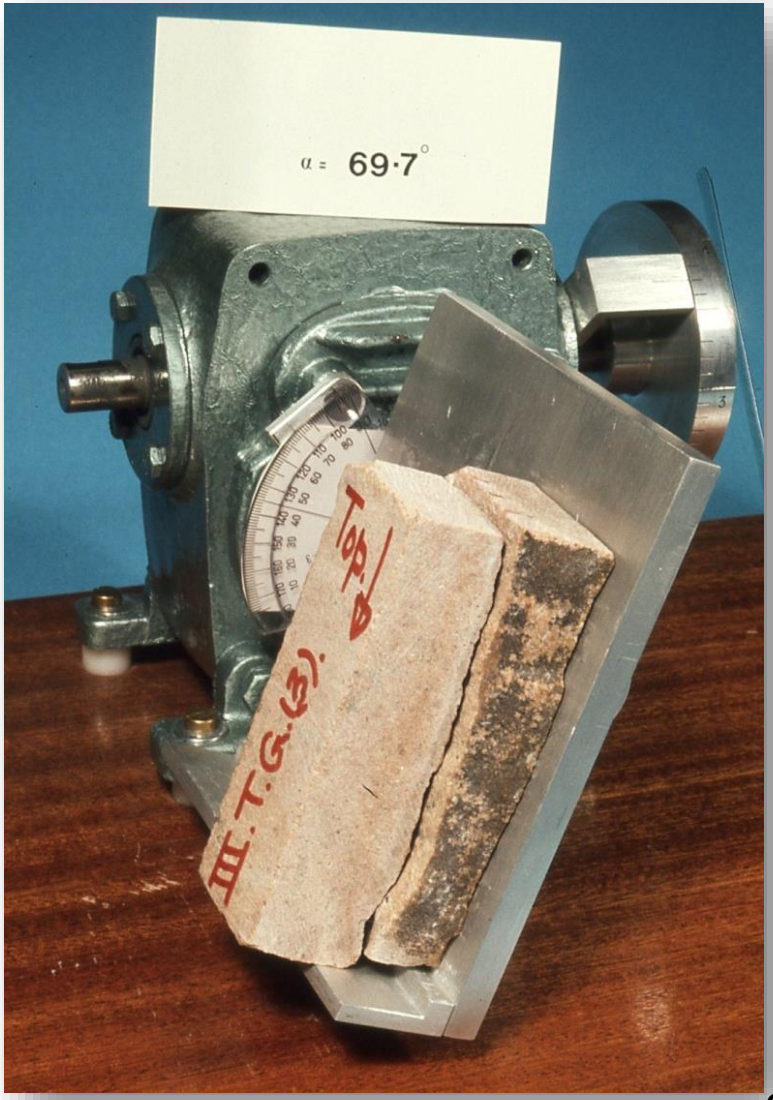
Φ_r = residual friction angle



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

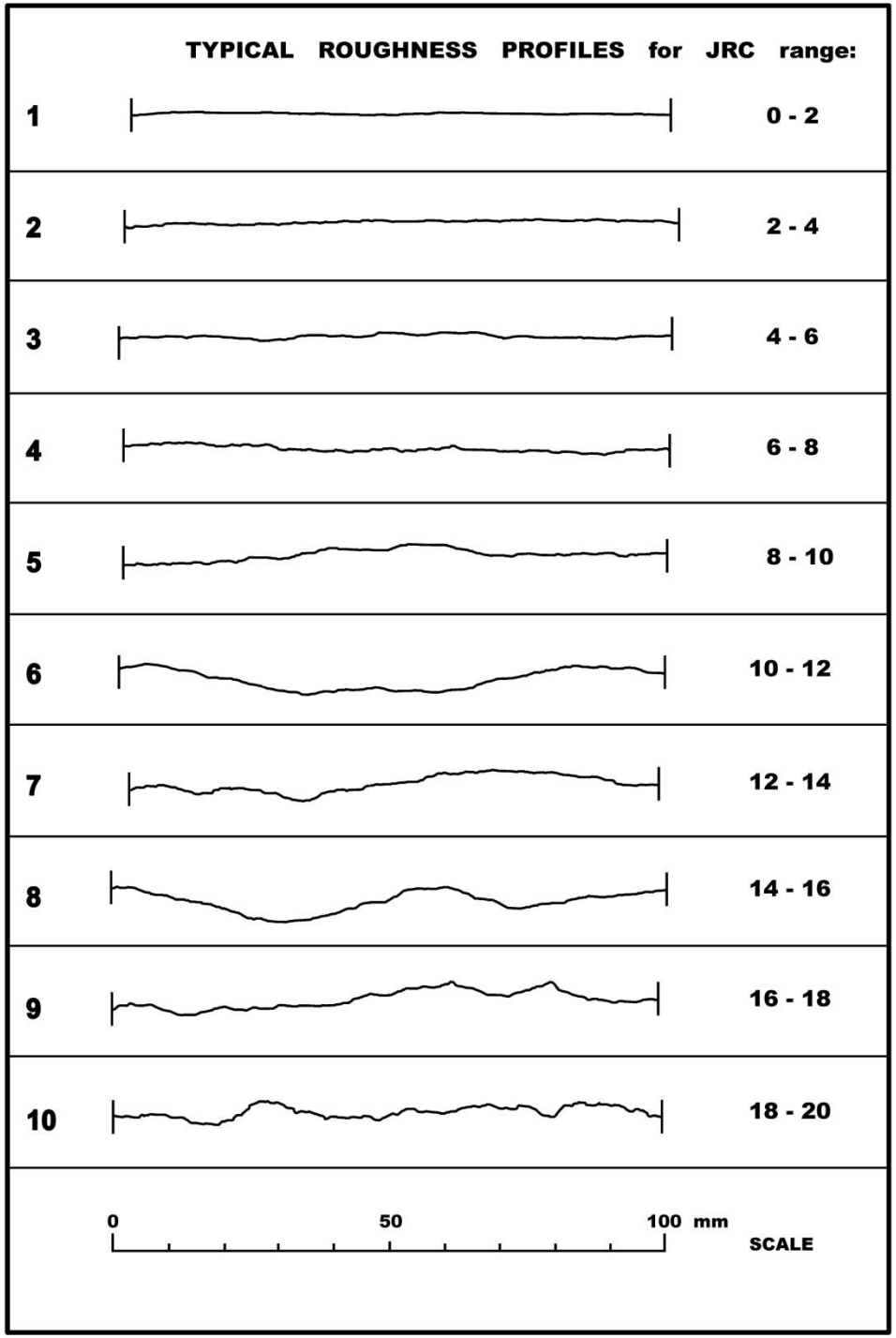
(Barton and Choubey, 1977)

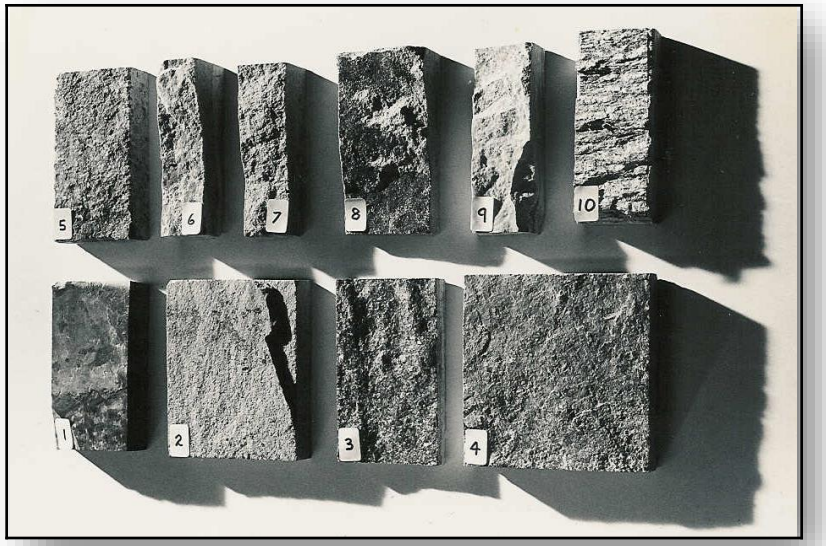


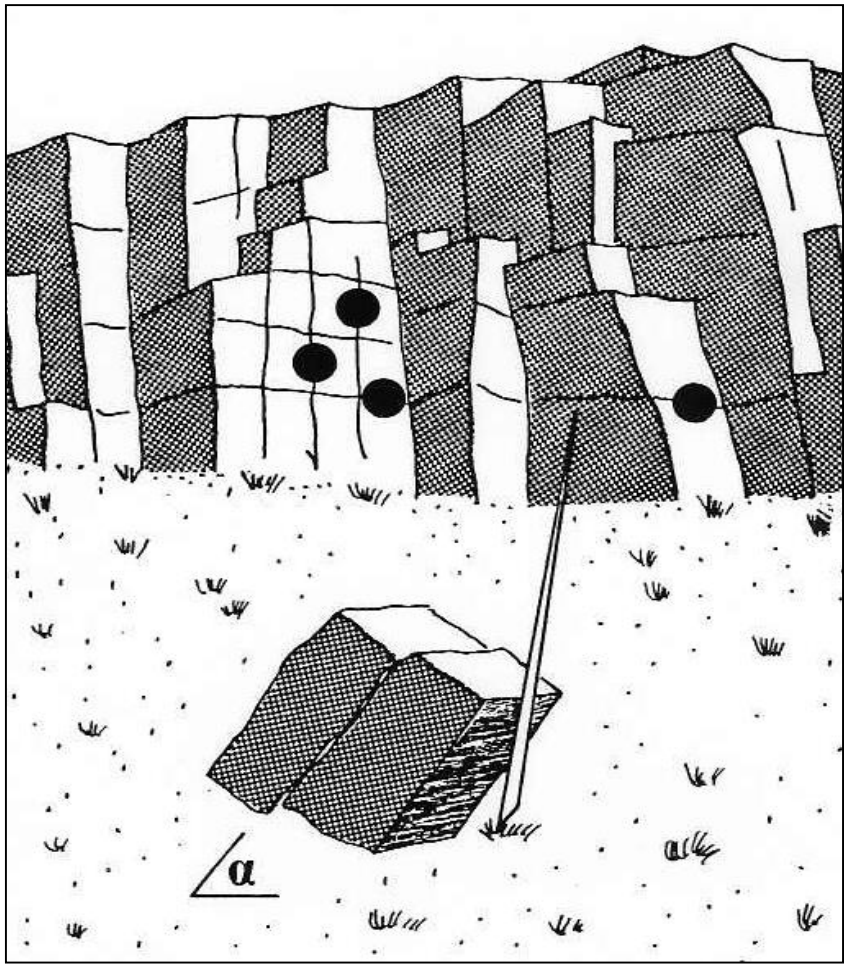


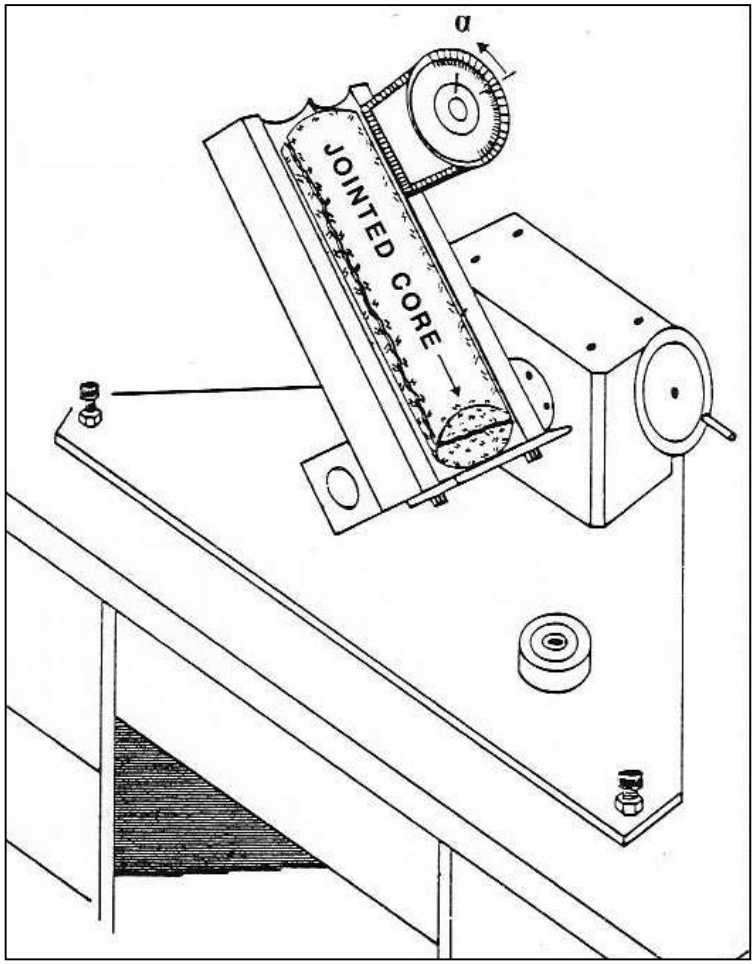
VISUAL MATCHING OF ROUGHNESS – for JRC: SIMPLE but LIMITATIONS

(Barton and Choubey, 1977)

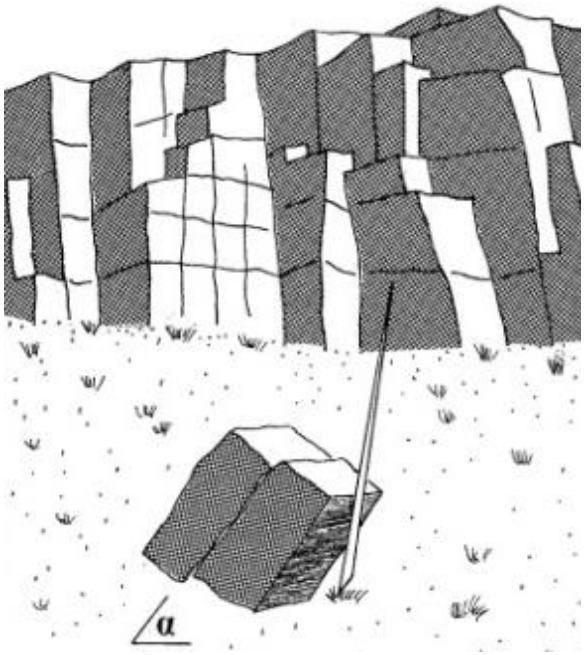


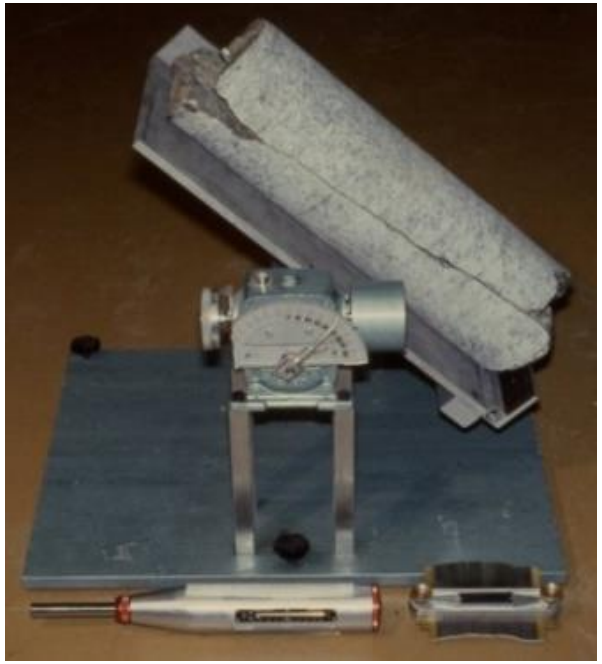




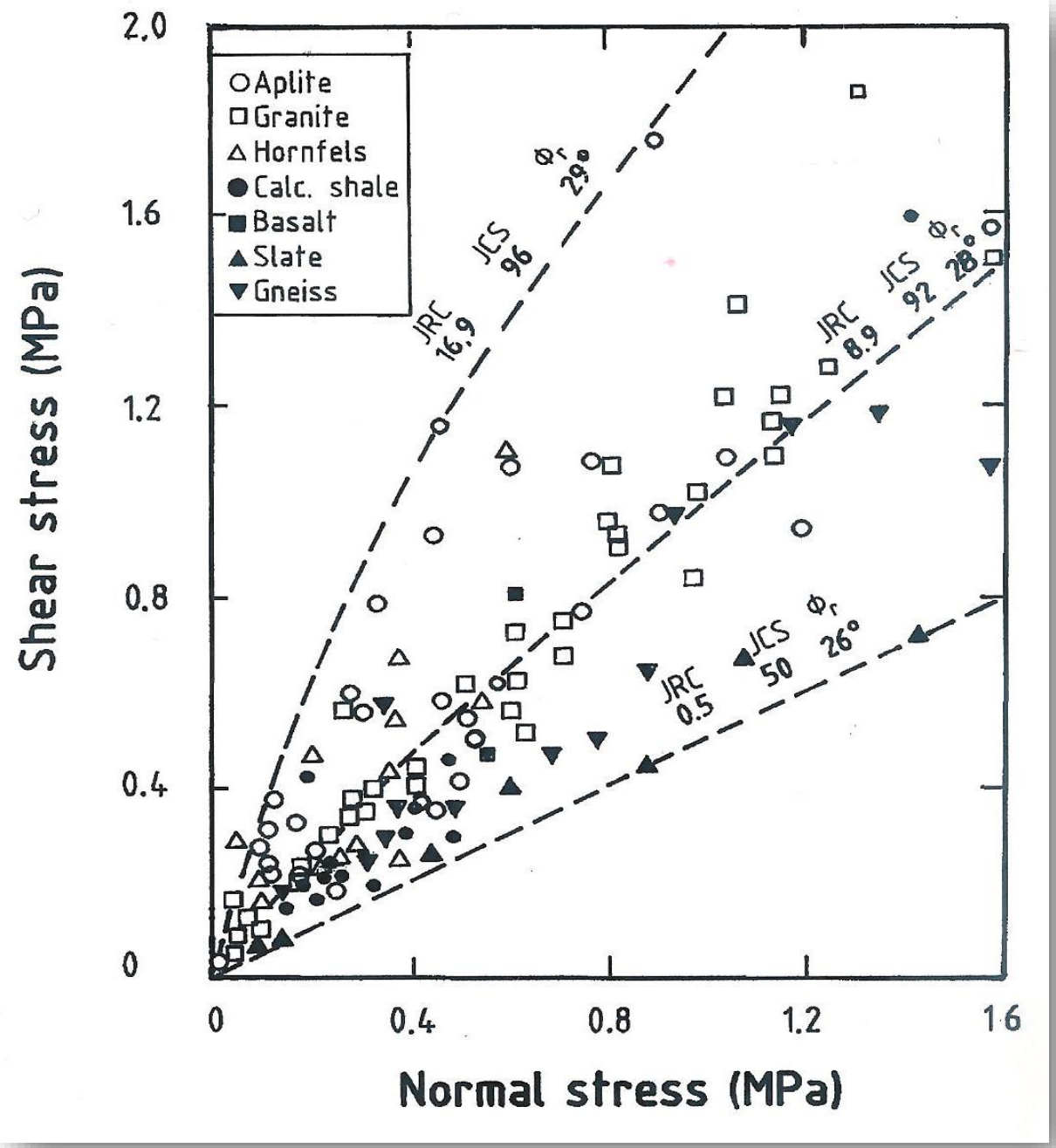


The ***tilt test*** has its origin in characterizing rock joints (Barton and Choubey,1977). Equipment is ultra-simple. Stress application is ultra-uniform (= gravity). Transfer: TerraTek to Schlumberger.





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



130 rock-joint samples

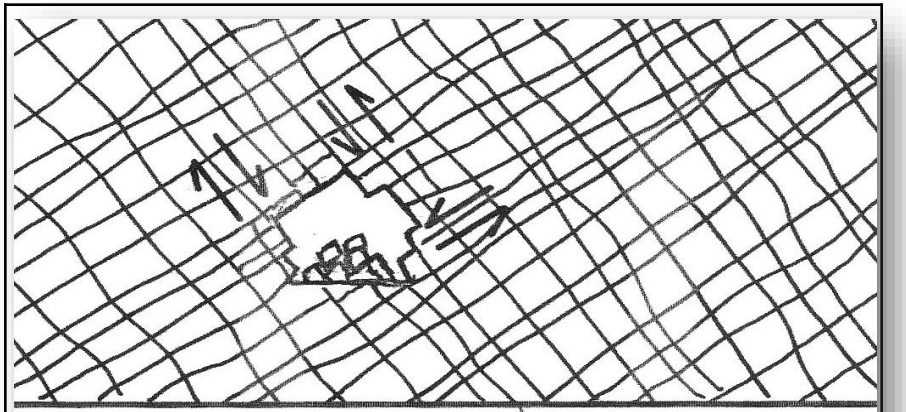
(Barton and Choubey, 1977)

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 $\varphi_r = 26^o$



BARTON-BANDIS (scaled)

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

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

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

Now: JRC JCS φb (now φr)

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

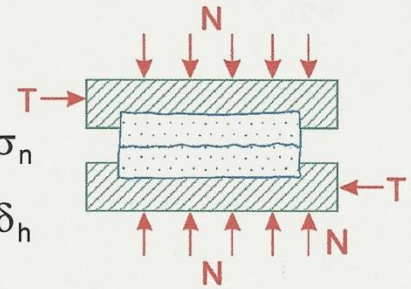
(Error due to *'downloadable rock mechanics'*)

EXAMPLE of ROUGHNESS CONTRAST – DST YUCCA MOUNTAIN (NON-LITHOPHYSAL UNIT)

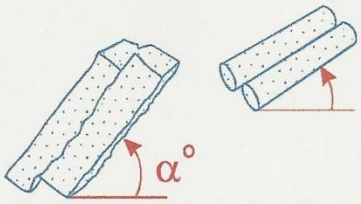
JCS < UCS

(UCS in all the 'islands' not cut by the waves)

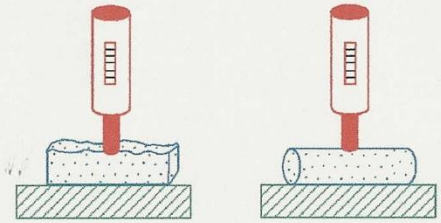
SHEAR BOX AND INDEX TESTING OF ROCK JOINTS



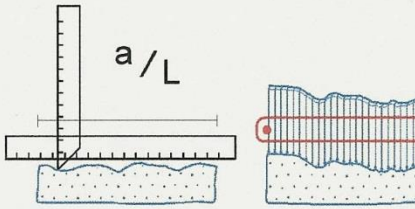
SHEAR BOX



TILT TEST



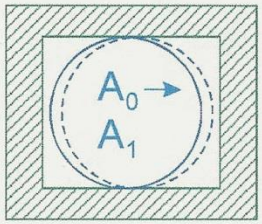
SCHMIDT HAMMER

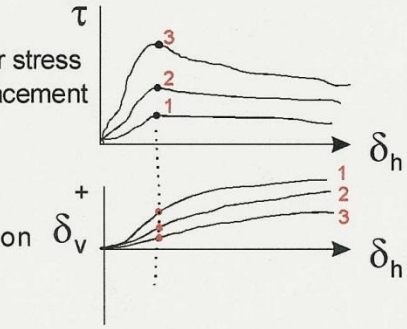


ROUGHNESS PROFILE

$= \frac{N}{A_1}$

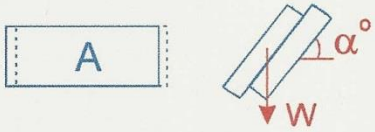
$= \frac{T}{A_1}$





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

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



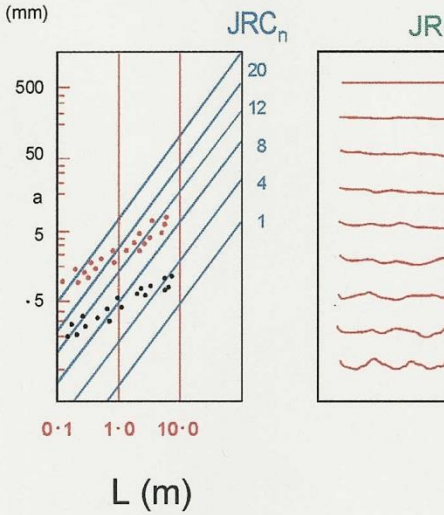
$$\phi_r = (\phi_b - 20^o) + 20 \, r_5/R_5$$

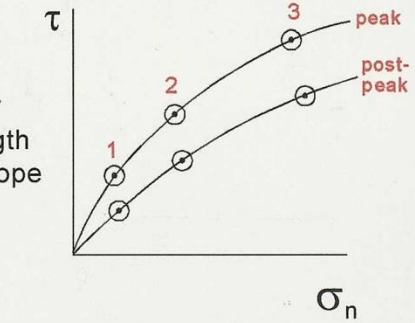
r	R
30	40
~~25~~	~~33~~
31	41
32	~~35~~
32	40
~~24~~	42
31	~~34~~
~~29~~	41
~~23~~	~~36~~
~~26~~	~~37~~

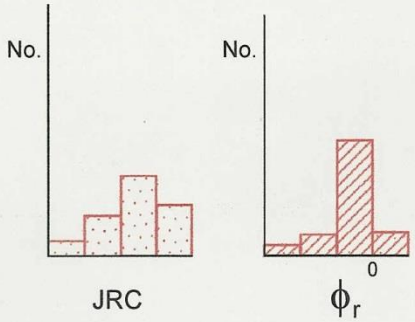
$\bar{r}_5 \rightarrow JCS_o$

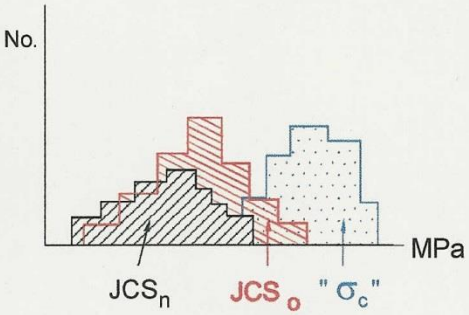
$\bar{R}_5 \rightarrow$ "σ_c"

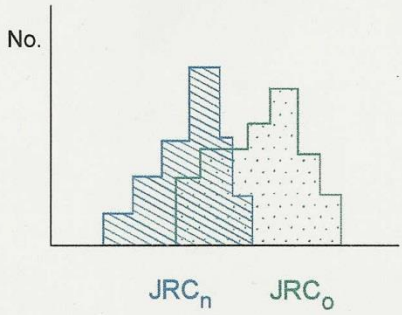
$\bar{r}_5 = 31{\cdot}1 \quad \bar{R}_5 = 40{\cdot}8$



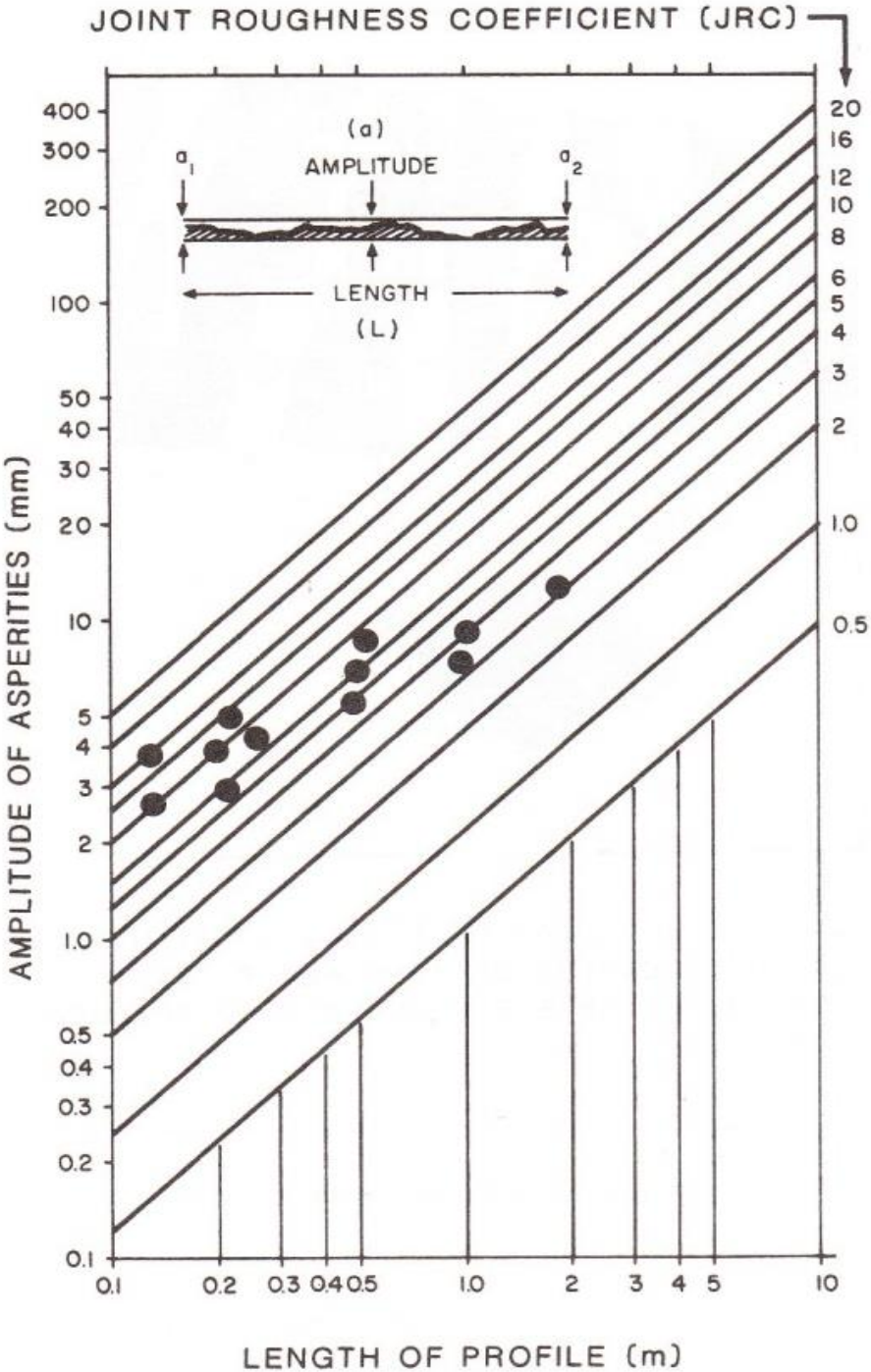






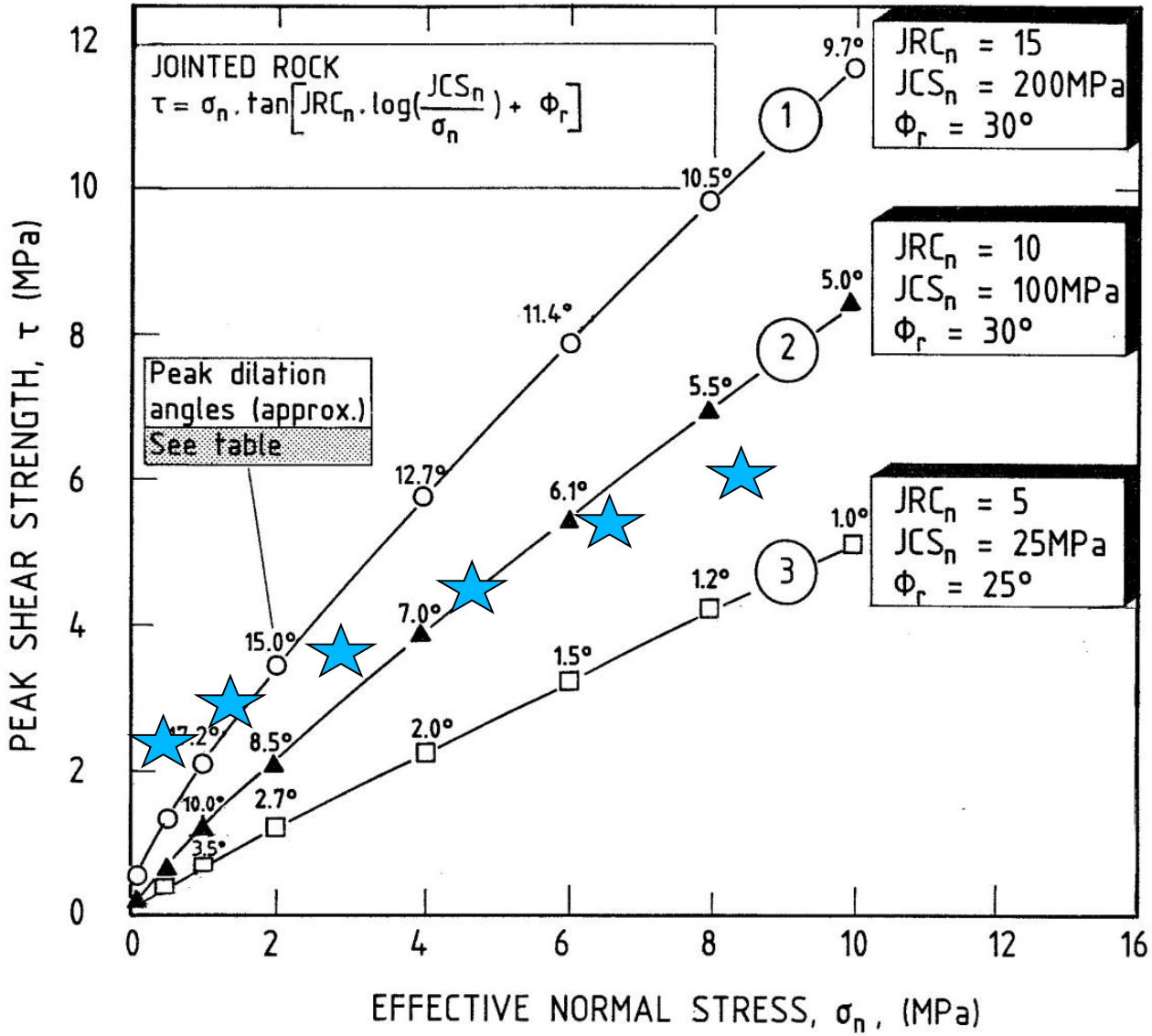


The ‘a/L’ method



ADDITIONAL CAUSE OF *'BELIEF IN COHESION'*

(why so many think it actually exists)

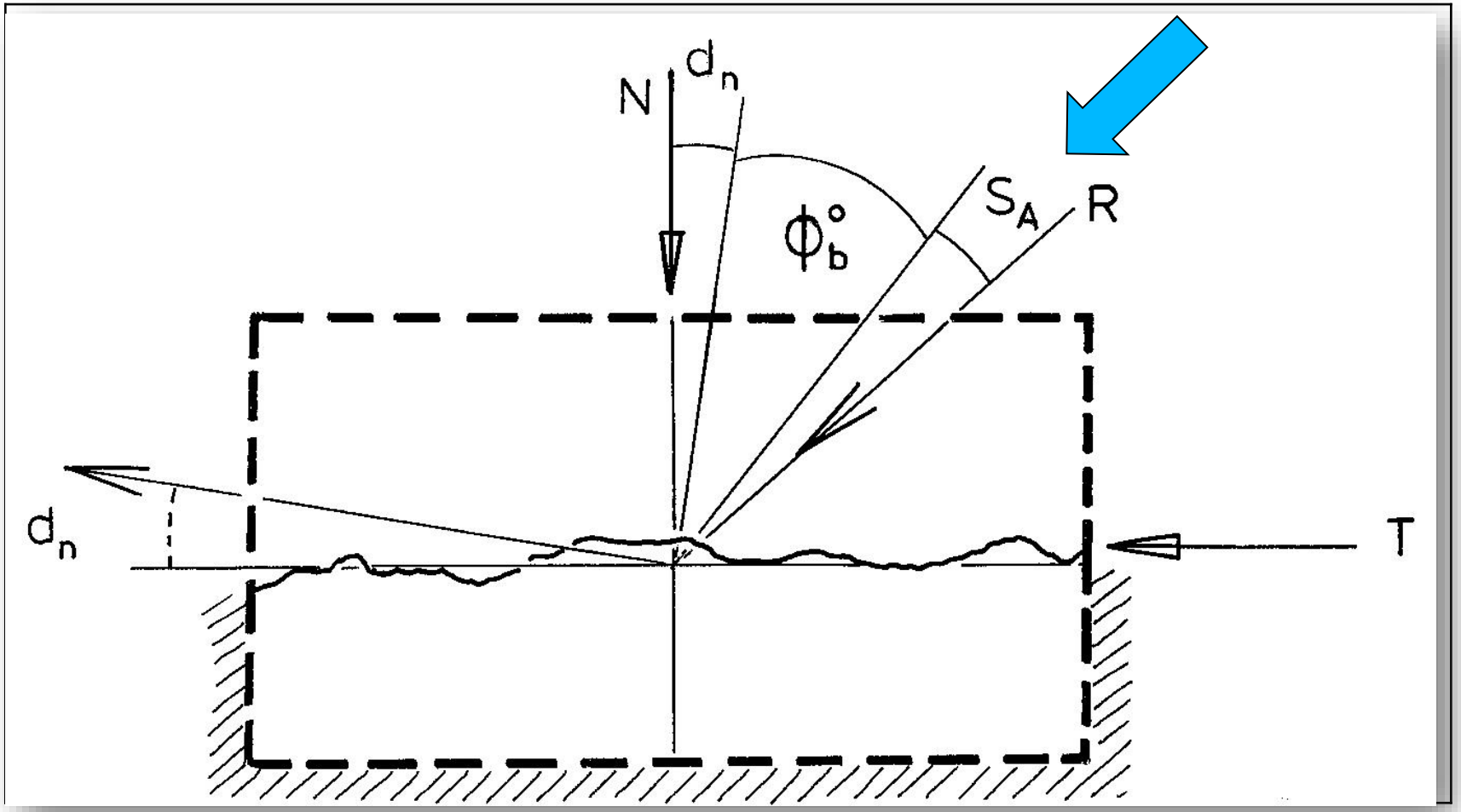


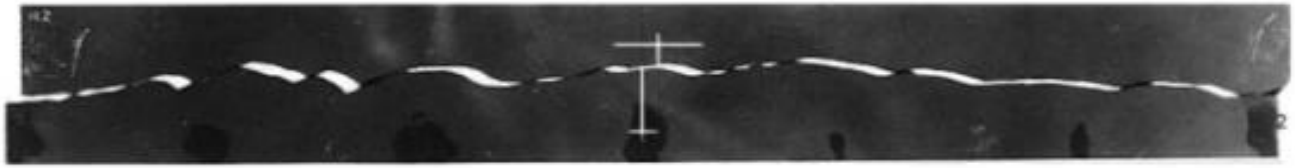
ISRM, TEST LABS recommendation of pre-loaded ***multi-stage shear testing*** ***(of same sample)*** if insufficient rock joint samples.

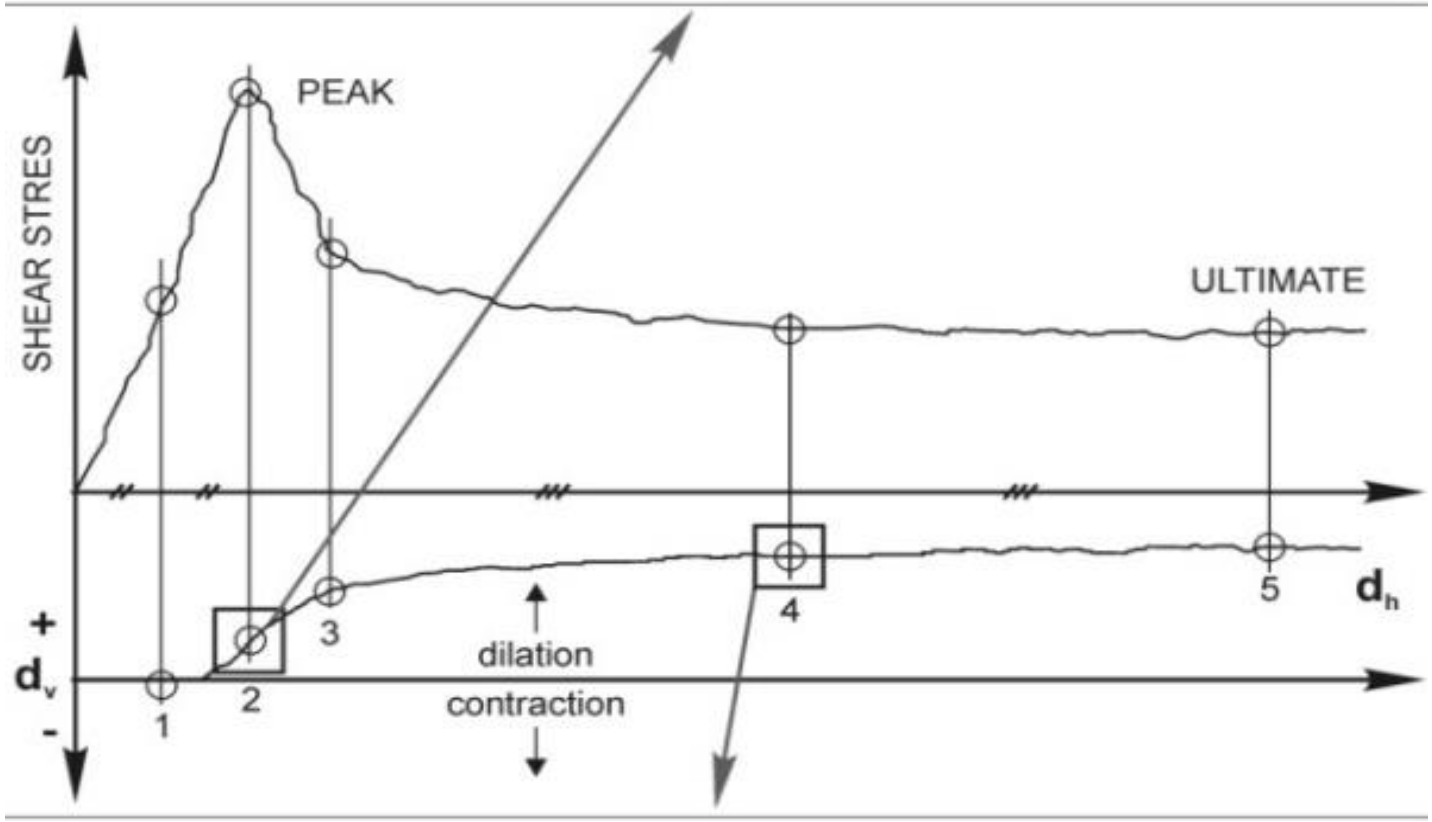
Results in 'rotation' of the strength envelope…… therefore artificial 'cohesion'

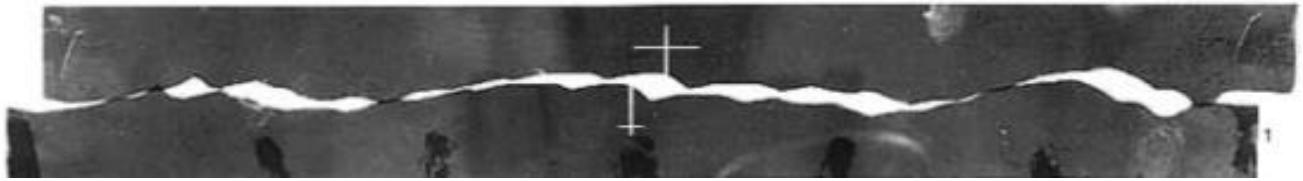
AN OVER-LOOKED COMPONENT OF SHEAR STRENGTH

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

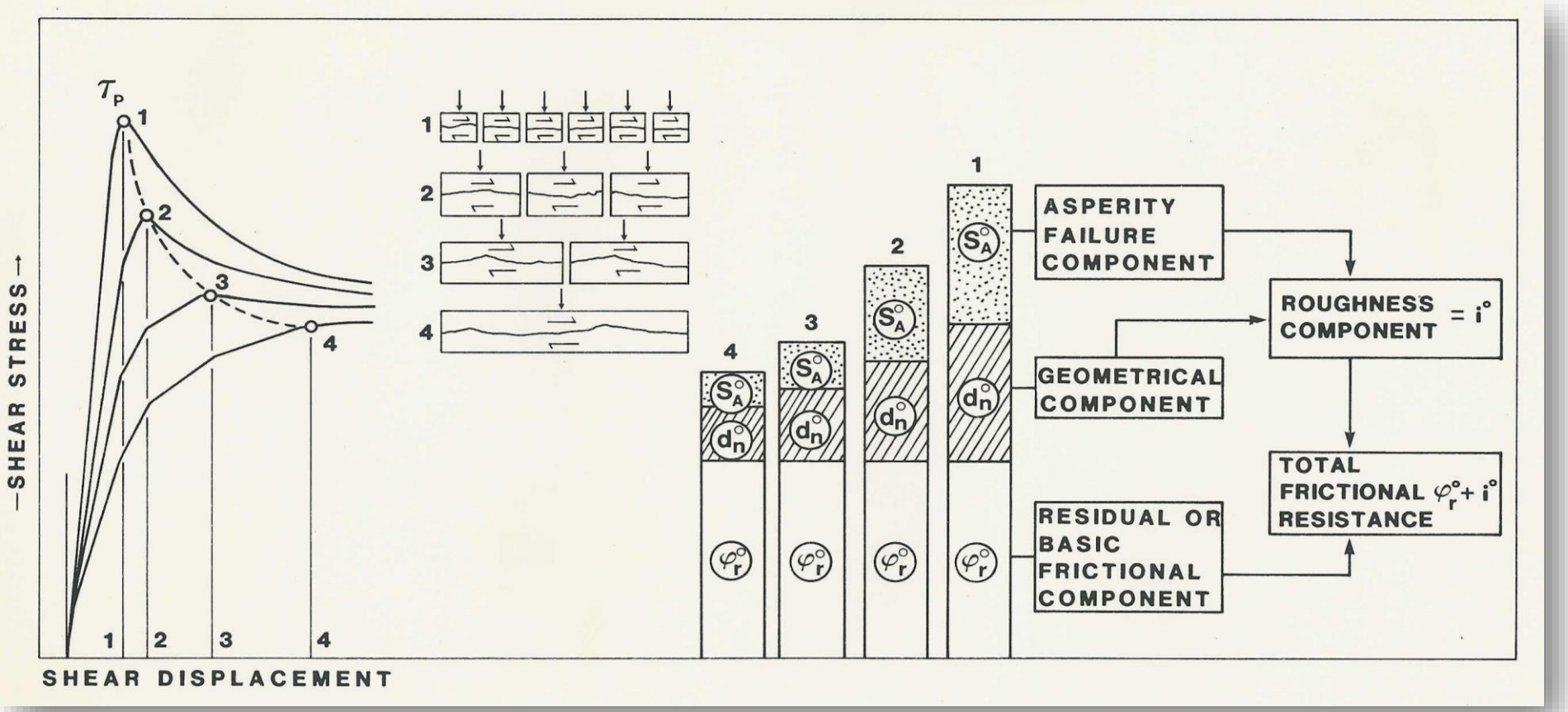








Asperity strength S_A due to highly stressed, failing asperities (i.e. has to be more than just a dilation angle).

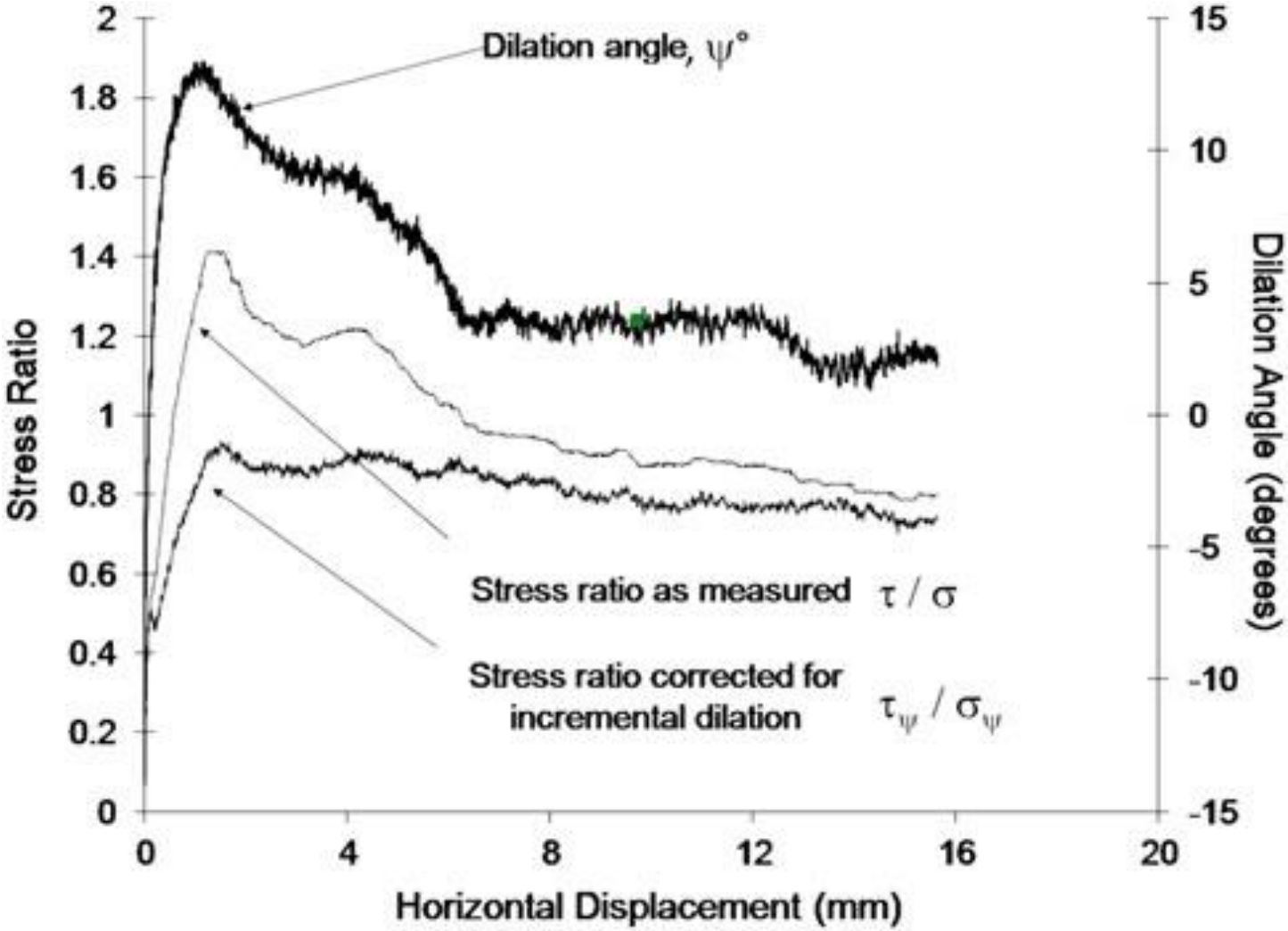


The asperity component S_A (Barton, 1971 and Bandis, 1980) .

***JRC* (or φ_r) cannot be back-calculated *by only subtracting dilation (d_n) from peak strength*.**

Φ_r or Φ_b would then be *dangerously too high* (and/or JRC would be incorrect).

This is a 'common' Ph.D error!

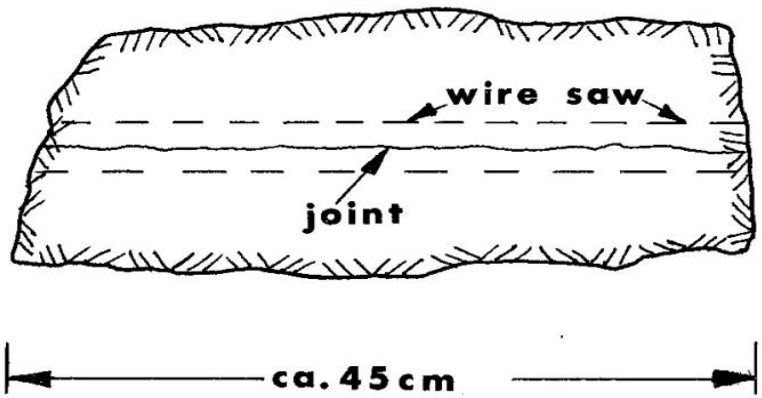


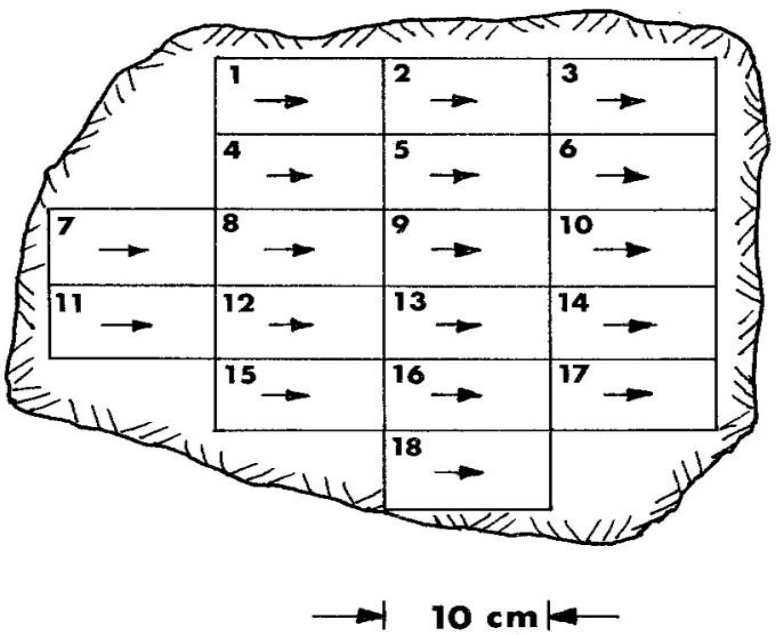
Hencher error: $\tan^{-1}(\tau/\sigma_n)$ minus $d_n \neq \phi_b$

What to do with so-called 'ϕ_b' values of 40°??

3. SCALE EFFECTS concerning ROCK JOINTS

IN SITU BLOCK

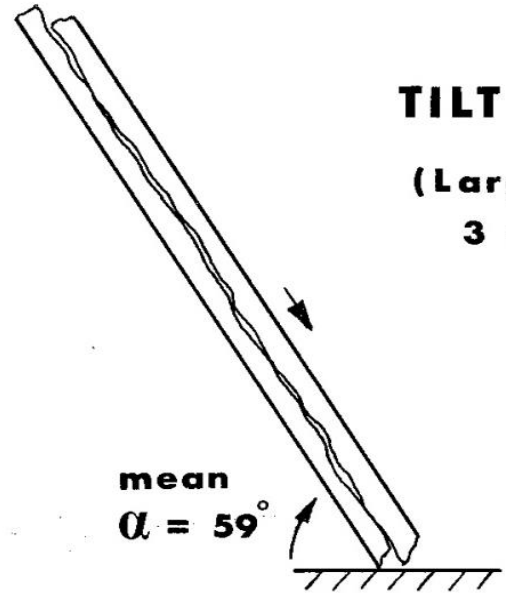




TILT TEST

(Large scale)

3 tests

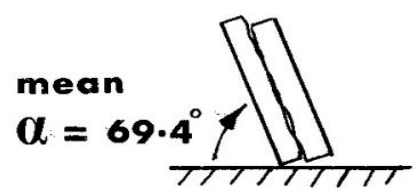


(1 sample)

TILT OR PUSH TEST

(Small scale)

54 tests



(18 samples)

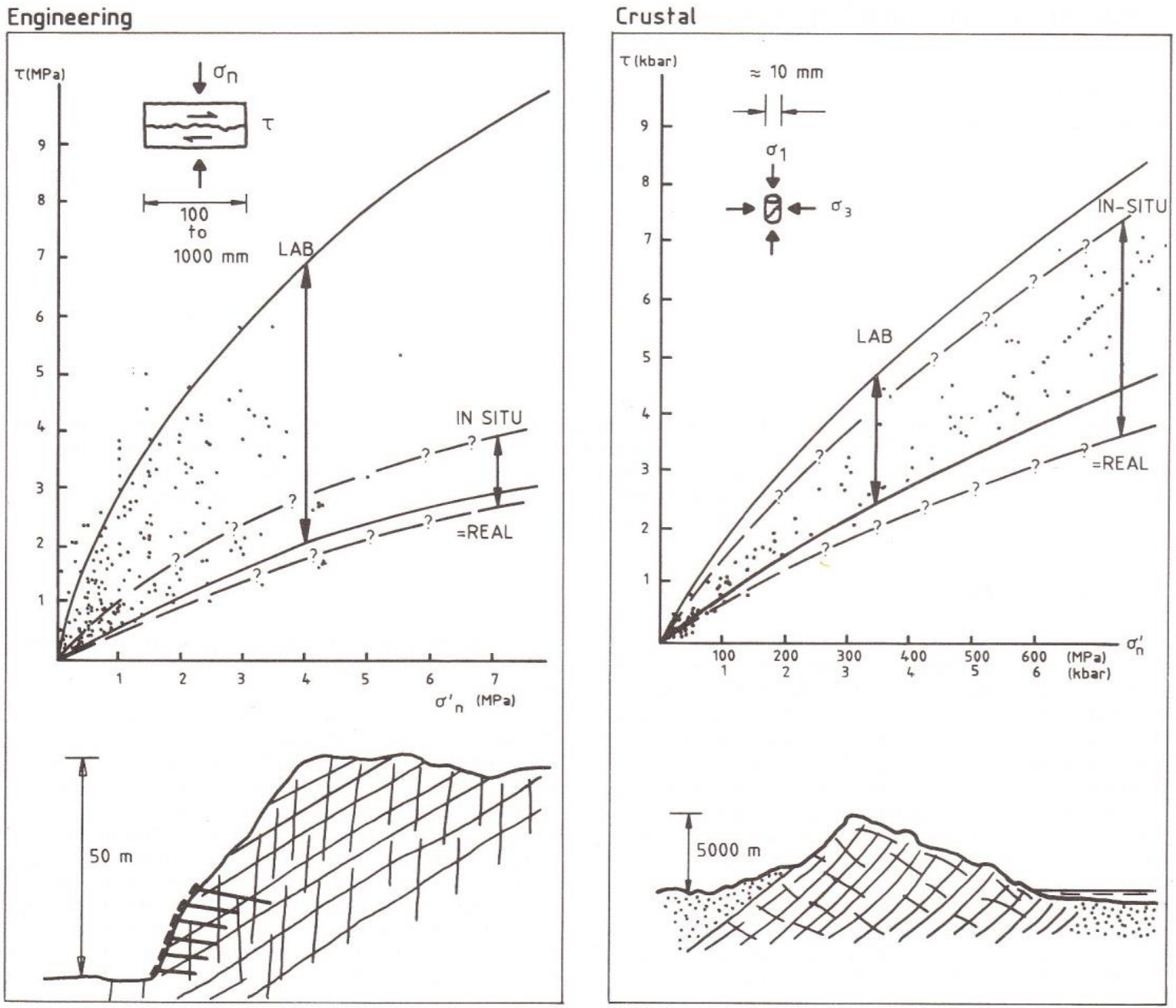


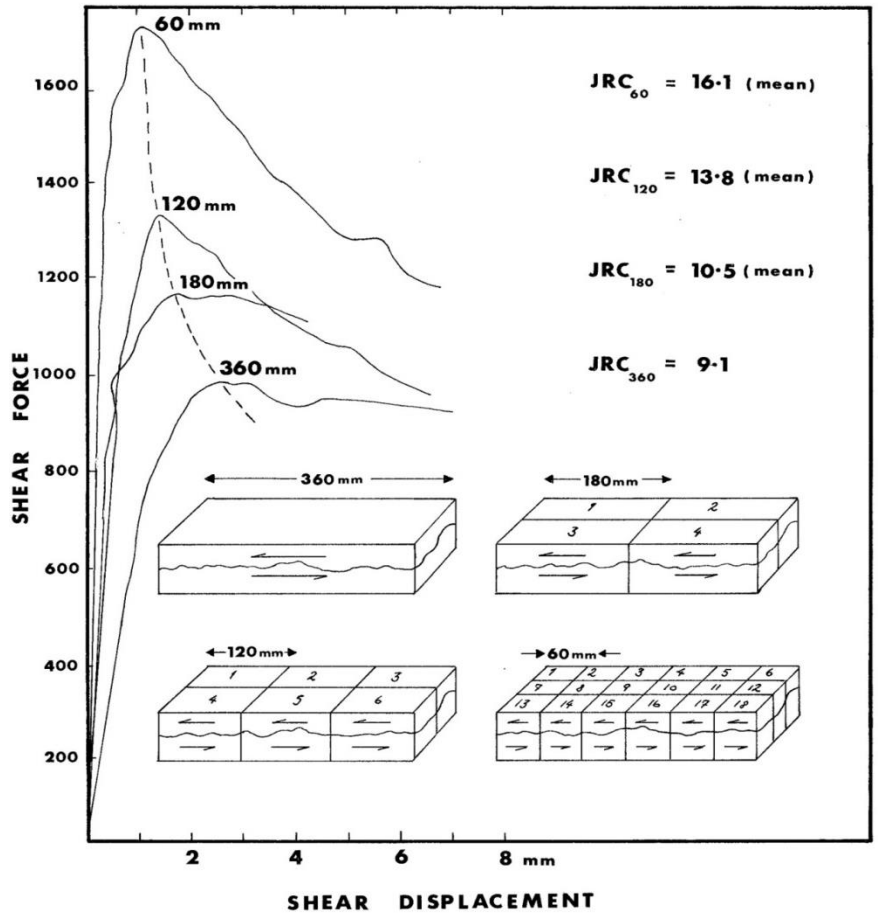
Figure 1. Different magnitudes of scale effect are expected under high and low stress. Adapted from Barton (1976). Additional data from Byerlee (1978).

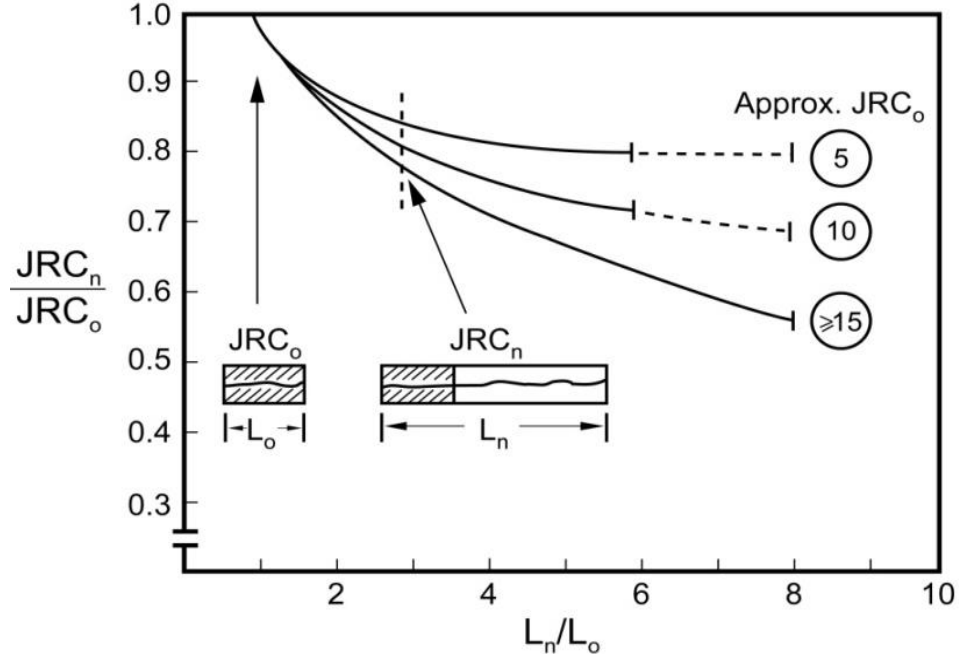
Many hundreds of DST and post -fracture triaxial data collected.

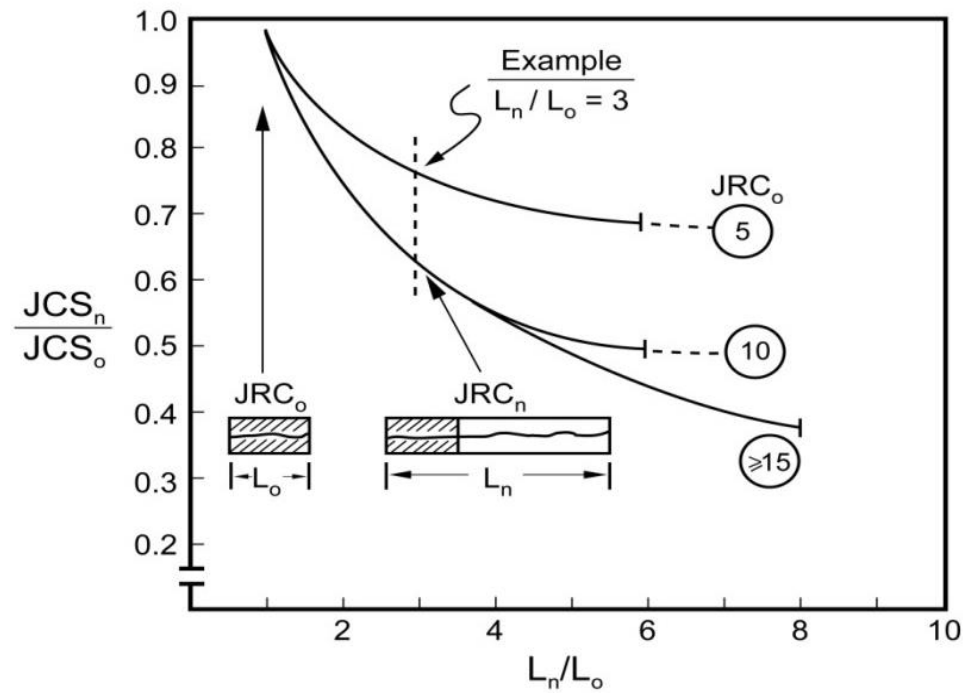
Barton, 1990.

(Loen scale-effects workshop)

Byerlee, 1968 suggests $\tau = 0.85\ \sigma_n$?



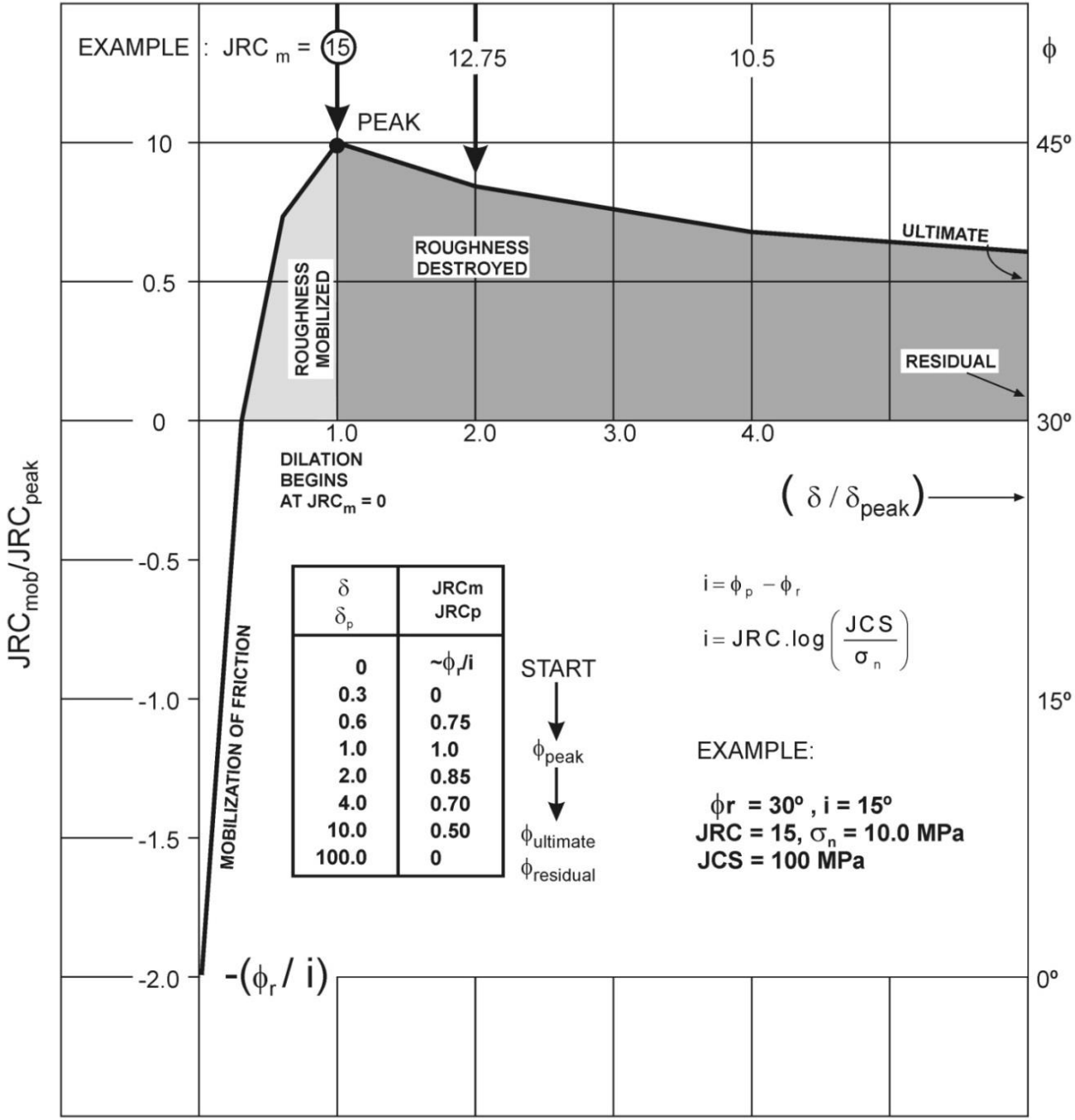




(Bandis et al. 1981)

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

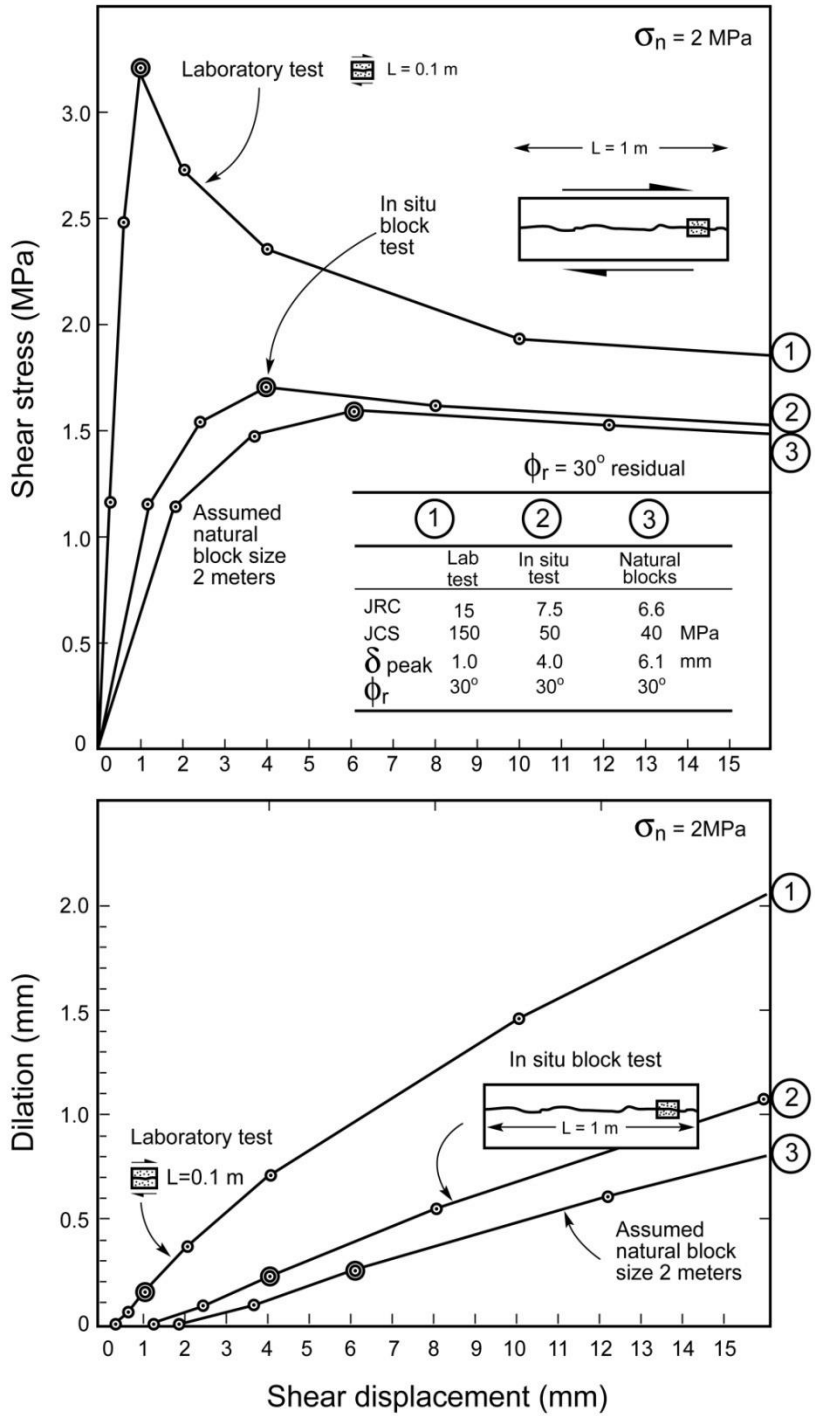
$$JCS_n \approx JCS_o \, [\, L_n/L_o]^{-0.03\,JRCo}$$



The JRCmobilized concept

(Barton, 1982)

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



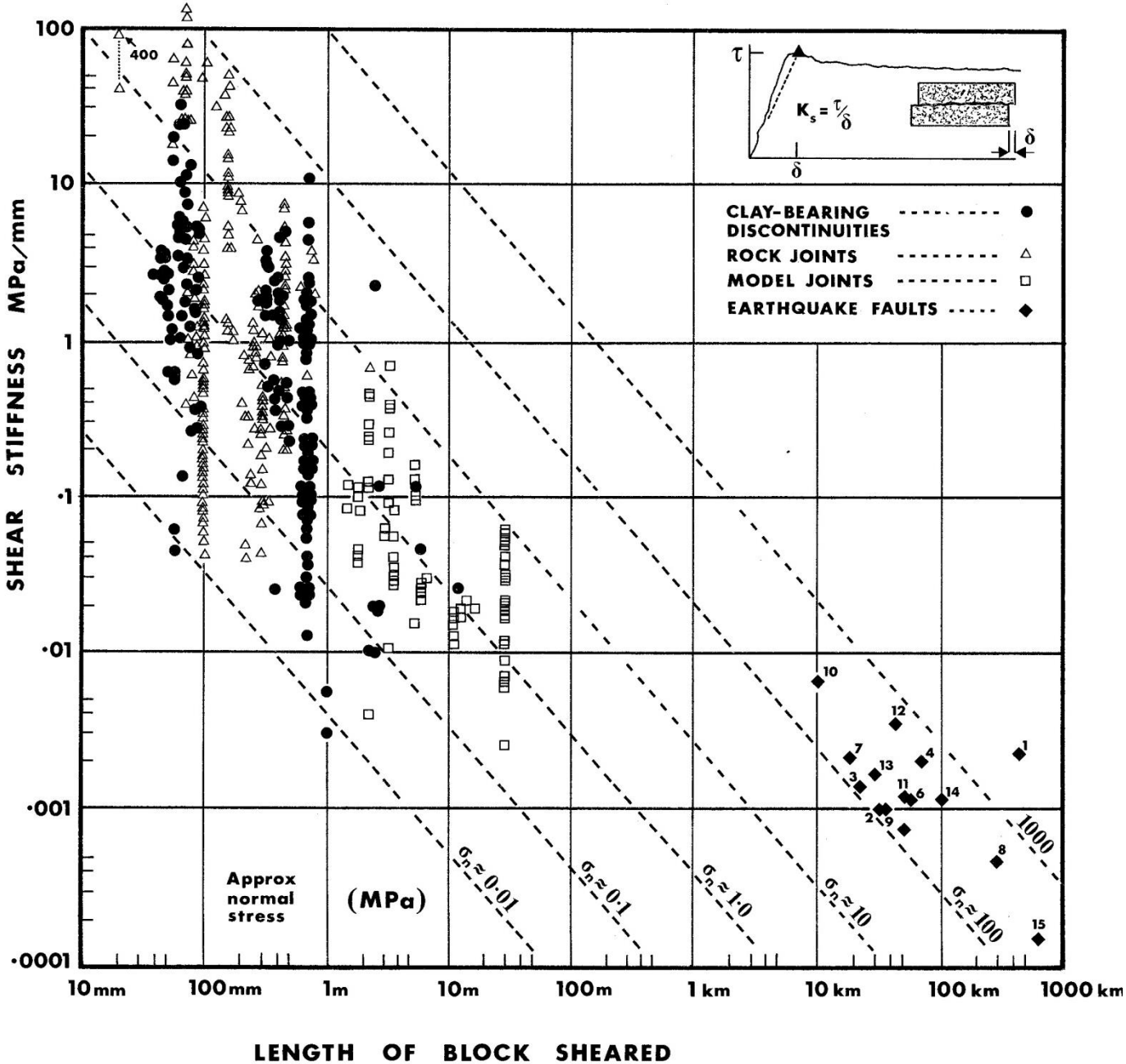
	1 Lab test	2 In situ test	3 Natural blocks	
JRC	15	7.5	6.6	
JCS	150	50	40	MPa
δ peak	1.0	4.0	6.1	mm
ϕ_r	30°	30°	30°	

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)

4. STRESS TRANSFORMATION ERRORS?

σ_1 and σ_3 transformed to τ *and* σ_n

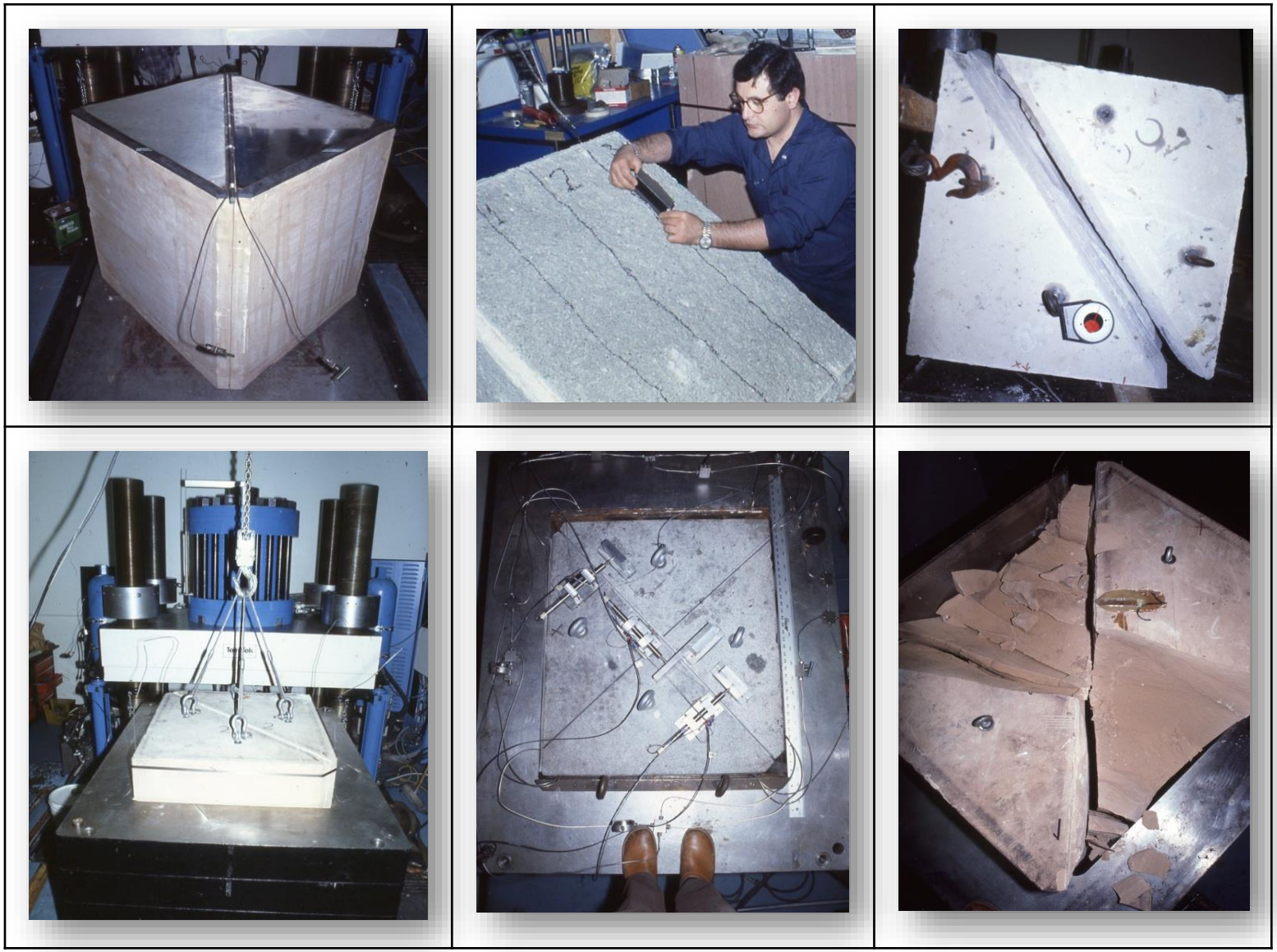
The classic equation assumes co-axial stress and strain, no discontinuity, no shearing, no dilation.

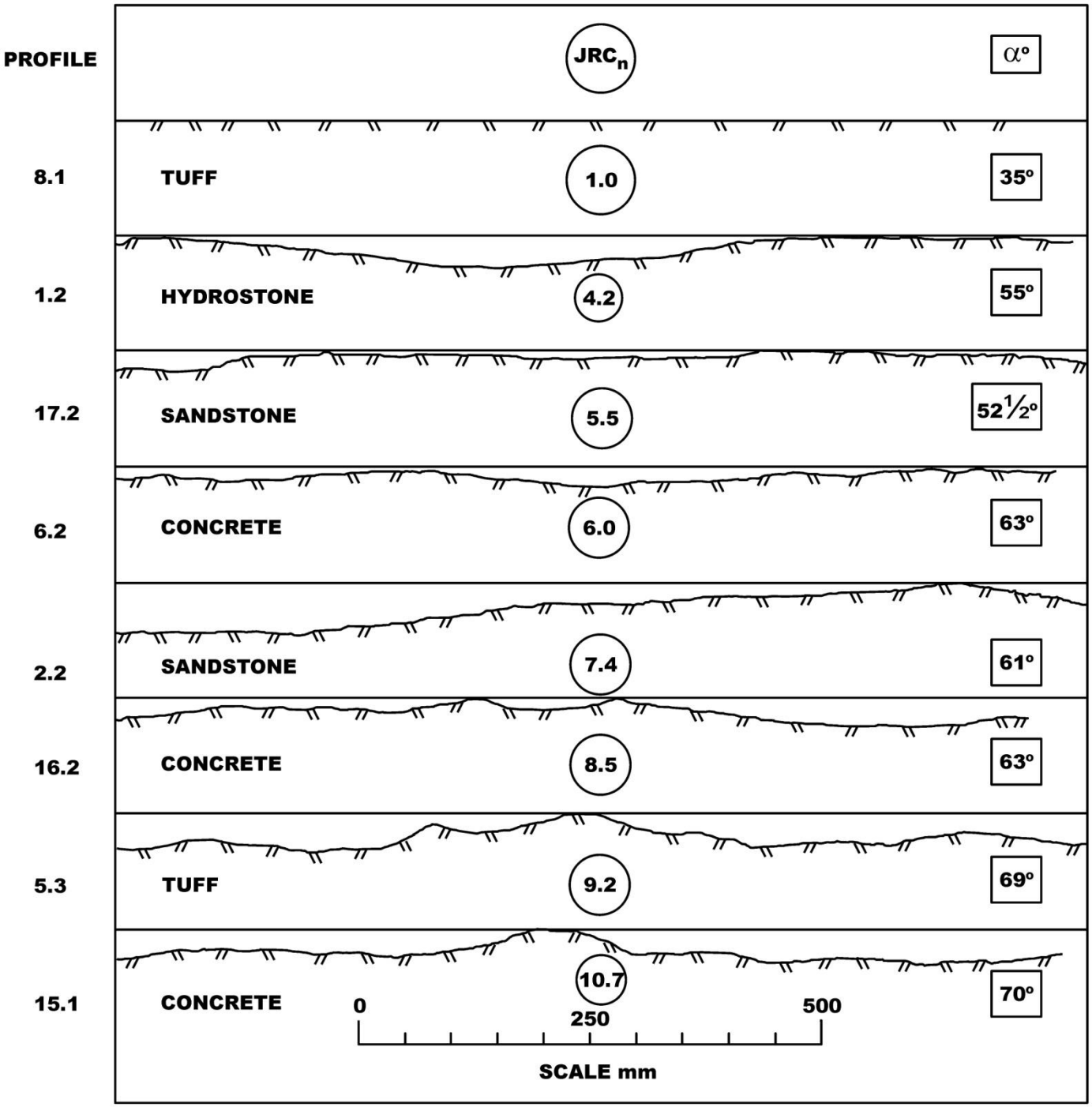
Potential failure surfaces in geotechnical materials (obviously) cannot satisfy these basic assumptions.

Rock joints, rock masses, rockfill, OC-clay, dense sand (i.e. most geo-materials) shear and dilate where we require the stress transformation.

What can we do?.....add mobilized dilation.

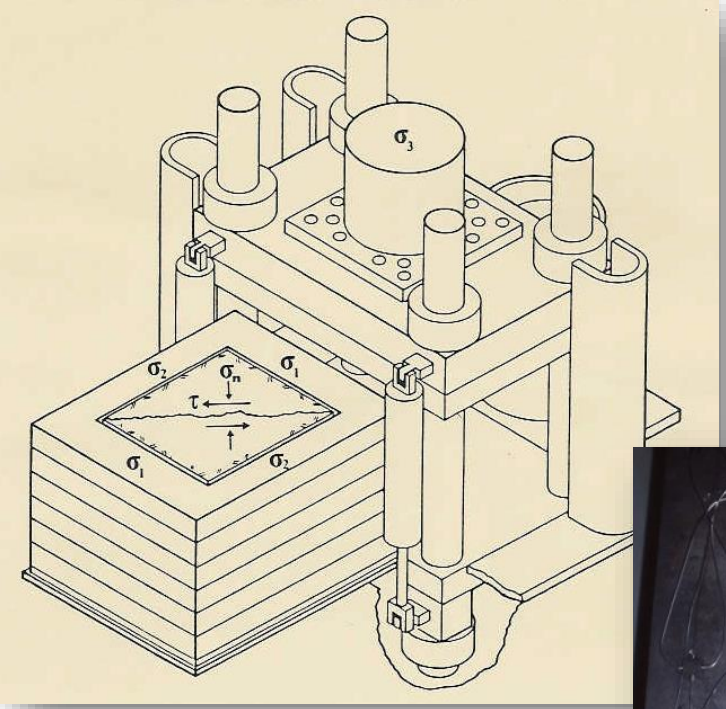
Diagonally fractured 1 m3 blocks, usually 'refusing' to shear when expected.(Initial diagonal fractures were generated in controlled σ_1 stress field. Then biaxial loading using flat jacks.)

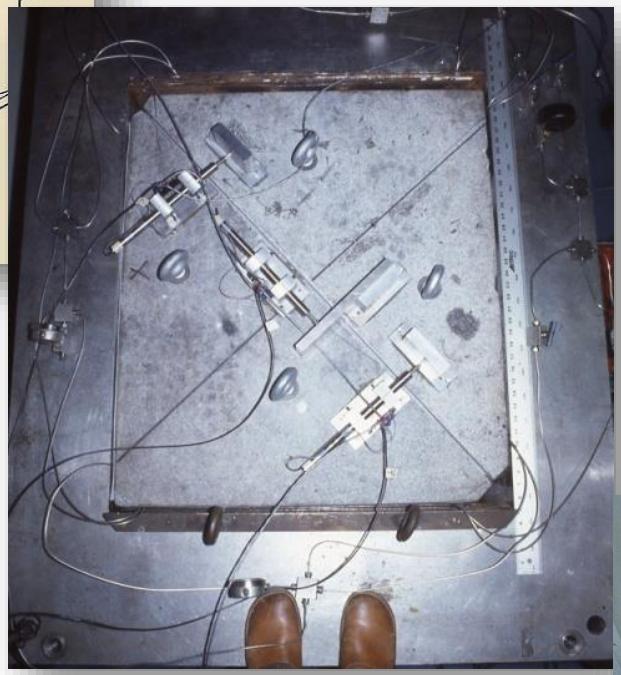




Very large-scale tilt tests (see angle α) with roughness profiles

(Bakhtar and Barton,1983)

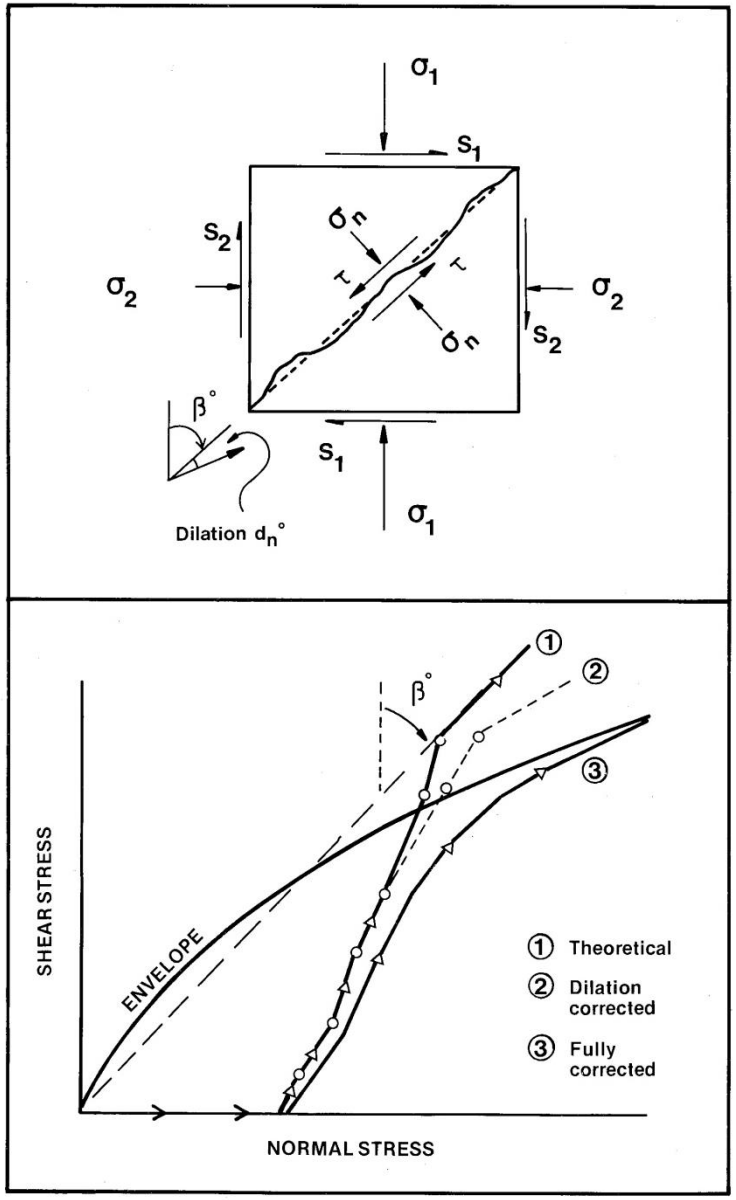




Nearly a *'rock mechanics injury'* – due to incorrect theory. The sample would not shear! Flat-jack burst at 28 MPa.

CORRECTION FOR *OUT-OF-PLANE DILATION* (AND BOUNDARY FRICTION) NEED TO BE MADE

(Bakhtar and Barton, 1984)



THEORETICAL

$$\sigma_n = \frac{1}{2}(\sigma_1 + \sigma_2) - \frac{1}{2}(\sigma_1 - \sigma_2)\cos 2\beta$$

$$\tau = \frac{1}{2}(\sigma_1 - \sigma_2)\sin 2\beta$$

DILATION CORRECTED

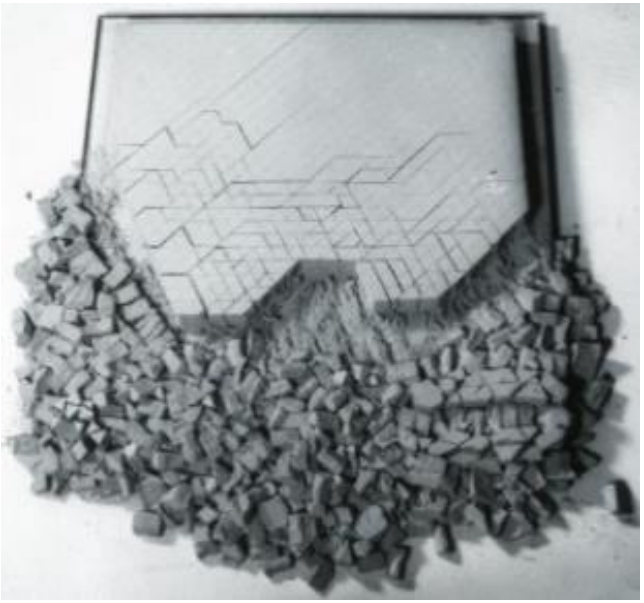
$$\sigma_n = \frac{1}{2}(\sigma_1 + \sigma_2) - \frac{1}{2}(\sigma_1 - \sigma_2)\cos 2[\beta + d_n \text{ mob}]$$

$$\tau = \frac{1}{2}(\sigma_1 - \sigma_2)\sin 2[\beta + d_n \text{ mob}]$$

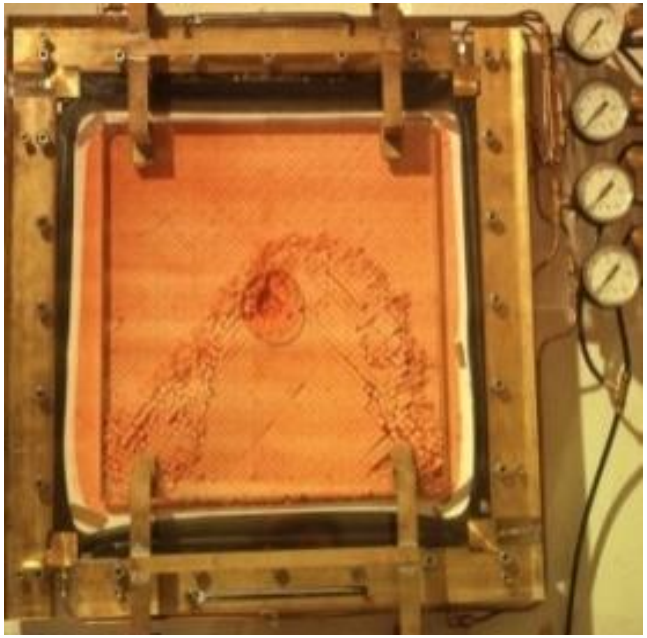
5. ROCK MASSES

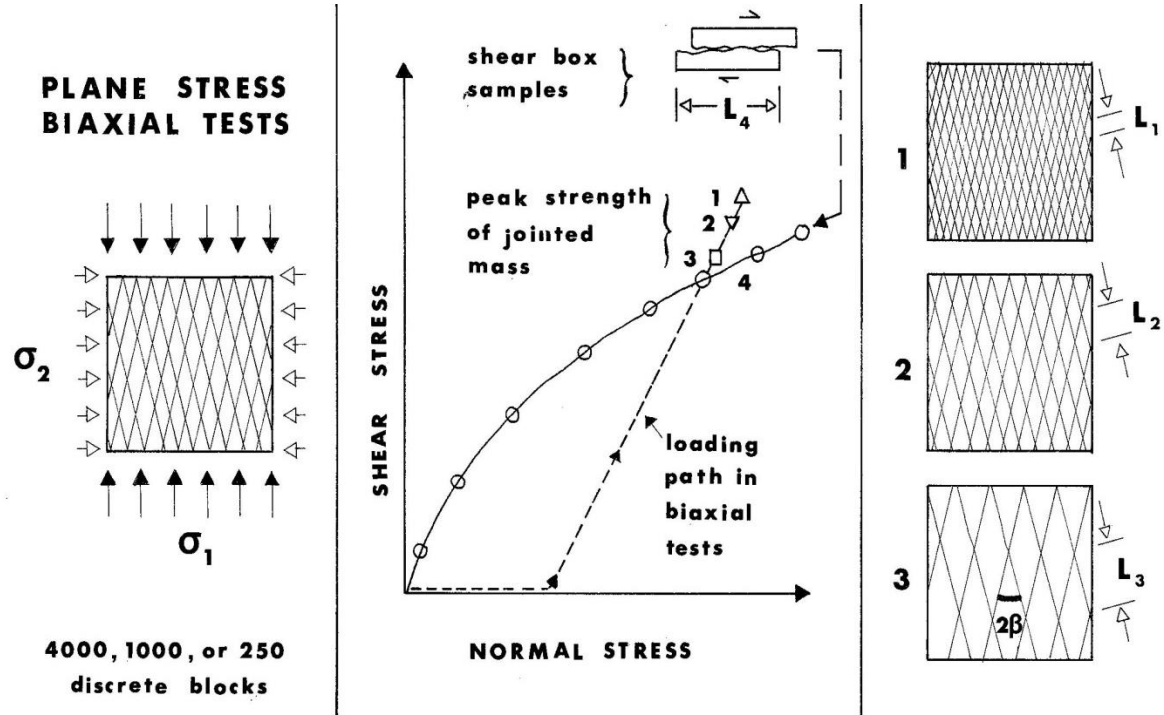
FAILURE IS PROCESS-AND-DISPLACEMENT DEPENDENT

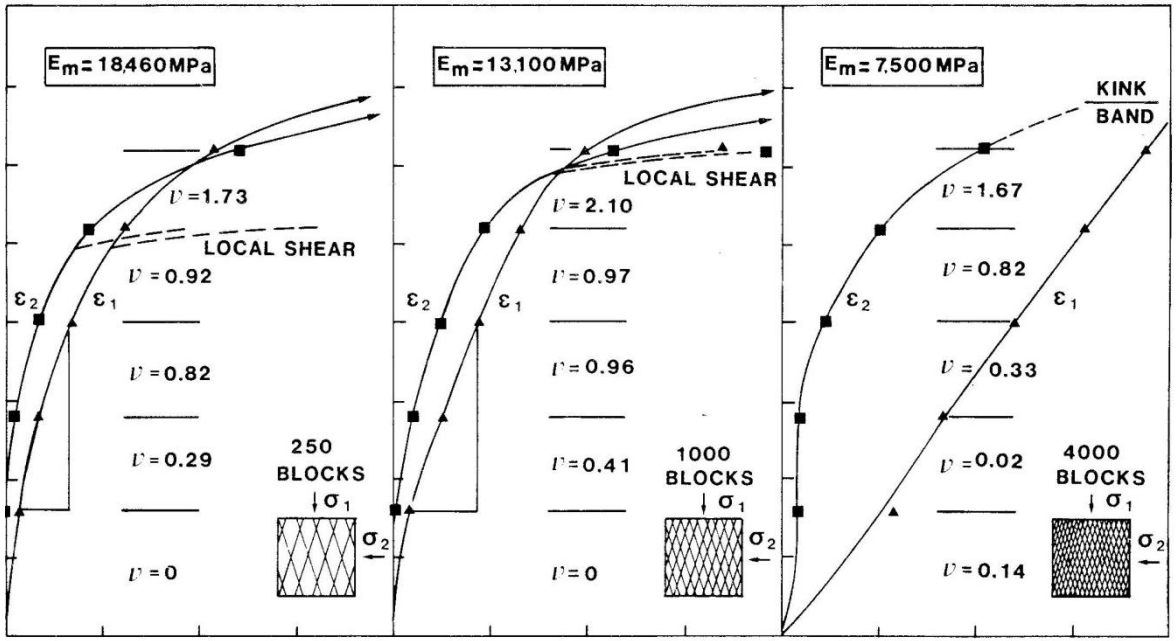
ONE THEREFORE CANNOT ADD ‘c’ and ‘$\sigma_n \tan \varphi$’

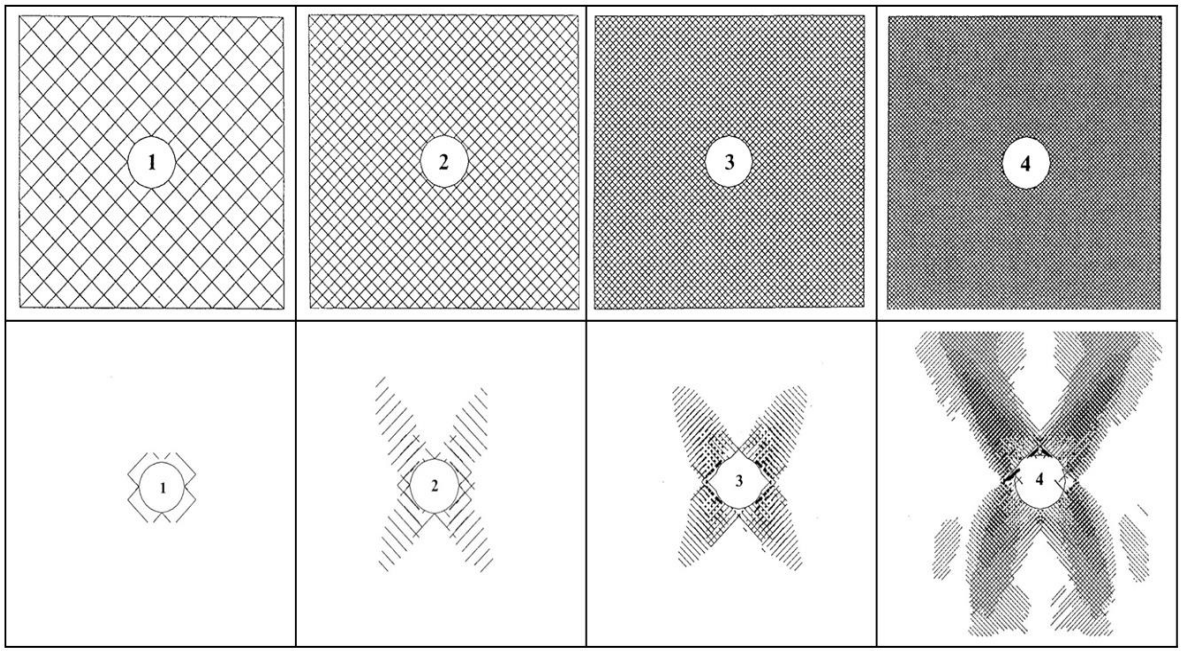


SHEAR STRENGTH OF 2D 'ROCK MASSES' APPEARS TO BE BLOCK-SIZE DEPENDENT. SMALL BLOCKS GIVE HIGHER STRENGTH, EVEN THOUGH (WEAK) FRACTURE ZONES ARE OFTEN ASSOCIATED WITH LOWER STRENGTH: DUE TO LOW Jr, HIGH Ja (i.e. CLAY-FILLINGS)









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

INCREASING 'POISSON RATIO' (>> 0.4999)

BLOCK ROTATIONS

TRANSLATIONAL SHEARING

LINEARIZED STRESS-'STRAIN' IF SMALL BLOCKS (SIMILAR TO BASALT CROSS-COLUMN LOADING)

THE REALITIES OF ROCK MASS SHEAR STRENGTH

PROCESS-AND-STRAIN-DEPENDENT FAILURE

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

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

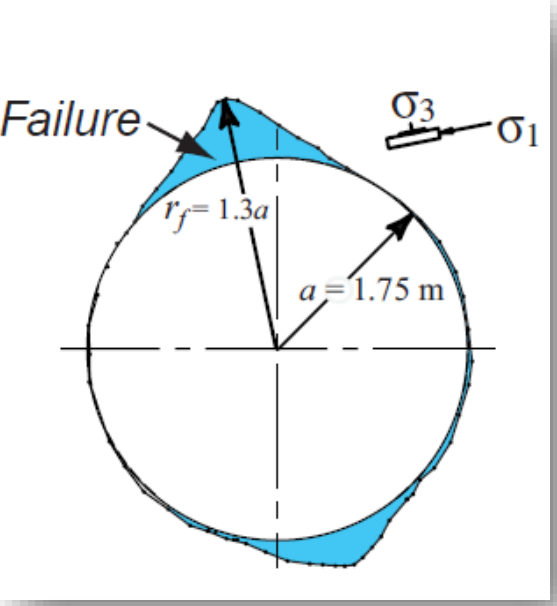
SURROUNDING NATURAL JOINTS with lower JRC, JCS, φ_r, SHEAR NEXT AT LARGER 'STRAIN'

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

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

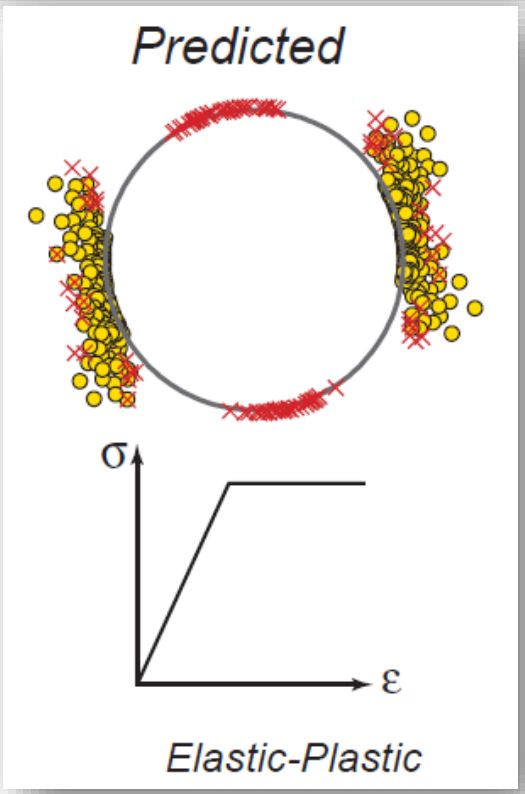
WHAT IF THE SCALE OF THE PROBLEM DEMANDS A CONTINUUM APPROACH?

CHANGE NEEDED



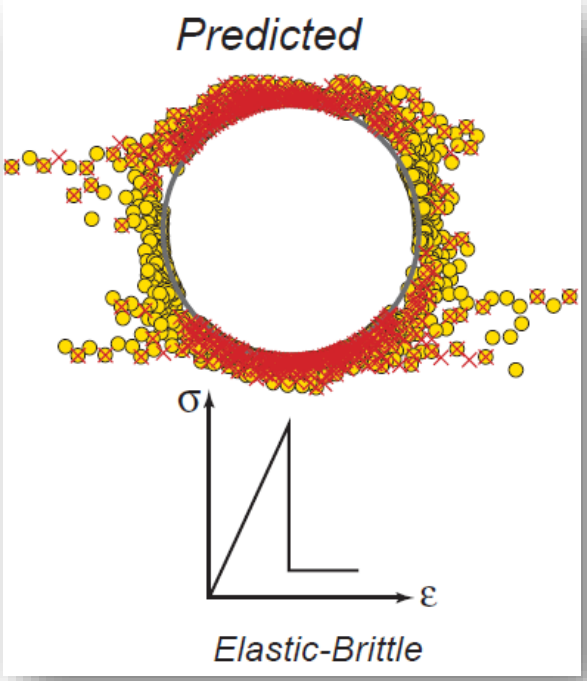
CONVENTIONAL continuum modelling methods.

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

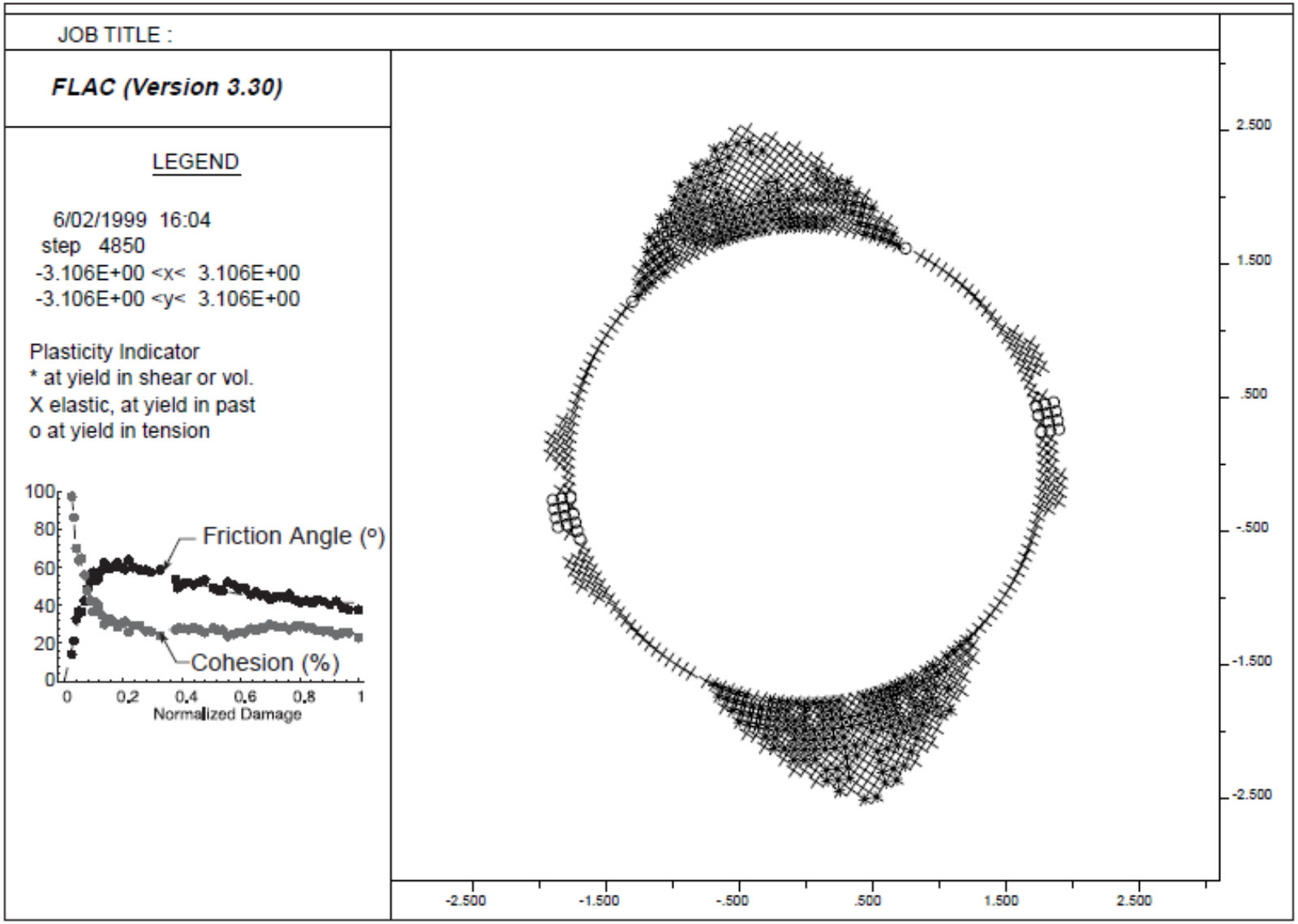


(Hajiabdolmajid, Martin and Kaiser, 2000 "Modelling brittle failure", NARMS)

So why performed by so many consultants?



× Shear failure ○ Tensile failure



Degrade cohesion, mobilize friction: excellent match.

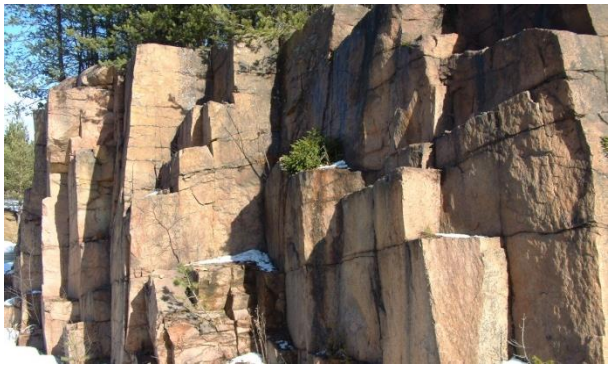
(Hajiabdolmajid, Martin and Kaiser, 2000 "Modelling brittle failure", NARMS.)

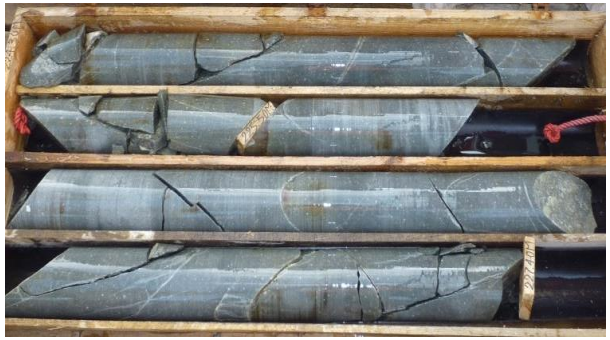
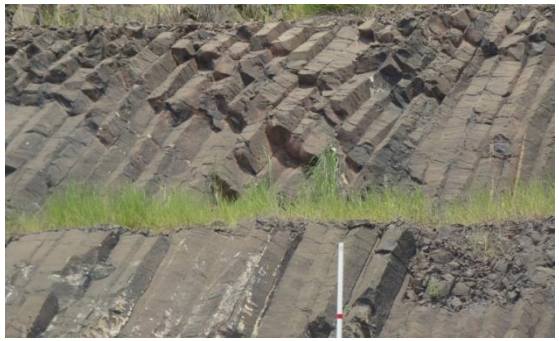
A FRESH APPROACH USING A COMMON SOURCE OF INPUT DATA 'Q'

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

(BUT AN AS YET LITTLE TESTED ESTIMATE OF

'c' AND 'ϕ')



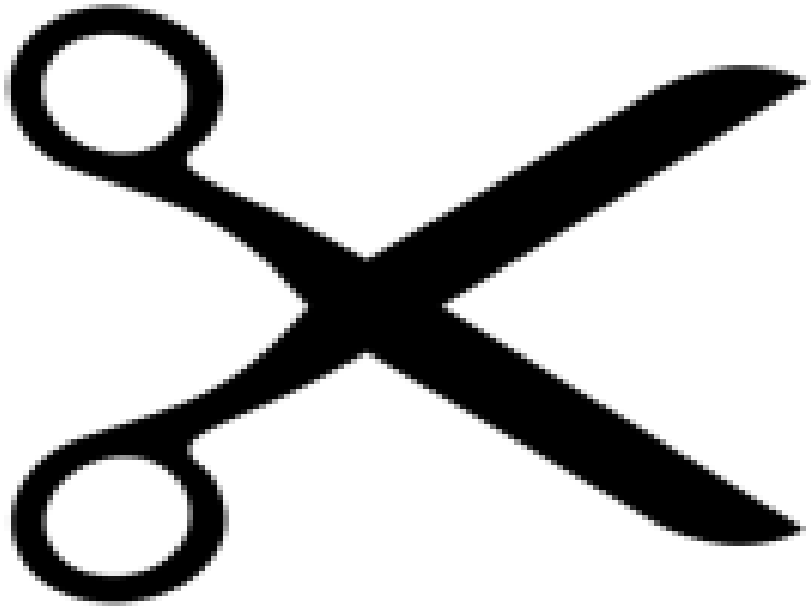



BROAD-REACH CLASSIFICATION NEEDED – LARGE NUMERICAL RANGE

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

0.001→1000, or 5→95, or F7→F1 ???

NEED SCISSORS, TO CUT THE Q_c-formula in TWO PARTS



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'

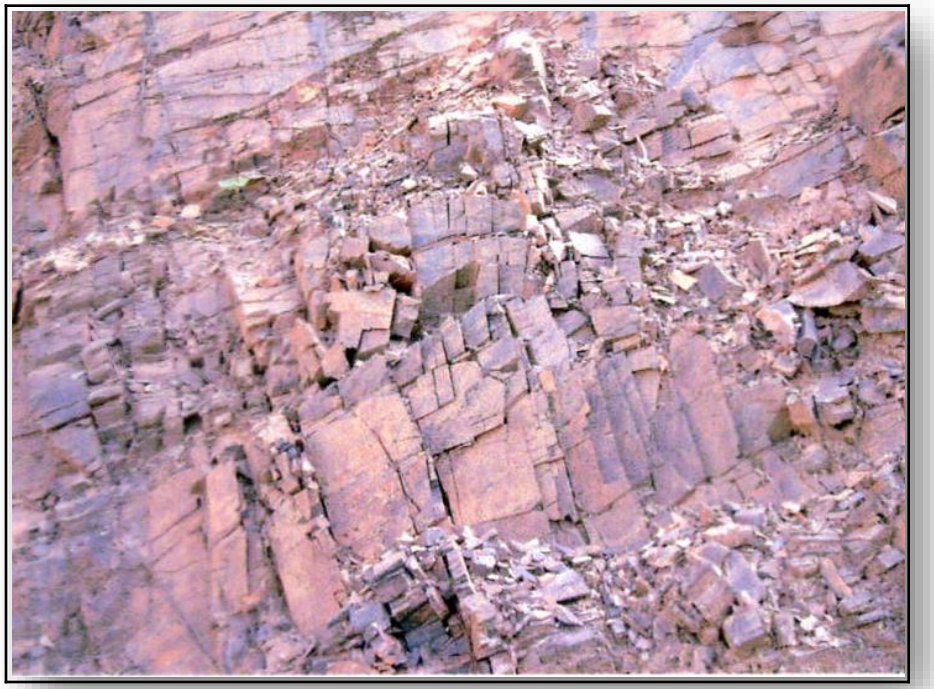
$$CC = \frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100}$$

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

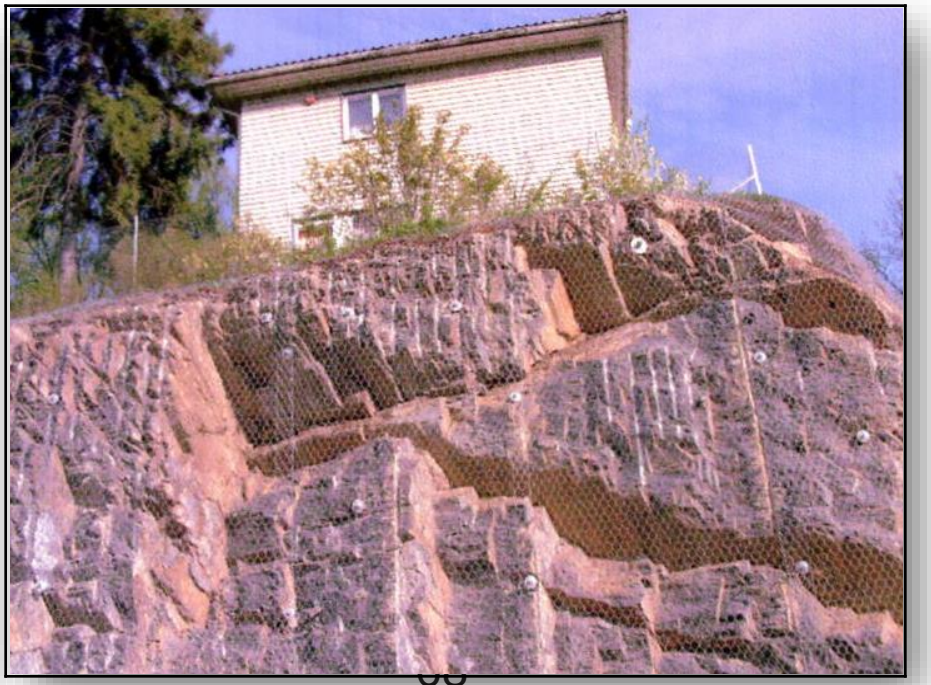
RQD	J_n	J_r	J_a	J_w	SRF	Q	σ_c	Q_c	*FC°*	*CC* MPa	V_p km/s	E_{mass} GPa
100	2	2	1	1	1	100	100	100	63°	50	5.5	46
90	9	1	1	1	1	10	100	10	45°	10	4.5	22
60	12	1.5	2	0.66	1	2.5	50	1.2	26°	2.5	3.6	10.7
30	15	1	4	0.66	2.5	0.13	33	0.04	9°	0.26	2.1	3.5

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

Low CC –shotcrete preferred



Low FC – bolting preferred



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

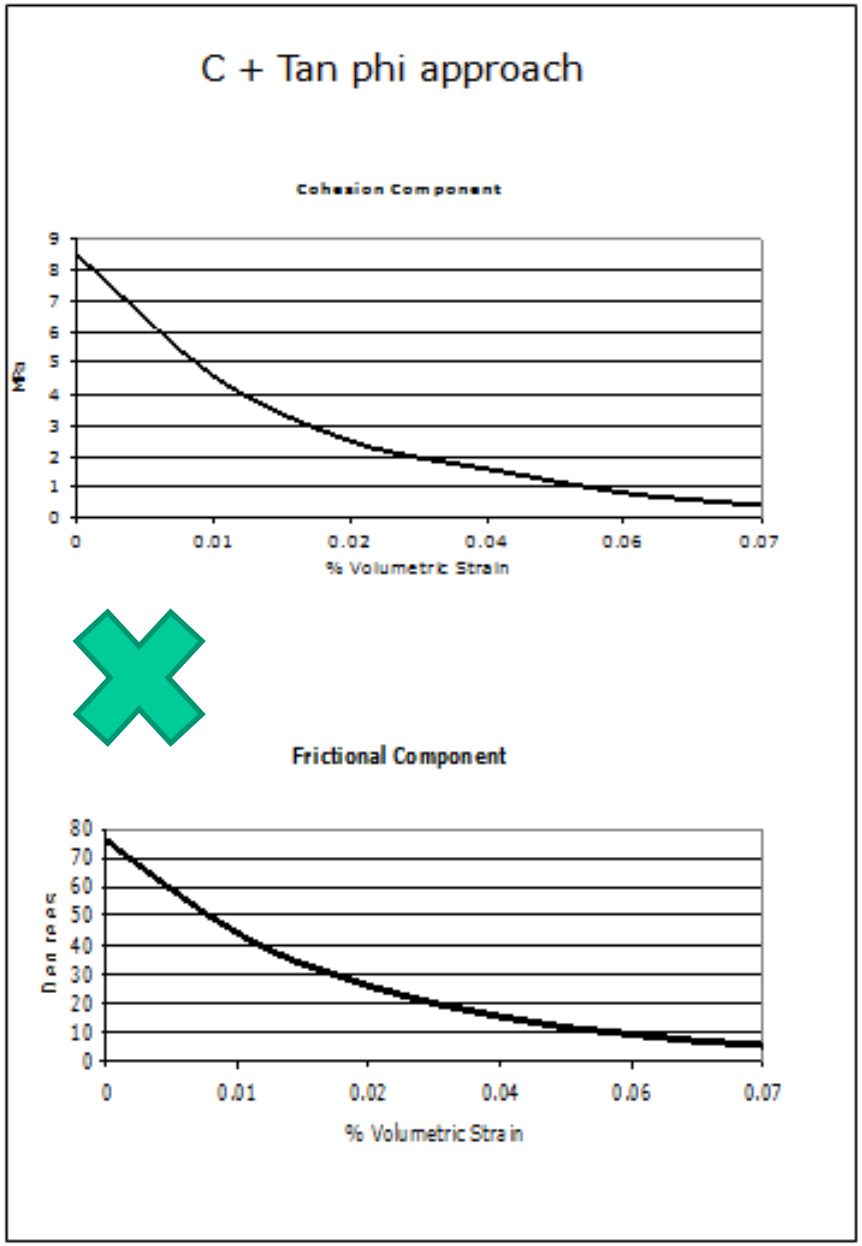
GSI (=RMR-5) based algebra for 'c' and 'φ'

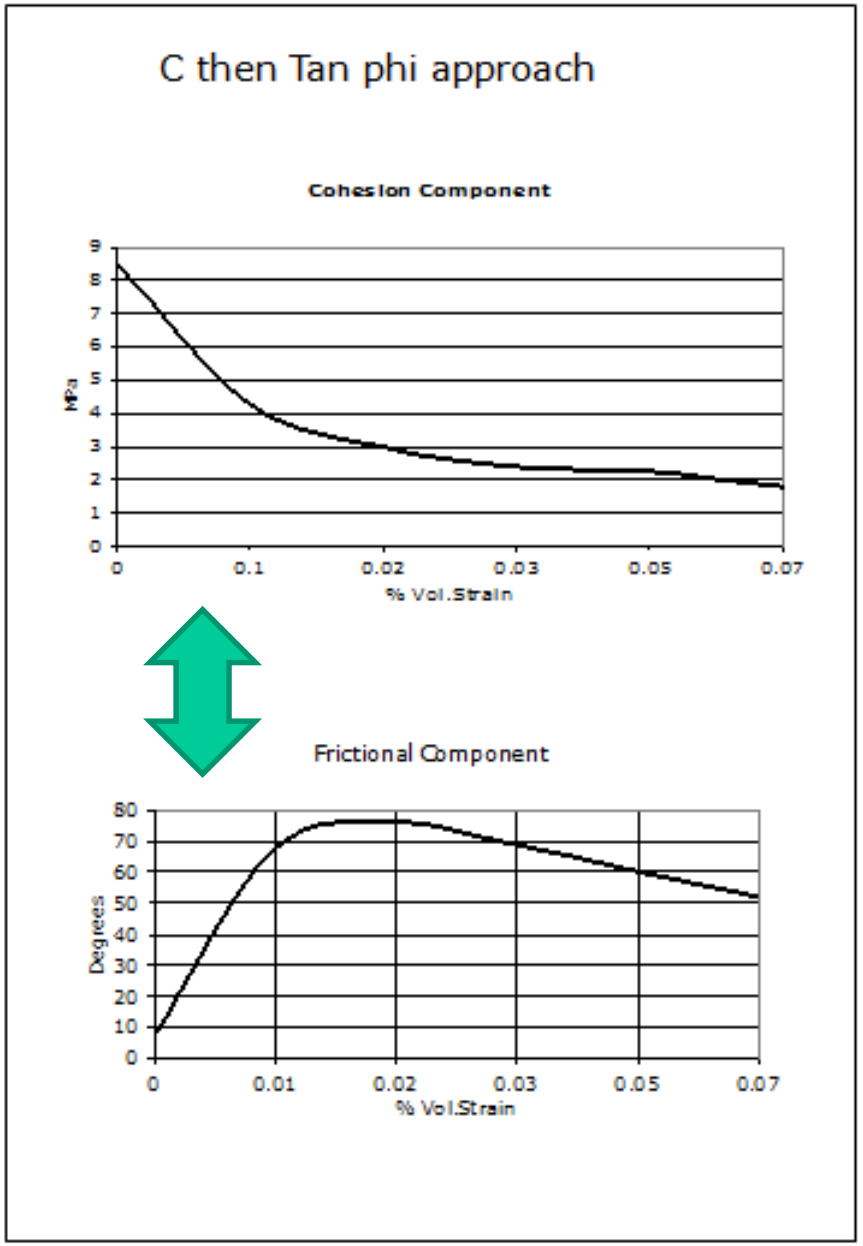
contrasted with

Q-based 'empiricism'

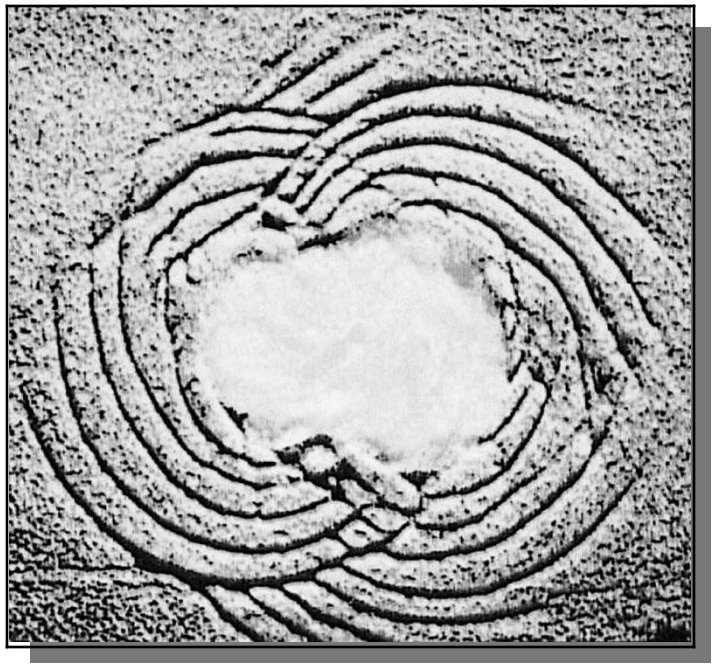
Note: ***shotcrete needed when low CC, bolting needed when low FC.***

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



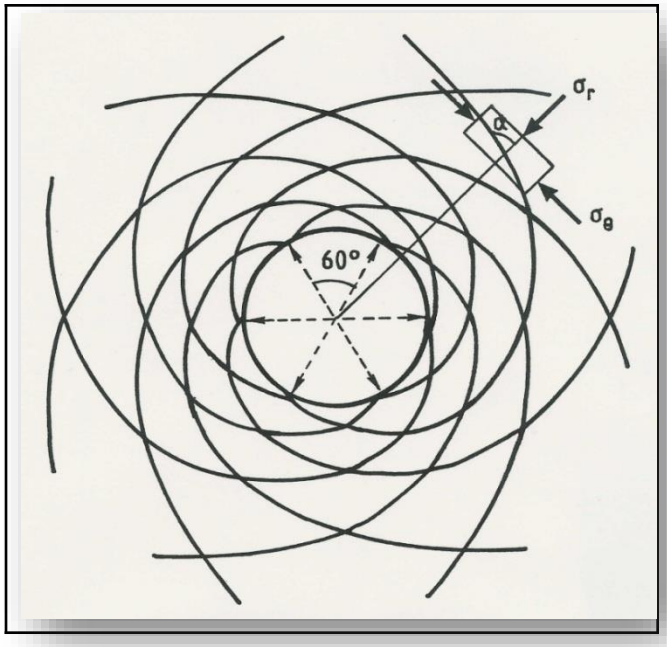


BUT – WHERE IS THE ***CONTINUUM BEHAVIOUR (?)*** WHEN FAILURE IS APPROACHING



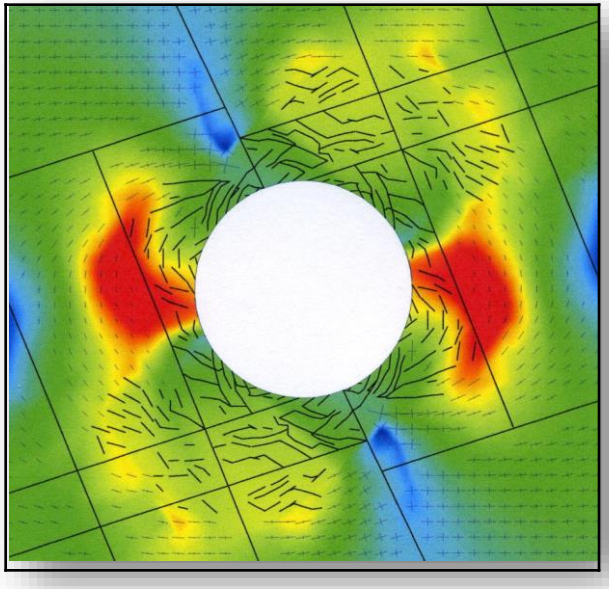
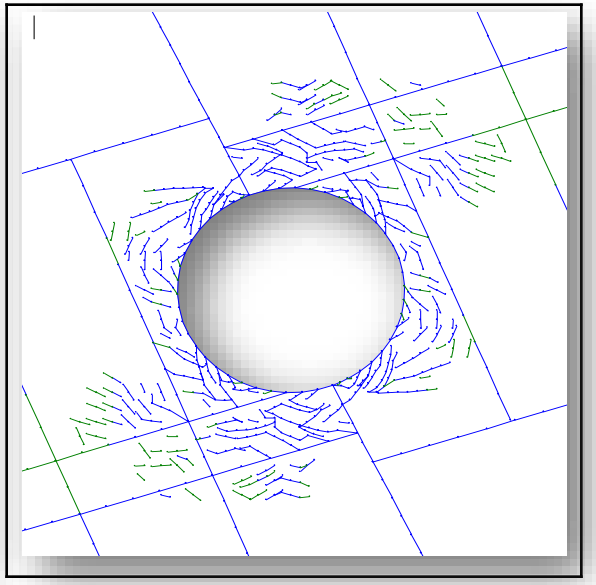
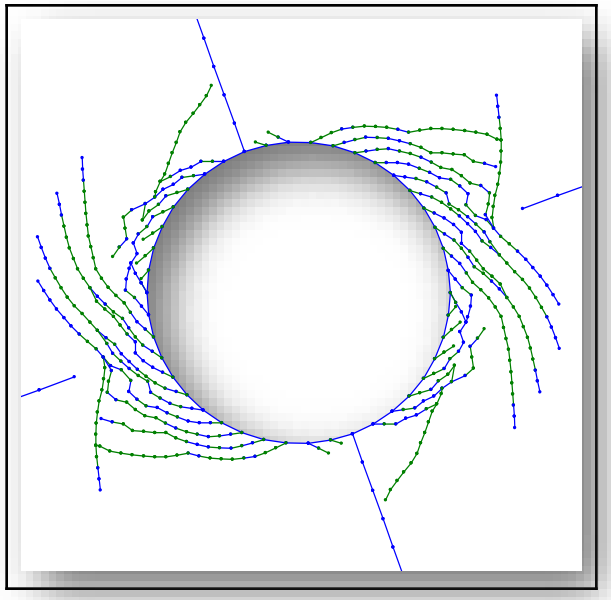
Borehole stability studies at NGI.

(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

FRACOD models by Baotang Shen, 2004.

(Stress-fracture/burst prediction in deep TBM tunnels. NB&A 2005 report). WHERE IS THE CONTINUUM WHEN FAILURE APPROACHES ?

CONCLUSIONS

1. NON-LINEAR SHEAR STRENGTH, NO COHESION FOR ROCK JOINTS

2. MULTI-STAGE TESTING 'GENERATES' ARTIFICIAL 'COHESION'

3. ARE WE TRANSFORMING STRESSES CORRECTLY IN OUR DILATANT GEO-MATERIALS (OC-clay, dense sand, rock joints, dense rockfill, etc?)

4. STRENGTH CRITERIA SHOULD BE DERIVED FROM ORIGINAL SOURCES, NOT FROM 'DOWN-LOADED' ROCK MECHANICS

5. CANNOT ADD 'c' AND 'tan φ' – THEY ARE NOT MOBILIZED TOGETHER (APPLIES TO MUCH OF 'POPULAR' MODELLING IN ROCK MECHANICS

6. INVITATION TO STRONG SWIMMERS: 'SWIM UPSTREAM' AGAIST THE CURRENT

STABLE BECAUSE OF SHEAR STRENGTH – MANY PARAMETERS INVOLVED – A QUESTION OF TASTE FOR WHICH ONES PREFERRED – COULD BE CC and FC FROM Q ? (BUT CC then FC).