Geotechnical Challenges in Mining, Tunnelling and Underground Structures (ICGCMTU2021) Malaysia. Invited keynote lecture.

Continuum or Discontinuum

GSI or JRC ??

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(WITH THIS PICTURE ON 1ST SCREEN, NO NEED FOR MICROSOFT, OR ME, TO DISTURB THE LECTURE AND COVER EQUATIONS SO YOU CANNOT READ THEM!)

THE DILEMMA NEEDING SERIOUS DISCUSSION

Should we spend the necessary *longer time* performing discontinuum models with discrete joint sets and non-linear properties? *And get more realistic designs for our slopes, mines and tunnels?*

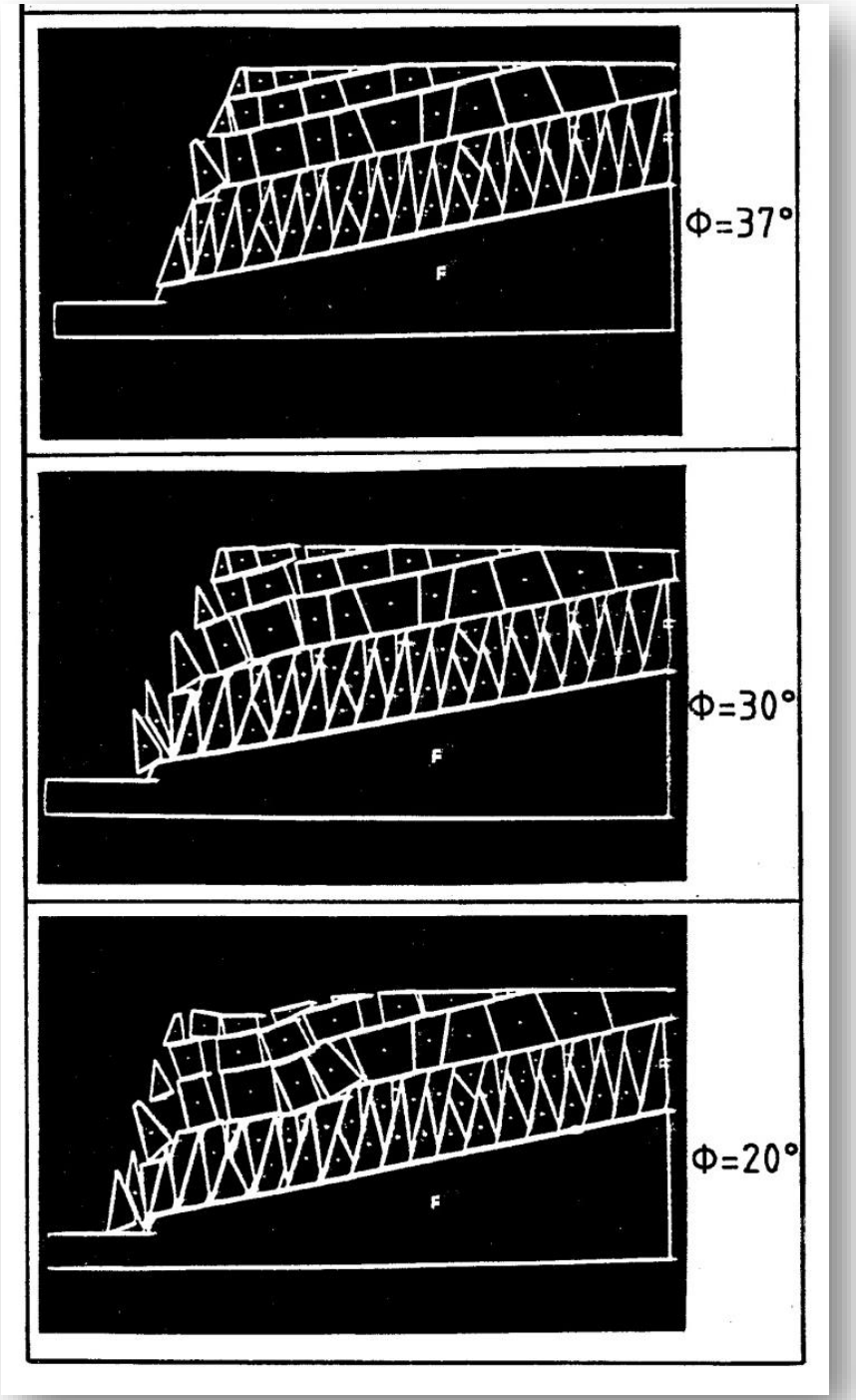
Or can we relax with GSI and let Rocscience software solve the many Hoek-Brown equations, happy in the knowledge that the nice colour plots of 'plastic zones' will impress our supervisors? And perhaps our clients are 'continuum' people also.

The consequences of our (career) choice is more important than most people realize.

WAS ‘ROCK MECHANICS’ for ROCK ENGINEERING

supposed to be so easy (with GSI)?

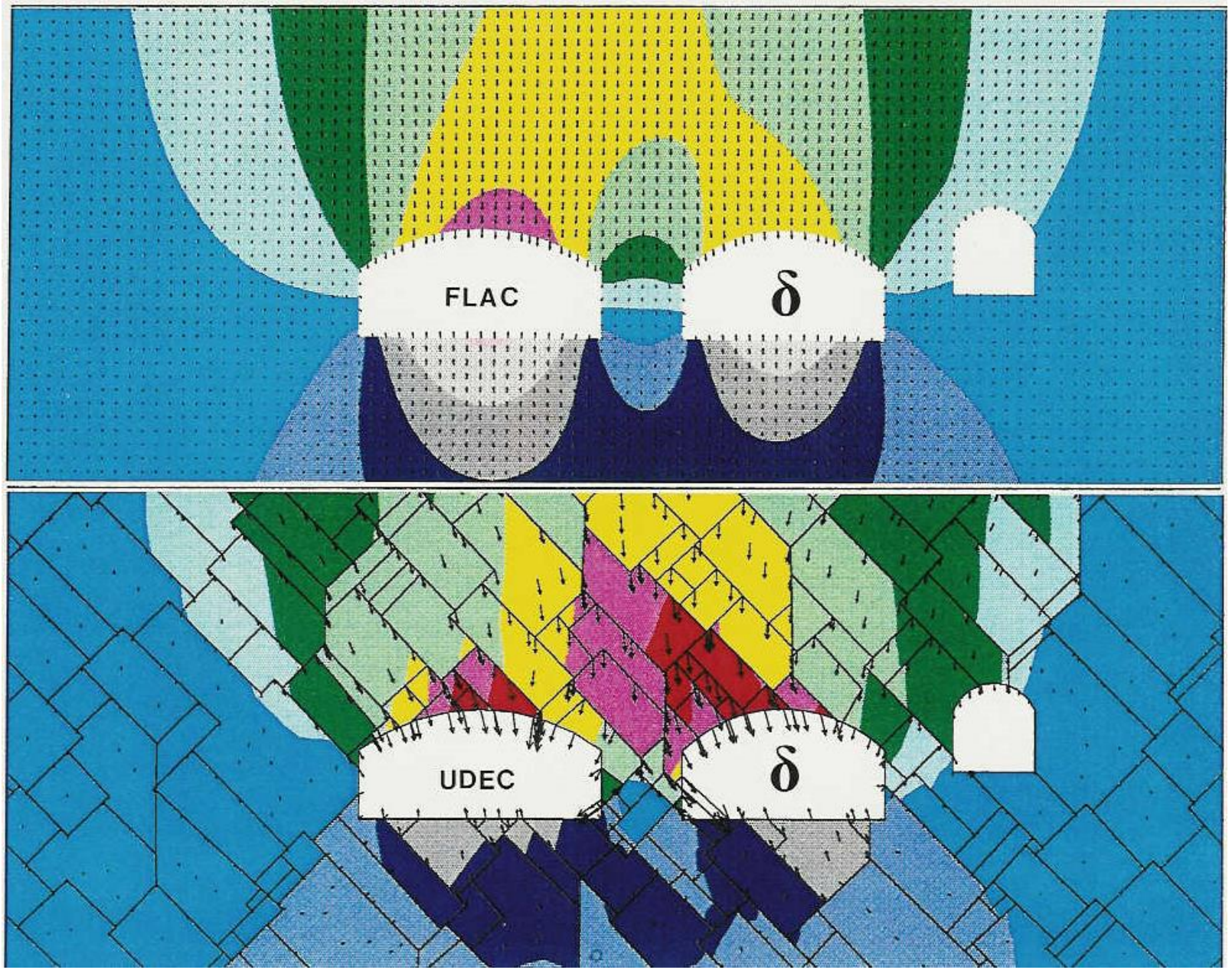
(AND WAS *CONTINUUM BEHAVIOUR* SUPPOSED TO BE A MODEL FOR JOINTED/FAULTED ROCK?)



μDEC with rigid blocks 1975
(φ=40º no block falls)
(*Cundall, Voegele and Fairhurst, 1977)*

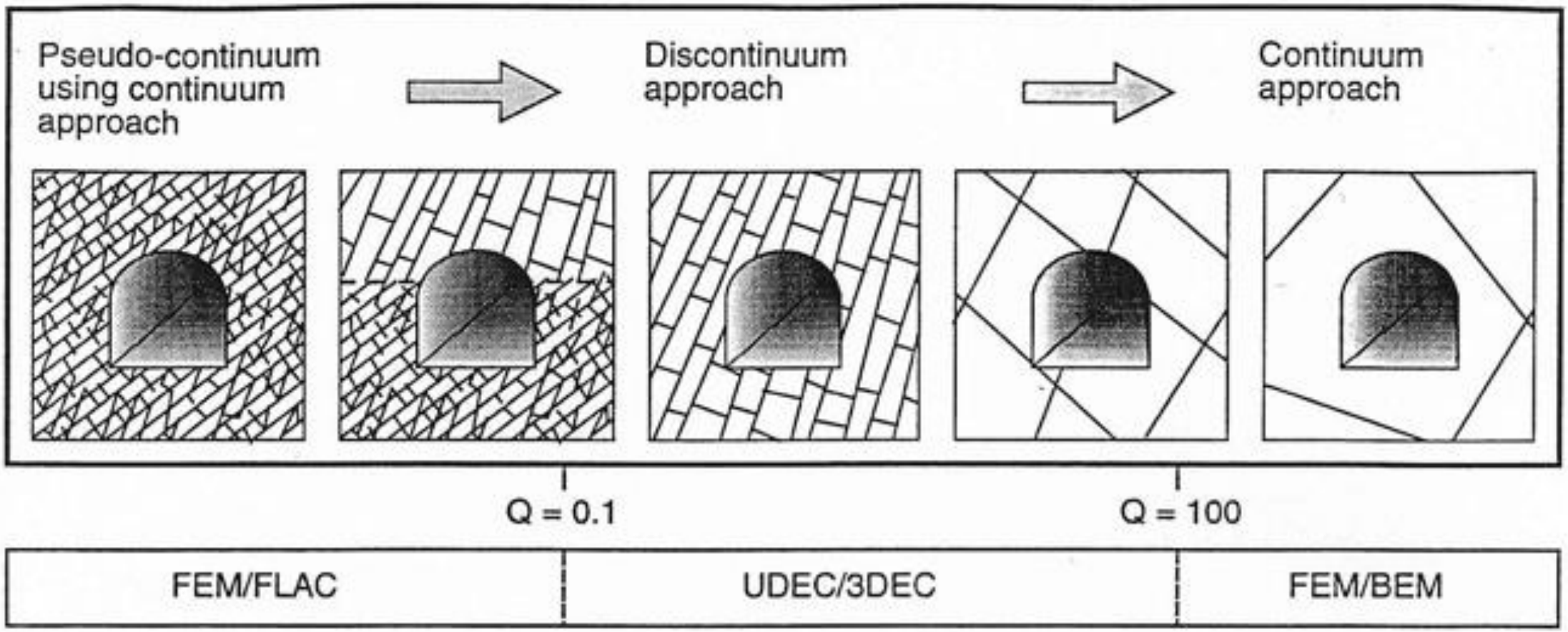
Soon to be followed (in 1980) by UDEC, then UDEC-BB (in 1985)

(see next example and contrast to continuum model)



We are surely learning more when including the jointing?

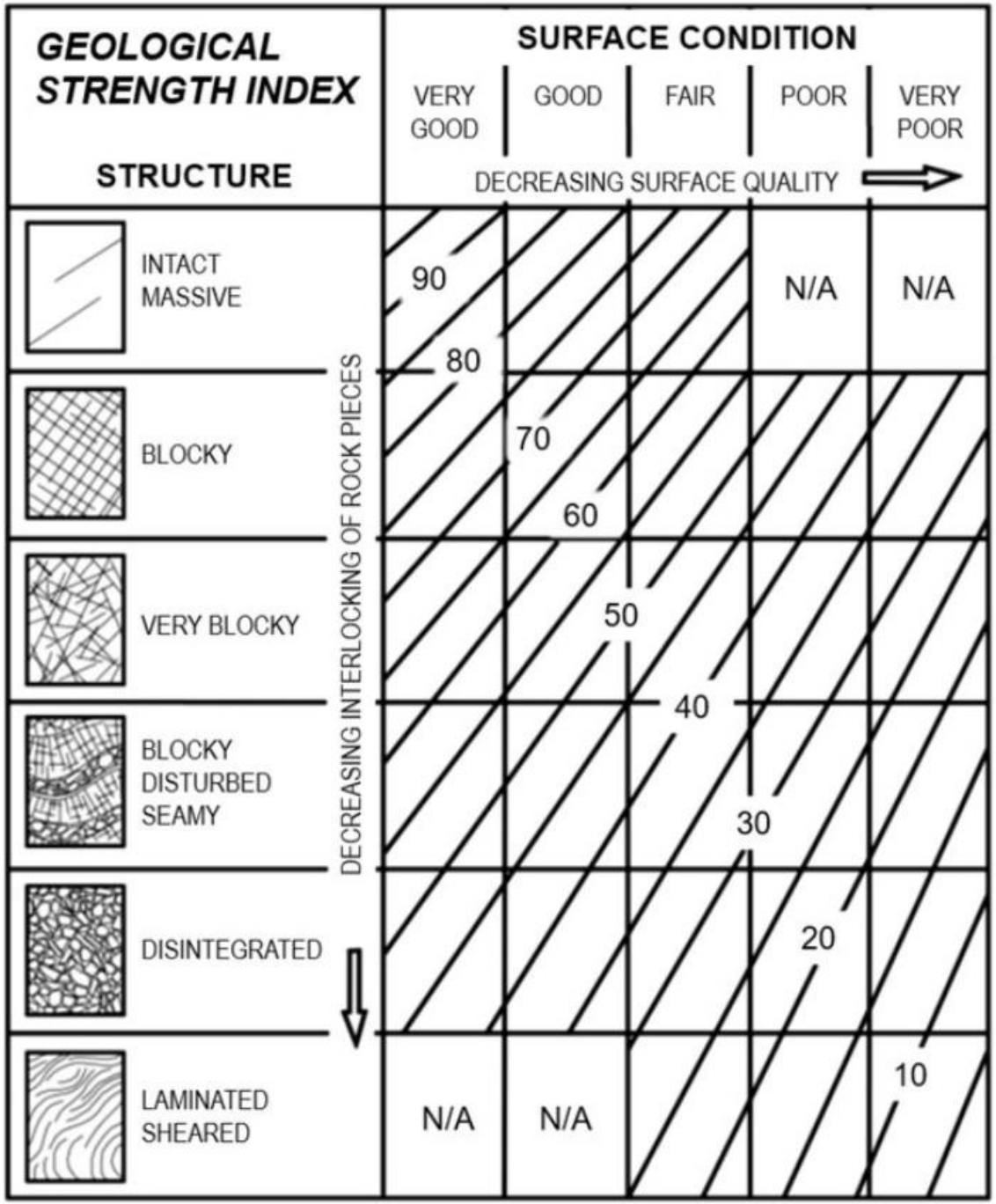
Affirmative!



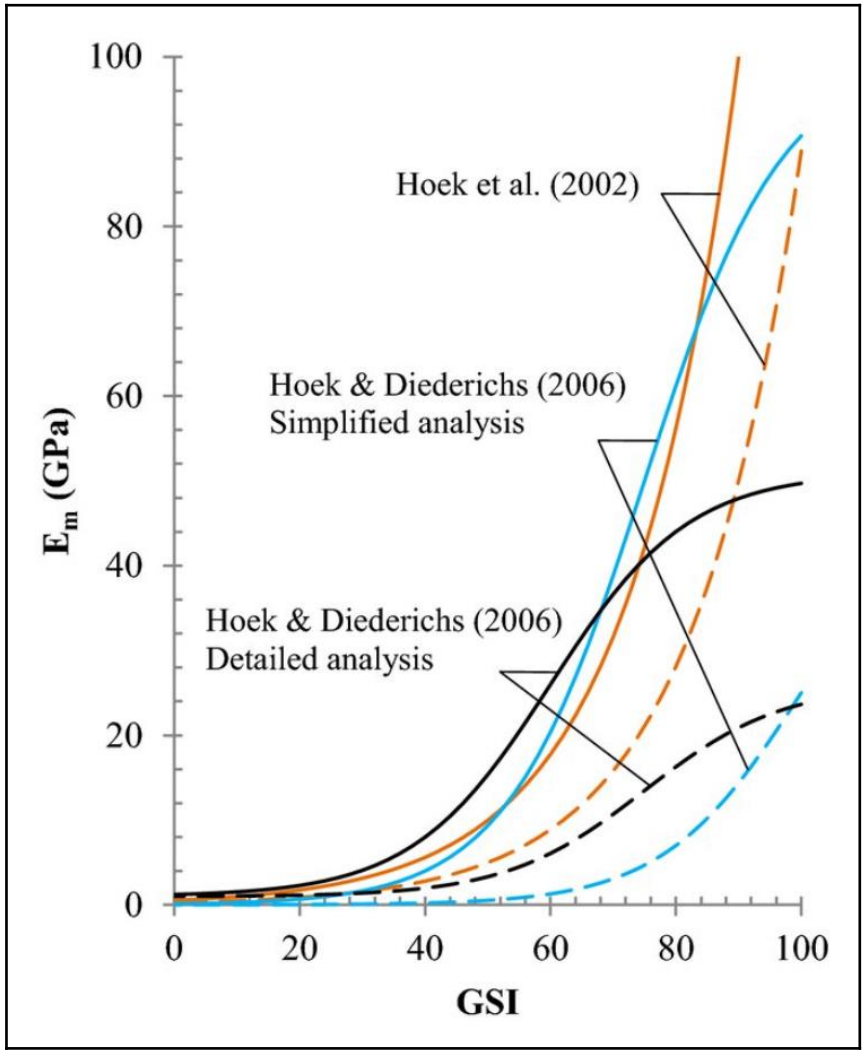
- **Heavily jointed weak rock = continuum (FEM)**
  Jointed/faulted rock = discontinuum (UDEC/3DEC) ...see examples
  Massive rock that may fracture = FRACOD...see examples

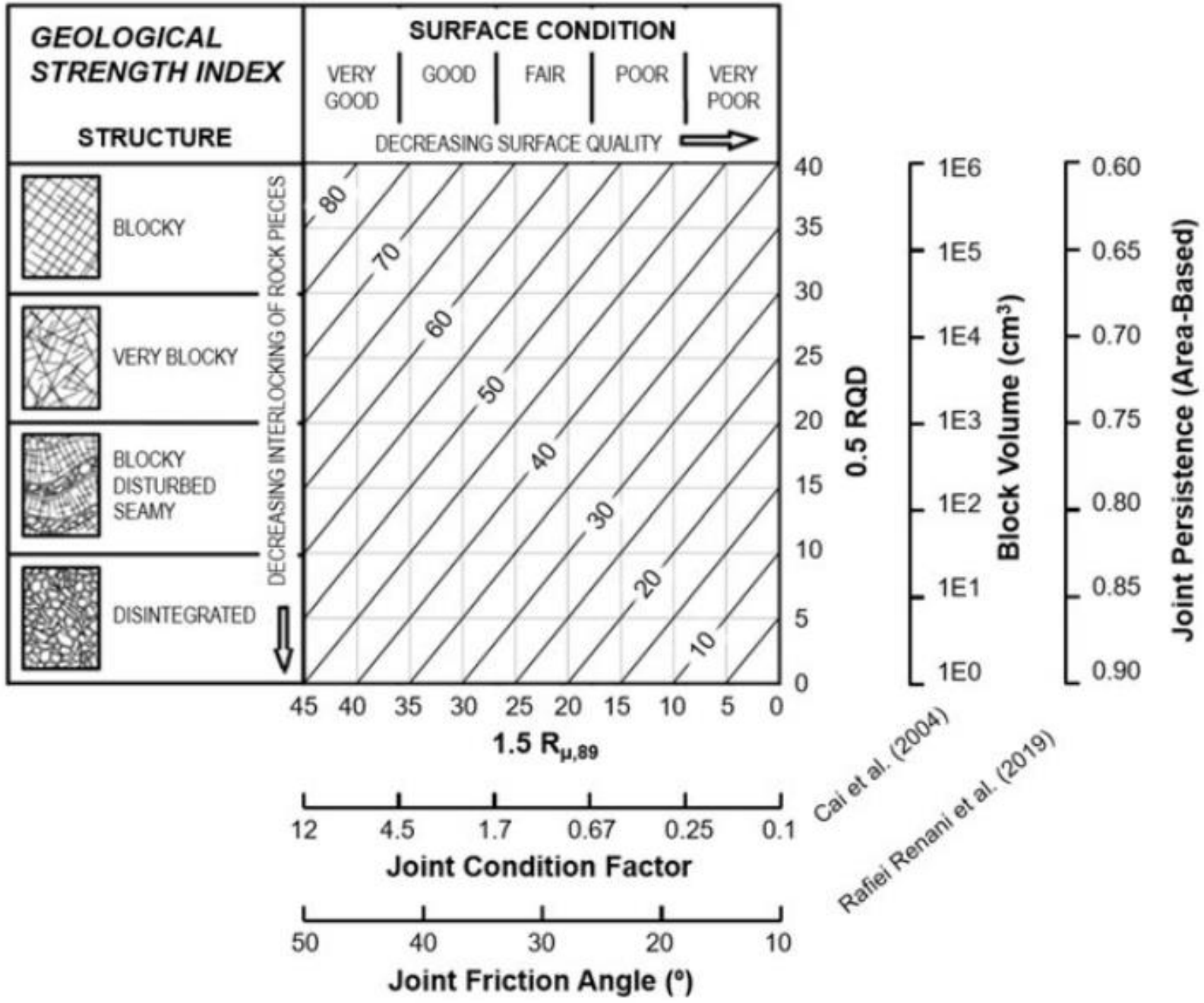
A BRIEF PRELIMINARY LOOK AT GSI

HOEK-BROWN et al. EQUATIONS
WILL BE REVIEWED LATER



The GSI chart stripped of its author's instructions. The hopeless deformation modulus advice.





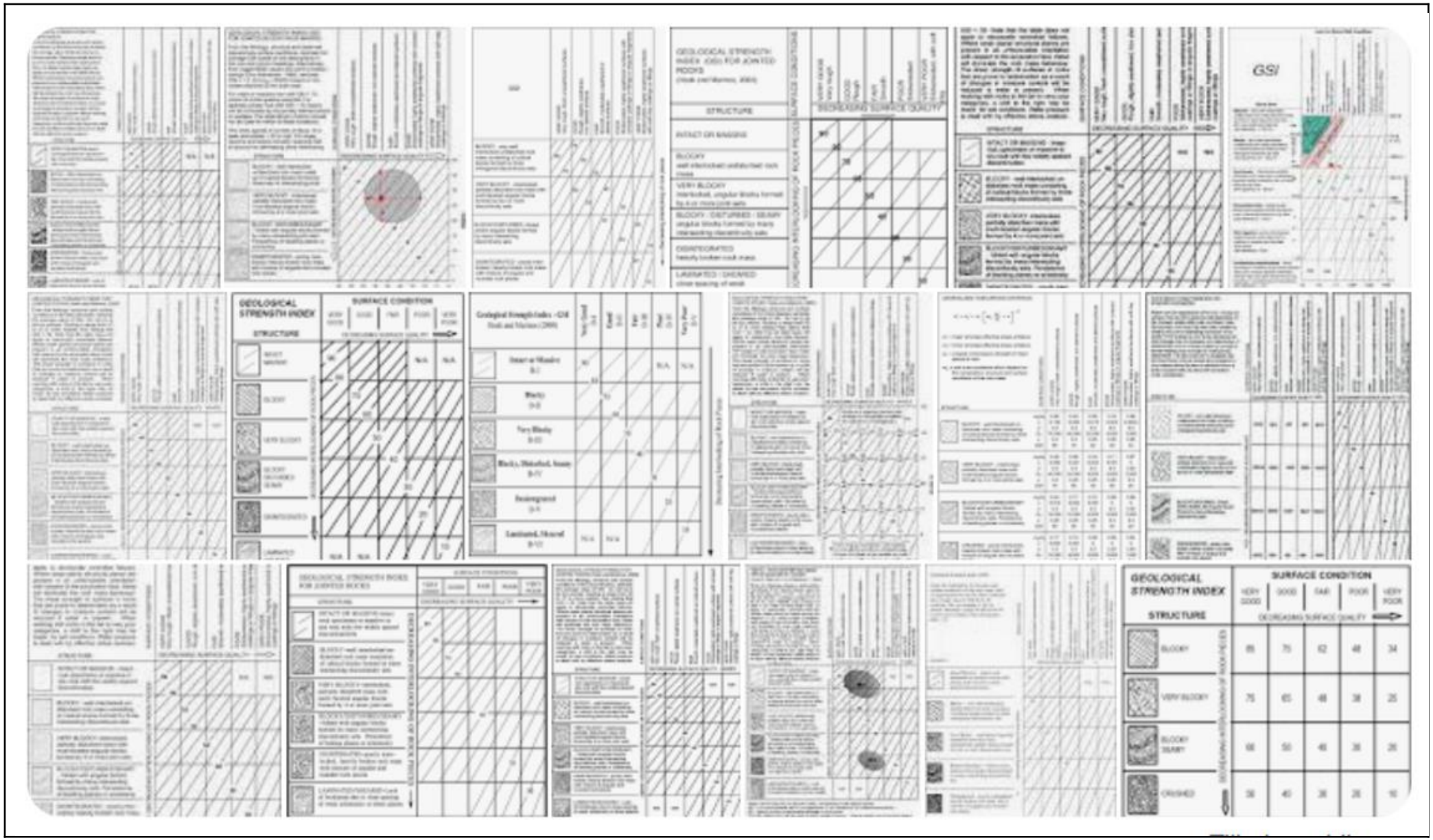
Some creative additions for 'better' quantifying GSI.

[Forty-Year Review of the Hoek–Brown Failure Criterion for Jointed Rock Masses.](#)
Renani and Cai, 2021

See also:

Ván and Vásárhelyi, 2014
for sensitivity analyses of GSI

THE GSI OPTIONS – an INCOMPLETE METHOD?



- **YOUR LECTURER REJECTS CONTINUUM MODELLING!**

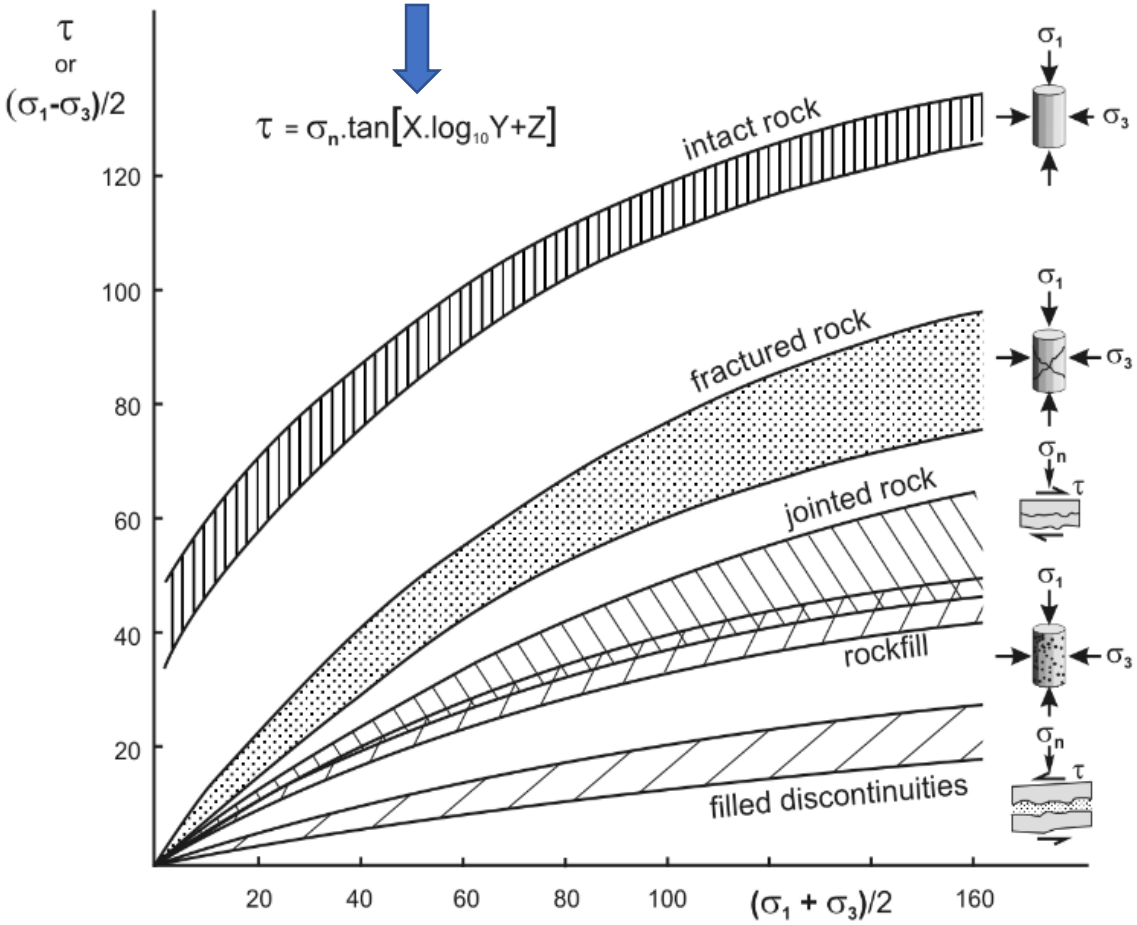
- **THE GSI HOEK-BROWN RS2 (FEM) METHOD IS USUALLY GIVING FALSE (CONTINUUM) RESULTS.**

- **ROCK *MASSES* ARE MORE INTERESTING (and more complex) THAN THIS!**

- **REAL BEHAVIOUR WILL CONSTANTLY BE A SURPRISE IN RELATION TO CONTINUUM PREDICTIONS (UNLESS THE CONSULTANT CAN GO BACK AND ALTER HIS INPUT DATA....GSI, D etc.)**

WHAT IS HELPING TO PREVENT SUDDEN COLLAPSE?

Progressive failure of components: **CcSs: crack, crunch, scrape, swoosh**

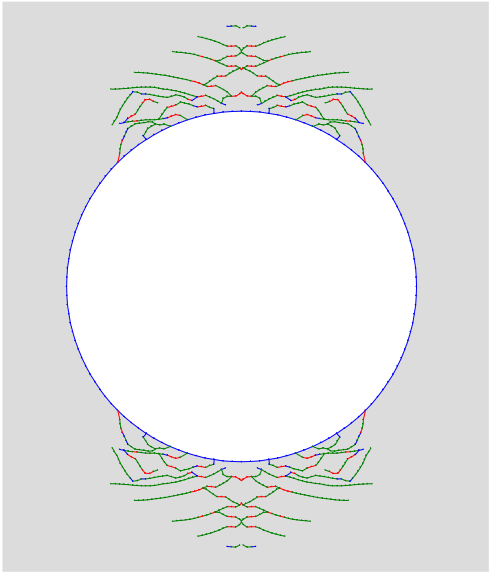


Bingham Pit: No casualties. Monitored. Progressive failure....i.e. ‘τ = c *then* σn tan ϕ’

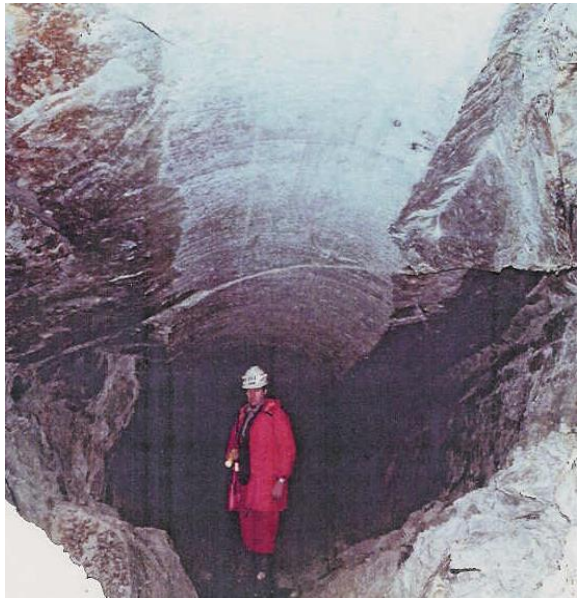
SLIDING ON A BASAL FAULT PLANE WITH WEDGE EFFECT SEEN TO THE LEFT. NO ‘SPOON-SHAPED’ FAILURE AS WITH CONTINUUM ANALYSES.

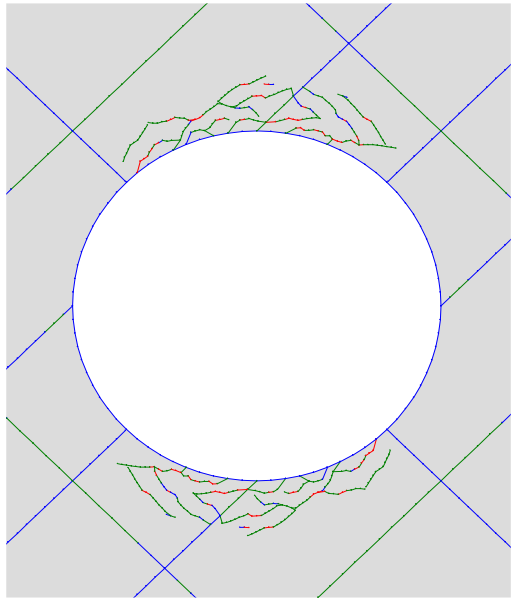
WE SEE DISCONTINUOUS BEHAVIOUR EVEN WHEN MASSIVE ROCK FRACTURES UNDER STRESS.

A ‘NEW’ CRITERION OF FAILURE IS CAUSED BY STRAIN NOT ***STRESS/STRENGTH***

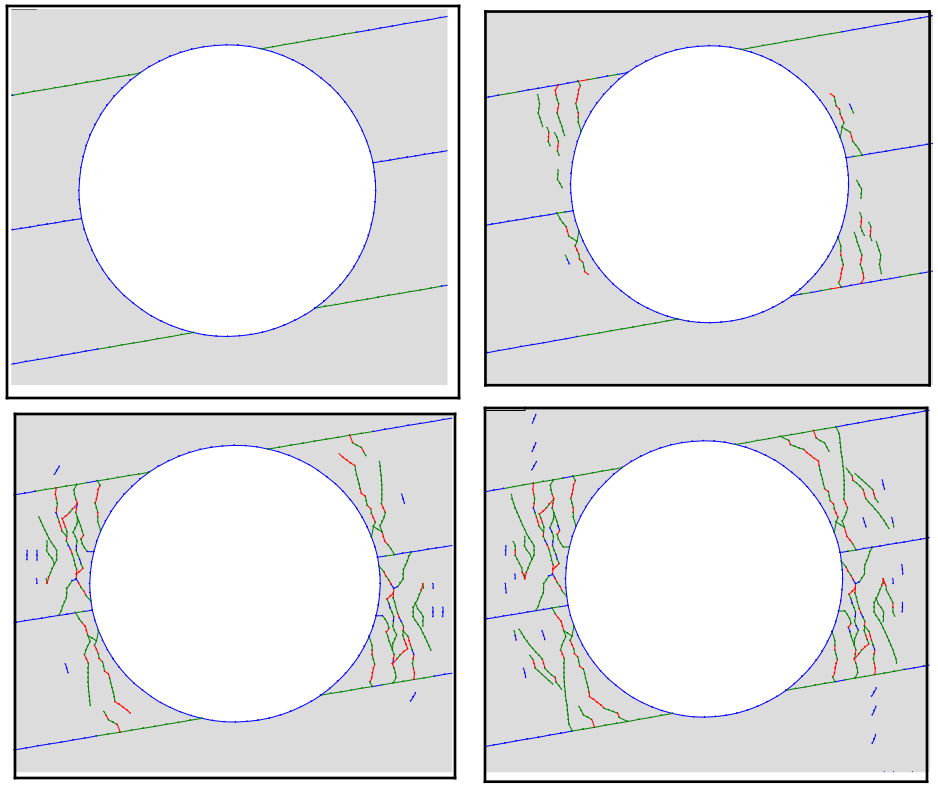


A selection of tunnel failure modes when higher stress:





- ❑ **Physical models**
- ❖ **TBM tunnel tragedy**
- ➢ **Numerical models (FRACOD B.Shen)**

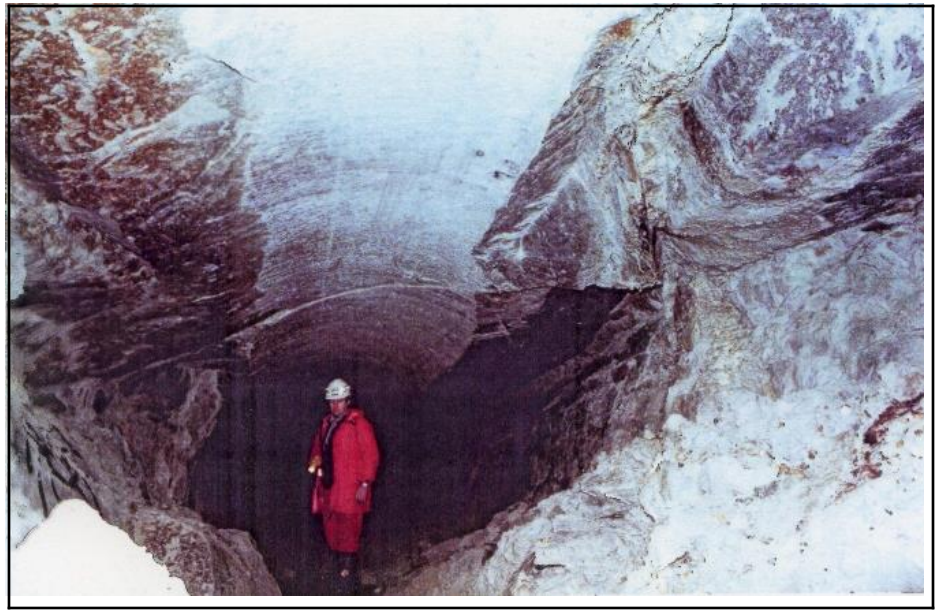


CRITICAL EXTENSION STRAIN:

Marks start of spalling which is cracking in tension. May get propagation in shear)

(Baotang Shen, in Barton and Shen, 2017)

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

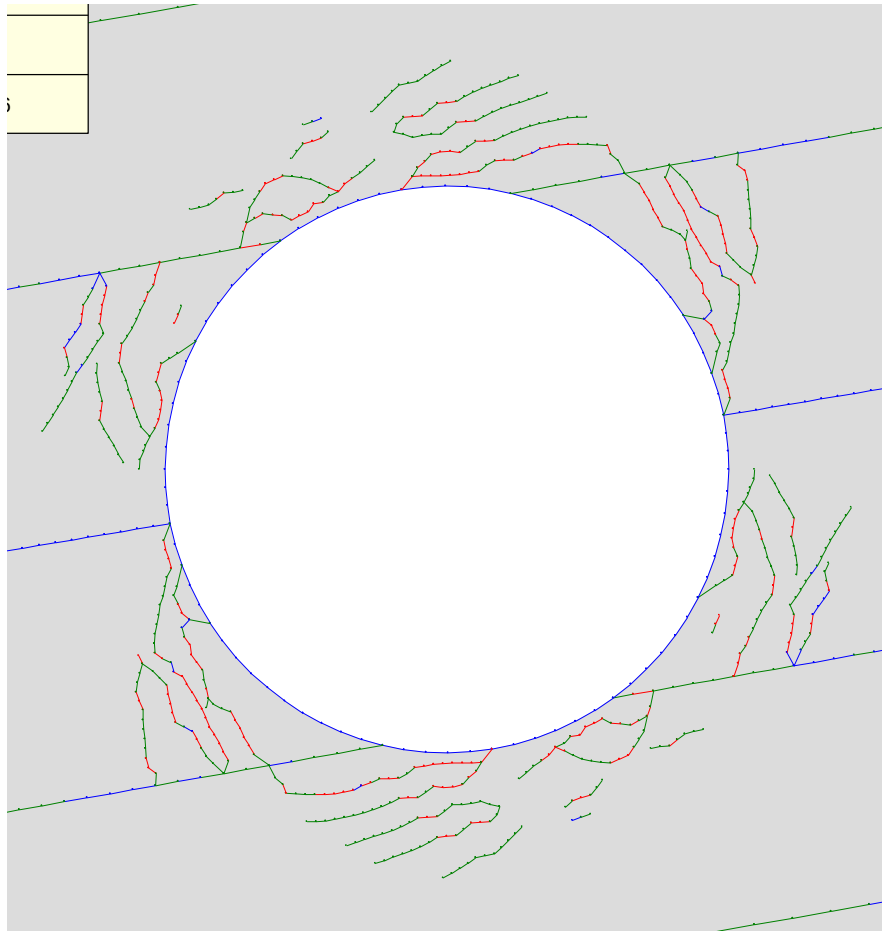


Example of FRACOD modelling of 1880 (Beamont/English) TBM in chalk marl). Here: assume

$\sigma_h = 1/3\ \sigma_v$ (due to nearby cliff)

WHAT IF THE STRESS HAD BEEN MAXIMUM IN THE HORIZONTAL DIRECTION? WHAT TYPE OF FAILURE?

THESE TWO FRACOD MODELS (by Dr. Baotang Shen) 'PROVE' THAT IT WAS THE HIGH VERTICAL STRESS WHICH CAUSED THE FAILURE.



Chalk-marl

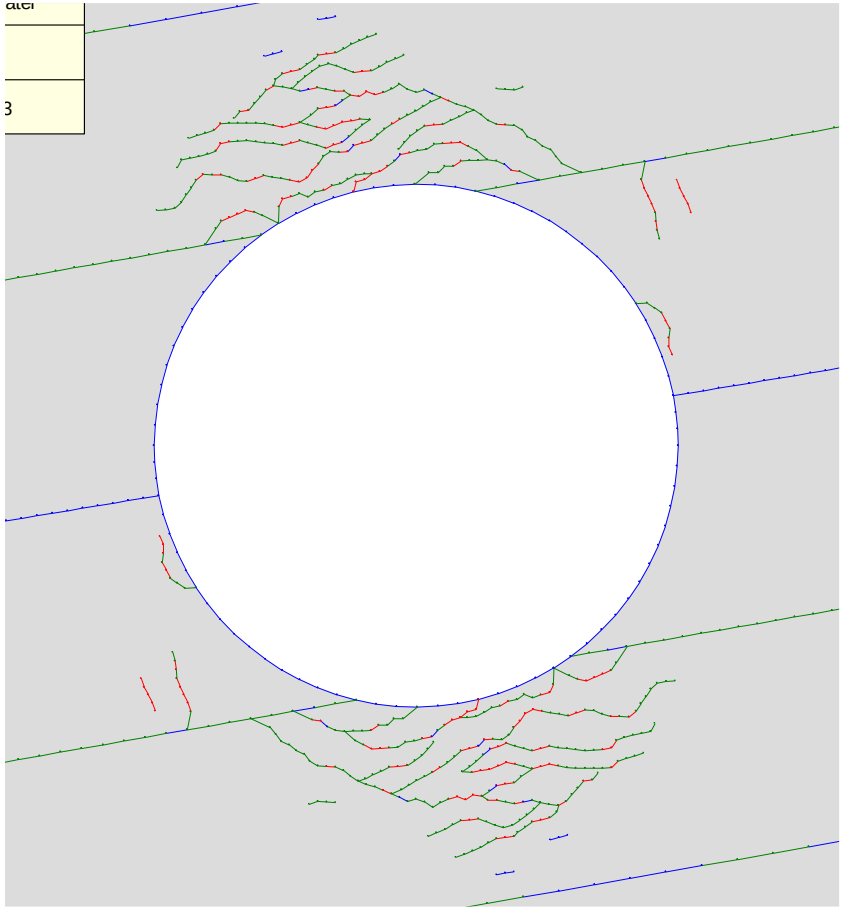
UCS = 6 MPa,

Left: $\sigma_h/\sigma_v = 1.0$

Right: $\sigma_h/\sigma_v = 2.0$

(tension fractures in red)

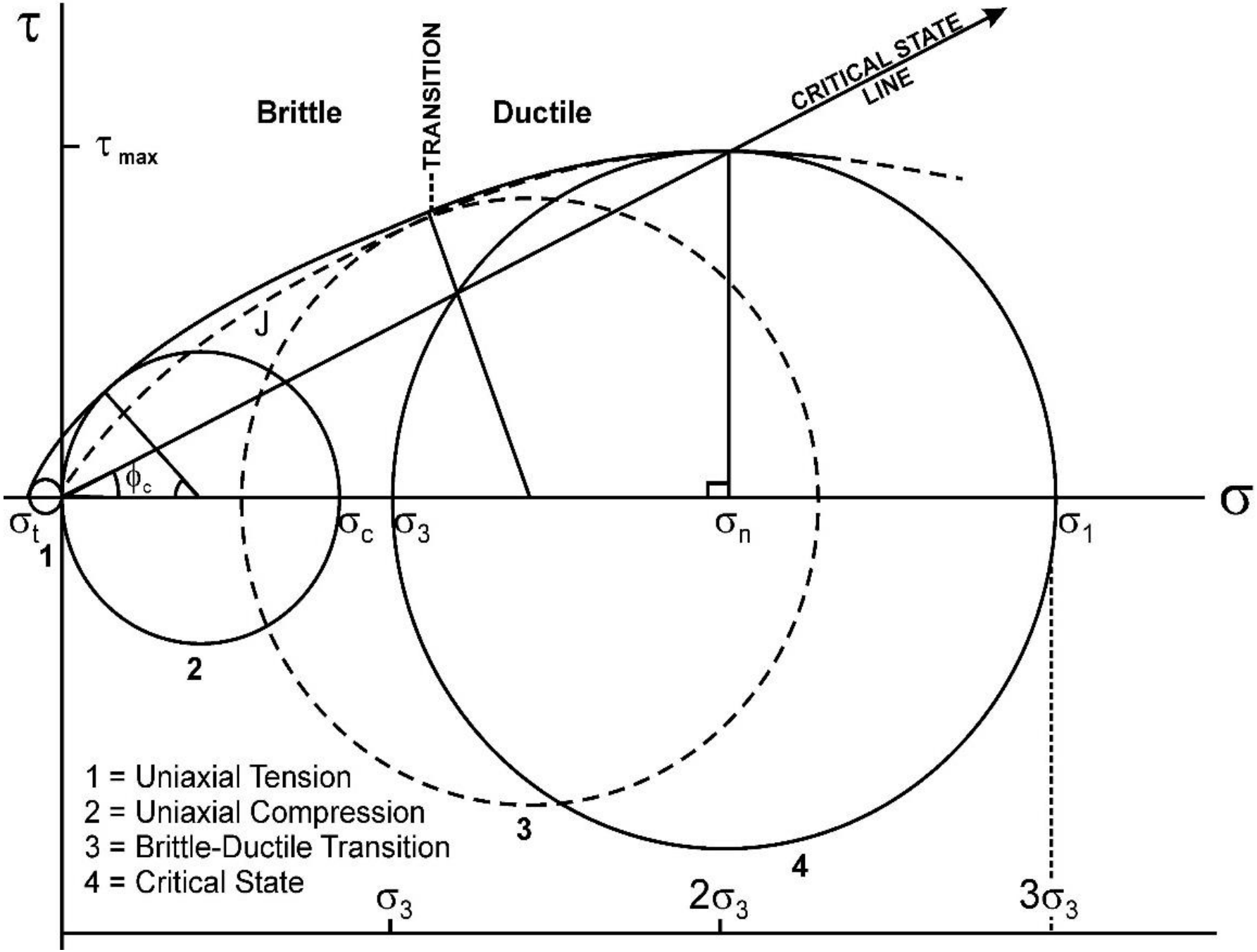
(green fractures propagation in shear)



A TEST OF MOHR-COULOMB: WE NEED THE COHESIVE STRENGTH OF INTACT ROCK.

IF ‘c’ IS VERY HIGH *AND IF THE ROCK MASS IS MASSIVE* WE FIND THAT FAILURE CANNOT BE BY SHEARING.

ANOTHER MODE OF FAILURE IS THE WEAKEST-LINK.



$$C = \tfrac{1}{2}(\sigma_c \times \sigma_t)^{1/2}$$

(lower-bound estimate)

Assumes linear envelope between tensile and compression circles.

Actual cohesion is higher due to curvature.

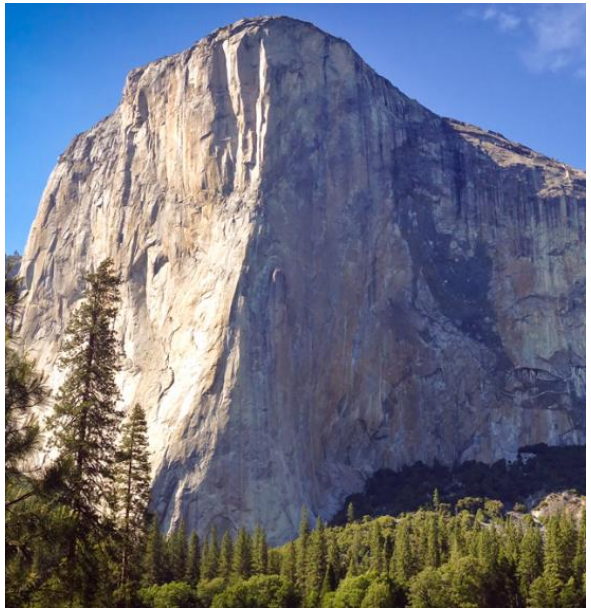
(Barton, 1976)

MOUNTAIN WALLS AND VERTICAL CLIFFS ARE A RICH SOURCE OF EMPIRICAL DATA – BECAUSE THEY HAVE HEIGHTS THAT ARE LIMITED BY THEIR ROCK PROPERTIES. (And tunnels have limited depths)

M-C? H-B? Linear or non-linear ‘c’ and ‘ϕ’

NO!.....σ_t/ν !

VERTICAL HEIGHT LIMITS OF CLIFFS AND MOUNTAIN WALLS – ***NEW EQUATION***:



El Capitain, Yosemite, California. granites, UCS = 100-150MPa.

Beachy Head, England. Chalks, UCS = 10 MPa (saturated ?)

West Temple, Zion, Utah. Sandstones, UCS = 50-75MPa.

Cappadocia, Turkey. Volcanic tuff, UCS = 1-2MPa.

$$H_{critical} \approx 100.\sigma_t/\gamma\nu \text{ (meters)}$$

(Have assumed $\sigma_v \approx \gamma H/100$ MPa)

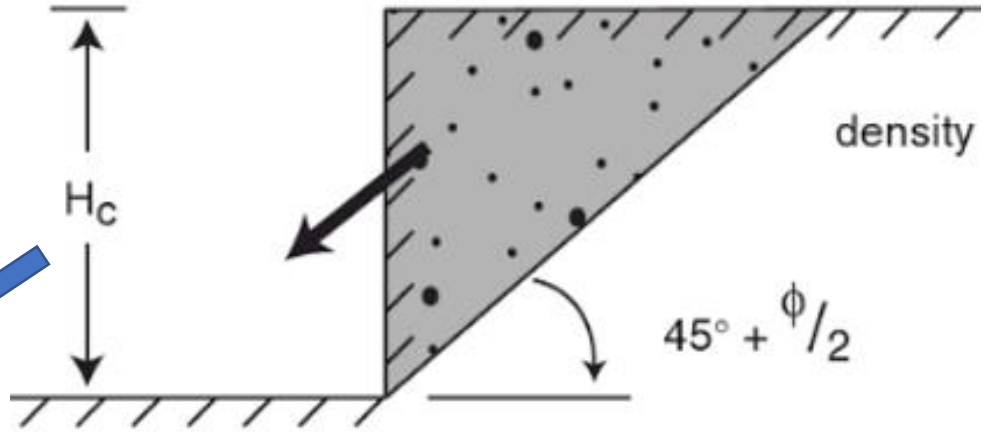
σ_t = tensile strength (MPa)

γ = density (when units are tons/m3)

ν = Poisson's ratio

Barton, 2016

‘SOIL MECHANICS’ THEORIES:



- ***2c/γ* . tan(45°+ φ/2) ≤ *Hc* ≤ *4c/γ* . tan(45°+ φ/2)**
- (Mohr-Coulomb, lower- and upper-bound)

- UNFORTUNATELY these classic solutions for soil are 3X to 6X IN ERROR WHEN ***EXTENSION FAILURE – and NOT SHEAR*** FAILURE OF **INTACT** ROCK IS OCCURING.

COMPARING 'SOIL MECHANICS' *SHEAR-STRENGTH-BASED* ESTIMATES of H_c WITH *EXTENSION-STRAIN* ESTIMATES

1. Sandstone

σ_c = 75MPa,

σ_t = 5MPa

$c = ½(75x5)^{1/2}$ = 9.7MPa

$$c = ½(\sigma_c \times \sigma_t)^{1/2}$$

2. Granite

σ_c = 150MPa,

σ_t = 10MPa

$c = ½(150x10)^{1/2}$ = 19.4MPa

1. Sandstone 'mountain wall' **($H_c = 2c/\gamma \times \tan(45°+\phi/2)$):**

H_c=2 x 9.7 x 1000/25 x tan (45° + 30.5°) = 3,001m !

(This is a 'lower-bound' estimate!)

2. Granite 'mountain-wall' **($H_c = 2c/\gamma \times \tan(45°+\phi/2)$):**

H_c=2 x 19.4 x 1000/27.5 x tan (45°+ 30.5°) = 5,456m !

(This is a 'lower-bound' estimate!)

BY COMPARISON *EXTENSION STRAIN THEORY*:

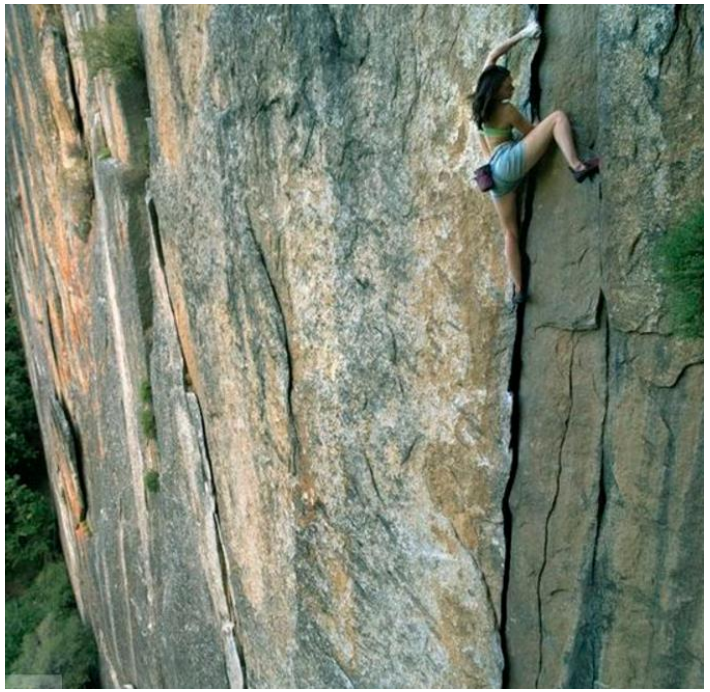
Sandstone: $100.\sigma_t/\gamma\nu$ = 100.5/2.5 x 0.25 = *800m*

Granite: $100.\sigma_t/\gamma\nu$ = 100.10/2.75 x 0.25 = *1,456m*

Lower-bound estimate M-C: $h = 2c/\gamma \tan(45° + \varphi/2)$		
σ_t = 0.2 MPa, σ_c = 2 MPa	γ = 2.0 t/m^3	h = 103m
σ_t = 0.5 MPa, σ_c = 5 MPa	γ = 2.0 t/m^3	h = 258m
σ_t = 5 MPa, σ_c = 50 MPa	γ = 2.5 t/m^3	h = 2,067m
σ_t = 10 MPa, σ_c = 100 MPa	γ = 2.8 t/m^3	h = 3,690m

Extension strain based: $H_c = \sigma_t/\gamma\nu$		
σ_t = 0.2 MPa, σ_c = 2 MPa	ν = 0.2	H_c = 50m
σ_t = 0.5 MPa, σ_c = 5 MPa	ν = 0.2	H_c = 125m
σ_t = 5 MPa, σ_c = 50 MPa	ν = 0.25	H_c = 800m
σ_t = 10 MPa, σ_c = 100MPa	ν = 0.25	H_c = 1,430m

For rock cliffs and mountain walls the choice is clear: do not use M-C.



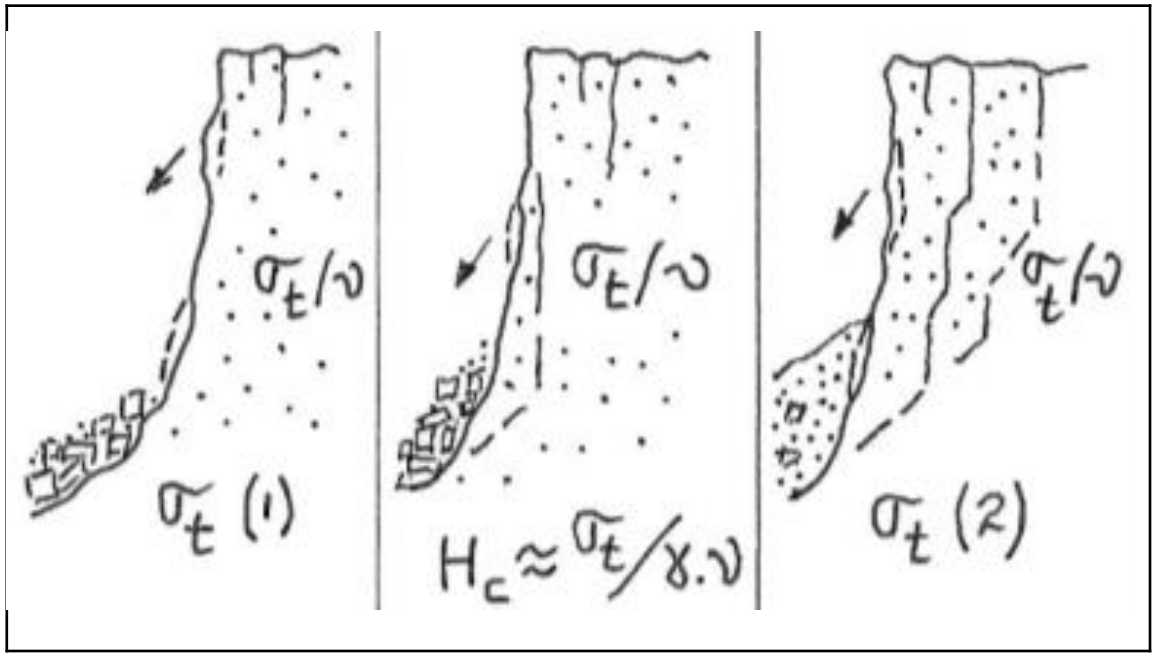
SHEETING JOINTS (AND ASSOCIATED CRACKS)

(WITH $H_c = 100\sigma_t/\gamma\nu$ (EXTENSION-STRAIN-FRACTURING) DO NOT NEED CURVATURE TO EXPLAIN SHEETING JOINTS)

Free-solo rock-climbing aces:

Steph Davis (see her book)

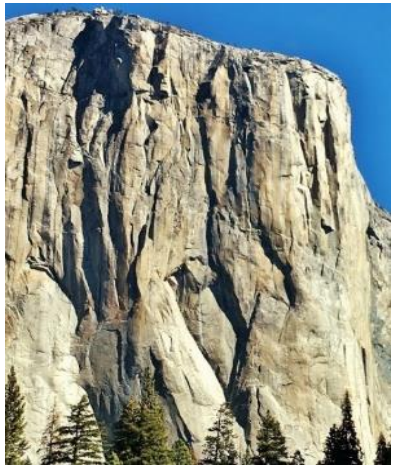
Alex Honnold (see his book)



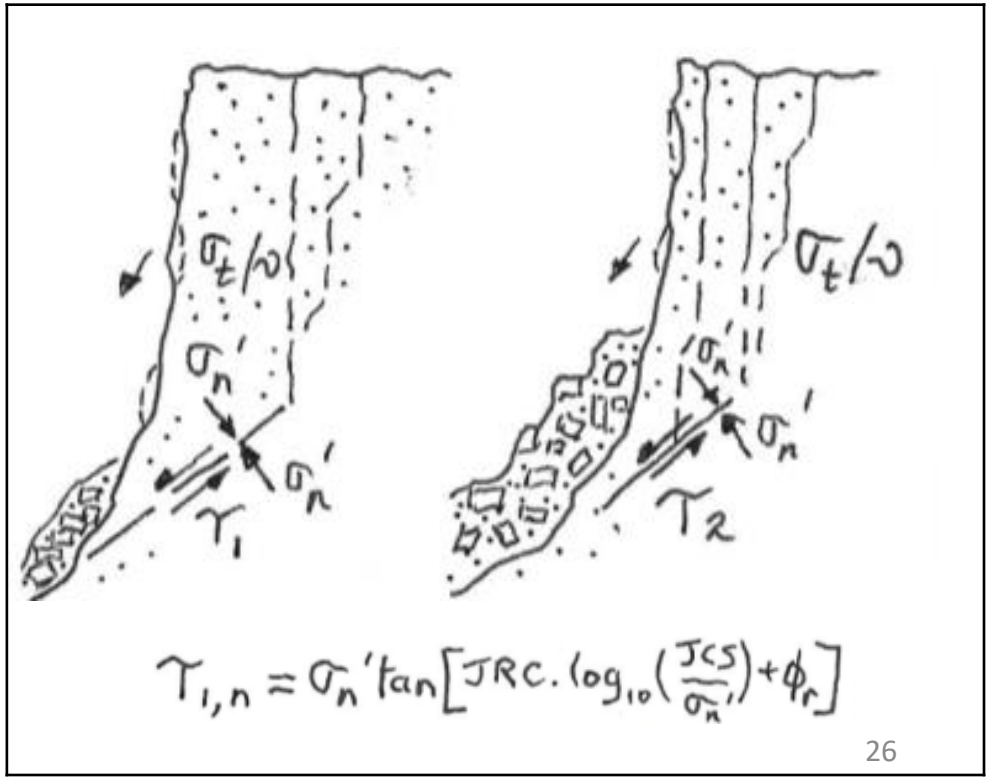
EXTENSION FAILURES CAUSE SHEETING FRACTURES, AND LIMIT ULTIMATE WALL HEIGHTS

(NOTE! σt REDUCES OVER GEO-MORPHOLOGICAL TIME-SCALES)

SHEAR FAILURE (AND TENSION CRACKS) THREATENING FUTURE MOUNTAIN ROCK AVALANCHE?

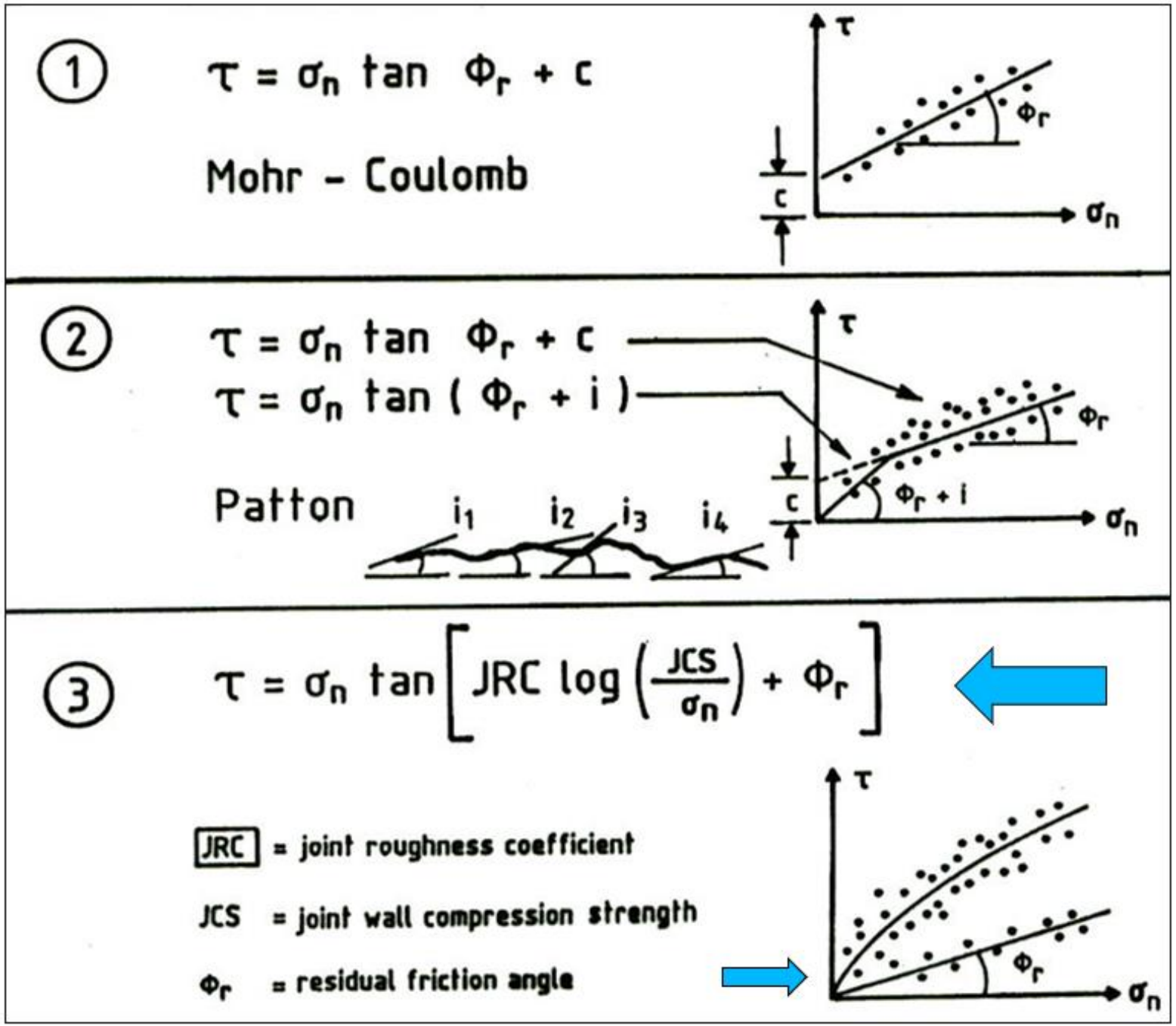


El Capitan, CA. and Holtanna, Antarctic.



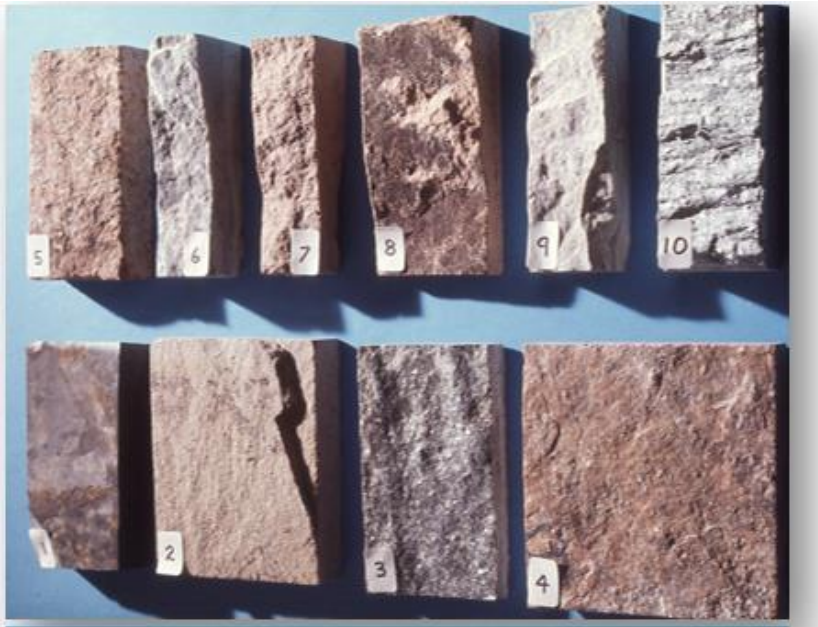
ROCK JOINT MODELLING

FOR UDEC / UDEC-BB



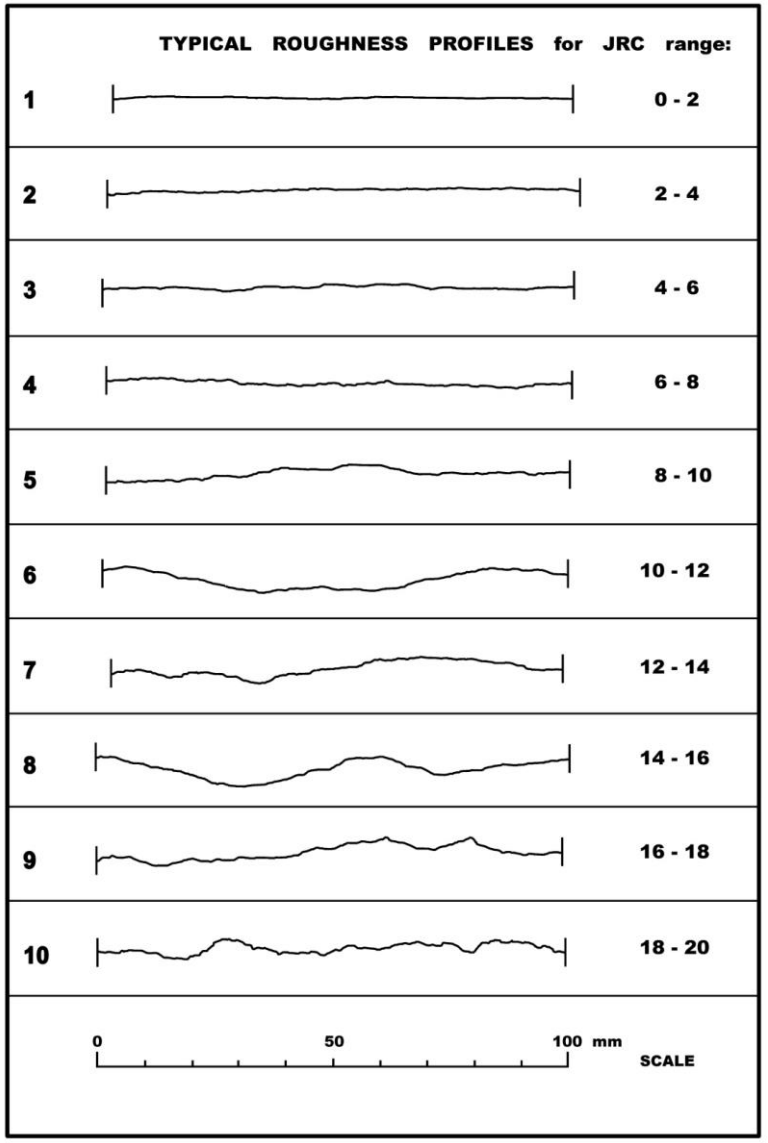
The third option for shear strength estimation includes JRC

Derivation of JRC: tilt test or profile matching

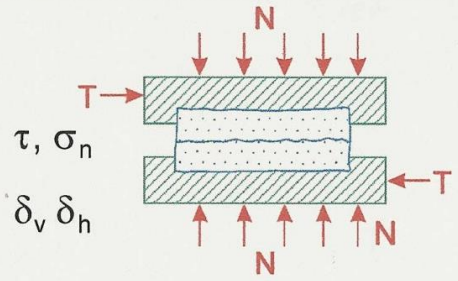


130 joint samples. Roughness measurement and ***tilt test***

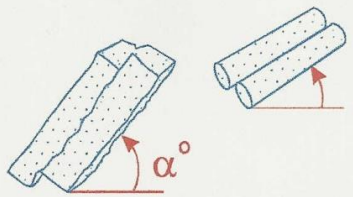
(Barton and Choubey, 1977)



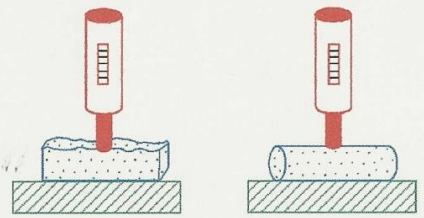
SHEAR BOX AND INDEX TESTING OF ROCK JOINTS



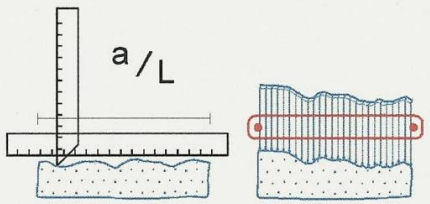
SHEAR BOX



TILT TEST



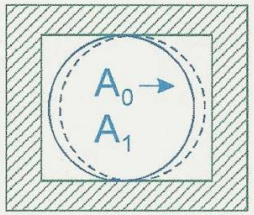
SCHMIDT HAMMER

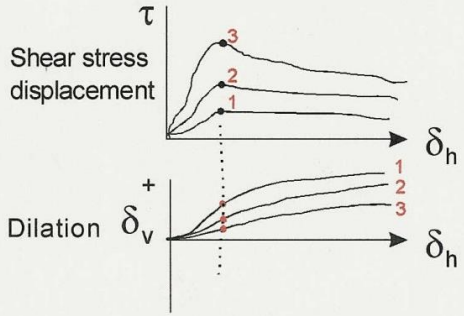


ROUGHNESS PROFILES

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

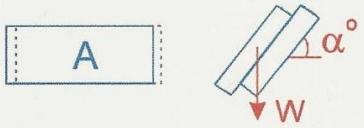
$\tau_n = \frac{T}{A_1}$





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

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



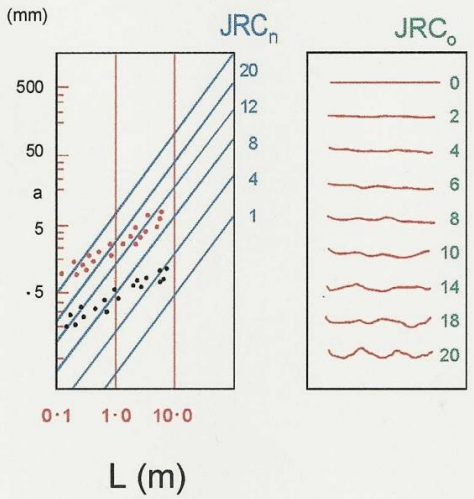
$$\phi_r = (\phi_b - 20^o) + 20 \, r_5/R_5$$

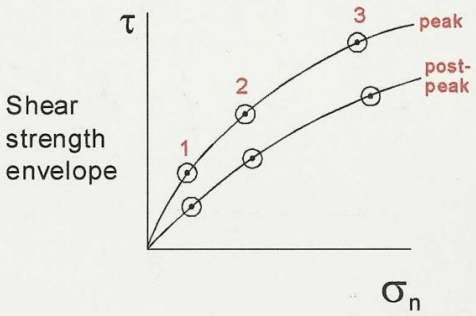
r	R
30	40
~~25~~	~~33~~
31	41
32	~~35~~
32	40
~~24~~	42
31	~~34~~
~~29~~	41
~~23~~	~~36~~
~~26~~	~~37~~

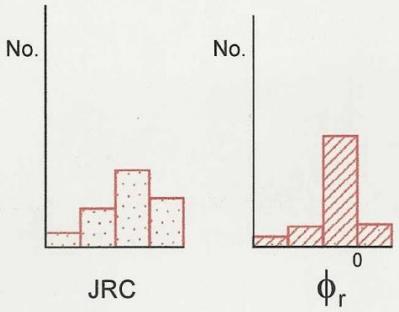
$\bar{r}_5 \rightarrow JCS_o$

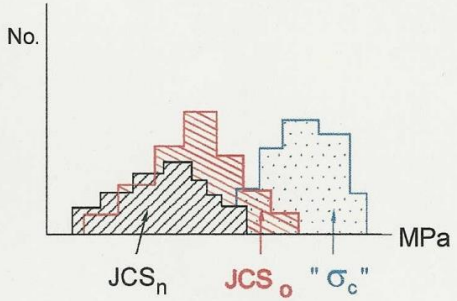
$\bar{R}_5 \rightarrow$ "σ_c"

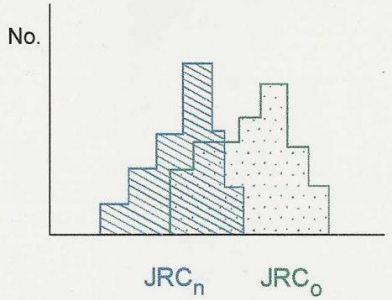
$\bar{r}_5 = 31{\cdot}1$ $\bar{R}_5 = 40{\cdot}8$



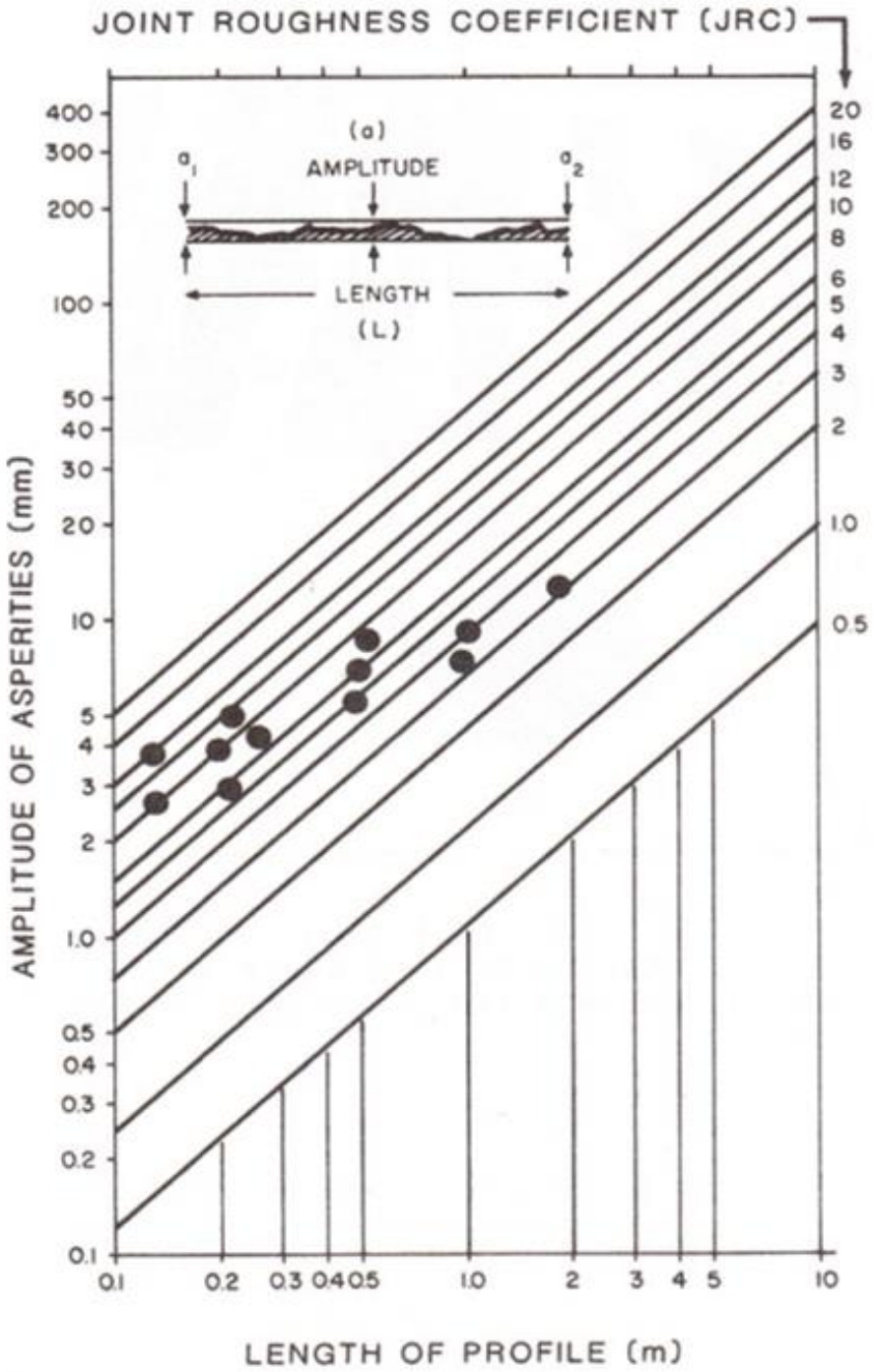






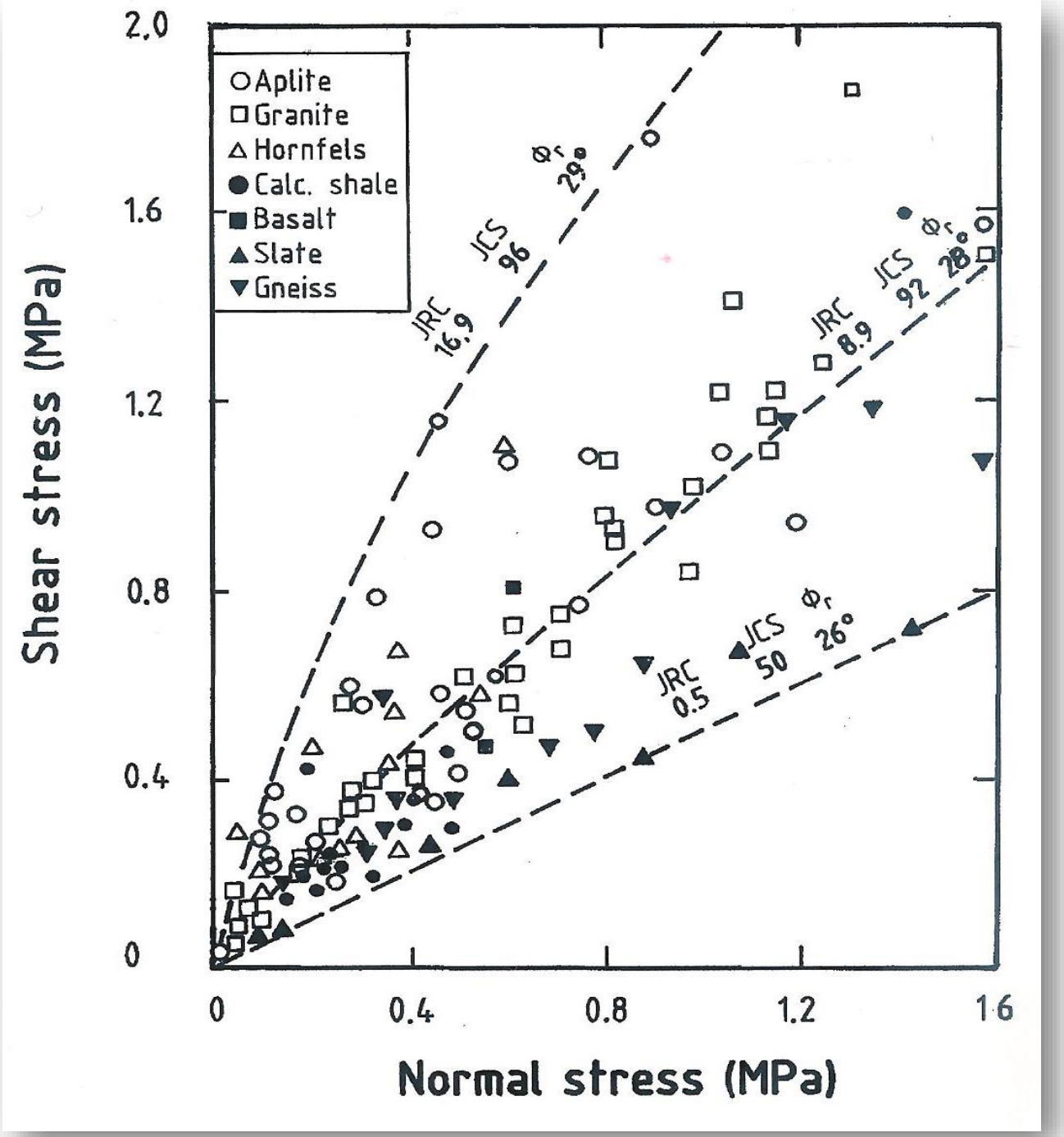


g:\desktop\coreldrw\542004\RockJoints.cdr



The 'a/L' method

While core logging it is easy to record a/L for different joint sets



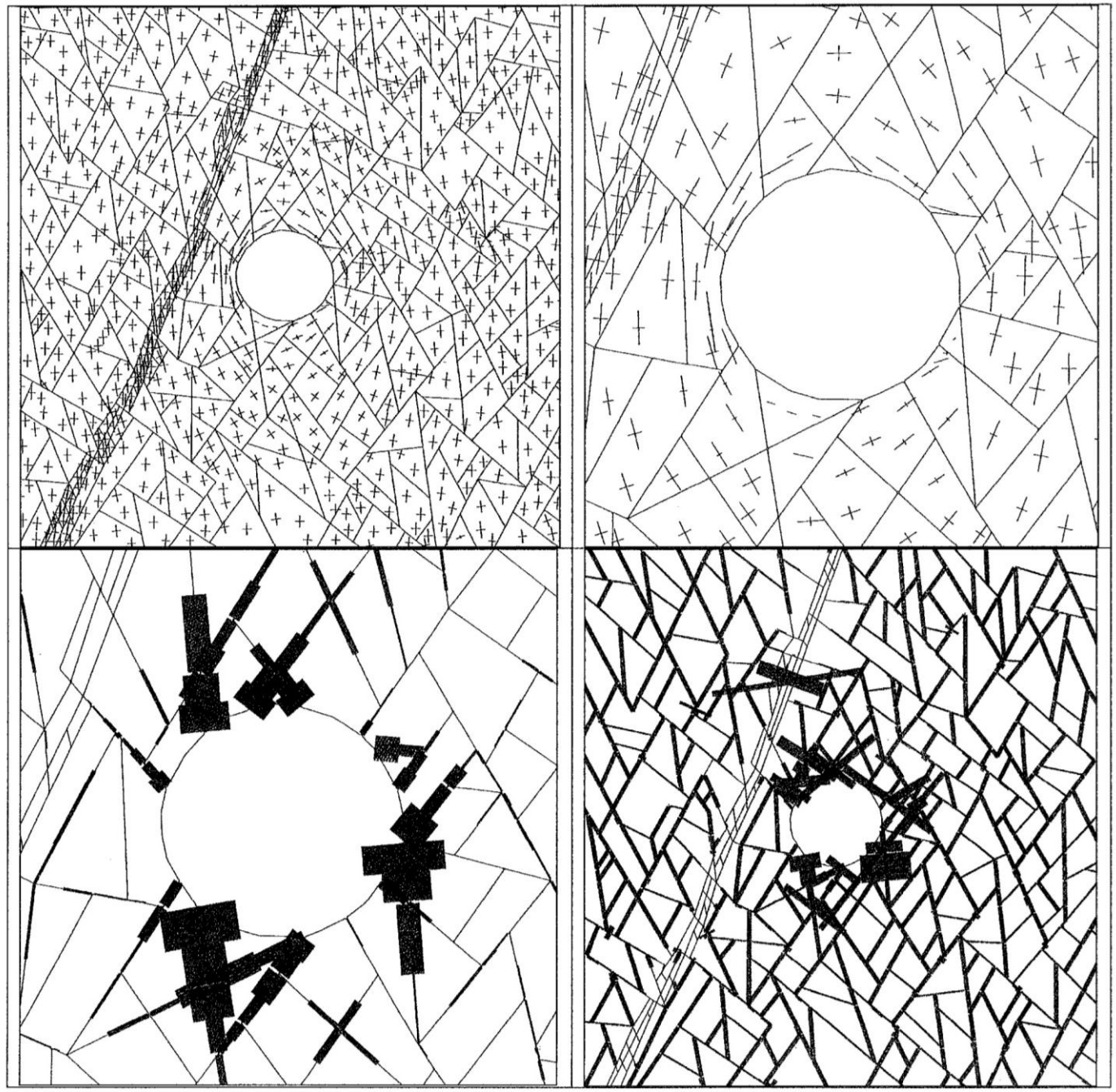
136 ROCK-JOINT SAMPLES
(Barton and Choubey 1977)

THREE CURVED **PEAK SHEAR STRENGTH** ENVELOPES SHOWN:

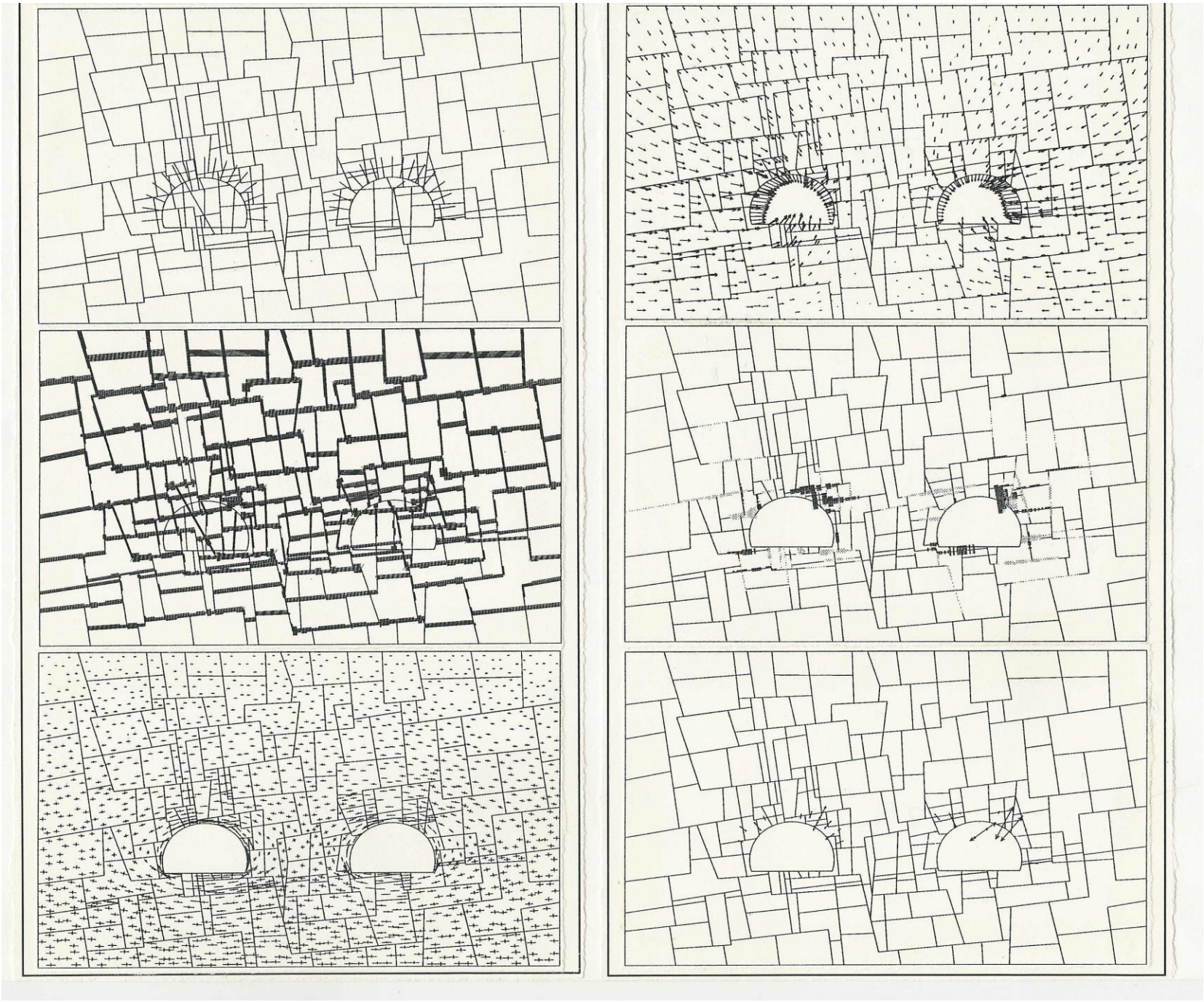
1.MAXIMUM STRENGTH WITH JRC = 16.9

2. MEAN PARAMETERS
JRC=8.9,
JCS=92MPA
ΦR=28º

3. MINIMUM STRENGTH
WITH ΦR = 26º



ALTHOUGH A LITTLE EXAGGERATED IN 2D, WE POTENTIALLY LEARN MUCH MORE FROM DISCONTINUUM ANALYSES



The interaction of tunnels.

We learn more from discontinua.

FINALLY,
A COMPARISON
OF SOME WELL-KNOWN
EQUATIONS

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'

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

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

Expression	Origin
$"\varphi" \approx \tan^{-1}\left(\frac{J_r}{J_a} \times \frac{J_w}{1}\right)$	FC from Q
$\varphi' = a\sin\left[\frac{6am_b\,(s + m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a) + 6am_b\,(s + m_b\sigma'_{3n})^{a-1}}\right]$	From GSI
$"c" \approx \left(\frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100}\right)$	CC from Q
$c' = \frac{\sigma_{ci}\left[(1+2a)s + (1-a)m_b\sigma'_{3n}\right](s + m_b\sigma'_{3n})^{a-1}}{(1+u)(2+a)\sqrt{1 + [6am_b(s + m_b\sigma'_{3n})^{a-1}]/[(1+a)(2+a)]}}$	From GSI

$$m_b = m_i.\,exp\left(\frac{GSI-100}{28-14.D}\right)$$

$$s = exp\left(\frac{GSI-100}{9-3.D}\right)$$

$$a = \frac{1}{2} + \frac{1}{6}.\left(exp^{-GSI/15} - exp^{-20/3}\right)$$

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

$$E_m(\text{MPa}) = 10^5\left(\frac{1 - D/2}{1 + e^{\left[\frac{75+25D-GSI}{11}\right]}}\right)$$

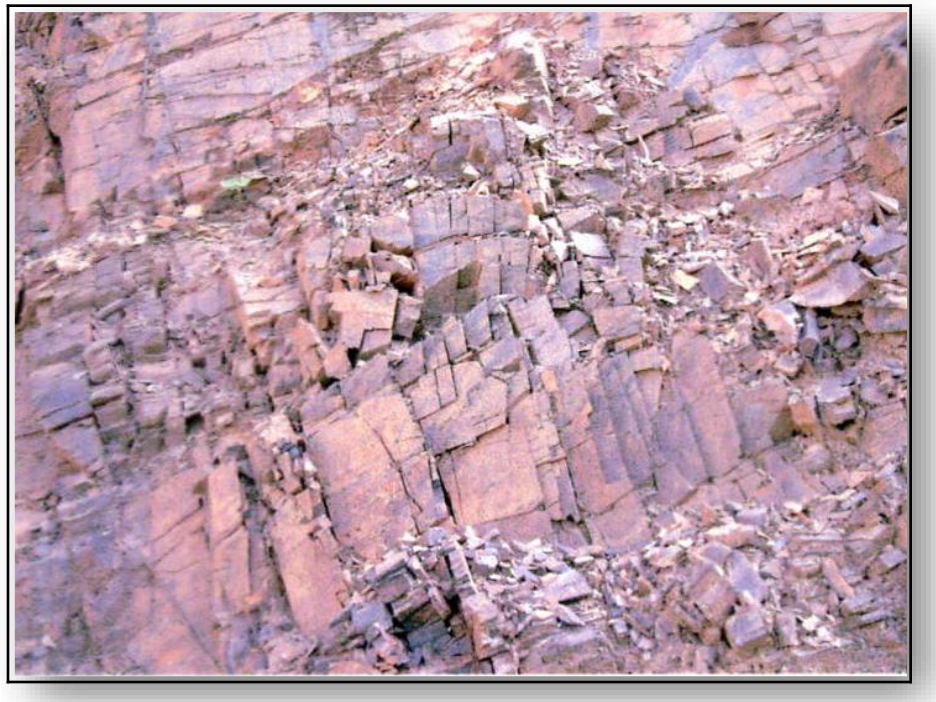
$GSI_1 = RMR_{1989} - 5 = R_1 + R_2 + R_3 + R_4 + R_5(=15) - 5$	(1)	Hoek et al. 1995
$GSI_2 = 1.5\ R_4 + 0.5\ RQD$	(2)	Hoek et al. 2013
$GSI_3 = 15\log\left(\frac{RQD}{Jn}\frac{Jr}{Ja}\right) + 50$	(3)	(This was not a Barton, 1995 suggestion)
$GSI_4 = 9\ln\left(\frac{RQD}{Jn}\frac{Jr}{Ja}\right) + 44$	(4)	Hoek et al. 1995
$GSI_5 = \frac{52Jr/Ja}{(1+Jr/Ja)} + 0{,}5RQD$	(5)	Hoek et al. 2013
$GSI_6 = \frac{26{,}5 + 8{,}79\ln Jc + 0{,}9\ln Vb}{1 + 0{,}0151\ln Jc - 0{,}0253\ln Vb}$	(6)	Cai and Kaiser 2006
$GSI_7 = 153 - \frac{165}{\left[1 + \left(\frac{Jp}{0{,}19}\right)^{0{,}44}\right]}$	(7)	Russo 2009

SOME OF THE ATTEMPTS AT IMPROVING THE QUANTIFICATION

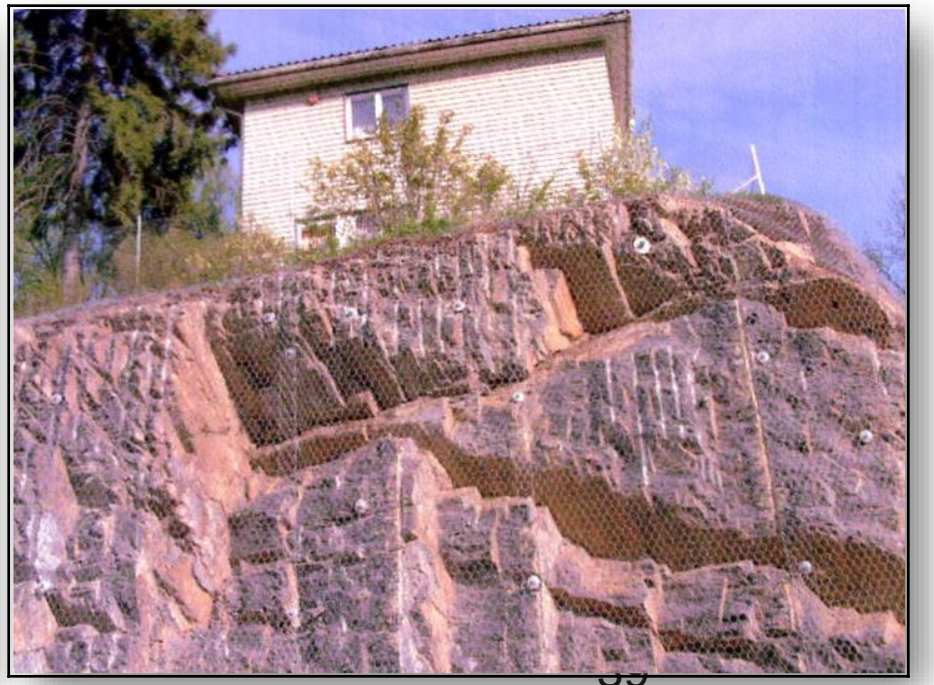
RQD	J_n	J_r	J_a	J_w	SRF	Q	σ_c	Q_c	*FC°*	*CC* MPa	V_p km/s	E_{mass} GPa
100	2	2	1	1	1	100	100	100	63°	50	5.5	46
90	9	1	1	1	1	10	100	10	45°	10	4.5	22
60	12	1.5	2	0.66	1	2.5	50	1.2	26°	2.5	3.6	10.7
30	15	1	4	0.66	2.5	0.13	33	0.04	9°	0.26	2.1	3.5

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

Low CC –shotcrete preferred



Low FC – bolting preferred



Nick Barton obtained a London University B.Sc. in Civil Engineering from King's College in 1966, and a Ph.D. concerning shear strength and rock slope stability from Imperial College in 1971. He worked for two long periods at NGI in Oslo, and for four years at TerraTek in Salt Lake City. Since 2000 he has had his own international rock engineering one-man consultancy, Nick Barton & Associates, based in Oslo and São Paulo. He has consulted on several hundred rock engineering projects in a total of 40 countries during 50 years, has 270 publications as first or single author, and has written two books, one on TBM prognosis, the other linking rock quality and seismic attributes of rock masses at all scales. He is currently writing a book with Bandis: Engineering in jointed and faulted rock (expected 2023). He has twelve international awards including the 6th Müller Lecture of ISRM. **He developed the widely used Q-system for classifying rock masses, and for selecting rock tunnel and cavern single-shell support in 1974. He was originator of the rock joint shear strength parameters JRC and JCS and co-developer of the resulting Barton-Bandis constitutive laws for rock joint coupled M-H modelling in 1982, which was incorporated as a sub-routine in UDEC-BB in 1985.** He has also developed the Q*tbm* prognosis method and Q*slope* for selecting maintenance-free rock cutting and bench-face angles. His chief areas of consulting activity have been in hydropower tunnelling and cavern construction and performance, nuclear waste disposal site characterization, metro tunnels and caverns, and site characterization assistance at high dams. He has given more than forty keynote lectures in international conferences.