

Using Deep Tunnels, Cliffs, Mountain
Walls, Models and Mountains: *to Explore
Failure Modes in Rock and Rock masses*

Nick Barton, NB&A, Oslo

CONTENT of LECTURE

1. FAILURE MODES IN DEEP TUNNELS: STRESS/STRENGTH? CRITICAL STRAIN?
2. CLIFF FAILURES IN ROCK – CAN WE USE ‘SOIL MECHANICS METHODS’?
3. CRITICAL TENSILE STRAIN APPROACH FOR CLIFFS AND MOUNTAIN WALLS
4. COMBINED FAILURE MODES. CCSS: Crack!! Crunch! Scrape! Swoosh!! M-C?
5. IS PREKESTOLEN (the ‘Pulpit’) SAFE TO STAND ON, 600m over LYSEFJORD?
6. BLOCK-ROTATION FAILURE MODES, AND STEAM!
7. THE HIGHEST MOUNTAINS ARE ‘ONLY’ 8 to 9 km HIGH – WHY IS THIS?
8. CRITICAL STATE SHEAR STRENGTH CURVATURE IS RELATED TO UCS

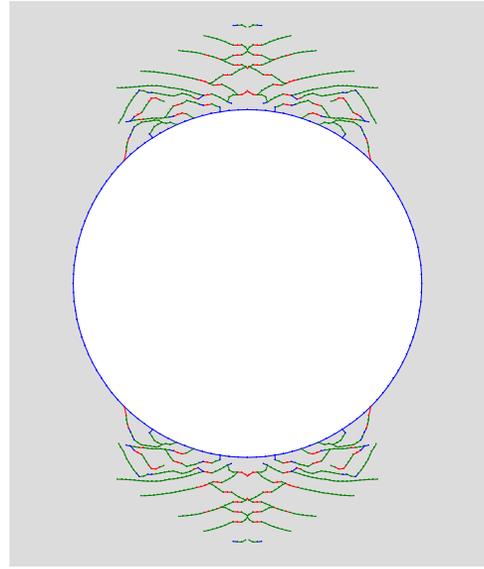
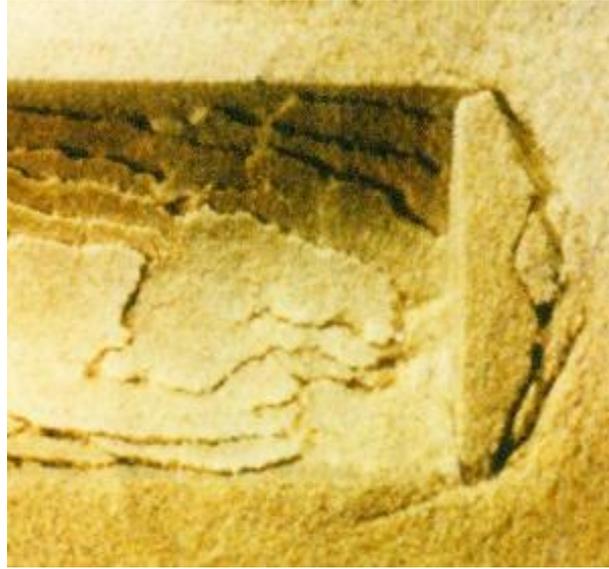


Baotang Shen, CSIRO-Australia
(FRACOD, σ_t / v , non-linearity)

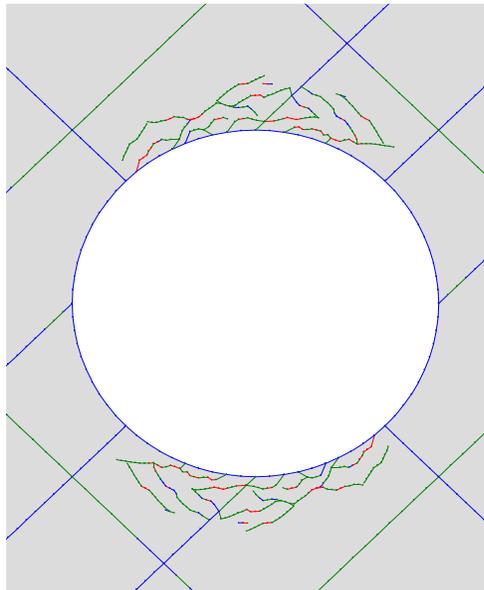
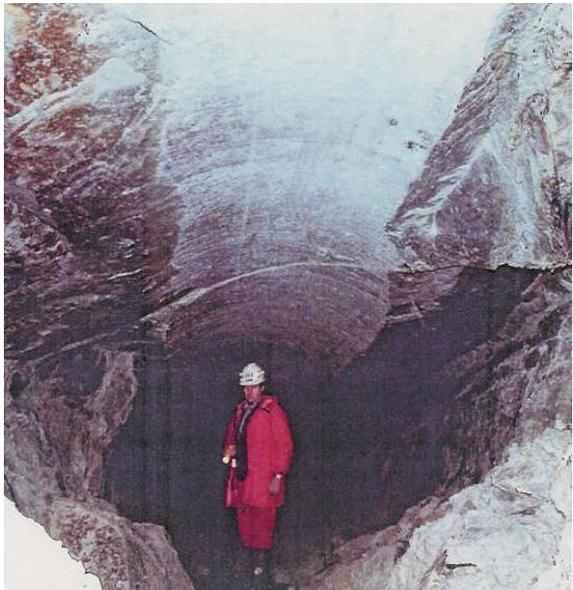


Mahendra Singh, Univ. Of Roorkee,
India. (σ_3 critical \approx UCS, non-linearity)

TWO VITAL CONTRIBUTORS TO FRACTURING MODES AND CURVED STRENGTH ENVELOPES



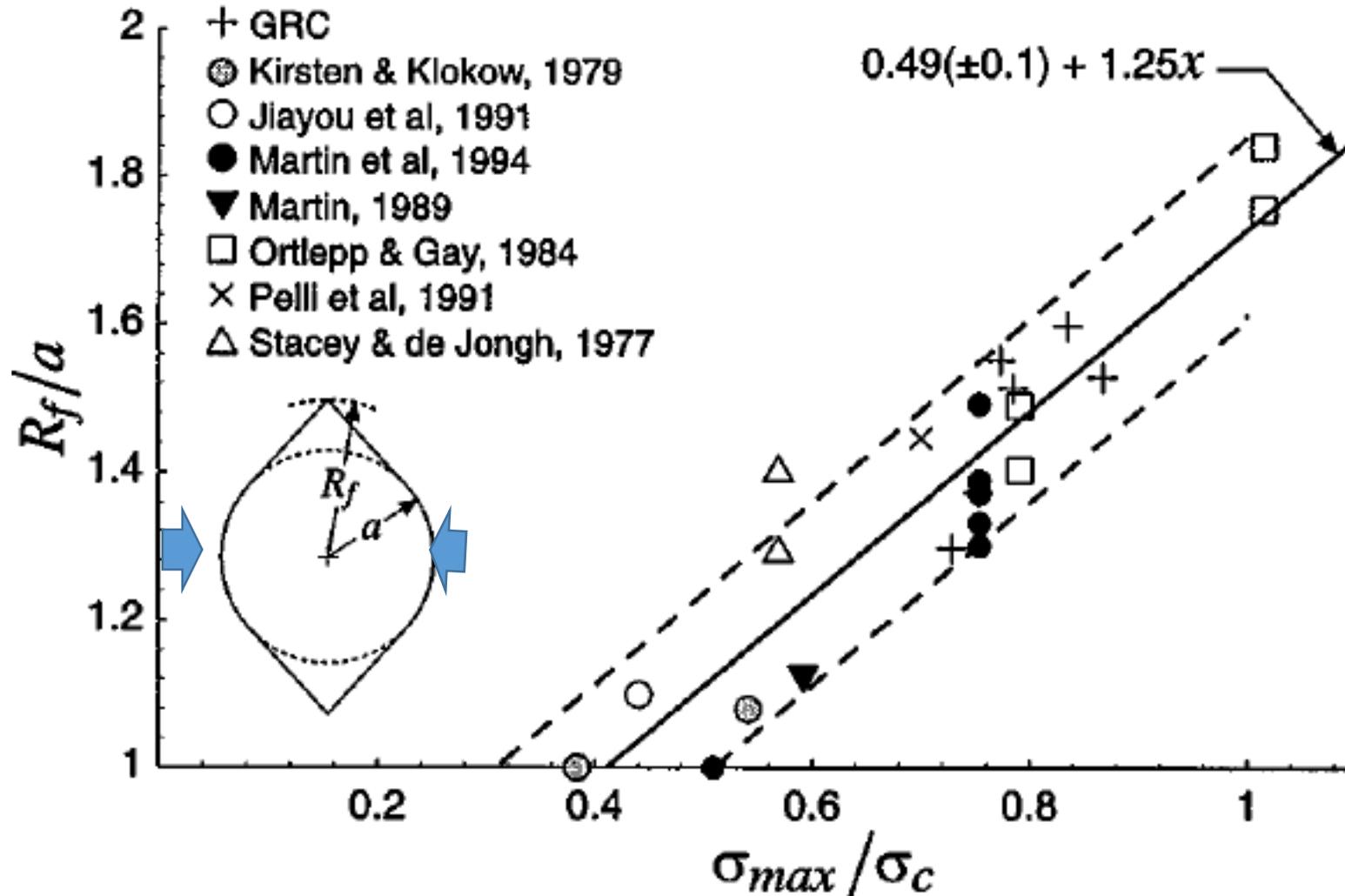
A selection of tunnel failure modes when higher stress:



**Physical models
TBM tunnels
Numerical models**

a) TUNNELS IN MASSIVE ROCK: STRESS (or strain) INDUCED FAILURE
 Traditionally expect 'stress-induced' failure when: $\sigma_{\theta \max} / \sigma_c > 0.4 \pm 0.1$

Maximum tangential stress estimate: $\sigma_{\theta \max} \approx 3\sigma_1 - \sigma_3$ ($\sigma_{\theta \min} \approx 3\sigma_3 - \sigma_1$)



(Martin et al. 1998)

THE 'Q-system' ?

As a briefest possible introduction
(for soil engineers):

Q means rock mass quality.

Q consists of *ratings for six parameters*.

$$Q = \frac{\text{RQD}}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{\text{SRF}} = \text{(Block size)} \times \text{(friction)} \times \text{('active stress')}$$



Q used here!



SUGAR LOAF MOUNTAIN, RIO DE JANEIRO

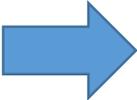
TOP END OF ROCK MASS QUALITY
SCALE.

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

i.e. >1000

SRF increases at great
depth $\sigma_{\theta \max} / \sigma_c$

BRAZILIAN HYDROPOWER PROJECT COLLAPSE IN FAULT

LOWEST END OF THE ROCK
MASS QUALITY SCALE. 

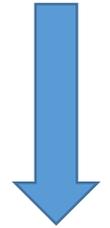
$$Q \approx 10/20 \times 1/8 \times 0.5/20$$

i.e. < 0.001



b) IN Q-SYSTEM, SAME EXPECTATION. If $\sigma_{\theta \max} / \sigma_c > 0.4$:
 higher SRF – lower Q-value – need heavier tunnel support.

CASE RECORD EVIDENCE

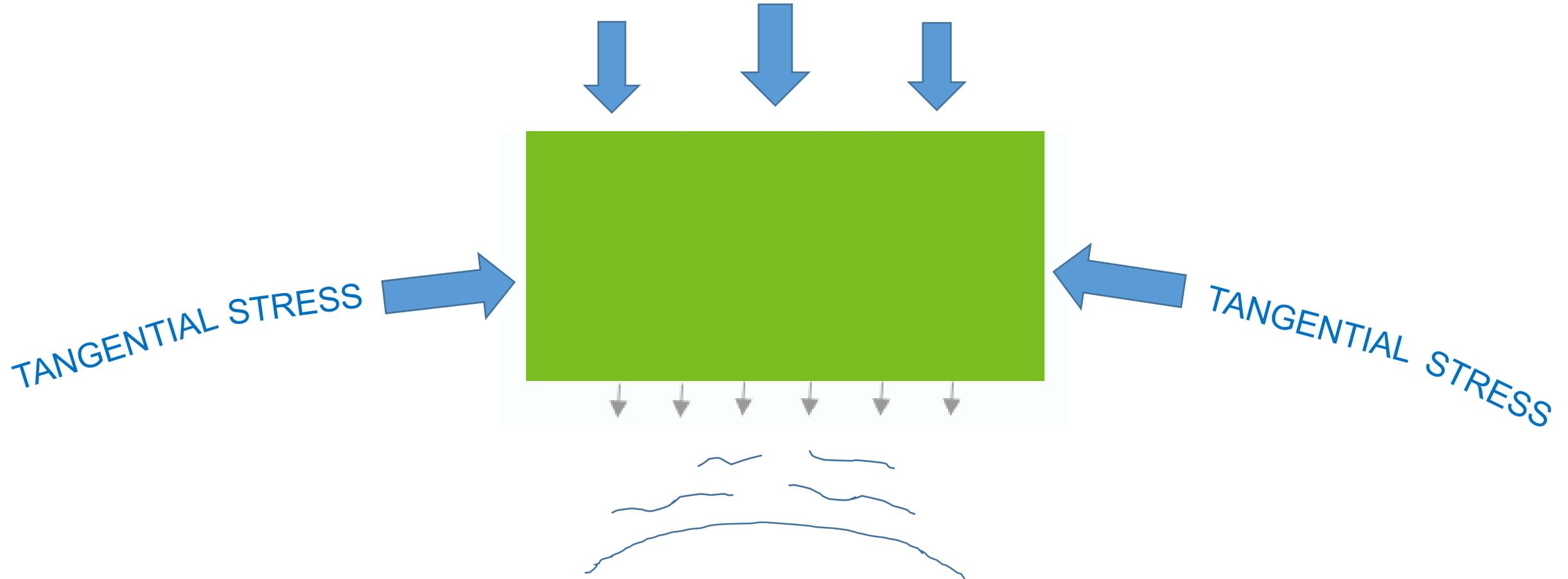


(Table 6b of Grimstad and Barton, 1993)

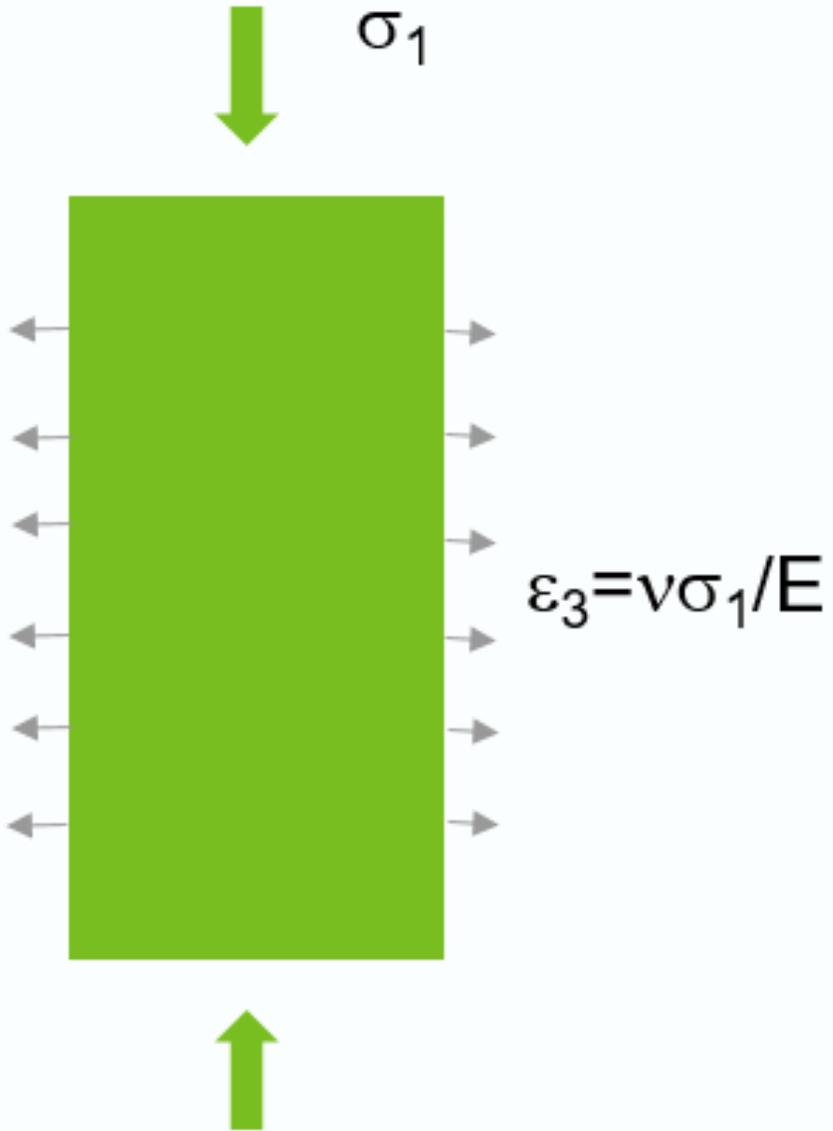
(1974)

<i>b) Competent rock, rock stress problems</i>		σ_c / σ_1	$\sigma_{\theta} / \sigma_c$	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

AROUND A TUNNEL: Poisson's ratio causes lateral strain



**NEXT TO THE TUNNEL MAY GET (TENSILE) CRACKING
– EVEN WHEN ALL STRESSES ARE STILL COMPRESSIVE**



TENSILE STRAIN CRITERION

(Proposed by Stacey, 1981, but fully applied by Baotang Shen in 2015-2016)

$$\varepsilon_1 = \sigma_1/E \quad \dots \quad \varepsilon_3 = \nu\sigma_1/E$$

Critical tensile strain for tensile failure:

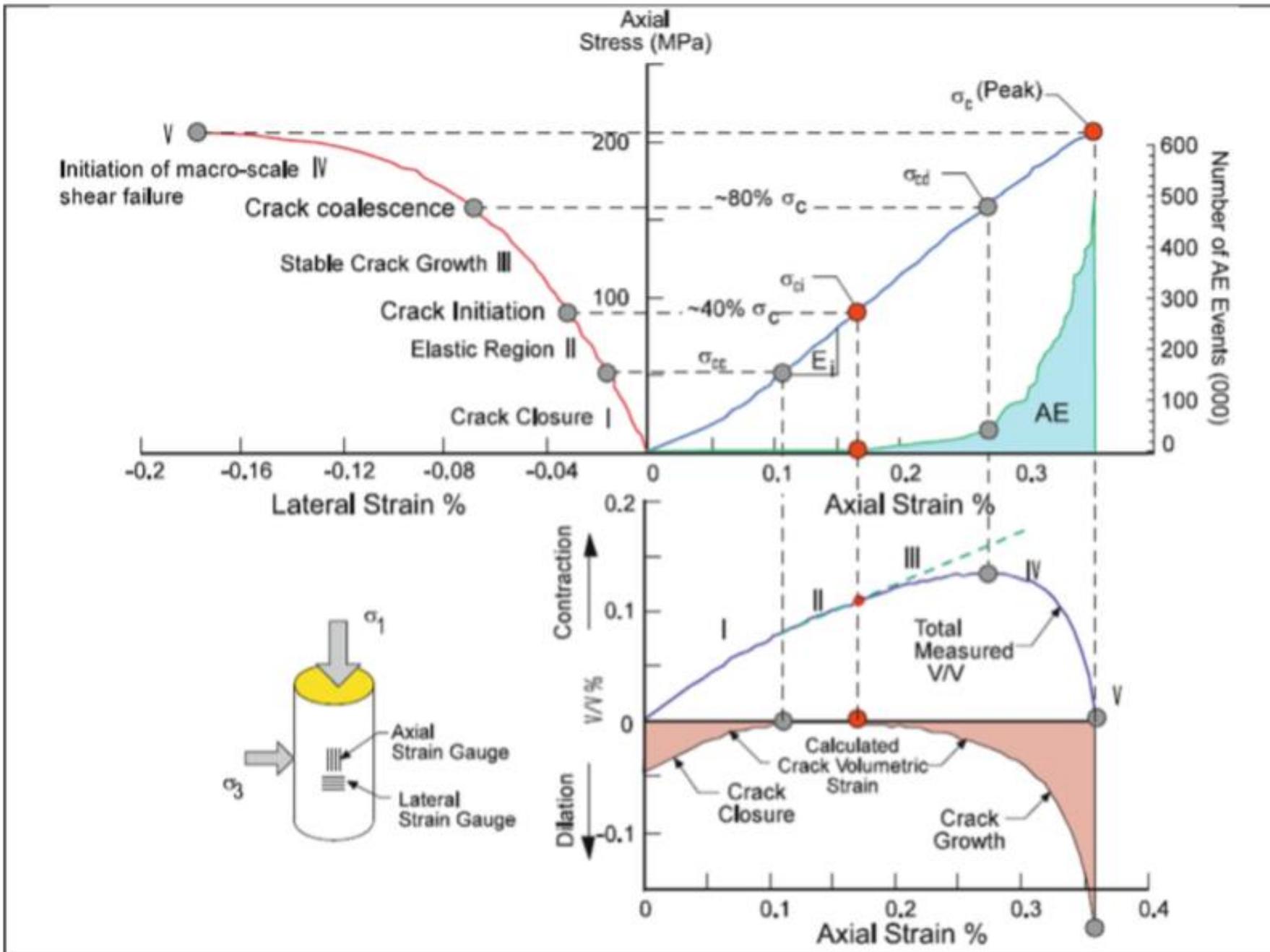
$$\varepsilon_c = \sigma_t/E \quad (\text{a definition})$$

Equate strains, and eliminate E.

Therefore:

$$\sigma_1 \text{ (critical)} = \sigma_t/\nu \quad (\approx 0.4 \times \text{UCS})$$

(We get '0.4' if $\text{UCS}/\sigma_t \approx 10$, and $\nu \approx 0.25$)



Martin, 1997
and others.

Note: AE also
initiates at
approx.
 $\sigma_1 = 0.4 \sigma_c$

CRITICAL EXTENSION STRAIN:

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

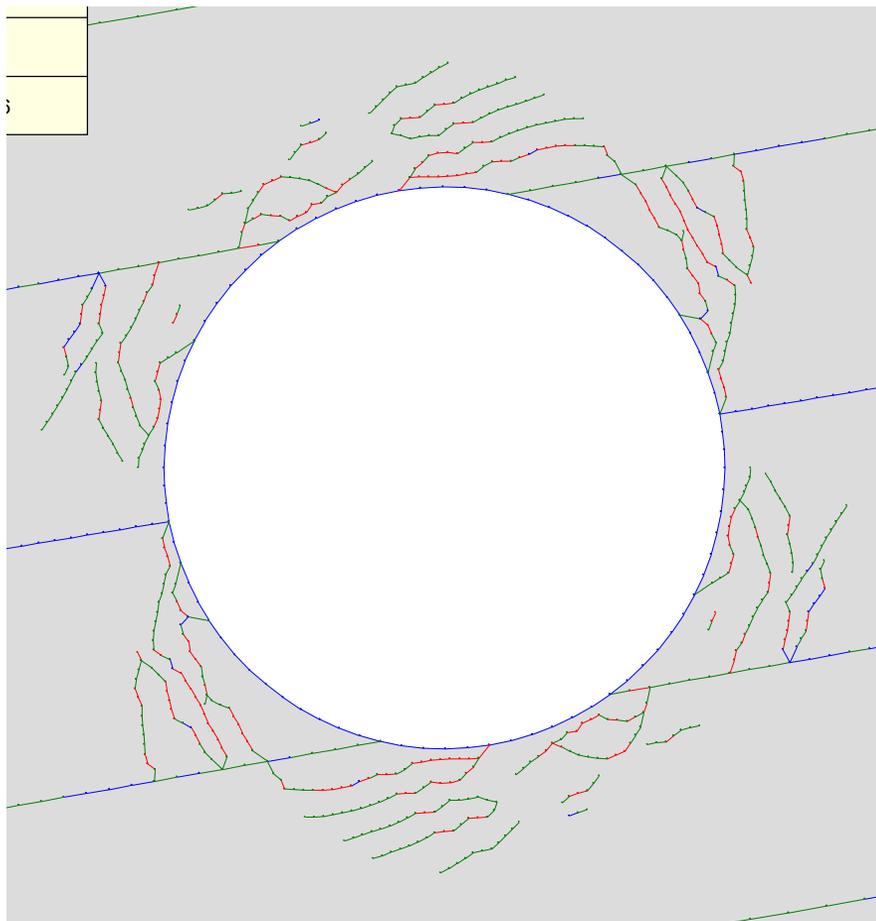
(Baotang Shen, in Barton and Shen, 2017)

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

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?

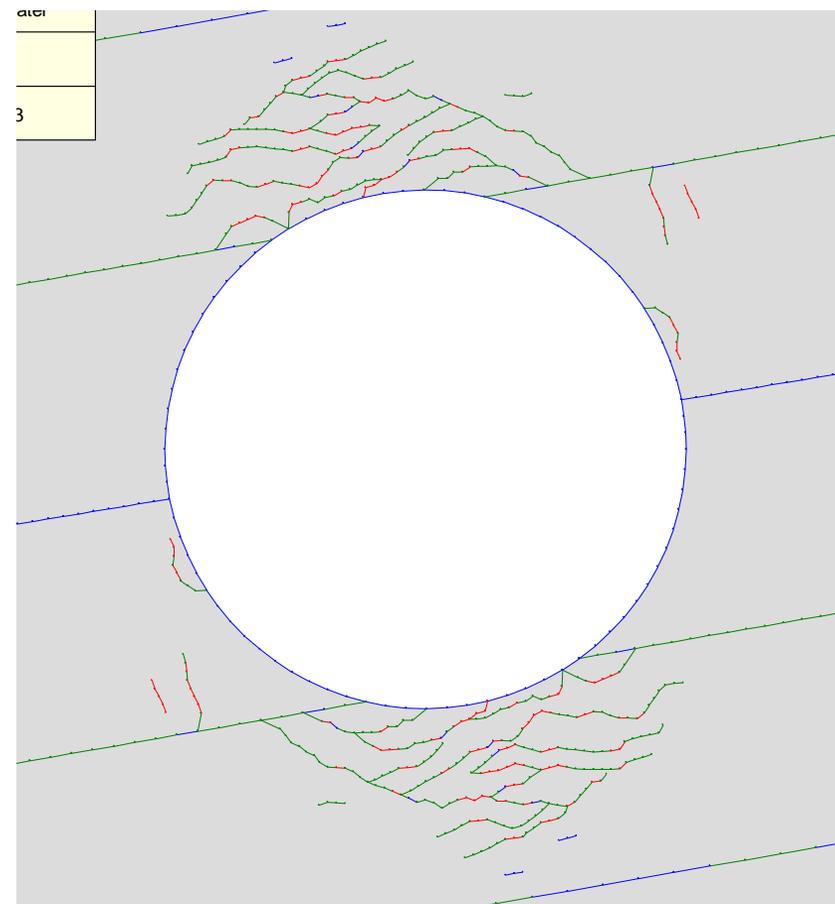
THESE TWO FRACOD MODELS (by Dr. Baotang Shen) 'PROVE' THAT IT WAS THE HIGH VERTICAL STRESS WHICH CAUSED THE FAILURE.



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)





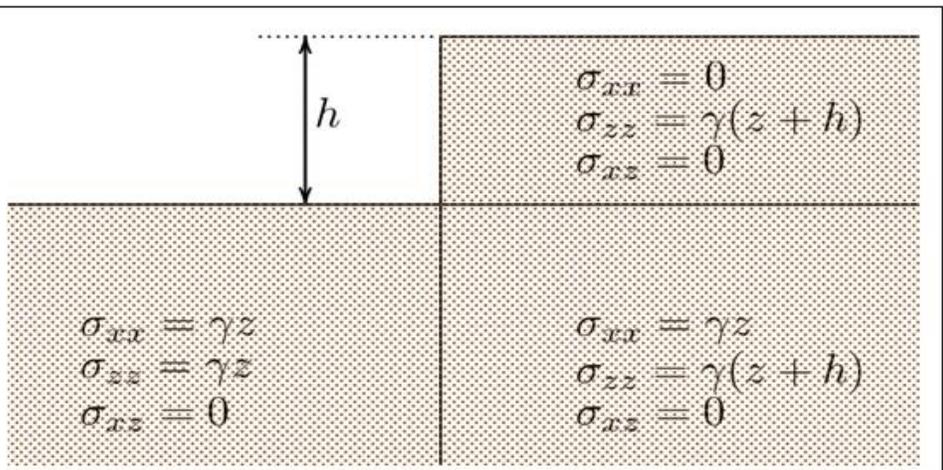
Tell-tale signs of *over-stress*, or is it *over-strain*?

(Jinping II)

2. EXTENSION-STRAIN FRACTURING *NOW APPLIED* TO FAILURE OF *VERTICAL CLIFFS and MOUNTAIN* *WALLS*

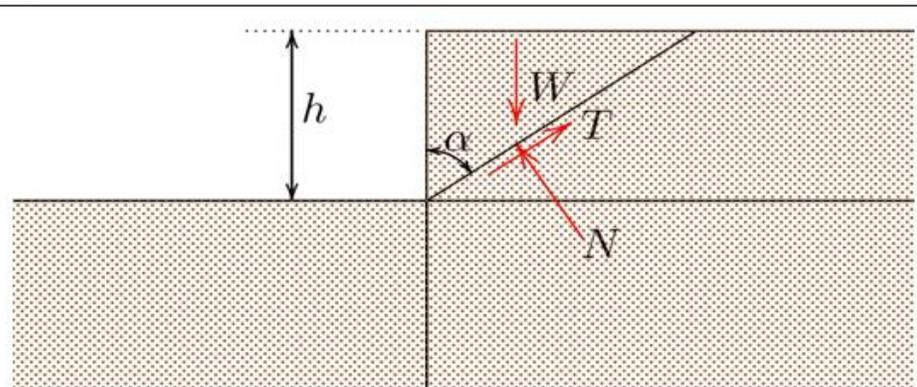


FIRST A LOOK AT CLASSIC SOIL
MECHANICS BASED SOLUTIONS TO THE
'VERTICAL-CUT' PROBLEM

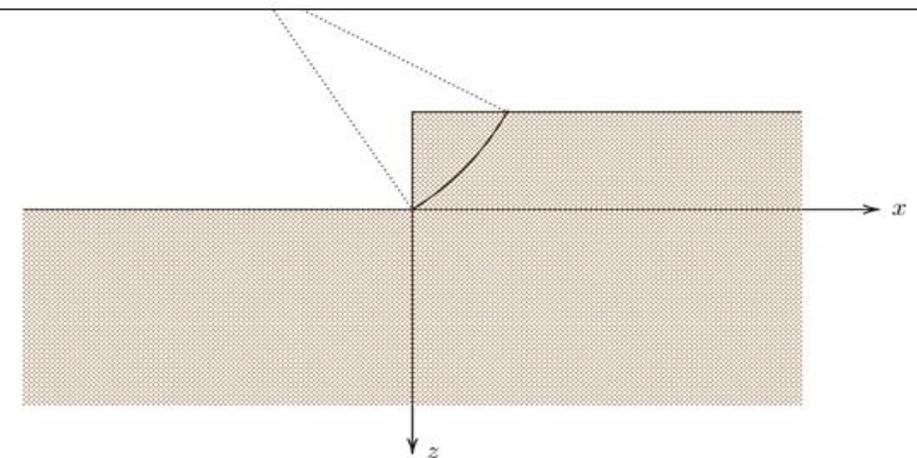


(a) Assumed equilibrium of three zones (of soil) gives a **lowerbound** solution for \underline{h} .

(Verruijt, 2001)



(b) **Upperbound** solution for \underline{h} involves a specific shear surface. (Verruijt, 2001).



(c) **Circular** failure surface.

(Fellenius, 1927)

Broad limits assuming **c and phi** :

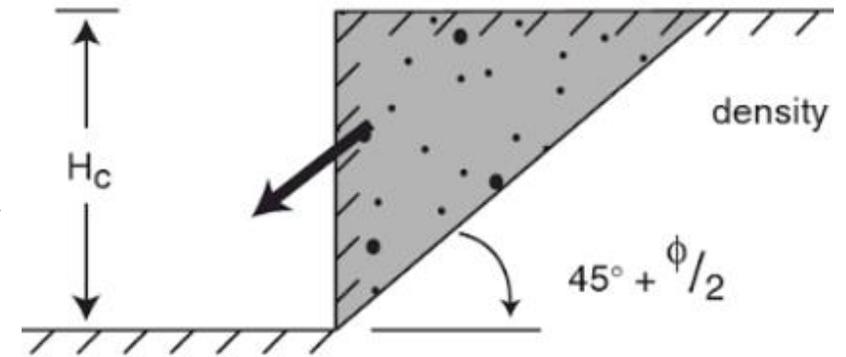
$h \geq 2c/\gamma \cdot \tan(45^\circ + \phi/2)$ lower bound

$h \leq 4c/\gamma \cdot \tan(45^\circ + \phi/2)$ upper bound

'SOIL MECHANICS' THEORIES:

$$2c/\gamma \cdot \tan(45^\circ + \phi/2) \leq H_c \leq 4c/\gamma \cdot \tan(45^\circ + \phi/2)$$

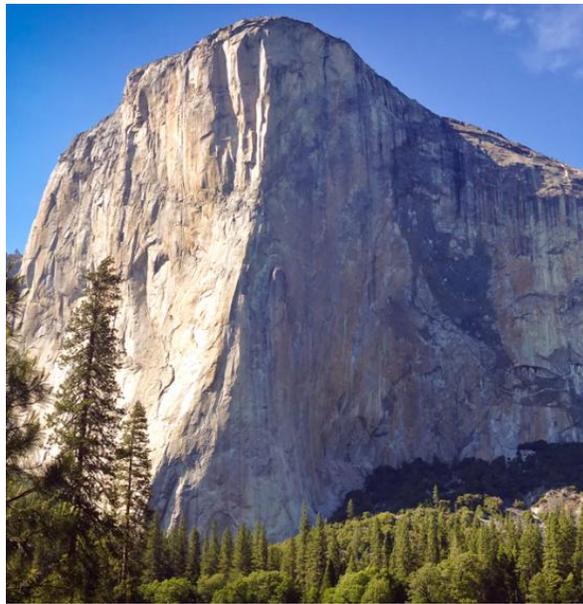
(Mohr-Coulomb, lower- and upper-bound)



(UNFORTUNATELY $\approx 3x$ to $\approx 6x$ IN ERROR WHEN **EXTENSION FAILURE – and NOT SHEAR** FAILURE OF ROCK IS OCCURRING)

TERZAGHI (1962): $H_{crit} = qc/\gamma$ (soil) ($\approx 2.5x$ IN ERROR FOR FAILURE OF STEEP SLOPES IN ROCK)

TERZAGHI SUGGESTED **NEED FOR JOINTING – TO EXPLAIN MUCH LOWER MOUNTAIN WALL HEIGHTS** *than predicted by this / his equation.* (ROCK IS BASICALLY TOO STRONG TO FAIL IN COMPRESSION?)



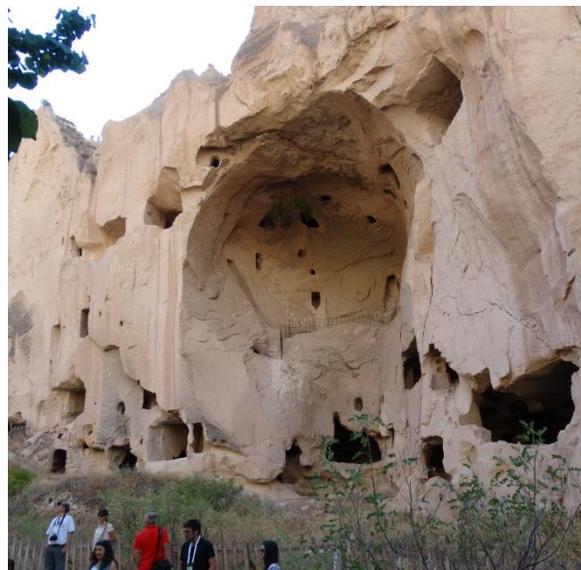
El Capitan, Yosemite, California.
granites, UCS = 100-150MPa.



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



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



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

3. VERTICAL HEIGHT LIMITS OF CLIFFS AND MOUNTAIN WALLS – NEW APPROACH:

$$H_{\text{critical}} \approx 100 \cdot \frac{\sigma_t}{\gamma \nu} \text{ (meters)}$$

(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

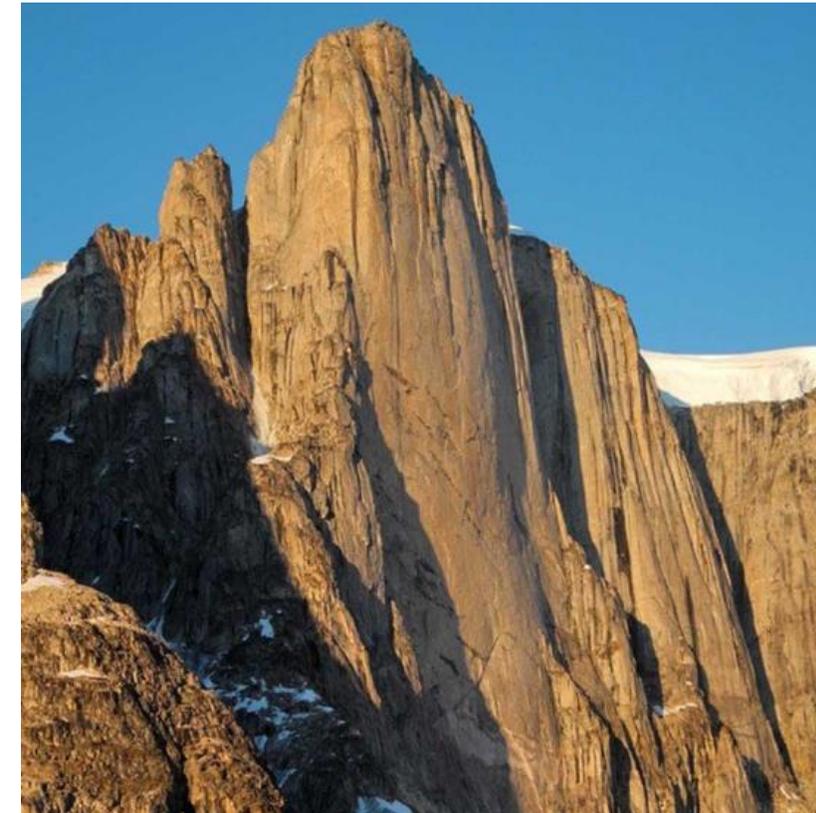


Great Trango Tower,
Karakoram, Pakistan:
1,340m



Mount Thor, Baffin Island,
Canada: **1,250m**

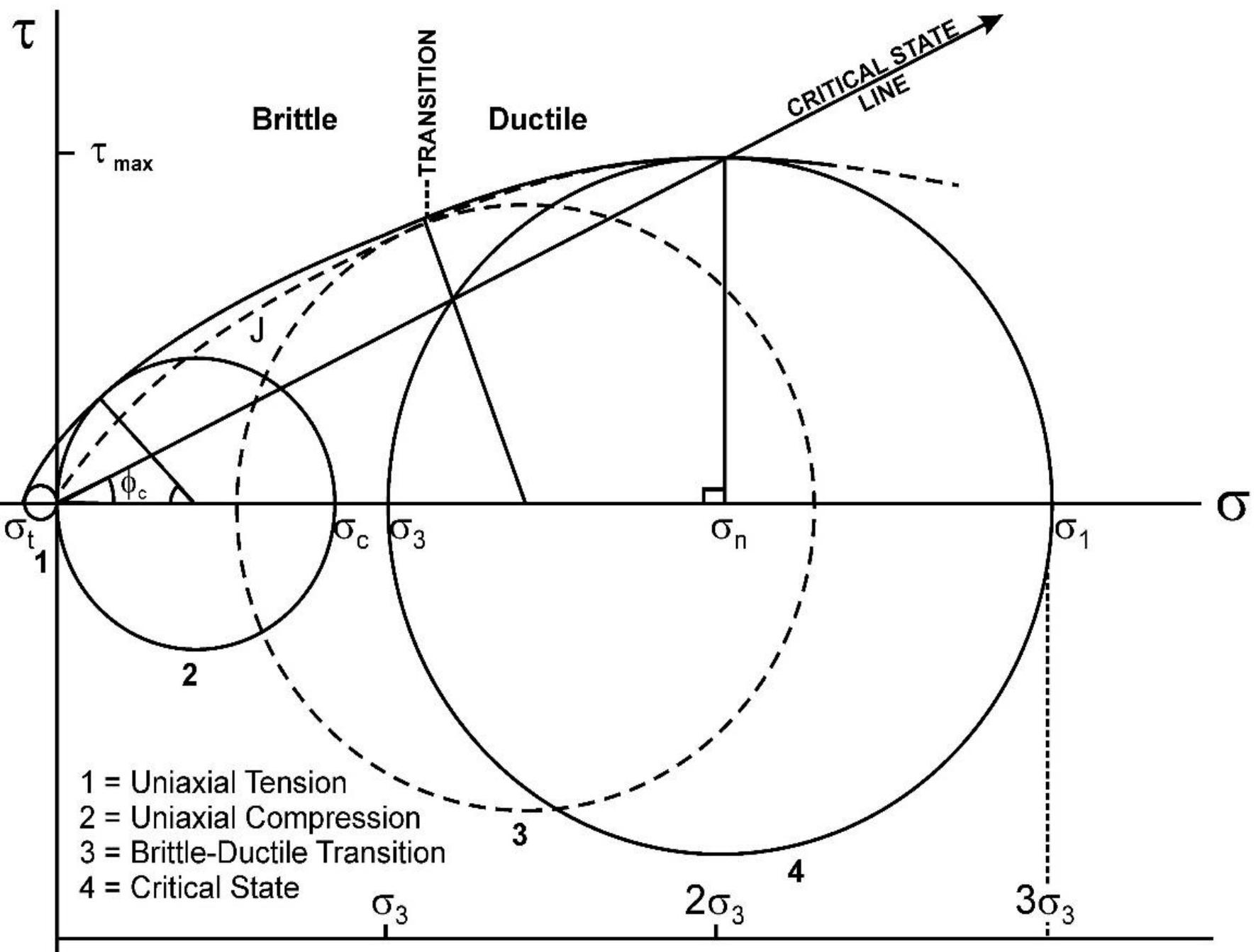
THREE OF THE HIGHEST 'VERTICAL' MOUNTAIN WALLS in THE WORLD



Mirror Wall, Greenland:
1,200m

ESTIMATING THE
COHESIVE STRENGTH
OF INTACT ROCK

(lower-bound)



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

(lower-bound estimate)

Assumes linear envelope between tensile and compression circles.

Actual cohesion is higher due to curvature.

(Barton, 1976)

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

1. Sandstone

$$\sigma_c = 75\text{MPa},$$

$$\sigma_t = 5\text{MPa}$$

$$c = \frac{1}{2}(75 \times 5)^{1/2} =$$

$$9.7\text{MPa}$$

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

2. Granite

$$\sigma_c = 150\text{MPa},$$

$$\sigma_t = 10\text{MPa}$$

$$c = \frac{1}{2}(150 \times 10)^{1/2} =$$

$$19.4\text{MPa}$$

Sandstone 'valley wall' ($H_c = 2c/\gamma \times \tan(45^\circ + \phi/2)$):

$$H_c = 2 \times 9.7 \times 1000/25 \times \tan(45^\circ + 30.5^\circ) = \underline{3,001\text{m}} !$$

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

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

$$H_c = 2 \times 19.4 \times 1000/27.5 \times \tan(45^\circ + 30.5^\circ) = \underline{5,456\text{m}} !$$

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

BY COMPARISON, USING EXTENSION STRAIN THEORY:

$$\text{Sandstone: } 100 \cdot \sigma_t / \gamma v = 100 \cdot 5 / 2.5 \times 0.25 = \underline{800\text{m}}$$

$$\text{Granite: } 100 \cdot \sigma_t / \gamma v = 100 \cdot 10 / 2.75 \times 0.25 = \underline{1,456\text{m}}$$

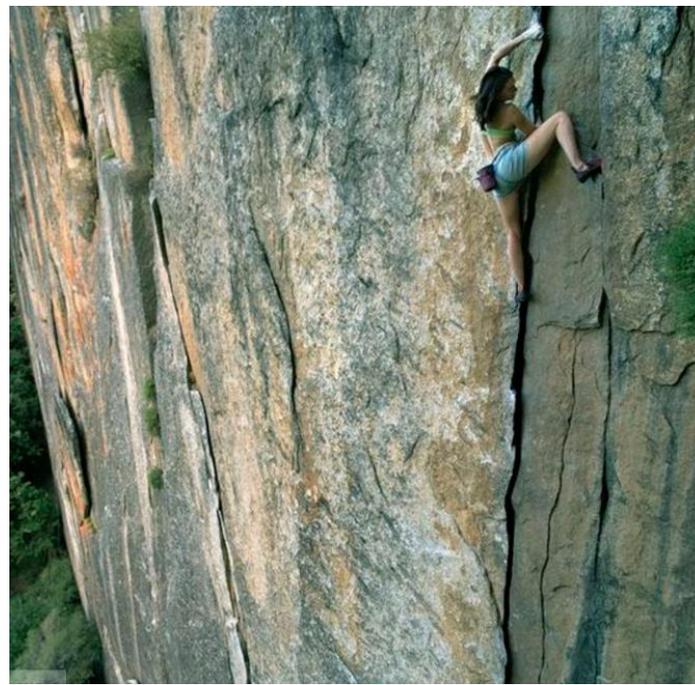
Lower-bound estimate M-C: $h = 2c/\gamma \tan(45^\circ + \varphi/2)$

$\sigma_t = 0.2 \text{ MPa}, \sigma_c = 2 \text{ MPa}$	$\gamma = 2.0 \text{ t/m}^3$	$h = 103\text{m}$
$\sigma_t = 0.5 \text{ MPa}, \sigma_c = 5 \text{ MPa}$	$\gamma = 2.0 \text{ t/m}^3$	$h = 258\text{m}$
$\sigma_t = 5 \text{ MPa}, \sigma_c = 50 \text{ MPa}$	$\gamma = 2.5 \text{ t/m}^3$	$h = 2,067\text{m}$
$\sigma_t = 10 \text{ MPa}, \sigma_c = 100 \text{ MPa}$	$\gamma = 2.8 \text{ t/m}^3$	$h = 3,690\text{m}$

Extension strain based: $H_c = \sigma_t/\gamma v$

$\sigma_t = 0.2 \text{ MPa}, \sigma_c = 2 \text{ MPa}$	$v = 0.2$	$H_c = 50\text{m}$
$\sigma_t = 0.5 \text{ MPa}, \sigma_c = 5 \text{ MPa}$	$v = 0.2$	$H_c = 125\text{m}$
$\sigma_t = 5 \text{ MPa}, \sigma_c = 50 \text{ MPa}$	$v = 0.25$	$H_c = 800\text{m}$
$\sigma_t = 10 \text{ MPa}, \sigma_c = 100\text{MPa}$	$v = 0.25$	$H_c = 1,430\text{m}$

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



SHEETING JOINTS (AND ASSOCIATED CRACKS)

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

Free-solo rock-climbing aces:

Steph Davis (see book)

Alex Honnold (see book)



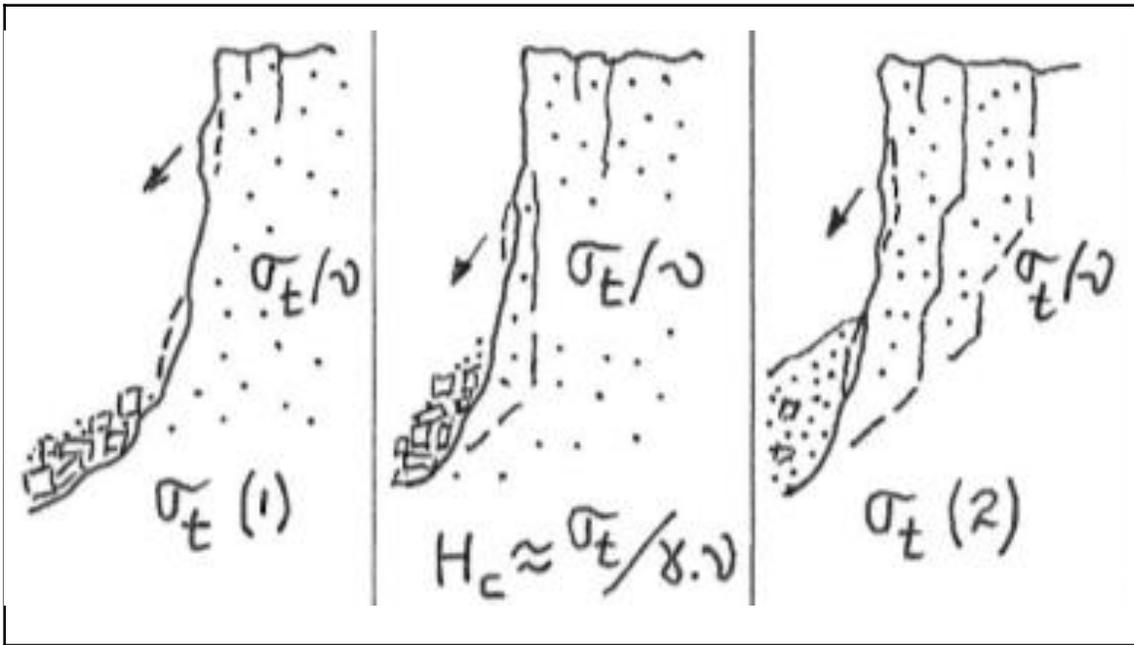
Honnold and Caldwell,
June 6th 2018: 'The Nose'

1 hour 58 mins 7 secs
(3,000 feet, 914m)

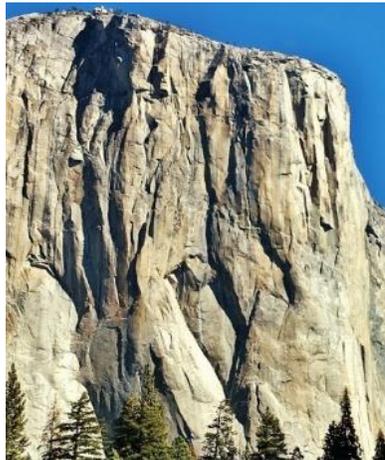


EXTENSION FAILURES CAUSE SHEETING FRACTURES, AND LIMIT ULTIMATE WALL HEIGHTS

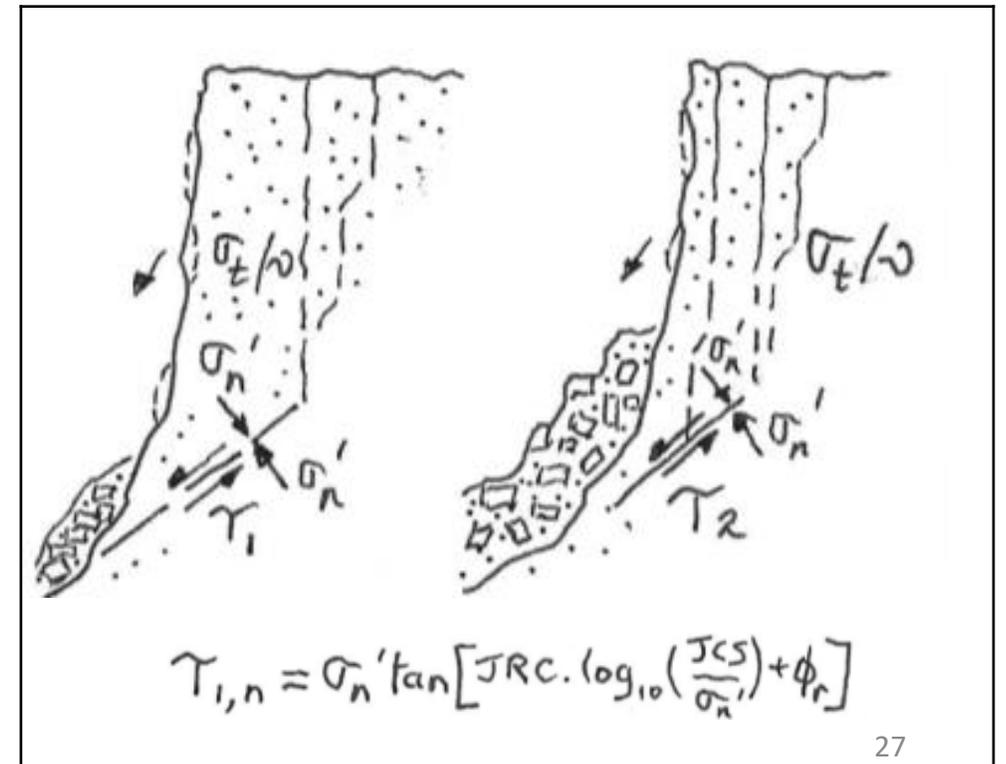
(NOTE! σ_t REDUCES, OVER GEOMORPHOLOGICAL TIME-SCALES)



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



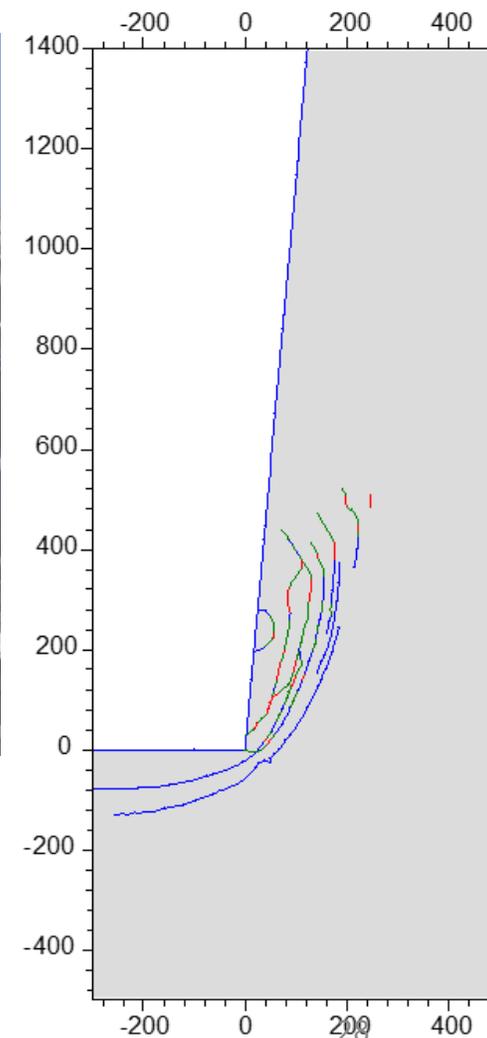
El Capitan, CA. and Holtanna, Antarctic.





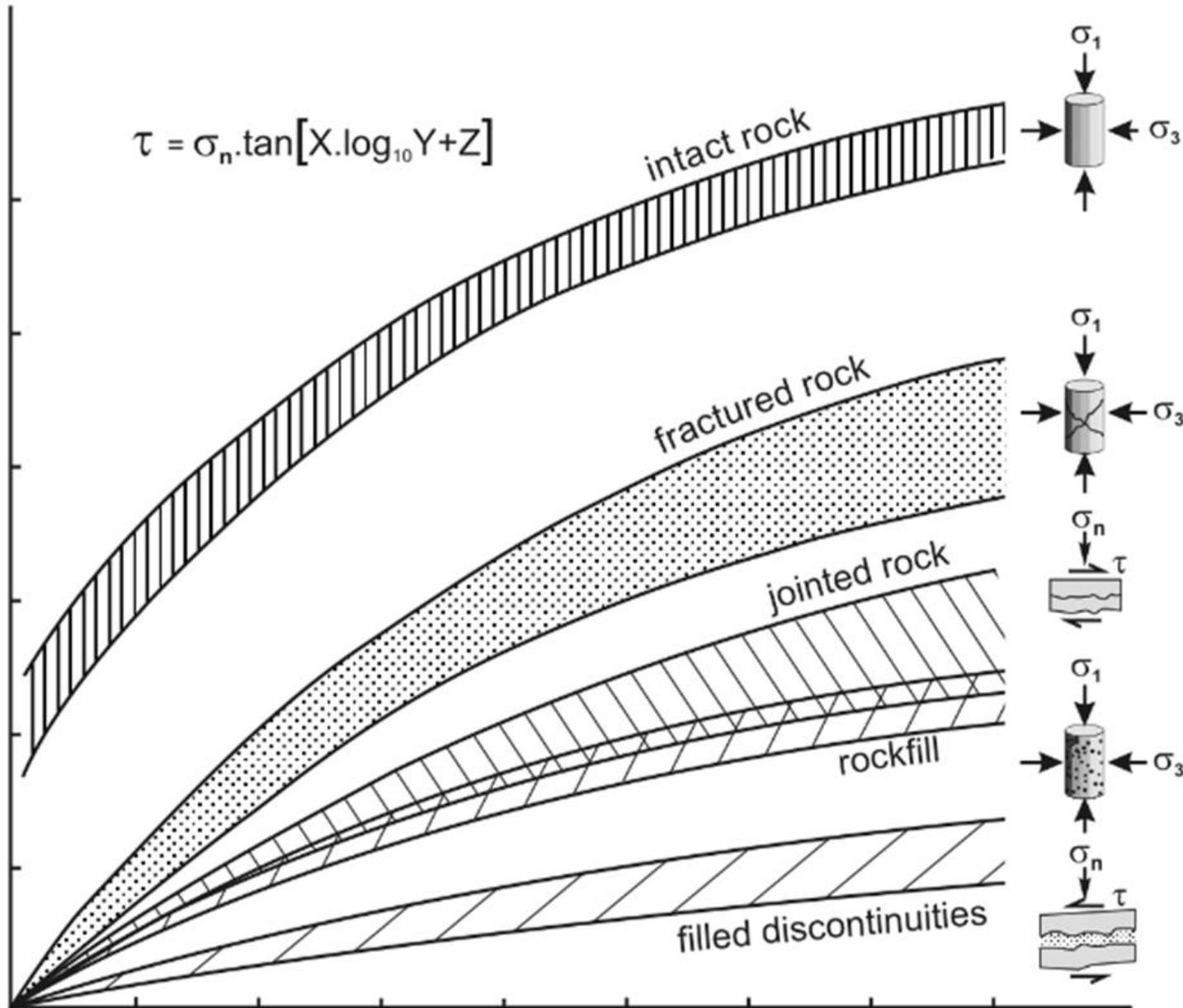
El Capitan, Yosemite.

What causes the shadows.....?



Extension fracturing, shear, and sheeting joints on El Capitan: evidence of a (slow) 'active' process. (FRACOD example, right)

4. COMPLICATIONS FROM
MULTI-COMPONENT
FAILURE-MODES
(slopes + tunnels)



FOUR COMPONENTS:

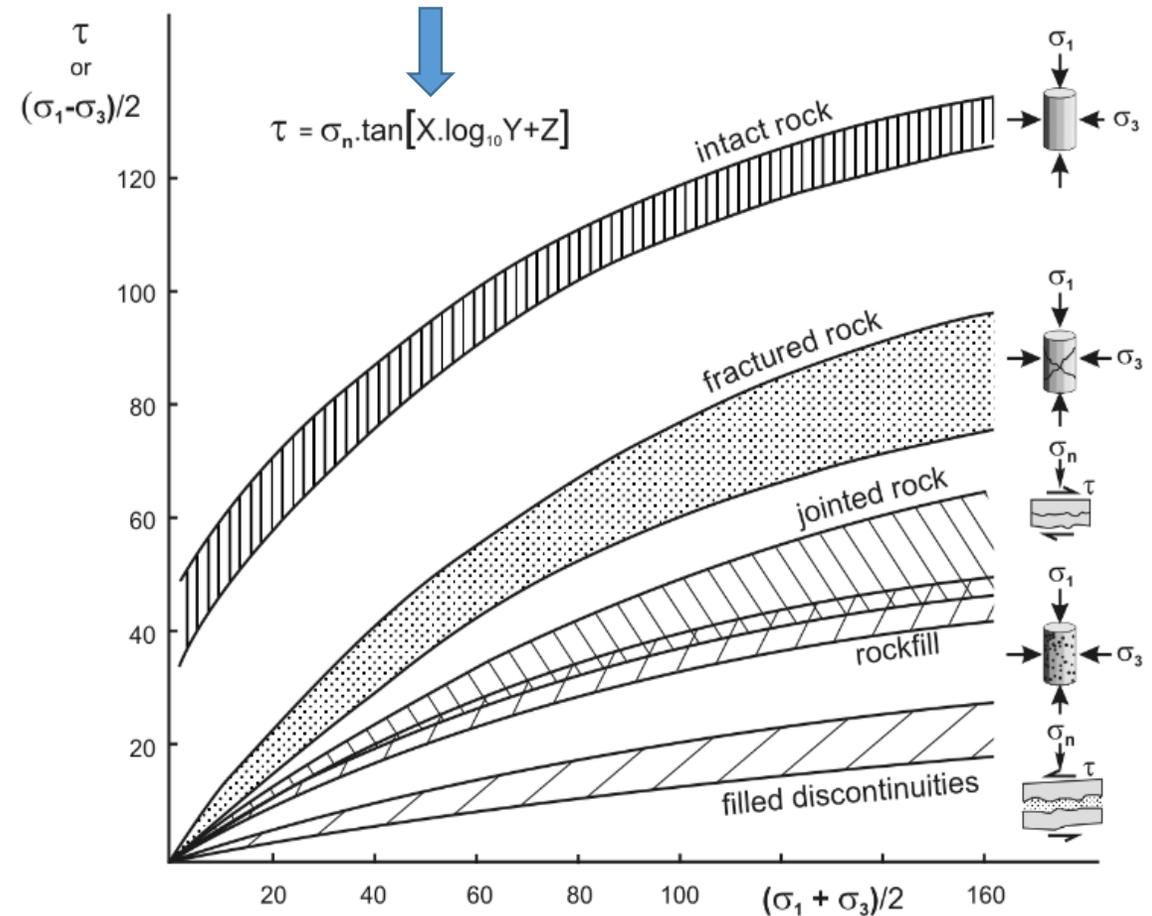
process-dependent shear failure of e.g. High open-pit slopes.

1. FAILURE OF INTACT 'BRIDGES'
2. SHEARING (OR NOT) ON THESE NEW, FRESH, ROUGH SURFACES
3. MOBILIZATION (OR NOT) ALONG THE ALREADY ESTABLISHED SHEAR PLANES INVOLVING JOINTS
4. FINALLY THE LIMITED STRENGTH OF ANY CLAY-FILLED DISCONTINUITIES OR FAULTS, WITH LOWEST SHEAR STIFFNESS

(Barton, 1999, 2013).

WHAT IS HELPING TO PREVENT SUDDEN COLLAPSES ?

('X' \approx 50, *then 20*, then JRC, then Jr/Ja) Barton, 1999

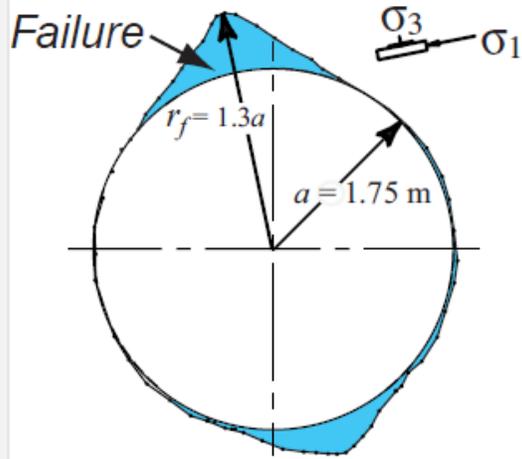


Bingham Pit: No casualties. Monitored. Progressive failure....i.e. ' $\tau = c$ *then* $\sigma_n \tan \phi$ '



A combination of low-strength rock and a sufficiently big stress range automatically leads to the need for non-linear shear resistance concepts.....

and *not linear Mohr-Coulomb* (for intact and jointed rock!)



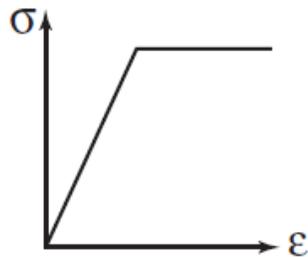
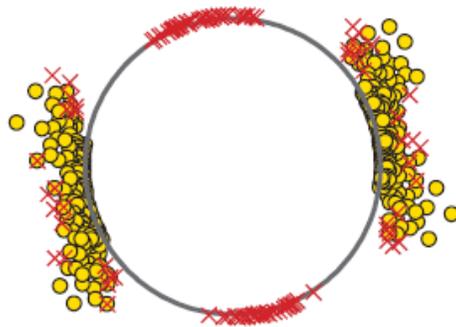
CONVENTIONAL continuum modelling methods for (highly stressed) TUNNELS

.....

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



Predicted

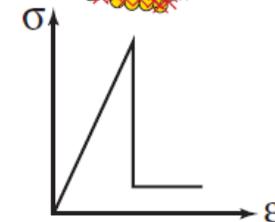
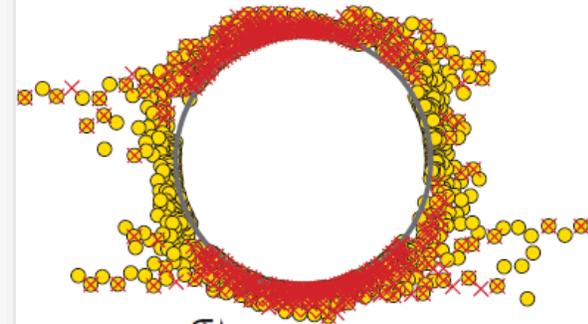


Elastic-Plastic

(Hajiabdolmajid, Martin and Kaiser, 2000)

NEED TO SEPARATE 'c' and 'phi'
degrade c, mobilize phi

Predicted



Elastic-Brittle

x Shear failure o Tensile failure

JOB TITLE :

FLAC (Version 3.30)

LEGEND

6/02/1999 16:04

step 4850

-3.106E+00 <x< 3.106E+00

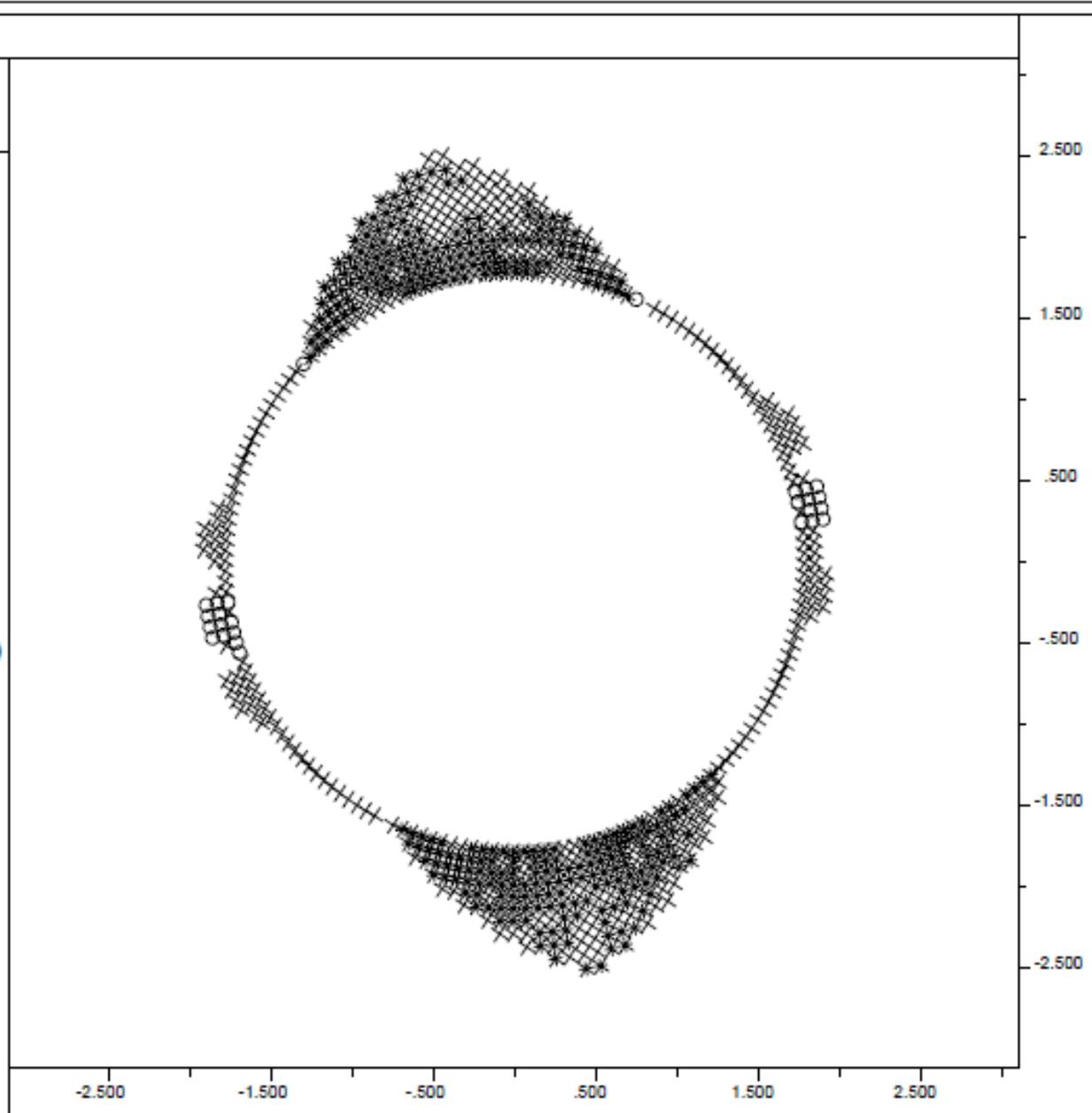
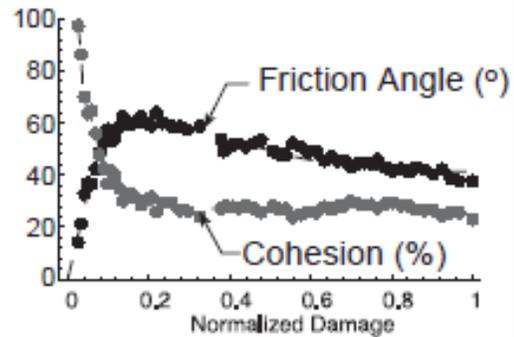
-3.106E+00 <y< 3.106E+00

Plasticity Indicator

* at yield in shear or vol.

X elastic, at yield in past

o at yield in tension



**Degrade cohesion,
mobilize friction:
excellent match.**

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

CAN WE GET ESTIMATES OF 'c' AND 'φ' FROM THE
Q-PARAMETERS?

YES – semi-empirical due to bolt and shotcrete
quantities – but estimates ONLY

(JUST LIKE THE *ESTIMATES-ONLY* FROM GSI, H-B!)

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

$$Q_c = RQD/J_n \times J_r/J_a \times J_w / SRF \times \sigma_c / 100$$

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

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

Cut Q_c into two halves \rightarrow 'c' and ' φ '

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

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

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



DO WE THINK ROCK MASSES KNOW ABOUT H-B (or Q-based) EQUATIONS? *WHAT WOULD BE THEIR OPINION*

Expression	Origin
$" \varphi " \approx \tan^{-1} \left(\frac{J_r}{J_a} \times \frac{J_w}{1} \right) \quad \leftarrow$	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
$" c " \approx \left(\frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100} \right) \quad \leftarrow$	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

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

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

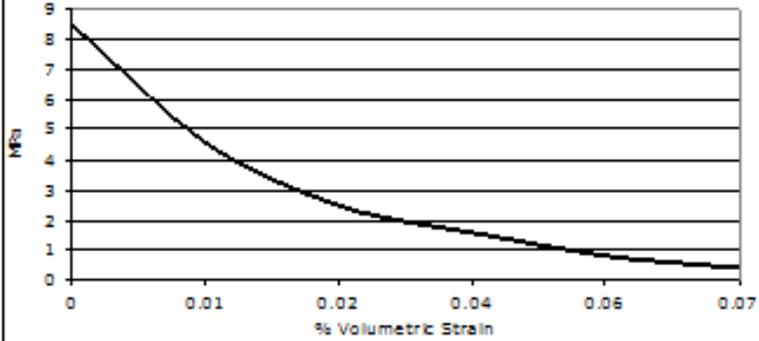
$$a = \frac{1}{2} + \frac{1}{6} \cdot \left(\exp^{-GSI/15} - \exp^{-20/3} \right)$$

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

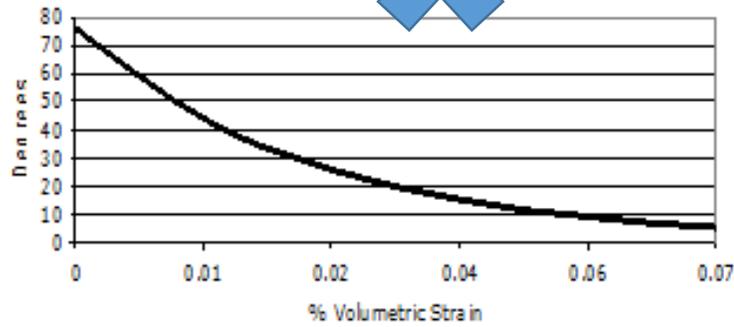
$$E_m \text{ (MPa)} = 10^5 \left(\frac{1 - D/2}{1 + e^{\left[\frac{75 + 25D - GSI}{11} \right]}} \right)$$

C + Tan phi approach

Cohesion Component

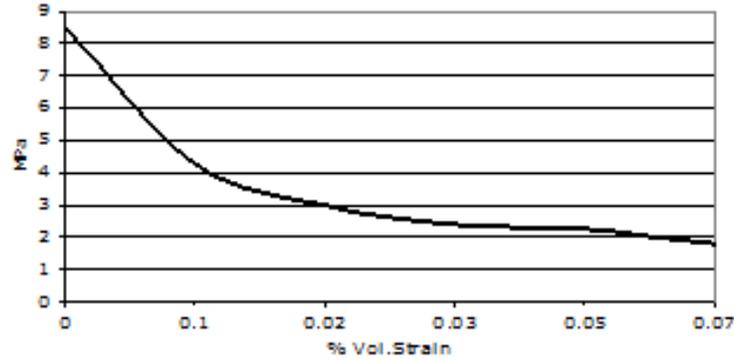


Frictional Component

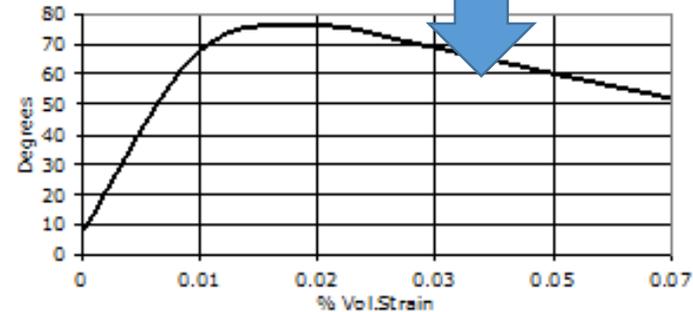


C then Tan phi approach

Cohesion Component



Frictional Component

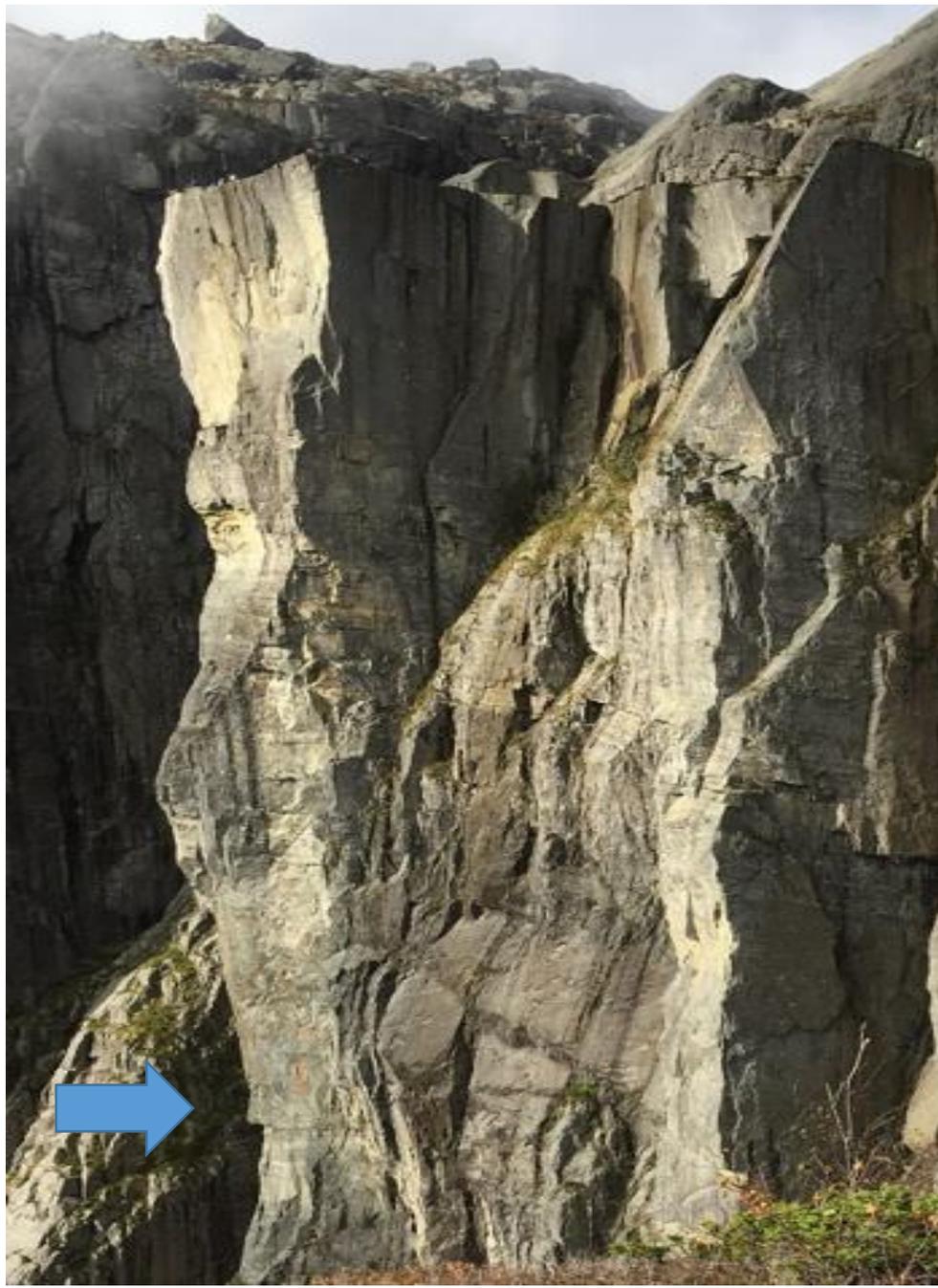


'c then $\sigma_n \tan \phi$ '

(as in Barton and Pandey, 2011)

5. Which side to stand at Norway's Prekestolen?





Back-wall
shear(?)

.....
tensile
opening
(?)

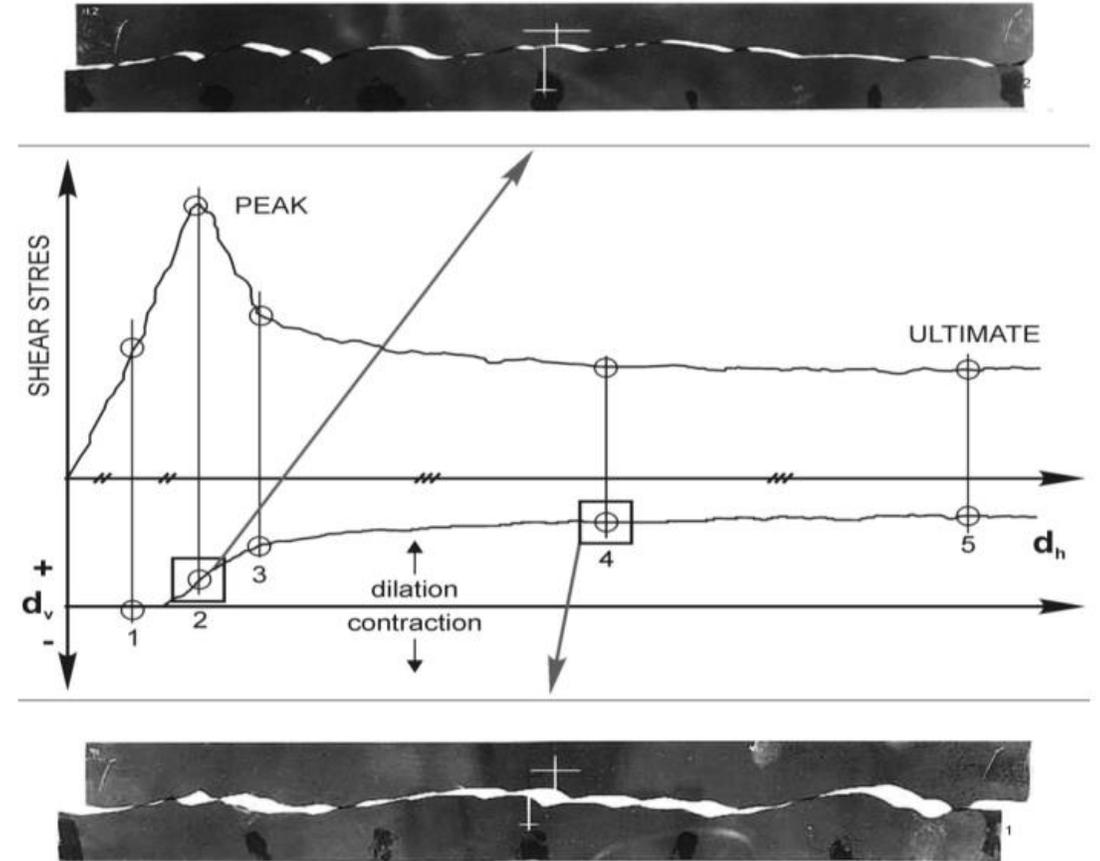
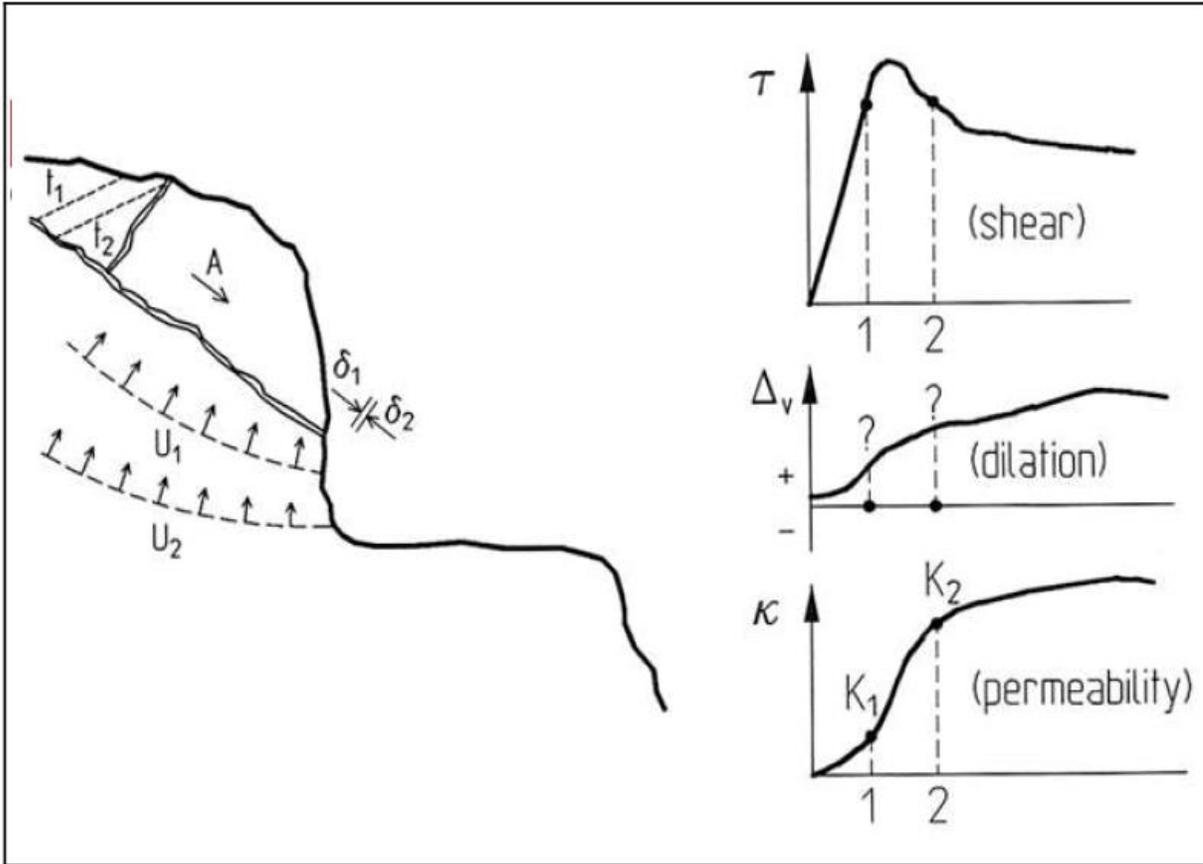
.....
triggering
by $\sigma_t/\gamma v$?

(Photos from
Katrina Mo,
NTNU, M.Sc.)



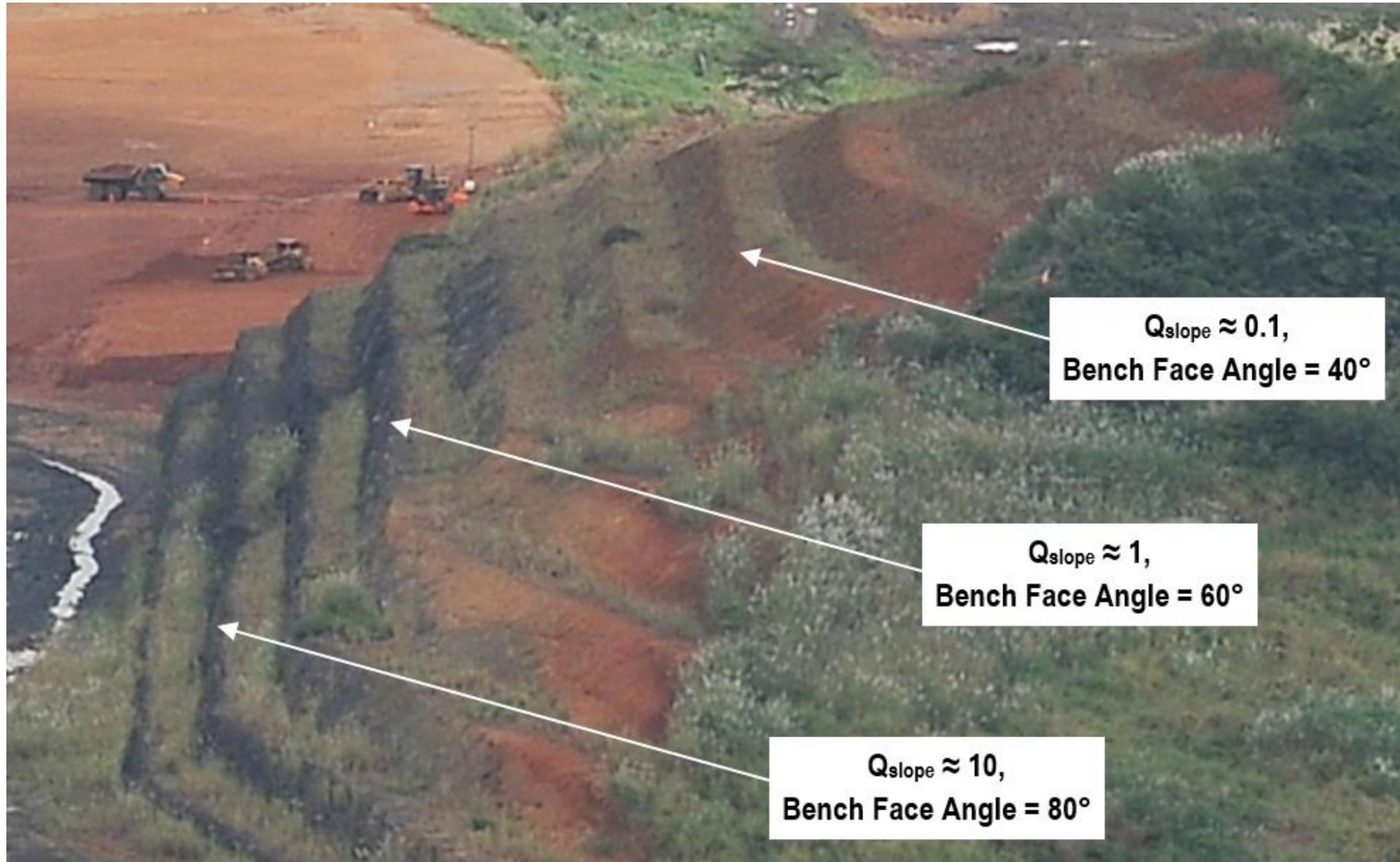
Apropos
crack apertures,
friction?
dilation?
normal stress?

**Slope angle estimates
for slopes
in jointed rock
Q-slope**

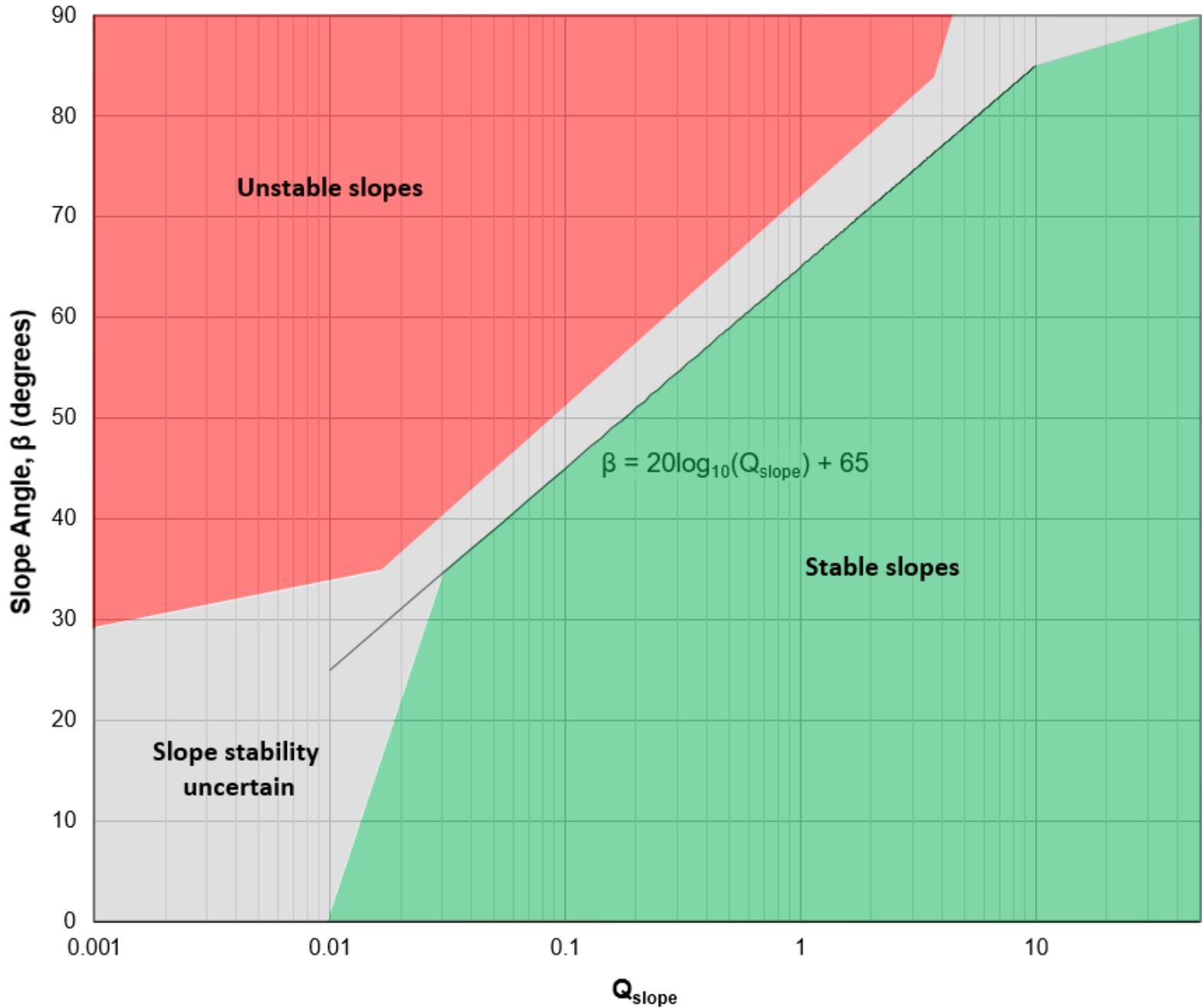


The stability of rock slopes in jointed rock obviously involves coupled (M-H) behaviour due to the effect of rain storms. Joint permeability may increase (shear-induced dilation), and later decrease (clay-and-silt filling).....[Shear tests from Barton, 1968.](#)

In a nutshell: Q-slope involves support-free slopes

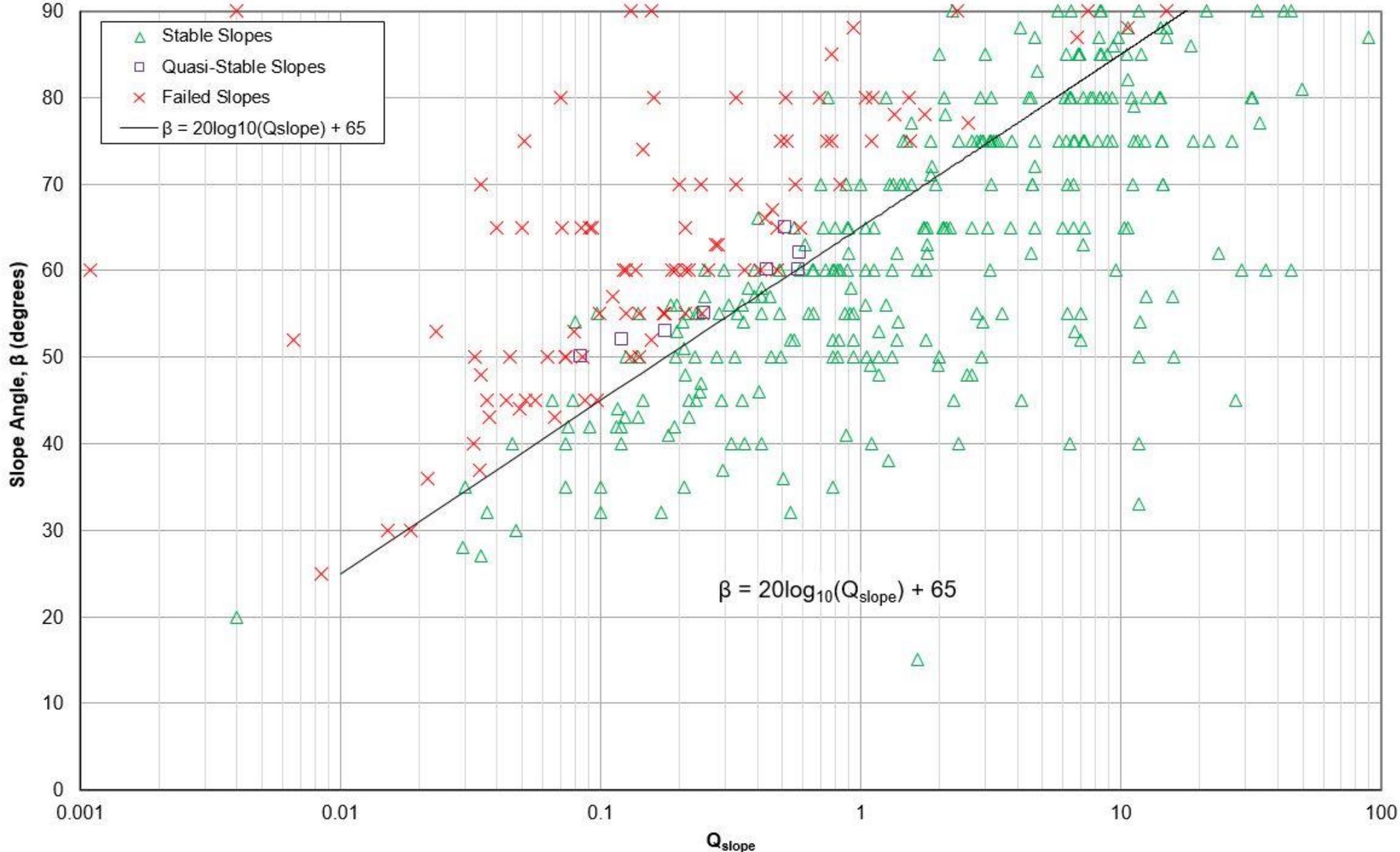


Q_{slope} Stability Chart



- The chart for collecting Q_{slope} case records.
- Bar and Barton, 2017, 2018

Q_{slope} Case Study Data



$$Q_{slope} = \frac{RQD}{J_n} \times \left(\frac{J_r}{J_a} \right)_o \times \frac{J_{wice}}{SRF_{slope}}$$

RQD/J_n are unchanged

(J_r/J_n)_o are unchanged, BUT have an orientation, and 'wedge' adjustment

J_w (now J_{wice}) has a new structure for slopes, including ice-effects and *tropical rainfall-effects*

SRF_{slope} has new categories tailored for slopes

Example 3: wedge failure. Bar and Barton, 2017.

RQD (%)	J _n	J _r		J _a		0-factor		J _{wice}	SRFa	SRFb	SRFc
		SET A	SET B	SET A	SET B	SET A	SET B				
50-60	9	1	4	3	4	0.5	0.9	1	2.5	2.5	1

$$\begin{aligned}
 Q_{slope} &= \frac{RQD}{J_n} \times \left(\frac{J_r}{J_a} \right)_0 \times \frac{J_{wice}}{SRF_{slope}} \\
 &= \frac{55}{9} \times \left(\frac{1}{4} \times 0.5 \right) \left(\frac{3}{4} \times 0.9 \right) \times \frac{1}{2.5} \\
 &= \mathbf{0.206}
 \end{aligned}$$

$$\begin{aligned}
 \beta &= 20 \log_{10} Q_{slope} + 65 \\
 &= 20 \log_{10} (0.206) + 65 \\
 &= \mathbf{51^\circ} \quad (\text{steepest slope angle})
 \end{aligned}$$

15° shallower than excavated and consistent with kinematic analysis

P-Wave Velocity and Q-value

- Motorway project in Panama (Barton, 2007)
- Forest clearing for roadway cuttings
- Seismic refraction used alongside core logging.
- Higher V_p correspond to higher Q-values
- Steeper bench-slope designs as ground conditions improve with depth

V_p (km/s)	Approx. Q	σ_c (MPa)	Bench Angle	Bench Height	Bench Width
0.5-1.5	0.05-0.2	2-5	<40°	4m	4m
2.8	0.8	25	50-60°	5m	4m
3.2	1.0	50	70-80°	5m	4m
4.4	8.0	100	80-90°	6m	4m

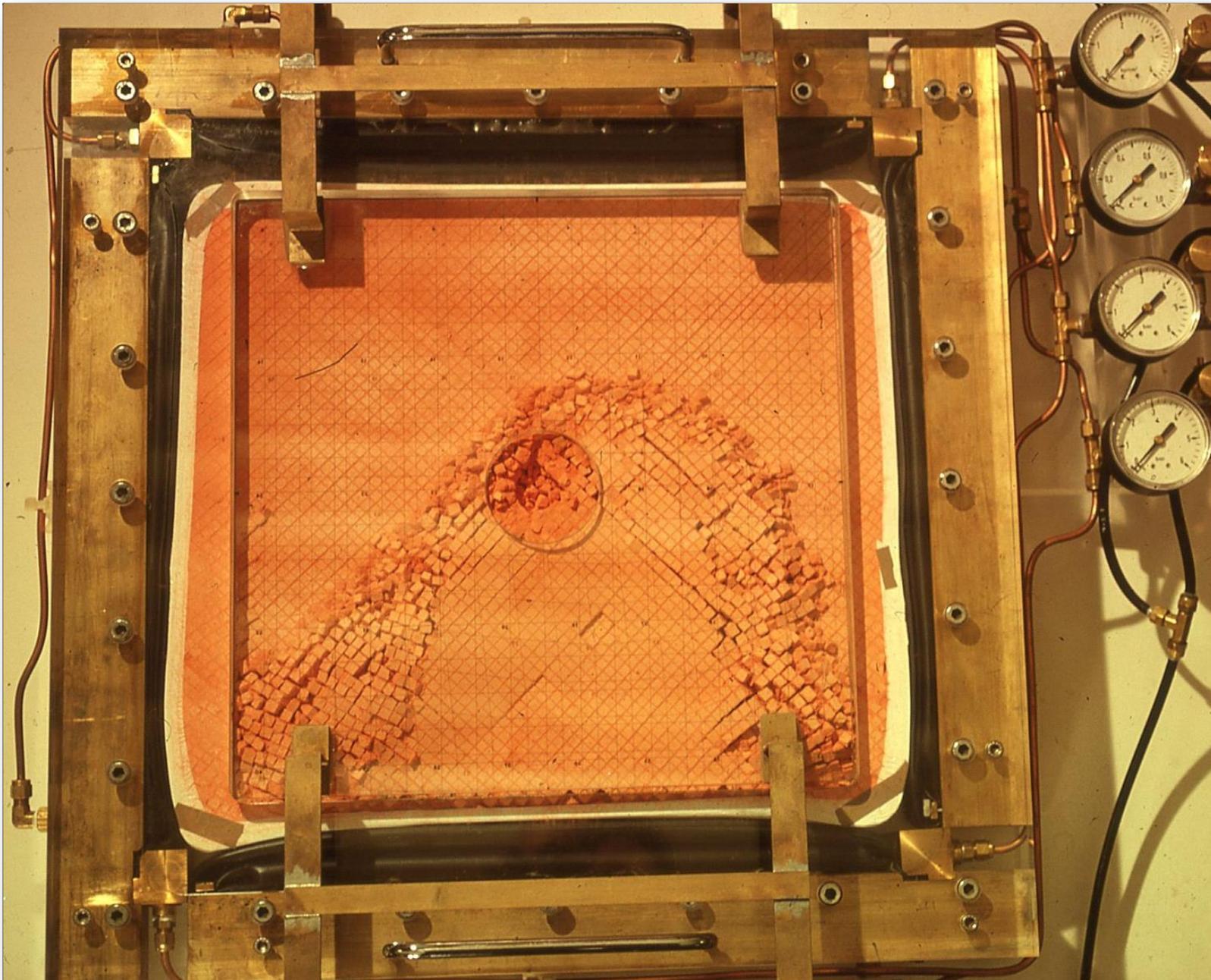




Natural structures (joint sets, bedding) may set the slope angle.

What about level of safety?

ROTATIONAL MODES WHEN SMALL BLOCK SIZES?



BIAXIAL LOADING

Scale-effect
investigations

250, 1000, or
4000 blocks.

“Always” got
rotational
failures with
small blocks!

APPROPOS: *BLOCK DEBRIS and ROCK SLIDES* (Front cover: eds. J.Clague, D. Stead)



FRANK SLIDE (Wikipedia)



ROCK-AVALANCHE TRAVEL DISTANCE VARIABILITY

SAY **0.5 to 1 km** TRAVEL DISTANCE EXPECTED
WITH 'AIR-CUSHION' (Chinese research) **2km**

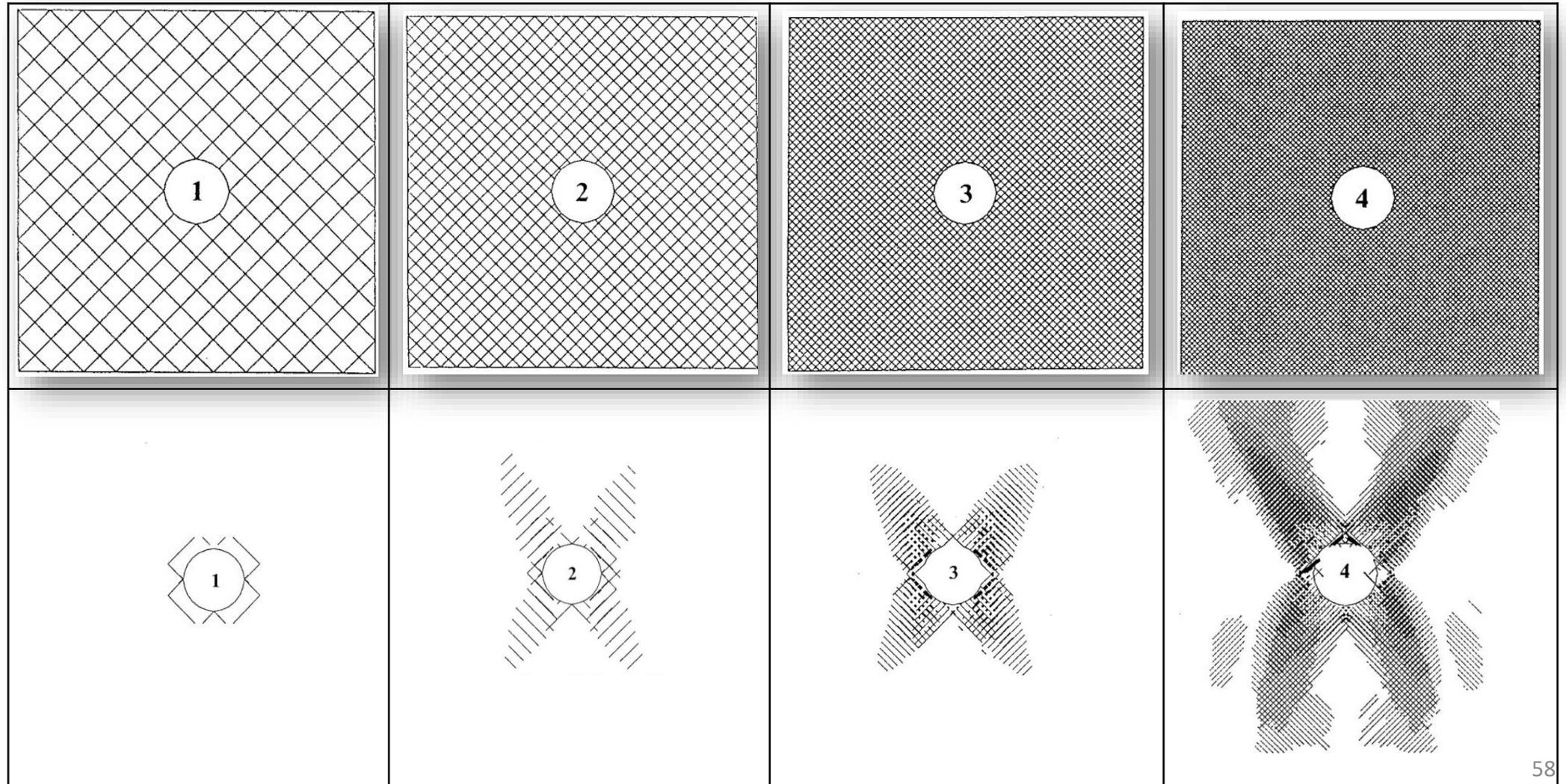
- REALITY (sometimes) is > **20 km**
- **SLIDE MASSES TOO HOT FOR RESCUE PARTIES to STAND ON**

WHY? Rotational friction, block crushing, extreme heating, ground water converted to steam..... 'steam-cushioned slides'

($V_2/V_1 = 1,400:1$)

SUCCESSIVE HALVING OF THE BLOCK SIZE – HAS DRAMATIC ROTATIONAL (degree-of-freedom) EFFECTS AROUND TUNNELS..... WITH *UDEC-MC*, ALSO *UDEC-BB*
(helps to explain *the drama of fault zones: worse with clay and water*)

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



**6. WHY ARE THE HIGHEST (15)
MOUNTAINS IN THE WORLD
'LIMITED' TO 8 - 9km?**



Mount Everest

8,864m

(from Wikipedia photo)

MISUSE OF 'TERZAGHI'
FORMULA: GIVES

$$H_c = 100 \sigma_c / \gamma$$

$$\text{e.g. } 100 \times 250 / 2.8 = 8.9\text{km?}$$

***No! HAS TO BE CONFINED
STRENGTH AT 9 KM DEPTH
AND THIS IS MUCH TOO HIGH!***

**CORRECT LOGIC SUGGESTS A
LOWER (CRITICAL STATE)
SHEAR STRENGTH LIMIT.**

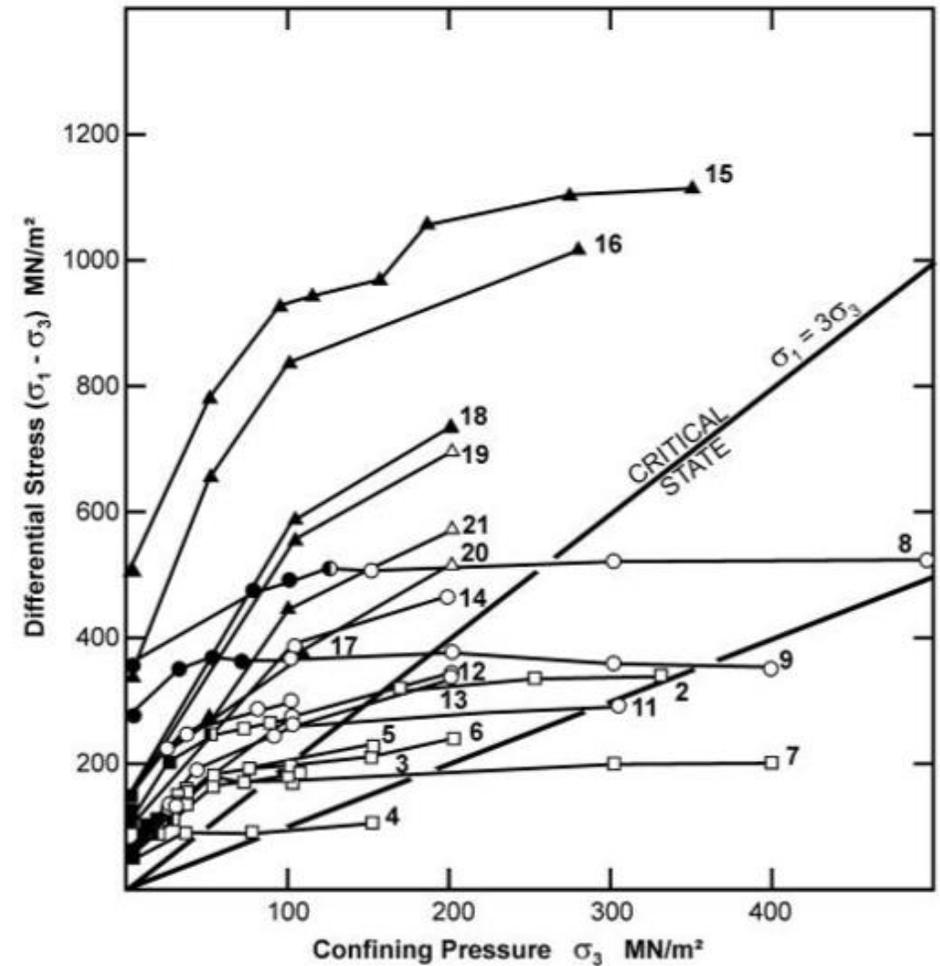
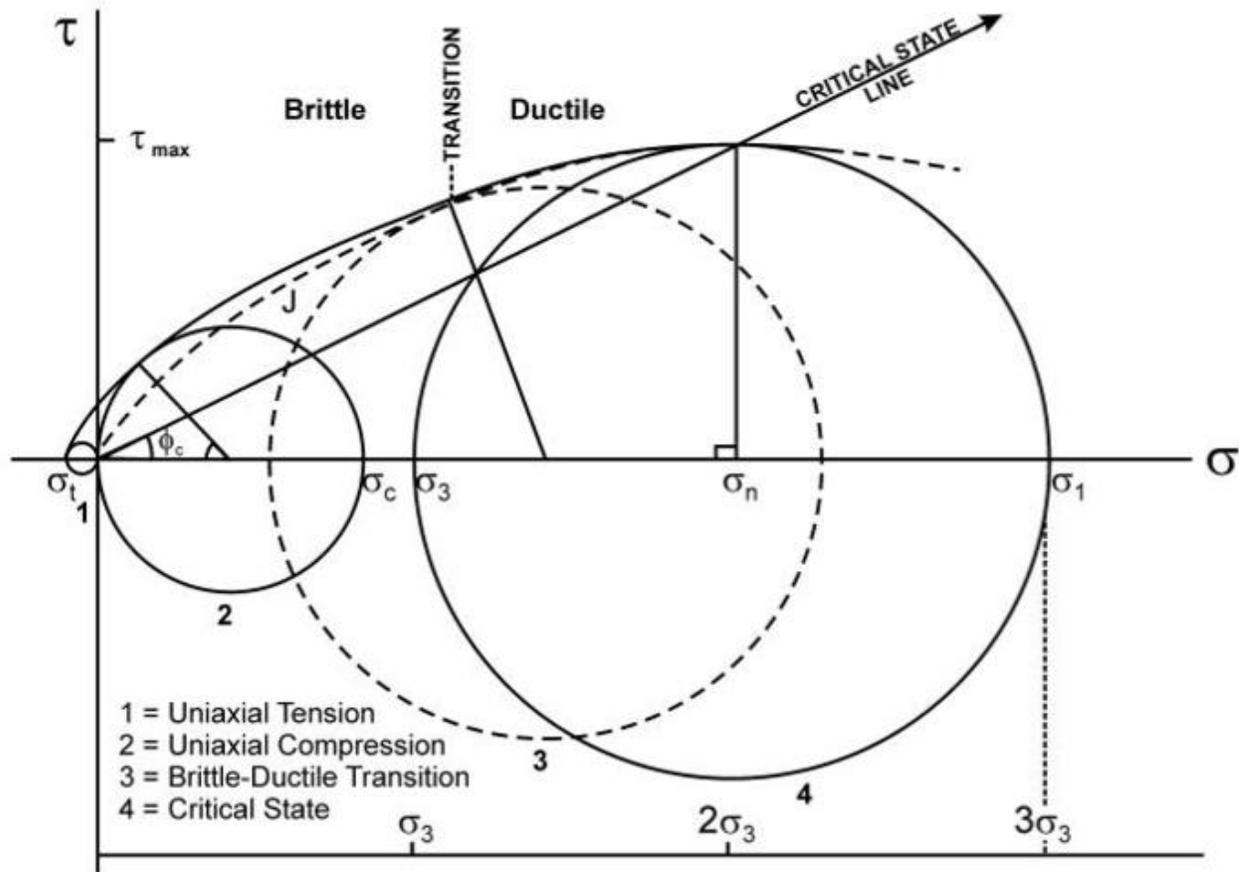
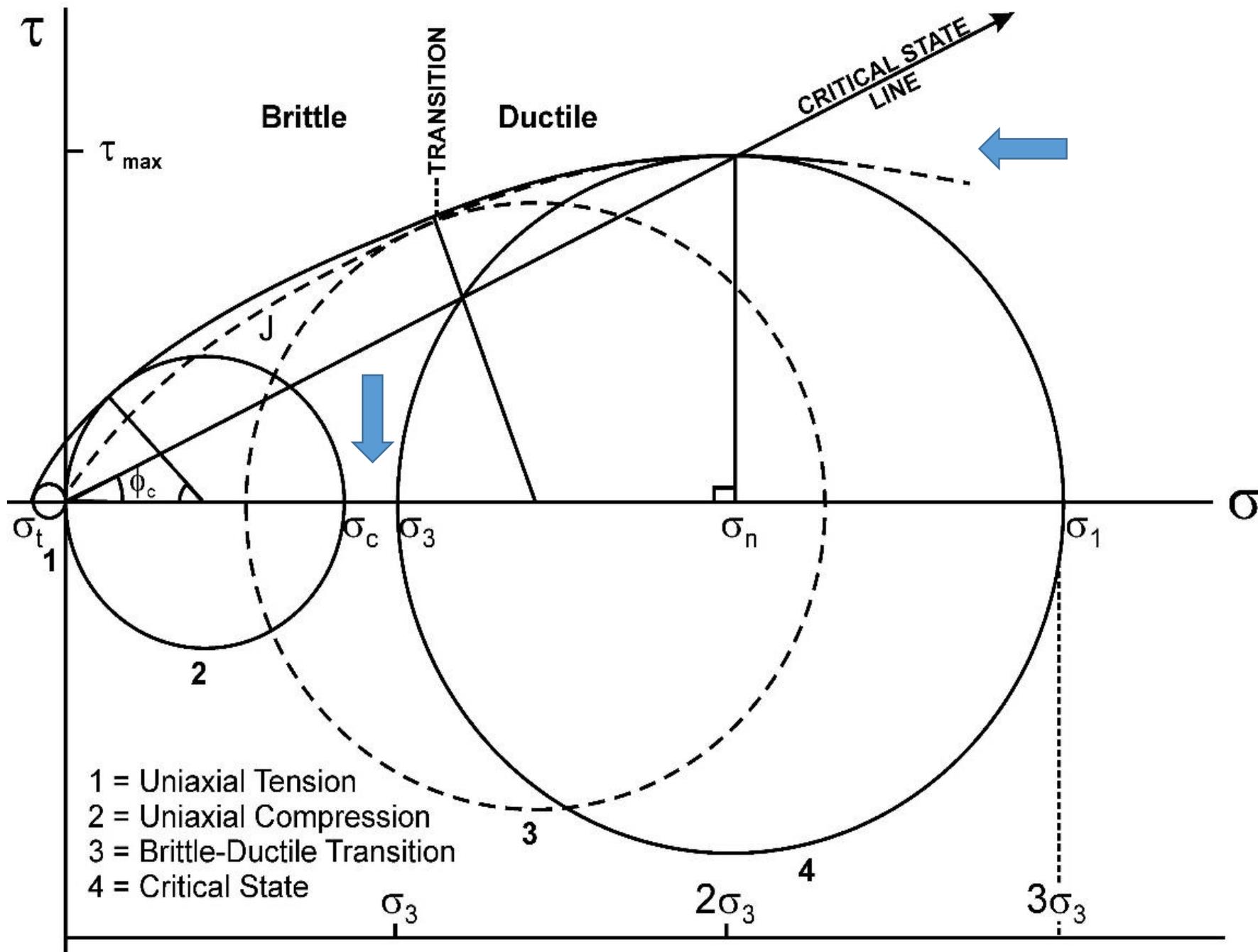


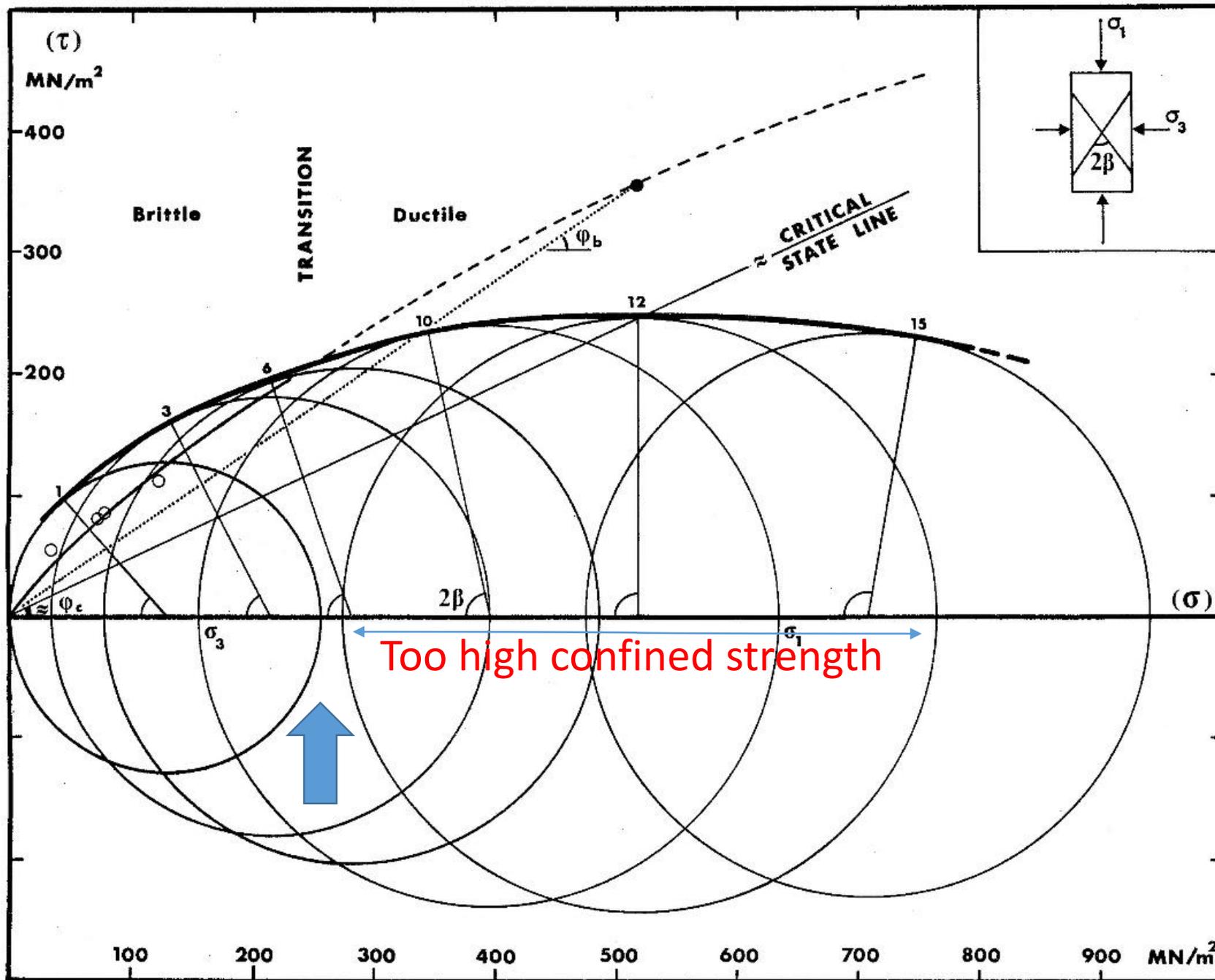
Figure 1. The suggested non-linearity and critical state suggestion of [Barton \(1976\)](#) which was based on sets of high-pressure triaxial test data as presented by [Mogi \(1966\)](#). One set is shown here.



THE MAXIMUM POSSIBLE SHEAR STRENGTH AT THE CRITICAL STATE

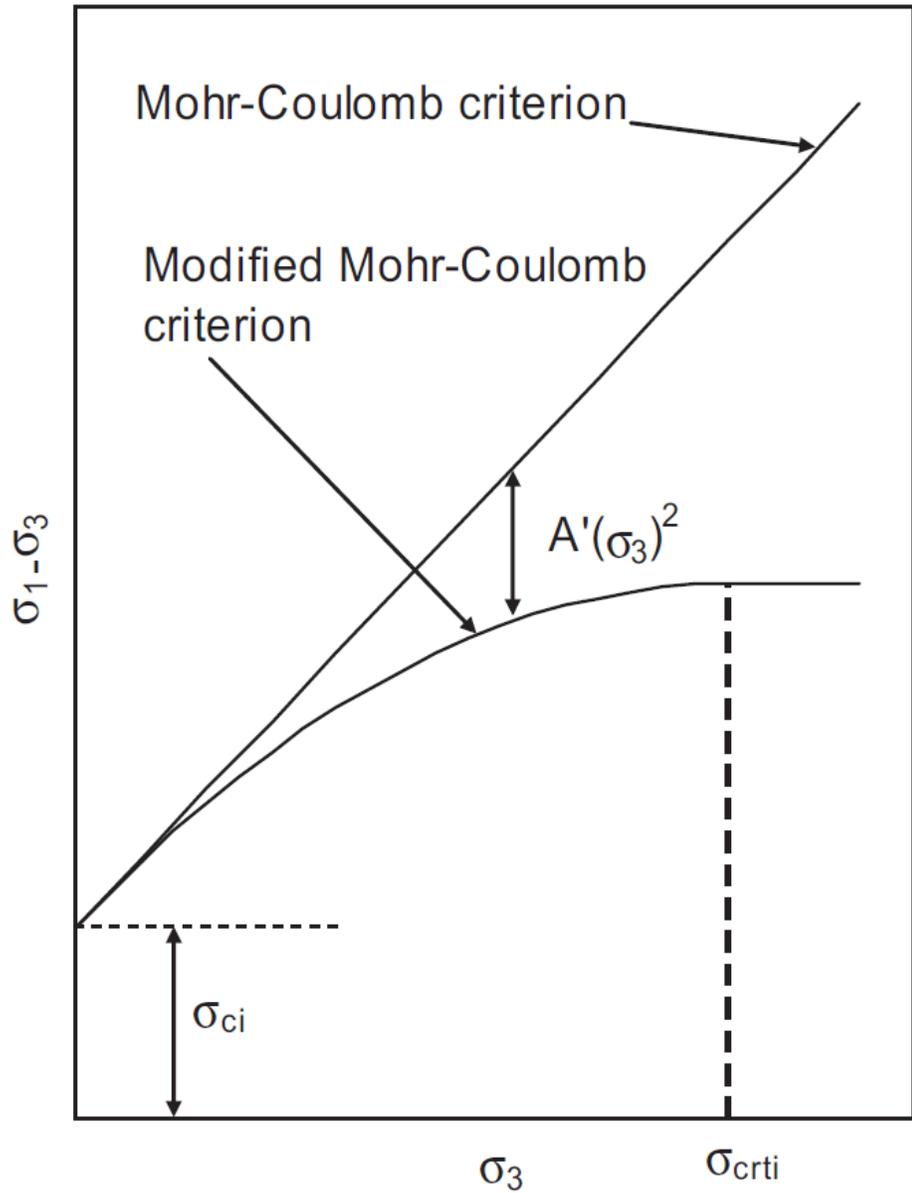
IS OF SIMILAR NUMERICAL MAGNITUDE TO UCS, SAY 200 MPa FOR A STRONG ROCK LIKE GRANITE.

Barton, 1976,
Singh et al. 2011

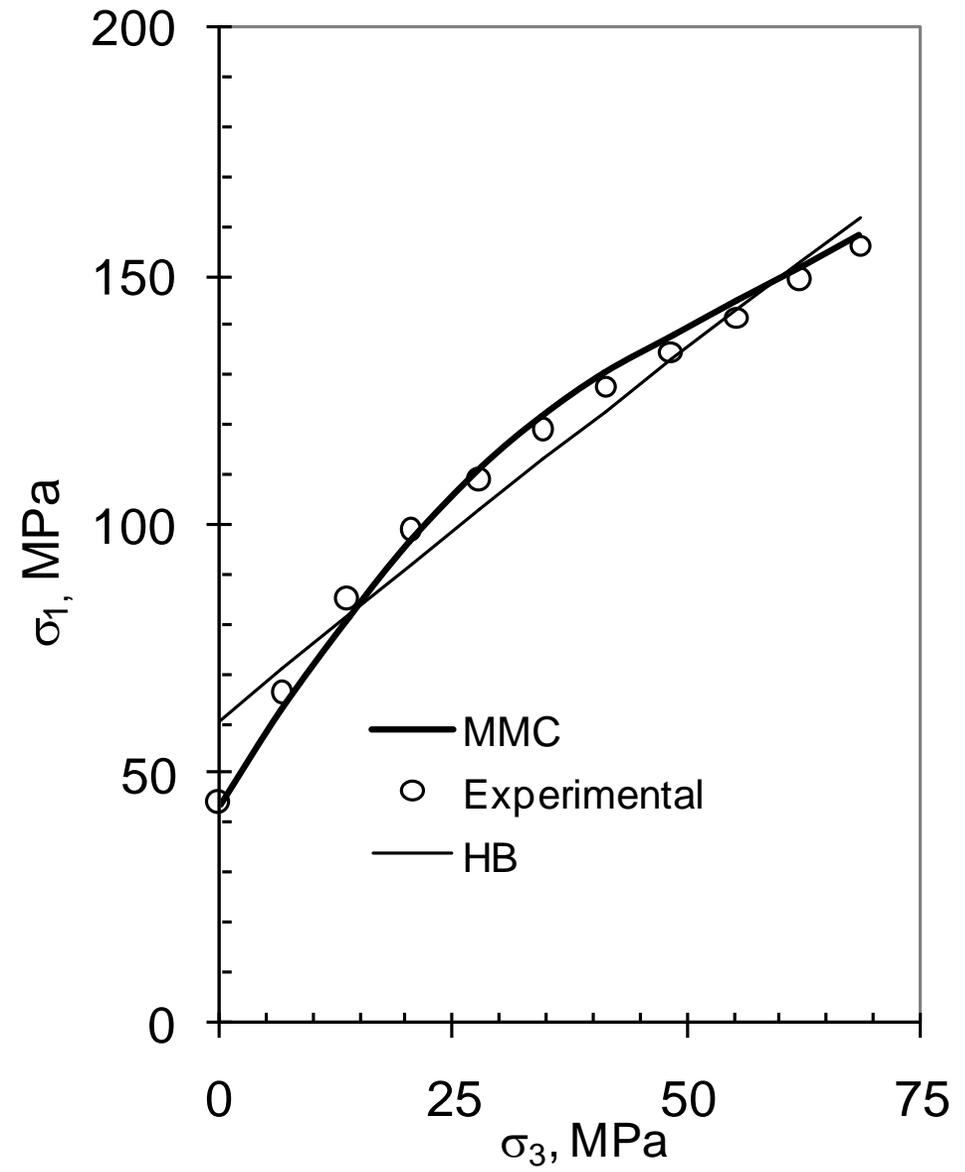


SHEAR
STRENGTH
LIMITS THE
HEIGHT OF THE
HIGHEST
MOUNTAINS

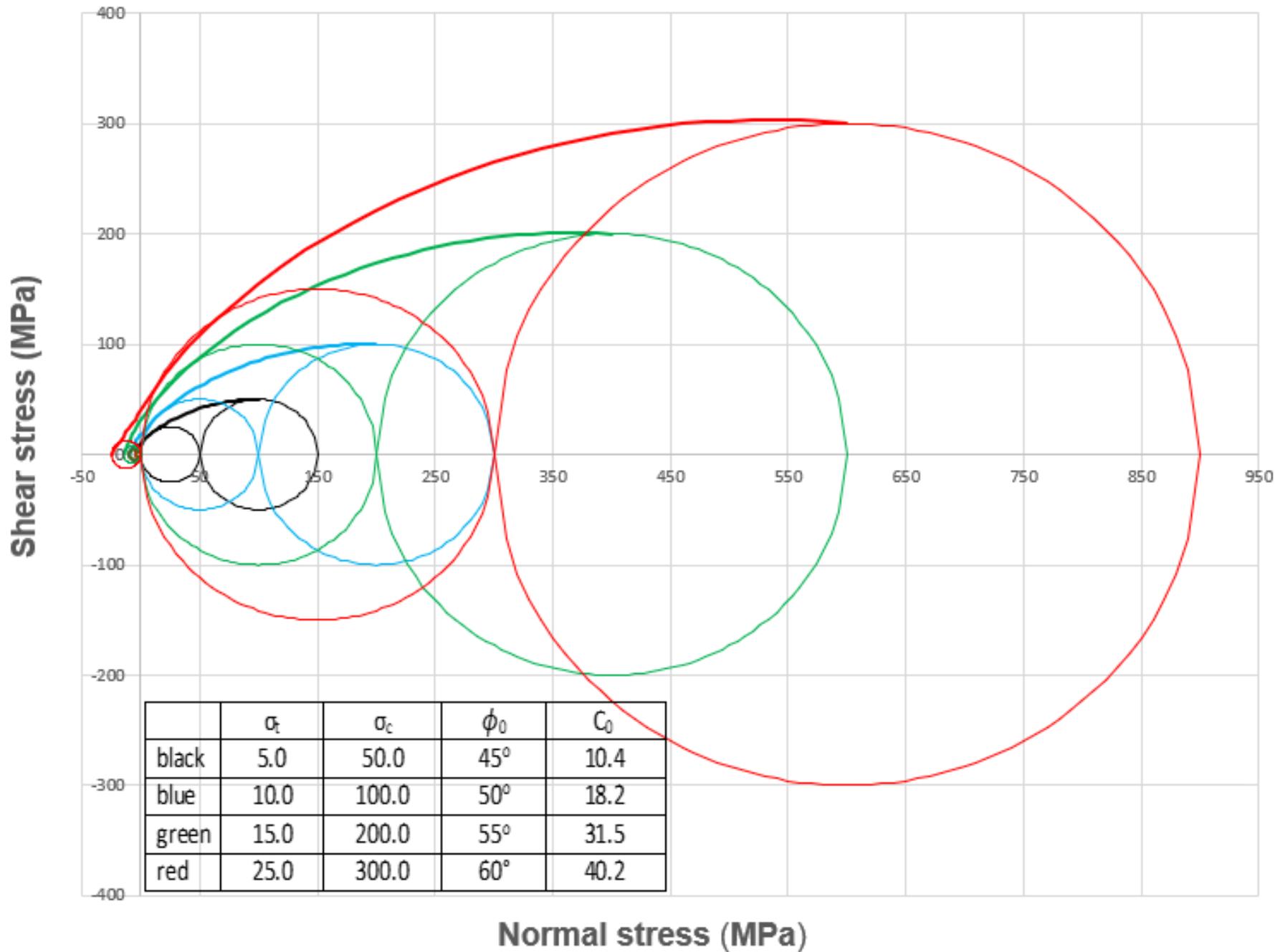
$$(\tau_{\max} \approx \sigma_c)$$



Singh, Raj, Singh, 2011



LIMESTONE. Singh and Barton, 2019



Approximate
nonlinear strength
criteria with critical
state for intact rocks.

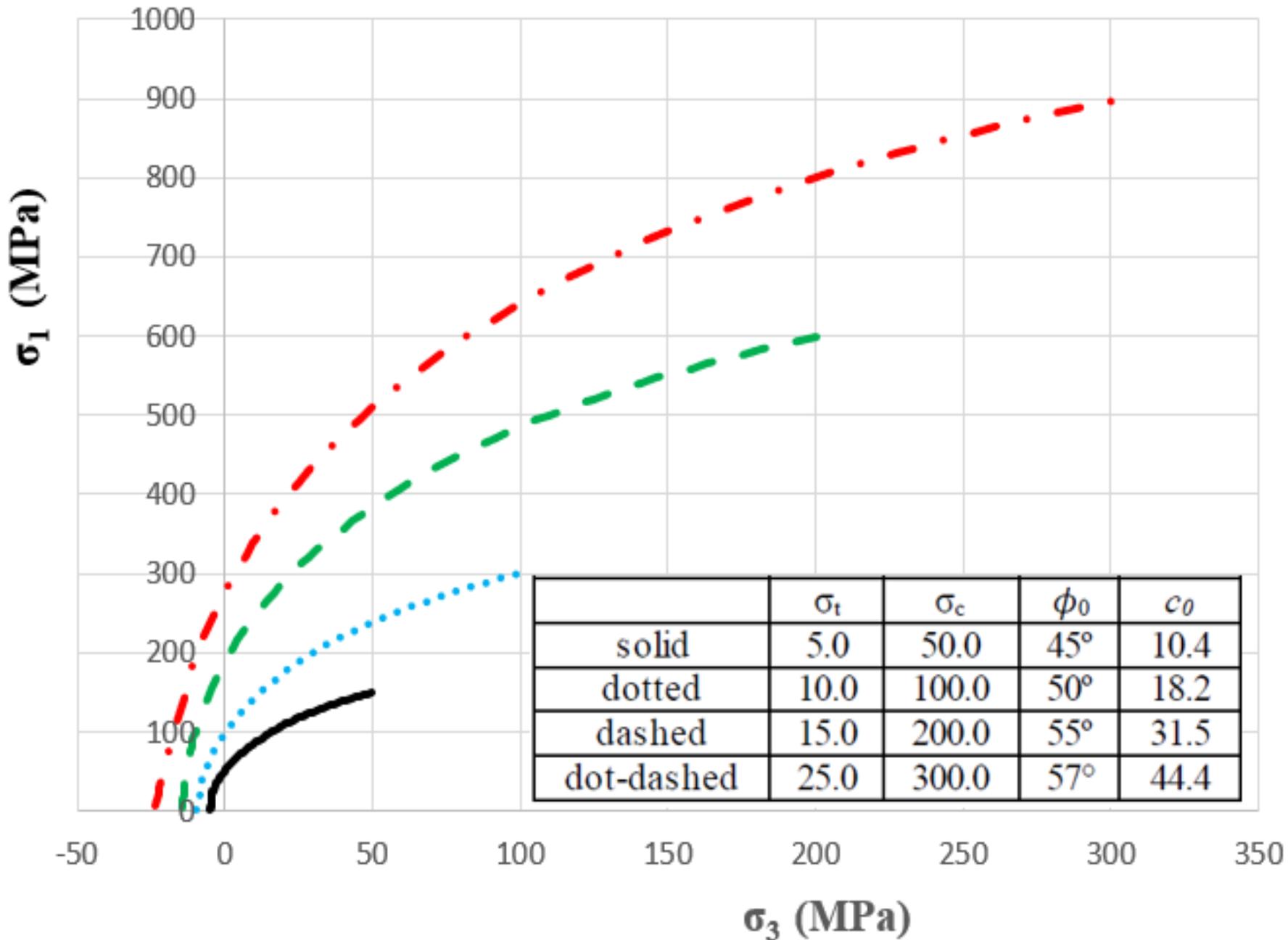
Shen, Shi, Barton, 2019

$$\phi = \phi_0 - (45^\circ - \phi_0) \frac{\sigma_n}{\sigma_t}$$

$$c = c_0 + (\sigma_t - c_0) \left(\frac{\sigma_n}{\sigma_t} \right)^2$$

$$\phi = \phi_0 \left(1 - \sqrt{\frac{\sigma_n}{2\sigma_c}} \right)$$

$$c = c_0 + (\sigma_c - c_0) \frac{\sigma_n}{2\sigma_c}$$



Confined strength is too high..... it is shear strength that is controlling mountain heights

CONCLUSIONS

1. DEEP TUNNELS IN HARD BRITTLE ROCK MAY FRACTURE/SPALL DUE TO INITIATION OF *EXTENSION FRACTURING, AND BURSTING IF PROPAGATION IS IN (UNSTABLE) SHEAR.*
2. THE FAMILIAR '*0.4 (+/-0.1) X UCS*' FRACTURE INITIATION STRESS IS ACTUALLY DUE TO σ_t/ν . THIS (ALSO) SIGNALS THE START OF ACOUSTIC EMISSION.
3. CLIFFS AND MOUNTAIN WALL HEIGHTS ARE LIMITED BY THE WEAKEST LINK (TENSILE STRENGTH) AND POISSON RATIO, CAUSING EXTENSION STRAIN IN (EVEN) A 3D *ALL-IN-COMPRESSION* STRESS FIELD. $H_c = \sigma_t/\gamma\nu$.
4. MULTI-COMPONENT STRENGTH: CCSS: Crack!!! Crunch!! Scrape! Swoosh!! IF NEEDING TO PERFORM CONTINUUM ANALYSES, DO NOT USE M-C, NOR H-B. (i.e. NOT '*c*' + $\sigma_n \tan \phi$) BUT *'c' then $\sigma_n \tan \phi$* DEGRADE *c*, MOBILIZE ϕ , THEN DEGRADE ϕ BEYOND PEAK.
5. THE 'LIMITED' HEIGHTS OF THE 15 HIGHEST MOUNTAINS (8 TO 9 km) IS CAUSED BY THE (CRITICAL-STATE) LIMITS OF SHEAR STRENGTH, *NOT BY AN IMPOSSIBLE UCS. CONFINED* COMPRESSIVE STRENGTH IS MUCH TOO HIGH. CURVATURE IS ACTUALLY GREATER THAN H-B.