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

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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, σt / ν, non-linearity)

Mahendra Singh, Univ. Of Roorkee, India. (σ3 critical ≈ UCS, non-linearity)

TWO VITAL CONTRIBUTORS TO FRACTURING MODES AND CURVED STRENGTH ENVELOPES

Stavros Bandis: shear strength in general

Neil Bar: Q slope case records

Eystein Grimstad: Q tunnel case records

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: $\sigma_{\theta\,max}/\sigma_c > 0.4 +/- 0.1$

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

(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*.

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

= **(Block size) x (friction) x (‘active stress’)**

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)

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

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)

$\varepsilon_1 = \sigma_1 / E$ …… $\varepsilon_3 = \nu\sigma_1 / E$

Critical tensile strain for tensile failure:

$\varepsilon_c = \sigma_t / E$ (a definition)

Equate strains, and eliminate E.

Therefore:

$\boldsymbol{\sigma_{1\,(critical)} = \sigma_t / \nu}$ **(≈ *0.4* x UCS)**

(We get '0.4' if UCS/σ_t ≈10, and ν ≈ 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)

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

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)

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 *h* .
(Verruijt, 2001)

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

(c) Circular failure surface.
(Fellenius, 1927)
Broad limits assuming c *and* ϕ :
$h \geq 2c/\gamma \,.\, \tan(45° + \phi/2)$ lower bound
$h \leq 4c/\gamma \,.\, \tan(45° + \phi/2)$ upper bound

‘SOIL MECHANICS’ THEORIES:

***2c/γ* . tan(45°+ φ/2) ≤ *Hc* ≤ *4c/γ* . tan(45°+ φ/2)**

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

(UNFORTUNATELY ≈ 3x to ≈ 6x IN ERROR WHEN ***EXTENSION FAILURE – and NOT SHEAR*** FAILURE OF ROCK IS OCCURING)

TERZAGHI (*1962*): Hcrit = *qc*/γ (soil) (≈ 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?)*

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

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

THREE OF THE HIGHEST ‘VERTICAL’ MOUNTAIN WALLS in THE WORLD

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

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

Mirror Wall, Greenland: **1,200m**

ESTIMATING THE COHESIVE STRENGTH OF INTACT ROCK

(lower-bound)

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

(lower-bound estimate)

Assumes linear envelope between tensile and compression circles.

Actual cohesion is higher due to curvature.

(Barton, 1976)

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

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

2. Granite

σ_c = 150MPa,

σ_t = 10MPa

c = ½(150x10)$^{1/2}$ = 19.4MPa

Sandstone 'valley wall' **(H_c = 2c/γ x tan (45°+ϕ/2):**

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

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

Granite 'mountain-wall' **(H_c = 2c/γ x tan (45°+ϕ/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, USING *EXTENSION STRAIN THEORY*:

Sandstone: 100.σ_t /γν = 100.5/2.5 x 0.25 = *800m*

Granite: 100.σ_t /γν = 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 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! σt REDUCES, OVER GEO-MORPHOLOGICAL 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)

USGS artist C.A.Weckerly

seems to have envisaged a future ‘failure mechanism’.

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).

REMEMBER: SCALE EFFECTS ON JRC, JCS GIVING BLOCK-SIZE DEPENDENT STRENGTH FOR THE ROCK JOINT COMPONENT (Bandis, 1980 – Univ. of Leeds Ph.D.)

IN SITU BLOCK

TILT TEST

(Large scale)
3 tests

(1 sample)

TILT OR PUSH TEST

(Small scale)
54 tests

(18 samples)

WHAT IS HELPING TO PREVENT SUDDEN COLLAPSES ?

('X' ≈ 50, then 20, then JRC, then Jr/Ja) Barton, 1999

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

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.

(Hajiabdolmajid, Martin and Kaiser, 2000

NEED TO SEPARATE

'c' and 'ϕ'

degrade c, mobilize ϕ

× Shear failure ○ Tensile failure

Degrade cohesion, mobilize friction: excellent match.

(Hajiabdolmajid, 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 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'

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

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

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

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

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

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

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

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

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

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 *then* σn tan ϕ’

(as in Barton and Pandey, 2011)

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

Back-wall shear(?)

.....

tensile opening (?)

.....

triggering by $\sigma_t/\gamma\nu$?

(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

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

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 (%)	Jn	Jr		Ja		0-factor		Jwice	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

$$\boldsymbol{Q_{slope}} = \frac{RQD}{J_n} x \left(\frac{J_r}{J_a}\right)_0 x \frac{J_{wice}}{SRF_{slope}}$$

$$= \frac{55}{9} x \left(\frac{1}{4} x 0.5\right)\left(\frac{3}{4} x 0.9\right) x \frac{1}{2.5}$$

$$= \mathbf{0.206}$$

$$\boldsymbol{\beta} = 20 log_{10} Q_{slope} + 65$$

$$= 20 log_{10}(0.206) + 65$$

$$= \mathbf{51°} \quad (steepest\ slope\ angle)$$

15° shallower than excavated and consistent with kinematic analysis

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

Qslope Case Study Data
Stable Slopes
Quasi-Stable Slopes
Failed Slopes
β = 20log10(Qslope) + 65
β = 20log10(Qslope) + 65
Slope Angle, β (degrees)
Qslope
0
10
20
30
40
50
60
70
80
90
0.001
0.01
0.1
1
10
100

P-Wave Velocity and Q-value

- Motorway project in Panama (Barton, 2007)
- Forest clearing for roadway cuttings
- Seismic refraction used alongside core logging.
- Higher Vp 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

$Hc = 100\ \sigma_c / \gamma$

e.g. 100 x 250/2.8 = 8.9km?

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

	σ_t	σ_c	ϕ_0	C_0
black	5.0	50.0	45°	10.4
blue	10.0	100.0	50°	18.2
green	15.0	200.0	55°	31.5
red	25.0	300.0	60°	40.2

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

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

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

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

	σ_t	σ_c	ϕ_0	c_0
solid	5.0	50.0	45°	10.4
dotted	10.0	100.0	50°	18.2
dashed	15.0	200.0	55°	31.5
dot-dashed	25.0	300.0	57°	44.4

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 $H_c = \sigma_t/\gamma\nu$.

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' + σ_n tan 'φ') BUT *'c' then σ_n tan 'φ'*. 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.