LESSONS LEARNED FROM LARGE FAILURES: MULTIPLE CAUSES INCLUDE ADVERSE DESIGN, GEOLOGY OR SUPPORT

ISRM SYMPOSIUM, CAPETOWN

Nick Barton, NB&A, Oslo. Norway

(www.nickbarton.com)

CONTENT of LECTURE

1.CAVERN COLLAPSE DUE TO CONCEALED RIDGE, LACK OF ARCHING STRESS

2. OPTIMISTIC TEMPORARY SUPPORT IN NATM TUNNEL: REAL-ROCK PROBLEM

3. STEEL SETS FAIL IN A CAVERN ARCH DUE TO NEGLECT OF A FAULT ZONE

4. LIMITATIONS OF LATTICE GIRDERS WITH OVER-BREAK, WITH BIG JOINTS

5. OPEN-PIT COLLAPSE DUE TO 'NOSE' AND NO TANGENTIAL CONFINEMENT

6. PROGRESSIVE FAILURE OF ROCK MASSES – FOUR COMPONENTS

7. *OVER-STRESSED/OVER-STRAINED*: THE '0.4'x UCS 'MYSTERY' EXPLAINED

8. EXTENSIONAL-STRAIN TENSILE FRACTURING, PROPAGATION IN SHEAR

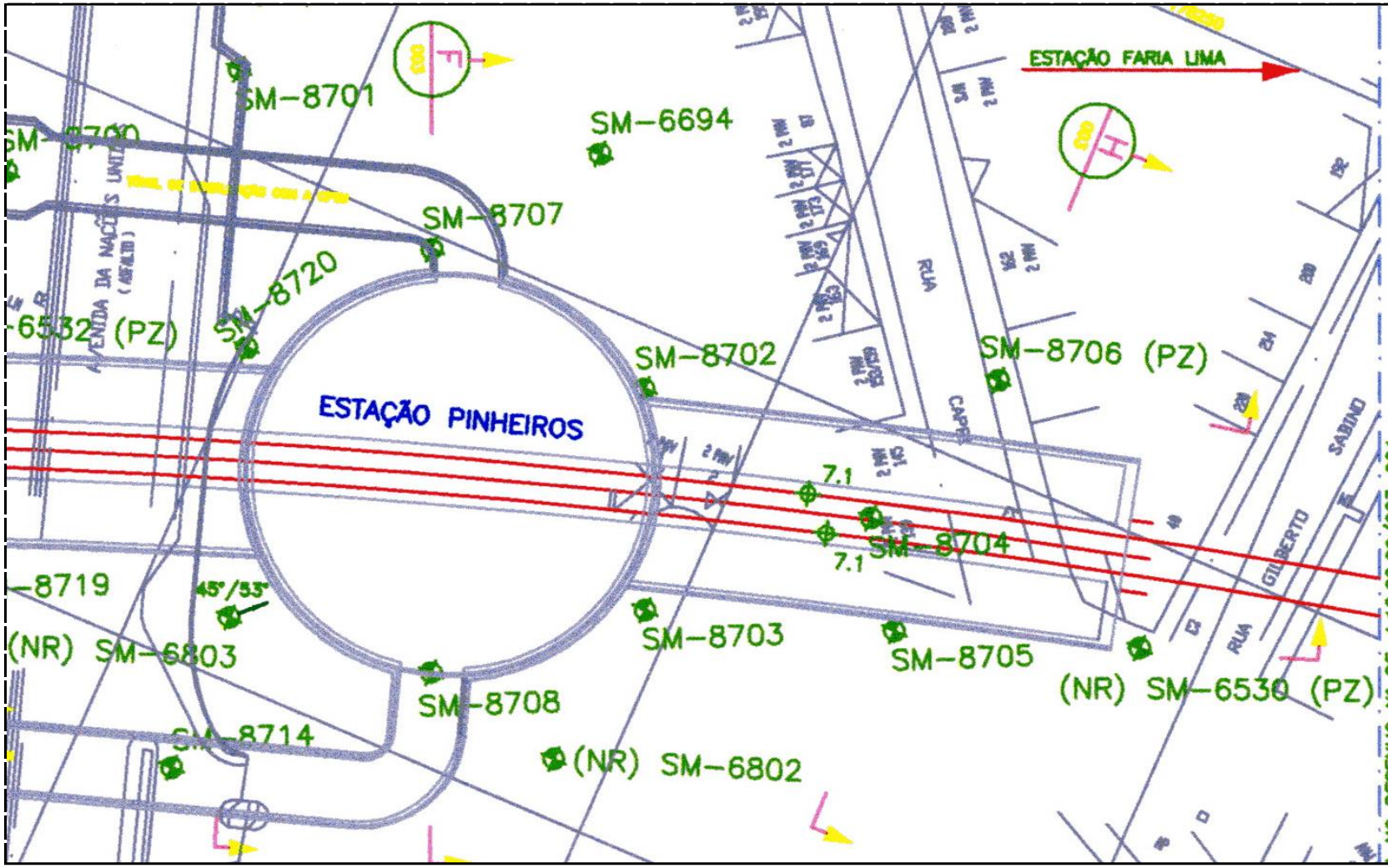
9. BENEFIT OFJOINTING IN DISSIPATING UNSTABLE (SHEAR) FRACTURING

Acknowledgements

Dr. Stavros Bandis † (former professor at Univ. of Thessaloniki)

Dr. Baotang Shen (senior scientist, CSIRO, Brisbane, Australia)

(For respectively their UDEC/3DEC and FRACOD/fracture modelling expertise, and long friendships)

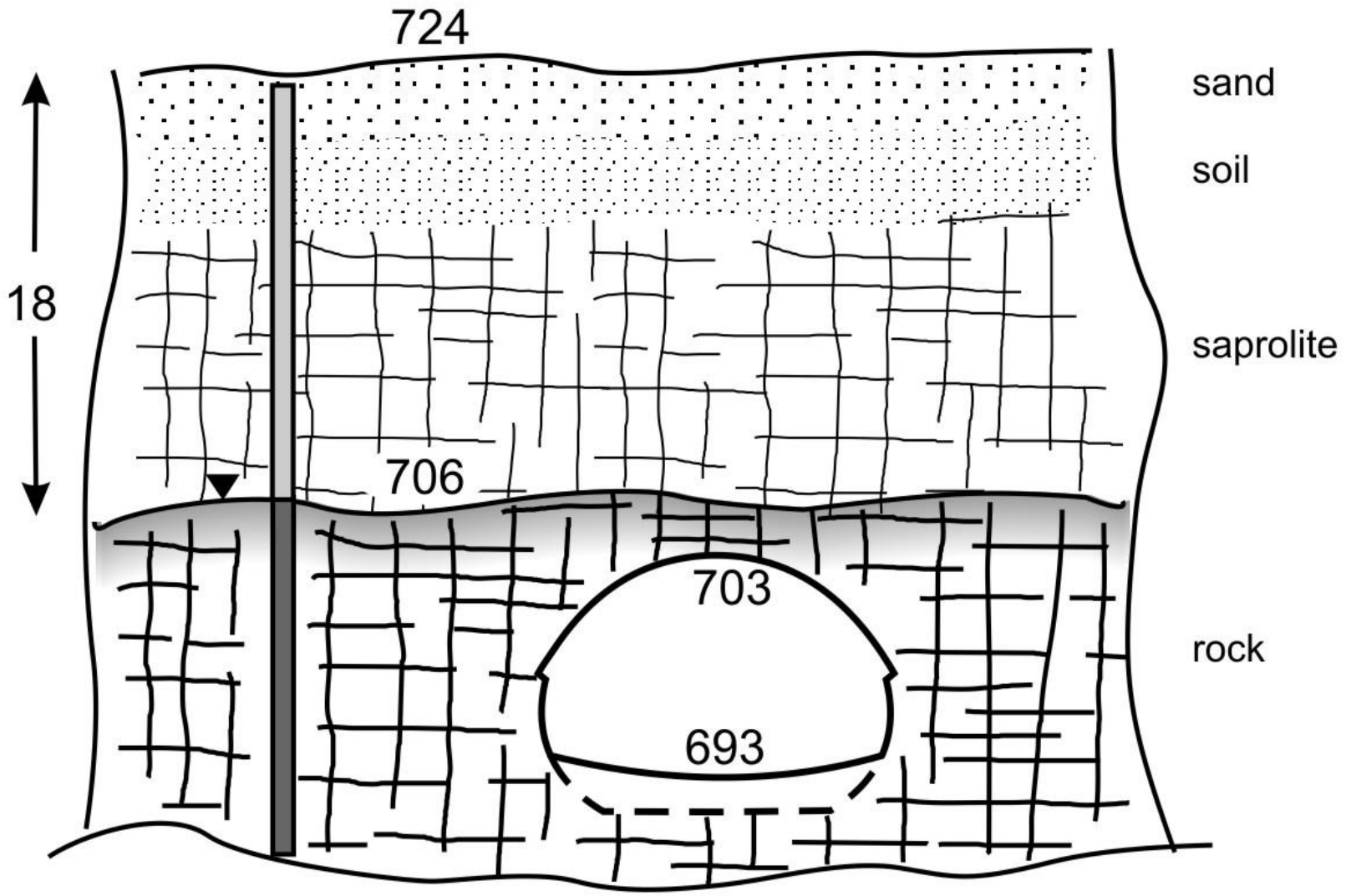


Eleven **boreholes around shaft and eastern station cavern.**

Exceeds international norm:

L borehole / L tunnel .....by a factor of ≈ 2 to 4?

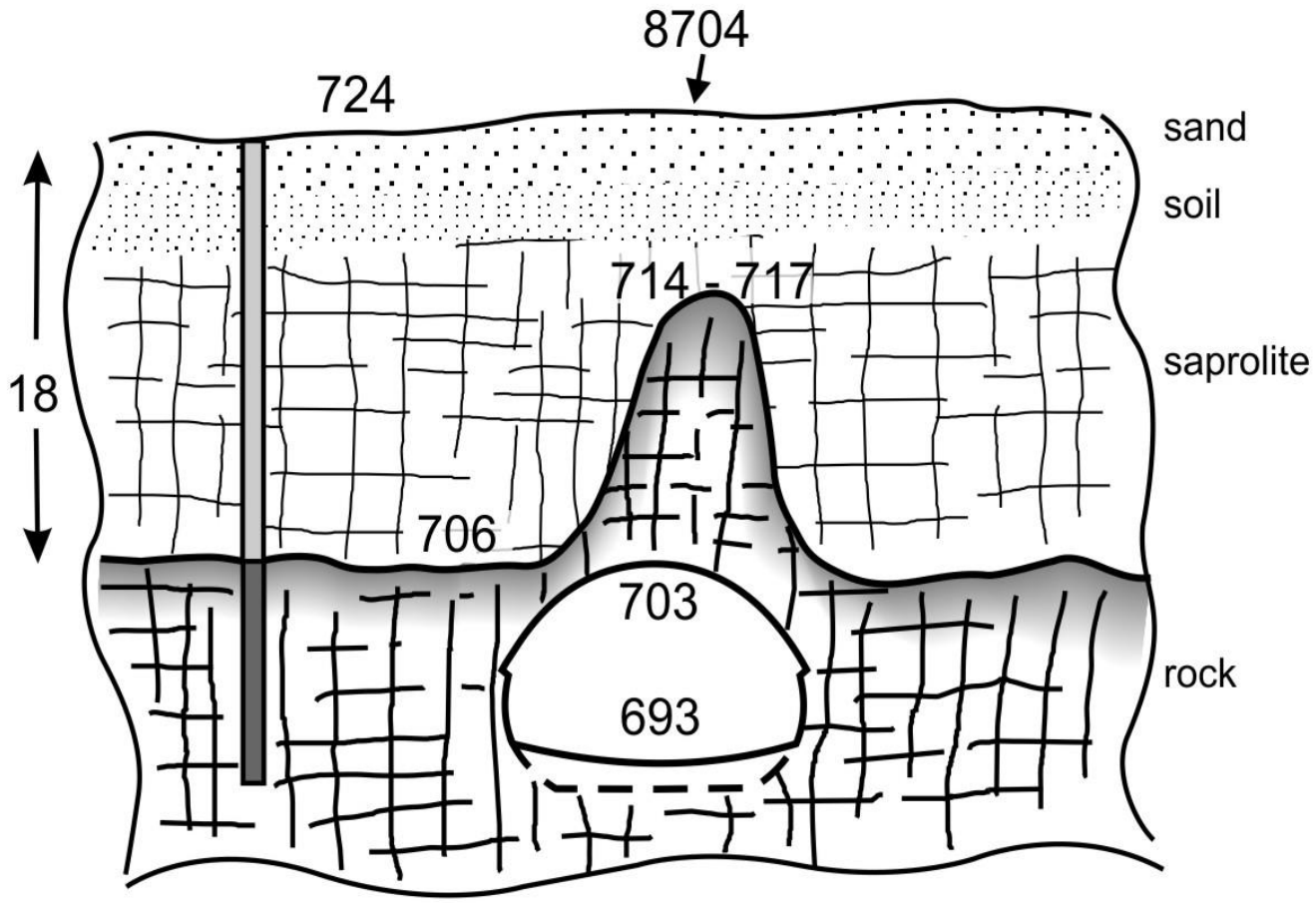
WHAT WAS EXPECTED – ON AVERAGE – CONCERNING ROCK COVER



Borehole SM-8704 drilled near centre of (future) station cavern.

Rock was encountered at 18 m depth, at elevation 706m. This is 3 m above the cavern roof.

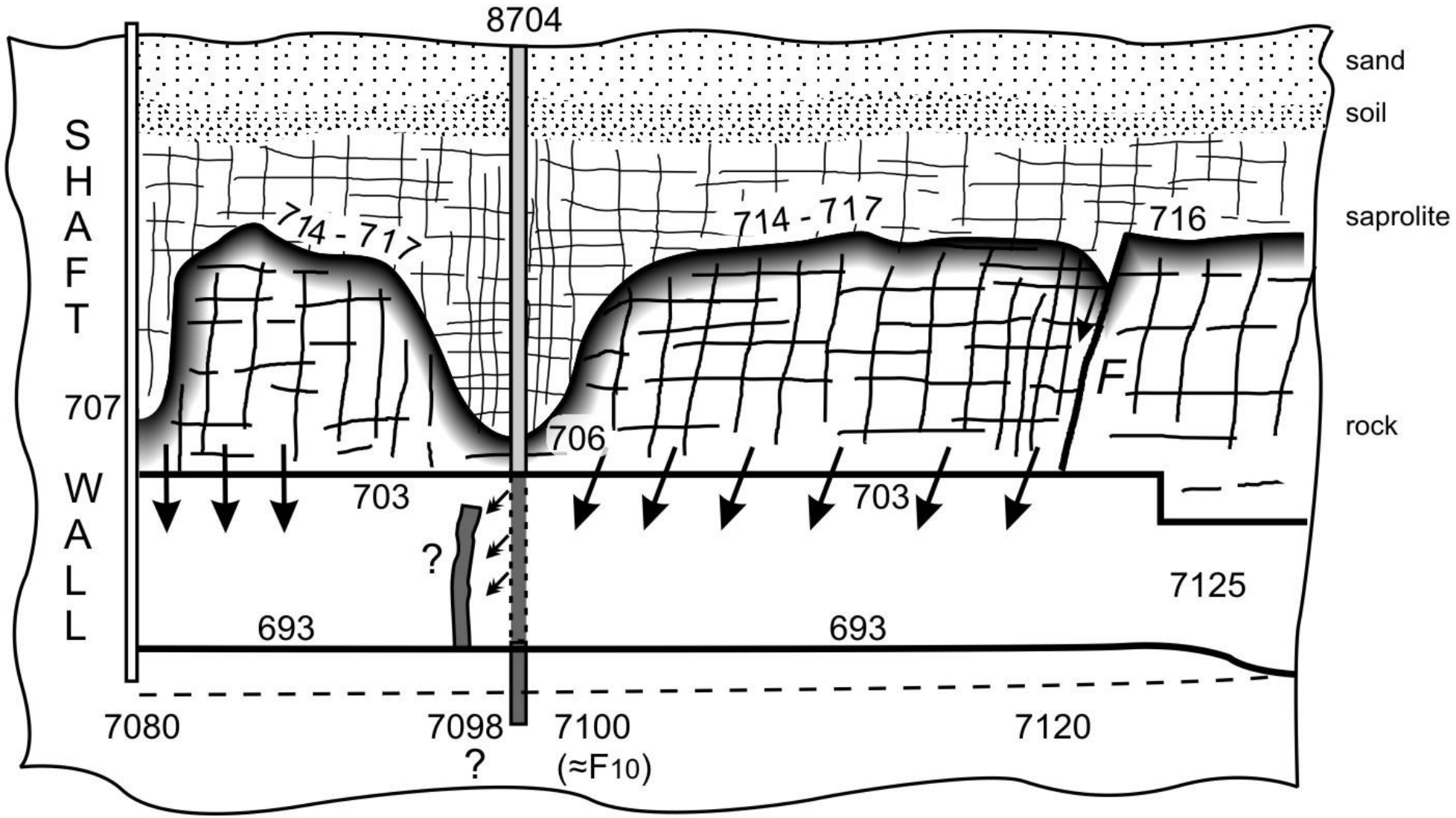
Low rock cover was 'confirmed' – almost the same as the mean of the five nearest holes.

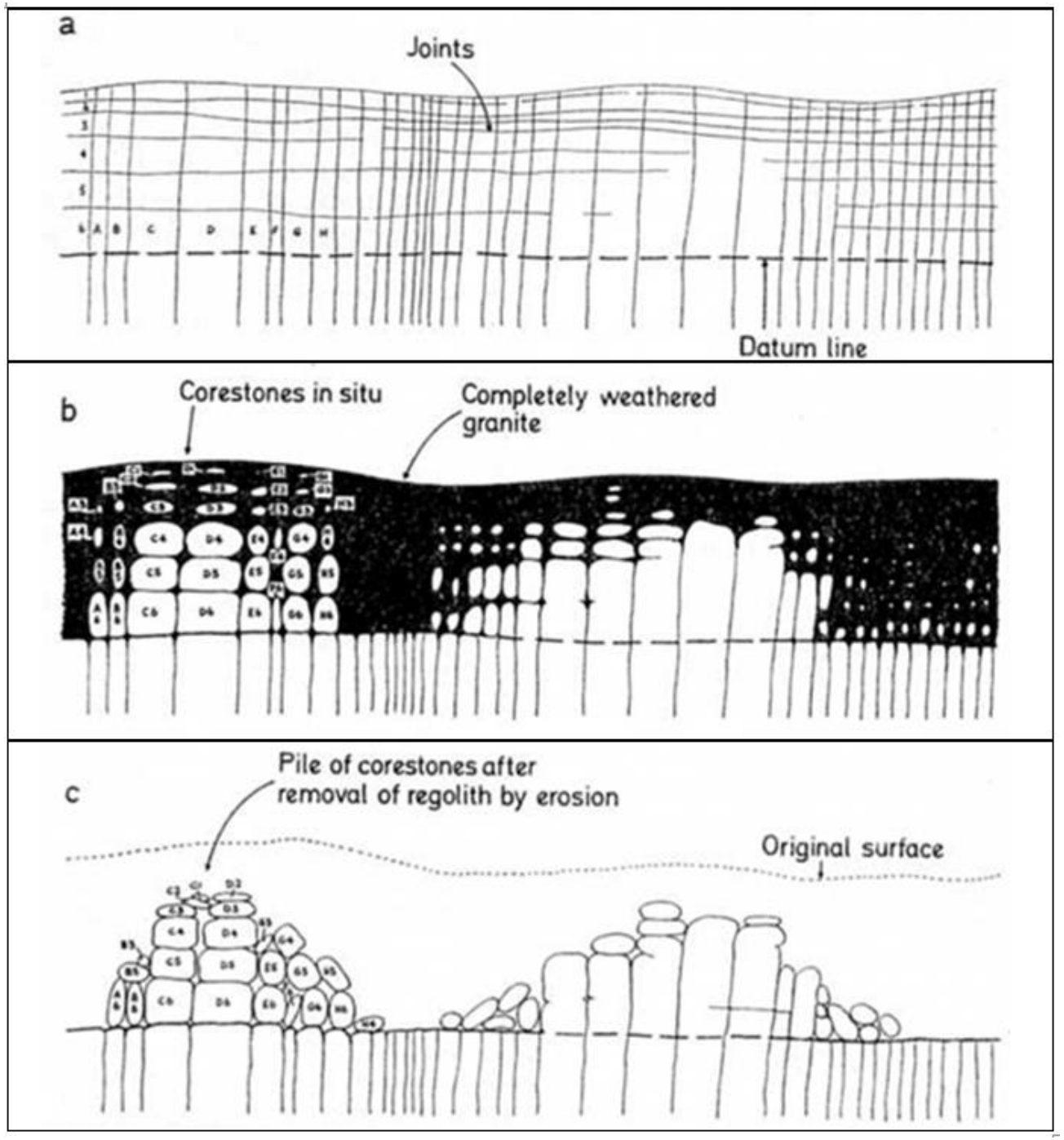


***THE EXTRAORDINARY REALITY:* SUB-SURFACE RIDGE-OF-ROCK THAT WENT UNDETECTED**

MOST OF THE COLLAPSED ROCK IN THE CENTRE OF THE CAVERN FELL 10m, REACHING ELEVATION 704-707m, i.e. REMAINING 1 TO 4m ABOVE THE (ORIGINAL) CAVERN ARCH.

THIS SHOWS WHY THE RIDGE WAS MISSED WHEN DRILLING SM-8704

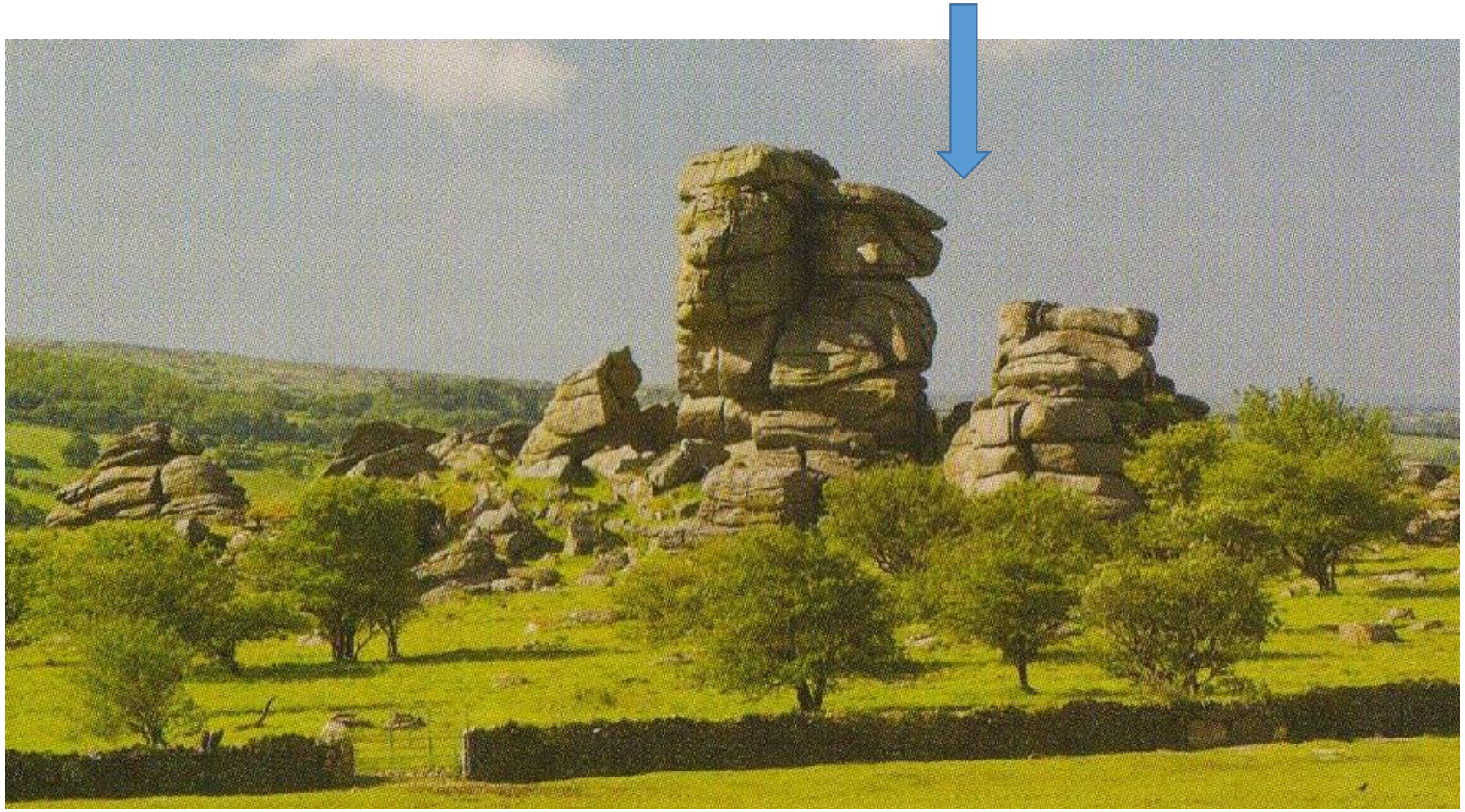




Linton, 1955. A classic model of differential weathering.

STAGE (b) is relevant to the origin of the cavern collapse

STAGE (c) is shown on next screen



'TORS' ON DARTMOOR, S.W. ENGLAND. PREVIOUSLY SURROUNDED BY SAPROLITE, SOIL AND SAND.

ARROW DEPICTS REASON FOR MISSING RIDGE WITH CENTRAL BOREHOLE ??

THE SUB-SURFACE IN AREAS OF DEEPLY-WEATHERED GRANITE, GNEISS, LIMESTONE, ETC. MUST BE EXTREMELY 'ROUGH' BENEATH THE SMOOTH SOIL/SAND COVER……happily for our cities existence!

NON-UNIFORM LOADING DUE TO CONCEALED and UNKNOWN STRUCTURE ABOVE ARCH.

NO BOLTS IN THIS CASE DUE TO ASSUMED LOW COVER, AND FRAGMENTED WEATHERED ROCK. BUT S(fr) ≈ 40 to 50 cm AND 'ROBUST' LATTICE GIRDERS.

BEFORE AND AFTER

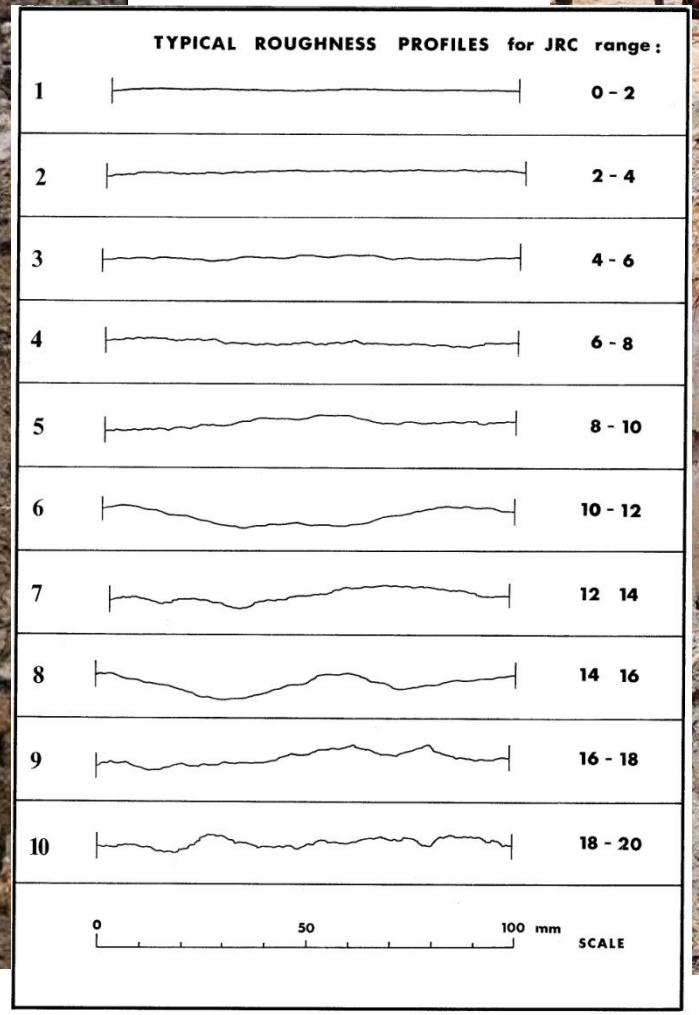
A first bench had been finished in relation to the top-heading seen in the left photograph.

January 2007 collapse. Collapse so sudden that pedestrians and minibus sucked into void. (Seven die)

Thin arrows mark same truck

The smooth discontinuity marking the limit of the collapse

Discontinuity marking ’end’ of collapse:
Very low JRC



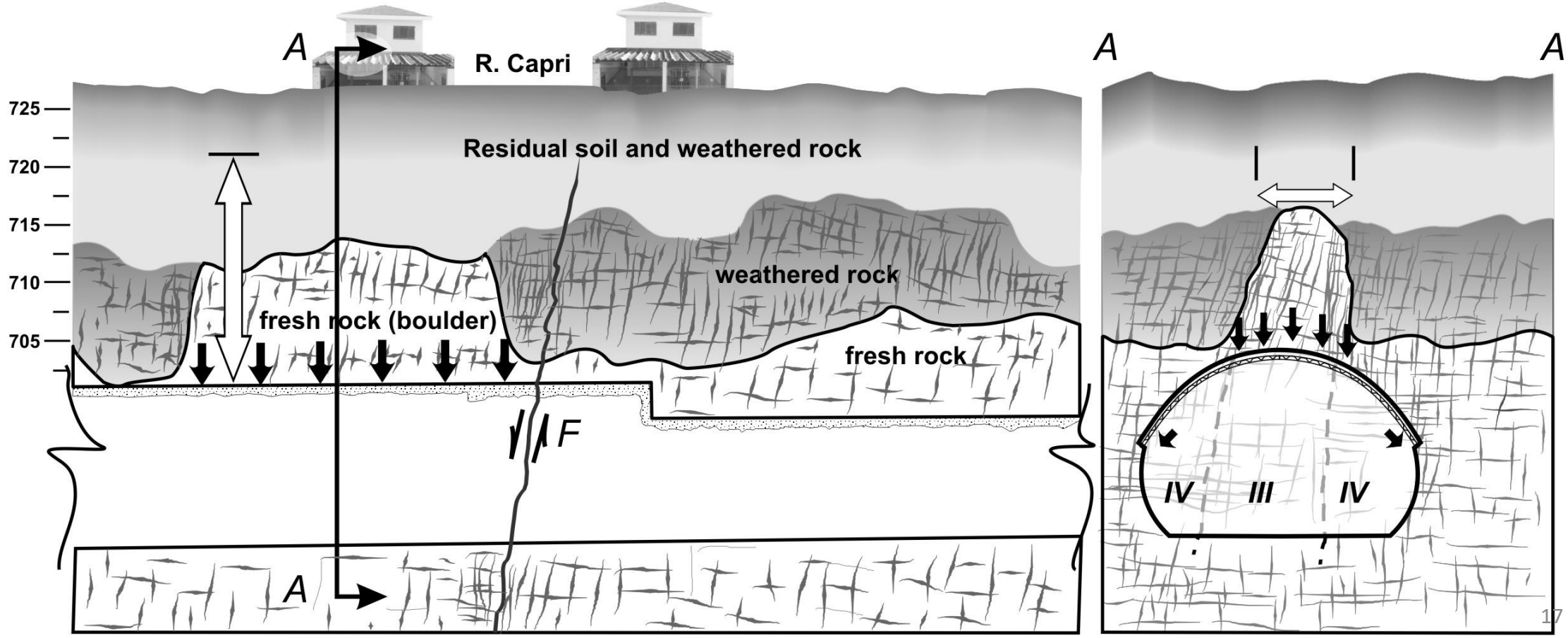
After many months of excavation – discover folded **25 x 25 x 32 mm** lattice-girders (and grouting tubes). Note 40-50 cm of S(fr) lower-left.

After many months of excavation:

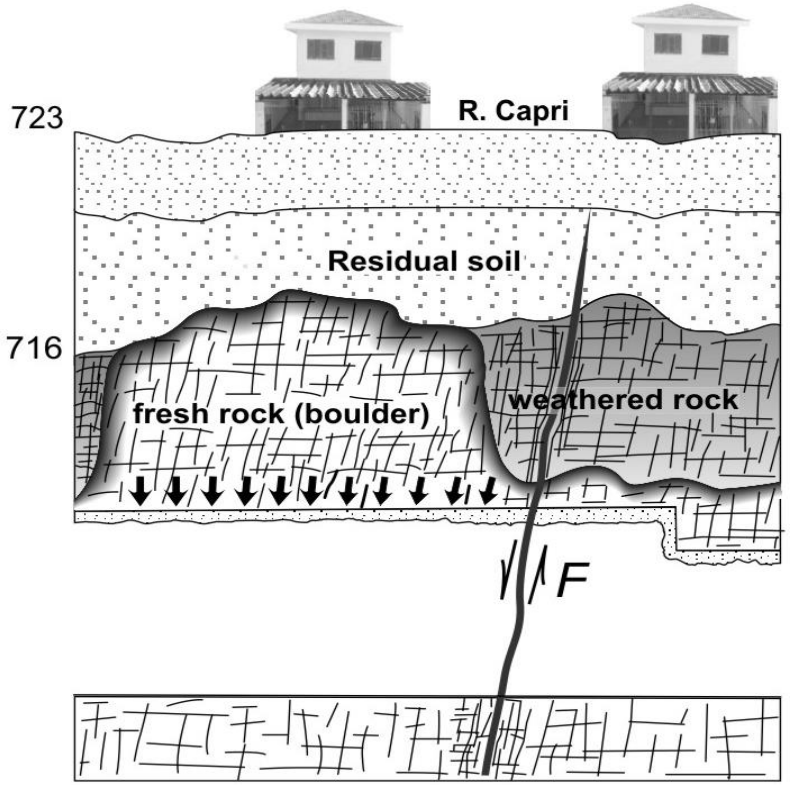
Smooth, weathered, curved, top-of-ridge-of-rock, length > 30 m.

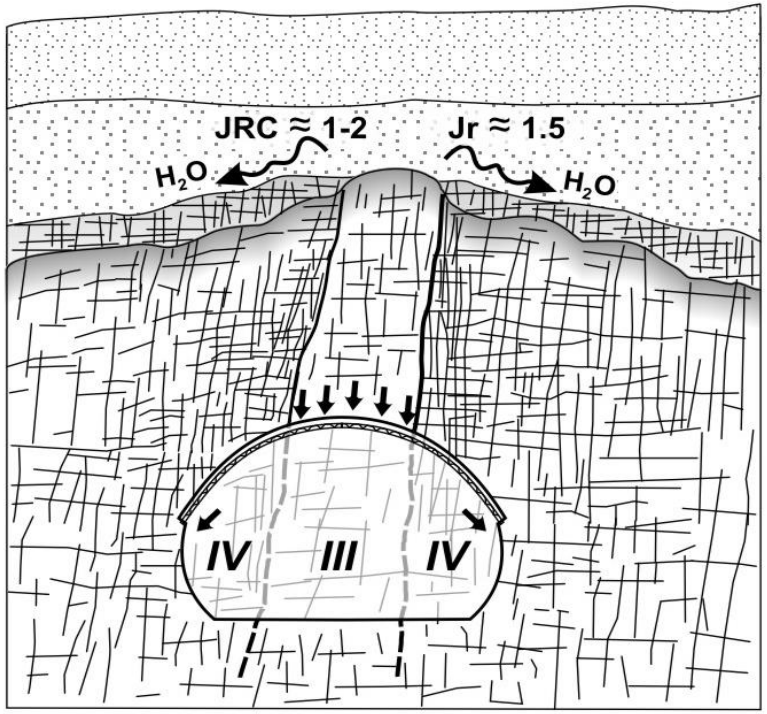
This jointed gneiss has fallen 8 to 10 m, but has remained like loosened ’pack-of-cards’.

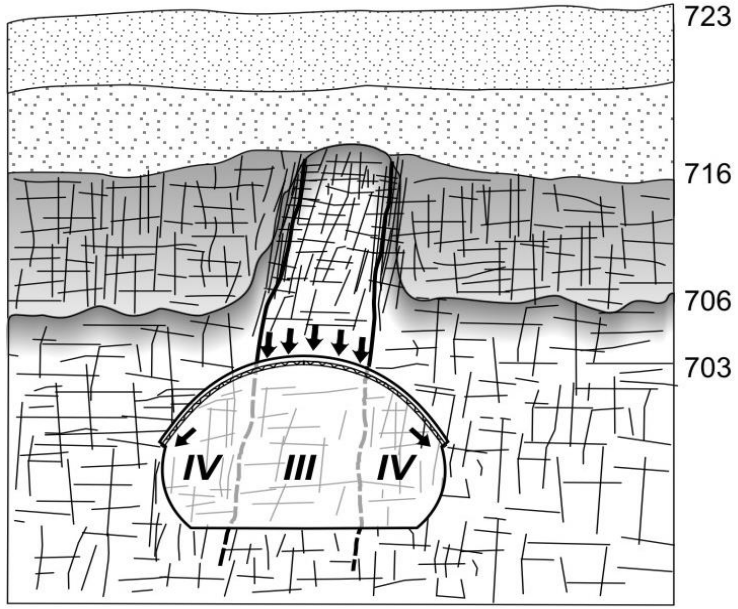
POSSIBLE CLUE CONCERNING PINHEIROS COLLAPSE GEOMETRY, FROM DISTANT IPT SEISMIC PROFILE: WITH RUA CAPRI, HOUSES, AND BOUNDARY DISCONTINUITY SUPERIMPOSED. (NOTE LACK OF RECORDED VELOCITIES DUE TO PROBLEMS WITH NOISE + DIFFERENTIALLY WEATHERED STEEP STRUCTURES).



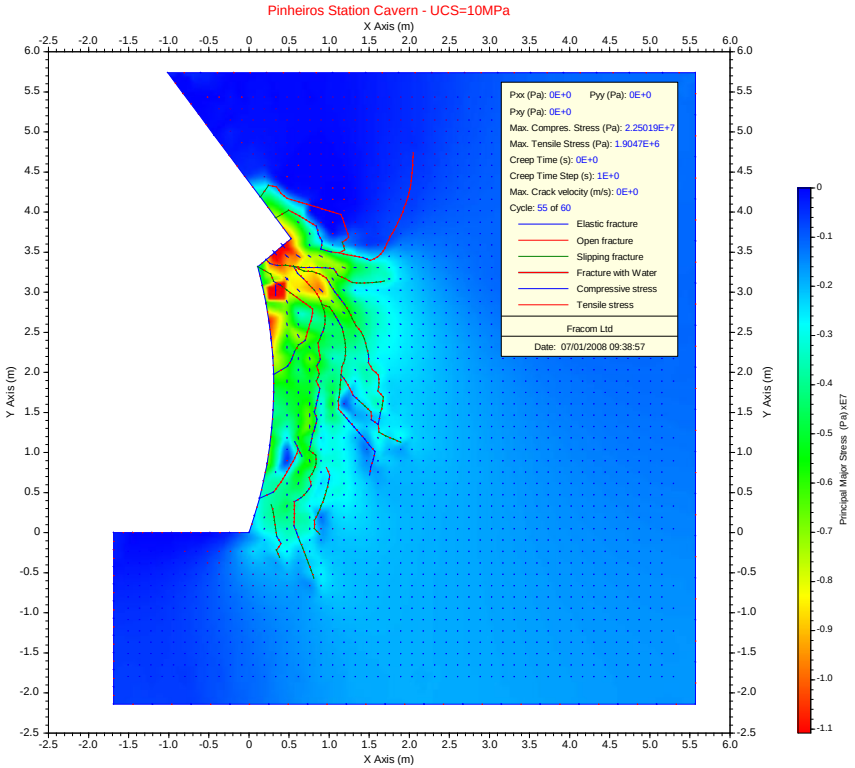
IMAGINED PROGRESSIVE FORMATION OF HIDDEN RIDGE-OF-ROCK







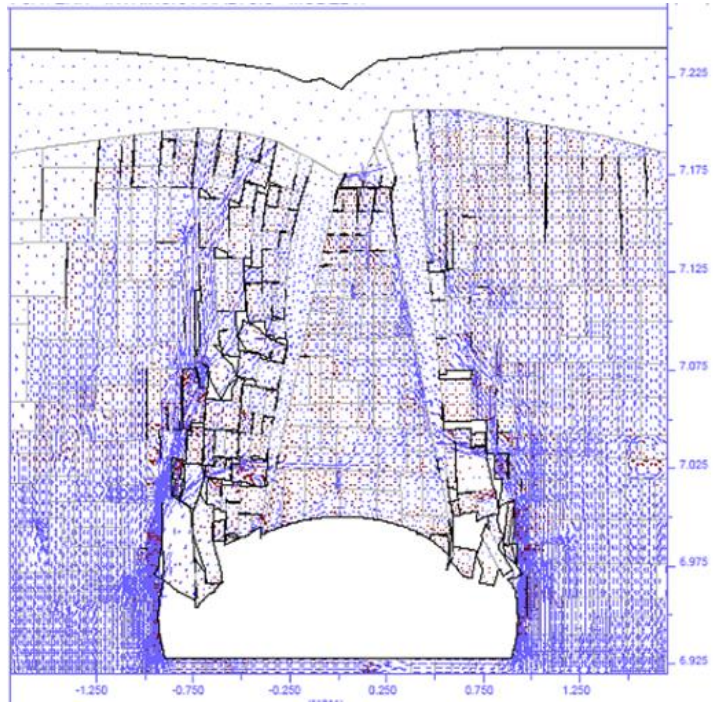
IN EFFECT THE (RMR III) RIDGE IS PRESERVED AND THE MORE JOINTED (RMR IV) SURROUNDINGS DETERIORATE, ***EVENTUALLY PROVIDING LOW-FRICTION BOUNDARIES WITH THE SAPROLITE.***

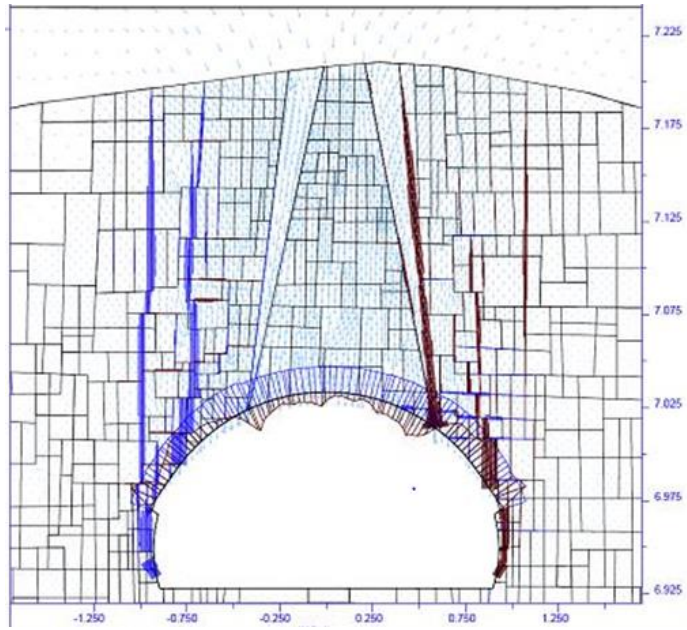


1. POSSIBLE EARLY FAILURE BENEATH 'ELEPHANT-FEET' IN 'LEFT' WALL. FRACOD MODEL BY DR.SHEN.

2. 'INTRINSIC' (WITHOUT SUPPORT) FAILURE WITH UDEC. (DR. BANDIS)

3. LOADING ON LATTICE GIRDERS AND DEVELOPMENT OF 'PLASTIC HINGES' PRIOR TO FAILURE. (DR. BANDIS)





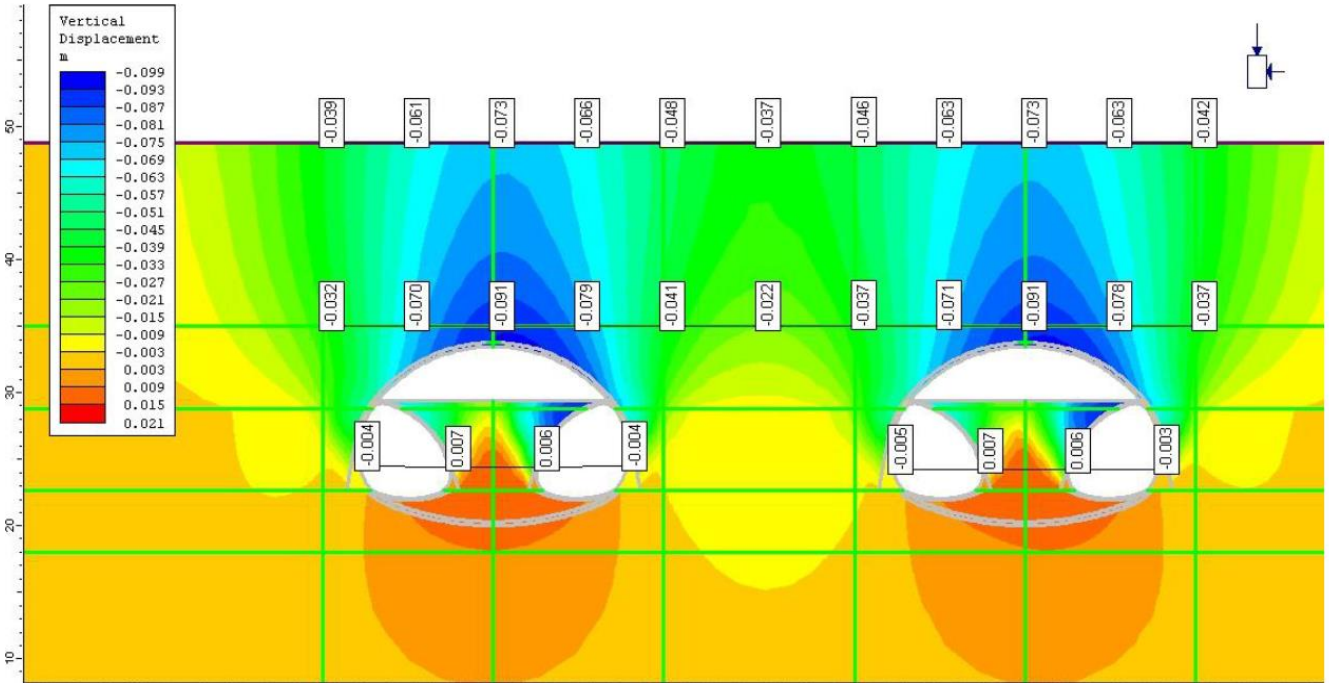
COLLAPSE # 2

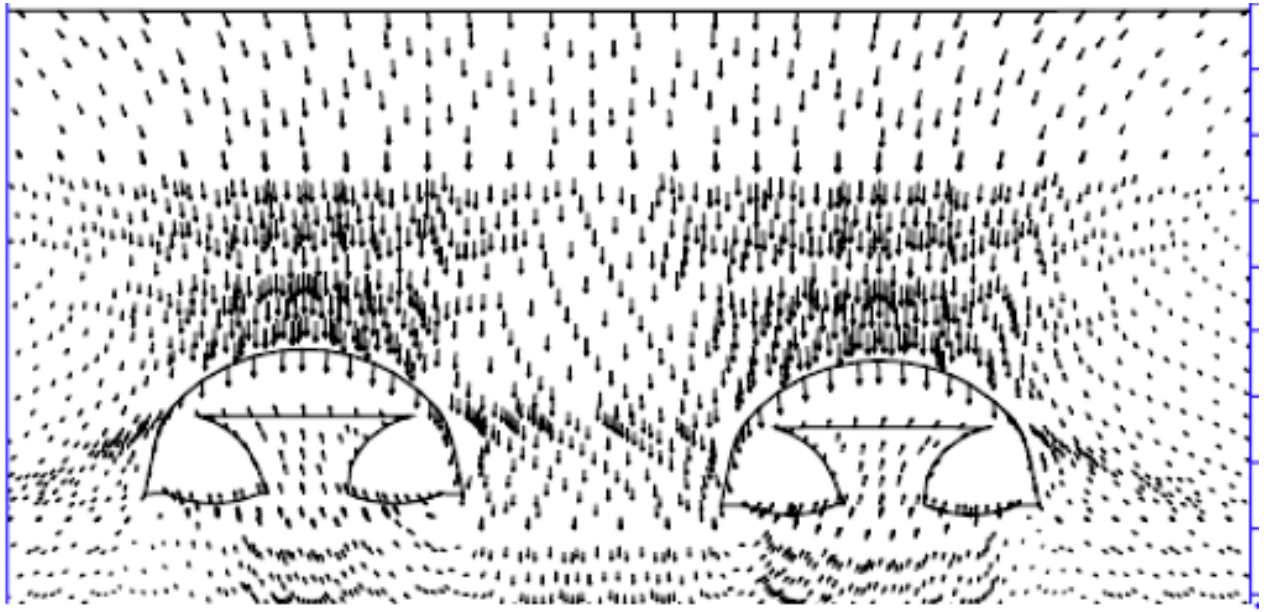
AN NATM TWIN-MOTORWAY TUNNEL

(with light, inadequate, temporary support, and anisotropic challenges from an actual rock mass)

OPTIMISTIC 'LIGHT' LATTICE-GIRDERS and UNREINFORCED SHOTCRETE. NO ROCK BOLTS due to INITIAL SAPROLITE.

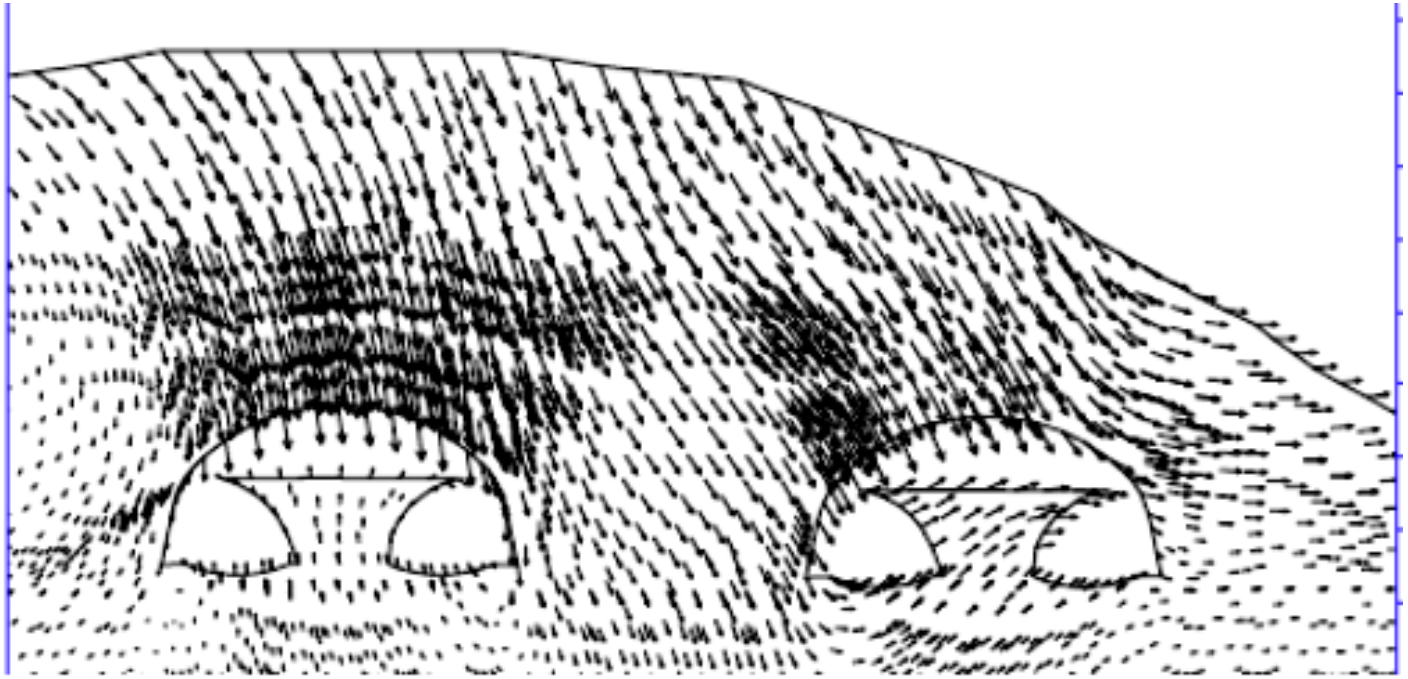
- **OPTIMISTIC AXISYMMETRIC UNIFORM LOADING ASSUMED.**
- **WHAT HAPPENS WHEN ACTUAL VERTICALLY JOINTED PHYLLITE?**
- **WHAT HAPPENS WITH OVER-BREAK AND WEATHERED DIKE?**



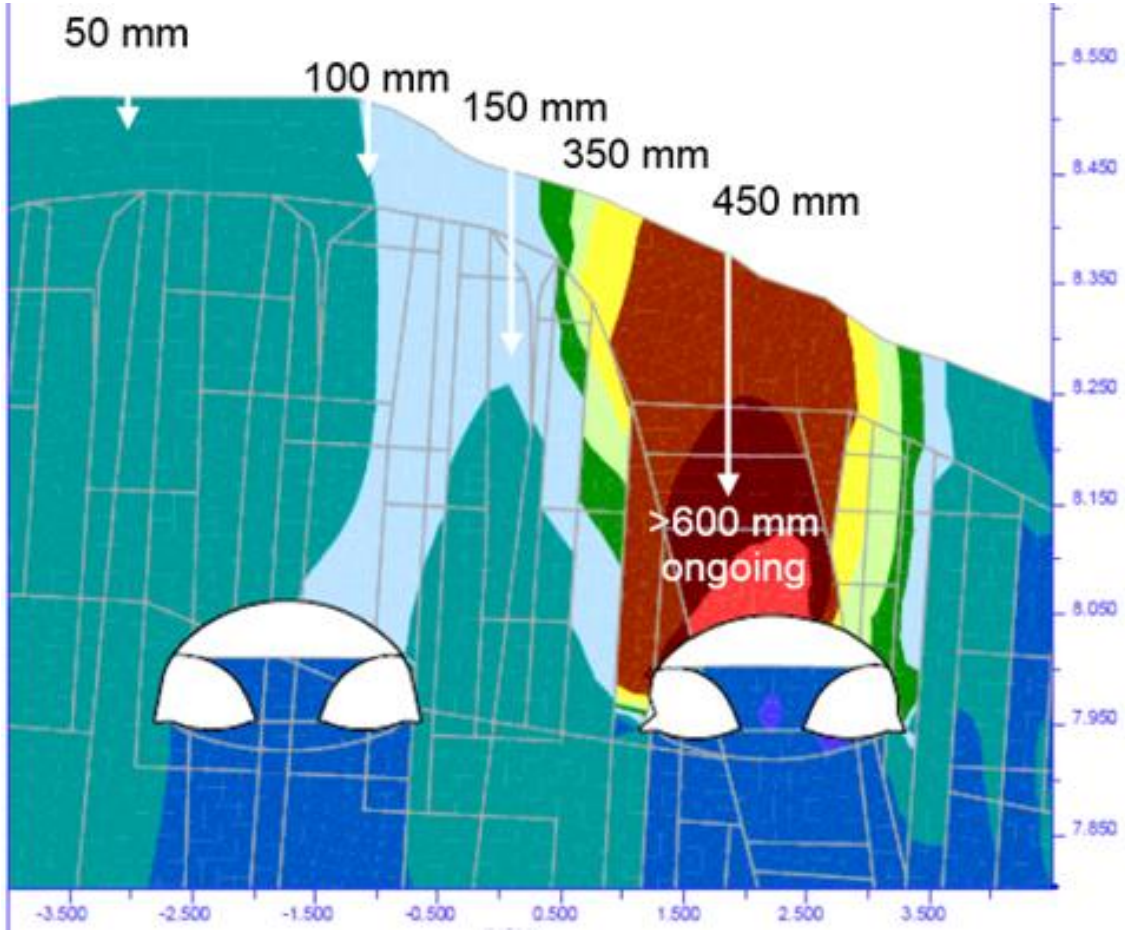


THE OPTIMISTIC ASSUMPTION

A NOT QUITE SO SIMPLE 2D 'REALITY' – WITH THE STRONG EFFECT OF A SLOPING HILLSIDE NOW INCLUDED

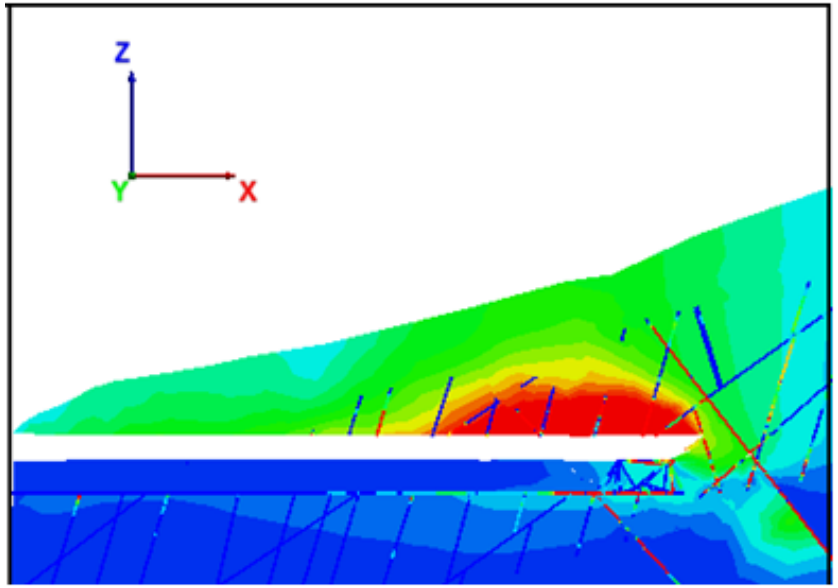


WEATHERED PHYLLITE (WITH STEEP DIP) AS SEEN IN THE NEIGHBOURHOOD OF THE TWIN TUNNELS

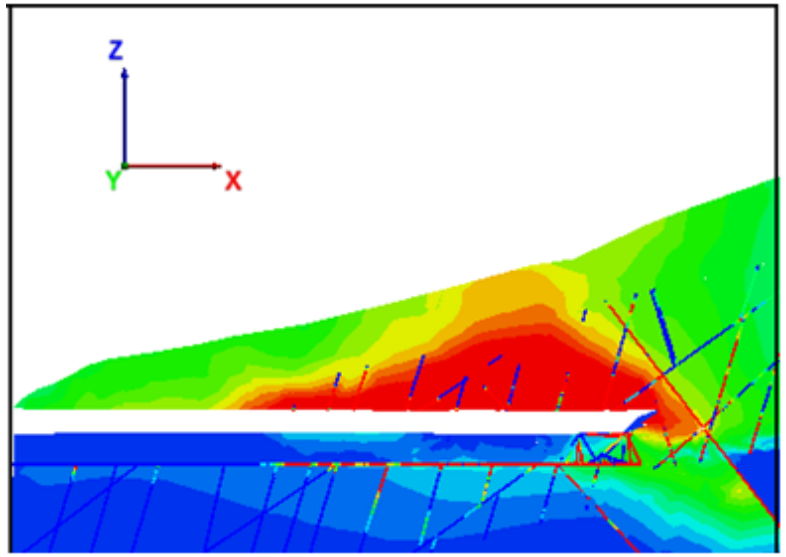


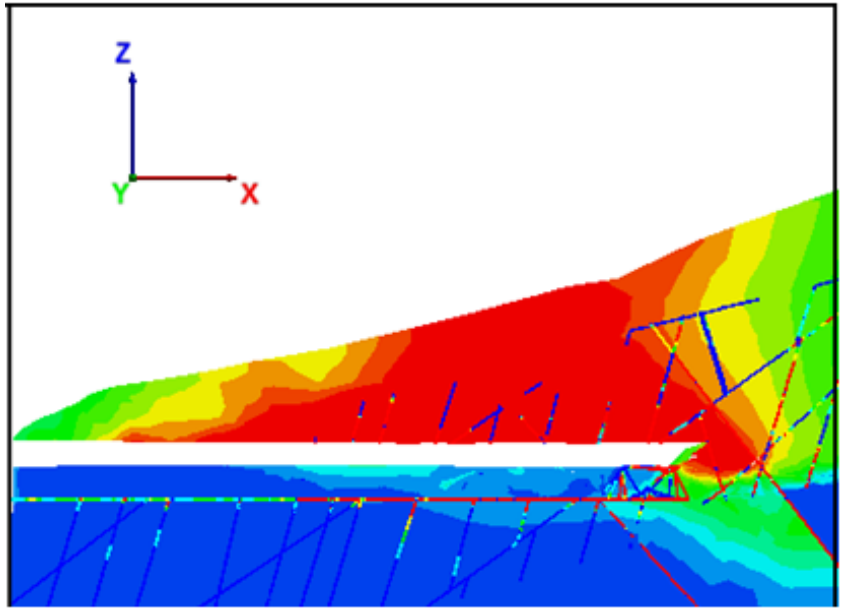
SOME OF THE HARSH 2D 'REALITY' (AS MODELLED), AND RAPID PROGRESSION TOWARDS MASSIVE FAILURE

(Dr. Stavros Bandis, UDEC)



Due to limitations of the design, retrogressive failure back to the portal. 140 m lost.





3DEC modelling by

Dr. Stavros Bandis

THEN THE LEFT-SIDE.....FIRST THE RIGHT-SIDE
(≈ 140 m COLLAPSE) .....(≈ 140 m COLLAPSE)

SOIL, THEN SAPROLITE, THEN WEATHERED PHILLITE, DIKE

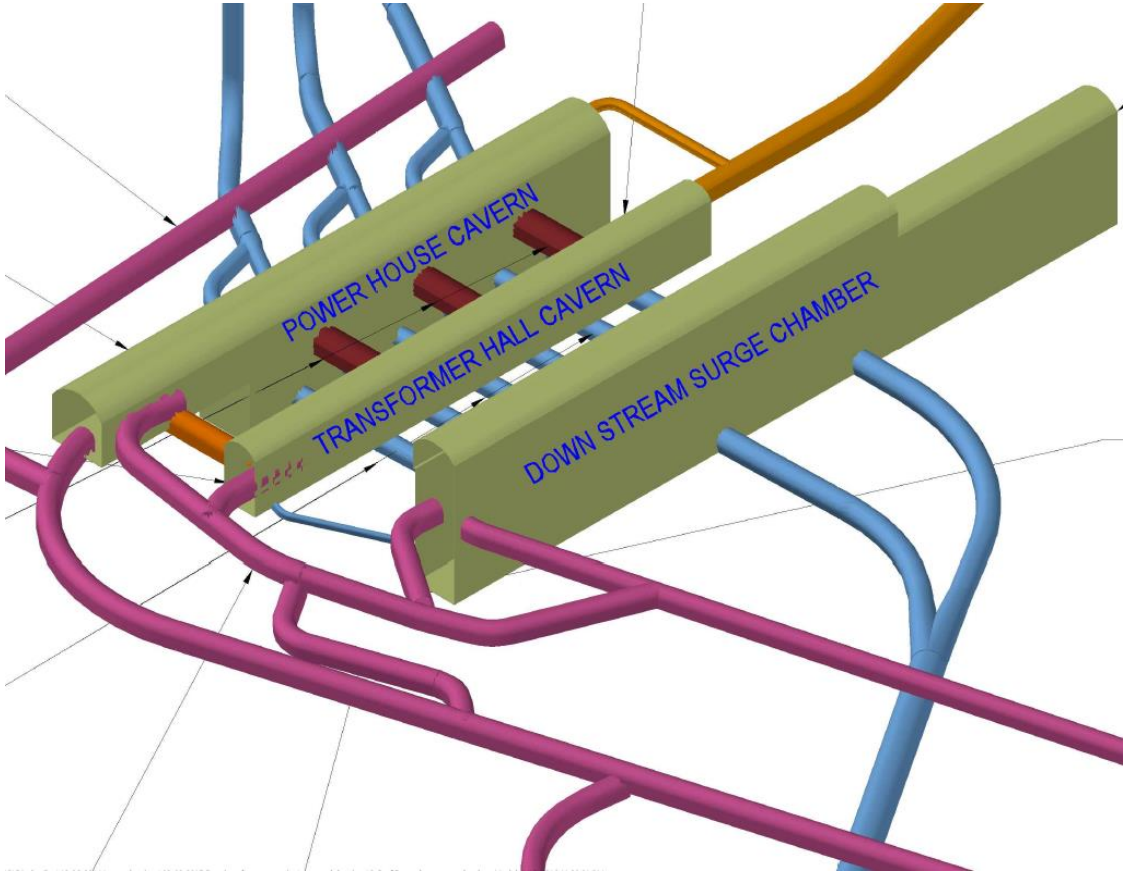
ROTATED ROAD, TREES AND TELEGRAPH POLE (after failure of second tube)

COLLAPSE # 3

CAVERN ARCH WITH STEEL SETS

(with no change of support when encountering fault zone on one side of the arch)

COLLAPSE IN PARTLY COMPLETED D/S SURGE CHAMBER ARCH.



- Tragically, six workers caught in the sudden collapse.
- First collapse ≈ 35,000 m3

The attempt to remove some 15,000 m^3 of the fallen rock, revealed the destruction of the 'heavy' steel sets in the arch.
(It was too dangerous to continue)

A total of 70,000 m^3 now fallen. A cavity with approximate dimensions L x H x W of (50-60)m x (40-50)m x (30-35)m has to be stabilized, then victims recovered.

SEVERE LIMITATIONS OF LATTICE GIRDERS AND STEEL SETS – IN OVER-BREAKING ROCK

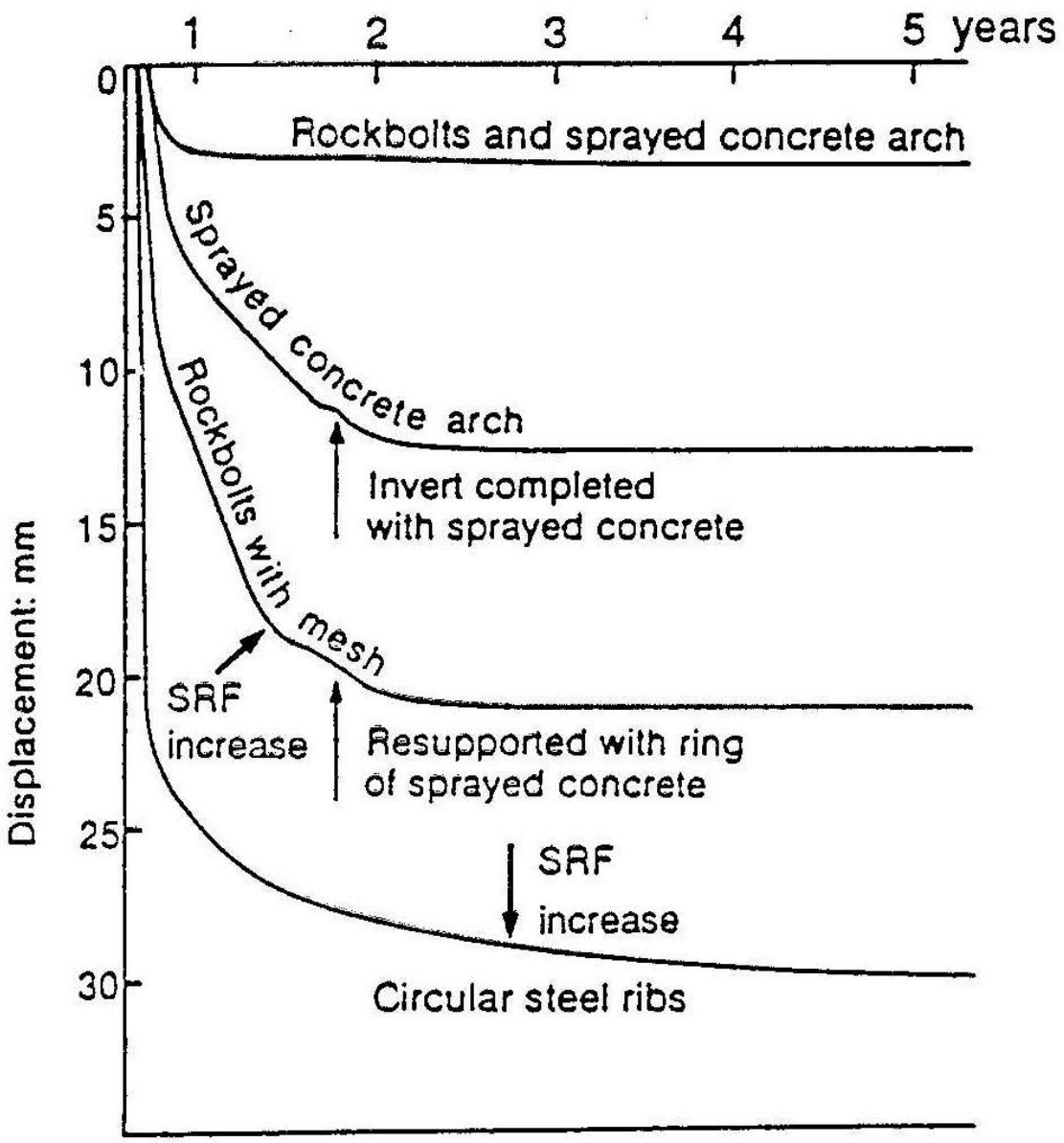
WHAT HAPPENS TO LATTICE GIRDERS (or steel sets) WHEN A TUNNEL CROSSES A MAJOR JOINT OR MINOR FAULT?

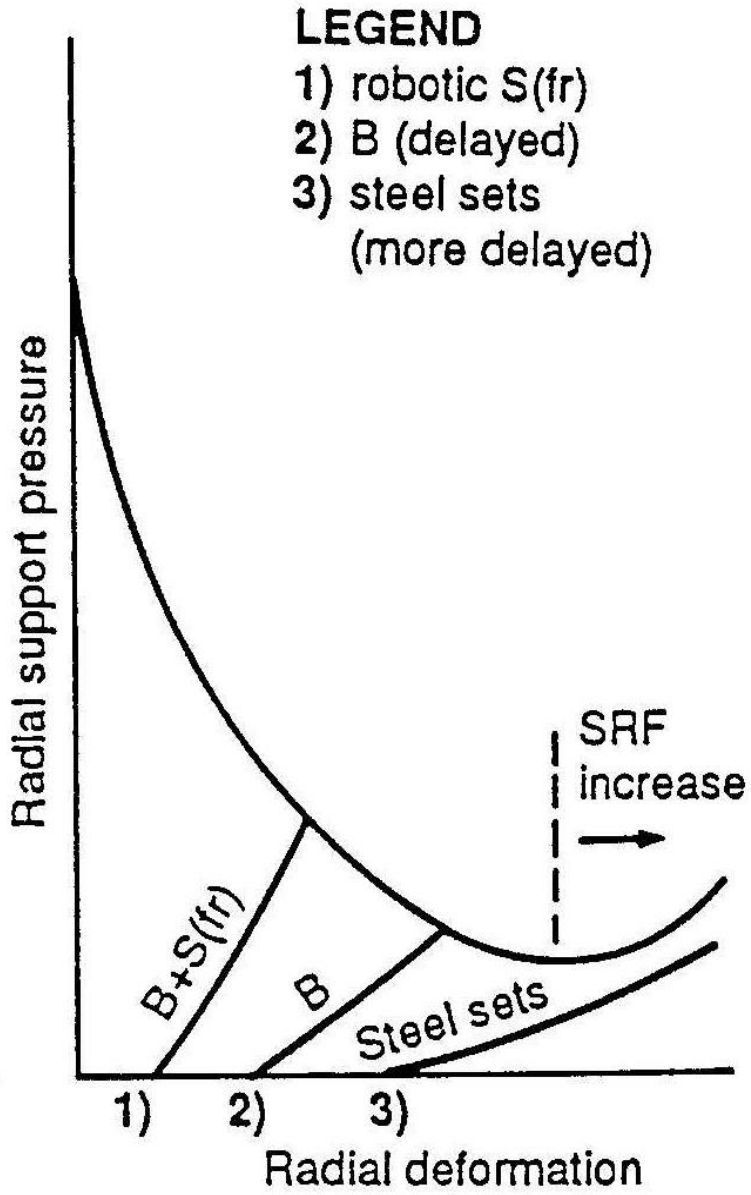
ADEQUATE CONTACT WITH THE ROCK? (IT IS NOT EASY)

1. **ADEQUATE FOOTING STIFFNESS?**
2. **ADEQUATE RESISTANCE WHEN 'POINT' LOADED?**
3. **OWN DEFORMABILITY?**

(ACTUALLY A VERY 'SOFT' SYSTEM)

UNSAFE WHEN WAITING FOR THE LINING.

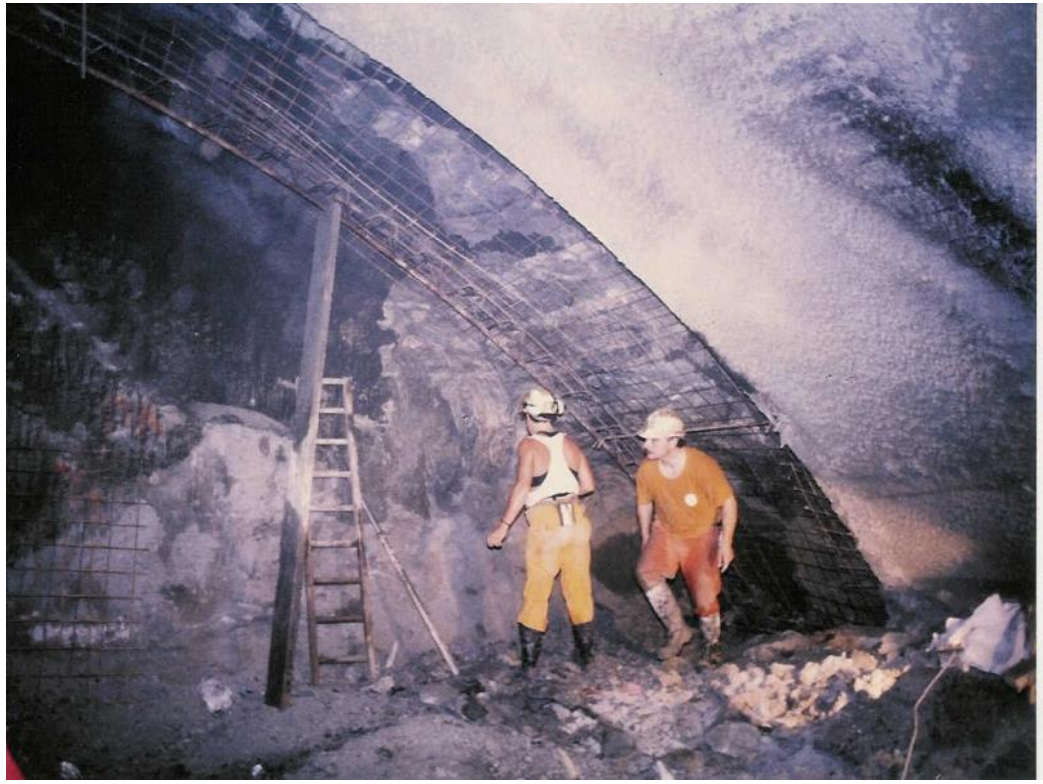




LEFT: Ward et al. 1983.
RIGHT: Barton and Grimstad, 1993.

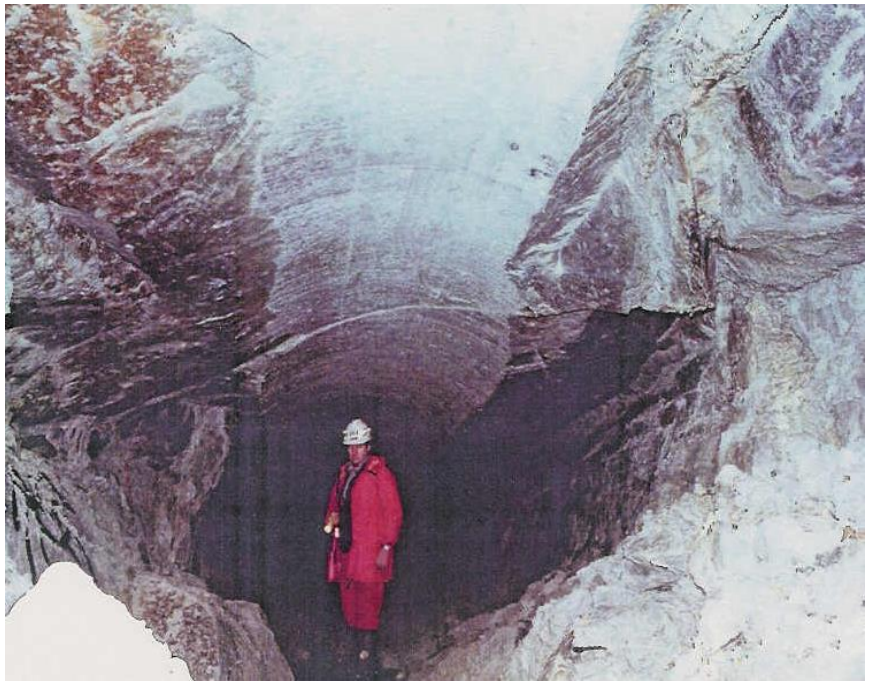
FINAL STAGE OF NATM – SHOWN BELOW – IS THE PERMANENT CONCRETE LINING.

AN EARLIER (TEMPORARY SUPPORT) PHASE OF THE SAME PROJECT.....NOT VERY CONVINCING LEVEL OF SAFETY. NOTE MESH AND LATTICE-GIRDER.



TUNNEL FAILURES AT *SMALLER SCALE* (IN MASSIVE ROCK)

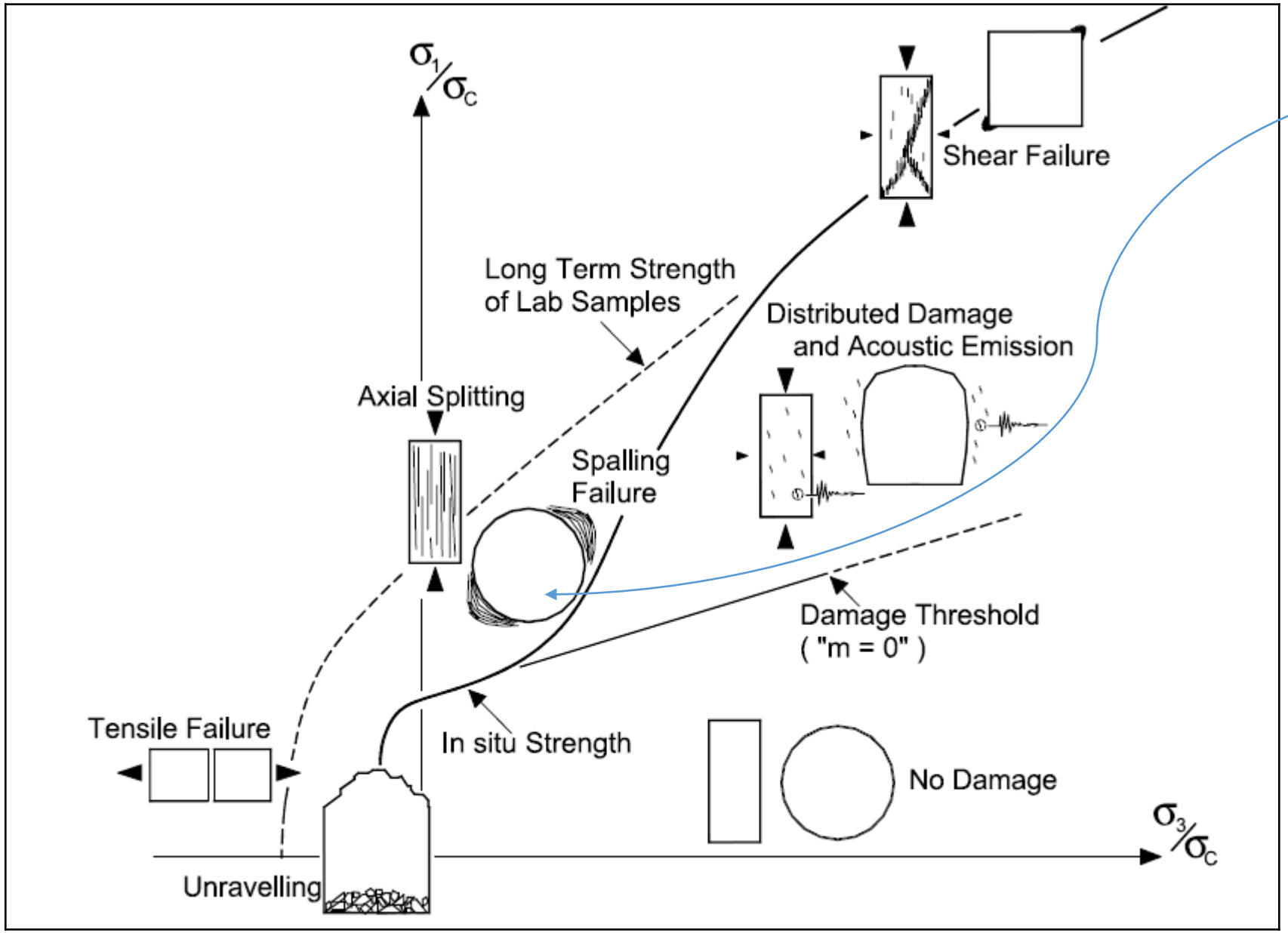
SHEAR STRESS (or extension-strain?)
INDUCED FAILURE?



WHAT DO WE (AUTOMATICALLY) THINK ARE THE REASONS FOR THESE ROCK FRACTURING EVENTS? (STRESS/STRENGTH?)

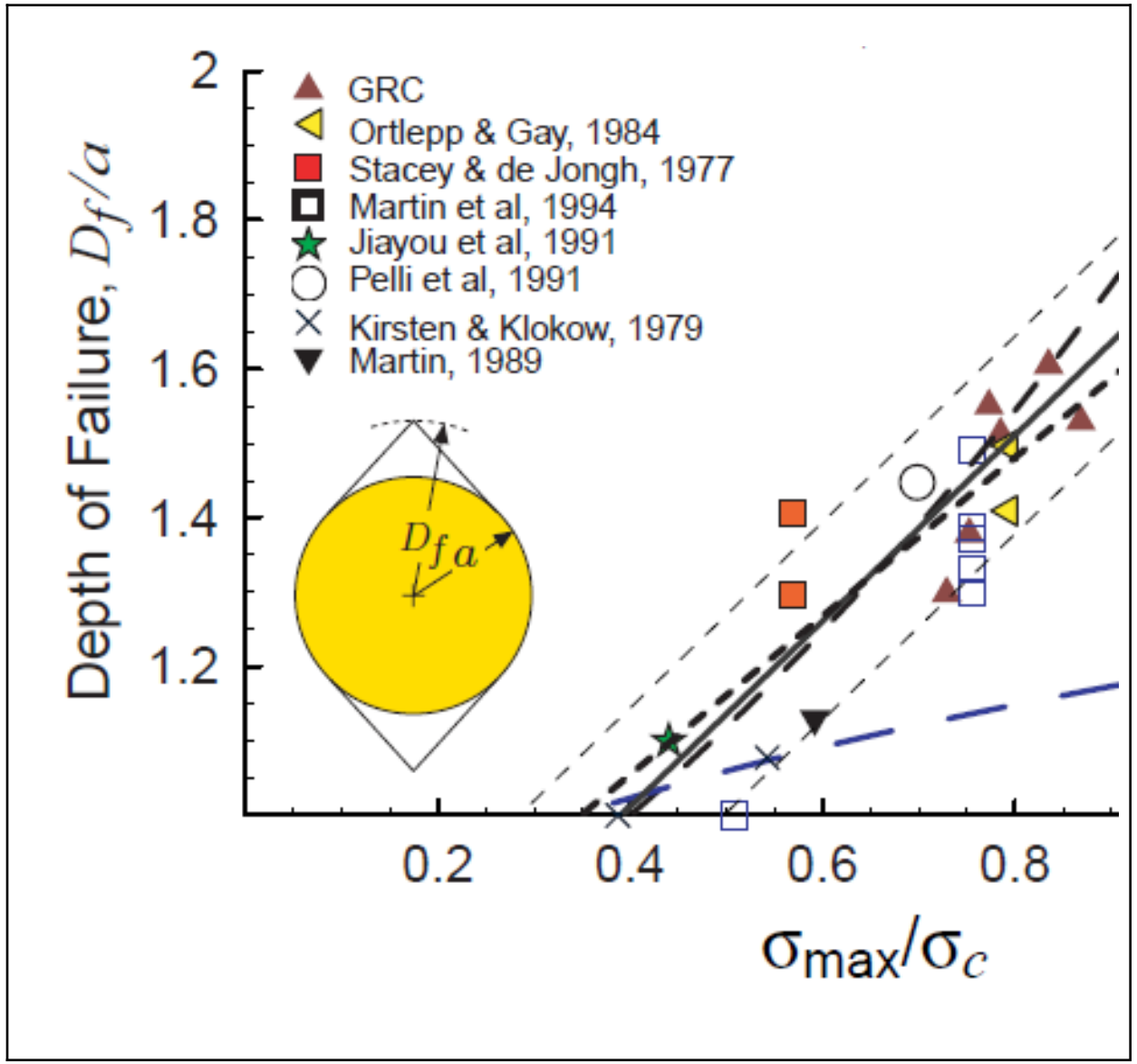
- **Top: Beaumont Tunnel, D = 2.2 m diameter, 1880 (yes!), chalk marl, UCS ≈ 4 to 9 MPa, $\Delta\sigma_\theta$ due to increase in vertical depth when passing under 70m chalk cliff ≈ 2 MPa.**

- **Bottom: Massive basalt, UCS ≈ 200 MPa, but extreme $K_0 = \sigma_h/\sigma_v \approx$ 20-25 (yes!) due to stress concentration in near-surface ridge (See later screen). 2-3m break-out (14 x 16m tunnels)**



(From Diederichs, 2003)

σ_t/v effect where 'Spalling Failure' (following Baotang Shen logic – itself based on earlier Dick Stacey insight from 1981)



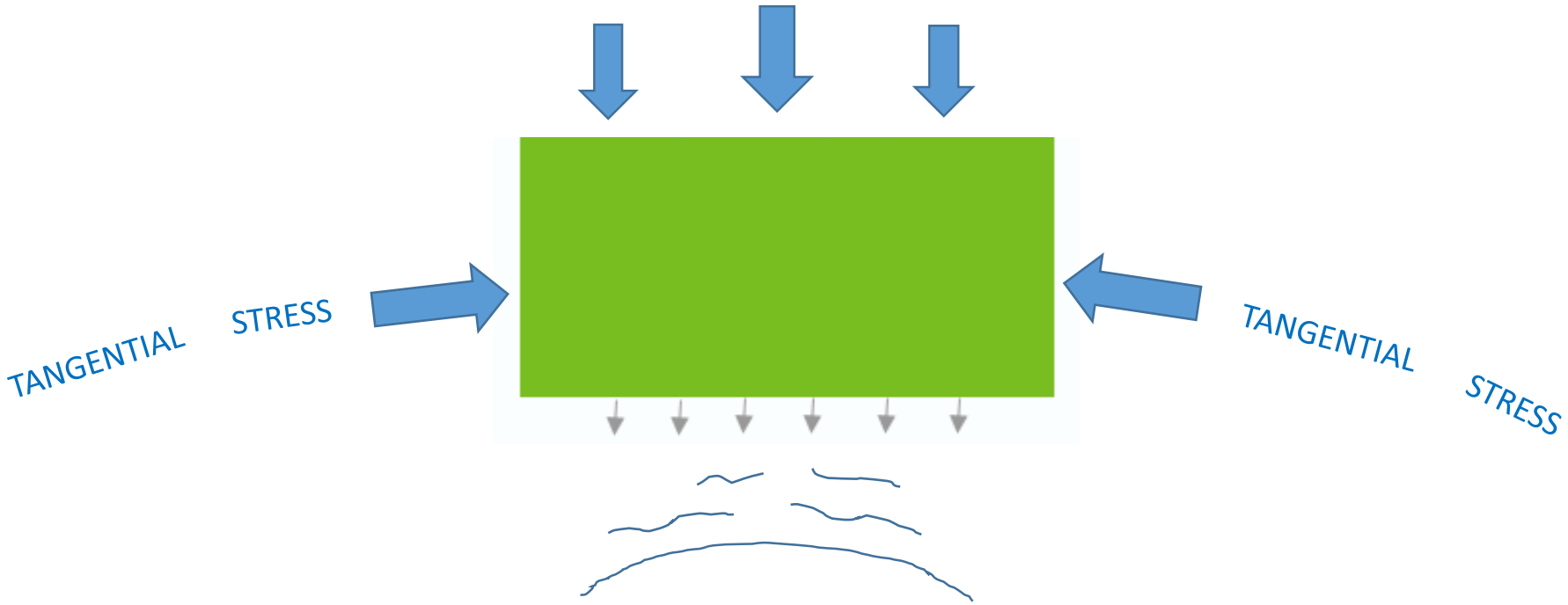
Simplified (from Martin et al. 2002)

IN Q-SYSTEM, SAME EXPECTATION. If $\sigma_{\theta\,max}/\sigma_c > 0.4$,
we need: high SRF – and lower Q-value – i.e. more tunnel support.

(σ_c = UCS unconfined compression strength)
(Table 6b of Grimstad and Barton, 1993)

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** effect causes lateral strain



NEXT TO THE TUNNEL MAY GET TENSILE CRACKING

– EVEN WHEN ALL STRESSES ARE STILL COMPRESSIVE

Two-dimensional equation applying to tunnel cross-section:

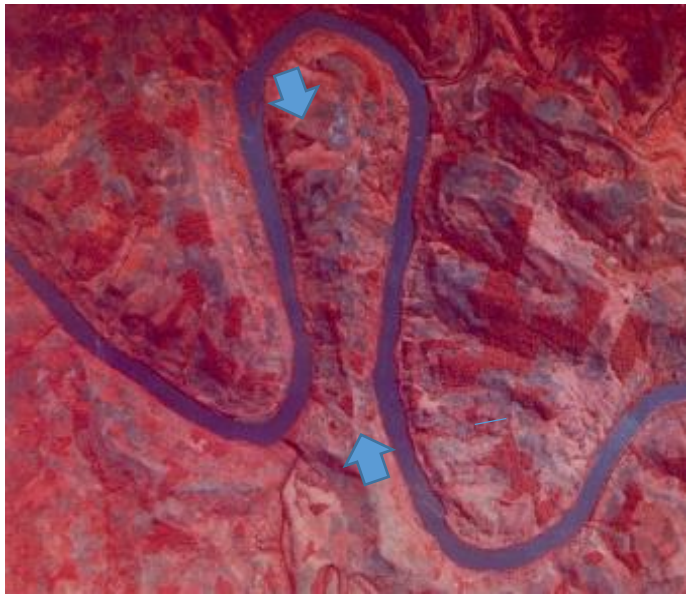
$\varepsilon_3 = [\ \sigma_3 - \nu.\sigma_1\ ]\ /E$ (after Stacey, 1981)

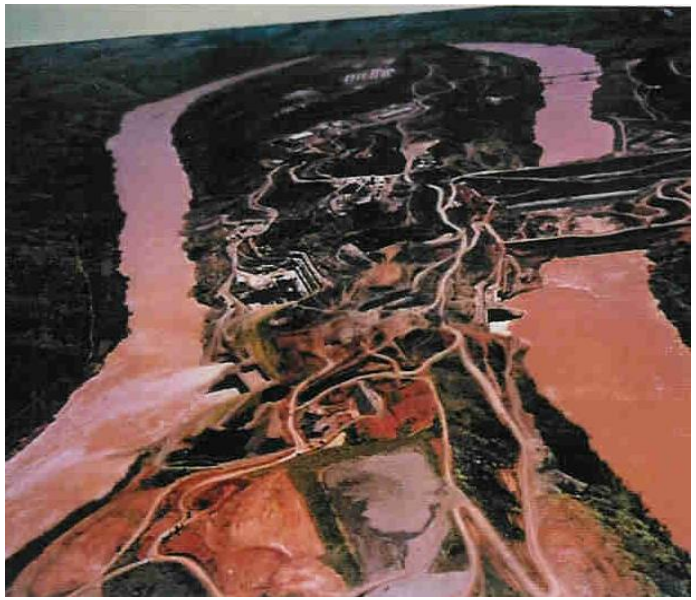
Tensile fracturing may occur if $\nu.\sigma_1 > \sigma_3$ if resulting $(-)\varepsilon_3 > -\varepsilon_c$.

$(-)\varepsilon_c = \sigma_t/E$ (This is the critical strain: strength σ_t just exceeded).

At (circular) tunnel wall, $\sigma_3 = 0$, and $\sigma_1 = \sigma_{\text{max. tang. stress}}$

$(-)\ \sigma_t/E = \varepsilon_c = (-)\ (\nu.\ \sigma_{\text{max. tang. stress}})/E$..... $\boxed{\sigma_{\text{crit. tang. stress}} = \sigma_t / \nu}$



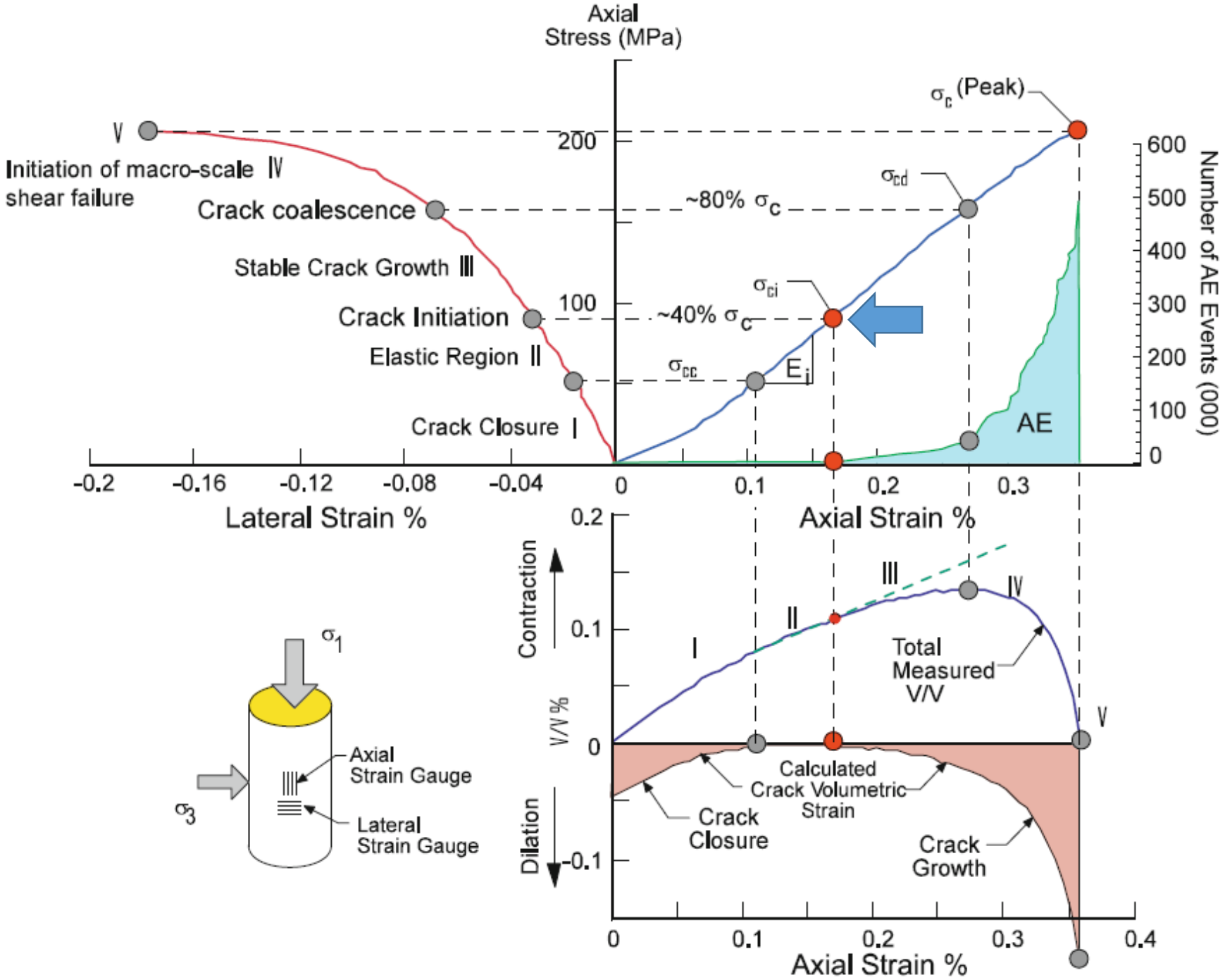


HYDRO-ELECTRIC PROJECT: ALL (10) TUNNELS (and spillway) SUFFERED ***EXTENSION CRACKING***.

$\sigma_{\theta\ max}$ /UCS ≈ 0.65-0.7 (from 2.5-3m break-out) ($\sigma_{\theta\ max}$ ≈ *130-140MPa)*

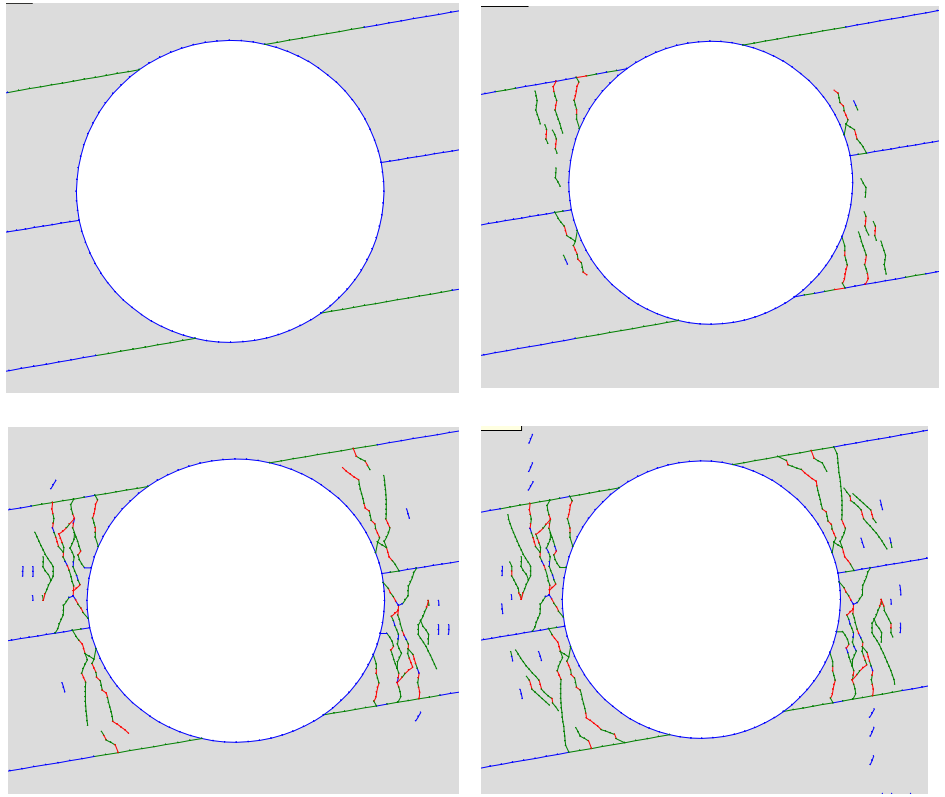
UCS OF BASALT ≈ **200 MPa**)

(BUT CRITICAL $\boldsymbol{\sigma_t / \nu}$ ≈ 20/0.25 ≈ *80MPa*. THIS WOULD BE START OF ***ACOUSTIC EMISSION, (i.e. 'POPPING') AND EXTENSION CRACKING***)



CANADIAN TRIAXIAL TESTS ON LAC DU BONNET GRANITES (from URL)

Martin's (1997) results show crack initiation and accelerated acoustic emission at about ***40% of the peak value of σ_c.***



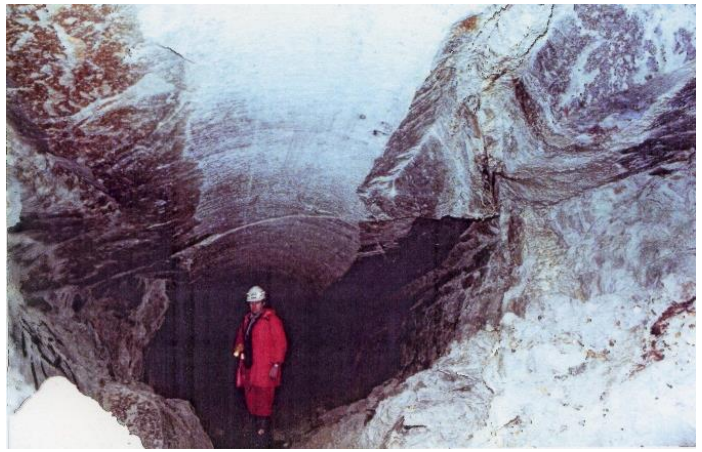
INITIAL CRACKING IS ACTUALLY CAUSED BY EXCEEDING THE ***CRITICAL EXTENSION STRAIN:***

Cracking in tension, then shear:

(Not ‘compression’ failure)

(Baotang Shen in : Barton and Shen, 2017)

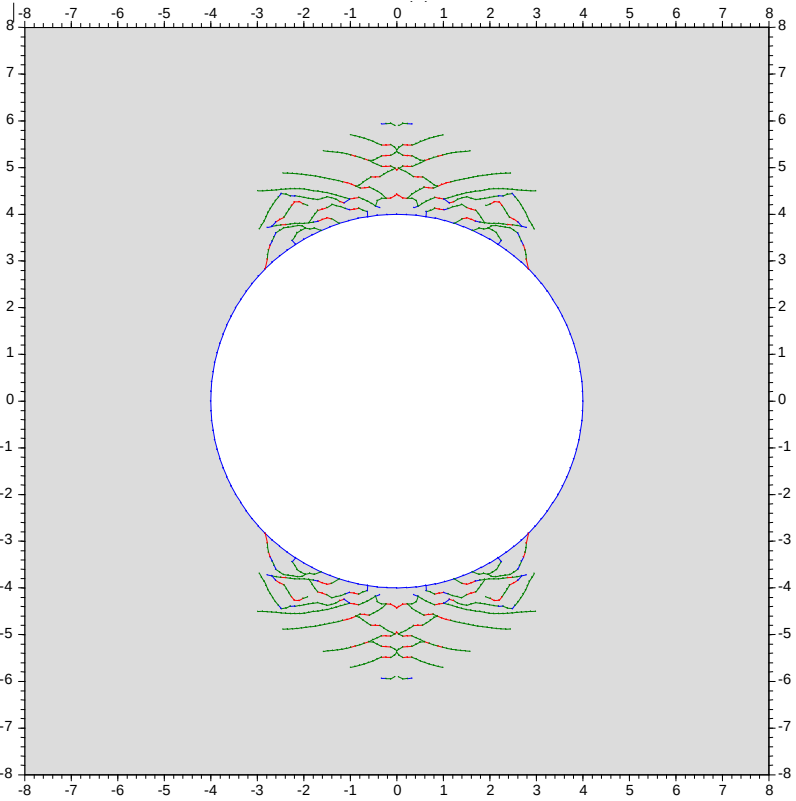
$$\sigma_{critical\ tangential\ stress} \approx (0.4 \times UCS) = \sigma_t / \nu$$

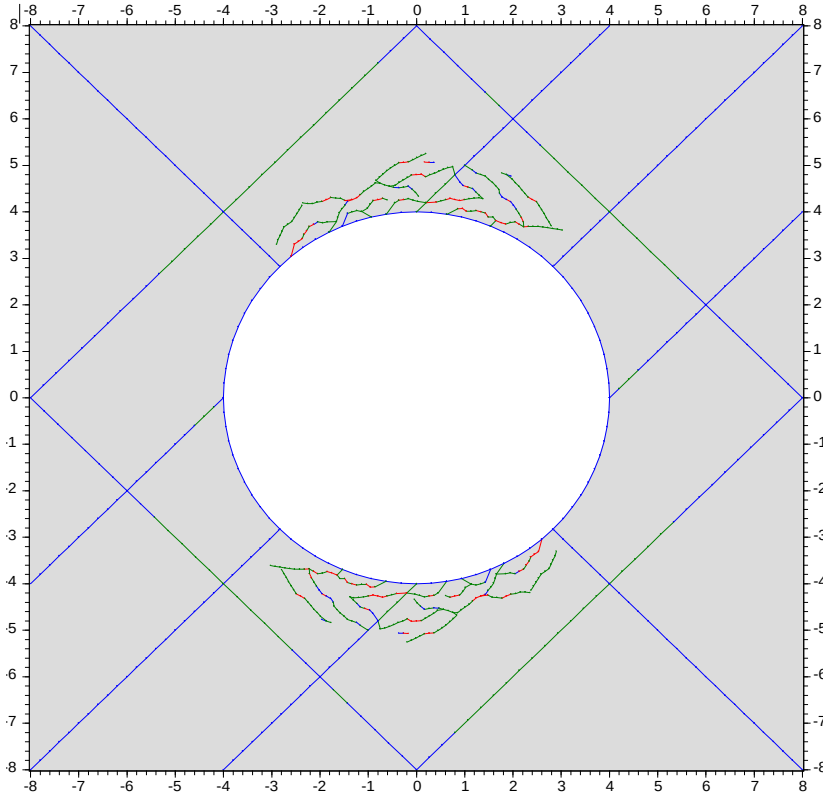


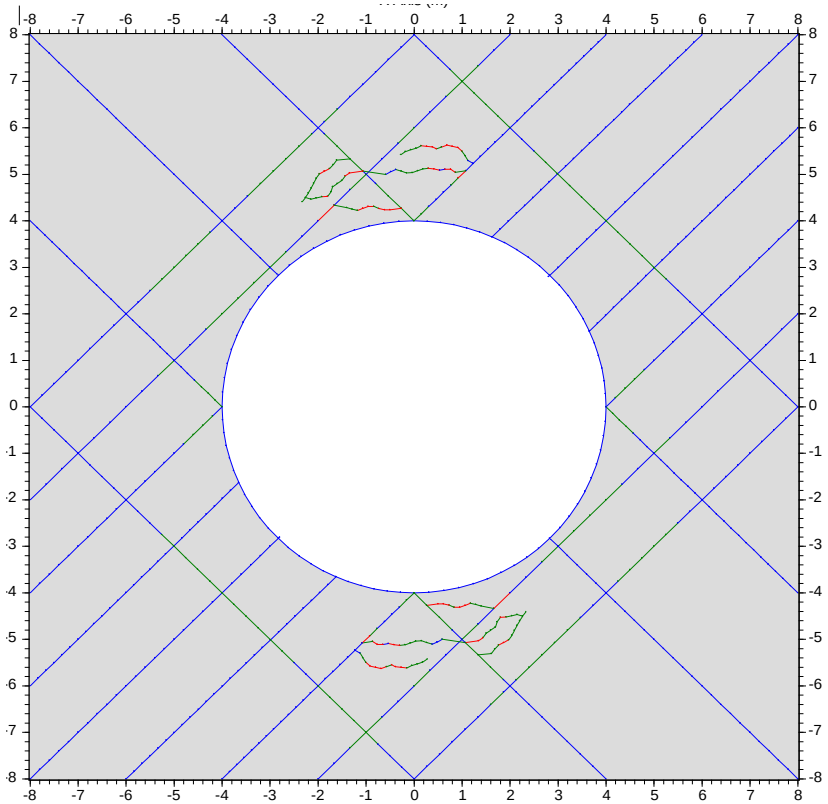
$$\sigma_t / \nu \approx \frac{UCS}{10\nu} = \frac{UCS}{10 \times 0.25} = 0.4 \times UCS$$

Tensile fracturing, followed by propagation in shear is partially dissipated by joint sets. B. Shen FRACOD tunnel models, at 1,000m depth.

(Barton and Shen, 2017)







Red: tensile fractures

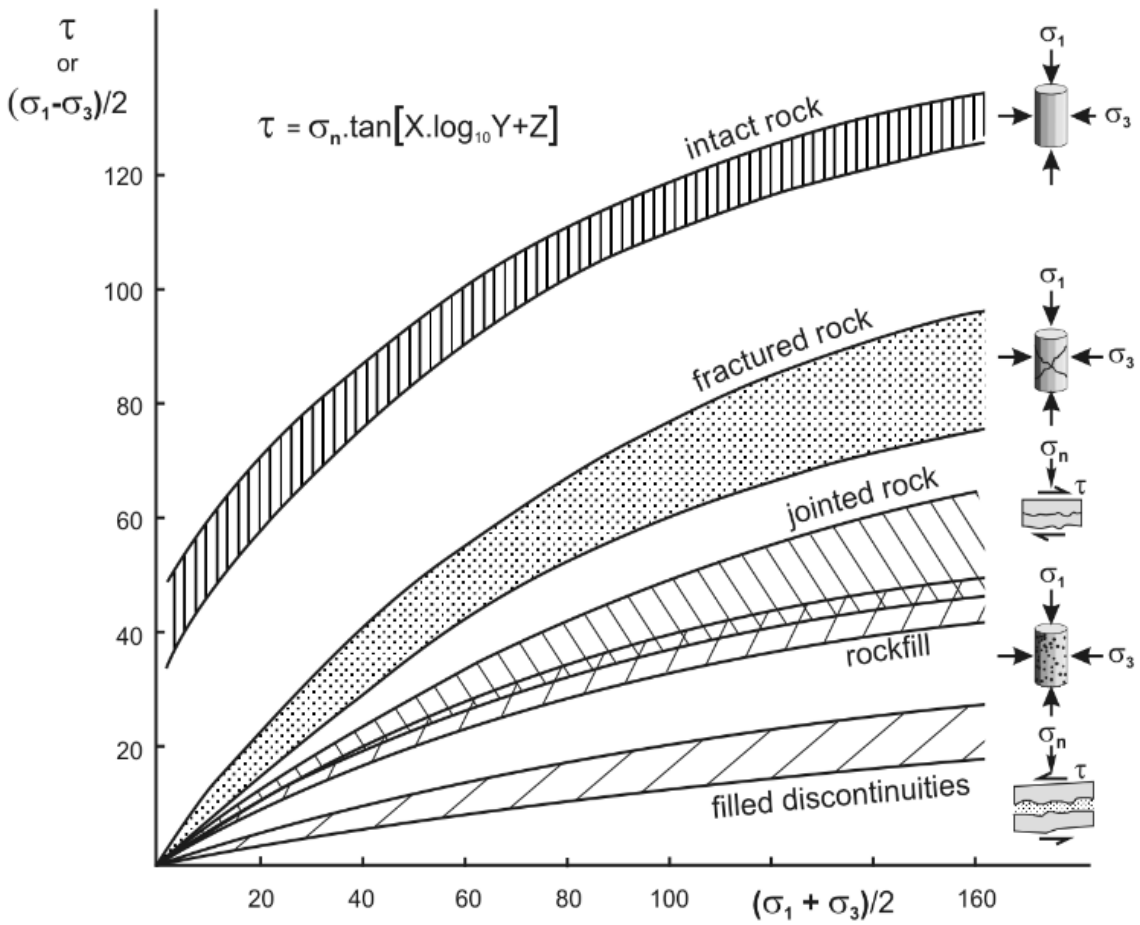
Blue: shear fractures

FINALLY A BRIEF DISCUSSION OF PROGRESSIVE FAILURE:

The shear strength of rock masses is ‘not so simple’ as M-C or H-B (linear or non-linear)

‘$c + \sigma_n \tan \phi$’

WHAT IS HELPING TO PREVENT SUDDEN COLLAPSES ?



No casualties. Monitored. Progressive failure. $\tau = c + \sigma_n \tan \phi$? **(Not +, *then!*)**

THE REALITIES OF ROCK MASS SHEAR STRENGTH

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

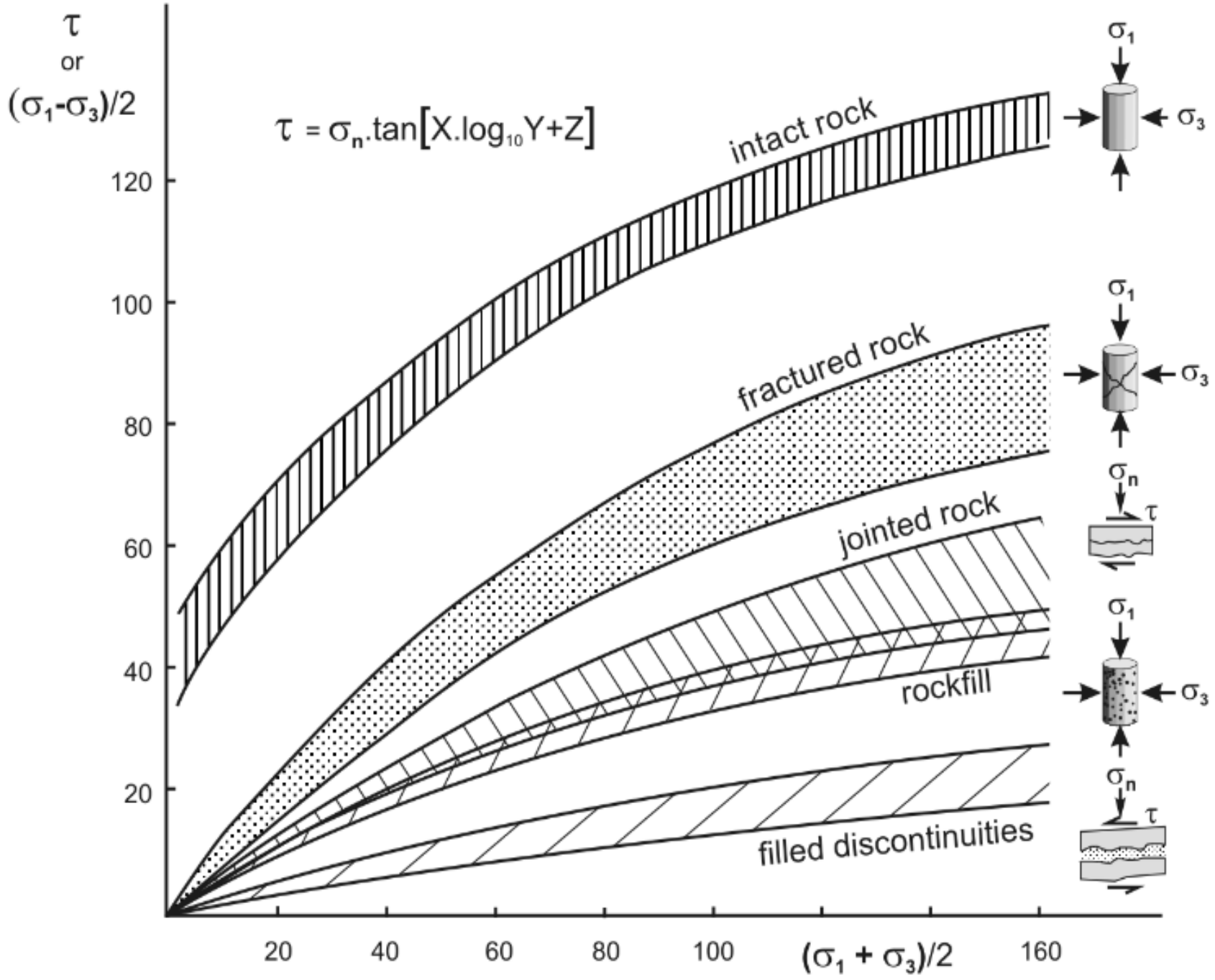
1. INTACT BRIDGES FAIL, ‘c’ fails AT SMALL ‘STRAIN’

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

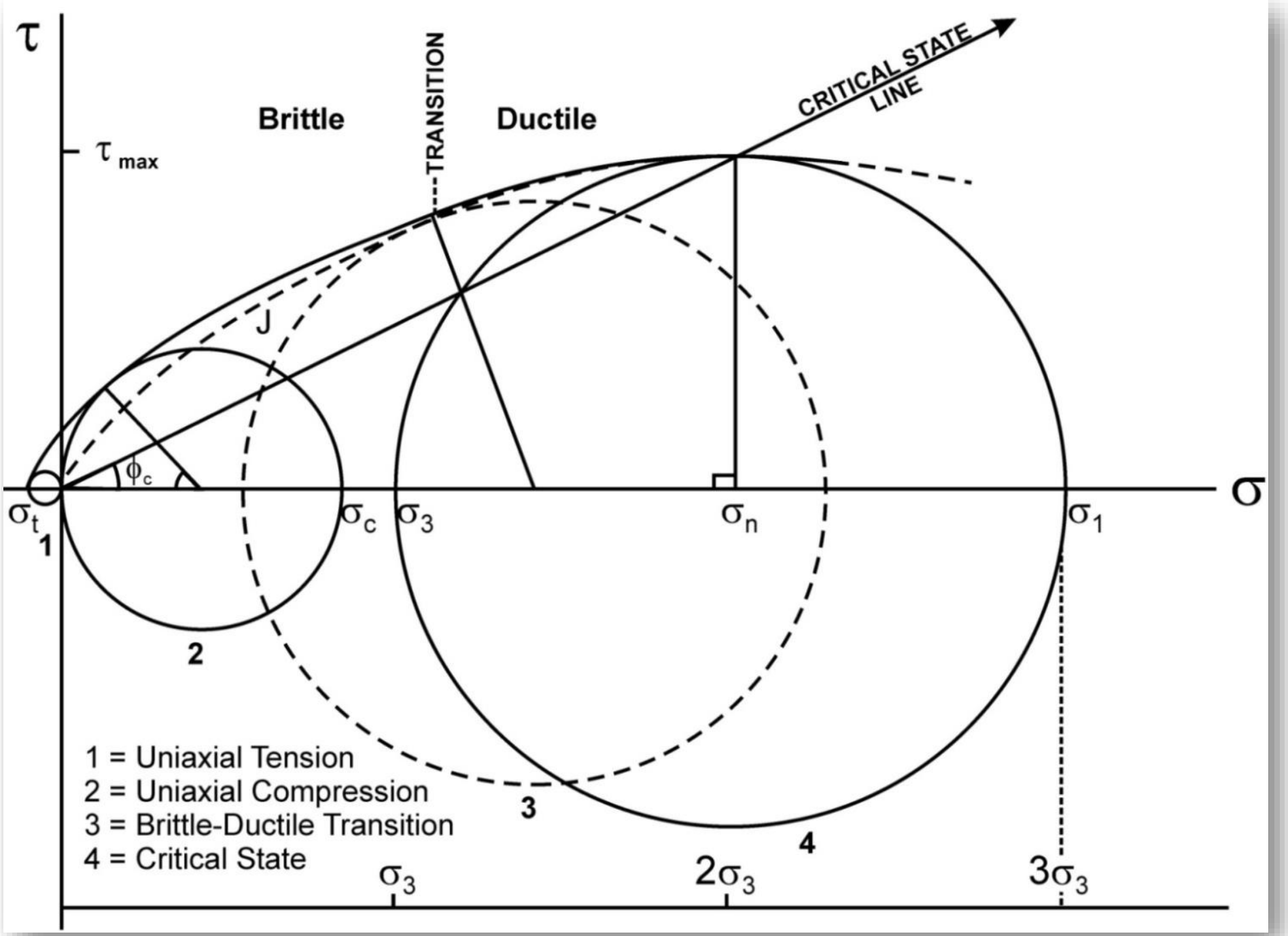
3. SURROUNDING NATURAL JOINTS with lower JRC, JCS, φ_r, SHEAR NEXT AT STILL LARGER ‘STRAIN’

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

SO PROGRESSIVE FAILURE IS LOGICAL!



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	$\left(\frac{\sigma_1 - \sigma_3}{\sigma_n}\right)$	ϕ_b
or	$\left(\frac{JCS}{\sigma_n}\right)$	ϕ_r
R	$\left(\frac{S}{\sigma_n}\right)$	ϕ_b
$[J_r / J_a]$ (from Q-system)		



Critical state concept allows the complete strength envelope to be defined.

Barton, 1976
Singh et al.2011
Shen et al. 2017

Failure of 70,000,000 m3 of waste rock and ore. Caused 5.1M and 4.1 M events, with 1 ½ hours separation.

Almost 3 km run-out distance. Clean-up > two years.

No loss of life due to monitoring, but a lot of buried dumper-trucks.

VISUAL LOGIC SUGGESTS THAT A ’CORNER’ HAS BEEN INVOLVED IN THIS LARGEST EVER OPEN-PIT FAILURE.

SO A LACK OF TANGENTIAL STRESS AROUND THE PIT.

CASA GRANDE, ARIZONA.....NOTE DESIRABLE CIRCULARITY

CONCLUSIONS

1. It is apparent that really large failures are caused by a multitude of adverse factors working together, seldom by just one or two over-sights. Small failures might be caused by designer or contractor short-cuts, or more likely by failure to log ‘today’s’ conditions, and react with more support. This emphasises the basic safety of NMT single-shell principles – as compared to ‘industrial’ tunnel support concepts.

2. The really big failures may need failed design, therefore inappropriate support. However, sometimes just the multiple effects of some exceptionally adverse and unanticipated ‘geology’, aided by a remarkably adverse location, prevent the needed arching, and can be the main cause. Designers and contractors can be blameless.

3. The tunnel and cavern failures illustrated amounted to 2 x 140 meters of tunnel collapse, 2 x 35,000 m^3 of progressive cavern collapse, and a 15,000 m^3 total collapse. A relative absence of sufficient tangential (arching) stress can be blamed in each case. If the rock mass shear strength or designed support are in question, massive failure may result.