Some aspects of stress, strain, strength, structure and (tunnel) support : Q and beyond

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CONTENT of LECTURE

1. Why we (obviously) need to link *stress – strain – strength – structure*?
2. Various examples of (anisotropic) S-S-S-S behaviour (slopes, tunnels)
3. Stress/strength (σ_θ/UCS) > 0.4 causes fracturing - or is it something else?
4. Extensional strain and some simple arithmetic that explains ‘ ≈ 0.4’
5. Examples of discontinuous behavior due to fracturing, and due to joints
6. (Examples from FRACOD developed by Baotang Shen, now CRIRO, Australia)
7. Practical aspects of tunnel support with Q: emphasis on why no steel ‘arches’
8. Over-break, over-stress (or over-strain), problems with anisotropic rock masses
9. Some lessons from failures / failed design concepts
10. Conclusions

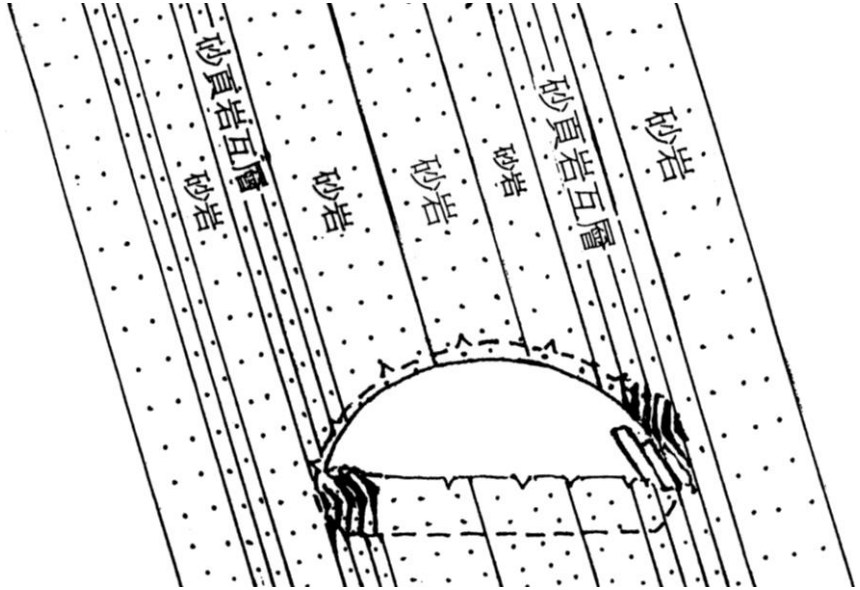
CASA GRANDE, ARIZONA. NOTE DESIRABLE CIRCULARITY. FAVOURABLE, VERY LARGE-SCALE TANGENTIAL STRESS WHICH *MAY* HELP TO STABILIZE MAJOR STRUCTURES IN MUCH THE SAME WAY AS JOINT SETS *MAY* BE STABILIZED AT TUNNEL SCALE.

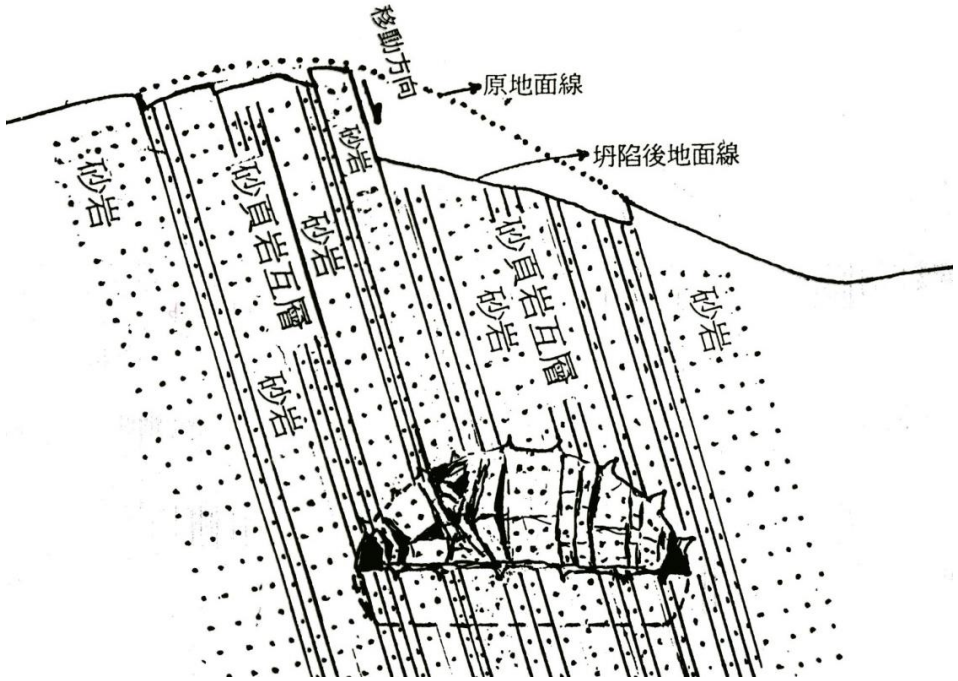
WOULD WE EVER DO ’THIS’ AROUND A TUNNEL? NO!

LACK OF CONFINEMENT AS IN THE OPEN PIT FAILURE.

IN THIS CASE: LACK OF *STRESS* CAUSING *STRAIN*, SERIOUSLY AFFECTED BY ADVERSE *STRUCTURE* WHOSE *STRENGTH* IS EXCEEDED).

THE INTERACTION OF *STRESS-STRAIN-STRENGTH-STRUCTURE* IN THE CONTEXT OF TUNNEL STABILITY (IN THIS CASE ALSO INFLUENCED BY THE STEEP SLOPE).

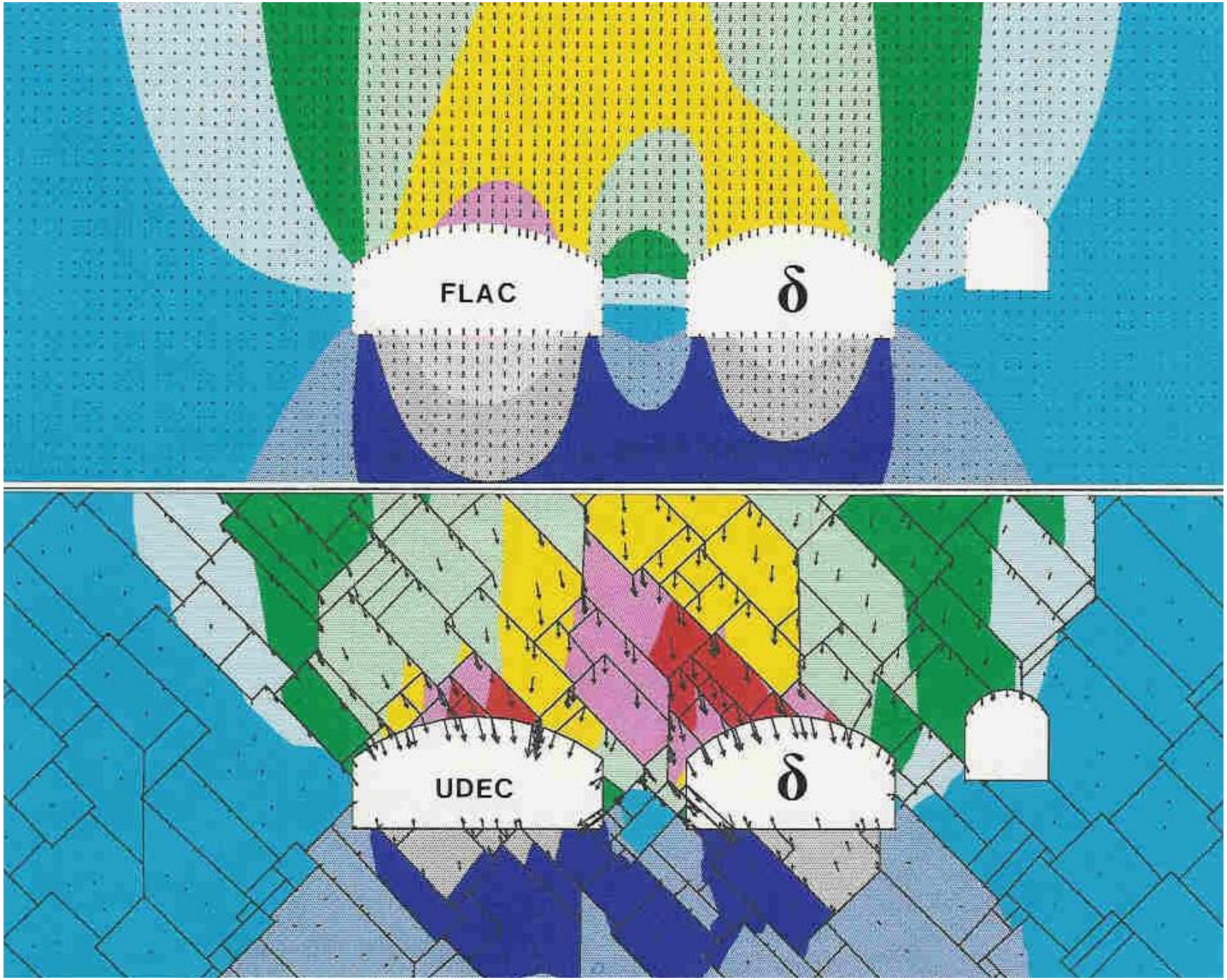




Shear *stress* – shear *strain* – causing shear *strength* of steep *structure* (bedding) to be exceeded.

(Buckling is a symptom but hardly the cause).

(A road tunnel excavation in Taiwan, from National conference in Taipei, 1996)

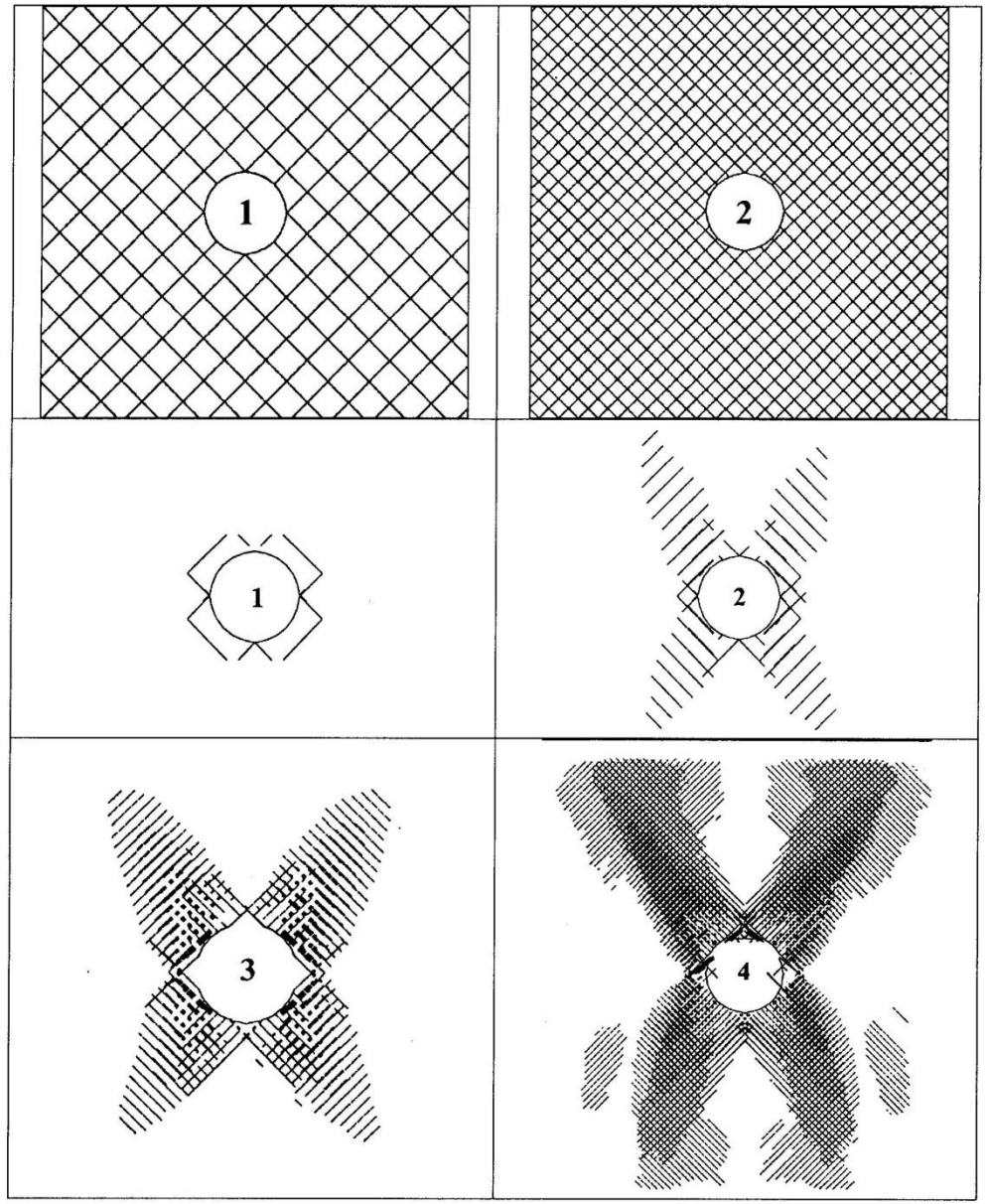


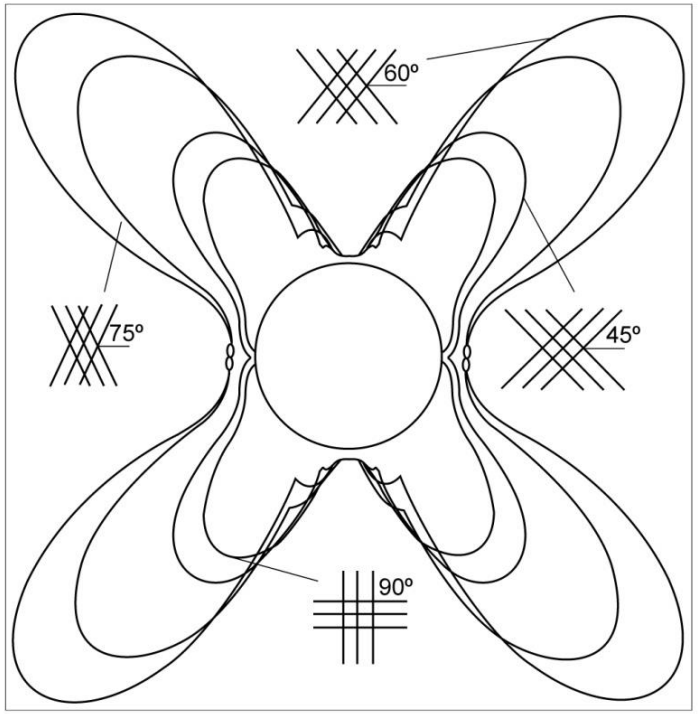
SS-S

(artificial isotropic response: no structure)

SSSS

(anisotropic response)





Reducinging the block size: shear *stress* exploits the reduced resistance to *shearing* and rotation.

Deeper-seated EDZ (and more chance of a stuck TBM)

If part of a fault zone, added stability issues due to water eroding clay/silt fillings, causing block falls, and possible TBM blockage.

(Low Q!)

(All SSSS responses)

ALL SSSS RESPONSES

(even the over-break)

THE EXAMPLES ILLUSTRATED ARE FROM BRAZIL AND CHINA.

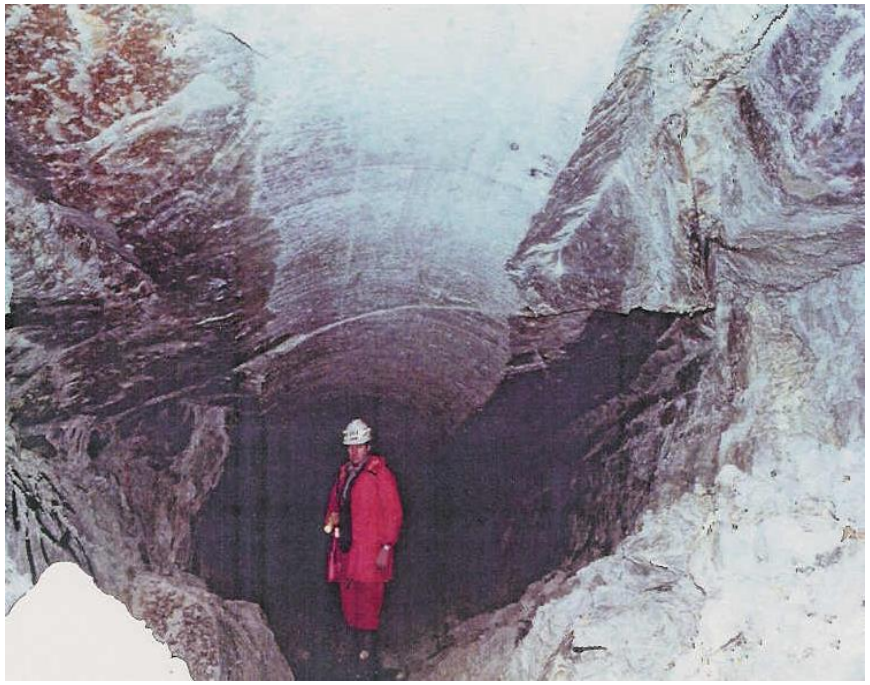
THREE OF THE CASES INVOLVE SHEAR ZONES OR FAULTS.

TUNNEL EXCAVATION CAUSED SHEARING AND WORSENING STABILITY, OR LOCAL TUNNEL COLLAPSE.

NOW SOMETHING SURPRISING

Max. tangential stress/strength (σ_θ/UCS) > 0.4 causes fracturing –

or is it related to something else ?

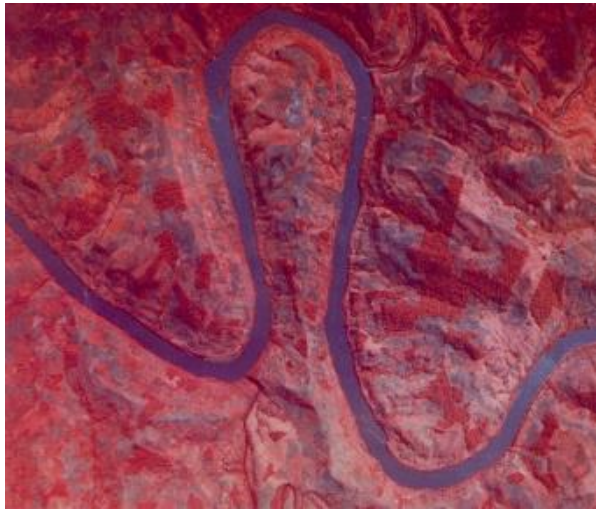


WHAT DO WE (AUTOMATICALLY) THINK ARE THE REASONS FOR THESE ROCK FRACTURING EVENTS? (STRESS/STRENGTH?)

- **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.**

- **Bottom: Massive basalt, UCS ≈ 200 MPa, but extreme K_0 ≈ 20-25 (yes!) due to stress concentration in near-surface ridge (See next screen).**

SOME OBSERVATIONS OF (STRESS-INDUCED?) FAILURE



ITÁ 1,450 Mw POWER PLANT, BRAZIL.

ALL TUNNELS CROSSING RIDGE:

- **FIVE DIVERSION AND FLOOD-CONTROL TUNNELS 15 X 17M AND 14 X 16M DIMENSIONS.**
- **FIVE 9M DIAMETER 53° INCLINED PRESSURE TUNNELS.**
- ***ALL SUFFERED STRESS-INDUCED (OR EXTENSION-STRAIN-INDUCED) FRACTURING.***

EXAMPLE: 2 TO 3 M DEEP BREAK-OUT IN THE ARCH AND EVEN MORE IN THE INVERT DUE TO EROSION EFFECTS.

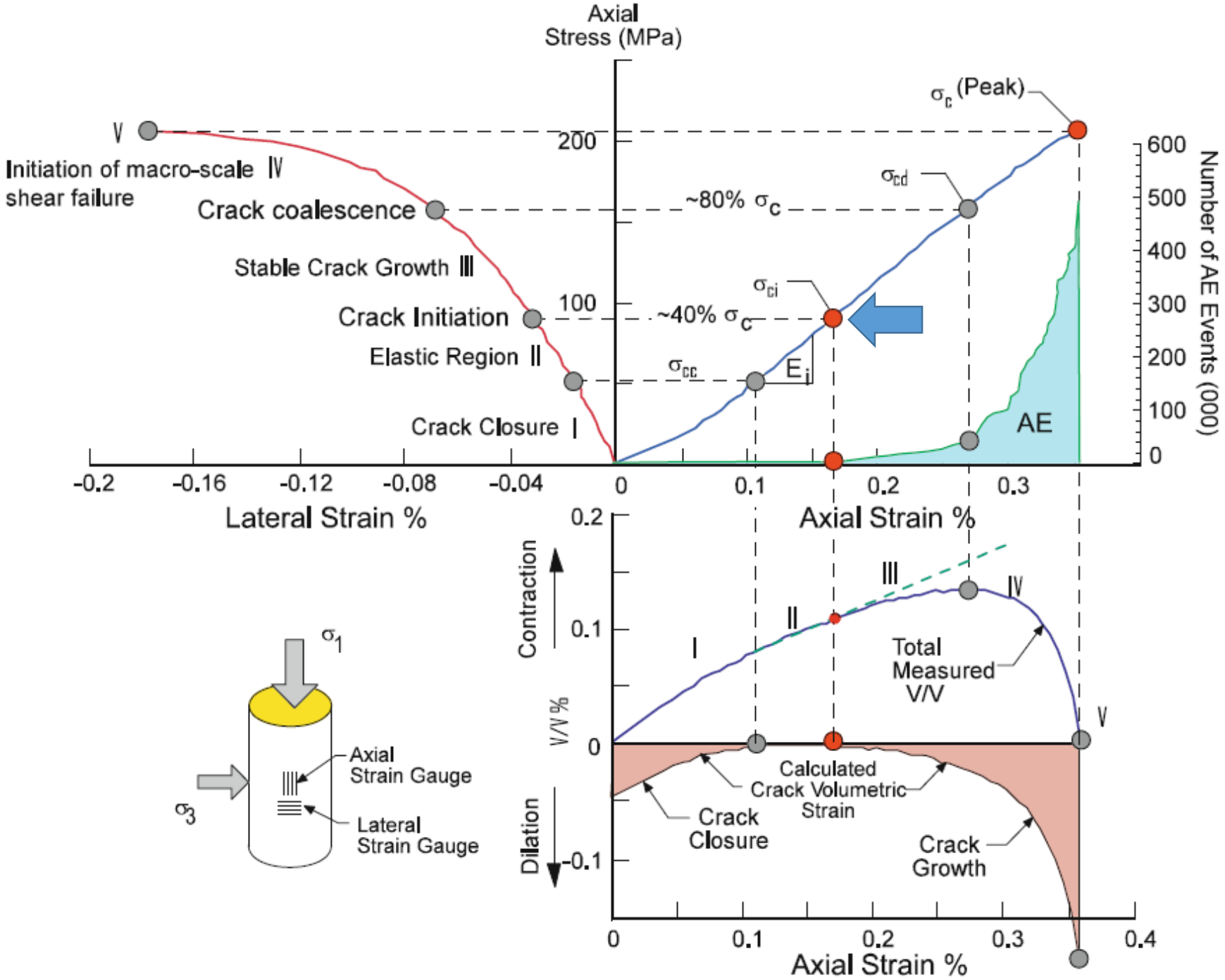
THE PREVIOUSLY ASSUMED 'STRESS-INDUCED' FAILURES (initiating at about $\sigma_\theta / \sigma_c \geq 0.4$) MAY ACTUALLY BE EXTENSIONAL-STRAIN-INDUCED TENSILE FAILURES

POISSON'S RATIO EFFECTS (AND LIMITED TENSILE STRENGTH) ARE RESPONSIBLE FOR THE *TENSILE FAILURE INITIATION*, EVEN THOUGH THE MORE DRAMATIC (FINAL) FAILURE PROCESS MAY PROPAGATE IN (THE UNSTABLE) *SHEARING MODE*

(See later FRACOD modelling by Baotang Shen).

THE ’0.4’ *MYSTERY (?)*

i) Lab-scale
ii) Tunnel-scale
iii) Extension / shear failure



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.***

IS FAILURE DUE TO ADVERSE STRESS/STRENGTH RATIOS ?

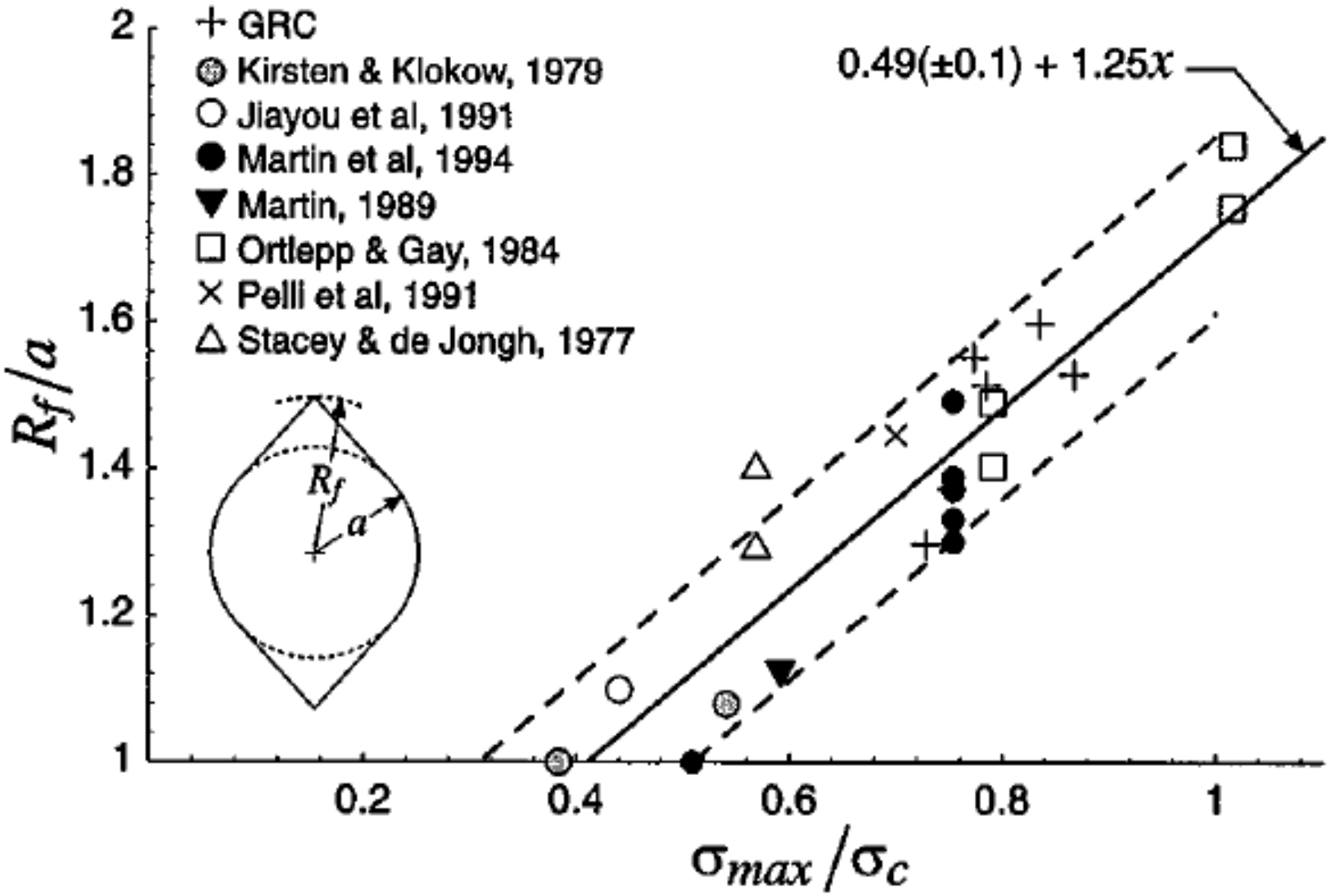
(THIS HAS BEEN ASSUMED FOR MANY YEARS – BY MANY of US)

note 'acceleration' of SRF beyond σ_θ/σ_c > 0.4 to 0.5

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

THIS IS THE Q-SYSTEM SRF TABLE FROM GRIMSTAD AND BARTON, 1993.

IN BARTON ET AL. 1974 WE USED THE RATIO OF σc /σ1, OR σt /σ1 TO ESTIMATE <u>SRF</u> FOR 'STRESS-INDUCED' FRACTURING.



Mining and nuclear waste related case records.

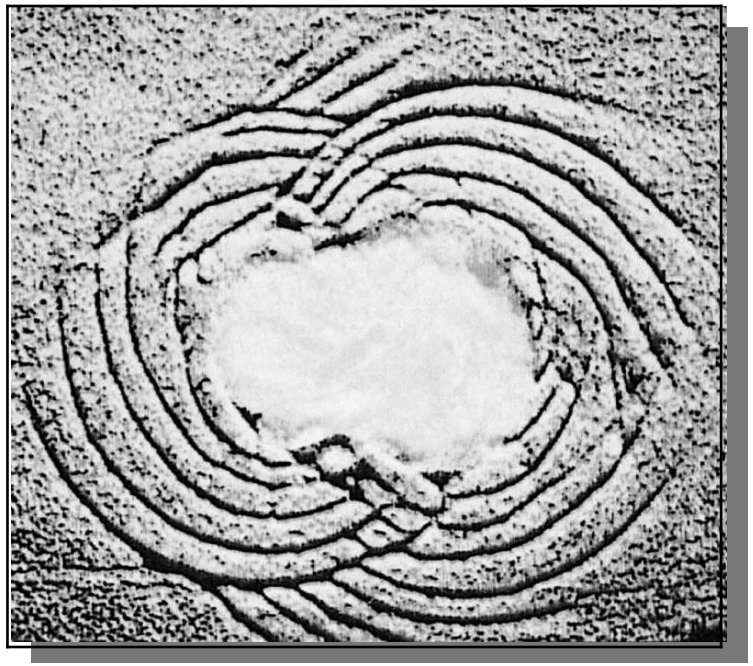
Depth of failure also linked to ratio of max stress to strength (σ_θ/σ_c)

(Martin et al. 1997)

NOTE ONSET OF OVER-BREAK WHEN $\sigma_\theta/\sigma_c \geq 0.4$. WHY 0.4?

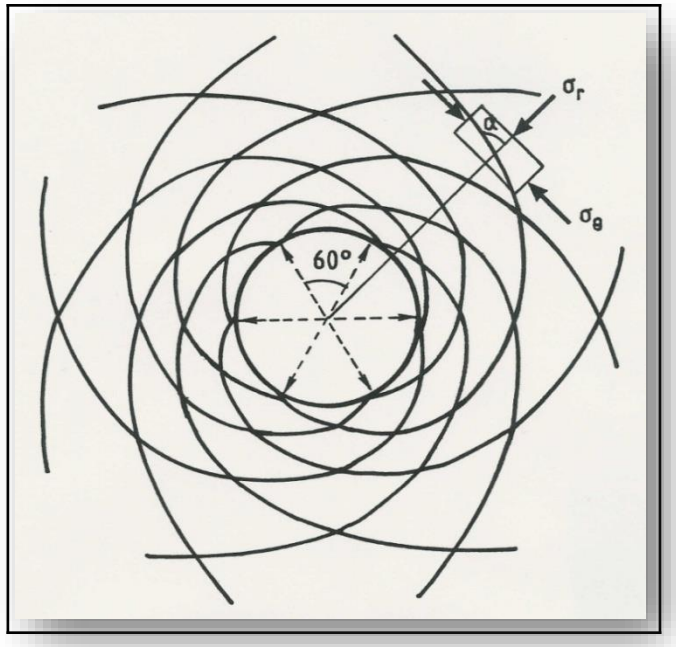
SCALE EFFECT?.....NO!!

IN THE CASE OF HEAVILY OVER-STRESSED WEAK ROCKS, SUCH AS WELL-BORES WITH LOCALLY INSUFFICIENT MUD-PRESSURE, *ONE MAY ONLY EXPERIENCE SHEAR FAILURE MODES*: TYPICALLY LOG-SPIRAL SHEARING (e.g. BRAY, 1967)

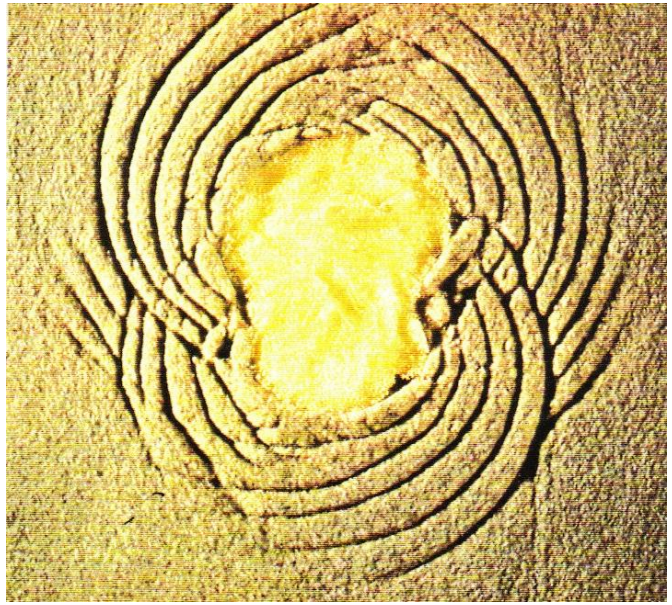


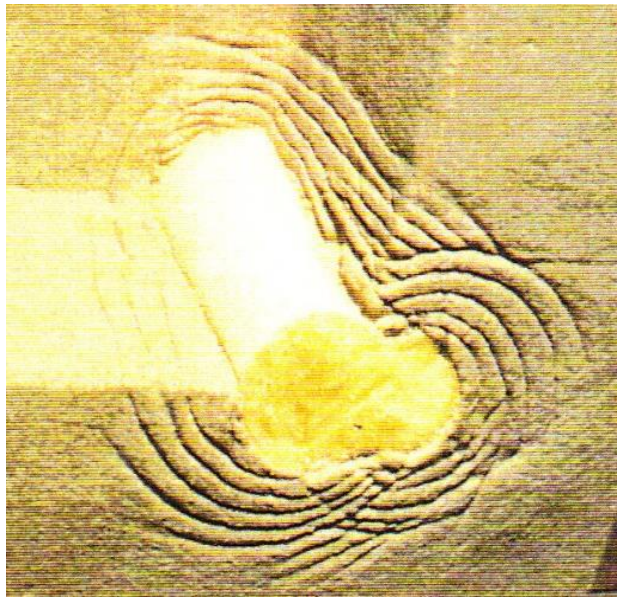
Borehole stability studies at NGI.

(Joint Industry Project). See Addis et al., SPE, 1990.

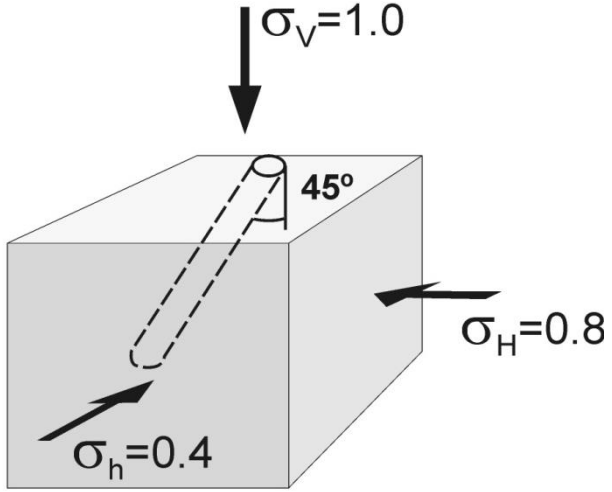


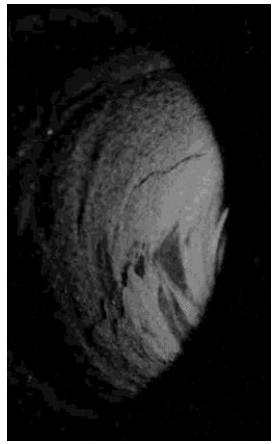
Drilling into σ1 > σ2 >σ3 loaded cubes (0.5 x 0.5 x 0.5 m) of very uniform *model sandstone* which was under high stress.

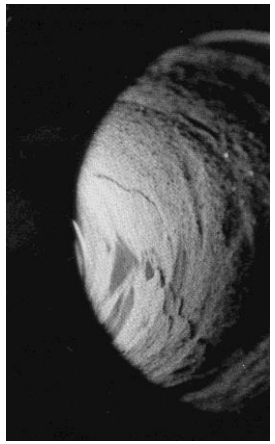


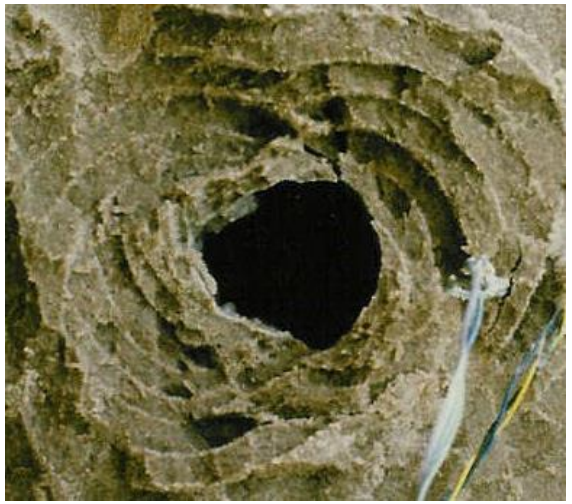


FROM NGI JOINT (OIL COMPANY) RESEARCH. NOTE THAT HOLES WERE *DRILLED*.
THESE ARE NOT *NOT 'LOADED CYLINDER' TESTS*.









Addis, Barton, Bandis, & Henry, 1990. Laboratory studies on the stability of vertical and deviated boreholes. *65th SPE*, New Orleans.

THE IMPORTANT ROLES OF *POISSON'S RATIO* AND *EXTENSIONAL* STRAIN

Classic three-dimensional equation for expressing extension strain (in the lateral direction) is as follows:

$$\varepsilon_3 = [\ \sigma_3 - v(\sigma_1 + \sigma_2)\ ] / E$$

where v is the Poisson's ratio of the intact rock and E is the Young's modulus.

When $v(\sigma_1 + \sigma_2) > \sigma_3$, (negative) extensional strain ($-\varepsilon_3$) will occur.

Stacey, 1981 proposed that if the calculated extensional strain was greater than the *critical strain*, then (extensional) spalling would occur around an 'over-stressed' (i.e. *over-strained*) underground opening.

Two-dimensional version applying to tunnel section:

$$\varepsilon_3 = [\ \sigma_3 - \nu.\sigma_1\ ]\ /E$$

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

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

At (circular) tunnel wall, $\sigma_3 = 0$, and $\sigma_1 = \boldsymbol{\sigma_{max.\ tangential}}$

$(-)\ \sigma_t/E = \boldsymbol{\varepsilon_c} = (-)\ (\nu.\ \sigma_{max.\ tangential})\ /E$…… $\boxed{\sigma_{tangential} = \sigma_t / \nu}$

DR. BAOTANG SHEN WAS ADDING A SUB-ROUTINE TO FRACOD WHEN HE NOTICED THE FOLLOWING. (SEE SHEN AND BARTON, 2016):

$$\sigma_{tangential} = \sigma_t / v = \frac{UCS}{10v} = \frac{UCS}{10 \times 0.25} = 0.4 \times UCS$$

Hence the frequent occurrence of the ratio 0.4 (±0.1) for $\sigma_{tangential}$ /UCS when spalling commences, with the related need for much higher SRF when utilizing the Q-system to select single-shell (NMT) tunnel support.

The ratio 0.4 typical in laboratory tests as well. Crack initiation and AE start for a logical *failure-mechanism* reason, as opposed to *failure-criterion* reason (a subtly differentiated goal mentioned by Bieniawski, 1967).

❑ WHAT DOES THE MOST REALISTIC FRACTURE MECHANICS CODE (FRACOD) SHOW IN RELATION TO EXTENSION-MODE/ SHEAR-MODE FRACTURE PROPAGATION?

- ❑ WHAT HAPPENS WHEN THE ROCK MASS IS PARTIALLY JOINTED OR WELL JOINTED?

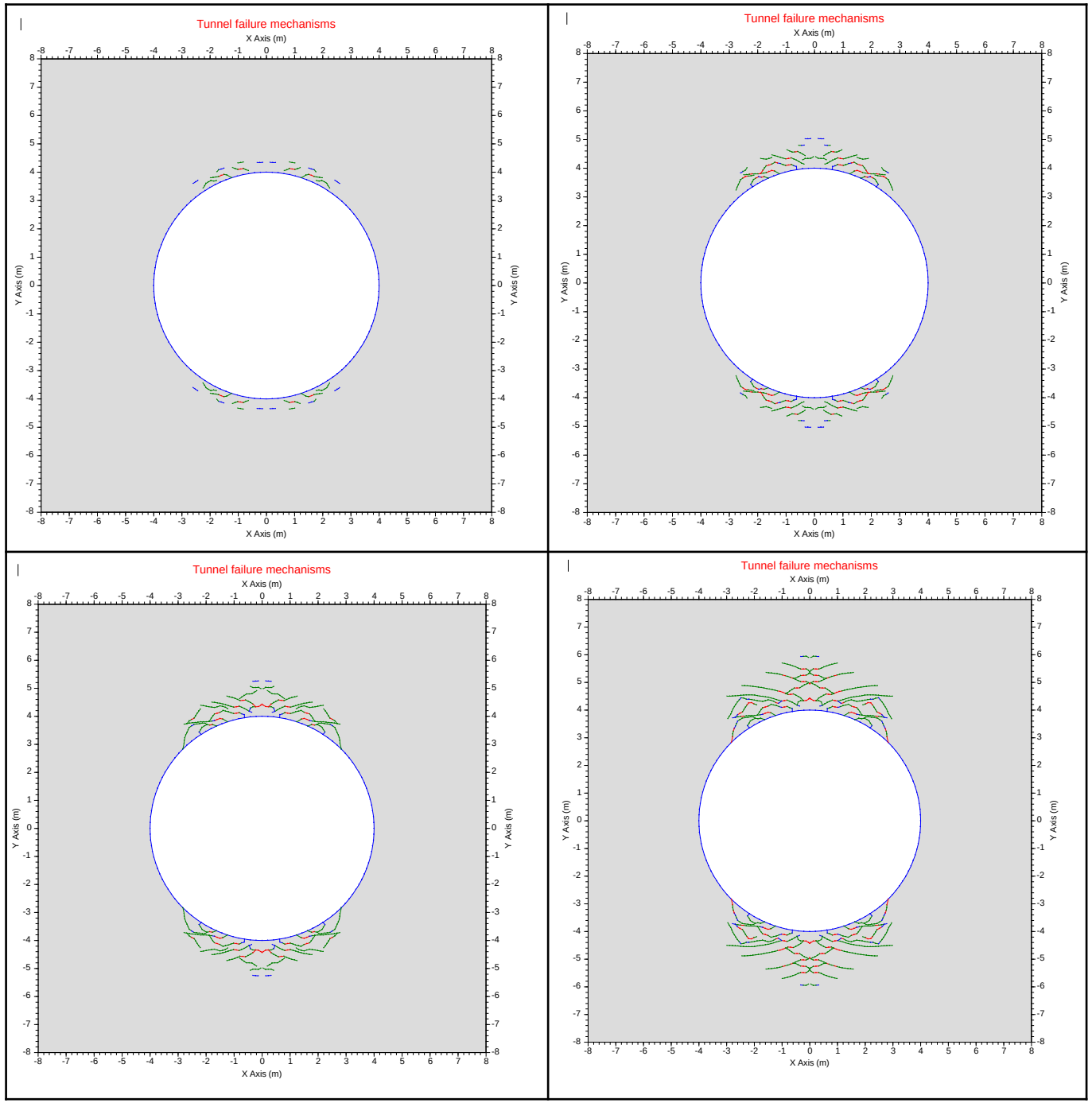
- ❑ DOES JOINTING PREVENT / DISSIPATE THE TENDENCY FOR FAILURE OF THE INTACT ROCK?

FRACOD is a (BEM) fracture mechanics code. It generates discontinuous behaviour in Mode 1 *and* Mode 2, so gives insight into something that ***cannot be modelled*** *with* isotropic elastic/plastic continuum assumptions.

IN SUMMARY

THE FRACOD-BASED APPROACH ALLOWS THE ACTUAL MODELLING OF FRACTURE INITIATION AND PROPAGATION, RATHER THAN THE USE OF THEORETICAL CONSTITUTIVE MODELS AND CONTINUUM MODELLING.

IMPORTANTLY, FRACOD ALSO CONSIDERS SHEAR (MODE II) FRACTURE PROPAGATION WHICH MOST TRADITIONAL FRACTURE MECHANICS APPROACHES NEGLECT.

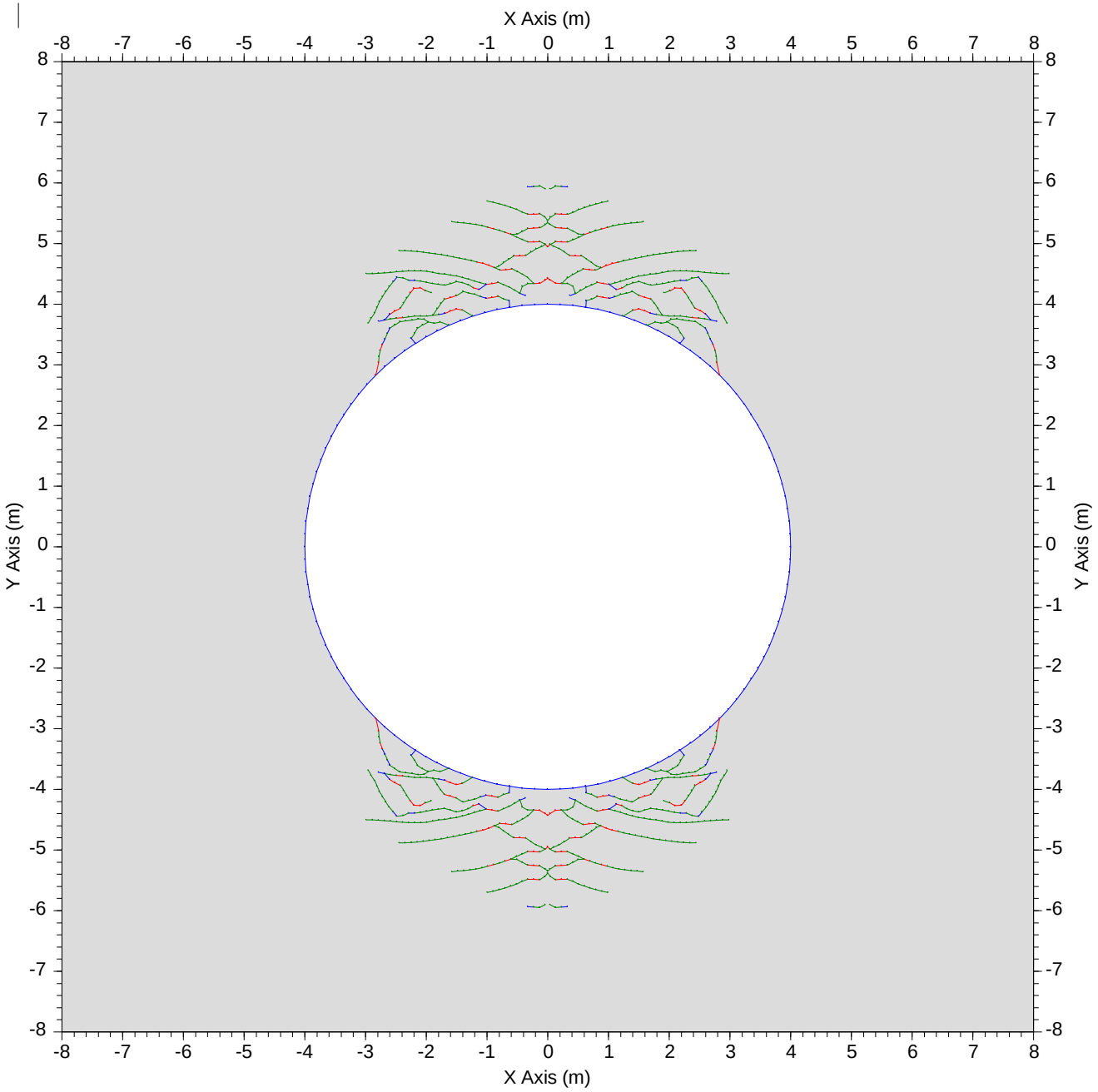


CASE 1. DEPTH = 1000m

ISOTROPIC ELASTIC, NO JOINTS.. DIAMETER = 8 M. (TENSION RED, SHEAR GREEN)

σ_{HMAX} = 50MPa; σ_V = 25MPa. UCS = 165MPa; COHESION C = 31MPa; INTERNAL FRICTION ANGLE ϕ = 49°; σ_t = 14.8MPa; *MODE I FRACTURE TOUGHNESS* K_{IC} = 3.8 MPa $m^{1/2}$ AND *MODE II FRACTURE TOUGHNESS* K_{IIC} = 4.7 MPa $m^{1/2}$.

(TEST DATA FROM ÄSPÖ DIORITE, FROM SKB AND POSIVA).

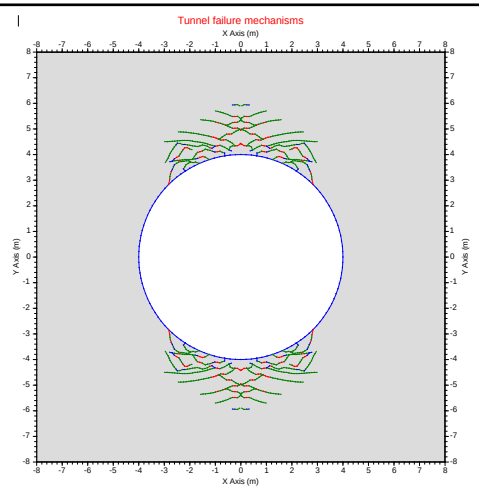
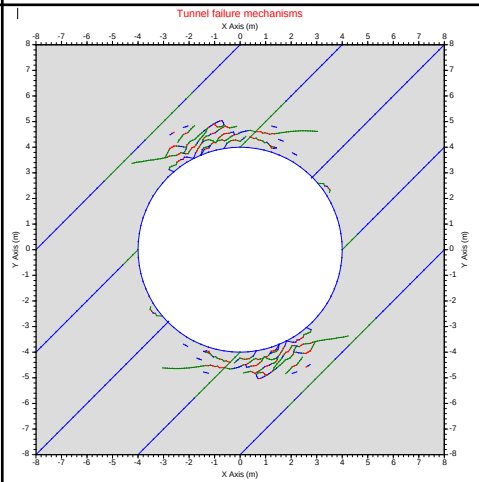
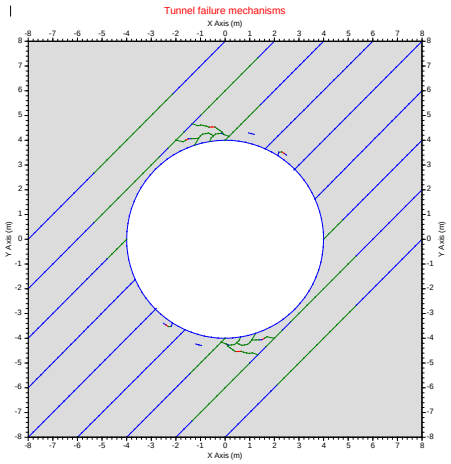
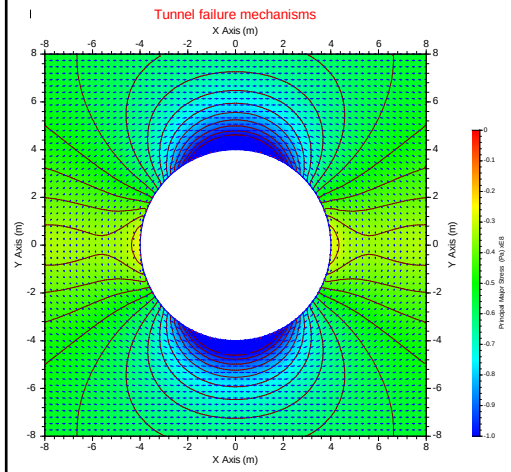
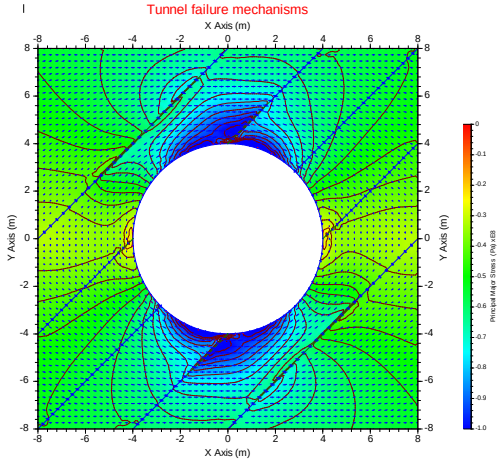
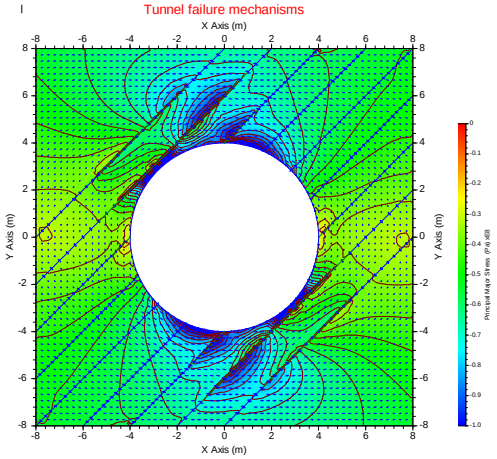


RED: EXTENSION:TENSILE FAILURE (MODE I)

GREEN: SHEAR FAILURE (MODE II) (Log-spiral)

FRACTURE INITIATION MAINLY CAUSED BY TENSILE STRAIN, THEREFORE LOCAL TENSILE FAILURE, BUT FRACTURE PROPAGATION DOMINATED BY SHEARING.

OVERALL BREAKOUT CAUSED BY MIXED FAILURE IN EXTENSION AND SHEAR FAILURE, WITH SHEARING AS THE DOMINANT MECHANISM.

No jointing	One joint set, spacing = 2.8m	One joint set, spacing = 1.4m
		
		
$R_f/a = 1.5$	$R_f/a = 1.25$	$R_f/a = 1.18$

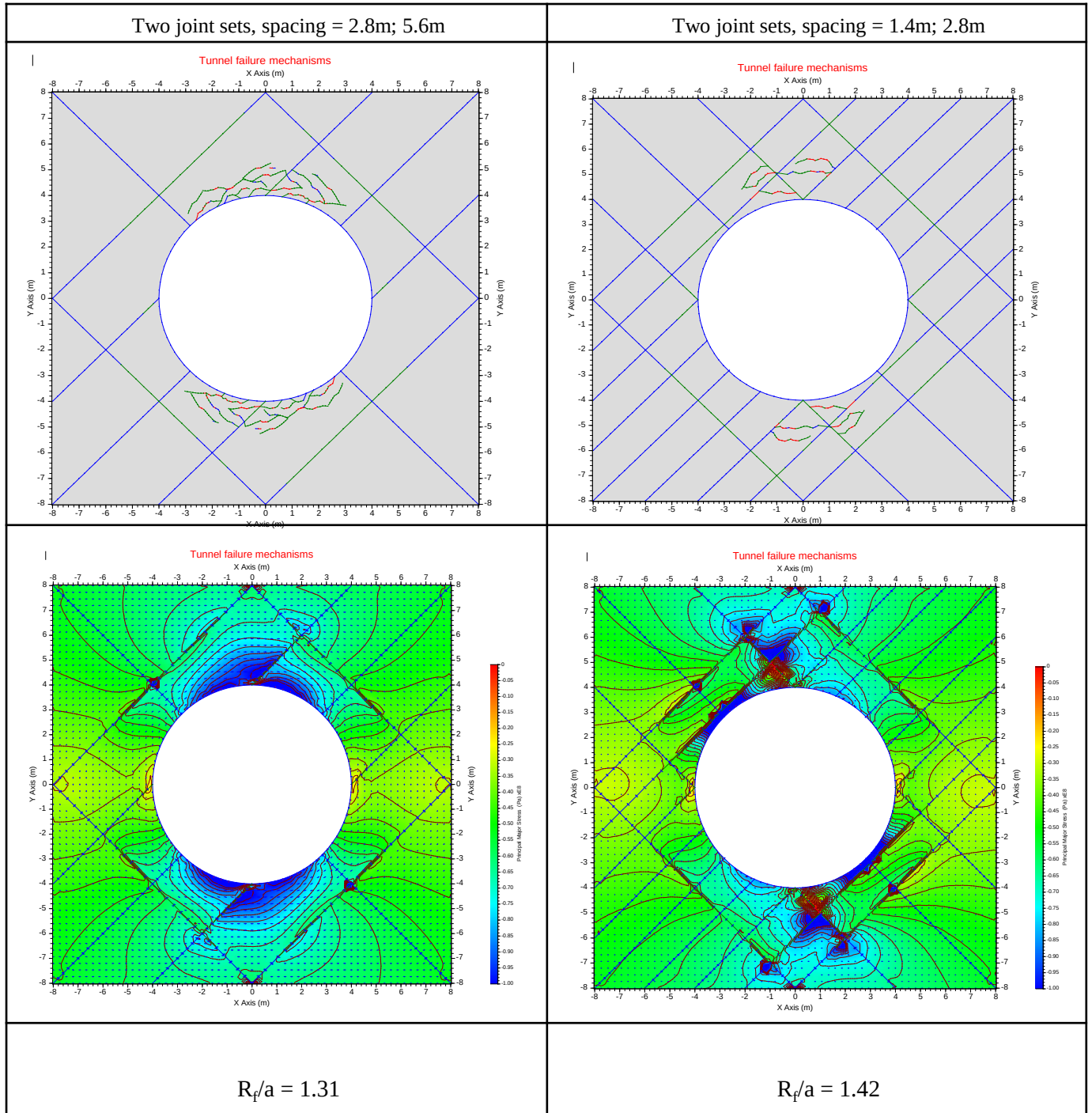
CASE 2. 1,000m DEPTH.

COMPARISON OF BEHAVIOUR WHEN INTACT ROCK IS REPLACED *BY ONE SET OF INCLINED JOINTING,* WITH TWO DIFFERENT SPACINGS (2.8 AND 1.4m c/c).

RE-DISTRIBUTED PRINCIPAL STRESSES SHOWN WITH BLUE CONTOURS.

SHEARING ALONG JOINTS, THEREFORE MUCH REDUCED FRACTURING OF INTACT ROCK, ESPECIALLY WHEN JOINTS ARE MORE CLOSELY SPACED.

NOTE STRONG REDUCTION IN R_f / a.



CASE 3. 1,000M DEPTH.

INTACT ROCK or *TWO SETS OF INCLINED JOINTING*, WITH TWO DIFFERENT SPACINGS (2.8 and 1.4 m c/c).

REDISTRIBUTED PRINCIPAL STRESSES = BLUE CONTOURS. RED CONTOURS INDICATE LOW COMPRESSIVE PRINCIPAL STRESS WHICH OCCURS NEAR THE INTERSECTION OF THE JOINTS DUE TO STRESS REDISTRIBUTION.

SHEARING IS EVIDENT, AND HENCE THE MUCH REDUCED FRACTURING OF INTACT ROCK, ESPECIALLY WHEN THE JOINTS ARE MORE CLOSELY SPACED.

Model at 2,000m depth (with reduced strength):

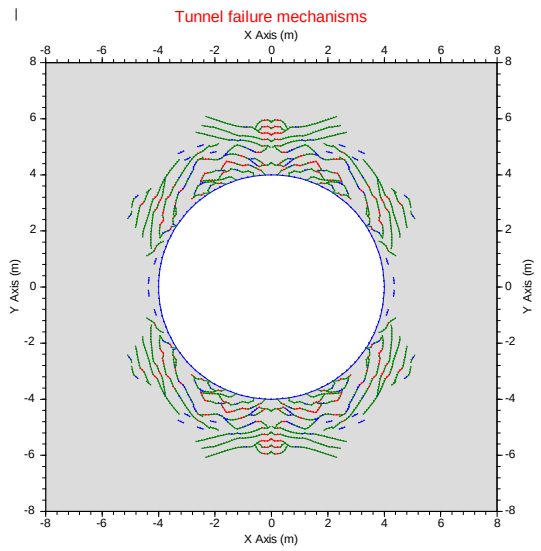
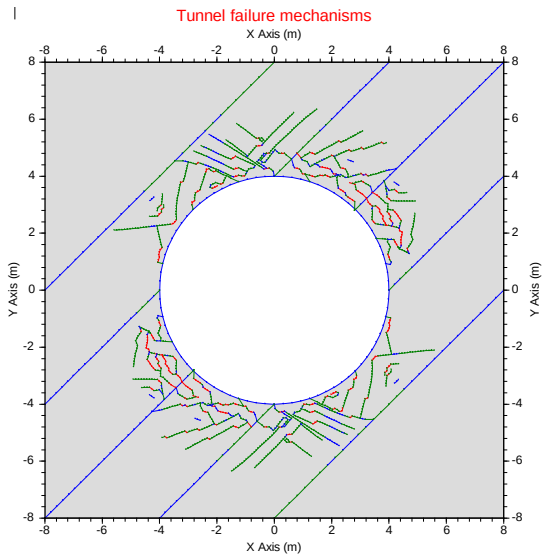
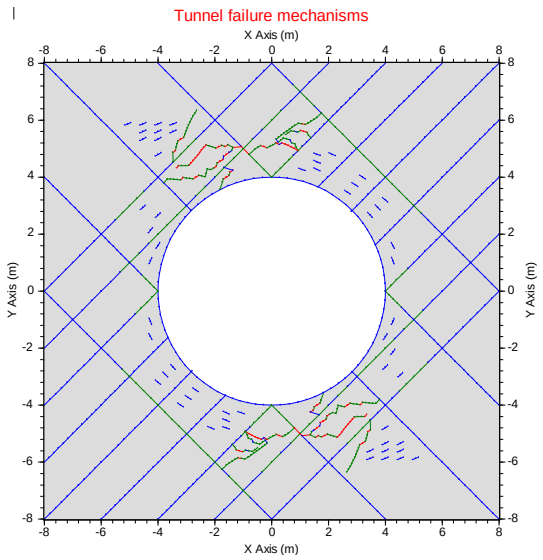
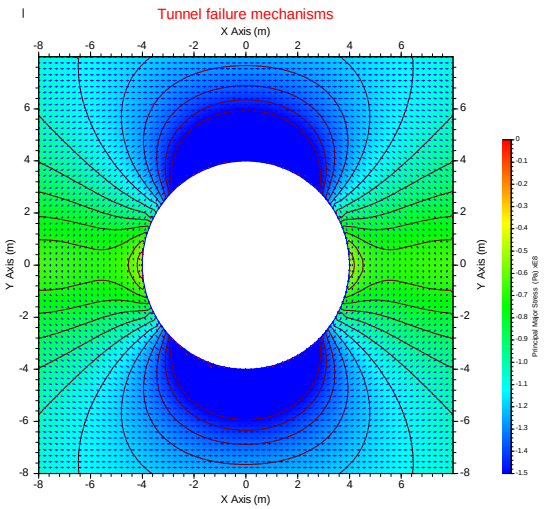
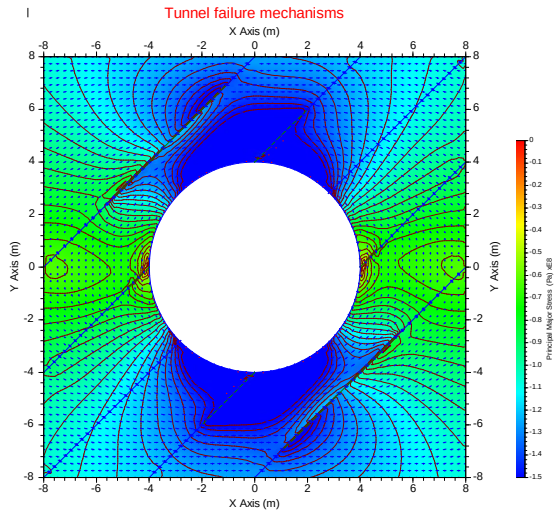
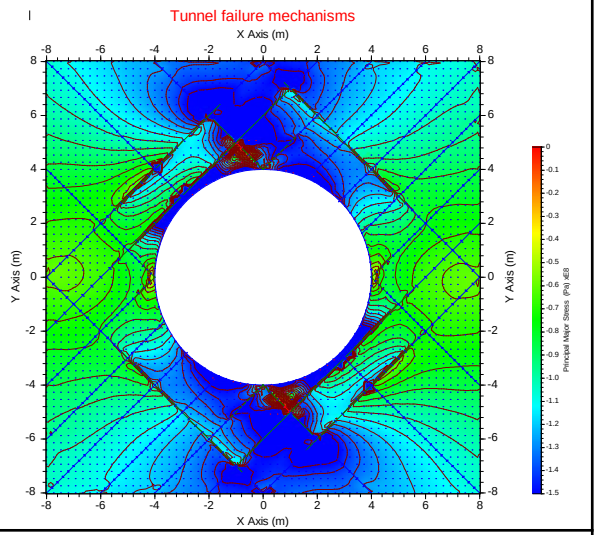
ASSUMED BOUNDARY STRESSES*:* σ_{HMAX} = 100MPA, σ_V = 50MPA.

UCS = 82.5MPa; COHESION c =15.5MPa; INTERNAL FRICTION ANGLE ϕ = 49°; TENSILE STRENGTH σ_t = 7.4MPa, MODE I FRACTURE TOUGHNESS K_{IC} =1.9 MPa $m^{1/2}$; MODE II FRACTURE TOUGHNESS K_{IIC} = 2.35 MPa $m^{1/2}$.

1. **NO JOINTS (MASSIVE ROCK)**

2. **ONE JOINT SET WITH DIP ANGLE OF 45° AND SPACING OF 2.8m**

3. **TWO ORTHOGONAL JOINT SETS, DIP ANGLE 45°, SPACING 1.4m (SET 1) AND 2.8m (SET 2);**

No jointing	One joint set, spacing c/c = 2.8m	Two joint sets, c/c = 1.4m; 2.8m
		
		
Rf /a >1.5	Rf /a = 1.25-1.38	Rf /a = 1.4

CASE 4. 2,000M DEPTH

COMPARISON OF FRACTURING BEHAVIOUR WHEN INTACT ROCK IS REPLACED BY *ONE SET, AND THEN TWO SETS* OF INCLINED JOINTING, WITH TWO DIFFERENT SPACINGS (2.8 and 1.4 m c/c).

THE DEVELOPMENT OF THE REDISTRIBUTED PRINCIPAL STRESSES IS SHOWN WITH BLUE CONTOURS.

SHEARING IS EVIDENT, AND HENCE THE MUCH REDUCED FRACTURING OF INTACT ROCK, ESPECIALLY WHEN THE JOINTS ARE MORE CLOSELY SPACED.

FRACTURE INITIATION MAINLY CAUSED BY TENSILE STRAIN, BUT FRACTURE PROPAGATION DOMINATED BY SHEARING.

OVERALL BREAKOUT CAUSED BY MIXED TENSION AND SHEARING WITH SHEARING AS THE DOMINANT MECHANISM.

THE KEY ADVANTAGES OF THE EXPLICIT FRACTURING *FRACOD* APPROACH:

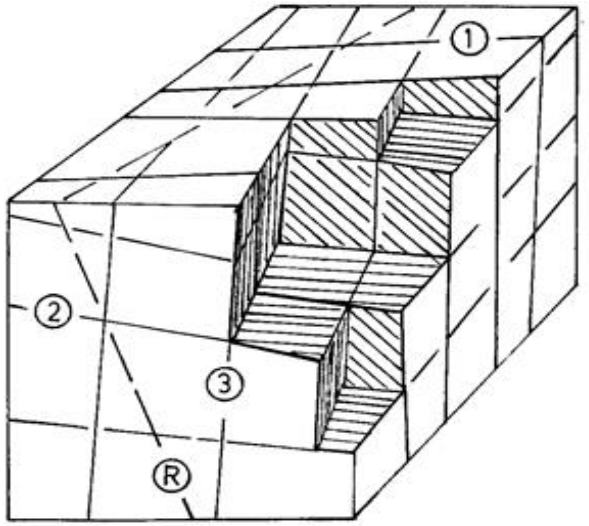
DO NOT NEED TO ASSUME THE FAILURE MECHANISM (TENSILE OR SHEAR) NOR VARY ROCK STRENGTH PARAMETERS DURING FAILURE (SUCH AS STRAIN-HARDENING OR STRAIN-SOFTENING).

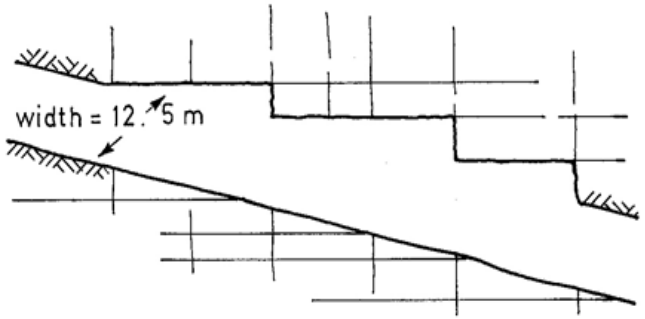
FORMATION OF NEW FRACTURES AND THEIR PROPAGATION CHANGES OVERALL ROCK MASS STRENGTH (AND OTHER PROPERTIES) IN THE MOST REALISTIC WAY. THIS IS FULLY CAPTURED IN THE FRACOD APPROACH.

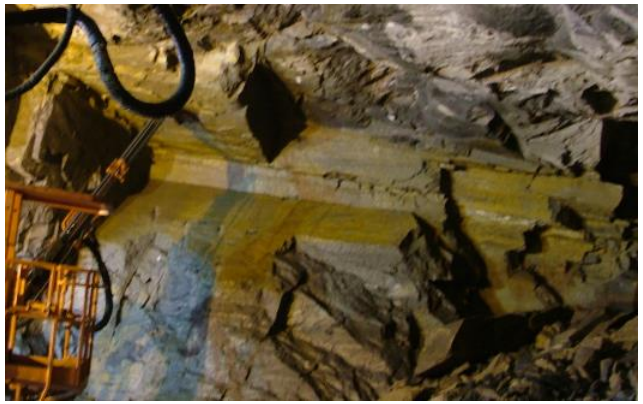
SOME PRACTICAL MATTERS CONCERNING TUNNEL/CAVERN SUPPORT

HOW TO AVOID THE WORST CONSEQUENCES OF OVER-STRESS / OVER-STRAIN / OVER-BREAK?

- **ONE WAY IS TO FOLLOW AN EMPIRICALLY DEVISED METHOD – SUCH AS 'Q'**
- **GSI, HOEK-BROWN, PHASE 2 IS NOT A SOLUTION: EXAGGERATED PLASTIC ZONE**
- **SRF TAKES CARE OF 'OVER-STRESS' (OR 'OVER-STRAIN')**
- **Jn/Jr ≥ 6 SUGGESTS WHEN OVER-BREAK MAY BE INEVITABLE**
- **BE AWARE THAT STEEL SETS / LATTICE GIRDERS ARE NOT PART OF 'Q'**
- **THE LATTER IS FOR A REASON – THEY CAN LEAD TO FAILURE AS TOO 'SOFT'**







OVERBREAK IF Jn/Jr ≥ 6

Jn = number of sets

Jr = roughness

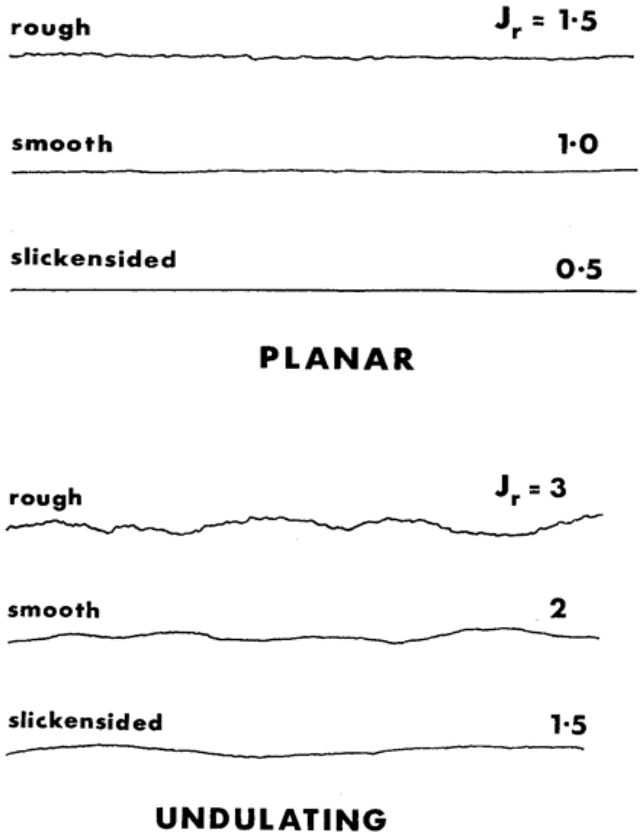
6/1.0 9/1.5

12/2 15/3

(DESPITE FOUR JOINT SETS, TOO MUCH ROUGHNESS AND DILATION)

In photos:

Jn/Jr = 9/(1-1.5)

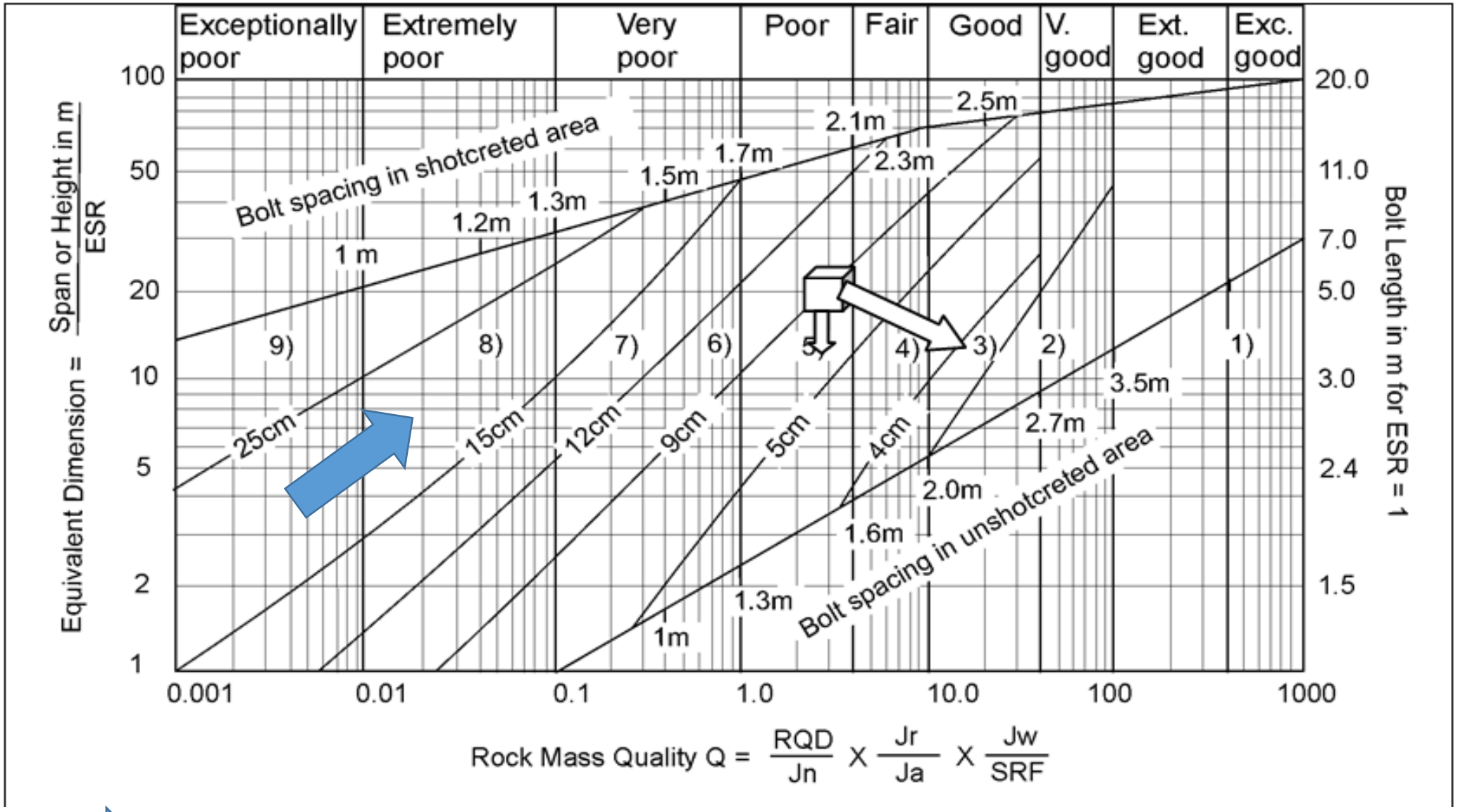


OVER-BREAK needing 4m of CONCRETE

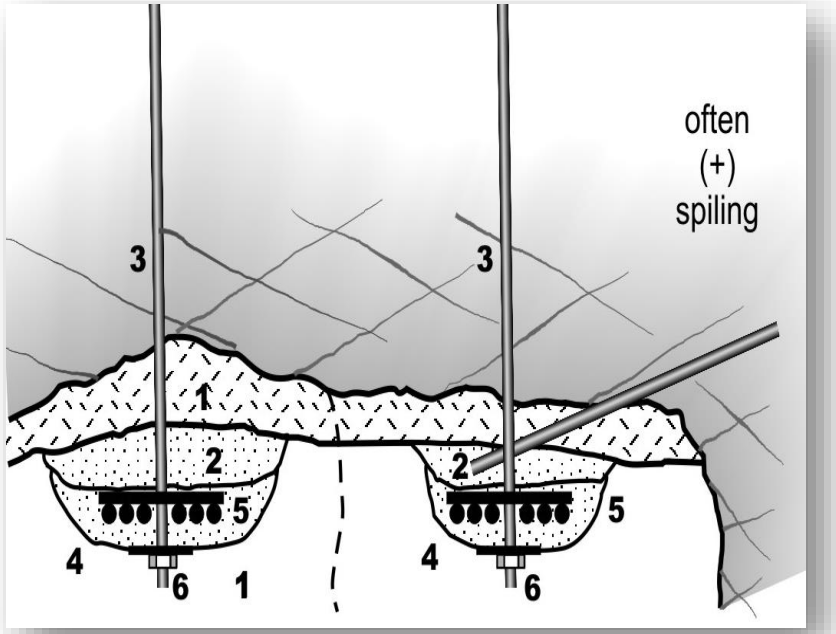
(or 3.5m void?)

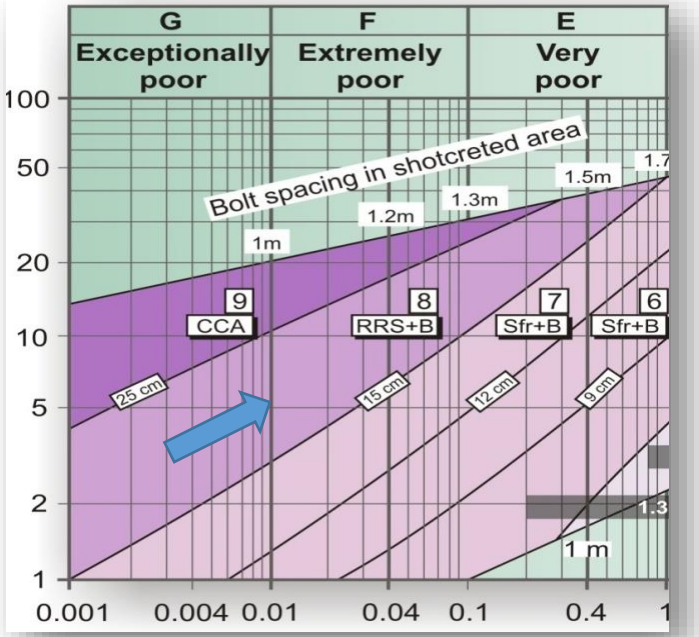
Reason: adverse Jn/Jr

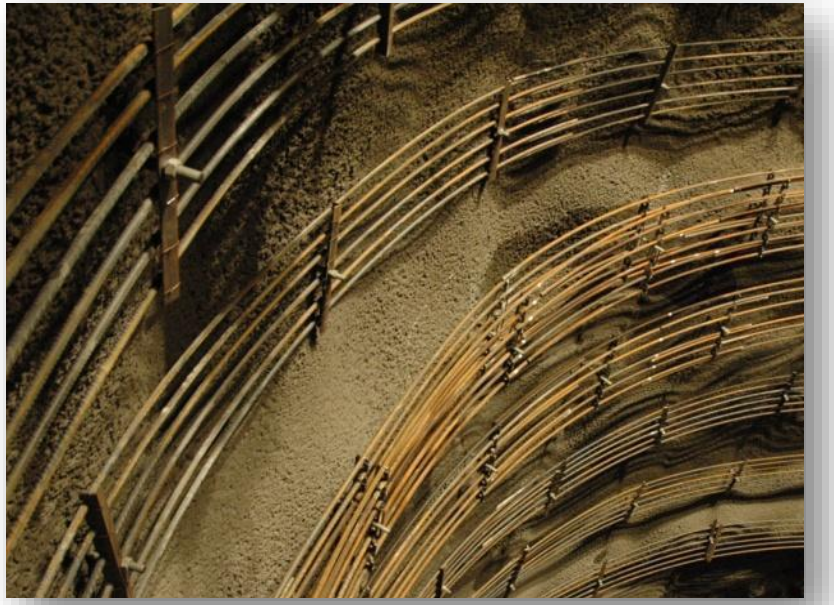
STEEL REINFORCEMENT IN NMT (Q) AND IN NATM

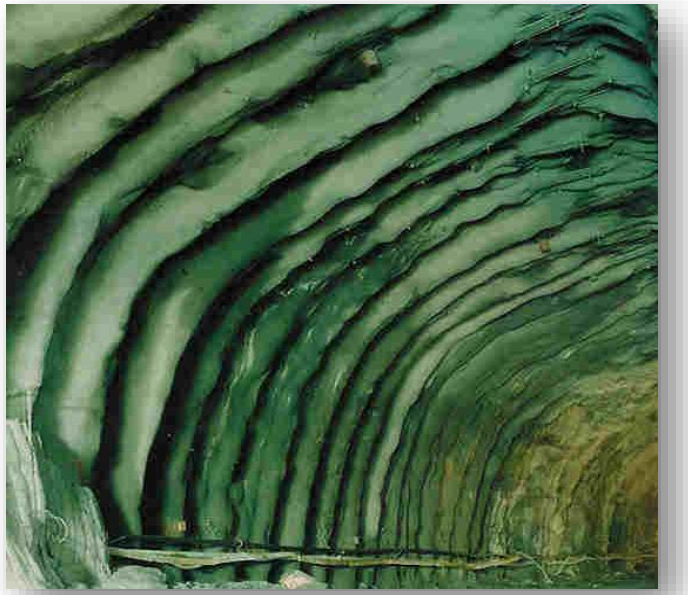


RRS (FOR TEMPORARY (NATM) SUPPORT: 5Q and 1.5 ESR)









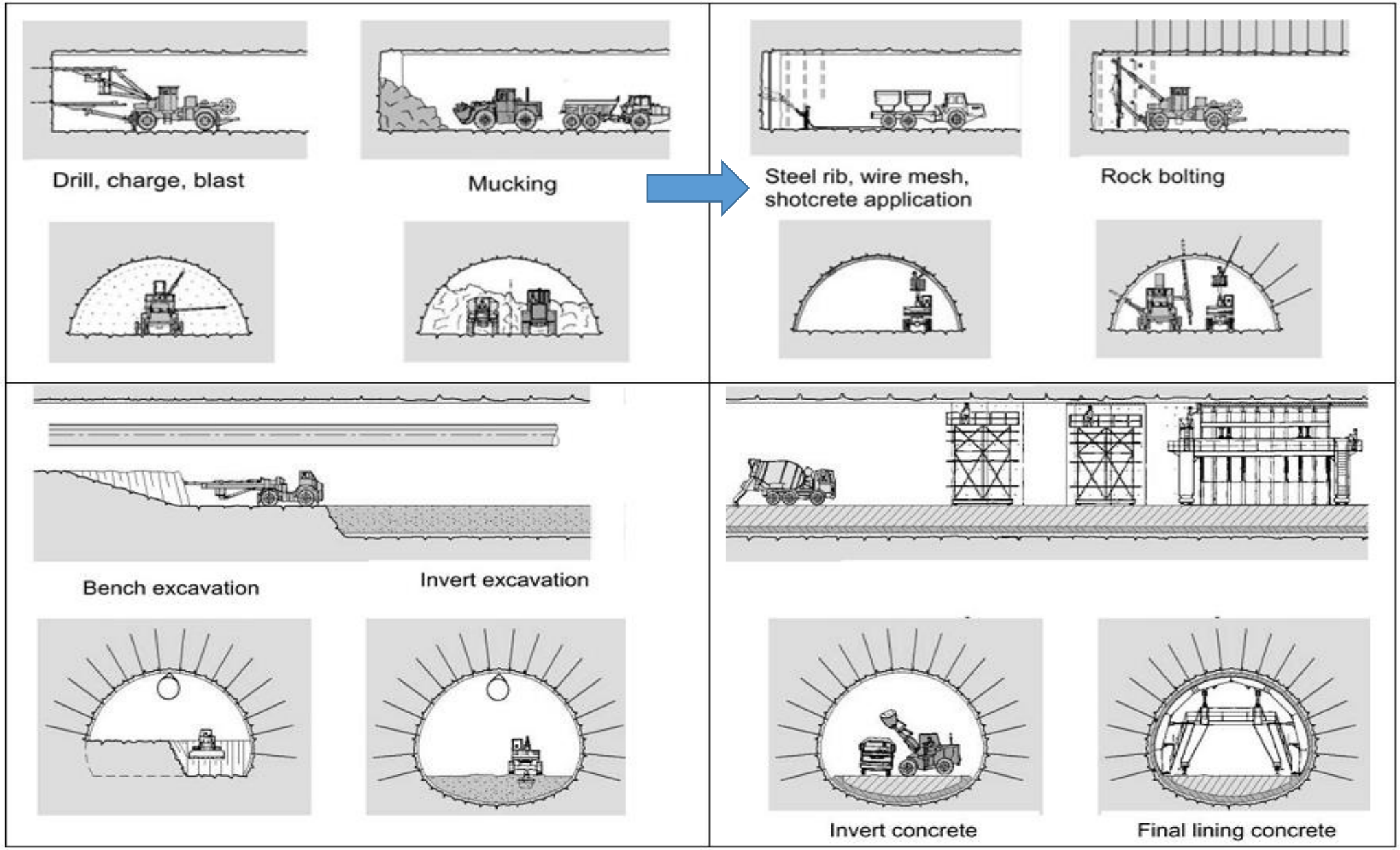
RRS (RIB REINFORCED SHOTCRETE):

STEEL- REINFORCED SHOTCRETE ARCHES, WITH SYSTEMATIC (RADIAL) BOLTING.

OBJECTIVE: LIMIT DISPLACEMENTS THAT CAN CAUSE LOSS OF STRENGTH

(Much better than deformation-inviting lattice girders)

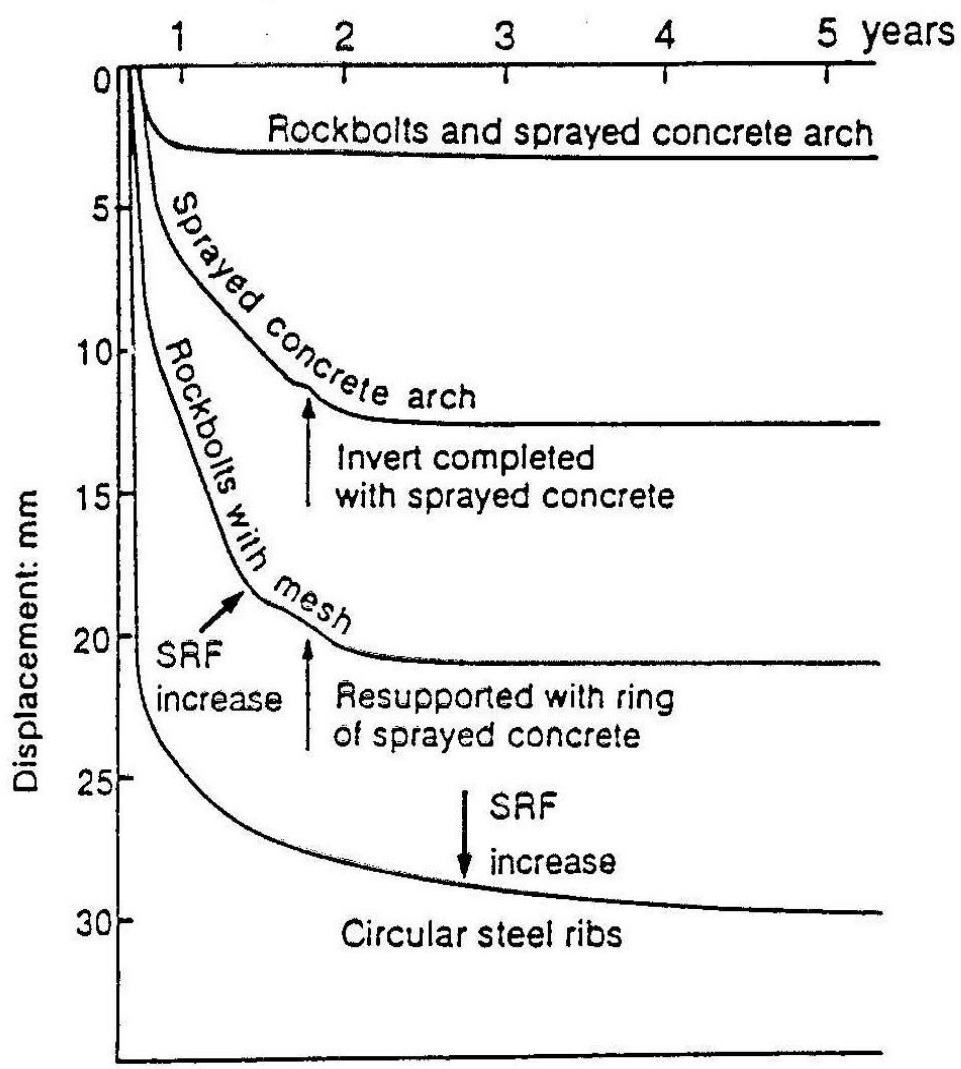
AUSTRIAN SOCIETY FOR GEOMECHANICS, 2010. NATM,
'THE AUSTRIAN PRACTICE OF CONVENTIONAL TUNNELLING'.

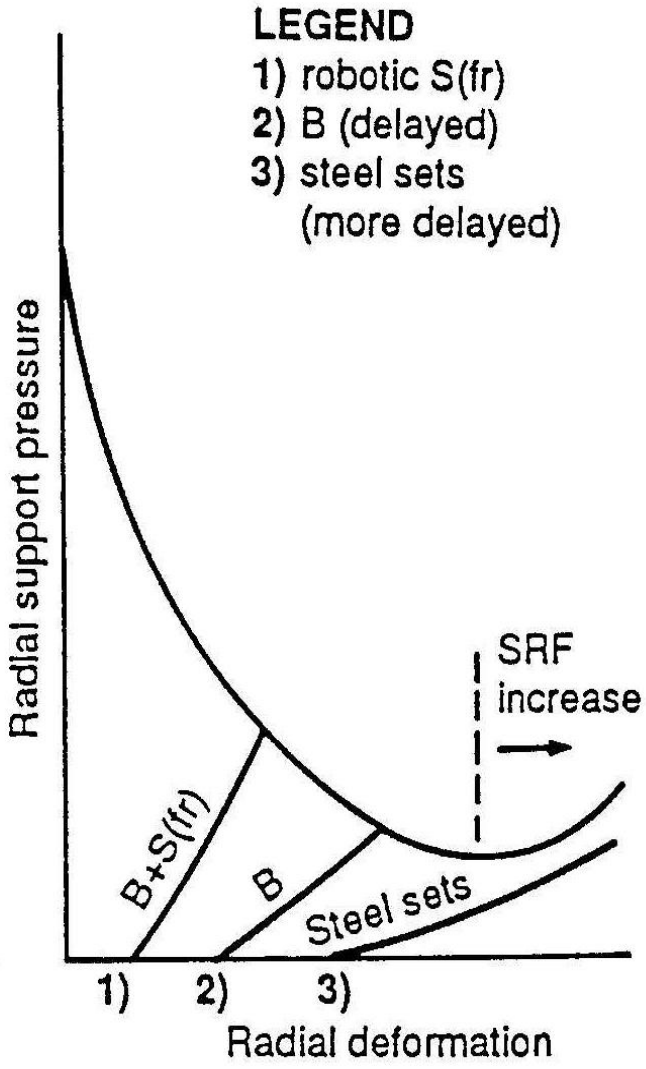


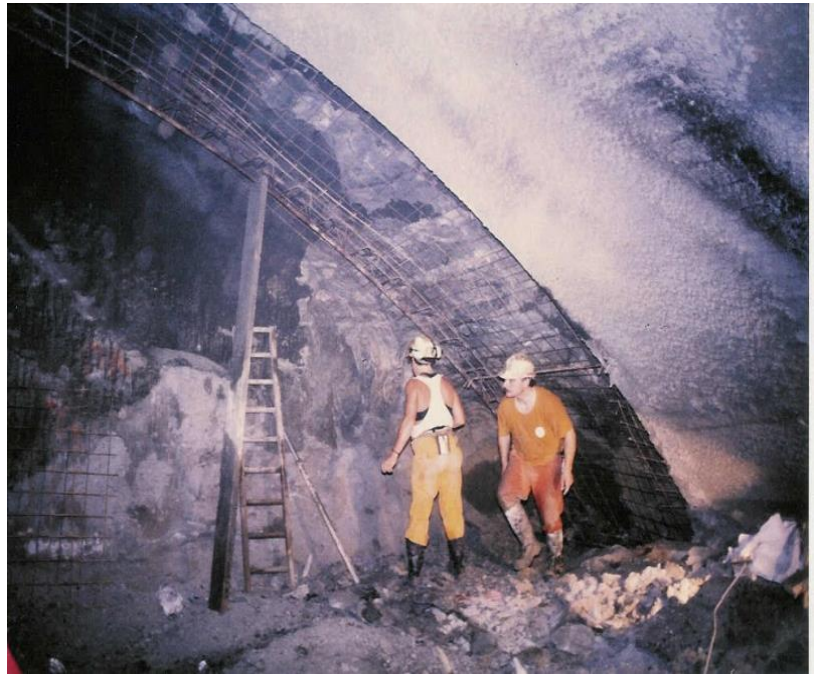
POTENTIAL PROBLEMS WITH (DEFORMABLE) STEEL ARCHES, LATTICE GIRDERS

Ward et al. 1983 Kielder experimental tunnel

Barton and Grimstad, 1994 Update of Q-system for NMT







THE STEEL IS *HARD,* BUT THE STEEL ARCH or LATTICE GIRDER ITSELF IS VERY *'SOFT'!*

COLLAPSE DUE TO *ANISOTROPIC SHEAR STRENGTH* AND UNEXPECTED STRUCTURE, 130m INTO ONE TUNNEL.

2 X 140m OF LATTICE GIRDERS AND SHOTCRETE COLLAPSED – DUE TO MARGINAL STABILITY.

SSSS!

140 m collapsed in parallel tube. Finally 280m collapsed. SSSS!

Top-heading of a cavern.

All three HEP caverns incorrectly oriented.

Had to back-fill (H=30-40m) all caverns to springline, to repair broken B(tg) and cracked shotcrete.

SSSS!

INCORRECT REINFORCEMENT AND SUPPORT TECHNIQUES.

AN OVER LOADED SHEAR ZONE IN THE ARCH WAS INVOLVED AND NON-Q ROCK REINFORCEMENT METHODS (steel arches) – THEREFORE INCREASING RISK.

CONCLUSIONS

1. Excavations in brittle rock, (when at sufficient depth and in the absence of significant jointing), will tend to develop extensional-strain tensile fracturing close to the excavation, followed by coalescence and propagation of fracturing. The latter is dominated by shearing and is intrinsically unstable. In general we will see interaction of stress, strain, strength and structure: a broad challenge for rock support.

2. The deeper the excavation the more dominant will be shearing, which is an unstable, dynamic, and possible rock-bursting associated process. Both processes become limited when one or more joint sets are present, due to shear-stress dissipation on the joints, as opposed to the need for more stress-dissipating fracturing of intact rock, in order to gain 'equilibrium'.

3. So *lack of jointing* is a source of risk in deep hard-rock tunnels, whereas the presence of jointing can sometimes be a source of risk in shallow tunnels. The Q-system tunnel support routines - designed long ago - to reduce risk in tunnel and cavern construction.

4. The commonly occurring fracture-initiation stress/strength ratio of 0.4 (±0.1) representing the ratio of maximum tangential stress (estimated from linear elasticity: $3\sigma_1$-σ_3) and the uniaxial compression strength of the rock (σ_c or UCS), was noted in deep road tunnels in Norway in the 1980's and marks the point where SRF is designed to *accelerate* , both in the Q-system of 1974, and in the S(fr) update of 1993, to provide appropriate *risk-reducing* reinforcement.

5. Apparently independent from Q, fracture initiation and depth of failure have been related to increasing ratios of σ_{max}/σ_c (also initiating at 0.4) in case records from deep mining and from later Canadian URL break-out studies assembled by Martin in 1999.

6. The stress/strength ratio of 0.4 marking (extensional-strain-based) tensile fracture initiation, due in part to Poisson's ratio, also marks the start of acoustic emission, both in the laboratory and *in situ.* It turns out that the ratio 0.4, or close to this number, is a logical result of simple arithmetic.

7. The arithmetic involves typical ratios of σ_c/σ_t (often about 10) and Poisson's ratio for (initially) intact rock (often about 0.25). The 'simplicity' behind the mysterious '0.4' was discovered by Dr. Baotang Shen when adding a new sub-routine to FRACOD.

8. One should be aware that steel sets or lattice girders should never be used in conjunction with the Q-system. They are too 'soft'. Good contact with the rock is difficult (for most contractors – though not in models!) Deformation is 'invited'/needed. Footings must also stop deforming.