# From Empiricism, Through Theory, To Problem Solving in Rock Engineering: 6th Müller Lecture

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**ABSTRACT:** The behaviour of the jointed-and-faulted-anisotropic-water-bearing media that we call rock masses, was an abiding pre-occupation of Leopold Müller. The author has been similarly pre-occupied. So starting with modest developments from tension-fractured physical models, and progressing via simple UDEC models to the real jointed and three-dimensional world in due course, a few of the lessons learned and subsequently applied in rock engineering practice will be described. The failure of jointed rock masses involves mobilization of the intact rock strength at smaller strain than that for the jointing. This was also discussed by Müller in 1966. Continuum modelling with conventional strength criteria is questioned. A simple solution is proposed. Excavation disturbed zones caused by joint shearing are compared with log-spiral failures. The reality of jointing as affecting the deformation of a large cavern, as affecting NMT pre-injection design in rock masses, and as affecting the compaction and subsidence behaviour of a reservoir are each illustrated. Discontinuous behaviour provides rich experiences for those who value reality, even when reality has to be simplified by some empiricism.

**SUBJECT:** General

**KEYWORDS:** rock joints, rock masses, physical modelling, empiricism, site characterization, tunnelling, rock failure

## 1 INTRODUCTION

*The deformation resistance of the material bridges takes effect at much smaller deformations than the joint friction: this joint friction makes partly up for lost strength*. (Müller, 1966). This opinion remains highly relevant, just as it was at the time of the first ISRM congress in Lisbon.

Why are so many young people, even some older, being so attracted by the ease of producing colourful appendices of questionable relevance to reality? Is simplicity a good enough reason, when Müller is no longer guiding us in more challenging directions? Is rock mechanics supposed to be 'easy'? Why are we so confident that 'c *plus* $\sigma_n$ tan φ' should be a guiding principle of shear failure in rock masses? In fact 'c *then* $\sigma_n$ tan φ' or degraded cohesion as friction is mobilized, is much more logical.

Of course rock masses are very complex media, so *some level of simplification* is needed. The question is whether the *real effects* of joints and faults can be 'lifted' into a continuum analysis. Empiricism is used in this process, but later the rock mass loses its identity. Are layers of algebra a justified next step to the empiricism? Popularity suggests yes. Common sense suggests no. Where is the anisotropy?

It would seem to be an advantage if classical British empiricists could influence the more recent *empiricism* in rock engineering. In the late 17th century John Locke (1632–1704) proposed in *An Essay Concerning Human Understanding* (1689) the influential view that *the only knowledge humans can have is a posteriori*, i.e., based upon experience. According to Locke, the *ultimate building blocks of knowledge are based on simple ideas*. George Berkeley (1685-1753), a minister, considered that *any order humans may see in nature is the language or handwriting of God*, and also insisted that *experience is the source of most knowledge*. The role of *experience and evidence* in the formation of new ideas, should be guiding the use of empiricism both in rock engineering and in rock mechanics.

David Hume (1711-1776) nevertheless claimed that *there is no certainty that the future will resemble the past*, which presents a potential challenge to our rock classification abilities, and fuel for the few Austrian, Australian, Italian and Norwegian critics of empiricism. New developments represent a threat to 'the establishment' in all subjects, but even critique that is blind may be a basis for advances and fine-tuning by those who are making developments, so should be welcomed, even if reluctantly.

A fundamental part of the scientific method is that all hypotheses and theories must be tested against observations of the natural world, rather than resting solely on *a priori* reasoning or intuition. On this basis the representation of an obviously anisotropic rock mass in a continuum model can be seriously questioned, despite its widespread practice.

Leopold Müller was one of our discipline founders, and one who constantly emphasized the importance of joints and discontinuities. His analysis of the Vajont rock slide (Müller, 1964) was a direct source of inspiration for one who started to study rock mechanics in 1966. The dramatic discussion of shear strength and elevated joint water pressure at Vajont, from Müller's landmark publication, certainly had important educational influence on the author's life-time pre-occupation with the importance of discontinua. This was a strong feature of early Imperial College and subsequent NGI work, with numerous valued colleagues. Discontinuous behaviour is also *interesting*.

Leonardo da Vinci (1452-1519) once gave advice that was distinctly helpful to one starting out in a relatively undeveloped field: *'If you find from your own experience that something is a fact and it contradicts what some authority has written, then you must abandon the authority and base your reasoning on your own findings'.*

As a result of the strong non-linearity of the peak shear strength envelopes for the tension fractures to be illustrated in Figure 2, the writer has followed a life-long habit of never using cohesion 'c' for representing the shear strength of *rock joints*. Shear tests on non-planar rock joints also demonstrate non-linear strength envelopes. Since 'c' may really be non-existent, convention should be questioned.

According to a MacGraw Hill dictionary definition: *The collection of empirical data* may be *guided largely by preliminary theoretical exploration of what to expect. The empirical method is necessary when entering hitherto completely unexplored fields, and becomes less purely empirical as the acquired mastery of the field increases. Successful use of an exclusively empirical method demands a high degree of intuitive ability in the practitioner.*

The above opinions seem to have some application to developments such as the Q-system and the Barton-Bandis model for rock joint behaviour, that will be used in various contexts in the field examples presented in this paper.

Figure 1. Confronted with this potentially unstable jointed rock slope, multiple reasons for the over-break and instability suggest themselves, at least to one involved with quantitative empirically-based methods. There are clearly adverse values of JRC, JCS, and $\phi_r$, and there are also adverse ratings of Jn, Jr, Ja (and Jw on occasion). However dip and dip direction must be the fundamental geometric/gravitational drivers in this apparent stability problem.

The development and application of these simple empirically-based parameters will inevitably form a part of this paper, as they have proved very useful for describing conditions in the form of readily understood numbers, rather than adjectives, as in the distant past.

Historically the discovery of empirical relationships has been important as a stepping stone to the later discovery of theoretical relationships, and on occasion, what was thought to be an empirical factor may later be considered a fundamental physical constant (or variable). One may claim that the rock joint parameters JRC, JCS and $\varphi_r$ introduced in the next section, have resemblance to physical variables of merit. However the form of the equation linking these parameters to the peak shear strength has yet to be explained on theoretical grounds, so its empiricism remains.

Likewise the approximation of the (Jr/Ja) part of the Q-formula to inter-block friction coefficient, shows close resemblance to this important physical variable, though the overall empirical combination in the form a/b x Jr/Ja x e/f anchors the Q-formula firmly in the realm of empiricism.

A more recent discovery that $Q_c \approx c \times \tan \varphi$ for the rock mass, may seem to defy rational explanation. However, the fact remains that a strong resemblance to *c and tan φ* is indeed present in two halves of the formula for $Q_c$, which equates to Q x $\sigma_c/100$, with the uniaxial compression strength expressed in units of MPa, thereby normalizing Q.

The fact that both the Q-*parameter development* and their *ratings* were strongly steered by case records concerning sufficient shotcrete (compensating for lack of cohesion), and by sufficient bolting (compensating for lack of friction), may add some slight credibility to this unexpected finding.

## 2. TWO-DIMENSIONAL ROCK MASSES: FICTIONAL BUT INFORMATIVE

The desire to model the behaviour of jointed rock slopes in late nineteen sixties post-graduate studies at Imperial College, (consistently and innocently called Empirical College in several countries in the Far East), led to tension-fracture models by the writer, and numerical modelling developments (pre-μDEC) in the case of student colleague Peter Cundall. The relative inflexibility and flexibility of the two approaches is readily imagined from Figure 2.

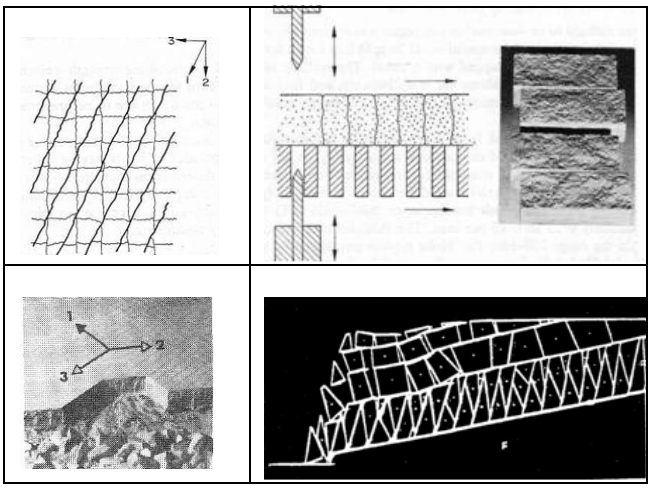


Figure 2. A study in contrasts: physical modelling using tension-fracture generation, and numerical modelling, using μDEC: example with a friction angle for the joints of φ = 20º throughout.

The single numerical slope model shown in Figure 2, demonstrated the influence of changed friction angles, and was reported in Cundall et al., 1977, but performed some years earlier, following Cundall, 1971 developments. The writer has optimistically assumed that the demonstrable *inflexibility of his physical-models*, contributed to Cundall's later developments of UDEC and 3DEC, which are so much appreciated by those desiring a closer approach to modelling the complex reality of excavating in jointed rock.

Despite the shortcomings of physical tension-fracture models, the writer was nevertheless excited to discover that the peak shear strength of these rough and clearly unweathered tension fractures could be described by a simple relation involving the *uniaxial compression strength* ($\sigma_c$) of the model material (Barton, 1971):

$$\tau = \sigma_n \tan [\ 20 \log (\sigma_c / \sigma_n) + 30^o\ ] \qquad (1)$$

This equation, and simple links to peak dilation angle, proved to be the 'end-member' of the Barton and Choubey, 1977 equation for the peak shear strength of rock joints, since *unweathered tension-fractures* clearly maximise surface roughness, wall strength *and* residual friction angle:

$$\tau = \sigma_n \tan [\ JRC \log (JCS / \sigma_n) + \varphi_r^o\ ] \qquad (2)$$

where the *joint roughness coefficient* (JRC), the *joint wall compression strength* (JCS) and the *residual friction angle* ($\varphi_r$) can each assume variable magnitudes, caused by variable roughness and variable weathering. The first equation was based on *direct shear tests* of more than 200 artificial samples, while the second equation was based on DST of 130 rock joint samples, some of them weathered.

Note that the angle $\varphi_r$ may be several degrees less than $\varphi_b$ (used earlier for flat *unweathered* surfaces), and JCS may be significantly lower than $\sigma_c$ for the same reasons. JRC should be calculated on this basis, if back-analysed from testing, as emphasized by Barton and Choubey, 1977. In addition there are block-size scale effects, to be mentioned later.

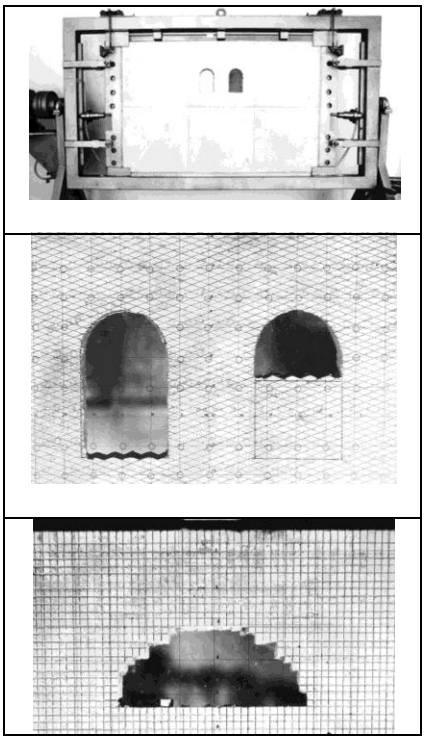


Figure 3. Physical models each of 40,000 blocks created for rock slope analyses in Barton, 1971, were followed some years later, by physical model studies of caverns for underground nuclear power plants, studying the generic effect of joint-set (fracture) orientation, anisotropy, and horizontal stress in models with 20,000 blocks. This research was performed in NGI, Oslo. Barton and Hansteen, 1979 also compared the physical results with FEM.

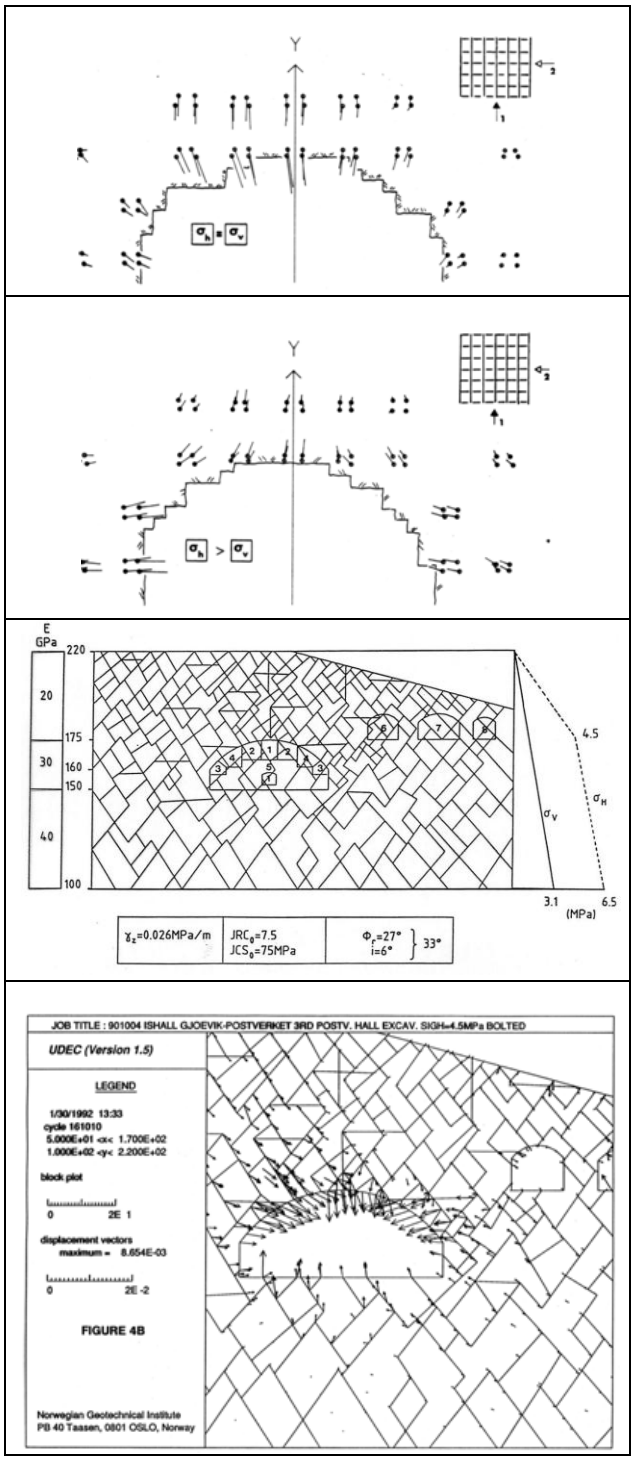
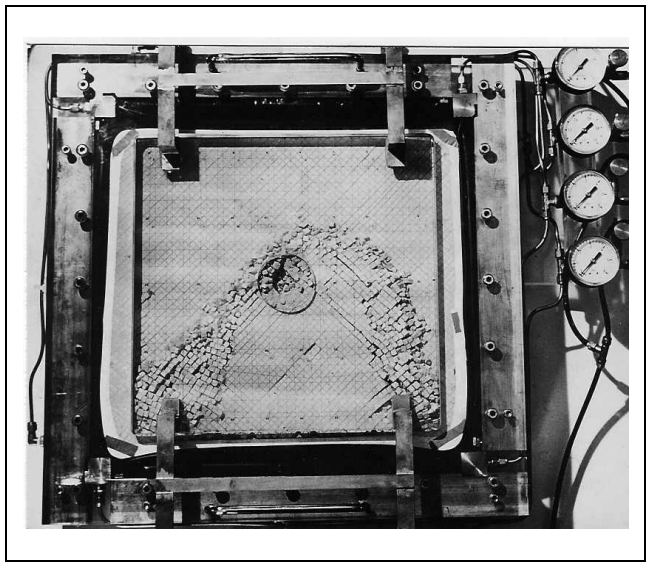



Figure 4. The contrasting effects of $k_0 = 1$, or $\sigma_H > \sigma_V$ on deformation vectors, were recorded in these pre-UDEC-BB cavern models (top two figures). These effects seemed to be accentuated by fracture-set orientations and by fracture-set anisotropy (set 1 was always dominant, set 2 was always 'stepped' where crossing set 1, due to the method of generation seen in Figure 2). The sensitivity to 'joint-set' orientation led to speculation 10 years later, when checking designs for the 62 m span Gjøvik Olympic cavern, that this huge rock arch could 'heave' or 'subside'. The reality, based on surface optical monitoring, and seven MPBX installed from the surface, integrated with three MPBX installed from the top-heading, was a 7 to 8 mm downward movement. The UDEC-BB models shown above, gave a good (Class A) prediction of 7 to 9 mm downward movement. (Barton et al., 1994).



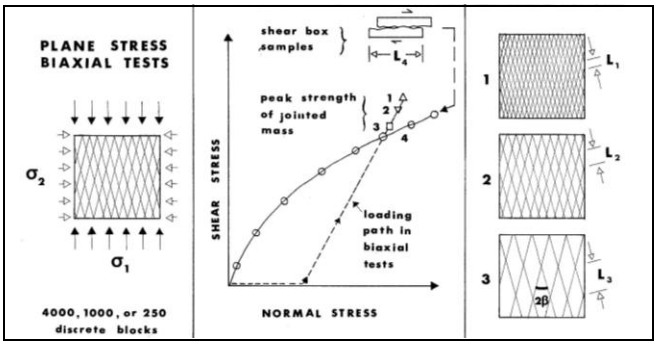



Figure 5. Biaxially loaded two-dimensional physical models with respectively 250, 1000 or 4000 discrete blocks also helped to give clues about scale effects caused by different block sizes. In the case of 'rock masses' formed with rough tension fractures, the shear strength of the mass tended to be *highest with smaller block sizes* (due to the tendency for block rotations), as also observed by Bhasin and Barton, 1997 in UDEC-BB models. UDEC-MC (Mohr-Coulomb) models did not show this block rotation effect with the same block sizes. These physical models were described by Barton and Hansteen, 1979, therefore pre-dating UDEC-BB.

As will be shown in the next figure, the performance of *UDEC-MC* models with *varied block sizes*, nevertheless showed a significant effect of block size, due to the greater freedom for block rotations when blocks were very small. Something resembling kink-band rotation could occur.

Although with 2D limitations, and highly idealized, Figure 6 demonstrates the potential for a much deeper EDZ in the case of *severely faulted or crushed rock*, if the assumption is made that rock in the neighbourhood of faults is often crushed or displaying smaller block sizes, even additional jointing. Core is often crushed and appears to show Jn = 15 (four sets) or worse in such locations.

With the possible addition of clay coatings or thin fillings of clay, or even a core of thick clay somewhere in the midst of a fault zone, one may expect Q-parameters in the neighbourhood of Q = 10/15 x 1/6 x 0.33/2.5 = 0.01. The severity of the rock mass characteristics, and effects on e.g. TBM progress, may be heavily dependent on the presence (or absence) of water in such cases. Outwash of clay may trap TBM, due to rock-block release, and frequent blocking of the cutter head by the falling larger blocks. Barton, 2008.

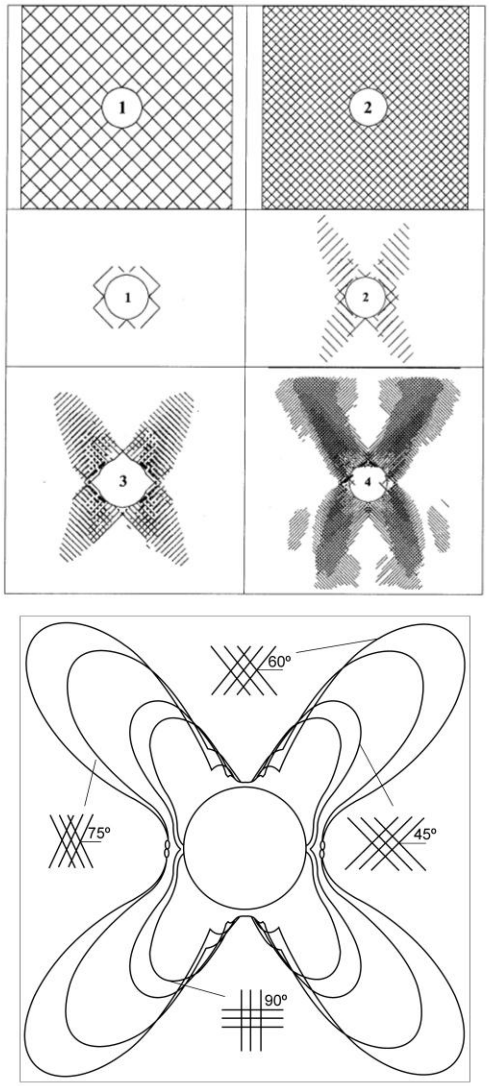



Figure 6. UDEC-MC models with successively halved block sizes but 45º/45º jointing, demonstrate a strong 'mass scale effect', that appears to depend on the greater degrees of freedom for block rotations, as block size reduces. The central plots show widely different penetration of inter-block shearing. Mohr-Coulomb based analytical solutions, also given by Shen and Barton, 1997, confirm the general trends. UDEC-BB models with 0º/90º block orientations, also demonstrate joint shearing in the same ≈ 45º sectors, confirming the penetration of the EDZ into these zones.

## MODELLING FAILURE THROUGH MASSIVE ROCK

The '45º sectors' EDZ seen in the previous figure, that are a feature of (idealized) jointed rock with two equal sets and any conjugate orientation (45º, 60º, 75º, 90º) of the jointing, forms a strong contrast to the development of stress-induced failure in the case of massive rock or intact model material. Joint-industry borehole stability studies performed for oil companies at NGI in the late eighties, provided strong evidence of shear failure development in the form of log-spiral surfaces, with physically measured shear displacements. This log-spiral form has recently been seen when inspecting TBM headrace tunnels in massive sections of marble and schists, where the estimated theoretical maximum tangential stress ($\sigma_\theta = 3\sigma_1 - \sigma_3$) was exceeding the 'limit' 0.4 x UCS, i.e. increased SRF in the Q-system.

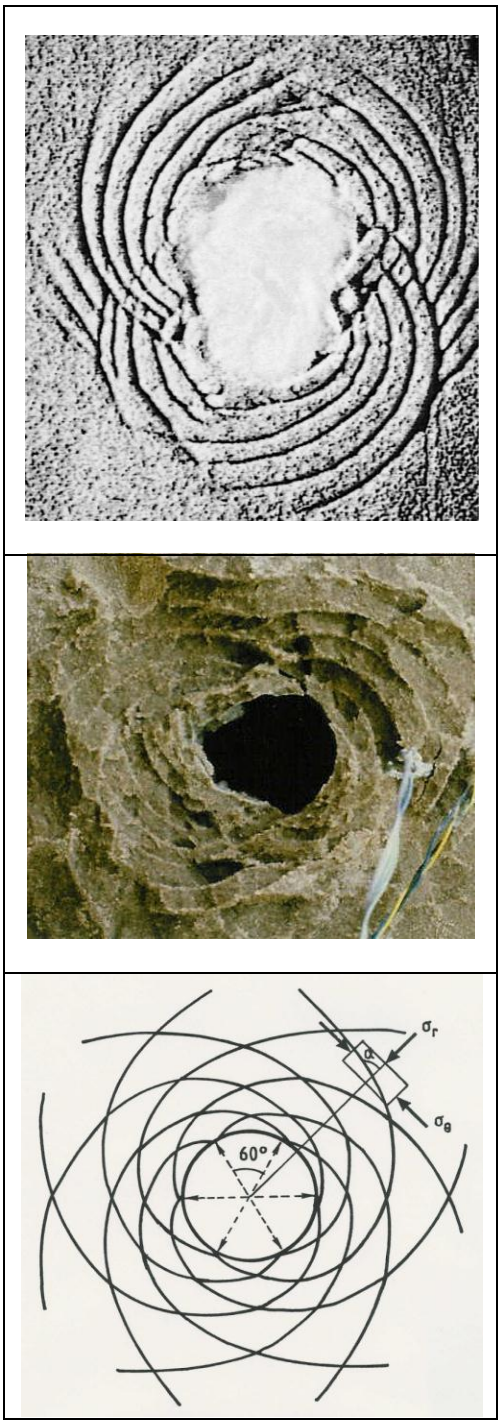



Figure 7. Top: log-spiral shearing as a result of drilling a 45° inclined borehole in the direction of $\sigma_H$ into a stressed block of model sandstone, with principal stress ratios $\sigma_v = 1$, $\sigma_H = 0.8$, $\sigma_h = 0.4$. Addis et al., 1990.The theoretical tangential and radial stresses acting on log-spiral shear surfaces are from Bray 1966 lectures, but these might be suppressed by dilation if rock strength is too high.

The borehole stability studies involved drilling into a 0.5 x 0.5 x 0.5 m polyaxial cell full of anisotropically loaded model sandstones. Drilling could be performed in various directions in relation to the three principal stress directions. Loading was provided by flat-jacks. Figure 6 shows some of the phenomena observed by 'slicing' the models after low pressure saturation with epoxy, in order to preserve the drilling-induced (and therefore stress-induced) failures.

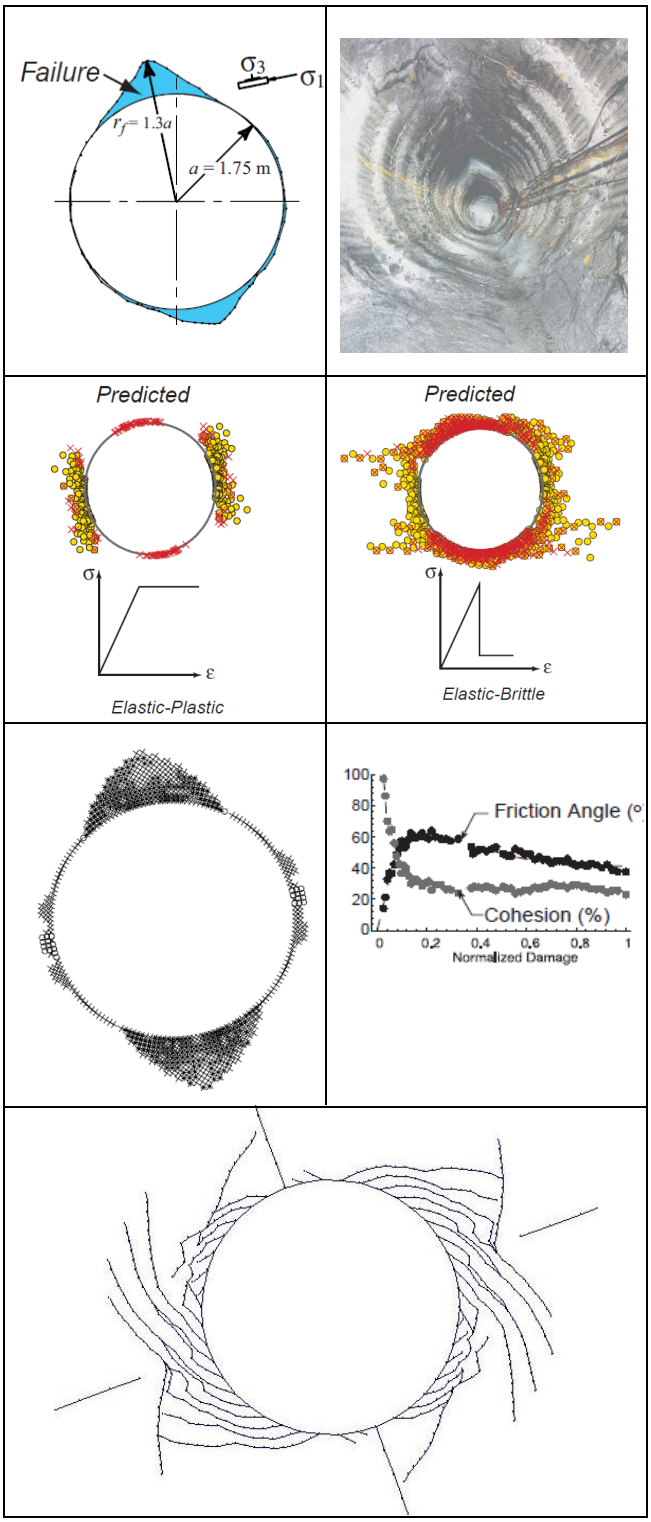



Figure 8. Top: The Canadian URL mine-by break-out that developed when excavating by line-drilling, in response to the obliquely acting anisotropic stresses. This is followed by an important demonstration of unsuccessful modelling by 'classical methods' given by Hajiabdolmajid et al., 2000. They followed this with a more realistic degradation of cohesion and mobilization of friction, which was applied in FLAC (their 6/02/1999 modelling date removed for clearer presentation). The bottom figure is a FRACOD fracture mechanics approach to modelling of stress-fracturing in a TBM tunnel (pers. comm, Baotang Shen, 2004). The project is now delayed, due to irreparable damage to the TBM from a rock burst. Advised drill-and-blast is finally occurring.

Attempts to model 'break-out' phenomena such as those illustrated are not especially successful with standard Mohr Coulomb or Hoek Brown failure criteria, because the *actual phenomena* are not following our long-standing concept of *'c plus $\sigma_n$ tan $\varphi$'*. The reality is degradation of cohesion at small strain and mobilization of friction (first towards peak, then towards residual) which occur at larger strain. We register closure or squeezing, and also can measure it, as *an apparent radial strain*. In reality it may be a tangential strain-related failure phenomenon.

The very important findings of Hajiabdolmajid et al., 2000 are summarised briefly by means of the first six figures assembled in Figure 8. The demonstrated shortcomings of continuum modelling with *'c plus $\sigma_n$ tan $\varphi$'* shear strength assumptions, should have alerted our profession for change already ten years ago, but deep-seated beliefs or habits are traditionally hard to change.

Rock masses actually follow an even more complex progression to failure, as suggested in Barton and Pandey, 2011, who recently demonstrated the application of a similar *'c then tan $\varphi$'* modelling approach, but applied it in FLAC 3D, for investigating the behaviour of multiple mine-stopes in India. A further break with convention was the application of peak 'c' and peak 'φ' estimates that were derived directly from mine-logged Q-parameters, using the CC and FC parameters suggested in Barton, 2002. For this method, an estimate of UCS is required, as CC (cohesive component) and FC (frictional component) are derived from separate 'halves' of the formula for $Q_c = Q \times \sigma_c / 100$.

The two or three classes of discontinuities (natural and induced) involved in pre-peak and post-peak rock mass failure, will also have quite different sets of shear strength properties. For instance *the new stress-induced failure surfaces*, if described with JRC, JCS and $\varphi_r$, might have respective numbers (at small scale) like 18 to 22, 100 to 150 MPa and 30° to 32° (i.e. rough and unweathered and strongly dilatant), compared to perhaps 4 to 8, 50 to 100 MPa and 27° to 29° *for potential joint sets*, or perhaps Jr/Ja = (1 to 2)/4 for any *clay coated discontinuities*, that might also be involved in post-peak behaviour of the rock mass.

The dilatancy obviously reduces strongly between these three groups of discontinuities. Furthermore, each of the above are features that begin to resist shearing at considerably larger strains/deformations than is the case for the also strongly dilatant failure of the 'intact bridges'. Why therefore are we *adding* 'c and $\sigma_n$ tan φ' in 'continuum' models, making them even poorer representations of the strain-and-process-sensitive reality?

Input data obtained via Hoek and Brown and GSI formulations that obviously ignore such complexity, nevertheless consist of remarkably complex algebra (e.g. Table 1) in comparison to the more transparent formulae for discontinuum codes, where $JRC_0$, $JCS_0$, $\varphi_r$, $L_0$ and $L_n$ and use of Barton-Bandis scaling equations, are sufficient to develop the key joint strength and joint stiffness estimates. ($L_0$ and $L_n$ are the lab-scale and in situ scale block sizes).

A demonstration of the simpler, even simplistic 'cohesive component' CC and the 'frictional component' FC for a variety of rock mass characteristics is given in Table 2. These parameter estimates have the advantage of not requiring software for their calculation – they already exist in the Q-parameter logging data, and the effect of changed conditions such as clay-fillings, can be visualized easily.

Table 1. The remarkable complexity of the algebra for estimating c' and φ' with Hoek-Brown based formulations (equations 4 and 6) are contrasted with the simplicity of equations 3 and 5, derived by 'splitting' the existing $Q_c$ formula into two parts, as described in Barton, 2002. ($Q_c = Q.\ \sigma_c/100$, with $\sigma_{ci}$ expressed in MPa).

| | | |
|---|---|---|
| FC | $"\phi" \approx \tan^{-1}\left(\frac{J_r}{J_a} \times \frac{J_w}{1}\right)$ | (3) |
| | $\phi' = a\sin\left[\frac{6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}{2(1+a)(2+a)+6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}\right]$ | (4) |
| CC | $"c" \approx \left(\frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100}\right)$ | (5) |
| | $c' = \frac{\sigma_{ci}\left[(1+2a)s+(1-a)m_b\sigma'_{3n}\right]\left(s+m_b\sigma'_{3n}\right)^{a-1}}{(1+u)(2+a)\sqrt{1+\left(6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}\right)\Big/\left((1+a)(2+a)\right)}}$ | (6) |

The relatively new parameters CC and FC, were 'hiding' in the Q-formula for 25 years, and needed the simple addition of a normalising (Q) *x $\sigma_c$ /100* for their final recognition.

It should be emphasised that all the Q-parameter ratings developed long ago by Barton et al. 1974, were successively improved by 6 months of trial-and-error fit to required rock bolting needs (bolting is compensating for *low frictional strength*), and to required shotcreting needs (shotcrete is compensating for *low cohesive strength*). Numerous tunnels and caverns spanning an order of magnitude of dimensions (3 to 30 m), and four orders of magnitude of rock quality (0.001 to 10) provided the guiding details of support needs.

The fact that $Q_c$ strongly resembles *c x tan φ* (different from *c + tan φ*), perhaps emphasizes the inherent utility of this number as a multiple-orders-of-magnitude descriptor of rock mass conditions. The simple peak 'c' and peak 'φ' estimates, as illustrated below, have potential limitations, but they are *transparent*, and can be rejected (with due care), if really considered unrealistic. In this connection it is considered that rock mass *cohesive strength* seems to be systematically under-estimated in numerical modelling.

Table 2. Illustration of parameters CC (MPa ) and FC° for a declining sequence of rock mass qualities, with simultaneously reducing $\sigma_c$ (MPa). Estimates of $V_P$ (km/s) and $E_m$ (GPa) are from Figure 9, whose derivation was described in Barton, 2002.

| RQD | Jn | Jr | Ja | Jw | SRF | Q | $\sigma_c$ | $Q_c$ | *FC°* | *CC* | $V_p$ | $E_m$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 | 1.25 | 26 | 2.5 | 3.6 | 11 |
| 30 | 15 | 1 | 4 | 0.66 | 2.5 | 0.1 | 33 | 0.04 | 9 | 0.3 | 2.1 | 3.5 |

There may be some additional advantages of this simple semi-empirical approach, because the pairs of parameters RQD/Jn and Jr/Ja are already being logged at a lot of mines, following Potvin and Matthews method, and the subsequent Modified Stability Graph, now in common use in Canadian and Australian mining for preliminary stope dimensioning.

The original technique has been well documented in the literature. The method is based on a graph of the ratio of stope face area and perimeter (the 'hydraulic radius') to the rockmass stability number N, which is a direct product of the factors: RQD/Jn ('block-size') and Jr/Ja (frictional strength) between the rock block surfaces. Three other

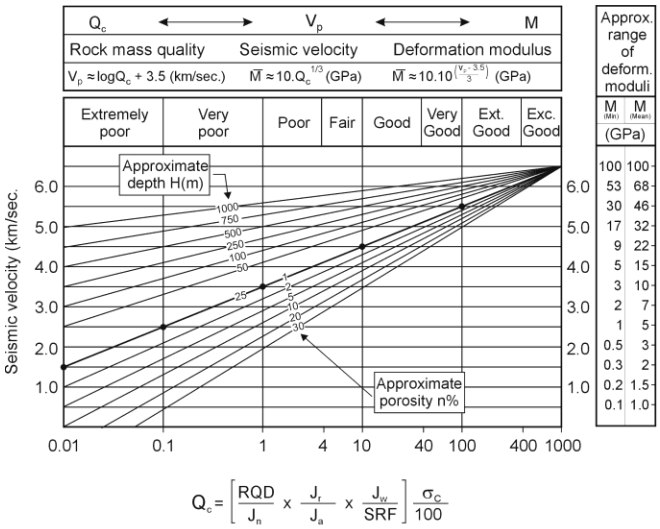



Figure 9.The integration of rock mass quality and seismic velocity, with adjustment (+ve) for depth (or stress level) and porosity (-ve), and rock strength. Estimates of static deformation modulus (right-hand column) should also be depth dependent. See example in Figure 4c that slightly pre-dates the above method of Barton, 1995.

factors are added in the N' method, described by Hutchinson and Diederichs. One of them is a necessary stress term to allow for the strength-to-induced-stress ratio, as traditionally used when estimating SRF for the Q-value.

## Q AND SINGLE-SHELL NMT FOR METRO TUNNELS

Chance or fate bought the writer to Norwegian line-where-needed, bolt-where-needed 'nominally unlined' hydropower tunnel territory in 1971. This region of Scandinavia eventually excavated about 3,500 km of such tunnels, 1,500 km of road and rail tunnels, and 200 underground power houses. In 1973 the Norwegian State Power Board (subsequently Statkraft) had requested a report from NGI, concerning a technical explanation as to why Norwegian hydropower caverns were displaying widely different magnitudes of deformation. This agency, now the owner of the most electrical generating capacity in the world, was apparently not hurt by waiting more than six months for their report, which could not actually be written until a rock-mass classification method had been developed.

The nature of the question meant that rock-mass *quality*, rock *support* needs (shotcrete) and rock *reinforcement* needs (bolts and anchors) for different sized openings, situated at widely different depths, needed to be linked to the different *deformations* recorded. This was a different and apparently more challenging problem than addressed when Beniawski developed RMR (rock mass rating) one year earlier. Why the Q-system (and Bieniawski's RMR) were not developed long before, may perhaps relate to the increasing use of more economic *single-shell* solutions. These are epitomised the world over in > 2000 big caverns of 20 to 60m span, for hydropower and other purposes.

The six orders-of-magnitude Q-value scale that was gradually developed during six months of trial and error, proved capable of answering the question posed, and has since been found to have simple links to joint, discontinuity and rock mass shear strength, deformation modulus, seismic velocity, seismic attenuation, as well as tunnel and cavern support needs, at depths from the surface to about 3 km depth. The guiding philosophy of support class selection has been to characterize the rock mass round by round, update prognoses, and use bolting and shotcrete of the best quality, or occasional concrete linings (but mostly unlined in the case of headrace and deep-pressure tunnels), to ensure faster and cheaper construction. Maximum water pressure head reached 1,000m about 20 years ago, where minimum rock stress justified this more sophisticated approach to rock engineering, than pressure-resistant linings with steel.

In contrast to *single-shell* tunnelling, the cost and schedule of metro construction that suffers from the 'short escalator syndrome', may cause just the *top heading* of the necessary *double-shell* NATM tunnels, to average no better than 10m/week, due to all the unnecessary mixed face conditions. Near-surface construction may also cause settlement damage to hundreds or thousands of houses, even far from the tunnel, if pre-grouting is unsuccessful, or not even performed. One may contrast the serious over-break seen in Figure 10, with the lack of over-break seen in Figure 11, where systematic pre-injection was performed throughout, resulting in improved Q-values.

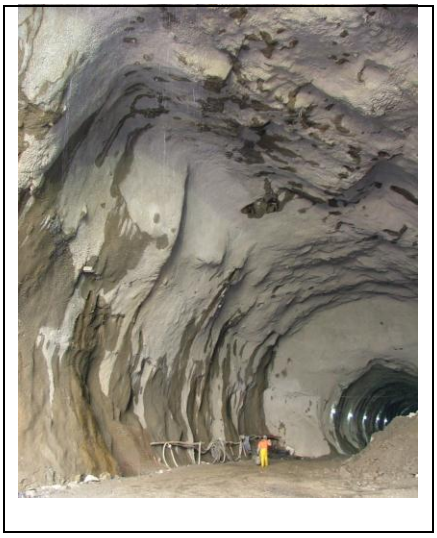


Figure 10. Over-break of 5 m and near break-through into the saprolite, following lack of pre-injection, and due to a cost-driving shallow metro choice by owners that took many years to complete.

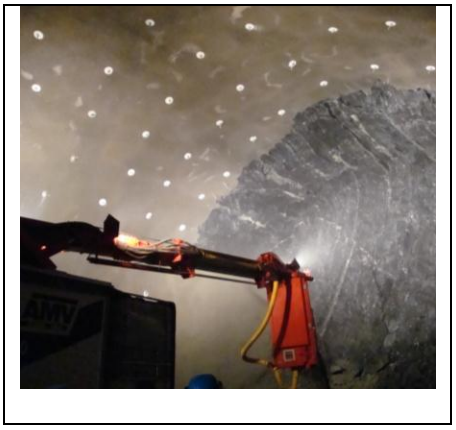


Figure 11. Systematic pre-injection and NMT permanent support (one final S(fr) layer is needed), gives an average 20 m per week for the completed tunnel, due to *Q-improved* rock mass properties.

Mixed-face construction can be avoided by *developing stations from underground* in better quality rock, or at least in the best geology within reasonable reach. This, of course, means London clay in London. The best available tunnelling horizon need not include much saprolite if the will is there to reach an 'economic' geologic horizon. Escalator advertising makes a 45 seconds stair journey more satisfying than 15 seconds; and this deeper 'rock engineering' approach may save years in construction time and give millions of city dwellers' earlier relief from traffic jams, due to five-year budgets extending to new projects.

## IMPROVING ROCK MASSES BY PRE-GROUTING

An important insight into the benefits of pre-grouting for tunnels was provided some years ago in Brazil, and involved what one might call pre-grouting for dams, since a grout curtain is also a form of 'pre-grouting'.Using boreholes spaced 4 to 8 m apart, Quadros and Correa Filho, 1995, measured the three anisotropic permeability tensors of the local rock mass. The same three holes were then used for local cement grouting, using just industrial cement and a small percentage of bentonite, as was normal in Brazil.

This was followed by re-drilling of the three holes in nearly the same locations, and a new round of 3D permeability measurements, using the three holes simultaneously as before, supplied with multiple piezometers, and a movable quadruple-packer assembly for pumping out water. These were used in rotation. The result was reduction of two permeability tensors (to 1/18$^{th}$ and 1/11$^{th}$ of the magnitudes), and rotation of all permeability tensors, implying joint-set sealing and reduced anisotropy.

As a result of these experiences, the writer argued (Barton, 2002) for the likelihood that most of the Q-parameters in a successfully pre-grouted rock mass, could change 'for the better'. In other words, due to partial or complete local sealing of one or more joint sets, logically starting with the most permeable set, there could be effects mostly better than the *very conservative* ones shown here:

$$Q = \frac{30}{9} \times \frac{1}{2} \times \frac{0.5}{1} = 0.8 \quad \text{changing to} \quad Q = \frac{50}{6} \times \frac{2}{1} \times \frac{1}{1} = 17$$

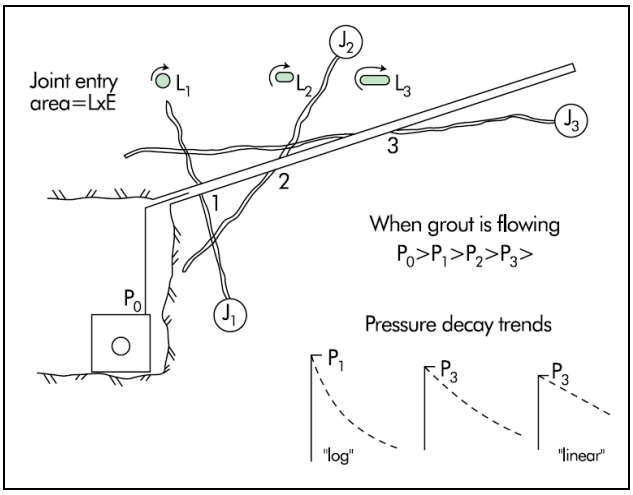



Figure 12. Pre-injection boreholes (40 to 60 in number, 25 to 30 m long) intersect numerous joint planes. Injection pressure must be high as pressure-drop is rapid away from each injection hole.

Pre-grouting at 5 to 10 MPa pressures is necessary to get a good result, like penetration of the typical 1 to 5 litres of grout per m$^3$ of rock in an assumed (mean) 5 to 6 m thick annulus of pre-grouted rock around a 100 m$^2$ tunnel, or approximately 500-1000 litres of grout per meter of tunnel. The logarithmic or linear pressure decay for Newtonian fluids, which should be even stronger for Bingham fluids, guarantees about 50% loss of pressure only 1 m from a given injection hole, while grout is still flowing in the joints. When flow has stopped, pressure must be reduced, otherwise hydraulic jacking will not be well controlled.

When an obstinate consultant prevents a contractor from using high enough grouting pressures, and limits use of micro-cement and additives, in a tight rock mass that nevertheless needs pre-grouting, due to overlying clay and risk of settlement damage, the result can be several of the following: abandoned settlement-damaged houses, leaking bolt holes, leaking shotcrete, at least 100,000,000 US $ of post-grouting, and one year of delays. There are examples with most of the above, in several different countries.

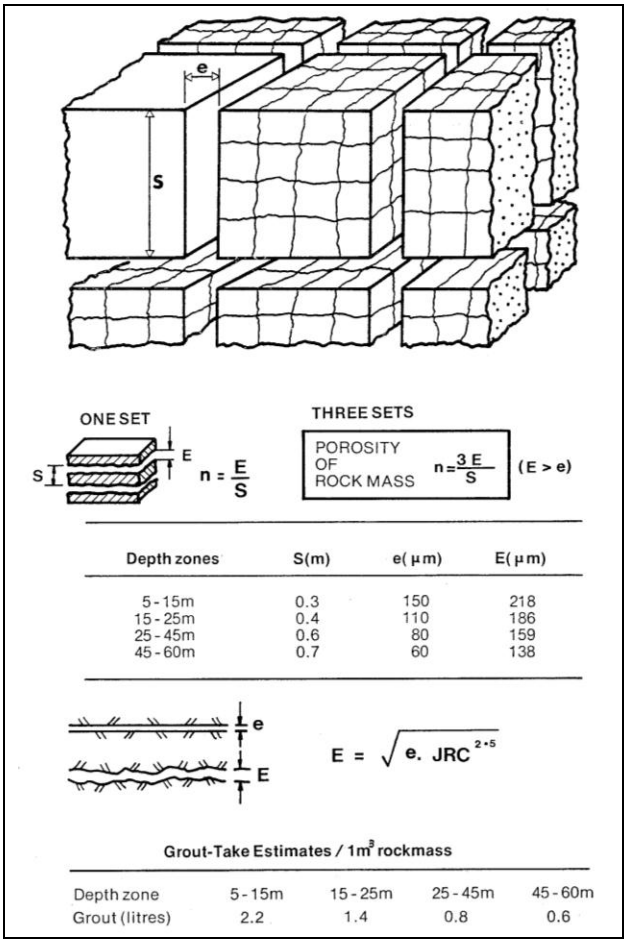



Figure 13. Application of a modified Snow, 1968 cubic network approximation to grout-take estimation at a dam site. This method (Barton et al., 1985) makes adjustment for larger physical apertures (E) than conducting apertures (e), using small-scale joint roughness $JRC_0$. So empiricism and theory are combined to estimate not only grout volume, but also appropriate size of grout particles, using another empirical-theoretical relation: $E \geq 3\ d_{max}$, (theoretical) or $E \geq 4\ d_{95}$ (empirical). The latter has also been shown to govern flow of blocks down ore passes in mines, where the mean physical joint aperture E is replaced by shaft diameter D.

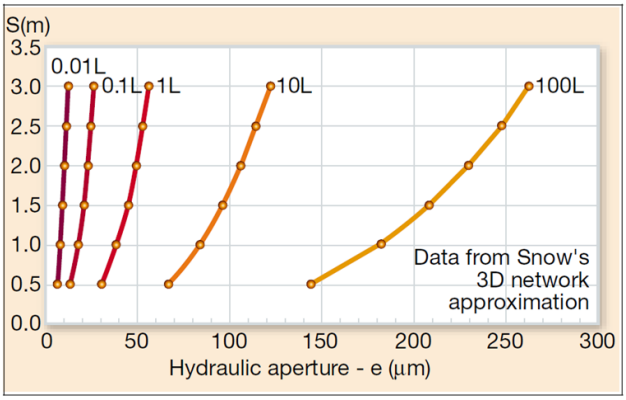



Figure 14. The estimates of mean hydraulic apertures (e) and mean spacing (S) of water-conducting joints are obtained from Snow's cubic network model, (Snow, 1968), in which permeability is assumed to be provided by two of the three joint sets, or by weaker contributions from the three sets when the flow gradient rotates.

The equation that can be derived from Snow's cubic network model, using non-American units, and the basis for Figure 14, is as follows:

$$e = (6LS \times 10^{-8})^{1/3} \qquad (7)$$

where e = mean hydraulic aperture (mm), L = Lugeons, and S = mean spacing of water conductors (mm) (Barton, 2004). The estimation of aperture (e) from Figure 14 and equation 7, depends on prior estimation of an average Lugeon value from a series of water injection tests (using constant packer spacing and *continuous test stages*. The all-important estimation of (S) is based on an assumed Poisson distribution of *water conductors* in relation to the *percentage of zero flow stages.* A final stage of the grout-take and grout-selection process is to estimate the equivalent physical joint apertures (E) using the $JRC_0$-based empirical relation from Figure 13, and compare it with the $\geq 4 \times d_{95}$ criterion for particle entry into the joints.

$$E \approx (e \times JRC_0^{\ 2.5})^{1/2} \qquad (8)$$

As a rough, and slightly conservative set of rules-of-thumb, the mean physical apertures may need to be ≥ 0.05, 0.1 and 0.4 mm, for respectively ultra-fine, micro-, and industrial cements. The approximate $JRC_0$ values can be estimated from the approximation: $JRC_0 \approx 7Jr\text{-}3$. This was derived from 1000 m of recent rail-tunnel core logging.

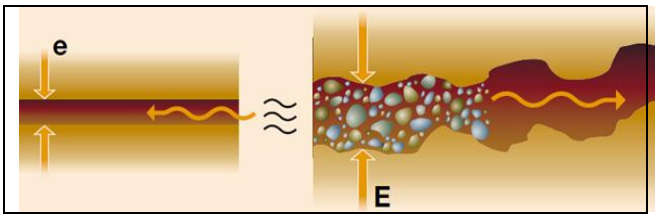



Figure 15. Blocks flowing down ore-passes, and micro-particles trying to penetrate into tight joints ahead of tunnel faces, need more space for more efficient flow, which can be created by reducing particle sizes and/or by increasing apertures locally, using *high pressure injection.* Fortunately joint aperture E is greater than e, which makes pre-injection easier than just considering the 3D network hydraulic aperture (e) trends shown in Figure 14.

## DEFORMATION COMPONENTS FROM BARTON-BANDIS MODEL

On the basis of the development of the empirical Barton-Choubey (1977) shear strength criterion (equation 2) involving the development of the joint characterization parameters JRC and JCS, Bandis made a remarkably thorough application and extension of these parameters, during studies of *scale-effects on shear strength* (using joint replicas), and also applied these parameters to numerous normal-loading tests on rock joints. His studies, and some of the author's multi-block studies, are summarised in Bandis et al., 1981 and 1983. Figure 16 shows a graphic summary of the joint closure (N) and shear components (S) of deformation, which were utilised in Barton, 1986 to explain the measured concave, linear and convex *load-deformation* trends of three different styles of rock mass. The dominance or combination of the N and S components are shown in the lower half of Figure 16, and confirmatory numerical modelling with UDEC-BB is shown in Figure 17.

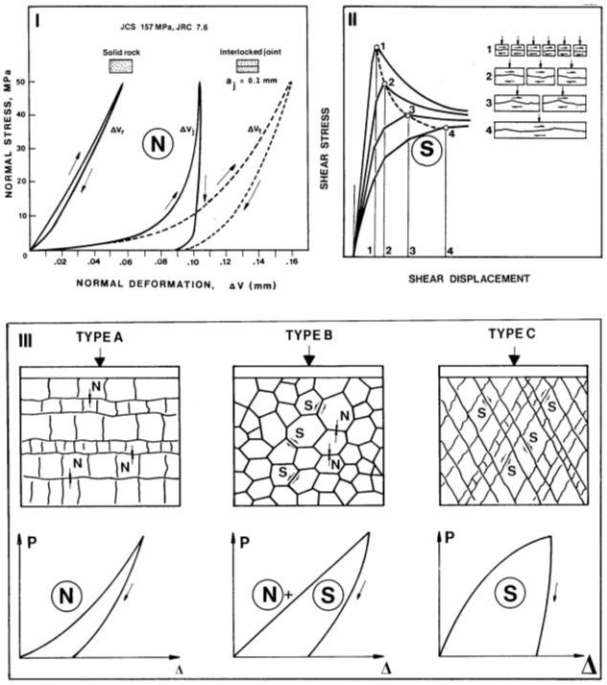



Figure 16. Joint deformation components N and S from Bandis et al., 1981 and 1983 applied to rock mass load-deformation understanding by Barton, 1986.

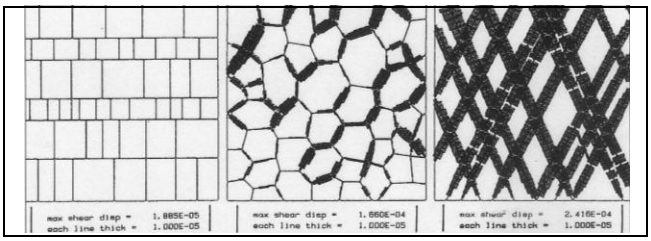


Figure 17. NGI's UDEC-BB modelling of the three contrasting 'rock masses' (Figure 16). The 'linearity' of the load-deformation for 'horizontally' loaded columnar basalt, matches the linearity of an in situ block test performed in the basalt Waste Isolation Project in Hanford, Washington, USA in the eighties. This model also assisted in recent major dam-abutment stiffness studies in China.

## MODELLING A COMPACTING SUB-SEA RESERVOIR

The Ekofisk reservoir in the North Sea was discovered during the very last attempts to find oil in this 'unpromising' petroleum province, at the end of the 1960's. With reservoir development from about 1971, and large-scale production from this giant off-shore field projected to last for 80 years, there was naturally concern when a supply-boat captain apparently reported that a reduced number of wave-baffle holes (to protect the temporary oil storage tank) appeared to be above sea levels in 1985, than at the beginning of production. These holes are spaced at 2 m c/c. (See effect of 13 years time interval in Figure 18).

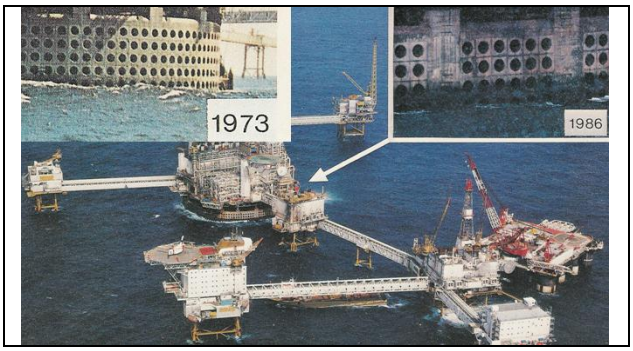



Figure 18. Compaction of the *jointed-chalk* reservoir of 300 m thickness at 3 km depth caused a rapidly increasing sea-bed subsidence that amounted to about 5 m when investigations began in earnest in 1985-1987. Following simultaneous raising of all platforms by 6 m in the late eighties, and provision of a 100 m diameter protective wall, and final re-location of operations away from the centre of the 9 x 14 km field, compaction has at least doubled, despite extensive sea-water injection for pressure maintenance (that also caused inevitable weakening of the chalk).

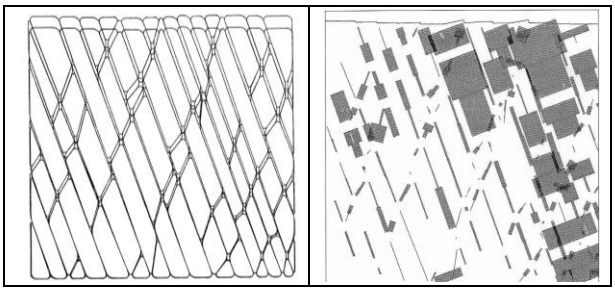


Figure 19. NGI and Itasca UDEC-BB 'uniaxial-strain' M-H modelling of a vertical 1.5 x 1.5 m 2D-element of jointed Ekofisk chalk, which had a porosity of 40%. Input data was obtained from JRC and JCS characterization of numerous joint samples that operator Philips recovered as part of the extensive exploration process. A vertical total stress of 62 MPa was applied, and internal pore pressure was reduced from 48 to 28 MPa, causing shrinkage of the porous matrix, and consequent space for down-dip joint shearing, despite one-dimensional strain. Barton et al., 1986.

The down-dip shearing that can occur despite one-dimensional strain, is a fundamental necessity for the continued conductivity of the joints, as the matrix is of low permeability, and cannot otherwise be drained. Of course 'rubble-ization' of the highest porosity chalk, caused by water-weakening and over-loading, is a further contributor to the excellent productivity of this huge reservoir.

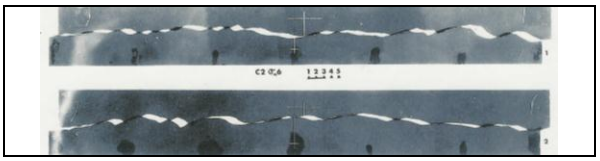


Figure 20. Rough tension fractures sheared as measured, give an exaggerated clue to the possibilities of maintained production despite compaction, in a reservoir with too low matrix permeability. For shearing to occur, steep dip is of course required. Note voids and contacts, and their contrary rotation. Barton, 1971.

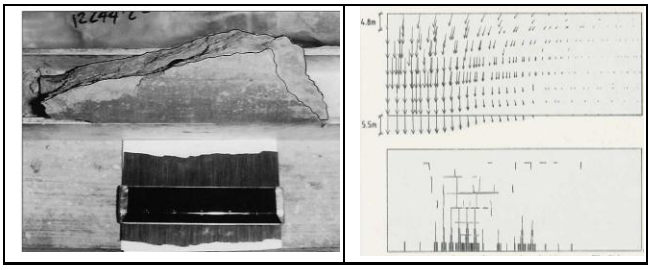


Figure 21. Left: An example of the steeply dipping conjugate jointing and its significant roughness. Right: One of NGI's axi-symmetric 10 km x 3 km subsidence studies with a 'bedded' UDEC model, showing a 4.8m/5.5m S/C ratio. Shearing events occur close to the reservoir and in the 'over-burden', and confirm one of the reasons for casing damage. High ratios of S/C, such as 0.85 *or more*, apparently matched estimates of S/C, that Philips obtained using radioactive targets fired into casing perforations.

There are several interesting observations to be added here, concerning the foregoing examples of discontinuum modelling at widely different scales. During exploration, slickensided joints in the chalk *were not observed*. According to Philip's geologist (H. Farrell, pers. comm.), slickensiding *was observed* in some much later cored holes connected with the water-injection operations, after 1985. Production was causing joint shearing, and is presently a seemingly ignored part of 4D interpretation (Barton 2006).

NGI's discontinuum approach to the modelling of Ekofisk compaction and subsidence, produced much higher ratios of S/C (subsidence divided by compaction) than that achieved by another team, who used 3D FEM continuum modelling with the large Dynaflow code. However, low values of shear stiffness appropriate to large scale bedding-planes, such as 0.01 MPa/mm were used (e.g. Barton et al., 1985), in contrast to typical 'lab values' of 1.0 MPa/mm. It is understood that *artificially low* values of shear modulus were needed to achieve better match to measurements, when modelling as a 3D continuum with Dynaflow.

## DISCIPLINE DISCREPANCIES IN EXPLORATION

While performing a wide-reaching literature search on geophysics reservoir-and-exploration topics (Barton, 2006), the writer discovered a seemingly serious discrepancy between geophysicist beliefs and the *presumed* realities of rock mechanics joint behaviour. The following example perhaps emphasises the need for real cross-disciplinary teams in petroleum and service companies, and a need to move beyond the belief that geomechanics is something that stopped developing after Mohr Coulomb. A case in point is geophysics expectation of stress-aligned reservoir fractures

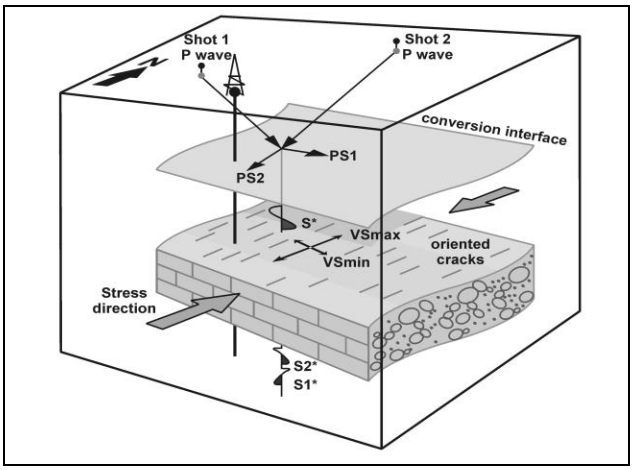



Figure 22. Wide-azimuth VSP exploration with P-wave sources, with conversion to pseudo-shear wave components. (Stenin et al., 2002). The assumed major-stress-aligned micro-cracks or single set of reservoir joints may however be closed by too high effective normal stress, and insufficient JRC and JCS (Barton, 2006), unless the more likely scenario of steeply dipping sets under shear is invoked, as for instance in Figures 19 and 23.

(micro-cracks or one set of joints) that are the assumed source of shear-wave splitting into fast and slow components. (See Figure 22 example). There may in reality need to be two joint sets involved, each under shear stress. 4D temporal rotation of fast S-wave axes, suggest this.

It is easy to show that unless joints in the reservoir are held open by mineral fillings, or unless for instance, a distinctly strong limestone with rough joints is involved, there will be insufficient apertures for production, at reservoir effective normal stress levels of tens of MPa. Barton-Bandis modelling with input data assumptions such as $JCS_0$ = 25 MPa and $JRC_0$ = 5, with moderate in situ block sizes, suggest closed stress-closure curves, and indicate that permeability can only be generated (or maintained) if shearing is occurring, as in Figures 19 and 22. If on the other hand, $JRC_0$ = 10 and $JCS_0$ = 50 MPa, some aperture can be maintained even without the benefits of shearing.

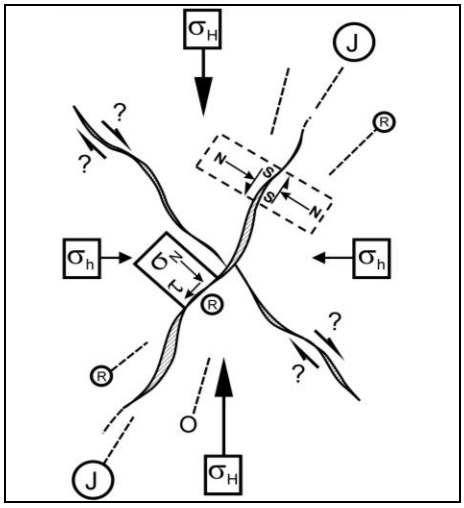



Figure 23. A scenario involving maintenance of permeability through shearing and slight dilation (exaggerated as in Figure 20) is more consistent with analysis of deep-well conductors and non-conductors, as emphasised by Zoback and co-workers at Stanford.

SHEAR STRENGTH AT EXTREME STRESS LEVELS

In 1976 the writer proposed a 'critical state' concept for the shear strength of intact rock at high stress, which involved both the expectation and the actual *horizontal orientation* of the Mohr strength envelope. This condition was shown or suggested by tectonophysicists' triaxial tests of various rocks at extreme confinement. Recently, this concept has been applied to better define the curvature of intact rock strength envelopes. A few tests at low confining pressures provide all the data needed for extrapolation to high levels of confinement. The elegant Singh et al., 2011criterion heralds a new era in rock mechanics understanding.

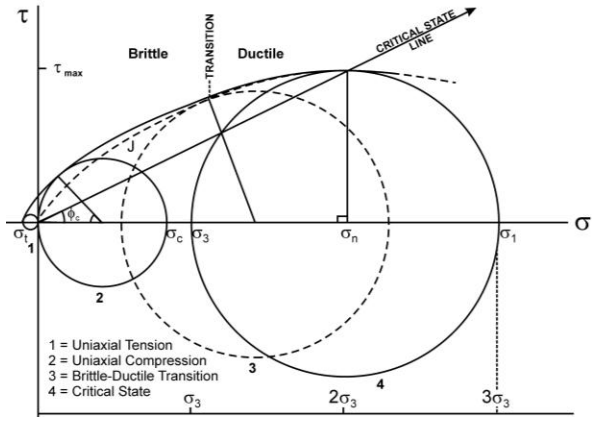



Figure 24. Critical state line for intact rock at extreme stress levels (Barton, 1976), suggested the limiting condition $\sigma_1 = 3\sigma_3$. Singh et al., 2011 show that most often, the critical value of $\sigma_3 \approx \sigma_c$. This seems to be logical, and helps to define a strength-envelope curvature which correctly matches test data at all confining pressure levels, and improves upon all existing strength criteria.

Since a blend of theory and empiricism has been promised in the title, with possible application to problem solving in rock engineering, the final figure to be presented will be of the tilt testing concept. This is at the other end of an extreme stress range. Tilt tests on joints of 10 cm and up to 1 m length, at normal-stress-at-failure of 0.001 to 0.01MPa, have provided $JRC_0$ and $JRC_n$ values for defining strength envelopes at three to four orders of magnitude higher stress.

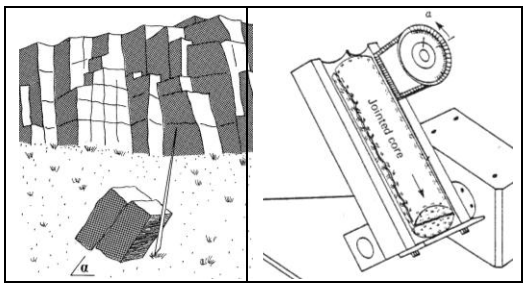



Figure 25. The tilt test result can be extrapolated from 0.001 to 10 MPa, and can be performed on samples of 10 cm to 1 m in size. The same method has also been used on 5m long as-built rockfills.

CONCLUSIONS

Jointed and anisotropic water-bearing rock masses most frequently represent the reality of engineering in rock. The variability of weathering, the presence of clay, and stress

magnitudes and permeabilities that usually vary spatially and with azimuth, represent challenges of quantification that require insight, some use of empirical methods like the *Q-histogram method,* and an absence of decimal places, which give a false sense of actual levels of understanding.

Those who restrict their rock mechanics experience to continuum modeling, inevitably miss many exciting insights in the rewarding field of rock engineering. It surely makes no real sense to collect data from discontinuous anisotropic media, and then convert it into isotropic continuum format.

Coloured appendices with their exaggerated 'plastic zones' may be misleading many clients, in view of the fact that shear strength is seldom 'c *plus* $\sigma_n$ tan $\varphi$', whether in linear or non-linear form. So many consultant offices in so many countries are producing coloured appendices that one can wonder if the colour is considered the most important sign of good-quality analyses. It may be wise for more young people to be educated for discontinuum modelling. This does require more understanding of rock mechanics.

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