Rock Engineering Challenges and Some Solutions for Hydropower Projects

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ABSTRACT: Hydropower projects in many mountainous regions, but especially in India and Kashmir, present geological, geo-hydraulic and construction challenges perhaps second to none. The tectonic influences, the intense jointing and continued deformation, the contrasting rock types, the high-pressure water - and barriers, the clay-filled fault zones, all combine to test the ingenuity of the designer and especially the contractor. The owner will always acquire something unique, due to the positive influence of human endurance and perseverance. This paper assembles some of the practical lessons learned by the writer during a forty-years professional career, spanning thirty five countries. The main topics will be headrace and pressure tunnels, both by drill-and-blast and by TBM, and how to make these more economic, and perhaps avoid big delays. There will be liberal use of the Q-system, also for its use in TBM prognosis through the Q_{TBM} method. Severe delays can be explained and may be mitigated. Single-shell NMT tunneling is preferred to double-shell NATM due to speed and cost.

1 INTRODUCTION

The writer has had the privilege of working on hydropower projects in many exotic places, during a 40 year time-span, and is hoping to continue during this next decade. Hydropower projects, almost by geographic necessity, can bring one to some of the most beautiful locations in the world. Once there, often after memorable travels, the rock mass related challenges occupy one for weeks or years, depending on the label 'expert' or 'designer' or 'contractor'. Thankfully, the rock mass and the hydrogeology know nothing about 'continuum analyses', and indeed demonstrate this repeatedly. Figure 1 is a simple demonstration of reality. As for medical doctors and their aging patients, gravity never takes a rest, and the rock mass seldom gets stronger, in fact usually weaker on an engineering time scale. There are jobs 'for everyone' – but some widely different answers and opinions. That is part of the fascination and challenge of rock engineering, especially when applied to the solution of hydropower problems, where there are many possible choices. Cost and time *can* be saved.

2 THE Q-SYSTEM BASED SINGLE-SHELL NMT METHOD

With 3,500 km of hydro-power related tunneling, about 180 underground power houses, and hydro-power competition with the investment needs of a growing off-shore oil industry, it was necessary to construct economic tunnels (and power-house caverns) in Norway. The Q-system development from 1974 always reflected this, and 50% of initial case records were from Norwegian and Swedish hydro power projects, with *fifty different rock types* in the first 212 case records. Contrary to popular belief, few cases from the Pre-Cambrian and mostly high quality bedrock could be used, unless they were challenging shear zones with clay-coated joints, and sometimes hydrothermally altered rocks with swelling clays. One cannot develop a rock mass classification system from cases of 'no support needed', when Q is so often in the range 10 to 100 in these basement rocks. Yet some believe Q cannot be used 'in their country' due to all the granitic gneiss that they imagine accounts for the Q-system development. This misunderstanding is perhaps understandable, but is nevertheless a pity.

Fig. 1 Chlorite-filled and graphite-coated discontinuities in a Norwegian headrace tunnel, causing over-break, a typical case record for Q-development. Fault-zone and swelling-clay induced failure in Ponte do Pedra, Brazil.

The basic Q-system based reinforcement and support components B + S(fr) meaning systematic rock bolting, and (since the 1993 update) fiber-reinforced shotcrete, were developed from challenging conditions, with weathered rock, clay-fillings, shear zones and fault zones, sometimes with swelling clay, like montmorillonite, due to hydrothermal alteration. However, Norwegian (and world) records for drill-and-blast, single-face advance rates from the last decade, of > 160 m/week, and more recently > 170 m/week, and even >100 m/week as a whole-project average, for *single-face* drill-and-blast progress, of course give evidence of plenty of good rock, plus well-proven methods by the contractors. Cycle-times may be <5 hours in the best quality rock masses. Nevertheless, due to slower rates in the Svalbard permafrost, not '>100m/week', but 108 m/week was actually the single face average advance rate in the 5.3 km of these coal-measure rocks. This is ultra-efficient 168 hours-a-week NMT tunneling. In the next section, the advantages of B+S(fr) compared to the much slower temporary support components of NATM, like steel-sets or lattice girders, bolts and mesh reinforced shotcrete S(mr) will be emphasized.

3 CONTRASTING NMT AND NATM

Single-shell NMT, which was developed from, and is therefore ideal, for headrace and pressure tunnels, with appropriate layout concerning minimum rock stress levels, is fundamentally different from NATM. Mesh-reinforced shotcrete S(mr), still so often used as part of NATM, has not been seen for 30 years in NMT, and rock bolts are permanent (and multi-layer protected from corrosion), like the PVC-sleeved, double-annulus-grouted CT-bolts that are widely used as they are so simple to install. Lattice girders (and steel sets) have *never* been used in Norway, and have never been advised, as they allow too much loosening. Instead, a more secure rib-reinforced shotcrete arch (RRS) is used, as illustrated in Figure 3. This is systematically bolted. It is therefore far safer and more cost-effective than lattice girders. NATM requires monitoring, in part because of the increased deformation that steel arches allow. Imagine comparing the fracture energy of B+S(fr) and lattice girders.

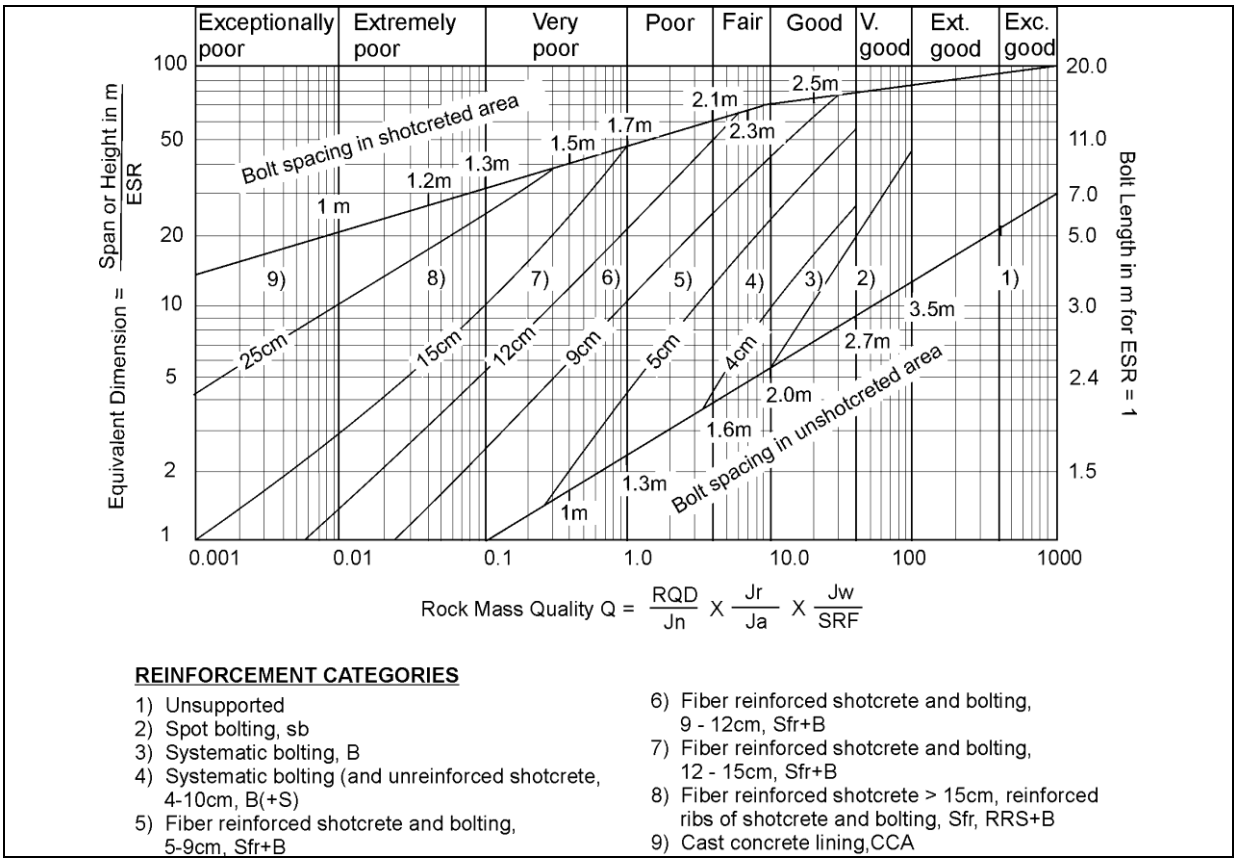

Fig. 2 The simple guide-lines of Q-based permanent, single-shell NMT support. Grimstad and Barton (1993).

The tunnel support measures B+S(mr) and lattice girders (or sliding steel sets in squeezing rock) are considered as temporary support in NATM, and design of the final concrete lining unfortunately ignores their long-term contribution. This is expensive, time-consuming, and can be considered wastage of resources.

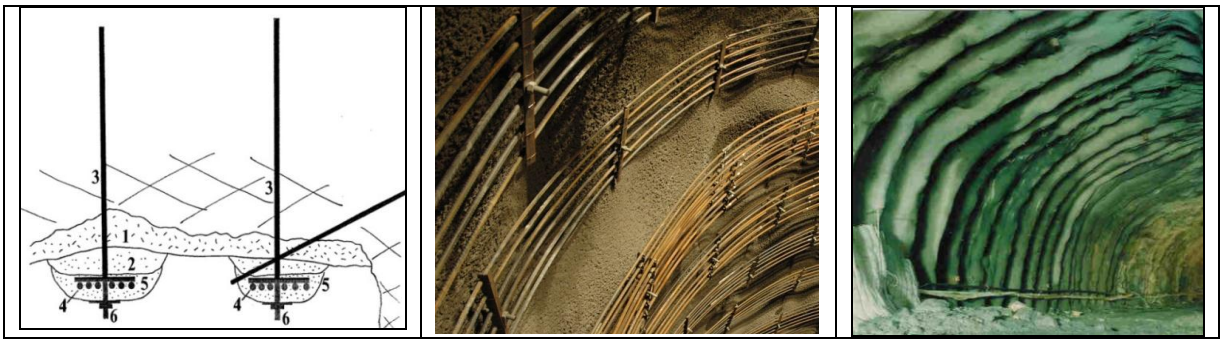

Fig. 3 In order to avoid 'standing-in-the-air' lattice girders, rib-reinforced, bolted shotcrete arches (RRS) are advised for the next-to-worst (category 8) rock masses, and of course in a headrace or pressure tunnel, will have local concrete, or 'fill-in' shotcrete, after stabilization. LNS photo in centre. N.Barton photo, right, Oslo station.

It is fundamental for fast tunneling to have robotic application of S(fr) readily available at the tunnel face. If there is some water inflow, sprayed arches of non-alkali accelerated S(fr) may be the best initial solution, as the water can drain between the arches and one is not fighting the water pressure. Too much water suggests the use of pre-injection, so that not only water problems are solved, but also rock mass quality improvements occur, which actually reduce the need for so heavy (multi-layer) support and bolting reinforcement. Bolt spacing can be increased if pre-injection is successful, but it has to be done well ahead of the face using high pressures of at least 5 MPa. In poor conditions packer depths behind the face can be increased.

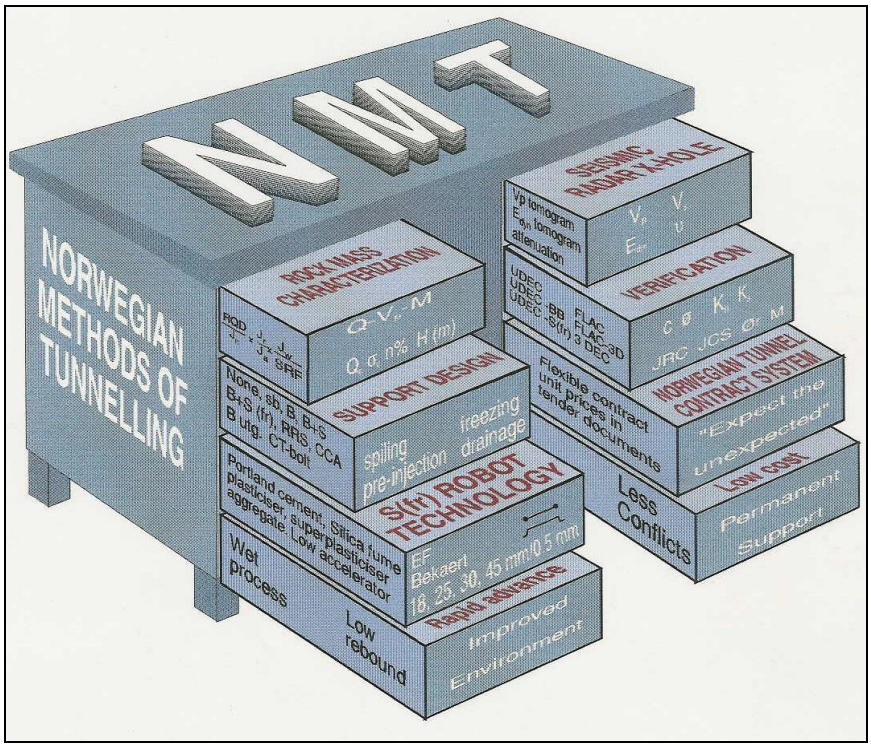

Fig. 4 A collection of practices that help to speed tunnelling, and to reduce overall tunnel costs, despite the high unit prices of some of the components, such as microsilica or silica fume in the shotcrete (and in the micro-cement for grouting. It is a false economy to use cut-price materials, like easily bent and broken steel fibre, but polypropylene 'Barchip' fully roughened fibres are proving to be an attractive competitor to the traditional stiff steel fibre. Pre-injection takes time, maybe 20 to 30 hours for the cycle of drilling and injecting, but the 'delay' may be an excellent investment in more secure, predictable tunnelling. The drilling of long probe holes is itself a good investment in reducing risk. Investment in faster equipment (drill jumbo, shotcrete robots) will pay off. When conditions are very poor, NMT includes a local cast concrete lining. If this is desired as final lining anyway, i.e. NMT first, CCA last with invert, the blasted section can be reduced in area, for hydraulic reasons. However the cost and time involved in final lining, compared to smoothing with thicker S(fr), needs careful comparison, thinking of both the higher unit cost, *but not total cost*, and the large saving of time.

Fig. 5 Warnings about the ineffectiveness of S(mr) and the obvious advantages of S(fr), from Vandevall ('Tunnelling the World') were not exaggerated. Despite its higher *unit* cost, one-process S(fr) will make a tunnelling project cheaper and faster, and also safer than three-process S(mr). If a final CCA lining (with invert concrete) is insisted upon, the reduced deformation of the NMT phase will help to add to the cross-section available for power production. Use of thicker S(fr) in 'one' process, and no cast-concrete liner, therefore quicker project completion, needs careful economic assessment. It may pay off handsomely. See Figure 6.

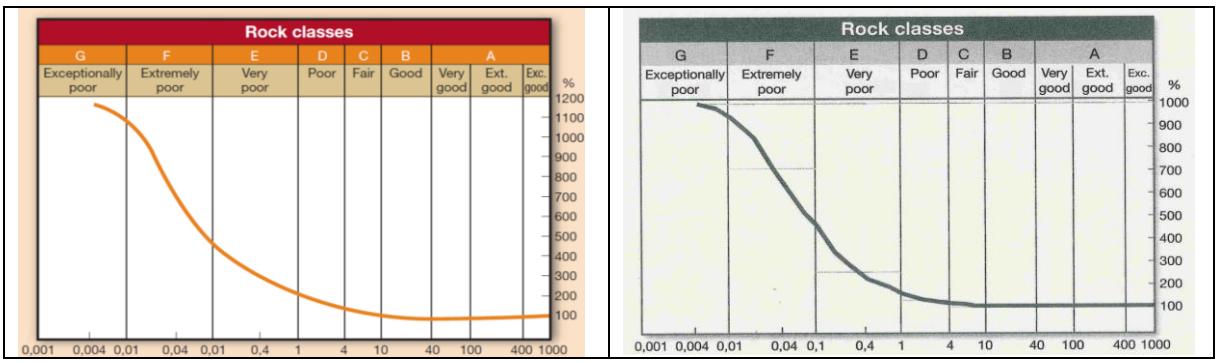

Fig. 6 Empirical data assembled by Roald, concerning relative cost (12:1) and relative time (10:1) as a function of the Q-value, for tunnel construction by contractors in Norway and Sweden who are employing B+S(fr) as final support. Barton, Buen, & Roald,(2001). Note that the cost and time for concrete-lined sections, is far to the left, at the top of each curve. Scandinavian contractors may *travel* with lattice girders and concrete-lining slip-forms, as part of their 'international baggage', but will use them as little as possible, preferably never.

As emphasized in Barton, 2011, in the third INDOROCK conference in Roorkee, it is in the steep area of the above two curves of relative cost and relative time, that one has the greatest benefit of performing pre-grouting, as both water and rock mass quality questions may be resolved. Progress will then be improved, and cost reduced, *despite* the investment in time (20-30 hours) and therefore local extra cost, when performing the pre-injection. If pre-injection is performed just as a canopy, leaving the invert unprotected, as has been tested in India, there is a risk in the invert of high inflows, more deformation, and possibly very muddy conditions. It is better to use 6 to 10 hours more, and include the invert in the pre-grouting. One can then control the water, as opposed to allowing its presence to remain, possibly in an even more troublesome manner.

4 TBM TUNNELLING WHEN HEADRACE TUNNELS ARE LONG AND DEEP?

The writer has had opportunities to work on TBM projects in various contexts, including deep road tunnels, deep mine access tunnels, deep petroleum-pipeline tunnels, shallow rail tunnels, but especially deep headrace tunnels for minor and sometimes major hydro-electric projects. Sometimes these have been through spectacular mountains in exotic countries. It has always been a surprise to register that 'because the tunnel is long' (which sometimes goes hand-in-hand with *deep*), the owners and designers chose TBM instead of drill-and-blast. This means that a significant investment is made for tunneling through a poorly investigated medium, quite simply because of depth limitations. While a mining company may have assessed mineral resources by means of ultra-deep boreholes, it is seldom that a hydro-power development will have borehole depths more than 500 m. Usually they is less deep than this, sometimes with unexpected consequences.

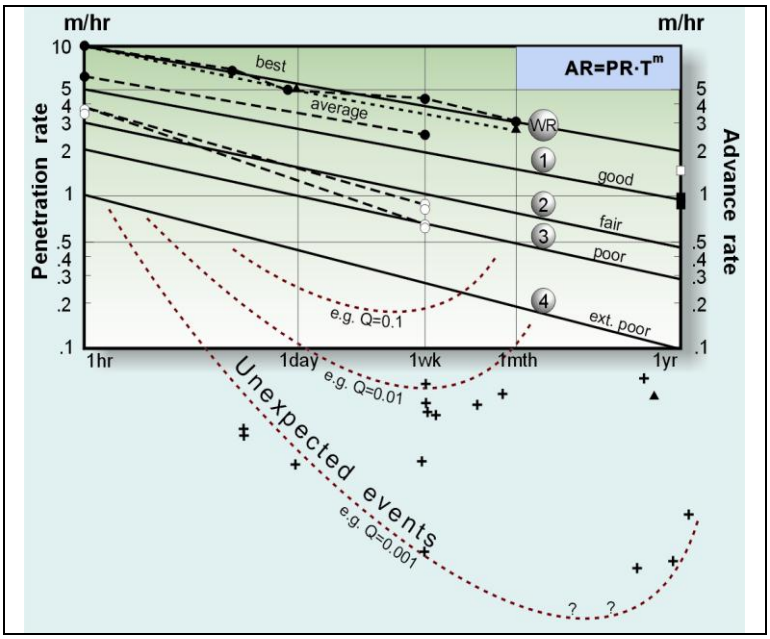

Fig. 7 A wide-ranging, 145 cases, 1,000 km search through open-gripper TBM case records led to these decelerating trend-lines. Below the lines of typical deceleration (negative gradient –m is mostly –0.15 to – 0.25), are curves representing some of the 'unexpected events', shown individually as crosses. These are seen to correlate with low Q-values, and include Dul Hasti in Kashmir. The development of an appropriate prognosis model: Q_{TBM}, followed on from this analysis of case records. The three lowest 'crosses' represent so serious conditions that TBM remain 'in the mountain', with drill-and-blast completion from the other end of the tunnel, sometimes with a new contractor following years of delay.

Figure 7 shows what was discovered from the case records: namely an inevitable decelerating trend, when analyzing advance rate (AR) of numerous TBM projects. *Deceleration* as tunnel length increases seems so far to be ignored, at least in public, by TBM manufacturers, consultants and contractors. The gradual deceleration of course comes after a 'learning curve' speed-up of penetration rate PR and actual advance rate AR, as a contractor's crews become familiar with what may be a new TBM. Even when breaking world records (see WR, top line), the 16 km in one incredible year also shows deceleration with increased tunnel length. Very respectable $AR_{after\ one\ year}$ averages of 1.0 m/hr represents 8.5 km in the year (curve #1, 'good'). At the other extreme, TBM 'remain in the mountain' forever, and drill-and-blast rescue, or tunnel completion from the other end, as for instance at Dul Hasti, Pont Ventoux, Olmos, and numerous other cases, is the alternative that should have been initiated many years earlier, while waiting for TBM delivery. Good advice is often ignored by *optimists*. *Pessimism* is sometimes a good investment. But don't ignore the advice in this paper, from whichever camp.

Less well documented case records using *double-shield* TBM, where it is difficult to characterize the rock mass, suggest about half the gradients of deceleration seen in Figure 7, because of concurrent PC-element lining efficiencies (of course at a price), with penetration of many smaller faults without major problems. In very rough terms we may be dealing with a deceleration gradient $m \approx -0.20$ in the case of open-gripper TBM, and $m \approx -0.10$ in the case of double-shield TBM. (In the latter, the PC element liner may be a continuous measure, or intermittent as-needed, as depicted in Figure 8). The utilization U from the classic equation AR=PR x U needs to be modified to the form $AR = PR \times T^m$, where T is the total time in hours. Time-dependence is clear.

When evaluating the possible advantages, or disadvantages of using a TBM for a long and deep headrace tunnel, the Q-value statistic is important, as can be imagined from the comparison shown in Figure 9, from Barton (2000). The modest 5 km in-one-year TBM project modeled here, shows the common need of central, well-jointed rock masses, if the TBM is to be consistently faster than drill-and-blast tunneling.

The inescapable dilemma is that as the TBM tunnel gets longer, a more-and-more 'central' rock mass quality distribution is needed, yet the *probable* reality, depicted in Figure 10, is that *the adverse extremes may themselves increase* as the tunnel gets longer. More hard rock at greater depth (HH in Figure 10) perhaps cannot be avoided, unless selecting drill-and-blast for this section. Pre-injection might solve some of the problems at the other extreme of low Q (F and FF imply fault zone, and serious fault zone, respectively).

Fig. 8 A headrace tunnel with intermittent (as-needed) PC-element liner, seen during inspection. The foreground is the initial drill-and-blast section of the tunnel, constructed while waiting for TBM delivery. Photograph by Dr. Nghia Trinh, SINTEF. Note that choice of drill-and-blast (for rough and larger section headrace tunnels) with permanent support by NMT (B+Sfr, the latter used to somewhat smooth the undulations/roughness) will always be cheaper and faster to construct than a final concrete lined tunnel. Less care on the temporary support phase of NATM invites more deformation. NMT in bad ground employs RRS (Figure 3), far superior to lattice girders.

Figure 9 was developed from application of the Q_{TBM} model for a moderately successful case (better than 'fair', curve #2, Figure 7), with 5 km in one year, assuming 8,000 hours of combined tunneling and maintenance time in the year. The D+B curves were estimated from Q-based cycle times in the different Q-value classes: from about 5 hours in the best rock to a day in the worst rock, with limited advance. Details of both models/assumptions are given in Barton, (2000).

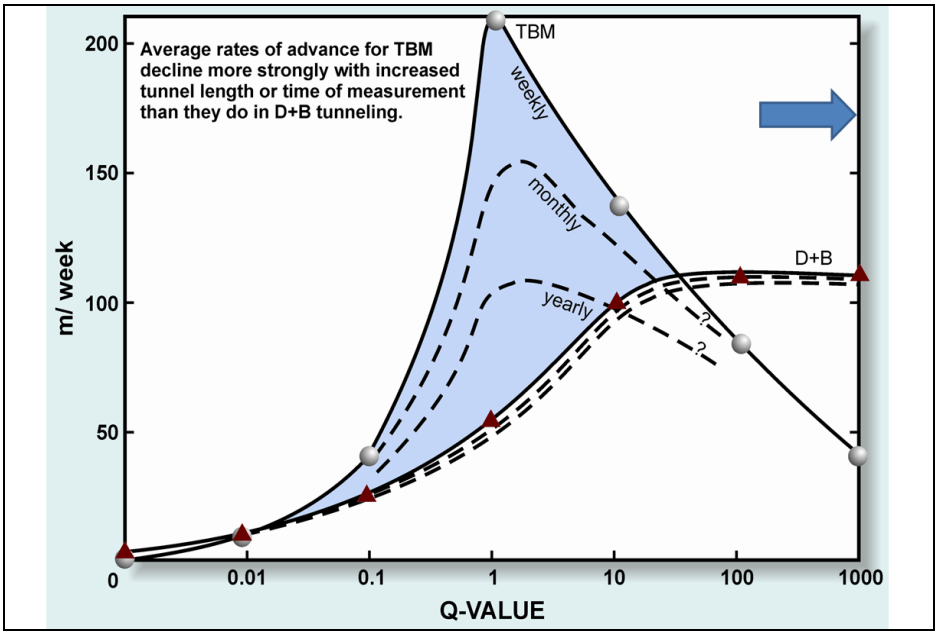

Fig. 9 Comparing drill-and-blast with an open-gripper TBM prognosis. In this example it is assumed that due to typical 'central values' of the machine-rock interaction parameters shown in Figure 11, $Q \approx Q_{TBM}$. The blue arrow shows the fastest drill-and-blast tunneling so far: 173 m in one week, one tunnel face. A project mean of 104 8/week in a one-face mine-access tunnel, through coal-measure rocks in Spitzbergen, also by a Norwegian contractor, has also been achieved. In very massive rock, with very frequent cutter change, drill-and-blast will be faster than TBM. It appears that it could also be faster if there is a *sufficient amount* of massive rock to slow the TBM, such that the TBM takes more than one year. Double-access for D+B, and only one TBM favours D+B.

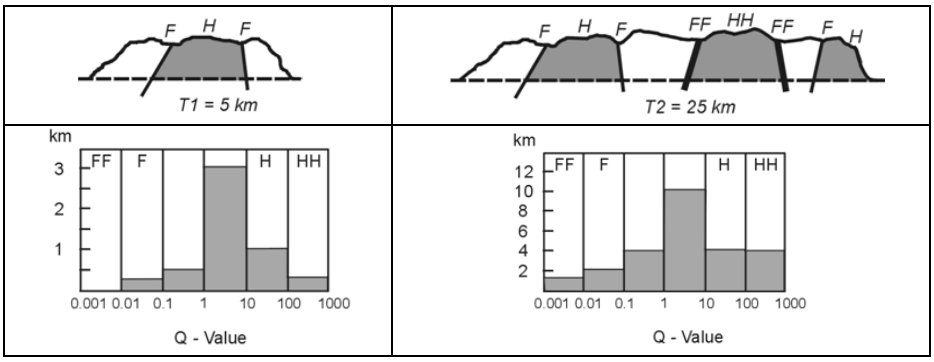

Fig. 10 The dilemma of rock mass quality distributions. Will they remain favourable when the tunnel is long, when even more central qualities are actually needed for continued good TBM performance? (HH signifies harder rock, and maybe higher stress, FF signifies a more major fault, and maybe higher water pressure on one side). We may also visualize a higher mountain range to the right, where FF, HH and FF occur. Part of the HH (extremely hard rock) effect will be due to the effect of confinement. In the Q_{TBM} model to be introduced in the next section, the biaxial stress-state on the tunnel face (σ_θ in Figure 11) is assumed to increase by an estimated 5 MPa per 100m. This is designed to reflect increased difficulty with maintaining penetration rate PR, due to higher confined rock strength. It is not infrequently that PR reduces despite increasing average cutter thrust (F in Figure 11). This is a sign of 'HH' (see conceptual Figure 10) and a temporarily (?) under-powered TBM.

5 THE Q_{TBM} PROGNOSIS METHOD

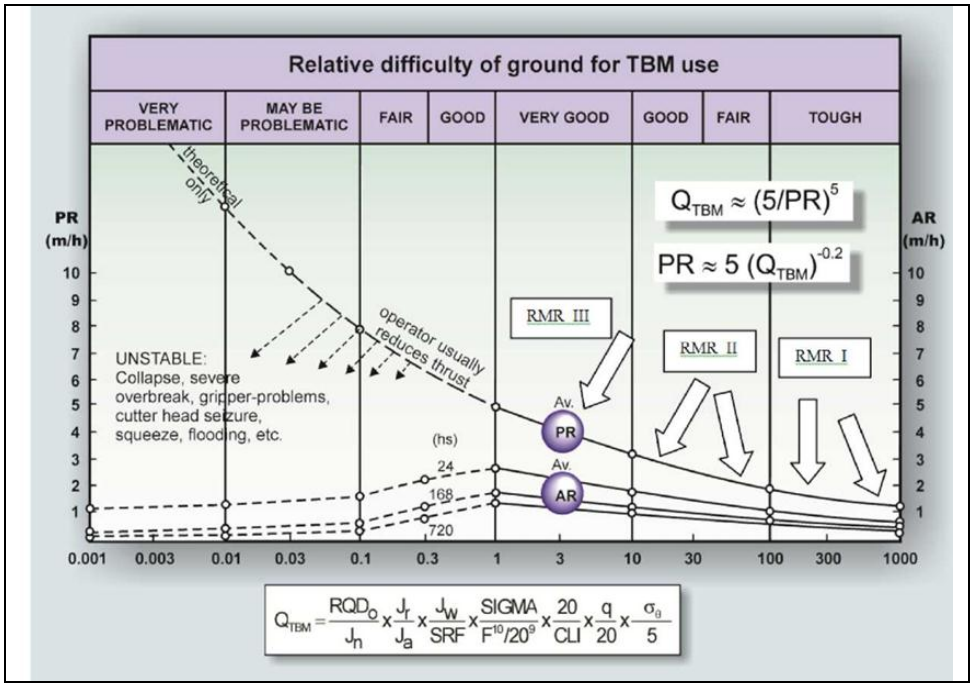

Fig. 11 This chart shows some elements of the Q_{TBM} method of TBM prognosis developed by Barton (2000). Note the 'new' adjectives to represent relative tunnelling difficulty. The 'familiar' 0.001 to 1000 logarithmic scale is used to emphasise that Q and Q_{TBM} can have similar values, as in Figure 9, with average 'central values' of F (20 tons), and SIGMA (rock mass strength = $5\gamma Q_c^{1/3}$ MPa) such as 20 MPa.

Note that the Q-value can have similar magnitude to the Q_{TBM} value, provided that sufficient cutter force F is available in relation to the estimate of rock mass strength SIGMA. Other parameters shown are quartz content (q)% and the cutter life index CLI. Low CLI, as in quartzite and granitic gneiss, with values as low as 5, are bad news for TBM penetration rate (PR) due to their hardness and rapid cutter wear, and advance rate (AR) is also adversely affected, with cutter-change as frequently as one-cutter per 1 to 3m TBM advance. In practice this means the need for many cutters to be changed every 24 hours, during the (extended) maintenance shift. High Q and high RMR rock mass quality values are equally bad news for TBM because low PR means low cutter-life.

6 WHY FAULT ZONES MAY DELAY TBM SO MUCH

There are unfortunately very good 'theo-empirical' reasons why fault zones are so difficult for TBM, with or without double-shields. (*Theological-empirical* means that lack of belief *in the empiricism* will be paid for, as believed to occur with religious dogma). We need three extremely basic equations to start with:

$AR = PR \times U$ (1)
$U = T^m$ (2)
$T = L / AR$ (3)
(Obviously the time T needed for length L must be equal to L/AR, for all tunnels and all TBM.)

Therefore we have the following:
$T = L / (PR \times T^m)$ But T appears on both sides: this equation can be rewritten as:

$T = (L / PR)^{1/(1+m)}$ (4)

This is a very important equation for TBM, if one accepts that *(-)m* is strongly related to low Q-values in *fault zones,* as shown by the empirical trend line, with adverse gradient to the left, in Figure 12.

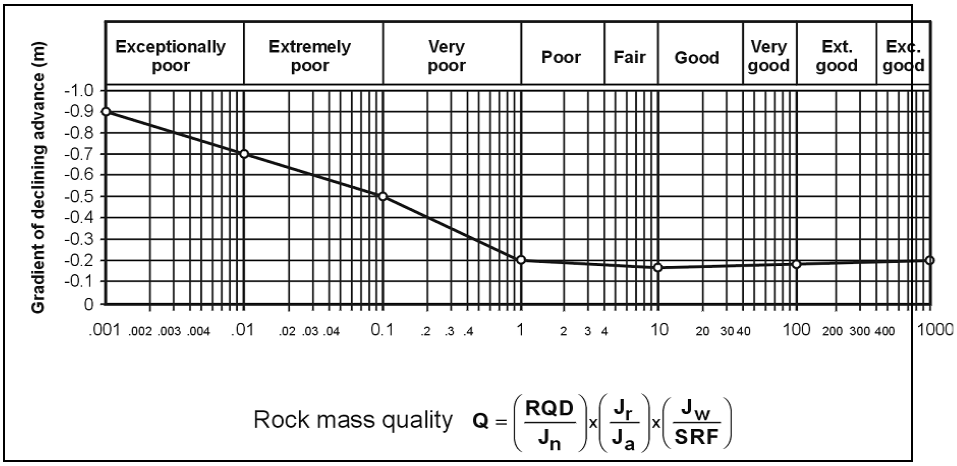

Fig. 12 Unexpected events seen delaying TBM performance in Figure 7, can be directly linked to too low Q-values, where steeper gradients of deceleration (-m) are seen. Just as in drill-and-blast tunnels, it is this region of the rock mass quality spectrum that may have greatest benefit from pre-injection.

Equation 4 is important because very *negative (-)m* values make the component $^{1/(1+m)}$ *too large.* If the fault zone is wide (large L) and PR is low (due to gripper problems and collapses etc.) then L/PR gets too big to tolerate a big component $^{1/(1+m)}$ in equation 4. It is easy (all too easy) to calculate an almost 'infinite' time for passing through a fault zone using this 'theo-empirical' equation. This also agrees with reality, in numerous, little-reported cases. The writer knows of several permanently buried, or fault-destroyed TBM (Pont Ventoux, Dul Hasti, Pinglin) and rockburst damaged or destroyed TBM (Olmos, Jinping II). There are certainly many more, and the causes can often be related to the logic and experience which are embedded in equation 4.

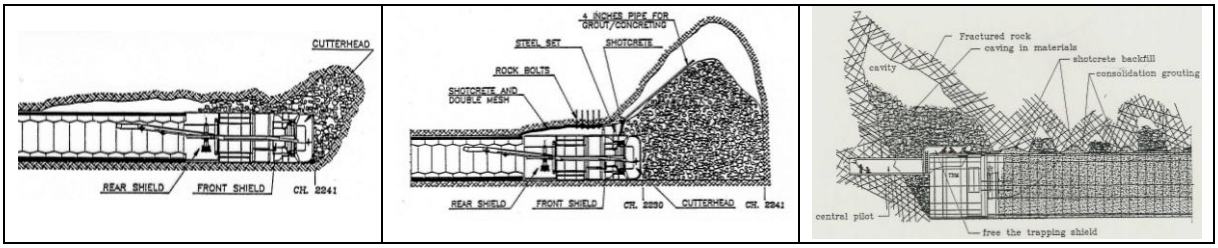

Fig. 13 Some examples of fault zones trapping TBM from Greece and Taiwan. Grandori et al., (1995), and Shen et al., (1999). Note that in the left and central example, the operator made the mistake of withdrawing the TBM (see ch. 2241 in both drawings). This is equivalent to penetrating, then unloading , a confined Vp at depth of 4 km/s, which becomes 2 km/s due to loosening, as if it was 'transported' to the surface. The compacted state of the fault is worth preserving, especially by probe drilling discovery, and high-pressure grouting, thereby reducing potential problems when the faulted rock is reached.

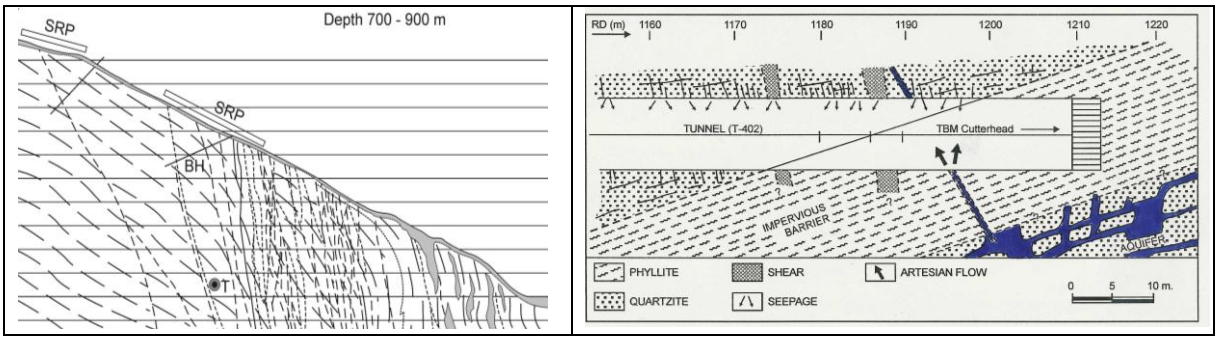

Fig. 14 The surprises that can occur with insufficient, or impossible levels of pre-investigation, due to depth limitations. Seemingly minor structures can 'sink' projects for 6 months to a year, and seem to have greatest negative impact in the case of TBM, because a contractor's ingenuity is hampered with 'something in the way' of pre-treatment. New generations of TBM (Figure 17) may help to mitigate some of these events, if used actively. (Left: fault swarms, discovered later, were missed by seismic and 200-300m deep boreholes. Right: Dul Hasti inrush/flooding event described and interpreted by Deva et a., 1994)

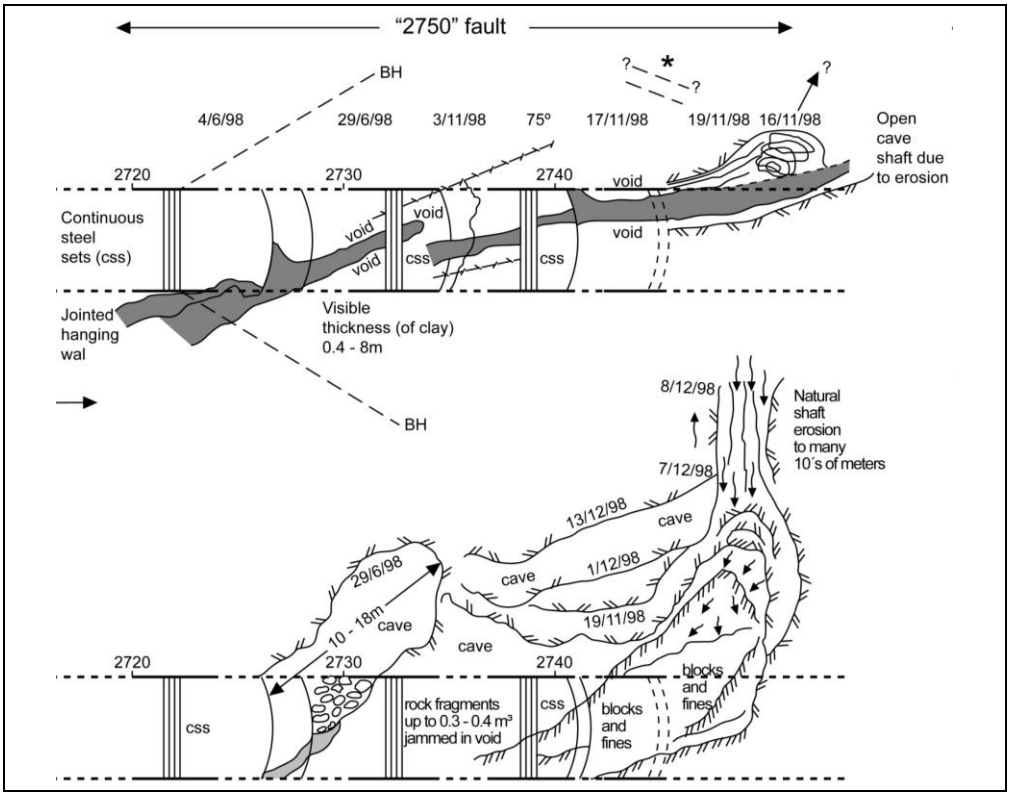

Fig. 15 A fault of moderate width with clay core, but with high water pressure on one side, proved to be an insurmountable problem for the poorly equipped 'inherited' TBM used at Pont Ventoux, in N. Italy. Note the 5 months of superimposed geologist's observations during 25 m of 'stop-go' lack of progress. After two or three such experiences, the TBM was abandoned, and the tunnel was completed by drill-and-blast from the other end. The curving tunnel, soon to run parallel to the mountain side, was becoming successively more sub-parallel to the swarm of (un-investigated) faults.

The large void above the finger-shield of this inherited TBM is shown in Figure 16, and the amount of water entering the tunnel, mostly from such faults, is easy to judge. The TBM tunnel was approximately 5m in diameter. Recently, Willis (2012) writing for Robbins Co., presented the 'All Conditions Tunneller' (ACT) depicted in Figure 17. This looks promising, as TBM manufacturers have gradually realized that TBM need pre-grouting facilities, just as do drill-and-blast tunnels. Of course it is a much greater challenge to provide such facilities for TBM, so it has taken a long time to see the gradually increasing number of holes in TBM shields.

Fig. 16 An example of the dreaded void (here 10m – 25m, at least) ahead *and above* of the cutter-head. In this location at Pont Ventoux, there appeared to be high water pressures on one side of the moderate-sized clay filled fault (Figure 15), resulting in erosion of a 'shaft' and regular falls of blocks, which successively trapped the cutter-head. Another less expected problem was the formation of a 'delta' of sand and silt just after the back-up, where water flow was slower. The muck-cars, reversed in by the train, had a habit of derailing before reaching their destination at the conveyor belt.

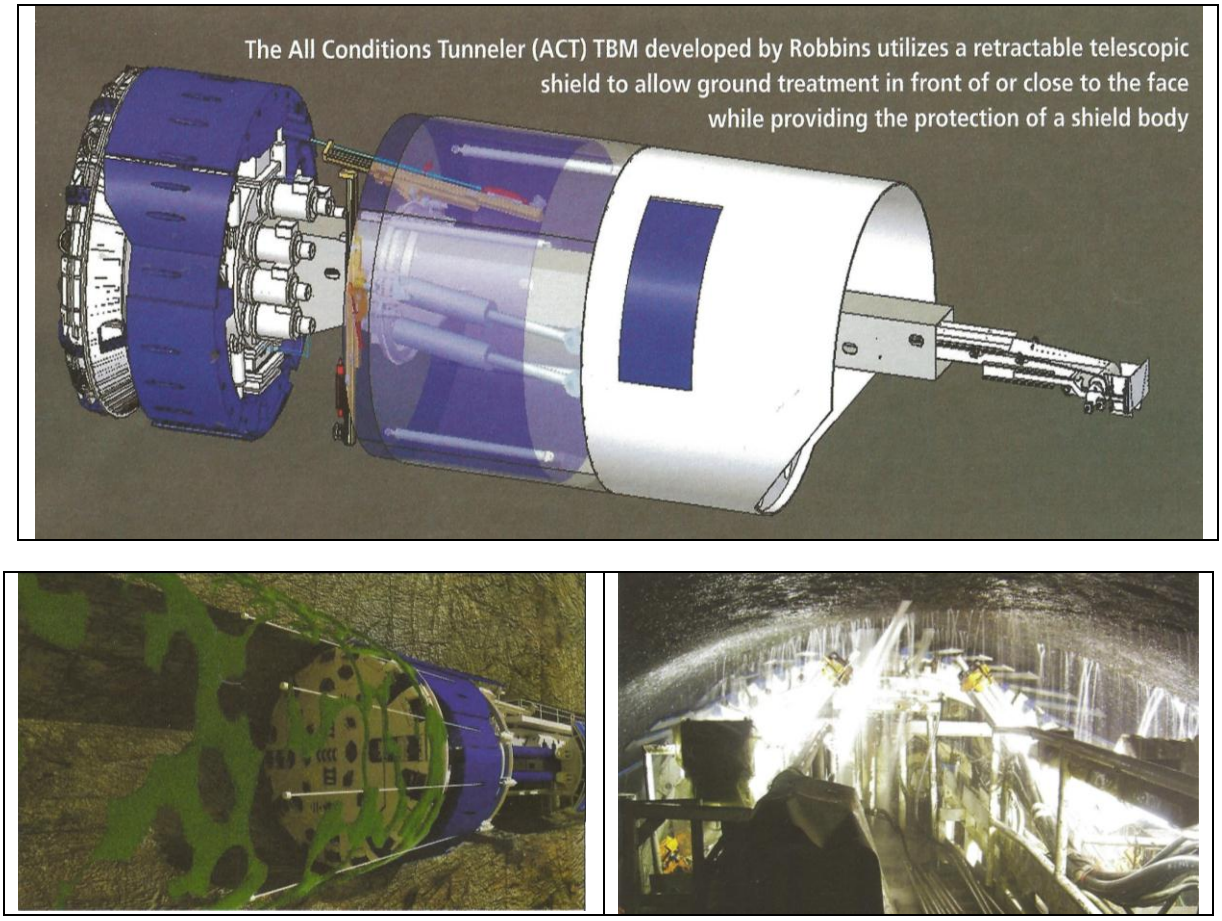

Fig. 17 Improved facilities for TBM probe drilling and pre-grouting. Willis (2012). The photograph shows an exceptionally wet tunnel, as reminder of conditions sometimes encountered, and the actual need of *pre*-injection.

7 NUMERICAL CONTINUUM MODELING

The Q-system based tunnel support selection chart shown in Figure 2, which is *a posteriori* in nature (based on experience), though widely used for two decades, is a recent target of some consulting company's critique. This has appeared it seems, since they discovered the colourful nature of *a priori* (based on assumptions) down-loadable methods of numerical continuum modeling, and black-box GSI-based algebra for estimating the assumed shear strength of rock masses. Their modeling sometimes suggested the need for longer rock bolts (than given in the Q-system) to 'secure' what actually may be imaginary 'plastic zones'.

Such has been proved in stringent legal surroundings – that plastic zones of many meters 'diameter' do not agree with the reality of a small-diameter and stable tunnel, excavated without need of shotcrete, which was in fact damaged by emergency emptying. It appears that 20m/hr may be a guarantee of failure, such as invert heave and block falls. On the other-hand 15 m/hr *may be* responsible for failure, while 10 m/hr seems to be acceptable: each of the above for competent jointed rock. In weaker rock provinces, 1 to 2 m/hr is generally advised.

The non-linear Hoek-Brown criterion, based previously on empirical data from numerous tests on *intact rock*, has been algebraically adjusted, in order to appear to 'work' for rock masses. This has led to many artificial 'plastic zones', also for stable tunnels. The reality is breakage of cohesion at small strain, and mobilization of friction (along near-by joint sets) at larger strain, as discussed and modeled in Barton and Pandey (2011). Both Mohr-coulomb and Hoek-Brown equations are in error. Also, do the two following Hoek-Brown equations, reproduced in Table 1, look as if they belong in rock engineering approximation?

Table 1. Those performing continuum analyses with Hoek-Brown, GSI-based 'input data' for 'c' and 'φ' are relying on this algebra (*and additional* underlying equations) to modify a valid empirical *intact rock* strength criterion, and make it appear to apply to rock masses. Something *a posteriori* has become an *a priori* method.

$\phi' = a\sin\left[\dfrac{6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}{2(1+a)(2+a)+6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}\right]$	$c' = \dfrac{\sigma_{ci}\left[(1+2a)s+(1-a)m_b\sigma'_{3n}\right]\left(s+m_b\sigma'_{3n}\right)^{a-1}}{(1+u)(2+a)\sqrt{1+\left(6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}\right)\Big/\left((1+a)(2+a)\right)}}$

What happens if a clay-filled discontinuity is found, or an extra joint set? Do we value and trust this obvious lack of transparency? The number of joint sets are absent from RMR, and therefore from GSI. (Jn is actually one of the most important parameters for rock mass stability, and the combination: Jn/Jr ≥ 6 helps to estimate the likelihood of over-break and therefore shotcrete volumes).

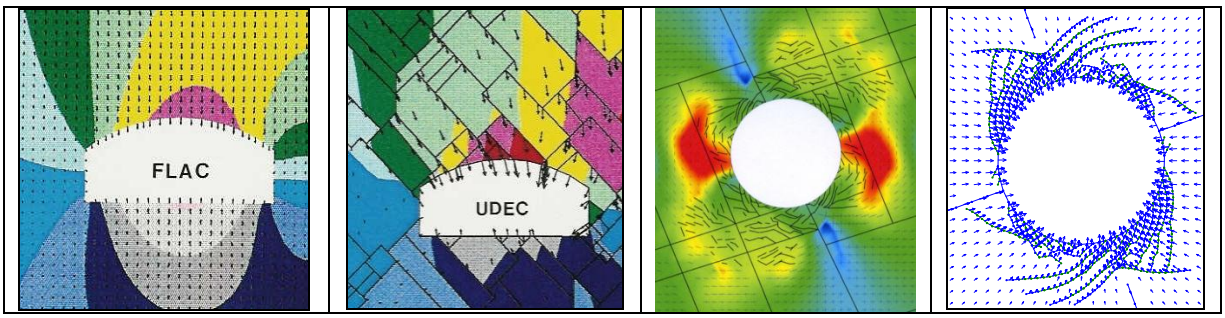

Fig. 18 Least understanding is derived from continuum modeling, much more is learned from discontinuum modeling, and when stresses are high, as in some ultra-deep headrace tunnels, stress-fracturing and rock-burst needs assessment: FLAC, UDEC-BB; FRACOD.(FRACOD modeling by Dr. Baotang Shen: priv. comm.)

8 CONCLUSIONS

1. Headrace and pressure tunnels for hydropower can be designed with NMT single-shell principles, using a larger cross-section drill-and-blast excavation, and deliberately choosing B+S(fr) as final support. Assuming all fault zones and any erodible or clay-bearing rock has been correctly treated, using Q-system based support and reinforcement, an additional conservatism can be added, by adding smoothing layers of S(fr). In a country like India, where tunnels tend to be time-consuming double-shell, similar to NATM and with final concrete lining, a change to 'NMT-plus' could have attractive cost and schedule benefits. 'NMT-plus' would need good quality shotcrete and fibres, and non-corroding rock bolts.

2. There are significant numbers of TBM projects that end up with difficult decisions to be made, namely to complete the projects by drill-and-blast from the other end of the tunnel. This on its own suggests that the TBM could or should have been used only on the better investigated portion of such projects, for instance the lower-cover section, with drill-and-blast started already from the other end, and deliberately chosen for the less investigated high cover sections in the case of hydropower tunneling through mountainous terrain.

3. The deliberate selection of *both* TBM and drill-and-blast may often be a simple matter of common sense, giving schedule advantages and cost savings. This is the preliminary level of hybrid tunneling. Very often, time is lost while waiting for TBM delivery and assembly, and great advantages could be gained by selective use of drill-and-blast for more than just the standard TBM assembly chamber and starter tunnel.

4. A second level of hybrid tunneling will be the deliberate choice, because of perceived advantages, of a) open-gripper TBM and drill-and-blast, or b) double-shield TBM and drill-and-blast. A third level of hybrid tunneling will be the deliberate choice, because of length of tunnel and perceived advantages, of both open-gripper and double-shield TBM, together with drill-and-blast on high cover and therefore poorly investigated sections. Clearly this would only apply at 'mega' projects with long tunnels.

5. TBM tunnelling and drill-and-blast tunneling show quite different performance in hard, massive abrasive rock masses, which are adverse for TBM performance, despite the excellent stability. TBM also exhibit adverse characteristics at the lowest end of the rock quality spectrum.

6. TBM gradually decelerate with time and tunnel length, even when breaking records. This is a natural process that should be a part of any realistic TBM prognosis, in preference to denial of its existence. Obviously, TBM that are operating in mostly favourable rock conditions, may record remarkable progress, and are therefore an excellent investment for part or all of many tunneling projects.

7. More uniform tunneling progress can be obtained, both in the case of drill-and-blast and TBM tunneling, if the advantages of systematic pre-injection through problematic stretches is better appreciated. The typical 24 hours 'delay' for a pre-injection 'umbrella' may save weeks or months in lost production. Some new designs for 'all-conditions' TBM, make pre-injection a viable proposition.

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