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Challenges and empirical solutions when tunnelling

Nick Barton reveals some highlights of his 40-year experience of working in the tunnel industry and reviews some important lessons when it comes to hydropower tunnelling.

Nick Barton 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, like over-night travel to Canyon Number 5 Railway Station in the incomparable Zagros Mountains of Iran, the rock mass related challenges occupy one for weeks or years, depending on the label 'expert' or 'designer' or 'contractor'. Return visits also occur, as projects progress. 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 with 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.

This review of some highlights and important lessons will start at the beginning. This was a still-remembered challenge: geological mapping of a short hydro power access tunnel in Western Norway in 1972, with Reidar Lien's instruction to also decide on suitable permanent support. How was one to decide, with limited engineering geological and tunneling experience? We all have to make that first professional start to our careers. Guidelines for rock mass description linked to tunnel support needs seemed to be lacking. Help was needed.

Intense study from 1973-74 and application in the decades following, have provided guidelines that can help in decision making, also for the less experienced, like the writer at that time, and many young people met on-site around the world. The Q-system development followed a question from the Norwegian State Power Board – 'what is the reason for all the different deformation magnitudes in Norwegian power station caverns'? From the start, excavation-size, depth and support quantities and methods had to be evaluated – against a newly created rock mass description – called Q. Luckily this had soon attained almost a log-scale, spanning six orders of magnitude of quality, during all the case-record analysis and re-analysis. A range from 10^{-3} to 10^{3} is not quite enough, but it represented a good start.

Figure 1. Clockwise – from top-left. The challenges of folded limestones at Bakhtiary Dam site in Iran. Characterization of macro-fractured columnar basalts at Baihetan Dam site in China. Fault-zone and swelling-clay induced failure in Ponte do Pedra, Brazil. Chlorite-filled and graphite-coated discontinuities in a Norwegian headrace tunnel, causing over-break.

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Some parametric simplicities

With Q_c (Q normalized by UCS/100) we now have an eight orders of magnitude rock mass quality scale, and as a result, embarrassingly simple equations linking Q_c to engineering estimates, like $E_{mass} \approx 10\ Q_c^{1/3}$ (GPa), $V_P \approx 3.5 + \log Q_c$ (km/sec), Lugeon $\approx 1/Q_c$, (but Q_{H2O} is better), deformation Δ(mm) ≈ SPAN(m)/Q. The latter as a central trend in the measured range from 1mm to almost 1000mm. Each of the above are depth dependent parameters, and are simply modified as required.

Remarkably, Q_c (=Q x UCS/100, with UCS expressed in MPa) can be split into two parts, and seems to represent the cohesive component CC, and the frictional component FC of rock masses, though of course nobody knows exactly what magnitude these should be in our ever variable rock masses. Q-system tunneling case records were apparently signaling lack of internal friction (= bolting needs) and lack of cohesion (= shotcreting needs). So for those who wish to perform scoping studies with continuum models, estimates of 'c' and 'Φ' are found. However, as pointed out, and utilized in Barton and Pandey, 2011, the cohesion should be degraded at small strain, and the friction mobilized at larger strain. It is actually unrealistic (actually incorrect) to add 'c' and σ_n tan'Φ', as in Mohr-Coulomb (or Hoek-Brown) shear strength criteria. This separation of cohesion and friction for rock masses, was suggested implicitly by Müller as long ago as 1966. When 'rock bridges' fail, and cohesion is broken, we know plenty about the remaining stress-dependent shear strength of the fractures and the rock joints that are then involved in the failure (e.g. from the Barton and Bandis, 1990 scale-dependent JRC, JCS and Φ_r criterion). Rock joints do not have cohesion unless stepped, despite what multi-stage testing of the same sample is erroneously suggesting.

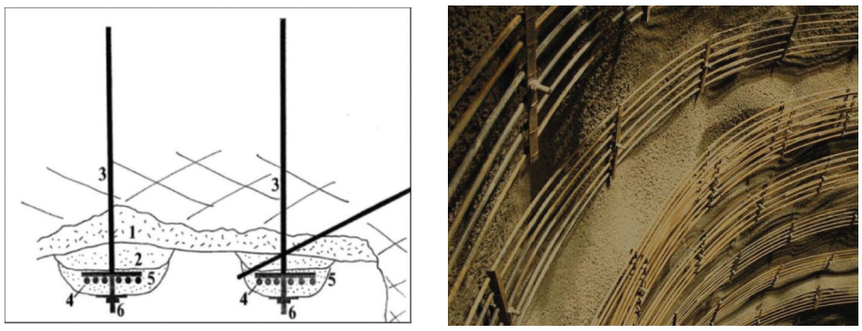

Figure 3. In order to avoid 'standing-in-the-air' lattice girders, rib-reinforced, bolted shotcrete arches (RRS) are advised for the next-to-worst (category 8) rock masses, and of course in a headrace or pressure tunnel, will have local concrete, or 'fill-in' shotcrete, after stabilization. LNS photo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

The Q-system based single-shell NMT method

With 3500km of hydro-power related tunneling, about 180 underground power houses, and some 1500km of road and rail tunnels, and hydro-power competition with the investment needs of a growing off-shore oil industry, it was necessary to continue to construct economic tunnels (and power-house caverns) in Norway. The Q-system development has 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 (mostly) high quality bedrock could be used – 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.

The basic reinforcement and support components B + S(fr) meaning systematic rock bolting, and (since 1993) 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 advance rates from the last decade, of > 160m/week, and more recently > 170m/week, and even >100m/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 108m/week was actually the single face average advance rate in the 5.3km of these coal-measure rocks. This is ultra-efficient 168 hours-a-week NMT tunneling.

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) as 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 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 cost-effective than lattice girders.

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 a wastage of resources.

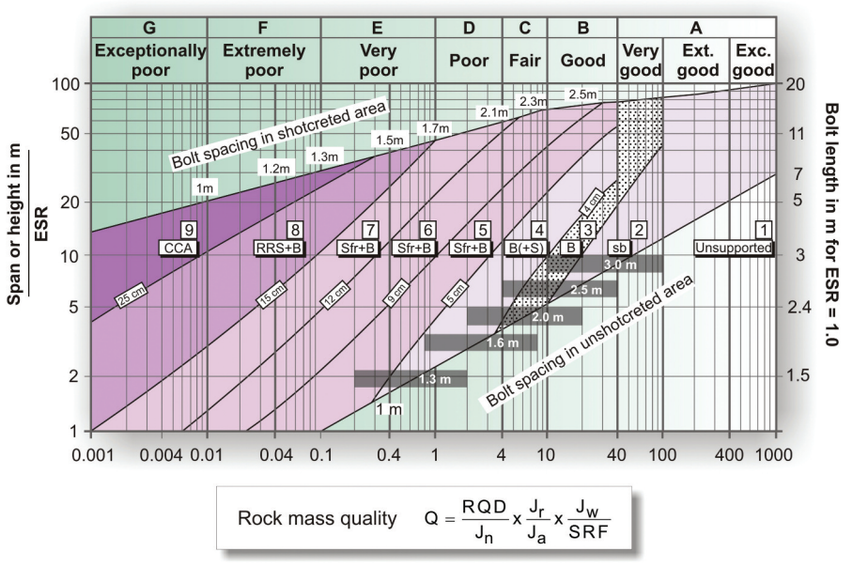

Figure 2. The simple guide-lines of Q-based permanent, single-shell NMT support.

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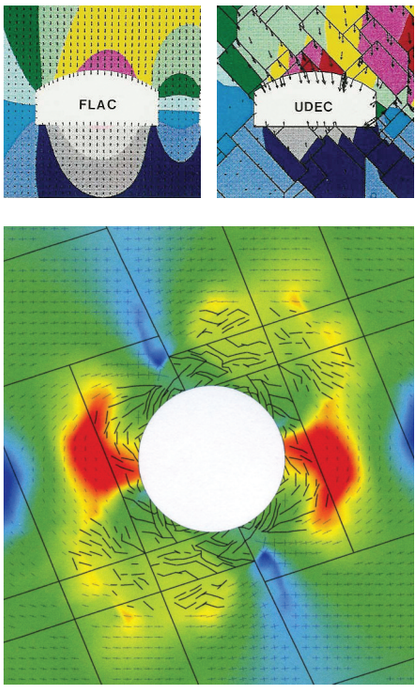

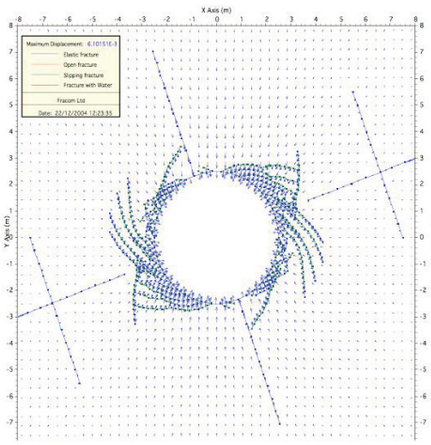

Figure 4. **Least understanding is derived from continuum modeling, much more is learned from discontinuum modeling, and when stresses are high, as in some ultra-deep headrace tunnels, stress-fracturing and rock-burst needs assessment: FLAC, UDEC-BB; FRACOD. (B.Shen, pers.comm.)**

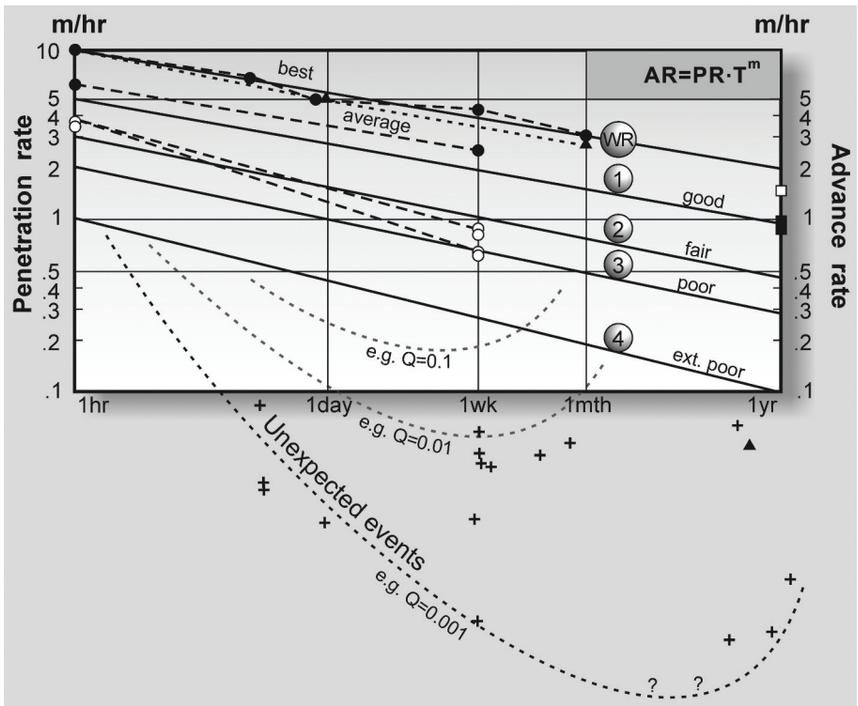

Figure 5. **A wide-ranging, 145 cases, 1000km search through open-gripper TBM case records, combined with personal experiences (including some of the crosses), led to these decelerating trend-lines, and the development of an appropriate prognosis model: Q_{TBM} .**

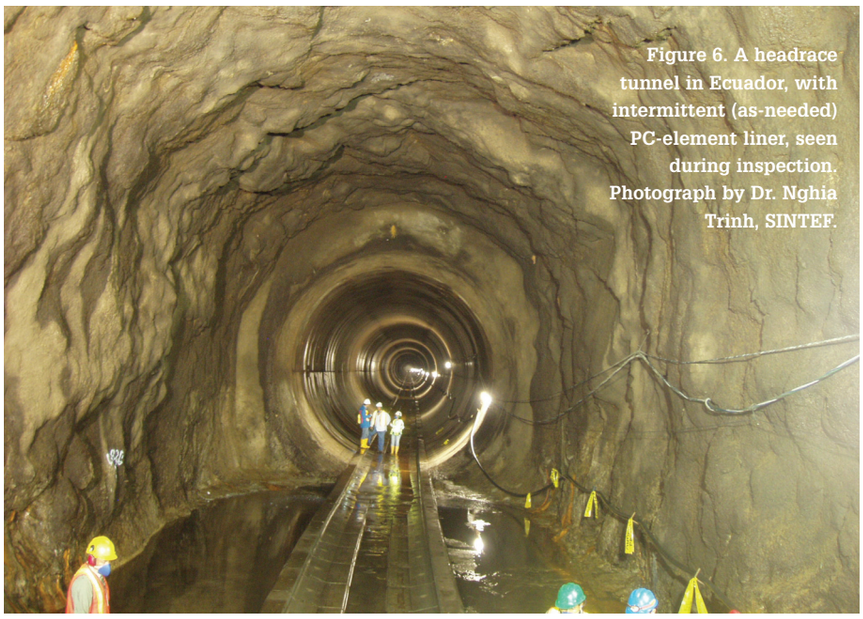

Figure 6. **A headrace tunnel in Ecuador, with intermittent (as-needed) PC-element liner, seen during inspection. Photograph by Dr. Nghia Trinh, SINTEF.**

Numerical continuum modelling

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 suggests 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.

The non-linear Hoek-Brown criterion, based previously on empirical data from numerous tests on intact rock, has been algebraically adjusted, in order to 'work' for rock masses. This has led to many artificial 'plastic zones', also for stable tunnels. Do the following equations look as if they belong in rock engineering approximation?

$$\Phi' = a \sin\left[\frac{6am_b\,(s+m_b\,\sigma'_{3n})^{a-1}}{2(1+a)(2+a)+6am_b(s+m_b\,\sigma'_{3n})^{a-1}}\right]$$

$$c' = \frac{\sigma_{ci}[(1+2a)s+(1-a)m_b\,\sigma'_{3n}](s+m_b\,\sigma'_{3n})^{a-1}}{(1+u)(2+a)\sqrt{1+(6am_b(s+m_b\,\sigma'_{3n})^{a-1})/((1+a)(2+a))}}$$

Those performing continuum analyses with Hoek-Brown, GSI-based 'input data' are relying on

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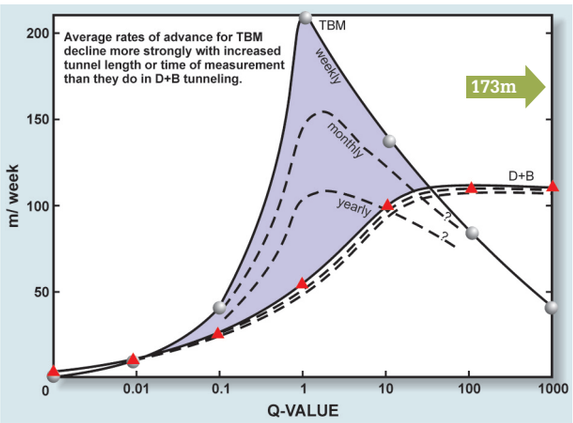

Figure 7. **Comparing drill-and-blast with an open-gripper TBM prognosis. The arrow shows the fastest drill-and-blast tunneling so far: 173 m in one week, one tunnel face.**

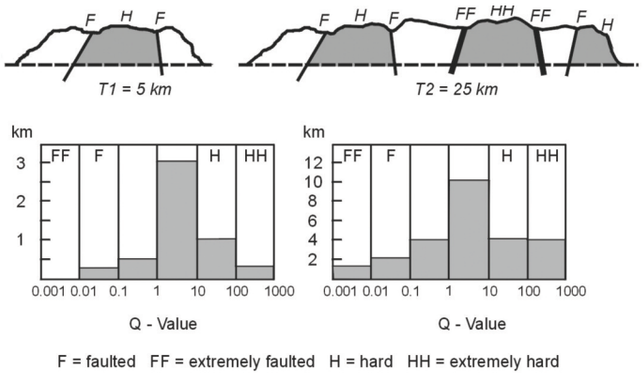

Figure 8. **The dilemma of rock mass quality distributions. Will they remain favourable when the tunnel is long, when even more central qualities are actually needed for continued good TBM performance. (HH signifies harder rock, and maybe higher stress, FF signifies a more major fault, and maybe higher water pressure on one side).**

this algebra (and additional underlying equations) to modify an intact rock strength criterion. What happens if a clay-filled discontinuity is found, or an extra joint set? Do we value and trust this obvious lack of transparency? 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 over-break and shotcrete volumes).

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 choose 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 500m. Usually they are not so deep.

Figure 5 shows what was discovered, but seems so far to be ignored, at least in public, concerning the deceleration of TBM as tunnel length increases. This 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 (like the WR top line), and 16 km in one incredible year, there is deceleration with increased tunnel length. At the other extreme, TBM 'remain in the mountain' forever, and drill-and-blast rescue, or tunnel completion from the other end, 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.

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 5, because of lining efficiencies (of course at a price), and penetration of many smaller faults without major problems. In very rough terms we may be dealing with a deceleration gradient m ≈ -0.20 in the case of open-gripper TBM, and m ≈ - 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 6. The utilization U from the classic equation AR=PR x U in fact needs to be modified to AR = PR x Tm, 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 7. 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 reality, depicted in Figure 7, is that the adverse extremes may themselves increase as the tunnel gets longer. More hard rock at greater depth (HH in Figure 8) 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.

Conclusions

Rock mass variability suggests that rock mass classification and empirical links to performance, may be better than *a priori* continuum modeling, with questionable complex algebra. Single shell Q-based NMT is ideal for hydro-power tunneling, and advance rates and cost are obviously favourable in relation to NATM and concreted final linings. Long deep tunnels may not favour TBM, despite the frequent belief 'because it's long we used TBM'. ■

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