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Strengths and Weakness of NMT and NATM and due Care with Numerical Modelling

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ABSTRACT

The Q-system was developed in 1973 from case records in which just bolts and shotcrete B+S or B+S(mr) were the principal reinforcement and support, with concrete-lined sections reserved for seriously faulted rock. A majority of cases were hydropower tunnels and caverns. In 1993, Grimstad and Barton published an overdue update incorporating B+S(fr) based on Grimstad's extensive collection of mostly road tunnel case records in which the revolutionary steel fiber reinforced shotcrete was used. This represented a paradigm shift. It was already seen in 1979 in a western Norway hydropower cavern, and in 1980 it was used for a central Norway road tunnel. The term NMT was coined in a multi-company World Tunnelling article by Barton et al. (1992) and also by Grimstad and Barton (1993). NMT emphasized single-shell tunnel and cavern support as compared to double-shell NATM with its final concrete lining. In these Norwegian updates, RRS-rib reinforced shotcrete arches were already described, and their design and selection were subsequently improved by Grimstad and his former NGI colleagues. In this *keynote paper, the differences between NMT and NATM will be emphasized, including the filling of over-break in the case of NATM, but not with NMT, and the use of 'soft' unbolted lattice girders in NATM compared to the stiffer bolted RRS arches. For water control, high-pressure pre-injection with stable grouts is common in NMT, while drainage fleeces and membranes and final concrete are standard elements of NATM road and rail tunnels. The paper contains some critical comments on numerical modelling, focusing on the illogical GSI and Hoek-Brown approximations, and the assumption that 'plastic zone' modelling might justify adjustments to empirical design routines. Using UDEC-BB or 3DEC more realistic behaviour is seen.

Keywords: NMT; NATM; Q; Shotcrete; RRS; GSI; UDEC-BB

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1. INTRODUCTION

The frequent assumption of those who feel they know best is that the Q-system only applies to typical hard jointed rocks. We actually make wider use of Q in NMT: the Norwegian Method of (single-shell) Tunnelling. The original case records included 50 different rock types in the initial two hundred or so cases that were analysed. There was deliberate choice of challenging cases such as clay-bearing and sheared rock masses, so that significant amounts of support could be included. If a more limited range of application of Q had been suggested, that would have been the result, since Q is an a posteriori empirical method. Significantly, the Q-system database and applicability were expanded in

1993 with Grimstad's incorporation of steel fibre-reinforced shotcrete S(fr) and the development of corrosion-protected sleeved (CT) bolts. Both have added to the reliability of B+S(fr) single-shell permanent support. The Q-system has been used successfully in rocks with UCS as low as 4 to 7 MPa (significantly jointed chalk marl in shallower parts of the Channel Tunnel: Barton and Warren (2019) and UCS up to at least 300 MPa for some granites, gneisses and quartzites. Development of the Q-system has also meant engagement in numerous tunnel and cavern projects in Norway and abroad since 1975, including experiences in water transfer tunnels, hydropower headrace and pressure tunnels in many countries. The economic advantages of single-shell tunnels for hydropower have made this form of water 'conveyance' very attractive in relation to more expensive concrete-lined alternatives.

There are tens of thousands of kilometers of single-shell or nominally 'unlined' tunnels, and all need sound design. This includes respect for moderate 1.5-2.5 m/s flow velocities. Q-based single-shell tunnels were never intended for 10 m/s river diversion flow velocities unless with full-profile S(fr), and especially protection of the invert.

The advantages of single-shell caverns for storage, sports halls, hydropower, and metro stations are clear, and it is a source of great surprise when one sometimes observes contractors having to place 3D drainage fleece, 3D membrane and 3D shuttering for the final concrete lining inside the complex shapes of metro stations. It is also a source of surprise when the need for pre-injection is not respected in metro and other tunnels under clay-rich city-foundation sediments, and serious building and road settlement damage results. There are cases of 0.5 km, 1 km and even 3 km distant damage to homes due to failure to pre-inject leaking tunnels (Barton and Quadros, 2019).

1.1 Tunnels in Jointed Rock Strangely Modelled as Continua

The author, probably like many others in the last two decades, has been 'impressed' (unfortunately negatively impressed) by all the colourful plots of 'plastic behaviour' in continuum modelling approximations, which has been a regrettable growing trend in rock 'engineering'. The problems start with the rock mass characterization being so crude with GSI and its limited 'one-way' joint condition scale on one axis. (Rough joints cannot be clay-filled or weathered, smooth, planar and slickensided joints need to be clay-filled and weathered. Clearly illogical, therefore why used?) The five or six sketches of rock masses on the other axis (optimistically added to and made 'anisotropic' by some Hoek co-authors) complete the choice of GSI. With much of engineering geology somewhat ignored, software-assisted application of page-wide Hoek-Brown equations for 'c' and 'φ' follows, with both these strength components incorrectly assumed to be mobilized simultaneously. If they were, we could not monitor progressive failure in tunnels or in open pits. In addition, there is the remarkable and extraordinary 16x and 12x repetition of GSI in the page-wide Hoek-Brown equations for 'c' and 'ϕ', even if we only use the three supporting equations for mb, s and a. (Barton, 2023; 2025). Thousands of GSI users seem to be unaware of this. The H-B estimates were derived, illogically, from the more reliable intact rock strength criterion of Hoek and Brown, instead of from more logical joint and fault strength criteria.

1.2 Tunnels in Jointed Rock Modelled as Discontinua

The reality of rock mass response to tunnel and cavern excavation is somewhat closer to what is illustrated in Figure 1. Considering joint-and-block response is, of course, more time-consuming and also approximate, but mechanisms of relevance can be understood, including the need for shotcrete support, bolt reinforcement (and how it is actually loaded), and perhaps the need for pre-injection. We need to consider and be able to model the excavation disturbed (and damaged) zone mechanisms, not the crude and incorrect 'plastic' zones which were proved in a formal international court case to be grossly exaggerated in relation to the recorded behaviour of reality. The small hydro tunnel, obviously without 'plastic' zones, did not need shotcrete support in 7 km. Instead, it was partly invert-damaged by too-rapid emptying. Closer to reality, we have two excavation-damaged zones (EDZ) and one damage zone. All can be compromised.

> *EDZ_1* - stress-redistribution in the solid rock surrounding and forming the tunnel.
>
> *EDZ_2* - deformation of the rock joints as a result of the stress-redistribution in the surrounding rock.
>
> *EDZ_3* - dynamic shock-loading causing blast-induced cracking, loosening, and enhancedlocal permeability.
>
> *EDZ_4* - lack of pre-grouting causing deeper-seated damage as a result of the blasting, plus unnecessary and deeper-located joint shearing, sometimes including more over-break.

Neglecting pre-grouting when needed, or performing it badly, potentially allows higher inflow than desired or allowed. In addition, there may be stability issues. The author has observed continuous wet shotcrete, leaking bolt holes, and grout penetration in only one of several joint sets (seen in hundreds of tunnel-face photographs) due to poor understanding of the need for high-pressure injection (Barton and Roald, 2023).

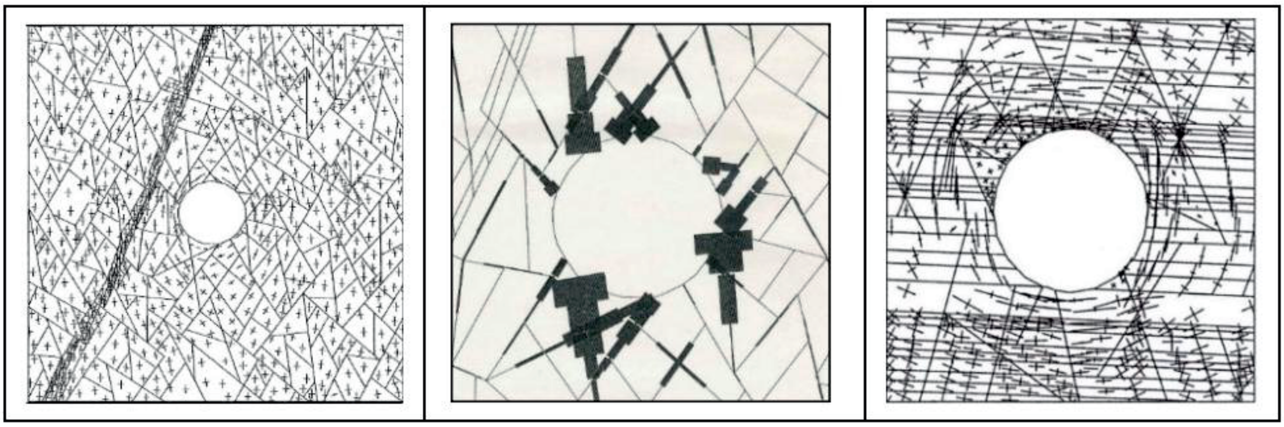


Figure 1- EDZ 1 and EDZ 2 in a welded tuff-ignimbrite and in interbedded sandstones (NGI, 1995)

The central diagram in the Figure 1, shows joint shearing magnitudes (of mm scale). EDZ 2 joint responses also include joint opening and closing. The first two EDZ mechanisms illustrated in the Figure are seen in distinct element models performed for a repository URL TBM access tunnel. These date from the 1990's work at NGI when we were consulting for UK Nirex. Drawdown and water flow in the joints can also be modelled (Fig. 2). Figure 3 shows the 62 m span Olympic ice-hockey cavern, which was supported with 10 cm of steel-fibre reinforced shotcrete and permanent rock bolts of 6 m length at 2.5 m c/c. The twin-strand anchoring was in case of unstable wedges. The NGI team were given the task of checking the design for the owner. The site investigation, modelling, Q-logging and performance are described by Barton et al. (1994). MPBX showed maxima of 7 to 8 mm deformation, as predicted by Chryssanthakis's UDEC-BB modelling. See the input table, bottom-left in Figure 3.

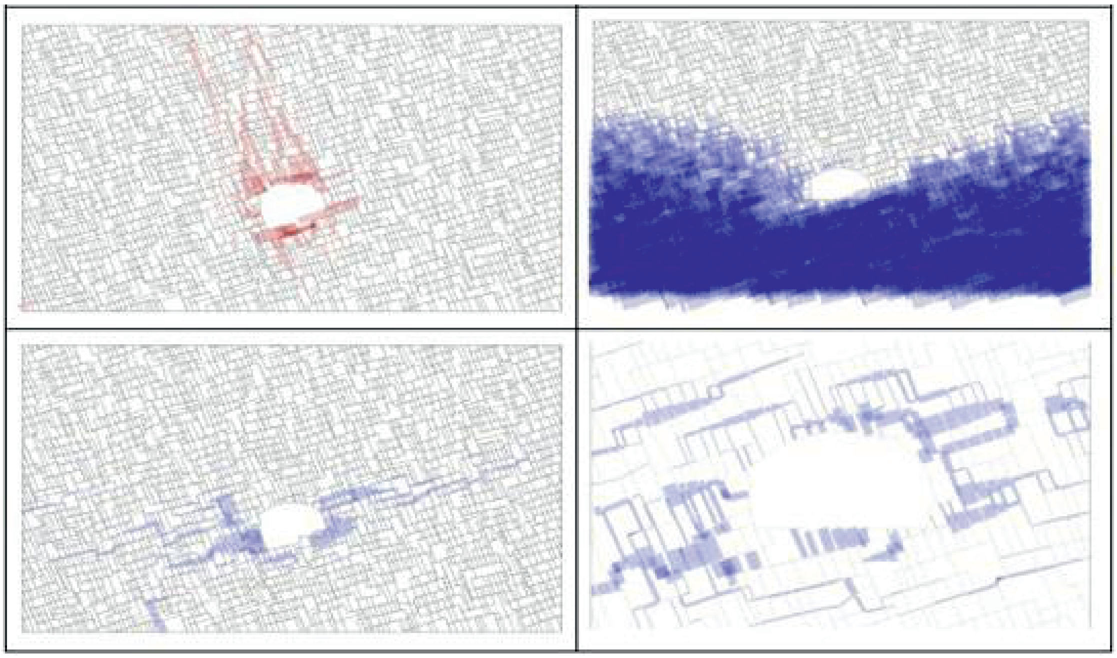


Figure 2 - Shear deformation (orange) on the steep joint set with lower JRC, and inflow in the more permeable higher JRC joints (Two stages of drawdown with lacking pre-injection)

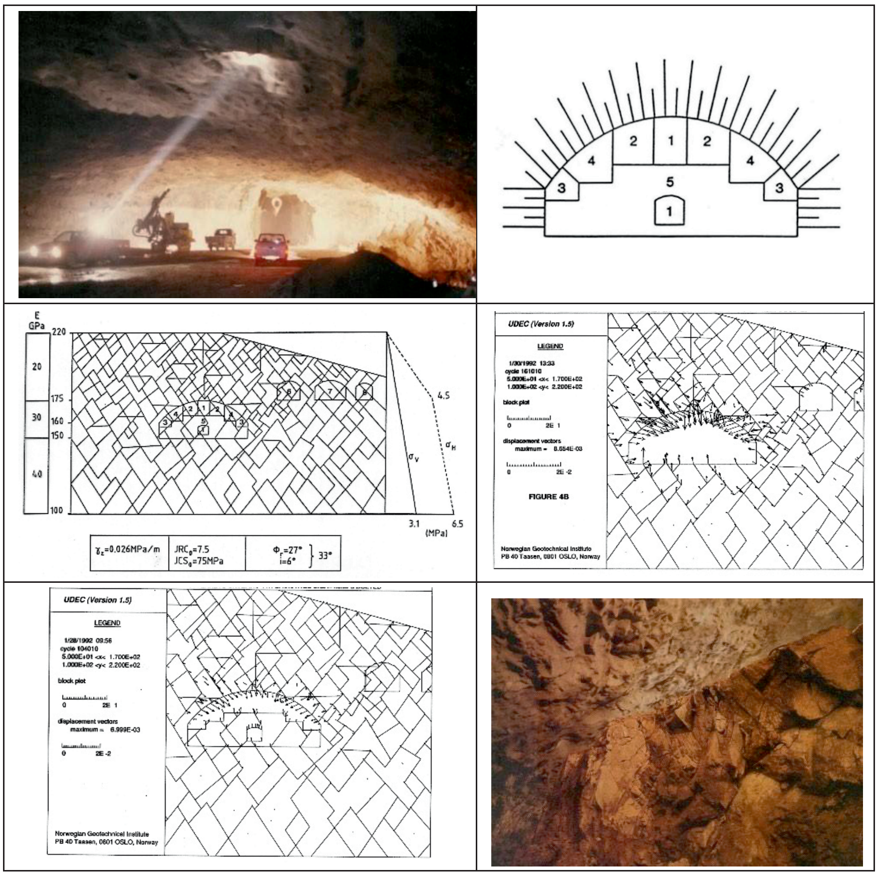


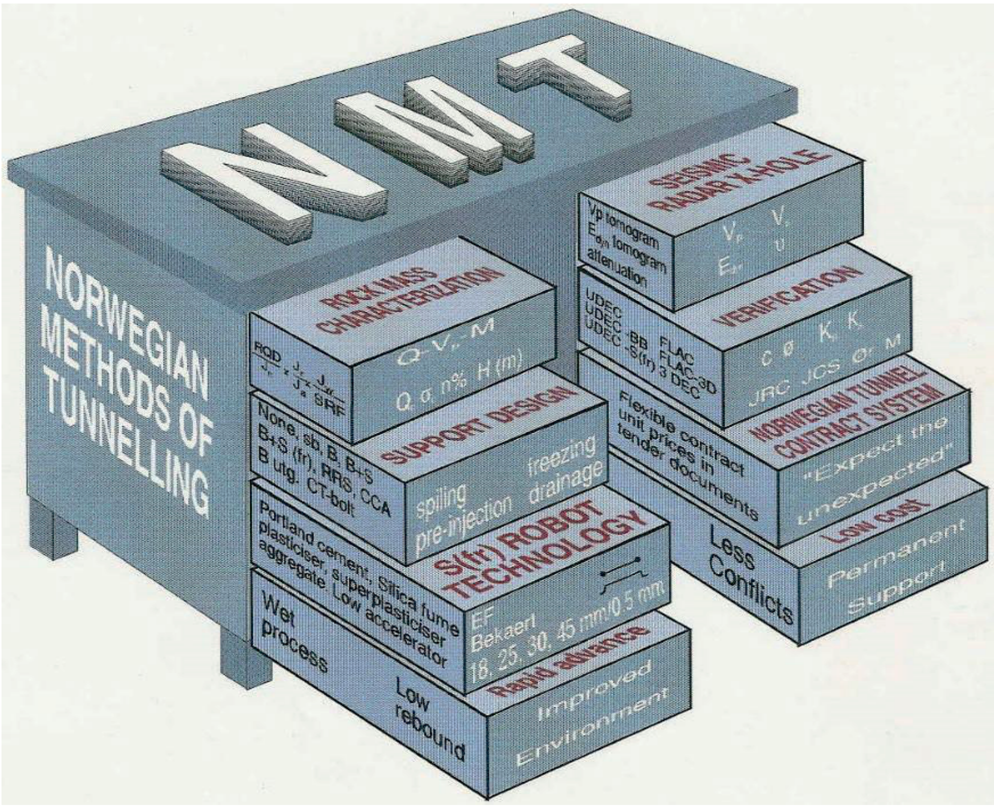
Figure 3- UDEC-BB modelling for design checks of the Gjøvik cavern (Barton et al., 1994)

In Figure 2, shear deformation on the steep joint set with lower JRC and inflow in the more permeable higher JRC joints indicated by coupled UDEC-BB modelling has been shown. Photos given in Figure 2 indicates design checks for a cavern. It is a perfect example of single-shell B+S(fr) cavern design. The huge arch is ‘supported’ with 10 cm of S(fr) and reinforced with permanent (sleeved) CT bolts. In other words, the 62 m wide rock arch is assisted in supporting itself.

Remembering the a posteriori origin of the Q-system, it is wise for today’s numerical modelers to think twice before proposing ‘longer rock bolts’ for Q-based design. Claims about deep ‘plastic’ zones when analysis methods are full of a priori assumptions and alarming page-wide opaque equations devoid of joint sets or clay fillings inevitably fail to convince. Regrettably, this is due to GSI, H-B, Phase 2/RS2 errors.

2. NMT PRINCIPLES AND PRACTICE BASED ON CASE RECORDS AND EXAMPLES

Since single-shell tunnelling is not practiced in some countries, except perhaps in their much larger hydropower caverns, some basic elements will be illustrated in this section. In the early 1990’s when a multi-author multiple-company group published some formal descriptions of the NMT for World Tunnelling, we were basically describing a viable and much-used alternative to the well-known double-shell method known as NATM (New Austrian Tunnelling Method). Principal aspects of NMT are listed in Figure 4, from Barton and Itoh (1994). As shown later, NMT has some considerable advantages in terms of speed and cost and actually presents a much smaller ‘environmental footprint compared to NATM due to the widely different effect of over-break on the concrete volume used in the two methods. This also affects the head-loss design philosophy for water transfer and headrace tunnels (Barton and Quadros, 2020).



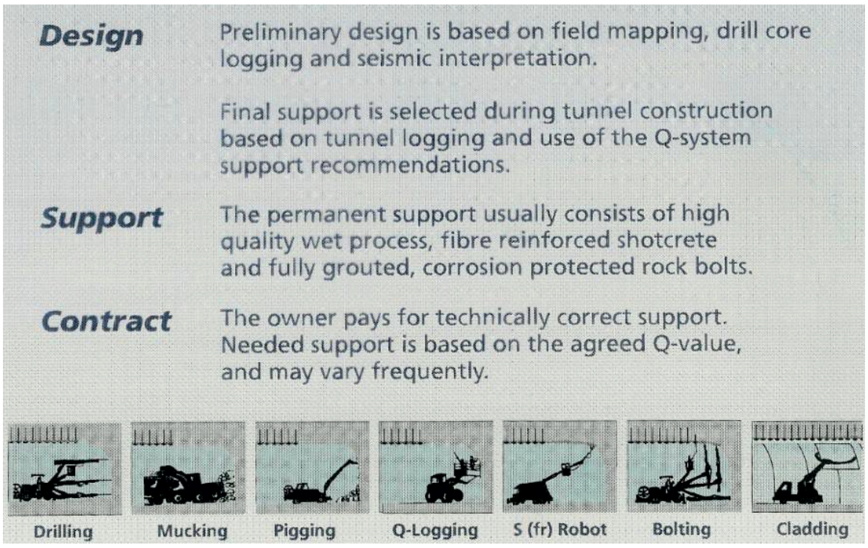


Figure 4 - Design and support according to NMT (Barton et al. 1992)

The first step in applying NMT is to collect Q-parameter data. While evaluating Q-parameter ratings, preferably with statistical variation, one is performing some measure of engineering geological activity, as compared to the corner-cutting crudity of GSI estimation, where any 'error', however defined, would have serious consequences for the values of input data in popular FEM continuum programs.

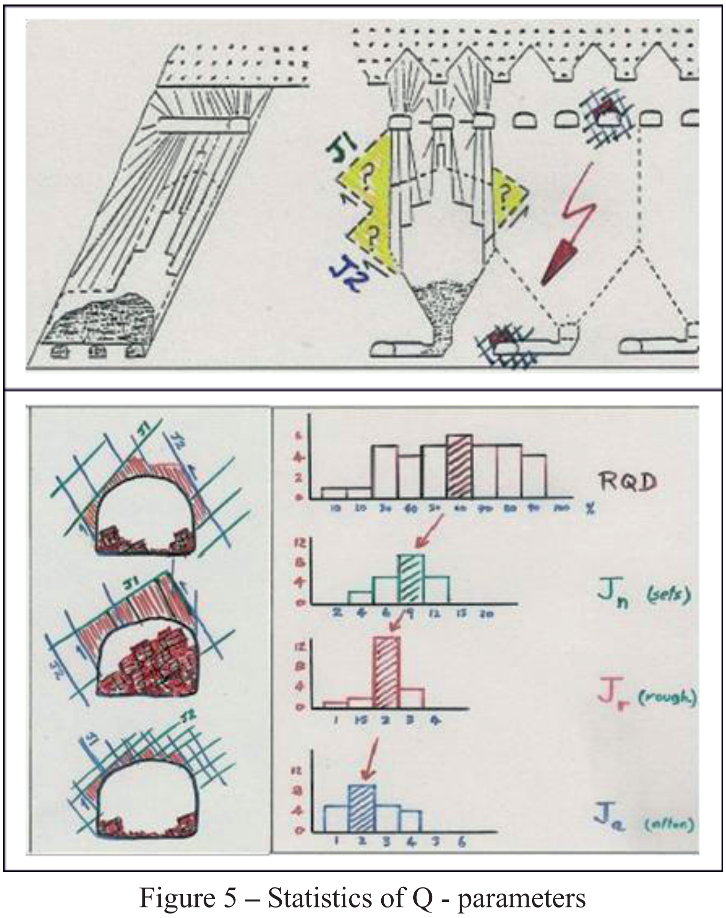


Figure 5 – Statistics of Q - parameters

The logging of Q-parameter statistics is found to be a powerful way to represent the inevitable variability of rock masses. The severe over-break in the long-hole drilling and draw-point drifts was due to some adverse ratios of J_n/J_r (> 6) and adverse J_a.

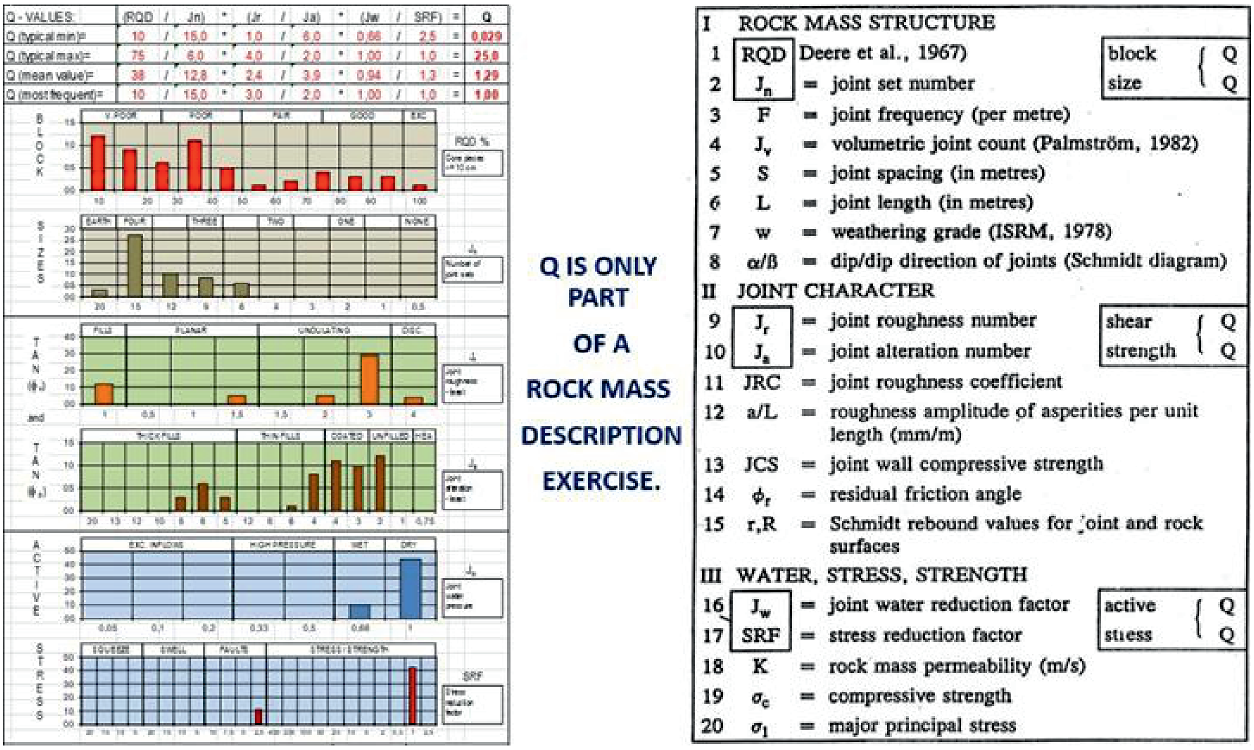


Figure 6 - Statistics of the six Q-parameters for domains in drill core, or specific lengths of tunnel advance

Q-parameters are just part of a formal engineering geological study (Fig. 6). The parameter ratios $J_n/J_r \geq 6$ specify likely overbreak, and $\tan^{-1}(J_r/J_a)$ represents friction angles, which are higher if there are joint sets with roughness, and lower with elevated Ja and clay fillings.

3. NMT SUPPORT AND REINFORCEMENT DETAILS

In this section, three generations of Q-support recommendations are reproduced for reference, starting with the 212 case records of Barton et al. (1974), then Grimstad and Barton (1993) and finally Grimstad's updated diagram with details of RRS (not the limited span NGI/NRA road tunnel version of 2015).

The Q_{TBM} prognosis model also took into account the more massive rock at 40 to 100 m depth using the higher velocity of many kilometers of seismic refraction (Fig.7). Note that E_{mass} and V_P both increase with depth or stress. The mean PR was 2.0 m/h, and the mean AR was only 0.5 m/h for 36 km of tunnelling with four Herrenknecht TBM, due to various hard rock and water inflow challenges (Macias and Barton, 2023).

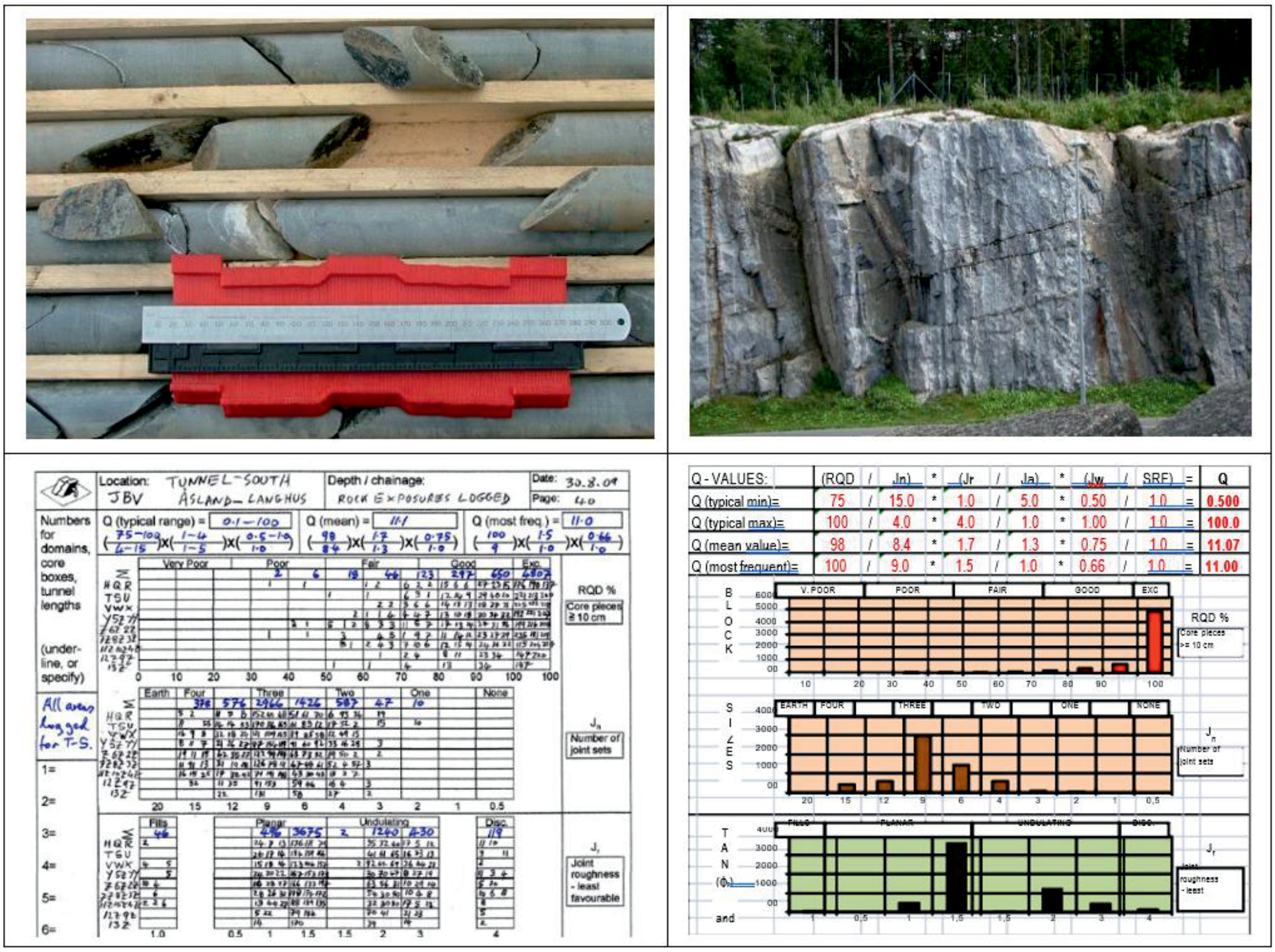


Figure 7- Q-parameter statistics for a 4x TBM rail project near Oslo

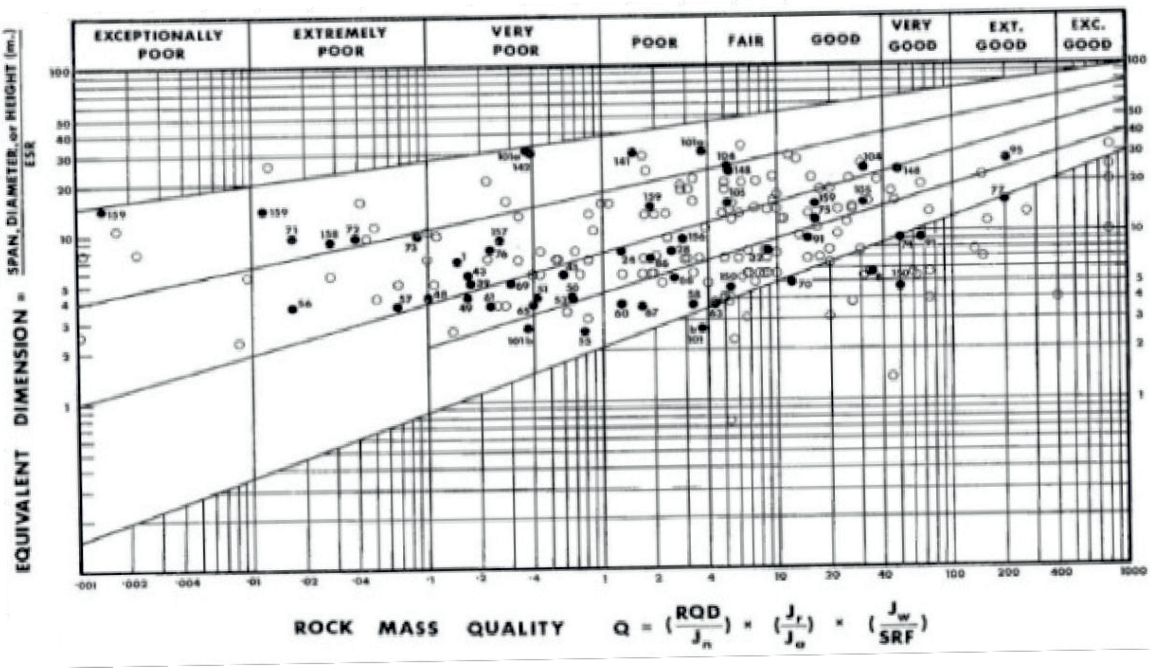


Figure 8 - The wide range of 212 tunnel and cavern case records inclusive of clay-bearing and faulted rock cases (Barton et al., 1974)

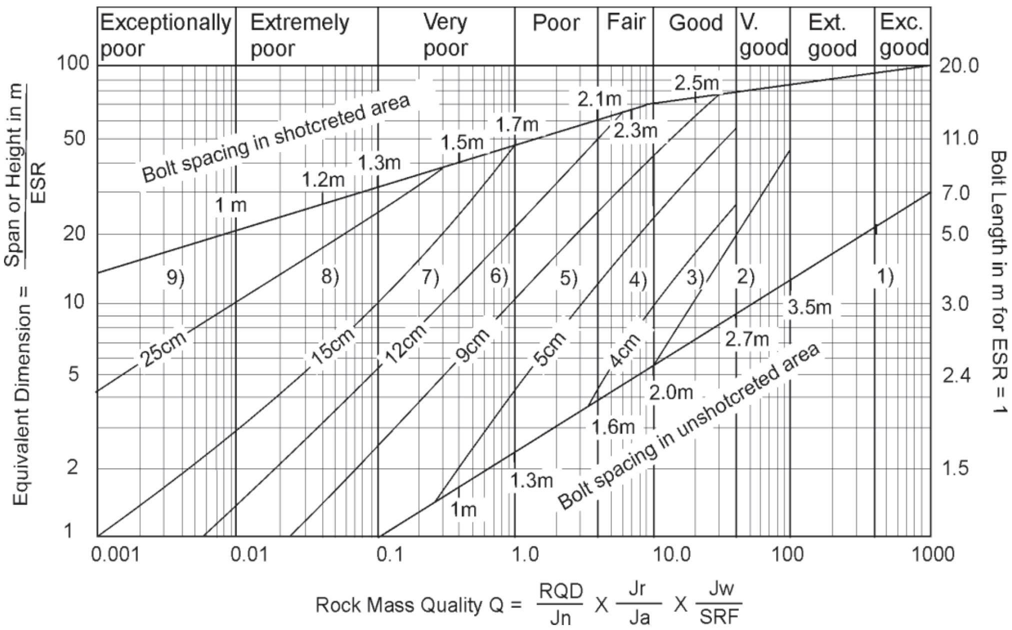


Figure 9 - The Q-system tunnel/cavern support and reinforcement scheme from Grimstad and Barton (1993)

The updated chart given in Figure 9 was the important update from S or S(mr) + B, to *S(fr)+B*. The latter could by now be PVC-sleeved CT bolts for improved longevity.

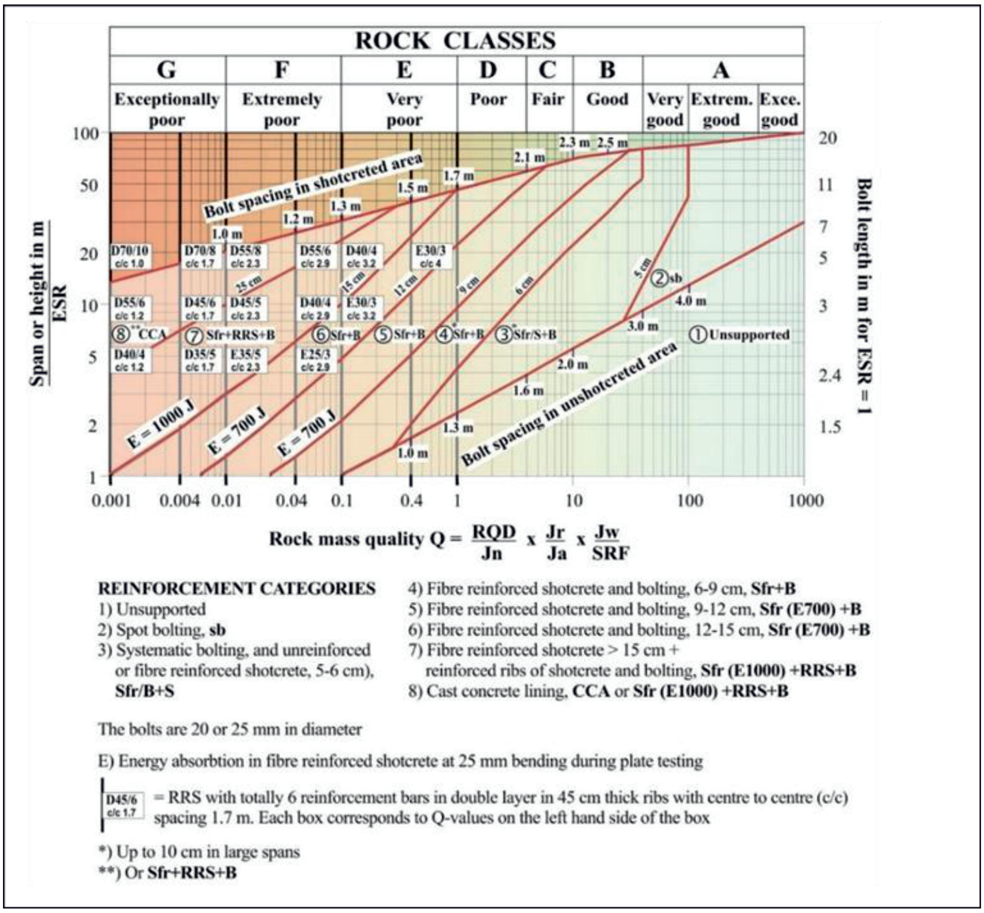


Figure 10- The support chart update of 2003 with the dimensioning of RRS (Grimstad et al., 2003)

Note the increased minimum thickness of S(fr) of 5 cm (Fig. 10). The Norwegian Road Authority decided some years ago to increase the conservatism (effectively using SRF = 0.5 in place of 1.0) and specify a minimum thickness of 8 cm.

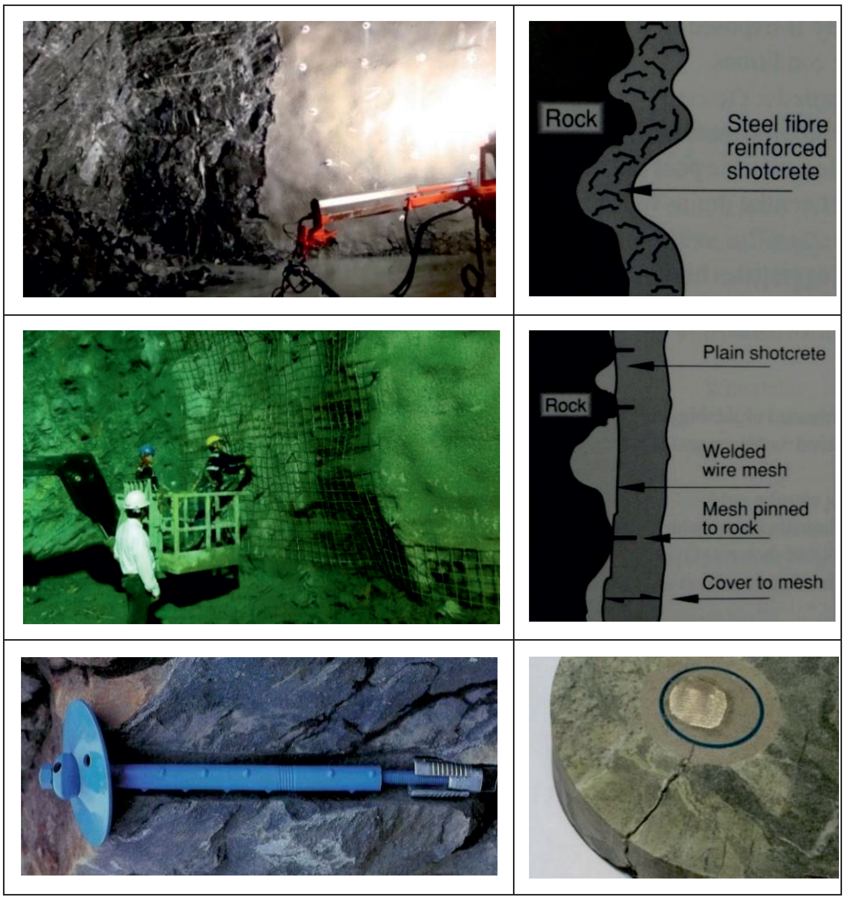


Figure 11- Illustrations of the S(fr) and S(mr) support options (Vandervall, 1990)

The CT bolt 'demo' demonstrates corrosion protection, even when an intersected joint cracks the outer annulus of grout, an inevitable occurrence when tunnelling. There remain four protective layers meaning that the extra cost of CT bolts guarantees an overall cost saving for the NMT-style single shell method as the bolts are permanent.

When tunnelling has to be done though faulted clay-bearing (low Q-value) ground, NMT solution is RRS (bolted rib-reinforced shotcrete arches) rather than the common though much more deformable lattice girders. Principles and examples of practice are illustrated in Figure 12. Specific Q-value related design choices were illustrated in Figure 10. Note that RRS are systematically (radially) bolted so are far more robust and therefore safer than lattice girders (LG), which have been known to buckle when strongly loaded. See later discussion and some examples of collapsed tunnels and caverns.

Note that the 28 m span railway station (bottom left photo in Fig. 12) was excavated to the left-side and then completed on the right-side, due to a deep sediment-filled valley in downtown Oslo. The National Theatre station has a 28 m span and indeed has a final concrete lining due to the very low Q-values. The unusual 'bent' shape of the RRS was requested architectural detail, prior to concrete lining.

The B+S(fr) reinforcement and support solutions for NMT excavations in jointed and clay-bearing rock masses is a very flexible solution as bolt spacing and shotcrete thickness are varied with Q. There is also the possible additional influence of conditional factors J_r/J_a (internal friction affecting bolt spacing and capacity) and RQD/J_n (affecting shotcrete thickness) as recommended by Barton et al. (1974).

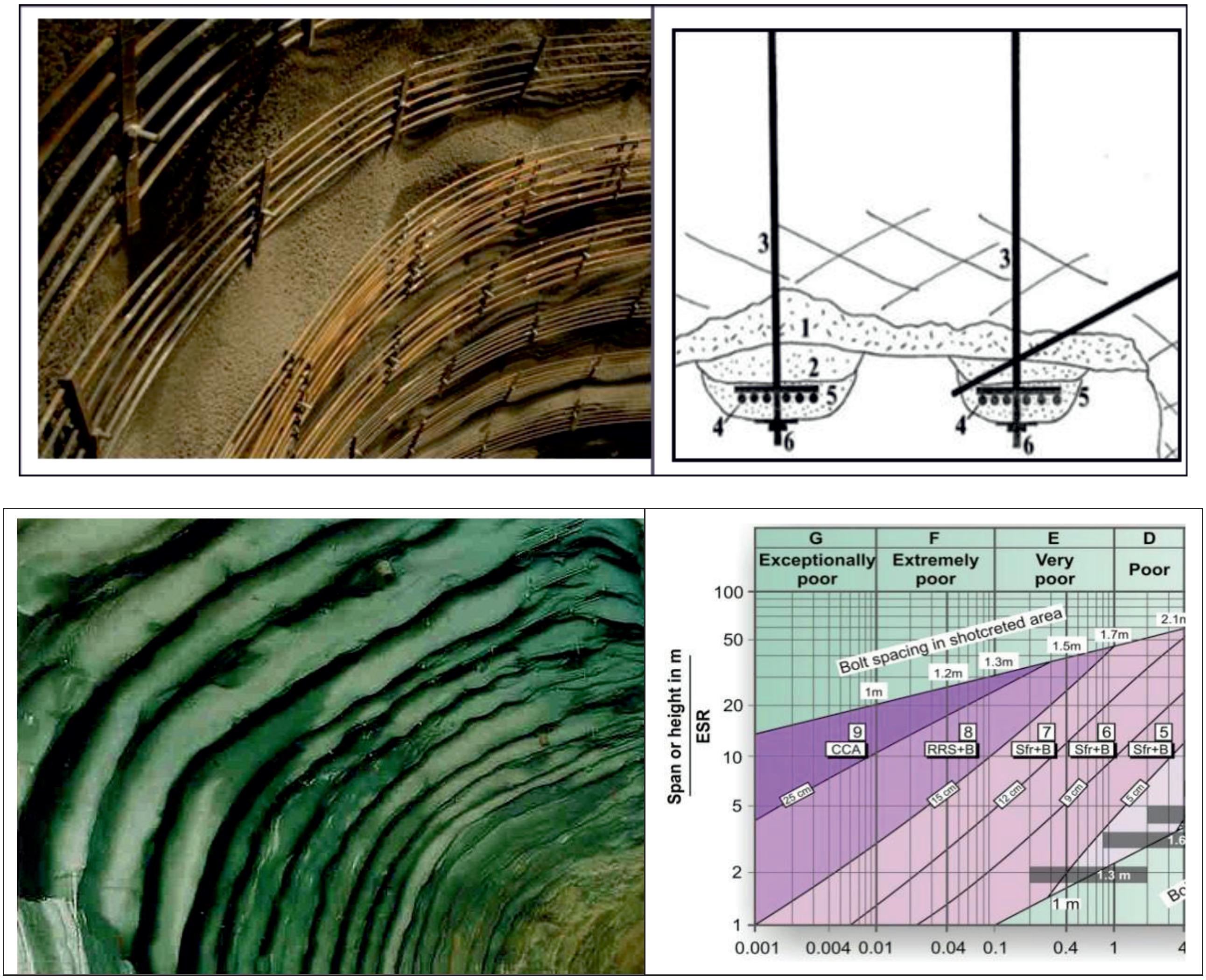


Figure 12 - The principles of RRS

In Figure 12, the top left picture shows single-layer and double-layer reinforcement; top right picture shows the order of installation 1 through 6; bottom left picture shows RRS arch in two parts for twin-face station cavern; bottom-right picture shows RRS and CCA locations (from 1993) with penetration of RRS into CCA areas shown in the more recent Figure 10 details (from 2003).

4. COMPARING NATM WITH NMT AND SUGGESTING HYBRID ECONOMIES

The two PowerPoint screens reproduced in Figure 13 emphasize the element of choice that is involved (left side), and the key differences (right side). On occasion, the author has strongly recommended a hybrid support with 'NATM-like' principles through the soil, saprolite and weathered rock at each end of a rock tunnel, and NMT economies in the central part, which may be over kilometers. As shown later in one special case, incorrect design caused 140 m of the NATM tubes to collapse at 3 months intervals as the lattice girders were over-loaded by vertical structure ignored in the design.

NMT or NATM? 1. SINGLE-SHELL METHODS OF SUPPORT (Sfr) + BOLT REINFORCEMENT (B) ARE USED IN 'ALL' THE WORLD'S HYDROPOWER GENERATION CAVERNS, OIL STORAGE CAVERNS, MINE ACCESS ETC. 2. BUT IN OTHER EXCAVATIONS (LIKE ROAD, RAIL, METRO TUNNELS) THERE IS A DECISION TO BE MADE: 'NATM' (expensive) or 'NMT' (cheap)?	1. 'SINGLE-SHELL' (NMT) (PRE-GROUTING?) + B + Sfr + (RRS?)............. NEEDS SMALL WORK FORCE ......(1/10 x NATM?) 2. 'DOUBLE-SHELL' ('NATM') (TEMPORARY: Sfr/Smr, B, STEEL/LATTICE GIRDERS, PERMANENT: FLEECE, MEMBRANE, CAST CONCRETE NEEDS LARGE WORK FORCE ......(10 x NMT?)

Figure 13 - The question of choice and key differences between NMT and NATM

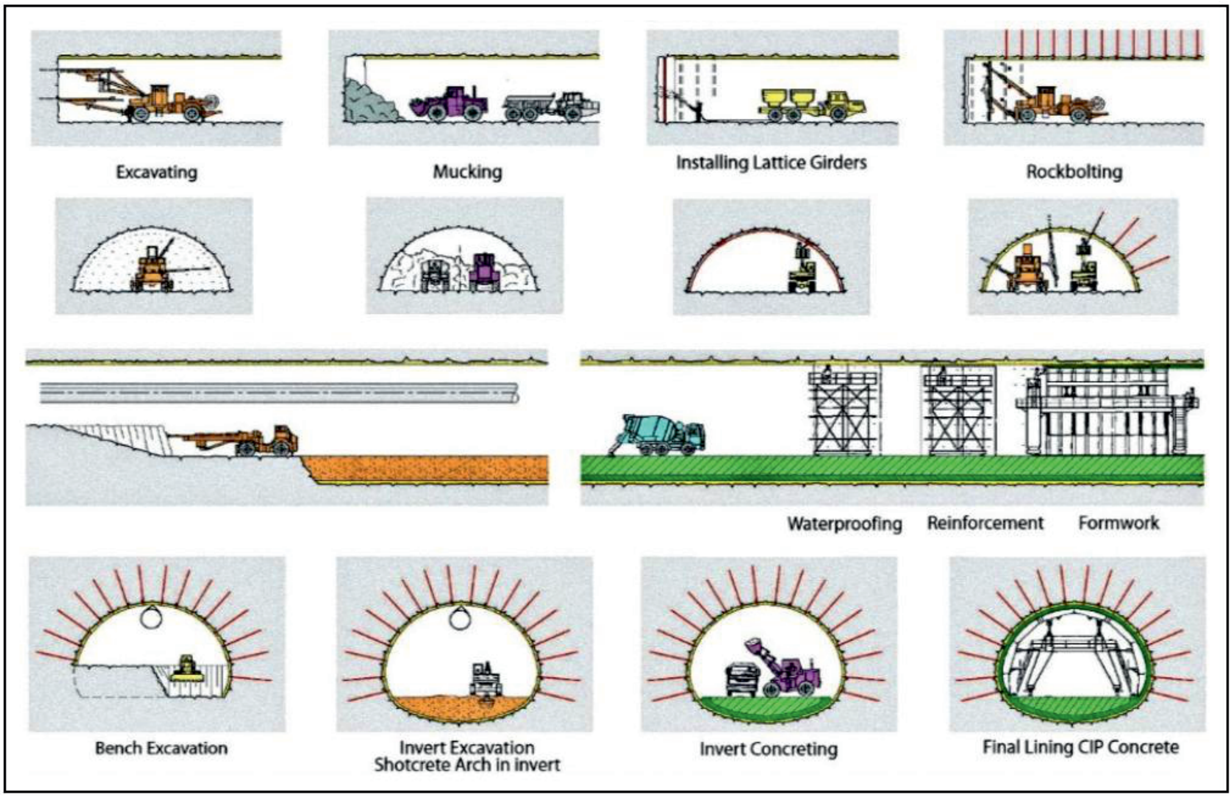


Figure 14 - Principal operations involved in NATM when a single top-heading can be formed ASG (2010)

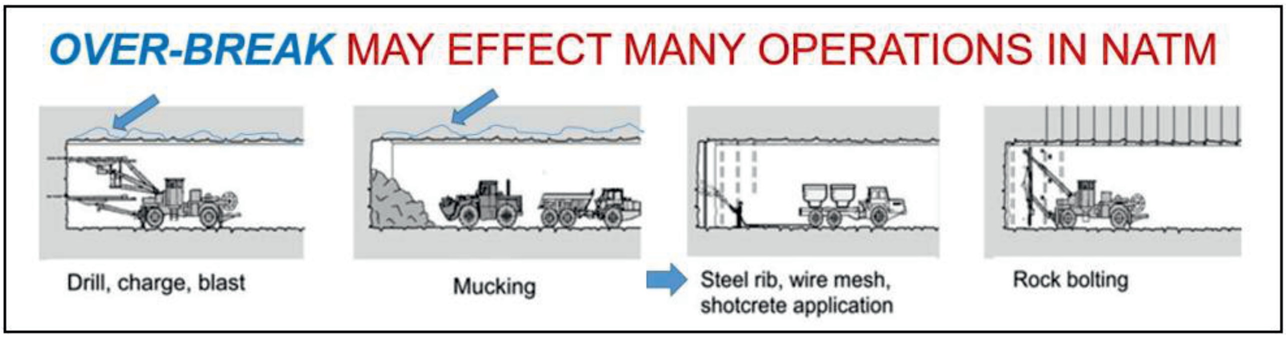


Figure 15 – Overbreak problems in NATM

In NATM practice, overbreak is ignored in design which may make operations far more time-consuming, and the effect of lattice girders and S(mr) are compromised (Fig. 15)**.**

The problem is that overbreak may affect most of the operations involved in forming NATM tunnels. Lattice girders and wire mesh are more time-consuming due to the need to fill overbreak, which may easily be double or locally three times more than a ‘nominal’ 30 cm thickness.
‘NATM: The Austrian practice of conventional tunnelling’. As suggested by Figure 15 overbreak that is ignored in design may make operations far more time-consuming, and the effect of lattice girders and S(mr) are compromised**.**

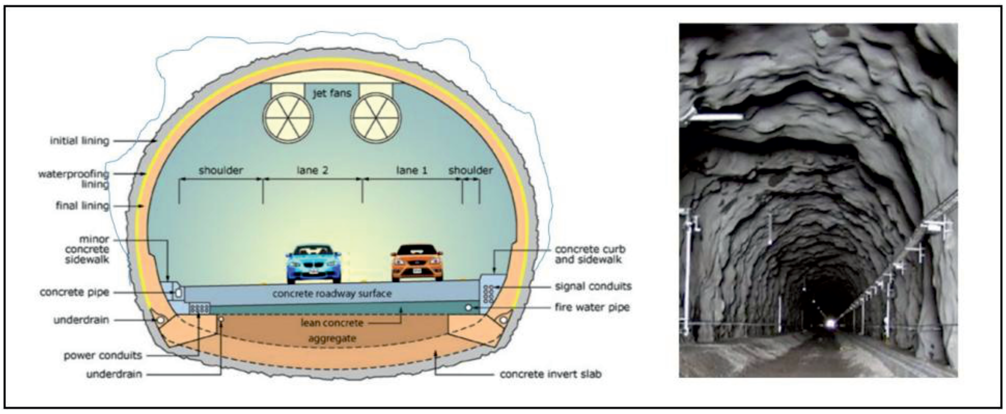


Figure 16 - The usual extent of overbreak is obviously less than 1 to 2 m, though not always. Its effect on NATM shotcrete-and-concrete-consumption as compared to NMT tunnels may be dramatic. The economic Botnia (Swedish) rail tunnel (which used Q-based B+Sfr) is ready for track-laying. S(fr) keeps the natural rock arch intact with internal reinforcement by the bolts. A load bearing ring is not the idea of NMT unless bad rock.

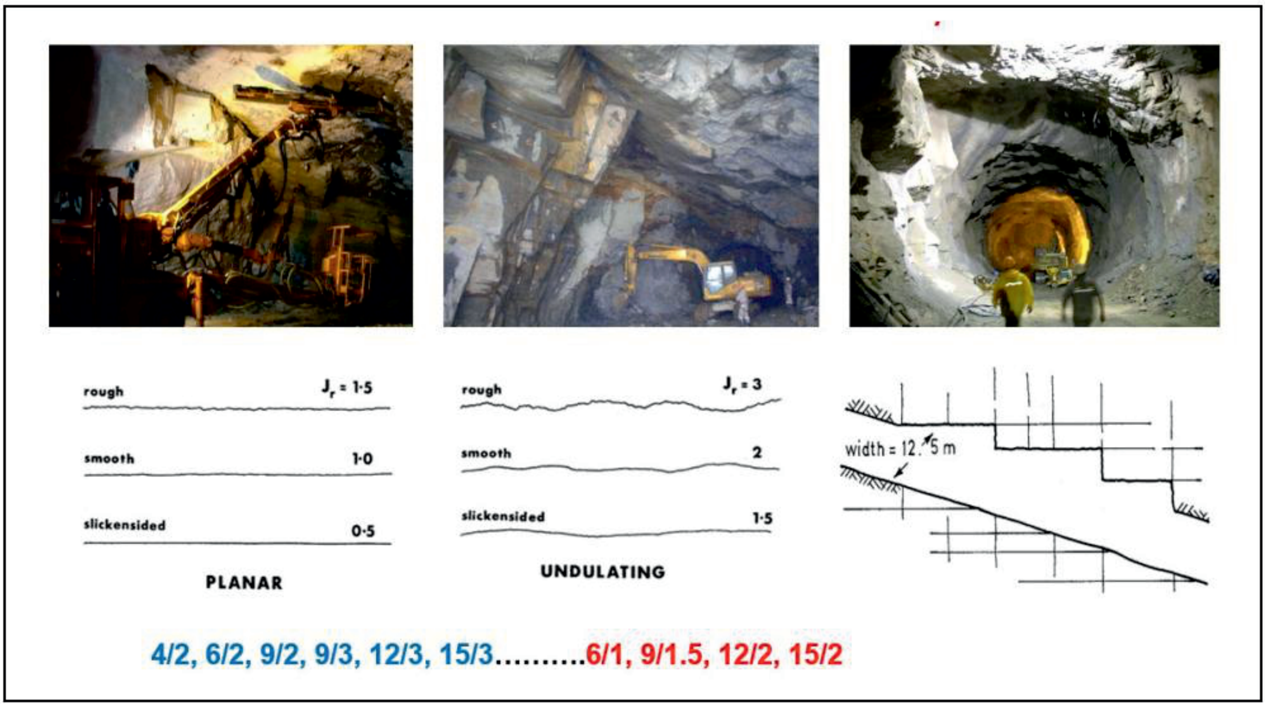


Figure 17- The overbreak claimed Q-based criterion (Jr/Ja ≥ 6)

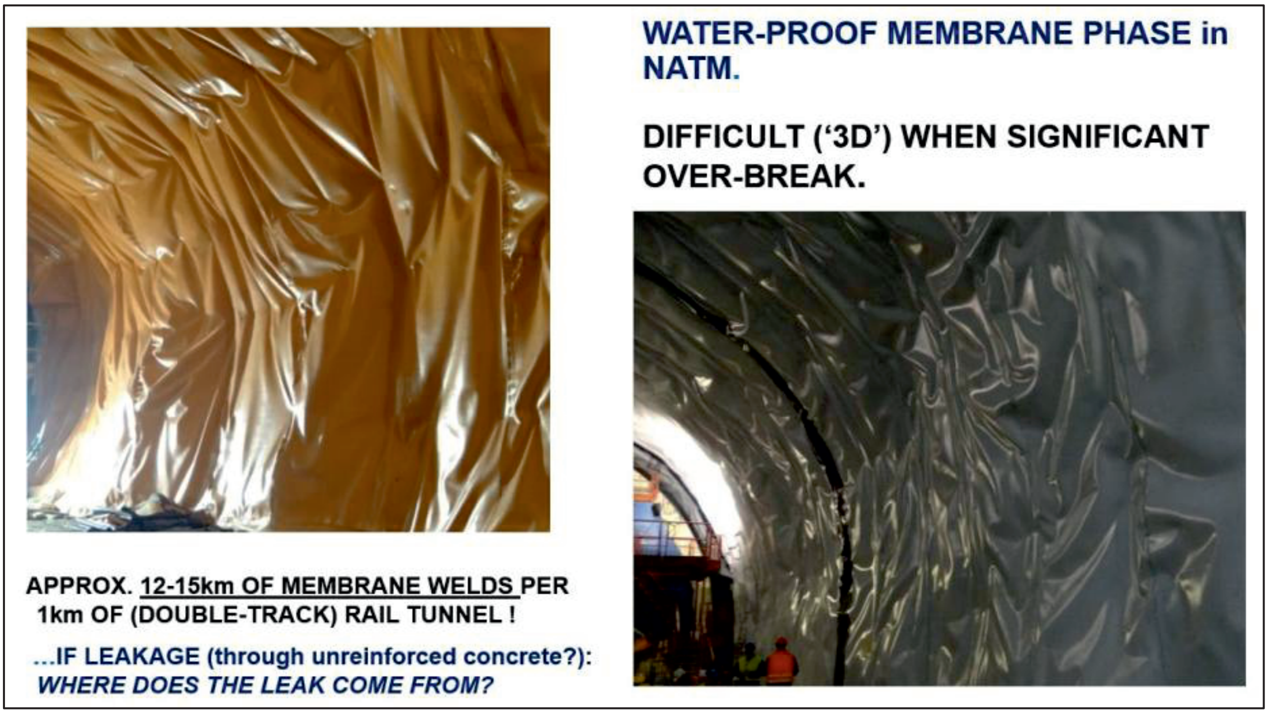


Figure 18- The question of weld integrity is the greatest risk beyond lattice girder folding

Figure 19- The remarkable labour-intensive NATM if used in a cavern instead of S(fr)

5. THE LATTICE GIRDER STAGE OF NATM IS SOMETIMES FULL OF RISK

Steel sets in smaller tunnels and lattice girders (LG) in larger ones, perhaps in the range 20 m and even up to 30 m span, is obviously convenient and is a form of psychological support for the tunnellers/miners. Its application in a 19 m span station cavern is illustrated in Figure 20. The necessity yet difficulty of making good contact with the tunnel or cavern arch needs emphasis, to minimize the risk of 'point loading'. Regrettably, since not bolted radially as with RRS (Fig. 12), the lattice girders (and steel sets) need good footing stiffness ('elephant feet' are a way of spreading load).

Finally, the steel has to deform longitudinally before it can actually build any significant resistance to the radial deformation that accompanies tunnelling. To the author, this suggests 10's of millimeters of deformation before radial tunnel support is actually achieved. In this period, there may be some loosening of the rock mass, and an actual need for the traditional NATM monitoring. The drawings in Figure 21 show evidence of the superiority of B + S (or B+Sfr) concepts, which are permanent designs in NMT (so taken more seriously), while temporary in NATM. The multiple design compromises (actually unintended errors) in NATM should be clear, and indeed, there have been some dramatic collapses due to the compound effects caused by lack of LG stiffness.

Figure 20 - Steel ribs (left) in an Ecuador HEP, and 32 mm × 25 mm × 25 mm lattice girders in a Brazilian metro station cavern (Barton, 2008)

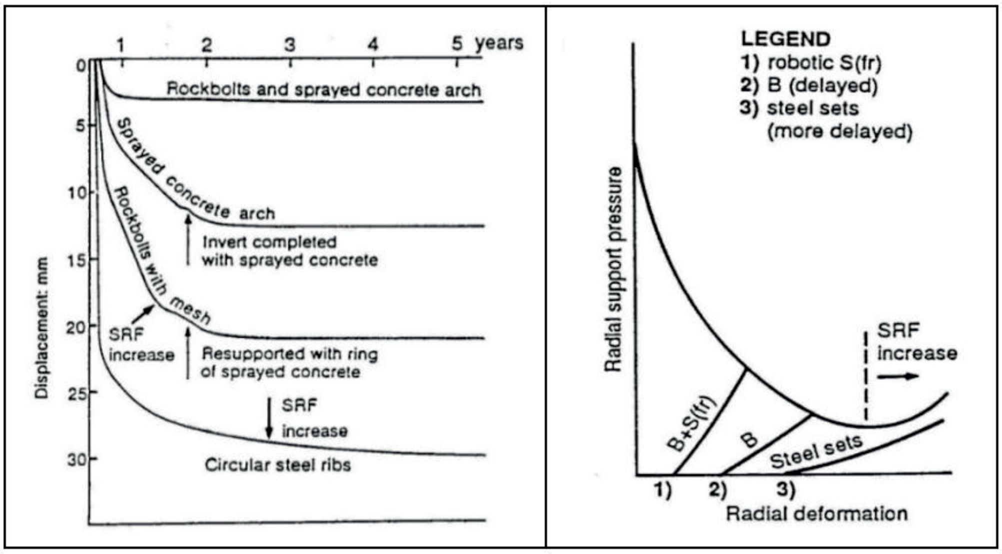


Figure 21- Left: Five years of monitoring of an experimental tunnel in mudstones (Ward et al., 1983)
Right: SRF loosening advisory from Barton and Grimstad (1994) in relation to steel sets

In Figure 22, the black curved arch at 20 m depth down the shaft is the top-heading of the northern end of the station platform cavern. The station cavern collapsed suddenly during construction despite multiple investigation boreholes. Differential weathering of the gneiss had allowed a 10 m high undetected ridge of rock to sit along the arch. Its vertical loading of the lattice girders and of the 40cm of S(fr) caused the tragedy. A final concrete lining had been planned, as is normal for NATM. Grouting through perforated tubes had apparently been insufficient to improve the shear strength of the clay-bearing rock mass (Barton, 2008).

Figure 22 - Aerial view of the Pinheiros Station shaft

Figure 23 – Severely overloaded vertically, lattice girders demonstrate insufficient strength

In view of the failure of lattice girders (Figs. 20 & 23), there is a need of religious monitoring in the case of NATM. In another NATM tunnel collapse example (Figs. 24 & 25), bolting could not be used due to the saprolite and deeply weathered rock near to the portals. A non-fatal collapse of 140 m of the southern tube (during a God-given Saturday afternoon barbeque for all tunnel workers) was followed three months later by an equal 140 m collapse of the second tube that is seen in the photograph. This showed signs of overloading of the haunches deeper into the hillside. There were really two chief culprits: a much too idealized symmetric numerical model design effort shown in Figure 24 (top left) and consequently too light LG. The omitted structural geological detail was vertically dipping phyllite: vertical loading was quite different from that modelled.

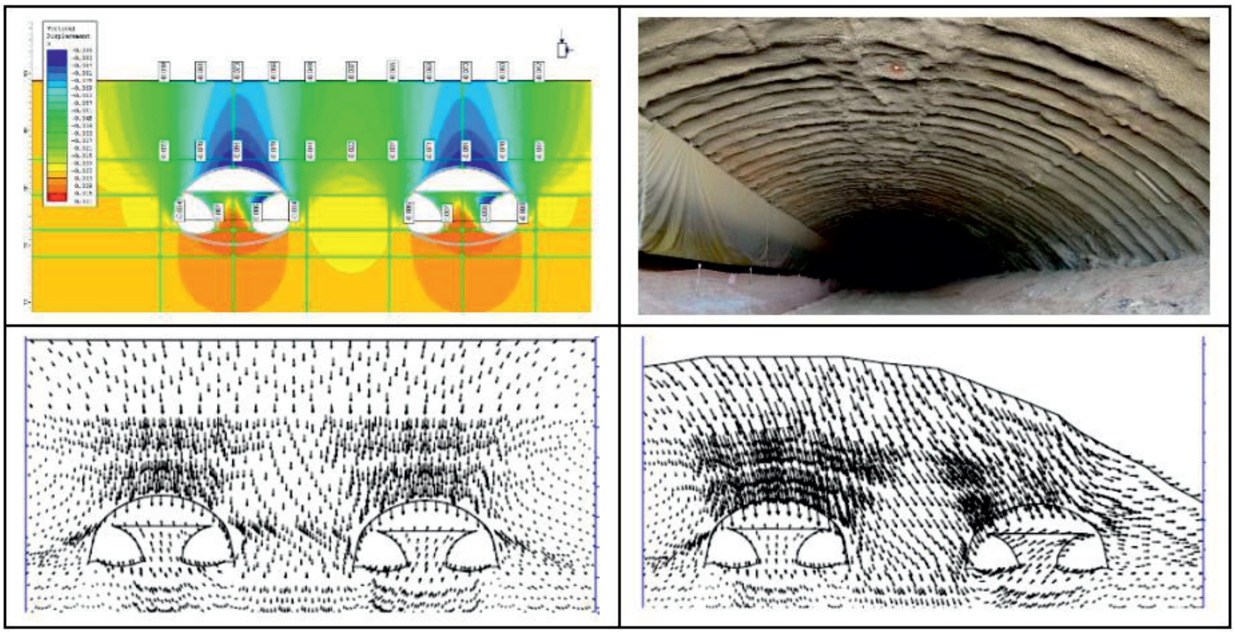


Figure 24 - Optimistic symmetric design in sloping ground of motorway tunnel

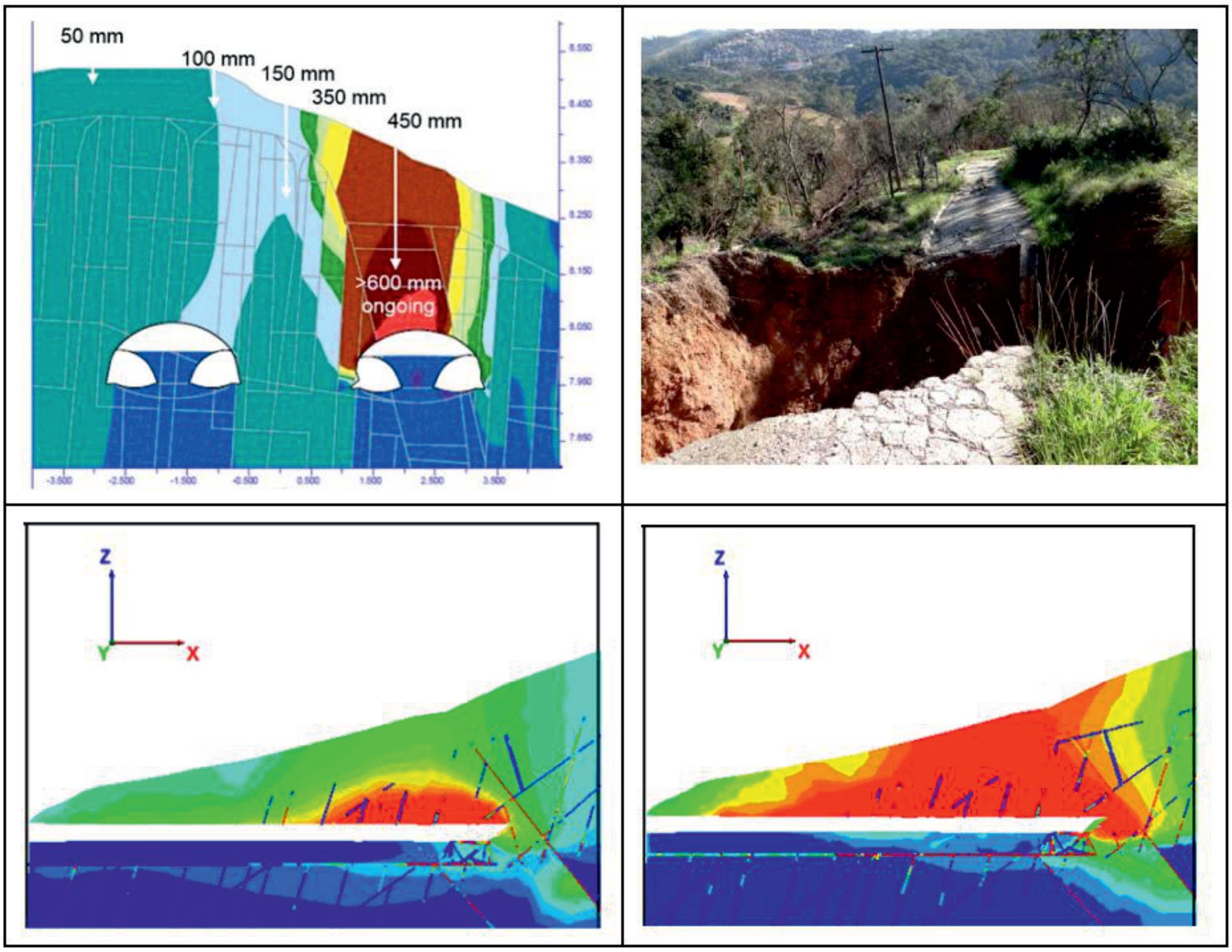


Figure 25 - UDEC and 3DEC models

The mentioned tunnel in Figure 24, suffered from too light lattice girders, too much overbreak and therefore 2 to 3 times the expected volume of shotcrete. Bolting could not be used due to the saprolite and weathering. A non-fatal collapse of 140 m (Saturday barbeque for workers) was followed three months later by an equal collapse of this second tube. NMT was used for the central kilometers in jointed granite, thus a hybrid approach. The UDEC models were performed by Stavros Bandis, who assisted NB&A at this time.

A progressive and catastrophic failure is seen progressing back from a weathered dike in surrounding low-shear-strength phyllite with adverse vertical foliation and jointing. A near-accident in a major international project, again due to lattice girder response to unexpected vertical loading (Fig. 26).

Figure 26- Cross-over cavern on the English side of the Channel Tunnel construction using NATM

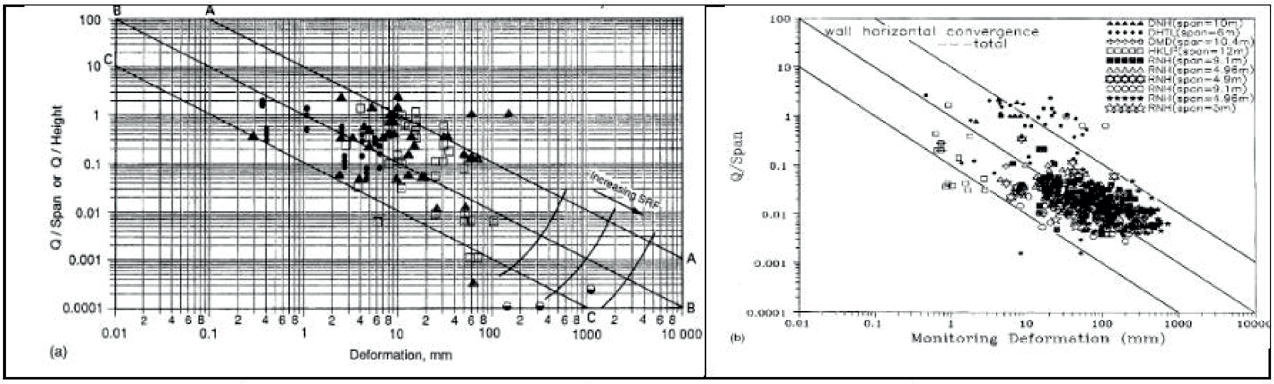
The cross over cavern (Fig. 26) was visited by the author during a Q-histogram logging project for owners Eurotunnel (on behalf of Geoengineering). The lack of LG bolting was all too apparent. Some days later, while in the tunnels with thousands of TML contractors, there was an 'event' in the incomplete cavern – a sudden 40 mm increment and local buckling damage to the LG. It is fortunate for many that emergency drainage holes controlled the sub-sea deformation.

The need for monitoring in the case of NATM designs for caverns and tunnels, which is often used to assist in the timing of the final concrete linings, is in the opinion of the author, due to the actual inadequacy of the temporary support method of LG, though of course this is extensively supplemented with temporary bolting and shotcrete, with deformable slots in the case of highly stressed cases. The author once observed a case with more than 60% of the fully grouted bolts broken in tension (Enasen Tunnel, Japan) due to premature grouting and uncontrolled deformation. Even the final concrete liner cracked on the 'NATM side' of this 1,000 m deep tunnel. In this case, the *double bottom- heading* Japanese contractor method was most successful, with several meters of concrete as foundation for massive steel arches, so stiffer deformation control.

Figure 27 depicts plots of numerous data collected from various tunnels and caverns. The developed correlations to assess tunnel deformation has also been established out of these data.

5. COST AND TIME FOR NMT TUNNELLING

Systematic application of the Q-system reinforcement (B) and support (Sfr) tables for three tunnel cross-sections 50, 90 and 130 m^{2,} illustrates relative costs across the Q-value spectrum in Figure 28. Both arch and wall support costs, and muck transport of 3 km, are included. NMT can be placed in the lower right with a typical Q-value range of 0.1 to 40. Due to the final concrete lining of NATM, the basic NATM cost is as if the Q-value were as low as 0.004 to 0.04, and therefore, NATM is several times higher in cost than NMT. Of course, when Q-values are very low, NMT tunnels, including RRS and maybe even a final concrete lining, will also be very expensive and time-consuming. NATM tunnels seem to always be 'as if' Q was very low.



$\Delta = \dfrac{\text{SPAN}}{\text{Q}}$	$\Delta_v = \dfrac{\text{SPAN}}{100\text{Q}}\sqrt{\dfrac{\sigma_v}{\sigma_c}}$	$\Delta_h = \dfrac{\text{HEIGHT}}{100\text{Q}}\sqrt{\dfrac{\sigma_h}{\sigma_c}}$	$k_o = \left(\dfrac{\text{SPAN}}{\text{HEIGHT}}\right)^2\left(\dfrac{\Delta_h}{\Delta_v}\right)^2$

Nathpa Jakri power station cavern

$\Delta_v = \dfrac{20{,}000}{100 \times 3} \times (6/35)^{1/2} = 28$ mm

$\Delta_h = \dfrac{50{,}000}{100 \times 3} \times (4/35)^{1/2} = 56$ mm

(SPAN = 20m, HEIGHT = 50m, Q = 3, σ_V=4 MPa, σ_h = 6 MPa, σ_C = 35 MPa).
(In the middle of the range of MPBX measurements for the arch and walls).

Gjøvik Olympic cavern

$\Delta_v = \dfrac{60{,}000}{100 \times 10} \times (1/75)^{1/2} = 6.9$ mm

(SPAN = 60m, Q_{mean} = 10, σ_V = 1 MPa at 40 m depth, σ_C = 75 MPa)
(Almost identical to that measured with nine MPBX, and almost identical to UDEC-BB modelling results).

Figure 27- Various empirical equations to assess tunnel deformations

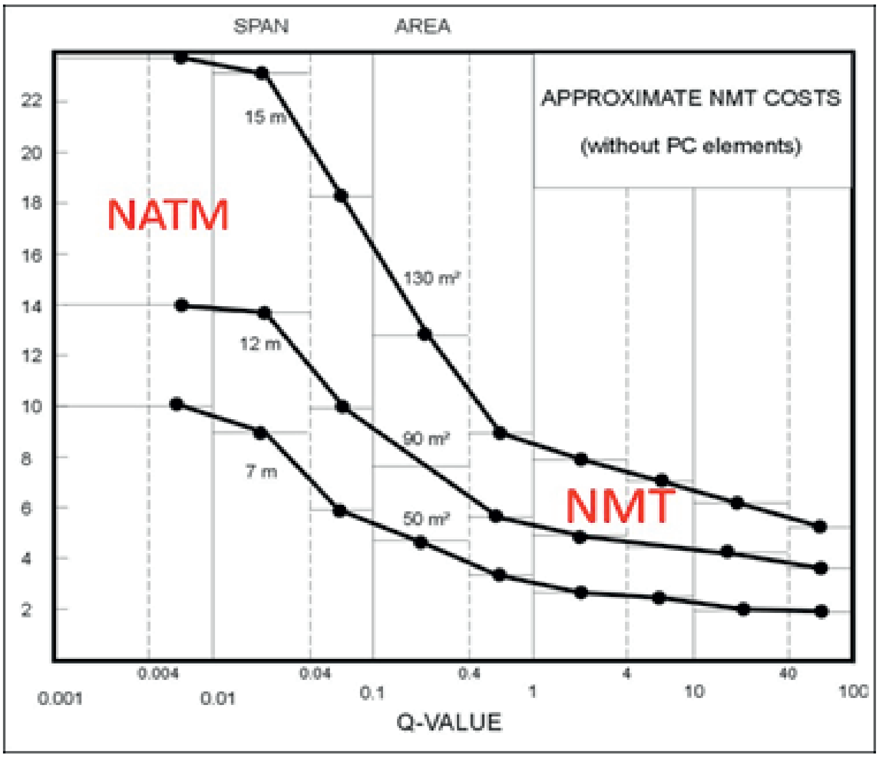


Figure 28 - Cost comparison chart of NMT and NATM for 3 tunnel X-sections

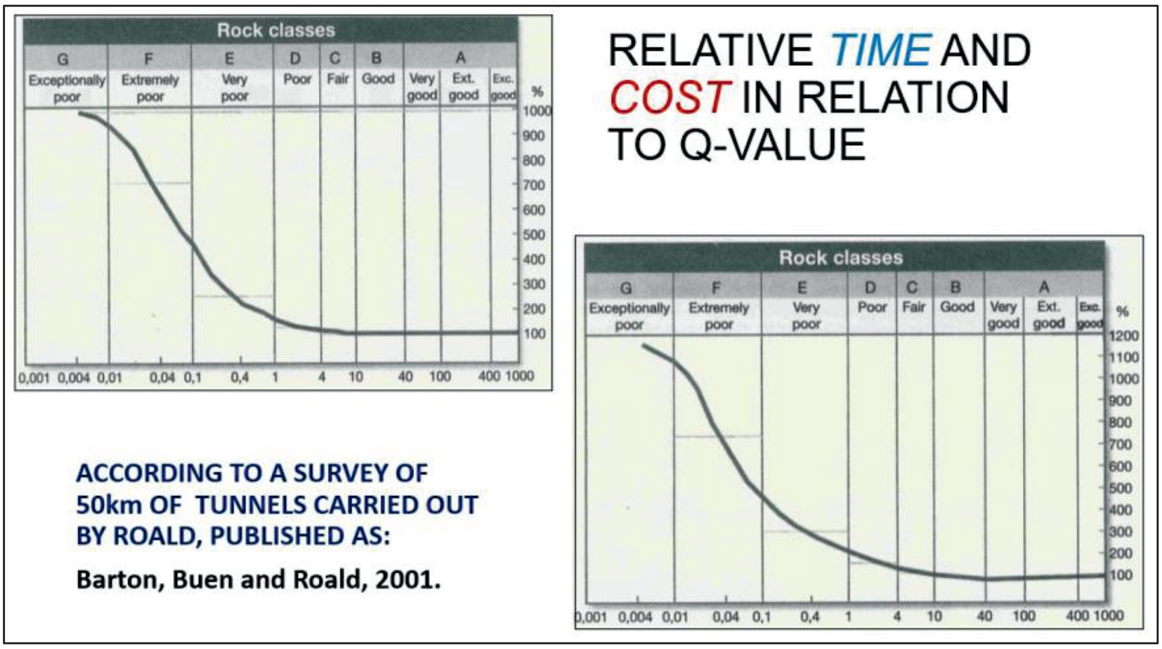


Figure 29 - Relative time and cost in relation to Q-value

Figure 28 shows cost comparative plots for NMT and NATM for three cross-sections of tunnels, viz. 50 m^2, 90 m^2 and 130 m^2. The plots shown in Figure 29 show data from 50 km of Norwegian and Swedish tunnels collected and plotted by Roald. Relative time (10:1 range) and relative cost (12:1 range). Virtual lack of support needs with Q>10 suggests moderate sizes of tunnels.

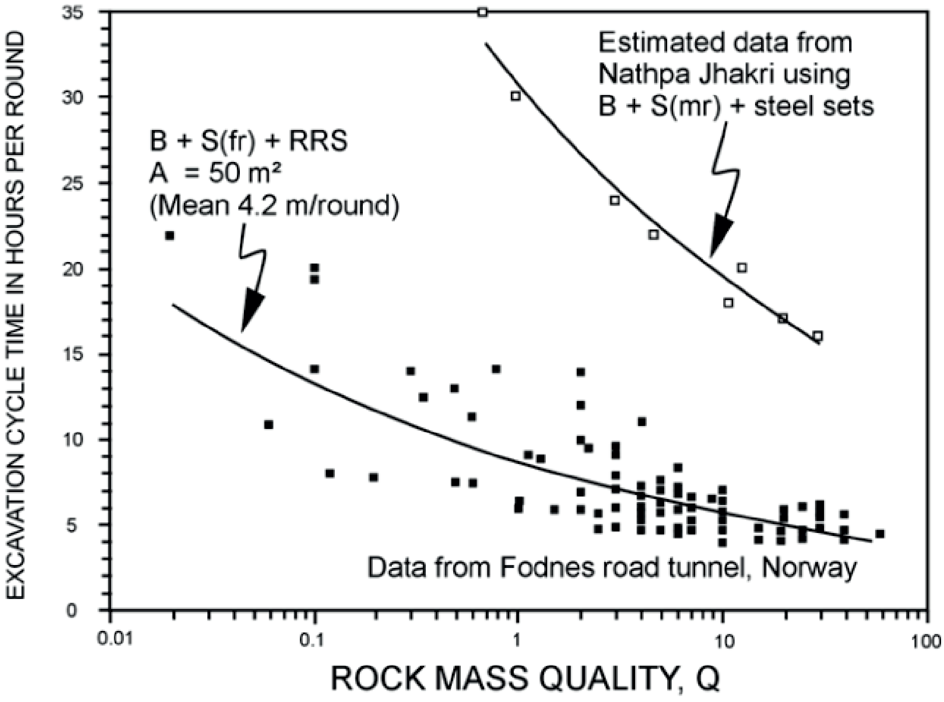


Figure 30 – Excavation cycle-time versus Q-value

Pre-injection and any post-injection repair (24 to 36 hours delay for each umbrella of injection at approx. 20 m intervals) will be added to NMT totals.

Cycle time is clearly an important driver of costs and time. It consists of drilling blast holes, loading with explosives, blasting, waiting for blast gas to clear, scaling, geological inspection, mucking, reinforcement and support. Figure 30 (Grimstad priv. comm.) shows observed Q-related cycle-time data from the Fodnes road tunnel (cross-section of 50-55 m^2). For comparison, the cycle time for labour-intensive temporary support methods in the Indian Nathpa Jhakri hydroelectric project (also collected some decades ago) is also shown.

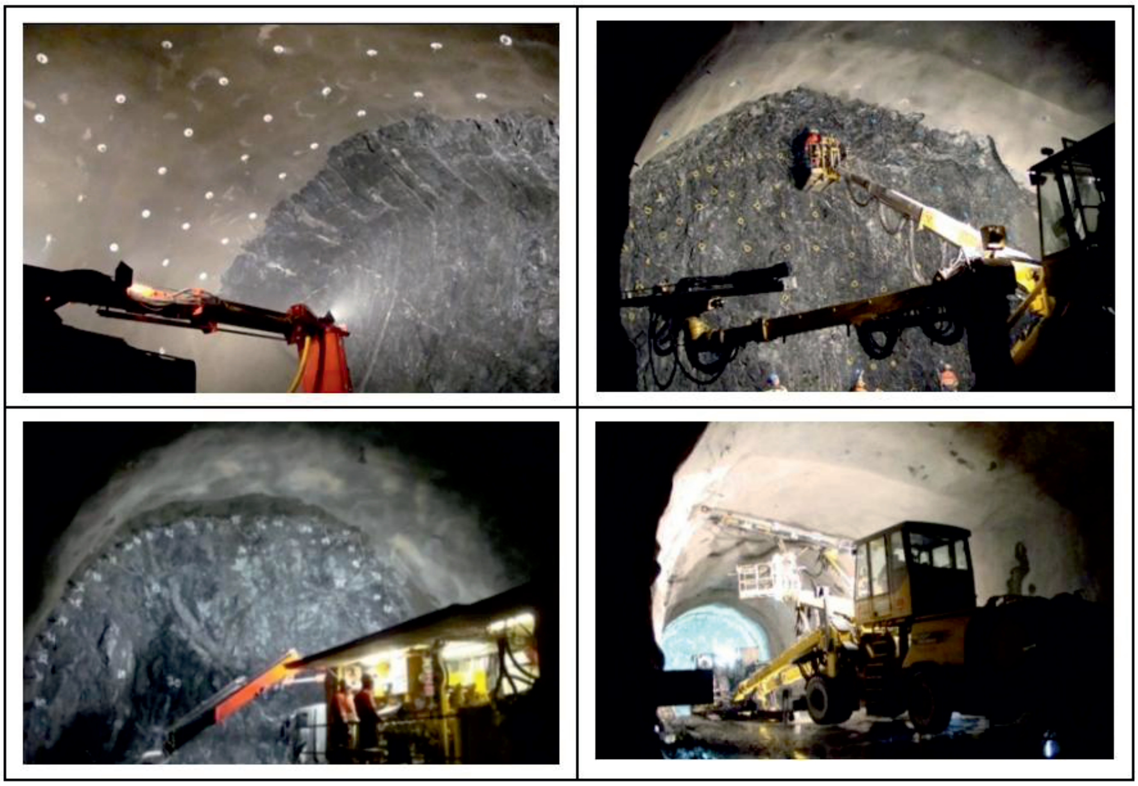


Figure 31- First layer of shotcrete, drilling blast holes, pre-injection, post-injection repair

Pre-injection is an important component of NMT, which increases the (low) cost by about 20% (if chosen). It generally takes 24-30 hours for 1.5 km of injection hole-drilling and injection per ‘umbrella’ screen. Details of good practice and poor practice were discussed in Barton and Roald (2022).

Note the dry shotcrete in this systematically pre-injected 5 km long Bærum rail tunnel, which did not require the designed infiltration wells. In contrast to this excellent result would be dark (wet) shotcrete and environmental damage. The result of using too low injection pressures and incorrect (filter-pump) grout penetration measurements. This Swedish (KTH) method disqualifies stable non-shrinking (Norwegian-style) micro-silica bearing grouts due to the extensional viscosity of these ideal grouts. (Barton and Roald, 2022).

An additional and obvious contributor to the cost or economy of NMT Q-based tunnels is the choice of the ESR number, which is seen in the left axis of Figures 8, 9 and 10. The two diagrams of Figure 32 are a convenient way of viewing the range of stable tunnel and cavern spans (marginally stable in the case of ESR = 5 for the case of mining stopes). The permanently unsupported and unbolted tunnels and storage caverns plotted in the left diagram have an upper boundary approximating ESR = 1.6. This is based on the lower boundary of Figures 8, 9 and 10, which corresponds to ESR = 1.0 in each case.

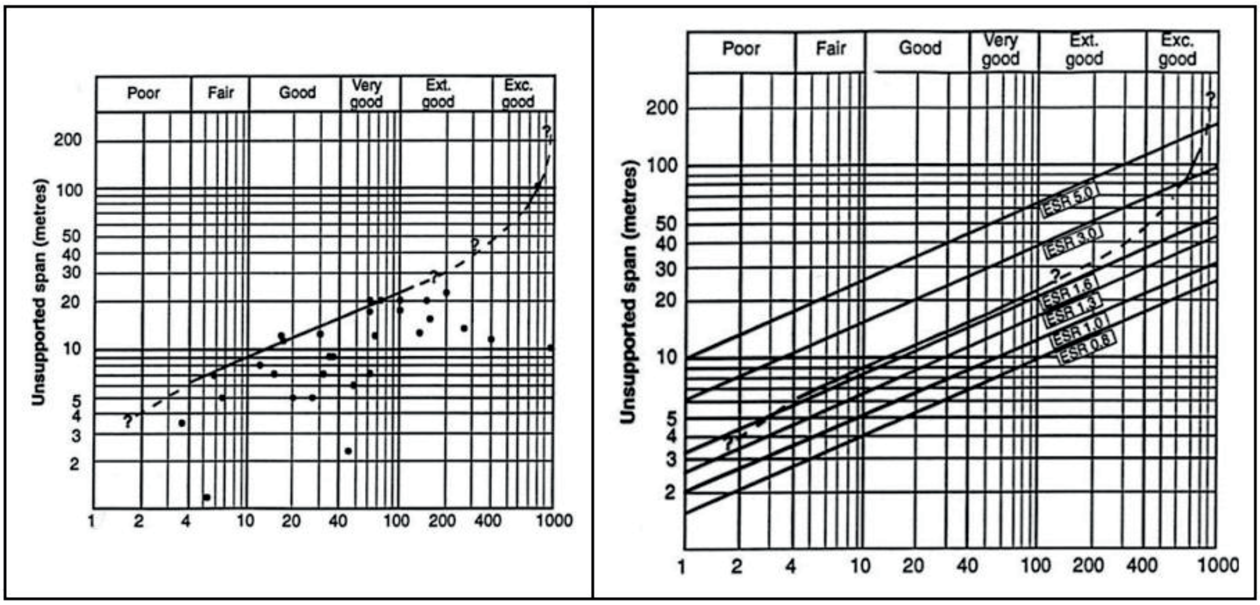


Figure 32- Left: case records of permanently unsupported tunnels and caverns; Right: illustration of ESR values from 0.8 to 1.6 for civil engineering, and from 3 to 5 for temporary mine opening (Barton, 1976)

The Norwegian Road Authority's choice of minimum S(fr) thickness of 8cm corresponds to an ultra-conservative ESR ≈ 0.5 needing an even lower line in the above right-hand plot in Figure 32.

Figure 33 - Construction of large machine hall in Austria using NMT

6. CONCLUSIONS

- NATM and NMT tunnels are based on radically different use of resources, with potential 5x to 10x differences in concrete volumes and costs and time differences that are also significant if accumulating time and cost to tunnel completion.
- One need not fear over-break with NMT. It is a much more expensive problem for NATM.
- Well-executed pre-injection is more reliable than 'membranes' due to the challenge of weld integrity. There may be 12 to 15 km of welds per linear tunnel kilometer in a big NATM tunnel.
- It is urgently necessary to be aware of the risks during the 'lattice-girder' stage of NATM. Lattice girders may deform too much before they begin to apply radial support. It is perhaps therefore that monitoring is always advised in NATM. RRS on the other hand supply radial support during their installation, due to the incorporated radial bolting.
- In NMT and NATM, preliminary support should be appropriate to the rock conditions as logged on site (not pre-conceived and uniform). When using the Q-system, NMT final support matches the rock conditions.
- NMT Q-based tunnel support S(fr) and reinforcement (B) methods for hydropower tunnels require respect for the typical moderate water flow velocities of 1.5 to 2.5 m/s. If used for river diversion and much higher flow velocities, then increases of B+S(fr) are needed with erosion protection, perhaps concrete, also of the invert.

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APPENDIX

For those who are numerically modelling planned NATM and perhaps NMT tunnels, be aware that GSI is repeated extraordinary 16, 12 and 10 times in these key Hoek-Brown equations for presumed 'c', 'ϕ' and 'σ_{cm}', based on substitution of just the three supporting equations for m_b, s and a. In fact, if the H-B equation for σ'3n is also included, the 16-times repetition of GSI for 'c' has to be increased to 16+10+10+10 = 46-times (!), and the 12-times repetition for 'ϕ' has to be increased to 12+10+10 = 32-times (!). Thousands of users of Rocscience software are presumably unaware of these fundamentally unreliable GSI/H-B methods. Plastic zones around tunnels may be grossly exaggerated (proved in an international court case), and rock slopes do not suffer circular failures as predicted when using GSI and H-B, unless the rock is very weak. Soil, sand, rockfill and saprolite may suffer circular failure. Joints and faults and competent rock prevent circular failure (Barton 2025). Pseudo-joint representation by Rocscience also gives suspect results. Joints need to be able to open, close and shear as in UDEC-MC, UDEC-BB and 3DEC-MC.

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

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

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right) \quad s = \exp\left(\frac{GSI-100}{9-3D}\right) \quad a = \frac{1}{2}+\frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right)$$

$$\sigma'_{c,m} = \sigma_{ci}\frac{\left[m_b+4s-a(m_b-8s)\right](m_b/4+s)^{a-1}}{2(1+a)(2+a)}$$

$$E_m(\mathrm{MPa}) = 10^5\left(\frac{1-D/2}{1+e^{\left[\frac{75+25D-GSI}{11}\right]}}\right)$$