5º CONGRESSO BRASILEIRO DE TÚNEIS E ESTRUTURAS SUBTERRÂNEAS
SEMINÁRIO INTERNACIONAL "LATIN AMERICAN TUNNELLING SEMINAR – LAT 2021"
01 a 04 de dezembro de 2021 / São Paulo-SP

TECHNICAL DETAILS OF SINGLE-SHELL NMT TUNNELS

N.R.Barton
NB&A, Oslo, Norway
E. Quadros
BGTech, São Paulo, Brazil

Abstract: Selected aspects of NMT are described in some detail. Tunnelling in jointed rock that may be clay-bearing and faulted is assumed, with a typical wide range of Q of at least 100 down to 0.01, or roughly RMR = 80 down to 20, but not needing double-shell NATM. Selected aspects to be discussed will be the three principle EDZ, two of them representing the load-bearing cylinder of rock where redistribution of principal stresses and joint deformation occurs, the third the disturbance due to blasting, which is much narrower. So-called 'plastic' behaviour via GSI, H-B, RS2 modelling is rejected since based on too many assumptions and complex page-wide equations. Case records suggest that combinations of bolting and fiber-reinforced shotcrete can provide stable tunnels at reasonable cost, but if some aspects are neglected, like under-dimensioned shotcrete thickness, lack of washing prior to shotcreting, and failure to record the presence of clay, then surprises can occur. Two important further conventions need to be adhered to. The Q-system based B+S(fr) reinforcement and support recommendation was never designed to accommodate or rely on lattice girders, which are far too 'soft' since unbolted and unevenly loaded. Single-shell Q-based tunnel design was also never intended to allow the passage of water at high velocities, such as 10m/s river diversion compared to the case-record expected 2m/s of typical headrace and pressure tunnels. When rock mass quality is compromised by fracture zones, or if permeability is too high and inflow from the surrounding rock mass needs prevention for ensuring both dry in-tunnel and stable external environments, then systematic pre-injection may be demanded. Injection of suitable stable grouts at high pressure improves the rock mass quality Q, and over-design of unadjusted Q-based support is then apparent. P-wave velocities, and deformation moduli are also improved by pre-grouting, as verified in formal dam-site studies in Brazil and Iran. In reality millions of kilometers of grout holes beneath the world's largest dams are giving the same evidence. Suitable stable grouts with their extensional viscocity must not be disqualified with filter-pumps. High injection pressures are needed, but do not hold pressure when flow ceases. Wet shotcrete, leaking bolt holes, and the need for post-injection indicate failed technology, if the objective was to pre-inject in one round only and prevent environmental damage.

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 analysed, with deliberate choice of challenging cases such as clay-bearing and sheared rock masses so that significant amounts of support were 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.

Development of the Q-system has 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. Significantly, the Q-system data base and applicability was greatly expanded in 1993, by Grimstad's incorporation of steel fiber reinforced shotcrete S(fr) and by 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 successfully used in rocks with UCS as low as 4 to 7MPa (significantly jointed chalk marl in shallower parts of the Channel Tunnel: Barton and Warren, 2019) and UCS up to at least 300MPa for some granites, gneisses and quartzites.

The economic advantages of single-shell tunnels for hydropower has made this form of water 'conveyance' very attractive in relation to more expensive concrete lined alternatives. There are tens of thousands of kilometres of single-shell or nominally 'unlined' tunnels, and all need sound design. This includes respect for moderate 1.5-2.5m/s flow velocities. Q-based single-shell tunnels were never intended for 10m/s river diversion unless with full-profile S(fr). 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 concrete 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 1km and even 3km distant damage to homes due to failure to pre-inject leaking tunnels (Barton and Quadros, 2019).

2 SOME BASICS ABOUT TUNNELS IN JOINTED ROCK

The authors, like many others in the last two decades, have been impressed (unfortunately negatively impressed) by all the colourful plots of 'plastic behaviour' in continuum modelling approximations, which has been a growing trend in rock 'engineering'. The problems start with the rock mass characterization being so crude with GSI and its limited RMR joint condition scale on one axis, and some five or six sketches of rock masses on the other axis (optimistically added to by later authors). This is followed by software-assisted application of page-wide Hoek-Brown equations for 'c' and 'φ' with both these strength components incorrectly assumed to be mobilized simultaneously.

We believe that the reality of rock mass response to tunnel and cavern excavation is much closer to what is illustrated below. 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' zone assumptions. These have been proved in a formal court case to be grossly exaggerated in relation to the recorded behaviour of reality – a tunnel without 'plastic' zones. In the following, we have two excavation disturbed zones, and one damage zone. All can be compromised.

EDZ_1 - stress-redistribution in the solid rock surrounding and forming the tunnel in rock

EDZ_2 - deformation of the rock joints as a result of the stress-redistribution in this surrounding rock

EDZ_3 - dynamic shock-loading causing blast-induced cracking, loosening, enhanced local 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.

The first two EDZ mechanisms which are illustrated in Figure 1 are demonstrated in distinct element models of TBM modelling performed for a spiral access tunnel. These date from the 1990's work at NGI in consulting for UK Nirex. Drawdown and water flow in the joints can also be modelled as in Figure 2. Figure 3 shows the 62m span Olympic ice-hockey cavern which was supported with 10cm of steel-fibre reinforced shotcrete and permanent rock bolts of 6m length at 2.5m 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, By, Chryssanthakis, Tunbridge, Kristiansen, Løset, Bhasin, Westerdahl & Vik in Barton et al. (1994). MPBX showed maxima of 7 to 8mm deformation, as predicted by Chryssanthakis's UDEC-BB modelling. See input table, bottom-left in Figure 3.

Remembering the *a posteriori* origin of the Q-system it is wise for today's numerical modellers 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 in relation to proven practice.

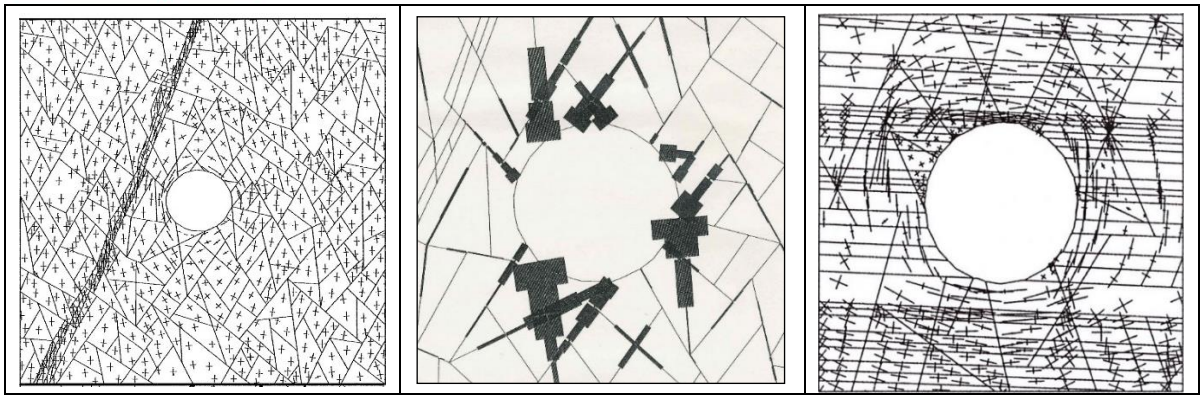

Figure 1: Examples of EDZ 1 (principal stress re-distribution in the tunnel surroundings) in a welded tuff-ignimbrite and in an interbedded sandstone sequence. The central diagram shows joint shearing magnitudes (of mm scale). EDZ 2 joint responses also include joint opening and closing, and with UDEC-BB also the physical and hydraulic joint apertures E and e. The latter is used in flow modelling. NGI modelling for UK Nirex in 1992. See Barton (2000) for details of EDZ 2 behaviour.

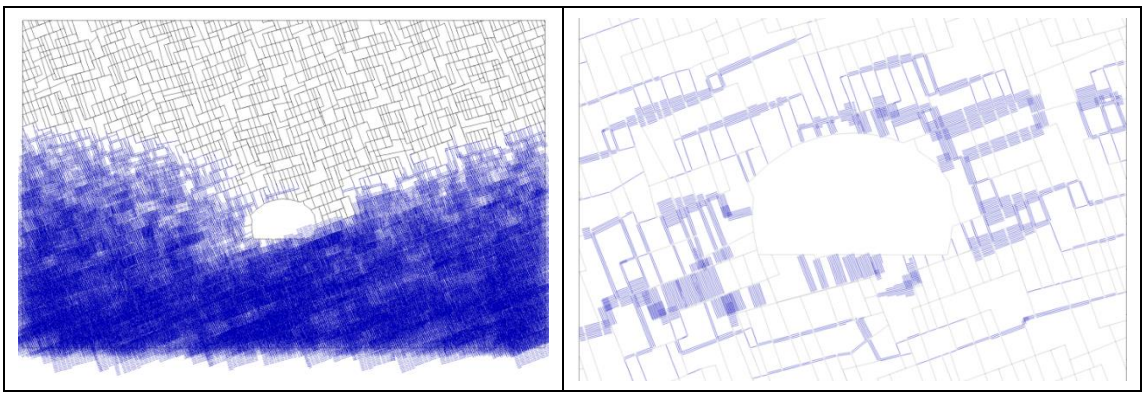

Figure 2: An example of coupled M-H modelling with water flow towards a tunnel causing drawdown. Note the anisotropic inflow (two different draw-down stages are shown) due to anisotropic permeability generated by the different joint apertures of the two modelled joint sets. (UDEC-BB modelling by K. Monsen, 2017: priv. comm.).

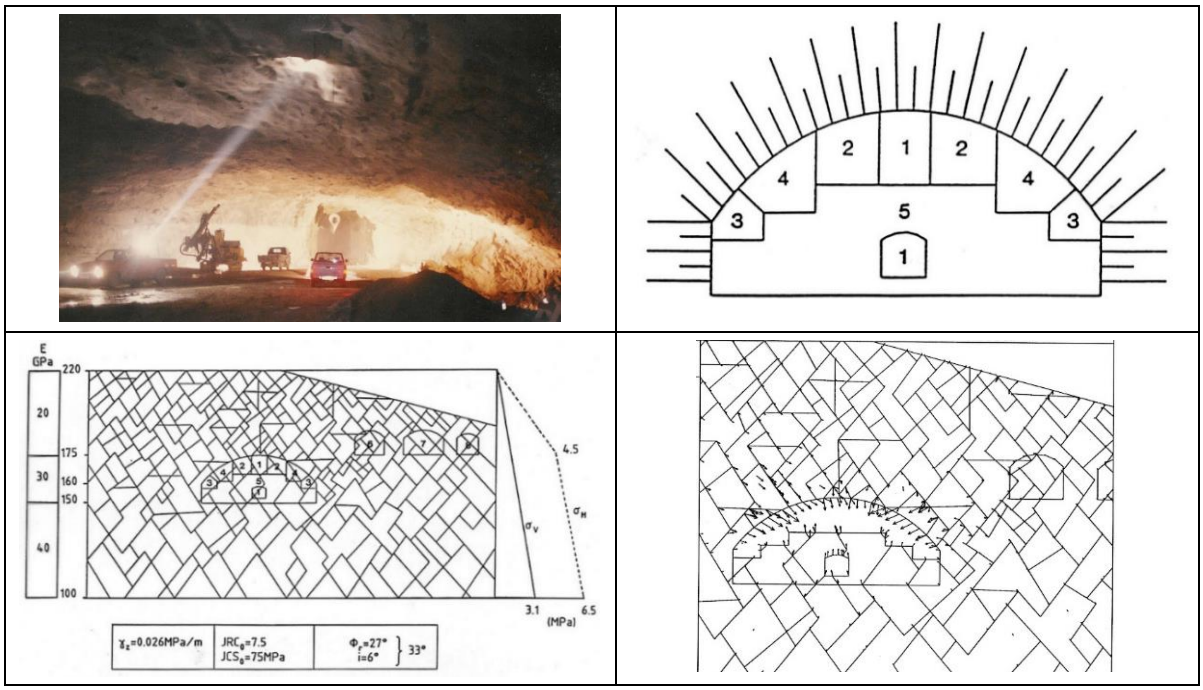

Figure 3: Numerical modelling with UDEC-BB for design checks of the Gjøvik cavern, which is an example of single-shell B+S(fr) cavern design. The huge arch is 'supported' with 10cm of S(fr) and permanent CT bolts. The rockmass is the support.

3 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 (the New Austrian Tunnelling Method). Principal aspects of NMT are shown in Figure 4. As we shall show later, NMT has some considerable advantages in terms of speed and cost, and actually presents a much smaller 'environmental footprint' than NATM due to the widely different effect of over-break on the concrete volume used in the two methods. This also affects the design philosophy for water transfer and headrace tunnels (Barton and Quadros, 2020).

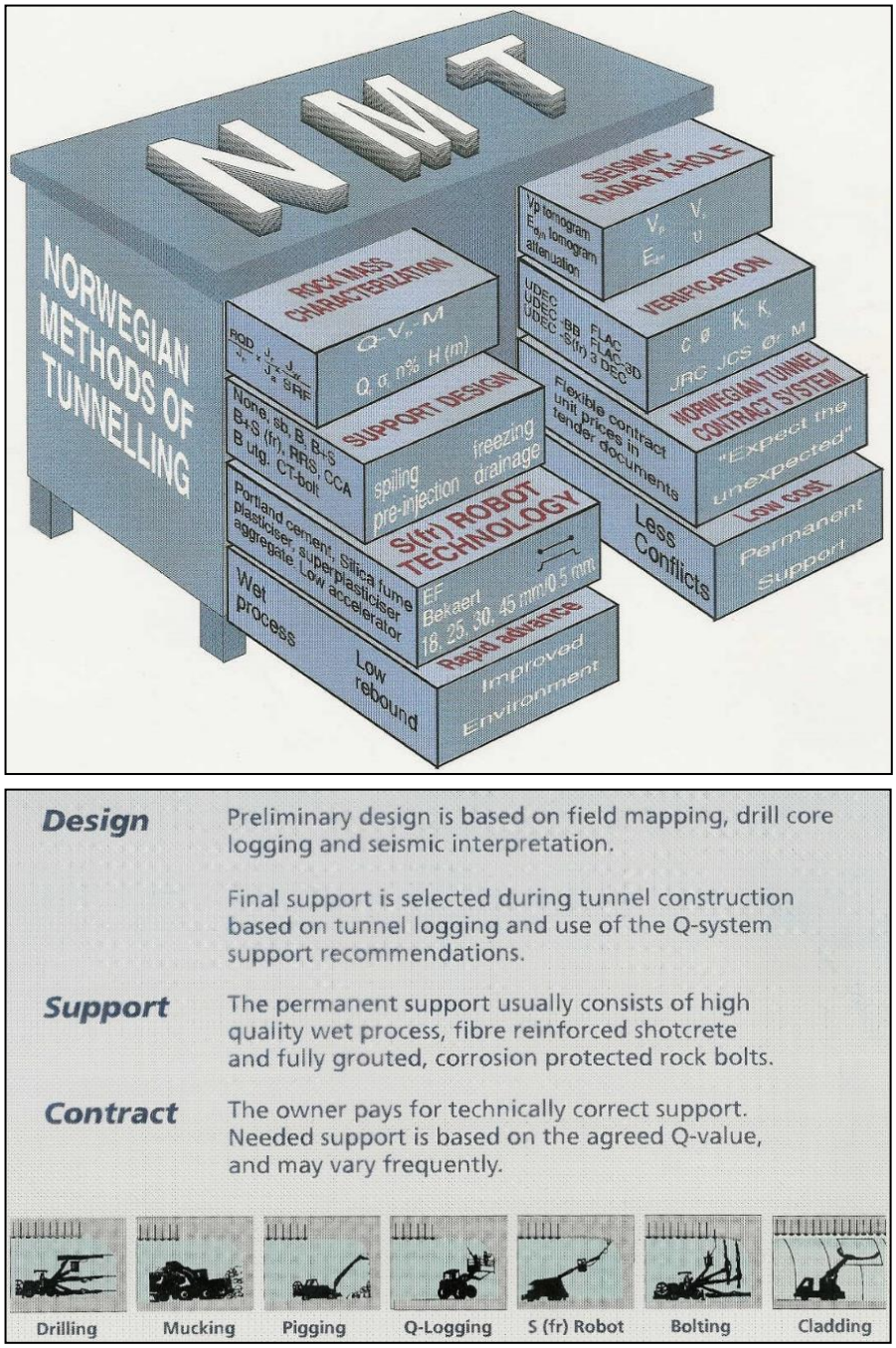

Figure 4: A diagrammatic and short descriptive summary of NMT. Many details given by Barton, Grimstad, Aas, Opsahl, Bakken, Pedersen, & Johansen from the companies NGI, Selmer A/S, A/S Veidekke, Enreprenørservice A/S, NoTeBy & Statkraft. (Barton et al. 1992). The first step in applying NMT is to collect Q-parameter data. Refer to Figures 5 to 8.

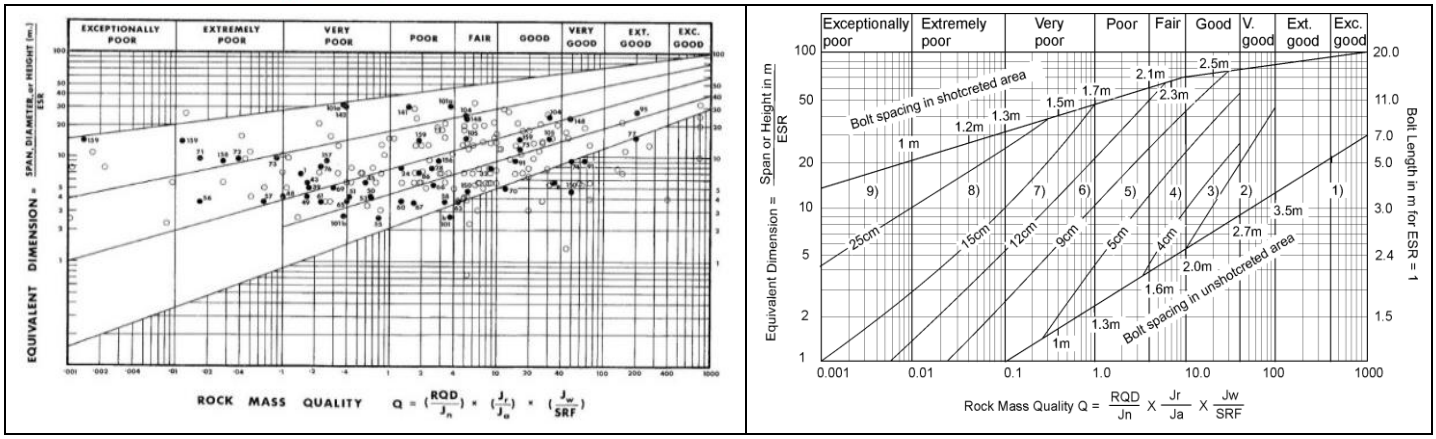

Figure 5: The initial spread of case records in the original development of Q is shown on the left. There were 50 rock types. On the right is the update to S(fr) from S(mr) from Grimstad and Barton (1993). Grimstad had collected 1050 additional (and independent) cases, mostly road tunnels that were not designed with the benefit of the Q-system.

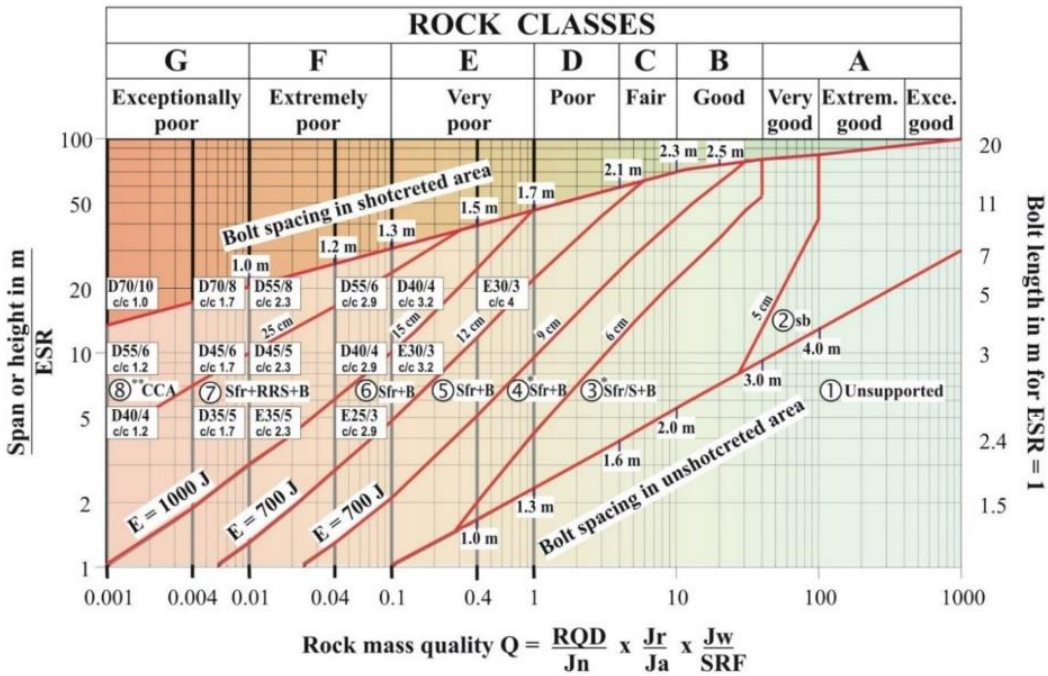

Figure 6: Grimstad and NGI modelling colleagues were responsible for the dimensioning of RRS (rib reinforced shotcrete). Note that the minimum S(fr) thickness is increased to 5cm to assist in more efficient curing and support. This complete version of the Q-based support chart is preferred to the truncated NGI version given in Google. (Barton and Grimstad, 2014).

Figure 7: An important part of Q-system application for NMT is of course core-logging and where possible, rock exposure logging. These are examples from the Bærum rail tunnel and from Follobanen. In the former roughness JRC was also logged to assist in pre-injection material selection based on interpretation of both Lugeon (and Q_{H2O}) and seismic refraction data.

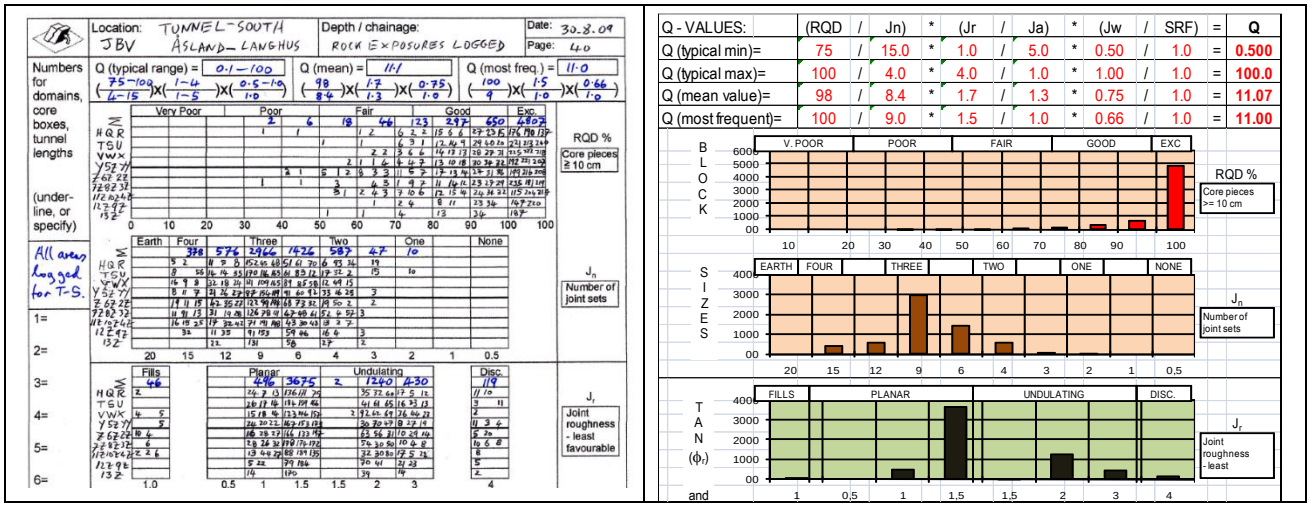

Figure 8: In the 1974 Q-system support tables, conditional factors RQD/Jn and Jr/Ja representing relative block size and frictional strength were specifically evaluated to emphasise the relative needs for more or less shotcrete and more or less bolting respectively for a given Q-value. Q-parameter statistics have been a feature of Q-application for the last 30 years. These two figures show part (just the first three parameters) of a major Q-parameter collection, and the full numerical statistics for surface exposure logging at Follobanen South using the Q-histogram method (Barton and Gammelsæter, 2010).

4 NMT SUPPORT AND REINFORCEMENT DETAILS

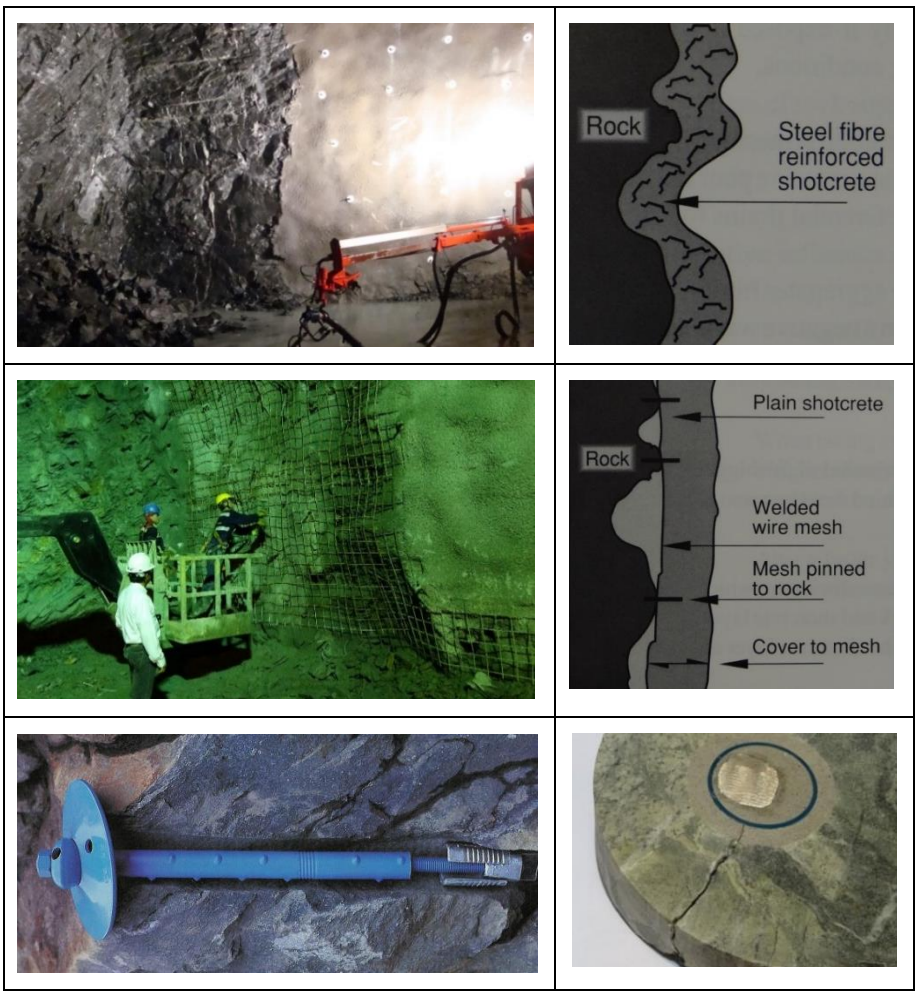

Figure 9: Since the proverbial figure is worth a thousand words we can start by illustrating the S(fr) and S(mr) options (the choice must be very clear). The realistic drawings are from Vandervall (1990). The CT bolt 'demo' demonstrates corrosion protection, even when an intersected joint cracks the outer annulus of grout: an inevitable ocurrence when tunnelling.

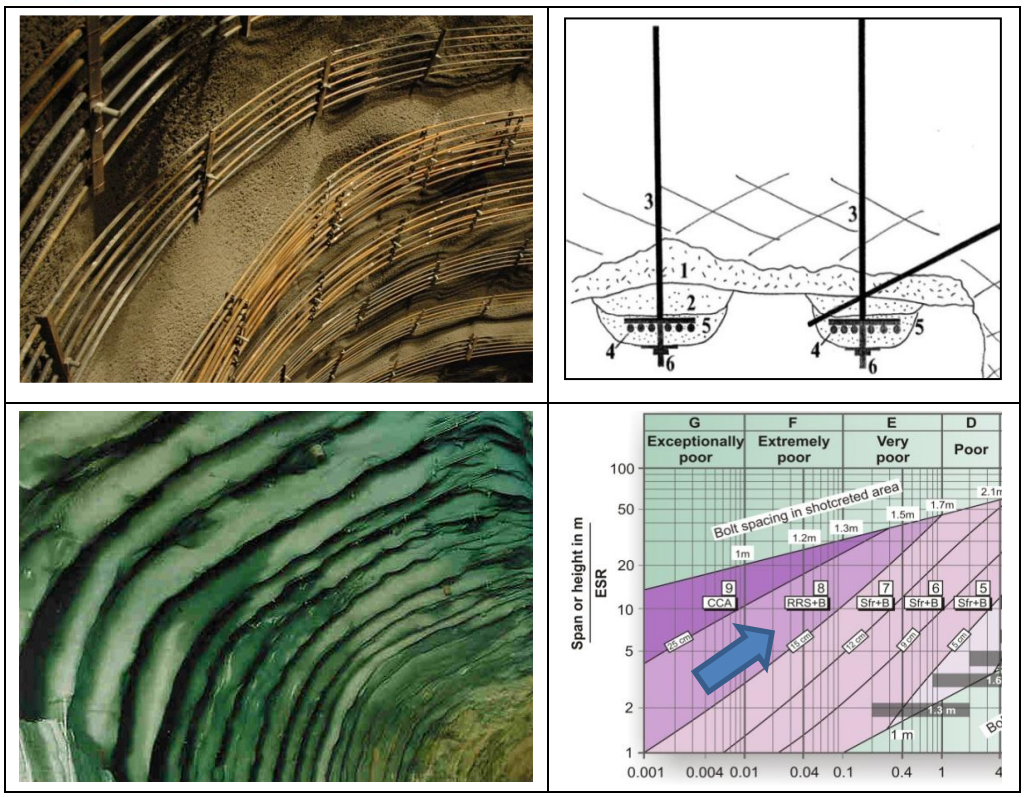

Figure 10: When faulted clay-bearing (low Q-value) ground has to be tunnelled through the NMT solution is RRS (bolted rib-reinforced shotcrete arches). Principles and practice are illustrated here, and design choices were illustrated in Figure 5. Note that the 28m span railway station (bottom left) was excavated left then right, due to a deep sediment-filled valley in down-town Oslo (National Theatre station). The shape of the RRS was requested detail, prior to concrete lining. Note that RRS are systematically bolted so far more robust and therefore safer than lattice girders, which have been know to buckle.

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, with the possible additional influence of conditional factors Jr/Ja (internal friction affecting bolt spacing and capacity) and RQD/Jn (affecting shotcrete thickness) as recommended by Barton et al. (1974). The manner in which the cost of NMT is impacted by tunnel dimensions and Q-value is shown in Figure 11.

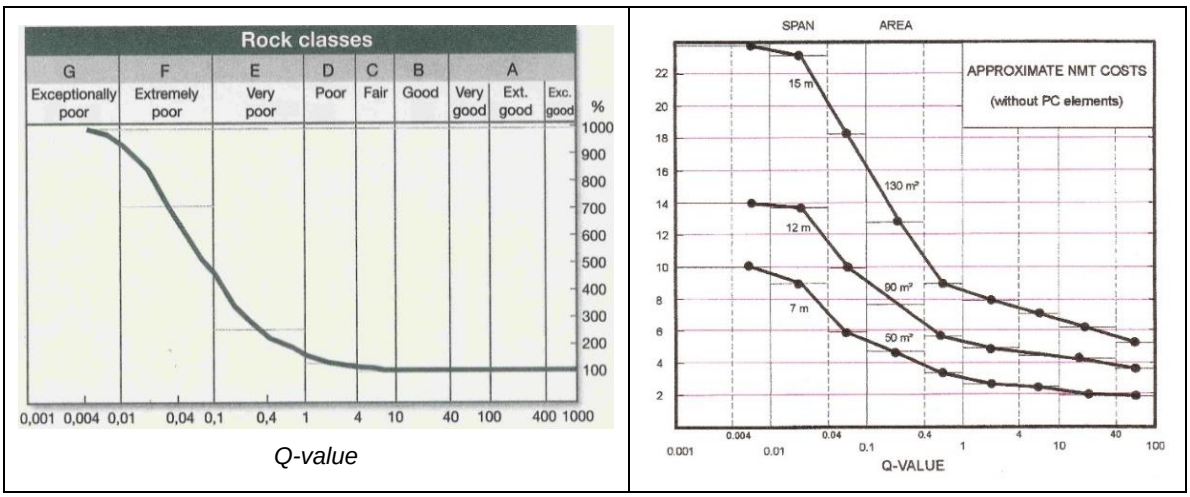

Figure 11: Left: Relative cost of tunnel construction (10:1 for cost, 12:1 for time) in relation to Q-value, according to a 50 km survey of tunnels in Norway and Sweden, carried out by Roald, and published in Barton et al. (2001). The three curves on the right were derived by a systematic application of the Q-system and include different quantities of arch and wall support when Q-values are sufficiently high, following Q-system instructions. All (five) curves give implicit comparison of relative time and cost of NMT compared to double-shell NATM where concrete is always used. Comparison does not favour NATM.

5 OVERBREAK EFFECTS ON NMT AND NATM

Overbreak caused by sufficient numbers of joint sets (typically three or more) and sufficiently planar joint surfaces is a sure way to maximise the cost difference and concrete volume differences between NMT (which does not fill over-break with shotcrete) and NATM which does have to fill overbreak, firstly using shotcrete to minimise 3D surface unevenness prior to drainage fleece and membrane fixing. Some years ago Barton (2007a) suggested that when the ratio Jn/Jr ≥ 6 there will be inevitable geologically-caused over-break. This has since been used in claims situations by international contractors. If a Q-parameter statistic (histogram) analysis based on drill-core logging indicates a predominance of Jn/Jr ≥ 6, this will influence the large-scale roughness. The Jn/Jr statistic in Figure 8b would suggest over-break problems if the tunnelling had been by drill-and-blast or TBM (as in the Channel Tunnel. Illustration of some extreme consequences of Jn/Jr ratios are shown in Figure 12.

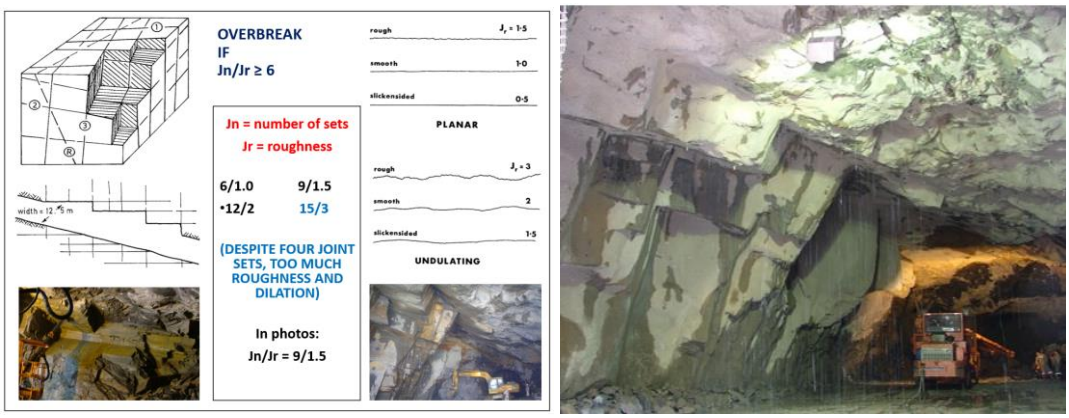

Figure 12: When there are sufficient joint sets to define blocks, together with the presence of the excavation for stress release, then the roughness of the principal joint sets will play an important role in determining if over-break occurs, or does not, for instance because Jr = 3. In a recent case: Jn =15 (four sets), three of them steeply dipping, and Jr = 1.5. Over-break was inevitable. Sometimes Ja > 1 also assists.

Figure 13: Examples of drainage fleece and membrane in a metro cavern and in a rail tunnel. In the latter with a curved perimeter of about 30m there will be the need for 12 to 15km of membrane welds per running kilometer of tunnel. If membrane leakage should occur and enter the tunnel via a shrinkage crack in the concrete, the actual leakage location will not be detectable, so post injection will likely fail.

Figure 14 (top-left) illustrates actual over-break in an NMT (B+Sfr and Q-system designed) rail tunnel in Sweden. Structural engineers might doubt the capacity of the shotcrete to take load when not in a cylindrical form. The reality is that the rock mass is providing a much wider (many meters thick) load bearing ring. The shotcrete merely holds the rock mass securely in place with the help of the systematic bolting. The shotcrete is not loaded in the traditional circumferential sense. The rough surface makes it extremely stable. The NATM road tunnel (top-right) is deliberately drawn with potential overbreak

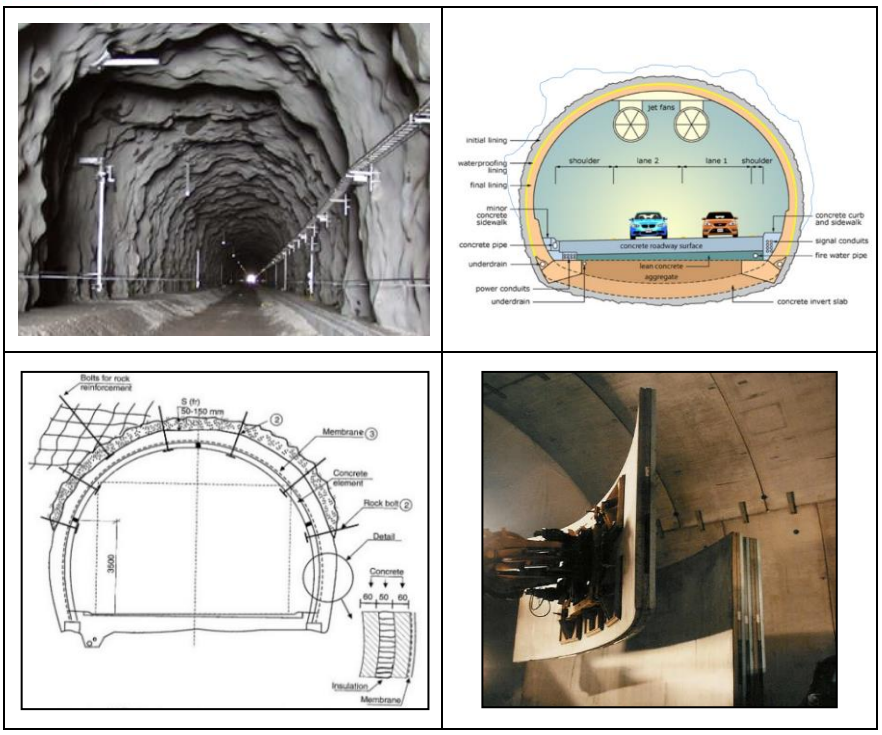

Figure 14: Examples of the different ways that over-break is tackled in NMT and NATM tunnels. In usual practice the top-left example will have been systematically pre-injected, but in remote areas under hills this may be unnecessary.

added, to emphasise that the concrete in this NATM tunnel is seldom an ideal 35cm thick cylinder. Concrete volume differences NMT:NATM may easily reach 1:10. The rough inner surface of the rail tunnel will not usually be acceptable in a main road tunnel, so in the NMT case a free-standing PC-element bolted (four-parts) liner is frequently used, as shown above from Kveldsvik and Karlsrud (1995) and as the final diagram ('cladding') in Figure 4b. The latter are road tunnels, but the PC-elements being lifted into place (Figure 14) are in a twin-track rail tunnel. An outer membrane sheet keeps the tunnel dry internally if environmental concerns of water inflow are relaxed for a short tunnel.

Figure 15: The use of lattice girders in the temporary support phase of NATM has it's own special challenges when there is over-break, since shotcrete volumes needed to spray in the steel bars may be very large. These two examples (a section of motorway and a section of cavern) failed dramatically and in one case tragically, due to non-uniform loading of the lattice-girders which are too 'soft' since unbolted, therefore can bend and also need non-yielding 'elephant feet'. Barton (2017).

6 PRE-INJECTION: IT'S PURPOSE AND CORRECT PRACTICE IN NMT

When rock mass quality is compromised by fracture zones, or if permeability is too high and inflow from the surrounding rock mass needs prevention for ensuring both dry in-tunnel and stable external environments (i.e. avoidance of settlement damage in sub-urban tunnelling), then systematic pre-injection may be demanded. Injection of suitable stable (micro-cement and micro-silica) grouts at high pressure actually improves rock mass quality, and over-design of unadjusted Q-based tunnel support is then apparent. P-wave velocities, and deformation moduli are also improved by pre-grouting, as verified in formal dam-site studies in Brazil and Iran. (Quadros and Correa Filho 1995, Zolfaghari et al. 2015). In reality, literally millions of kilometers of grout holes beneath the world's largest dams are likely to be giving the same evidence. Velocity increases of 1 to 2 km/s are proof from cross-hole V_P.

Those who disqualify suitable grouts with filter-pumps have misunderstood the high extension viscosity of ideal stable grouts. If they do not allow high enough injection pressures they have failed to see the need of local hydraulic jacking of the joints close to the holes since pressure drops rapidly while flow is occurring. If they suggest holding pressure when flow ceases they have misunderstood the actual risk of hydraulic fracturing of grouting already done. These mistakes are compromising this profession. Tunnels showing wet shotcrete, and the need for second injection rounds, and the need for injecting leaking bolt-holes are each representing failed technology, and are giving pre-grouting, indeed even tunnelling, a bad name. This has been happening in some countries for two decades and can be partly blamed on misunderstanding of the filter-pump and its *automatic de-selection* of the best, stable, micro-cement / micro-silica-based grouts. (Roald and Saasen, 2004).

As an analogy to grout flow in a rock joint: consider *flow of mixed traffic*: cars, vans, lorries, busses along a busy winding road with parked cars and busses on both sides. The parked vehicles are the 'slow' particles (at 100,000:1 scale) in the parabolic flow within a rough rock joint, with maximum speed in the centre as in hydraulic laminar flow. Traffic (the micro-cement and micro-silica, with its extensional-strength viscosity) moves slowly but surely – and continuously fills the road (whose width may vary). Place a multi-entrance tollbooth in the way where widest and see what happens. Even without extensional strength between the items of traffic - as in the case of an ideal grout - the traffic flow is hugely delayed. This is the filter-pump screen effect. Proponents of the filter pump propose 'many times wider roads to take the traffic': i.e. joint apertures of 8 to even 12 x d_{95} instead of the laboratory measured grouting of joints with known hydraulic aperture which convert to approx. 4 x d_{95} to 4 x d_{98} in terms of mean physical aperture. (Barton and Quadros, 2019 and Bhasin et al. 2002).

When the information seen in Table 1 is available, as it was in personally logged core for three rail tunnels, and including seismic refraction data and Lugeon testing, it is common to see the Q-value trends such as those shown in this table.

Table 1: Easily remembered potential links between Q_c (= Q x UCS/100), Lugeon or permeability, and seismic velocity. These approximate links may be experienced when no clay is present in the rockmass. Barton, 2006.

Q_c	0.1	1	10	100
≈ Lugeon	10	1	0.1	0.01
≈ K m/s	10^{-6}	10^{-7}	10^{-8}	10^{-9}
V_P km/s	2.5	3.5	4.5	5.5

If on the other hand such (easily remembered) trends seem to be 'violated' it may be because of the presence of clay-filled joints. It is not then logical to have Lugeon values increasing with lower (joint frequency-related) Q-values, because the clay fillings or coatings are likely to reduce the permeability. It is then that Q_{H2O} can be utilized with a reversed Ja/Jr in place of the standard Jr/Ja. Depth-dependent permeability can then be estimated (Barton, 2007b, Barton and Quadros, 2019). On the above basis an estimate of the likely range of tunnel inflow can be made, for instance using the pre-injection sensitive

equation of Brantberger et al. (1998). The final decision is the level of acceptable tunnel inflow: 1 to 4 l/min/100m (a demanding but achievable goal), or 8 or 12 l/min/100m. The latter two hardly protect the surface environment and are too high for the tunnel, allowing drips and potential icicle formation.

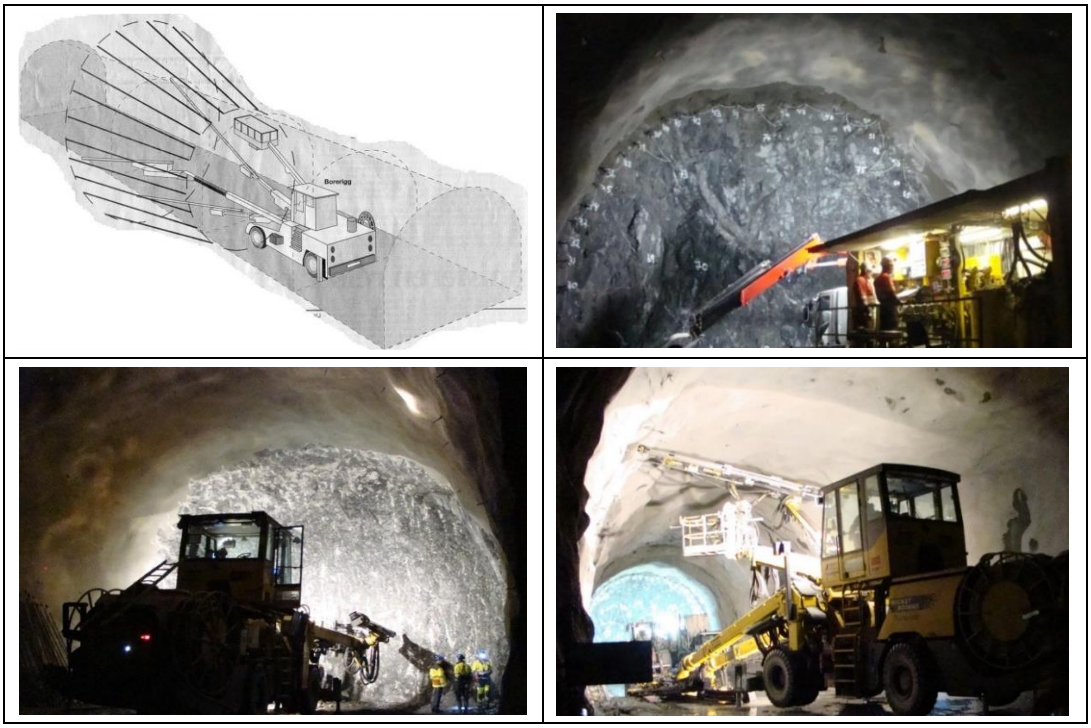

Figure 16: Pre-injection 'umbrella' and application throughout the 5km long double-track Bærum Tunnel. The drill jumbo in the bottom-right photo is being used for a single-point post-injection. Note the intended dryness of the shotcrete, despite high local permeabilities due to numerous igneous dikes. Pre-injection displaces the water. Here 1 l/m/100m was achieved.

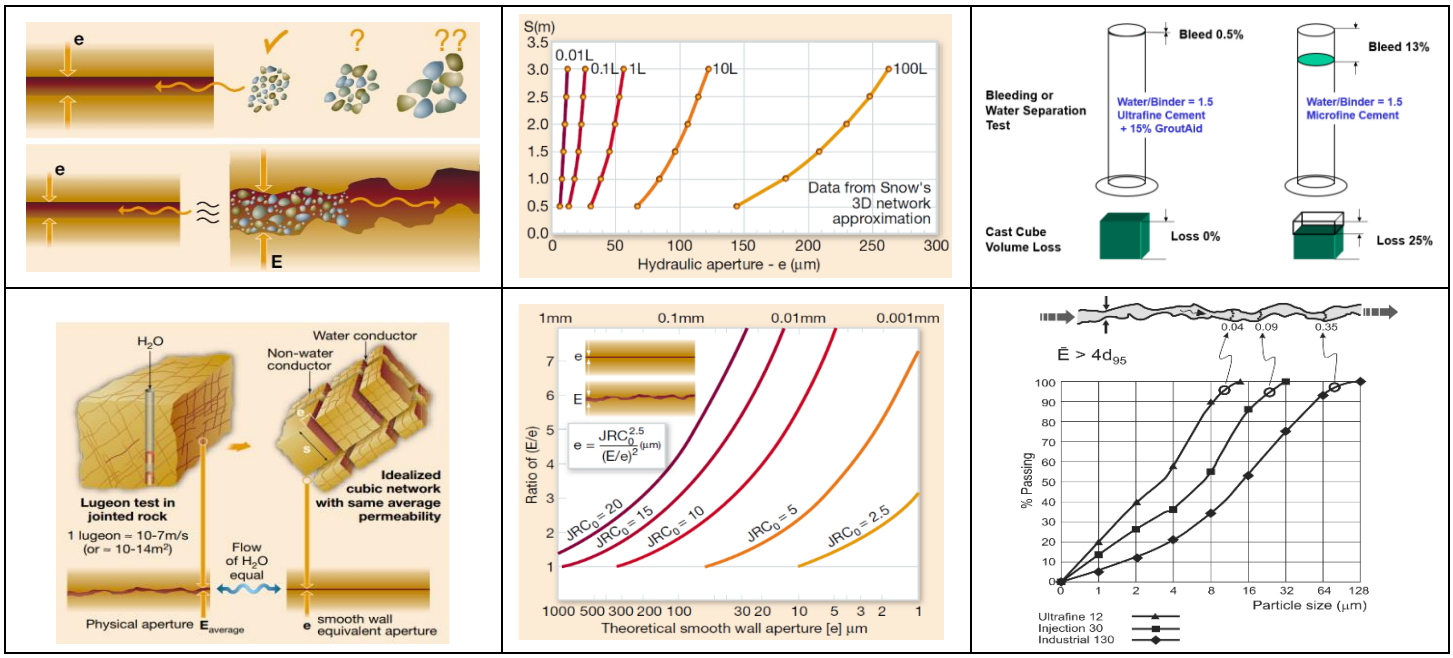

Figure 17: Grout selection must be based on recognition of the different apertures E and e. The former is physical and is the subject of grouting. Conversion from the hydraulic e to E is via joint roughness JRC (see estimation in Figure 7a). Based on a modified cubic 'Snow' network (bottom-left: with two of three joint sets conducting on average, whatever the direction of the gradient of flow) a value of 1.0 Lugeon (top-middle diagram) already suggests e ≤ 50μm. A need for ultrafine cement is implied (bottom-right), but usually E>e so micro-cement (+ micro-silica) may be suitable. Note the volume loss comparison.

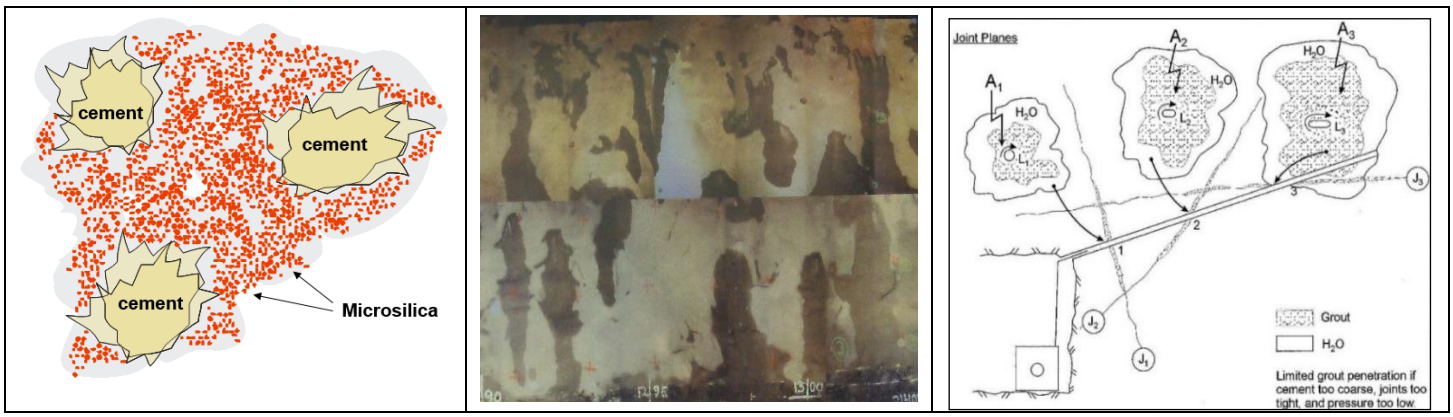

Figure 18: These first two figures, and the volume loss comparison in Figure 17 are from Roald (Priv. Comm. 2010). Incorrect pre-grouting strategy causes a 'coffee-filter' effect with more water in the rock mass than intended for the reasons given in the inset: grout is too coarse, joints are too tight, pressures is too low. (Barton, 2011).

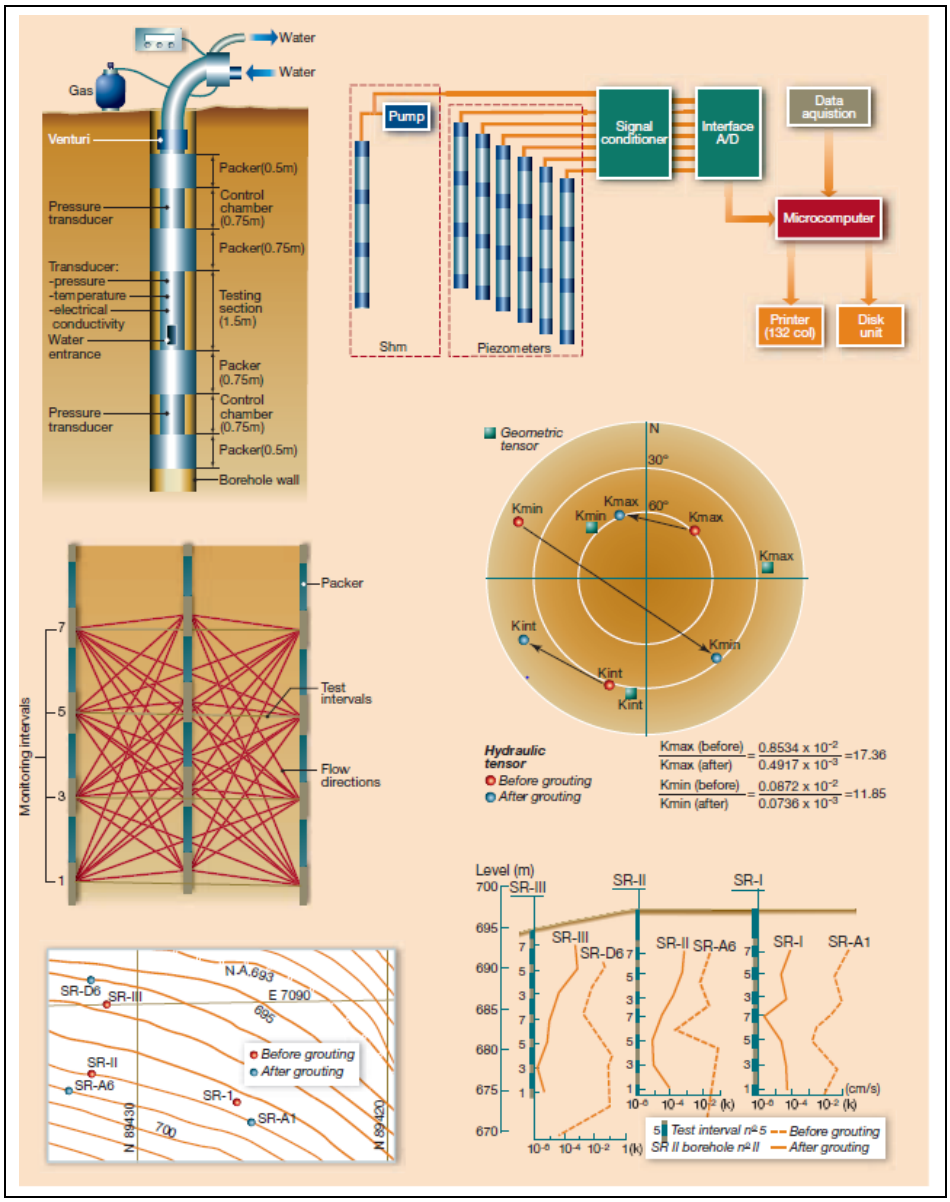

Figure 19: 3D permeability testing before and after dam-abutment grouting. Note permeability tensor rotations implying successive sealing of the most permeable joint sets. This supports the Q-parameter improvements suggested in Barton (2011) and verified by Zolfaghari et al. (2014). (3D before-and-after grout testing by Quadros and Correa Filho, 1995).

Table 2: The Q-system parameters are ideally suited for studies of Q-improvement-by-grouting. 3D permeability test data from Brazil and Iran suggest the successive sealing of joint sets. This means that not only the effective RQD is increased, but Jn will be reduced and Jr/Ja will then apply to different (less permeable) joint sets. Logically speaking Jw will become 1.0, and SRF might even be reduced if applying to a groutable fault. Note that air/water high pressure flushing of clay fillings and coatings prior to grouting has been verified as a successful technique and was used in a major underground project in India.

CONSERVATIVE MODEL	MORE REALISTIC MODEL
RQD increases e.g. 30 to 50%	RQD increases e.g. 30 to 70%
Jn reduces e.g. 9 to 6	Jn reduces e.g. 12 to 4
Jr increases e.g. 1 to 2 (due to sealing of most of set #1)	Jr increases e.g. 1.5 to 2 (due to sealing of most of set #1)
Ja reduces e.g. 2 to 1 (due to sealing of most of set #1)	Ja reduces e.g. 4 to 1 (due to sealing of most of set #1)
Jw increases e.g. 0.5 to 1	Jw increases e.g. 0.66 to 1
SRF unchanged e.g.1.0 to 1.0	SRF improves e.g. 2.5 to 1.0 due to consolidation of loose material
WET CONDITIONS	**WET CONDITIONS**
Before pre-grouting *Q = 30/9 x 1/2 x 0.5/1 = 0.8*	*Before pre-grouting* Q = 30/12 x 1.5/4 x 0.66/2.5 = 0.2
Vp ≈ 3.4 km/s E_{mass} *≈ 9.3 GPa* *K ≈ 1.3 x* 10^{-7} m/s **10 m Tunnel: B 1.6 m c/c, S(fr) 10 cm**	*Vp ≈ 2.8 km/s* E_{mass} *≈ 5.8 GPa* *K ≈ 5.0 x* 10^{-7} m/s **10 m Tunnel: B 1.4 m c/c, S(fr) 13 cm**
DRY CONDITIONS	**DRY CONDITIONS**
After pre-grouting **Q = 50/6 x 2/1 x 1/1 = 17**	***After pre-grouting*** **Q = 70/4 x 2/1 x 1/1 = 35**
Vp ≈ 4.7 km/s E_{mass} ***≈ 25.7 GPa*** ***K ≈ 5.9 x*** 10^{-9} ***m/s*** **10 m Tunnel: B 2.4 m c/c**	***Vp ≈ 5.0 km/s*** E_{mass} ***≈ 32.7 GPa*** ***K ≈ 2.9 x*** 10^{-9} ***m/s*** **10 m Tunnel: sb (spot bolts)**

7 CONCLUSIONS

1. Tunnelling in jointed rock causes stress redistribution in the intact blocks and deformation of the joints through slight shear, opening and closure in decending magnitudes from mm to µm. We do not call this 'plastic behaviour' just because the combination GSI, H-B, RS2-FEM creates an illusion of continuous behaviour. The popular method is based on too many assumptions, including the incorrect simultaneous mobilization of 'c' and 'φ' in page-wide opaque equations that lack joint sets, roughness, clay-filling, and water. EDZ 1 and EDZ 2 are preferred images, and EDZ 2W in case of coupled H-M modelling with UDEC-BB and water inflow to the excavation(s).
2. Single-shell tunnelling using B+S(fr) with quantities based on the use of the Q-system has been termed NMT for the last 30 years, and was 'launched' as an acronym by several tunnelling companies in Norway, as an alternative to more costly and time consuming double-shell NATM.
3. Because of the frequent tendency of jointed rock (with UCS from 4 to 400MPa) to suffer overbreak, especially when drill-and-blasted and also when TBM bored (as in some difficult kilometers in the Channel Tunnels), there will be a large discrepancy in volumes of shotcrete and concrete used in NMT and NATM. In the former, overbreak is not filled.
4. When conditions are demanding and lattice girders are relied upon as part of the temporary support as in the case of NATM, there is a more secure alternative in NMT termed RRS or rib-reinforced shotcrete arches. These are bolted systematically and by design are in intimate contact with the tunnel periphery. The ability to bolt is assumed. Revert to NATM if almost soil.
5. When pre-injection is needed to ensure dry tunnels and protection of the outer environment, do not let designers 'select' grout using the filter pump which disqualifies the best stable grouts due to extension viscocity. Grout flows in rough interlocking planes, not through 3D 'toll-booths'. Allow high pressure pre-injection: more than 50% is lost 1m from each borehole when flowing.

8 REFERENCES

[1] BARTON, N.; WARREN, C. Rock mass classification of chalk marl in the UK Channel Tunnels using Q. Ch. 18. *Soft Rocks Mechanics and Engineering*. 2019. Eds. Kanji, He, & Sousa. Springer.

[2] BARTON, N.; QUADROS, E. 2019. Understanding the need for pre-injection from permeability measurements: what is the connection? J. Rock Mech. And Geotech. Eng. Vol. 11, Issue 3, June 2019, 576-597.

[3] BARTON, N. TBM Tunnelling in Jointed and Faulted Rock. 2000, 173p. Balkema, Rotterdam.

[4] BARTON, N.; BY, T.L.; CHRYSSANTHAKIS, P.; TUNBRIDGE, L.; KRISTIANSEN, J.; LØSET, F.; BHASIN, R.K.; WESTERDAHL, H.; VIK, G. Predicted and measured performance of the 62m span Norwegian Olympic Ice Hockey Cavern at Gjøvik. Int. J. Rock Mech, Min. Sci. & Geomech. Abstr. 31:6: 617-641. 1994. Pergamon.

[5] BARTON, N.; QUADROS, E. Some lessons from single-shell Q-supported headrace and pressure tunnels. ISRM, Eurock, Trondheim, 2020, OnePetro, 8p.

[6] BARTON, N.; GRIMSTAD, E.; AAS, G.; OPSAHL, O.A.; BAKKEN, A.; PEDERSEN, L.; JOHANSEN, E.D. Norwegian Method of Tunnelling. WT Focus on Norway, World Tunnelling, June: 6p. August: 5p. 1992.

[7] BARTON, N.; GRIMSTAD, E. Tunnel and cavern support selection in Norway, based on rock mass classification with the Q-system. Norwegian Tunnelling Society, NFF, 2014. Publ. 23, 39p.

[8] BARTON, N.; GAMMELSÆTER, B. Application of the Q-system and Q_{TBM} prognosis to predict TBM tunnelling potential for the planned Oslo-Ski rail tunnels. *Nordic Rock Mechanics Conf*., Kongsberg, Norway, 2010.

[9] VANDEVALL, M. Dramix - Tunnelling the World. NV Bækert S.A, 1990.

[10] BARTON, N.; LIEN, R.; LUNDE, J. Engineering classification of rock masses for the design of tunnel support. Rock Mechanics. 6: 4: 189-236, 1974.

[11] BARTON, N.; BUEN, B.; ROALD, S. Strengthening the case for grouting Tunnels & Tunnelling International, Dec. 2001: 34-36, and Jan. 2002: 37-39, 2001.

[12] BARTON, N.Barton. N. Rock mass characterization for excavations in mining and civil engineering. Proc. of *Int. Workshop on Rock Mass Classification in Mining*, Vancouver, 2007a.

[13] BARTON, N. Future directions for rock mass classification and characterization – Towards a cross-disciplinary approach. Invited lecture. Proc. of *1st US-Canada Rock Mech. Symp*., 179-188. Vancouver, 2007b.

[14] KVELDSVIK, V.; KARLSRUD, K. Support and water control in Oslo. World Tunnelling, Vol. 8, 167-171, 1995.

[15] BARTON, N. Minimizing the Use of Concrete in Tunnels and Caverns – Comparing NATM and NMT. Keynote lecture, GeoMEast International Conference, *Sustainable Civil Infrastructures (SCI): Innovative Infrastructure Geotechnology*, Sharm El-Sheikh, Egypt, 36p, 2017.

[16] QUADROS, E. F.; CORREA FILHO, D. Grouting efficiency using directional (3-D) hydraulic tests in Pirapora Dam, Brazil. Proc. 8th ISRM congress, Tokyo. Fujii, ed., 823-826, 1995.

[17] ZOLFAGHARI, A.; SOHRABI BIDAR, A.; MALEKI JAVAN, M.R.; HAFTANI, M.; MEHINRAD, A. Evaluation of rock mass improvement due to cement grouting by Q-system at Bakhtiary dam site. Int. J. Rock Mech. & Min. Sci. 74, 38-44, 2015.

[18] BHASIN R,; JOHANSEN P.M.; BARTON, N.; MAKURAT, A. Rock joint sealing experiments using an ultra fine cement grout. Proc. N. American Tunnelling Conf., Seattle, Ed. Ozdemir, 2002.

[19] ROALD, S.; SAASEN, A. Penetrasjon i fjell for ulike sementbaserte injeksjonsbruk. 2004. *(Penetration of various cement-based injection materials in rock).*

[20] BARTON, N. Rock Quality, Seismic Velocity, Attenuation and Anisotropy. Taylor & Francis, UK & Netherlands, 729 p. 2006.

[21] Barton, N. An Engineering Assessment of Pre-Injection in Tunnelling. Keynote lecture. Indian Conf. on Rock Mechanics and Tunnelling. Roorkee, India, 2011.

[22] BARTON, N. Future directions for rock mass classification and characterization – Towards a cross-disciplinary approach. Invited lecture. Proc. of *1st US-Canada Rock Mech. Symp*., 179-188. Vancouver, 2007b.

[23] BRANTBERGER, M.; DALMALM, T.; ERIKSSON, M.; STILLE, H. Controlling grout spread in tunnel grouting – Analyses and Developments of the GIN-method. Tunnelling and Underground Space Technology. Vol 15, No 4, 343-352, 1998.