International Society for Rock Mechanics and Rock Engineering
Norwegian Group for Rock Mechanics

ISRM International Symposium
Eurock 2020 – Hard Rock Engineering
Trondheim, Norway, 14-19 June

Some Lessons from single-shell Q-supported headrace and pressure tunnels

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Abstract

Development of the Q-system has meant engagement in water transfer tunnels, hydropower headrace and pressure tunnels in many countries since 1974. The support requirements of single-shell tunnels, were initially dominated by Norwegian and Swedish hydropower projects. The Q-system data base was greatly expanded later, by Grimstad's incorporation of steel fiber reinforced shotcrete S(fr).

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. Interesting controversies arise in occasional hearings and court cases. One side may demand concrete-lined tunnels, the other defends 'nominally-unlined,' with Q-system based support and reinforcement where needed. Once the question 'what about rocks in the turbines?' was even heard.

Empirical *a posteriori* experience, related to the theoretical laminar-flow paraboloidal 3D velocity distribution, and a glance at the Hjulström-Sundborg river-erosion diagram, should convince the wise designer that flow velocities need to be limited to about 1.5 to 2.5m/s so that no fallen rock blocks ever reach the 'rock trap', which will likely contain silt and sand and perhaps floating pumice, when a tunnel system is emptied for inspection and maintenance. Too high flow velocities in lightly supported river diversion tunnels, with too thin shotcrete, have on occasion had dramatic consequences.

Remembering the *a posteriori* origin of the Q-system it is wise for numerical modellers to think twice before proposing 'longer rock bolts'. Claims about deep 'plastic' zones when analysis methods are full of *a priori* assumptions and alarming opaque equations devoid of joint sets, inevitably fail to convince. Unavoidable overbreak caused by high Jn/Jr ratios, and full-scale roughness losses, will also be briefly addressed. Minimum rock stress greater than water pressure is of course fundamental as well.

Keywords

Headrace tunnels, flow velocities, shotcrete, roughness, head-losses

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1 The single-shell alternative

1.1 Some background controversies

The development of the Q-system for assisting in the design of single-shell (shotcrete and rock bolts or unlined) tunnels and caverns dates back to 1974, with the initial use of B+S(mr) or systematically bolted and mesh reinforced shotcrete linings, based on case records of this period. There were also cases of unreinforced shotcrete, and no support at all, each interpreted from Q-values, span and tunnel use (Barton et al. 1974). Since this early period there was the remarkable development of steel-fiber reinforced sprayed concrete, seen by the first author in a Norwegian hydropower cavern in 1979, and used systematically in a Norwegian road tunnel by 1981 (pers. comm. Eystein Grimstad). The S(fr) development represented a paradigm-shift in tunnel support regarding safety, speed and economy. Yet there are still those in doubt that sprayed reinforced concrete is reliable. This can be due to cost-cutting on the necessary additives, such as micro-silica, and insufficient attention to cleaning the rock.

Even in recent times one may encounter consultants who propose mesh-reinforced shotcrete when there is over-break (their assumption of reduced tunnel stability?) while S(fr) 'may be used' when there is no over-break. This remarkable reversal of logic emphasises that one should not be too surprised either, when S(fr) is considered by some to be merely temporary support, or when pre-grouting is also considered as a temporary measure, despite the several millions of kilometres of grout curtains below thousands of the world's larger dams. Strange opinions also include an occasional and remarkable: 'what about rocks in the turbines' heard when traditional concrete-lined tunnellers are confronted with the 'nominally unlined' B+S(fr) or unlined alternative. It has to be said that Norway's 99% hydropower electricity supply (when not trading fossil fuelled seasonal alternatives in Europe) would be in dire straits, if the safe zone of water flow velocities (usually below 2m/sec) had not been solved at least 50 to 60 years ago.

1.2 Rock traps are for sand and silt

Even 3m/s water flow velocities in relatively smooth TBM bored headrace tunnels does not result in the transport of fallen pieces of rock, nor slabs of rock, nor de-bonded shotcrete to the mis-named 'rock trap'. The key of course is the parabolic (in 2D) or paraboloidal (in 3D) velocity profile when laminar flow is occurring. Pieces of jointed rock that fall, often during the drainage-for-inspection process, do not get transported, and the simplified, preliminary reasons can be seen in Figures 1 and 2.

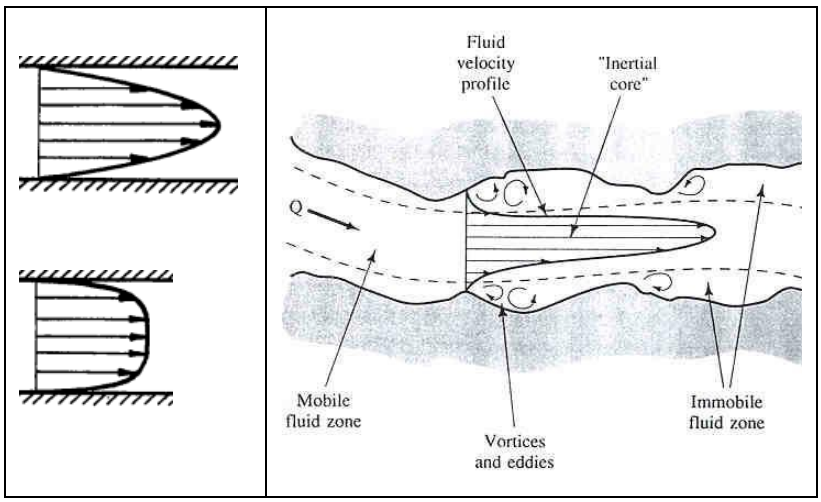

Fig.1 Left: Laminar (parabolic in two dimensions) and turbulent flow are contrasted. The more comprehensive diagram attributed to Schlichting (1960) suggests most convincingly why fallen or eroded rock blocks or de-bonded shotcrete do not get transported towards the mis-named 'rock trap' if the so-called viscous sub-layer remains (cylindrically) thick enough.

Fig. 2 is a demonstration (by Liu 2001) of why, in the tunnel context, rough stones/blocks on the floor of a TBM tunnel may not be picked up until greater than e.g. 4 or 5m/s flow occurs. For a rough-invert drill-and-blast tunnel, the boundary (next-to-the-wall) velocity will be higher in exactly the region where not desired, and loosened stones/stabs/wedges may get picked up and transported, especially if the velocity reaches 8 to 10m/s. This was the unfortunate case of a large river diversion tunnel that suffered severe damage, and a huge erosion/collapse cone up to the surface, eventually compromising the whole power plant due to a cascade of hydraulic problems. It is very important to be aware that Q-system based tunnel support, which was originally based on numerous hydropower projects, was not

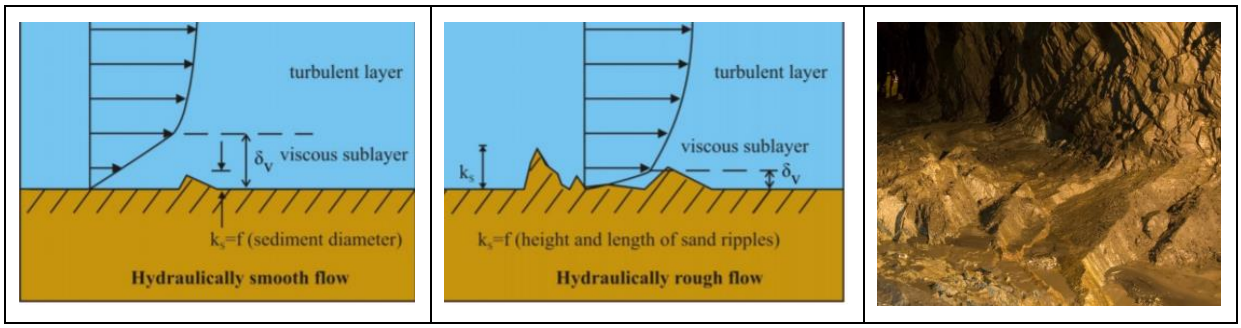

Fig. 2. If the flow velocity is too high, as in some river diversion tunnels, then higher than desirable velocities will occur too close to the tunnel periphery and increase the risk of erosion and damage of the shotcrete, and the possibility of progressive 'plucking' of the rough invert and lower walls, which may not be shotcreted. This has led to general tunnel failure and major collapse on rare occasions. It is easily avoided by conservative and complete lining and reinforcement, including the invert. (Modified, after Liu, 2001).

designed to resist (river diversion) flow velocities many times higher than the typical 1.5 to 2.5m/s of drill-and-blasted single-shell headrace tunnels. An 8.4km long TBM-driven tunnel (see Fig. 3) that was inspected following emptying, had a theoretical cross-section of 38.5 m^2, giving a maximum water flow velocity of 3.0 m/s, while the initial drill-and-blasted tunnel of some 1.5km length had a cross-section of 43.7 m^2, and a maximum flow velocity of 2.6 m/s. In *neither case* was transport of fallen stones, slabs or larger blocks noted (such as illustrated in Fig. 4), and even gravel-sized particles of several mm diameter did not reach the rock trap.

Compared with an undulating river bed and probably *smoothed* stones and pebbles, whose behaviour will likely follow the empirical Hjulström diagram (Fig. 5), it appears that the *rough particles* representing fallen rock pieces from the walls of 'nominally unlined' tunnels do not get transported along the rough invert of a drill-and-blasted tunnel, nor along the much smoother TBM driven tunnel. This particular TBM tunnel in massive schist was left largely unlined due to high Q-values. There were nevertheless sections with more jointing (Fig. 4) and significant numbers of joint-delineated stones and slabs had fallen during several years of operation, and some during the dewatering itself.

Fig. 3. An 8.4km long TBM and 1.5km long D+B headrace tunnel, photographed during inspection. The double-shield TBM was used to assemble pre-cast element liner on thirteen occasions where rock mass conditions were unfavourable. Most of the TBM tunnel was unsupported. Transport of fallen blocks did not occur, even along the long, smooth unlined parts of the TBM tunnel. The 'rock trap' contained silt and sand and (floated-in) pumice. Photograph by Dr. Nghia Trinh, SINTEF.

The photographs in Fig. 4 show some of the accumulated rock blocks/stones that had fallen or were scaled in the first 4 km of this 8.4km long TBM headrace tunnel. A lot of the block sides were planar

Fig. 4. Joint delineated blocks of rock that caused minor over-break and fell to the invert at intervals have beencollected from the TBM-driven and drill-and-blast sections of the headrace tunnel, following emptying for inspection and maintenance.

joint surfaces. These block-falls were caused by joint intersections or by occasional stress-or-extension slabbing as depths were up to several hundreds of meters. The fallen stones were probably plucked from the walls due to the 2.5m/s to 3.0m/s flow velocities. In a later emptying of the same tunnel, the total was 8 fallen rock-blocks in the same first 4 km, and 20 blocks in total in the 8.4 km long TBM tunnel. The key message provided by all these fallen blocks is that they are not transported to the 'rock-trap'. They may be loosened by 2.5m/s to 3m/s flow, but they do not get transported, and they in no way affect head loss or power production. These mostly water-in-rock conduits (we have more than 4,000km of these in Norway) represent a very economic method of hydropower development, and the good economy remains even when lower Q-values, or deliberately conservative design, results in significant quantities of B + S(fr).

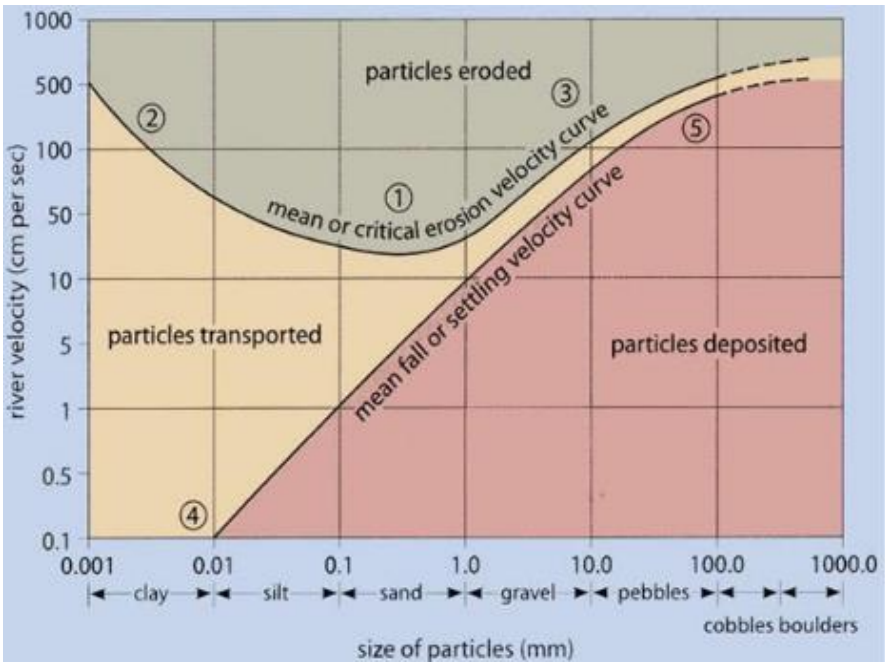

Fig. 5. The river erosion based Hjulström-Sundborg diagram appears to show a threat to 'blocks in the rock trap' (and perhaps later 'in the turbines'!) only when velocity reaches about 5m/s for the case of 100mm stones. Note that this river-based diagram is being recruited to give approximate guidance for deeper flows in headrace tunnels which may (also) have a free surface. Note that a typical headrace tunnel velocity of 2m/s is shown capable of eroding and moving approx. 20-30mm size gravel, which presumably would be partially smoothed in the case of river-bed transport.

In approximate terms, these river-sediment transport data also demonstrate why conservative velocities are used in 'nominally unlined' hydropower tunnels, where the desire is to run a plant for as many years as possible (say a typical 15 years in Norway), without having to empty 'the rock trap'. At 1 to 2m/s, and with the inevitable invert roughness, there is only sand and silt in these traps. (Pumice that has floated in at the top of the column of water, may be discovered when emptying, in the case of plants where such volcanic debris is eroded by rivers upstream of the reservoirs).

1.3 Tunnel support with water retention and drainage ability

When using a tunnel in rock as a large conduit for water, whether for water supply or delivery of water to the high pressure end of a hydropower project, it is clearly necessary to rely on the combined ability of the surrounding rock mass to more or less retain the water, *and* to allow drainage for occasional

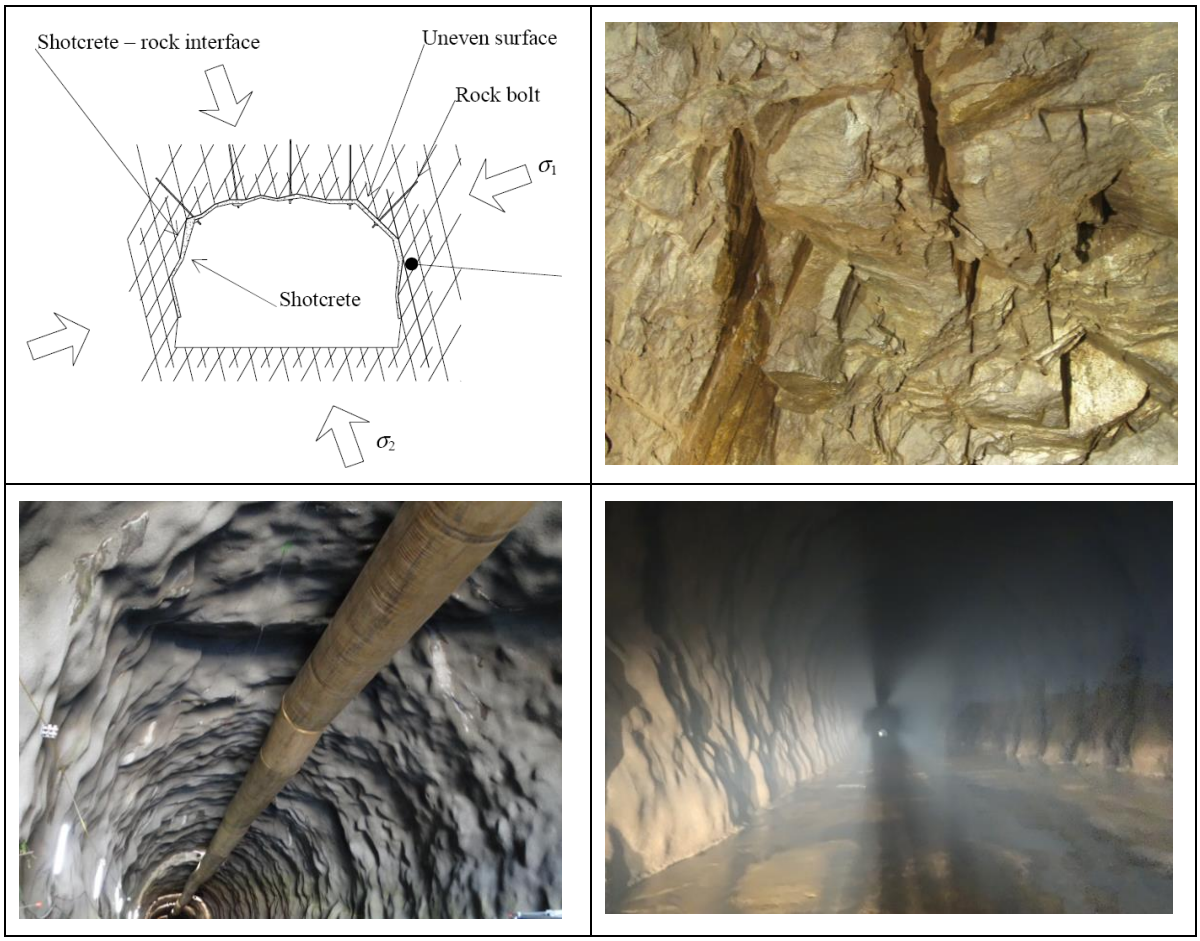

Fig. 6. Alternative single-shell tunnel liners. Top-left: typical B+S(fr) support with the traditionally reduced thickness of shotcrete in the walls when Q-values and rock strengths are high enough (Barton et al. 1974, Malmgren 2005). Top-right: unlined headrace tunnel section with sufficiently low rock mass permeability (schist at several hundreds of meters depth). Such tunnels can be drained at rates of 5m/hour without problems. Bottom-left and right: conservative application of S(fr) down to RRC invert. When a headrace tunnel with an almost 100% cover of S(fr) is used, slow drainage rates (< 1m/hr) are needed, and drainage holes in the invert are required. There will be some inevitable drainage along the invert-wall joint.

inspection and maintenance. The retention part of this balancing act may depend on a suitably high surrounding joint-water pressure, or in its absence, a suitably low rock mass permeability, or use of an invert-wall-arch-wall-invert shotcrete (or concrete) lining, with suitable impermeabilization of the invert using for instance roller-compacted concrete (RCC). The nominally unlined D-shaped section with enlarged cross-section to prevent greater head-loss, has very clear economic advantages, and has indirectly influenced the development of the single-shell B+S(fr) Q-system support recommendations, as the expensive alternative of *concrete lining* is only applied/recommended where rock mass conditions dictate its use (i.e. for Q = 0.01 or worse) – or of course if exceptionally high flow velocities are to be passed in the case of shorter-term river diversion.

It is important to admit, as developer of the Q-system, that the case record data base concerning hydropower, was *principally hard rock* rather than soft rock based (despite an initial 50 rock types in the first 200 case records) and consequently was reflecting the ability of many rock masses (many from Scandinavia) to function well when surrounding water conducting tunnels, without the need for the continuous lining (impermeable Sfr) or local reinforced cast concrete, as when water sensitive or soft rock is involved. Here one must act conservatively and visualize the future use of the given tunnel for conducting water, and use appropriately conservative Jw ratings, having regard for the need to tolerate drainage when the tunnel needs occasional inspection. Erosion must also be prevented during normal low-velocity operation. The Q-system is specifically designed for significant thickness of S(fr) when clay-fillings are present. This does not mean that one resorts to an expensive concrete lining, but rather that the S(fr) that is applied is continuous and sufficiently thick, for instance 5cm as a minimum.

Study of the Q-system support charts of 1993 (Grimstad, Barton 1993, Barton, Grimstad 2014) shown in Figure 7a and b, indicates that there has been a degree of increased conservatism in the 'fair' 'good', 'very good' region, with Q from 4 to 100, with minimum thickness increased from 4 to 5cm, which helps to improve the near-impermeabilization, and helps the curing conditions for the shotcrete,

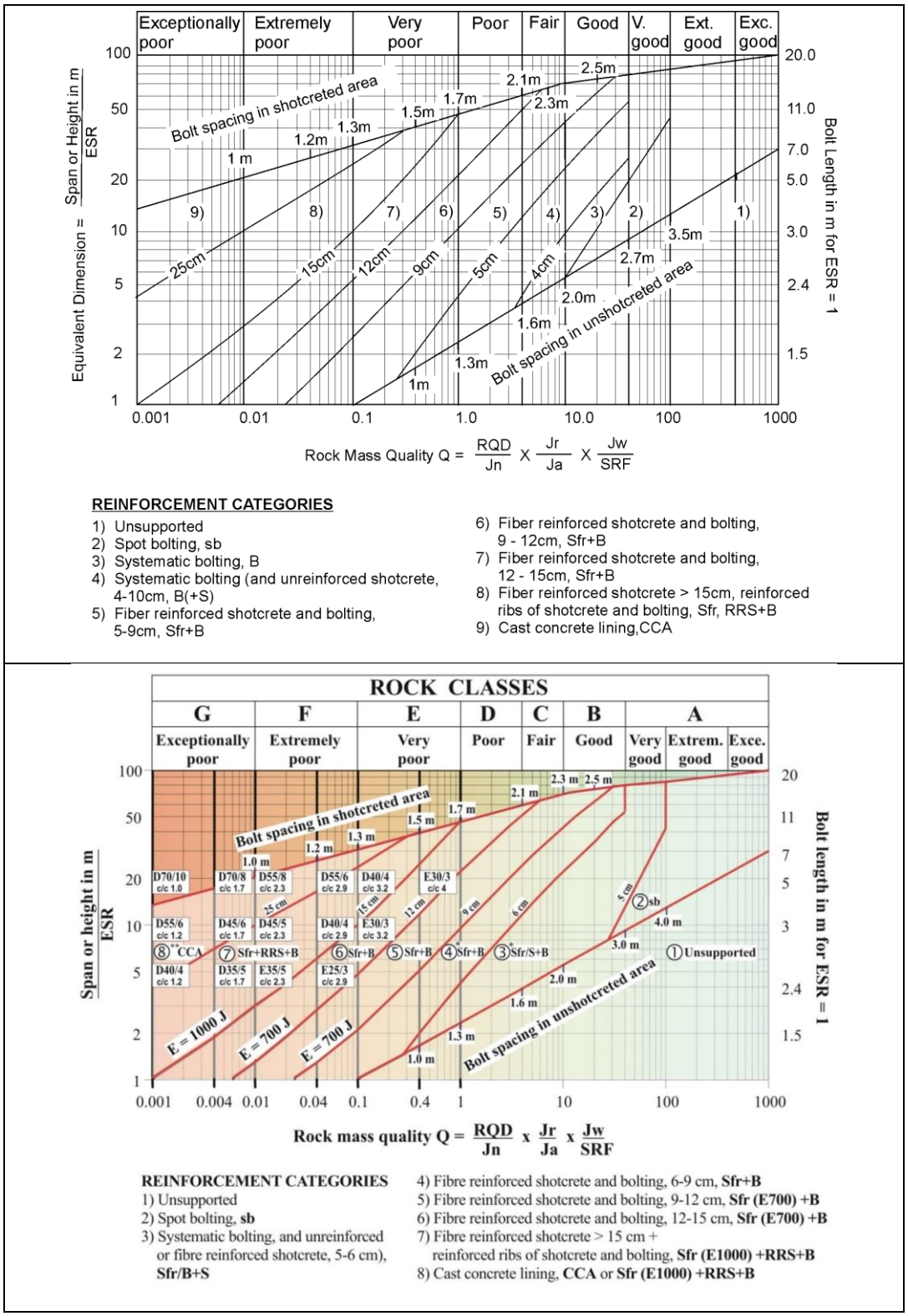

Fig. 7a and b. Q-system based tunnel (and cavern) support using systematic bolting and fiber-reinforced shotcrete combinations of B+S(fr). (Grimstad, Barton, 1993 and Barton, Grimstad, 2014). Note that S(fr) has extremely low permeability when used on high modulus rock masses. Deformations may be of millimeter scale during excavation and may remain in the few millimetre class even when conducting water at high pressure. **Note 1**: The central trend of hundreds of deformation recordings is: Δ mm ≈ SPAN (m)/Q, for instance 1mm = 10/10. This simple empiricism can outperform UDEC-BB analyses, if modellers have exaggerated the continuity of the jointing, as is frequent. **Note 2**: On occasion, modellers using continuum codes and opaque strength criteria, in effect adding c and $\sigma_n \tan \varphi$ in linear or non-linear forms, have suggested both plastic behaviour where it does not exist (as verified in a court case) and the perceived 'need' of increasing the length of rock bolts in relation to Q-system *a posteriori* empiricism. (L ≈ 2 + 0.15 SPAN/ESR m). Greater circumspection is needed when the analyses and parametric estimates are subject to seriously suspect and largely opaque *a priori* methods.

as compared with too thin application and premature drying. For Q-values below 1 ('very poor') the spacing of the bolts and the thickness of the shotcrete remains, and in the case of unstable conditions the provision of rib-reinforced shotcrete is specified, following the design suggestions of Grimstad. Here there will need to be consideration of roughness losses, with the deliberate over-excavation and

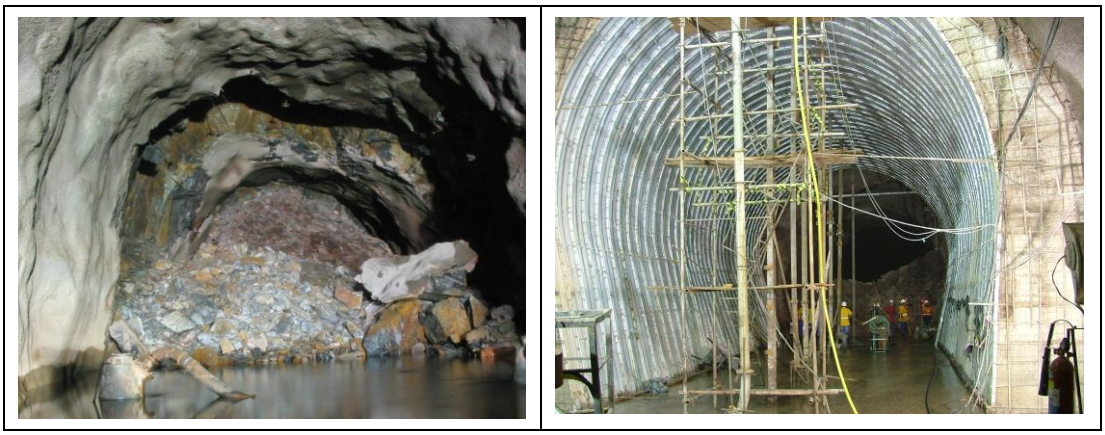

Fig. 8. A serious fault zone with swelling clay (montmorillonite) demanded a robust repair, but the serious reduction in cross-section as illustrated needs to be avoided when possible. There were several instances like this in a renowned tailrace tunnel.

smoothing between ribs, or even local cast concrete, in the case of very low, fault-related Q-values. One needs to avoid a drastic reduction in cross-section, seen in the before-and-after sequence in Fig. 8.

2 Over-break, roughness, head loss

An important consideration for headrace and water transfer tunnels is of course the head-loss one will experience due to the selection of a drill-and-blast construction and single-shell support. An early assessment in this respect is the degree of over-break (i.e. large-scale roughness) one is likely to experience, due to the nature of the anticipated rock masses along the planned route of the tunnel.

Fig. 9 illustrates the interaction of two of the Q-parameters in particular: Jn and Jr. Some years ago Barton (2007) 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. Clearly if a Q-parameter statistic (histogram) analysis based on drill-core logging indicates a predominance of Jn/Jr ≥ 6, this will influence the likely large-scale roughness and therefore potential head loss, and/or the need for a larger volume of 'smoothing' shotcrete. In particular, it would indicate the need for protection of the invert (not just the arch and walls) in the case of planned high-velocity river diversion.

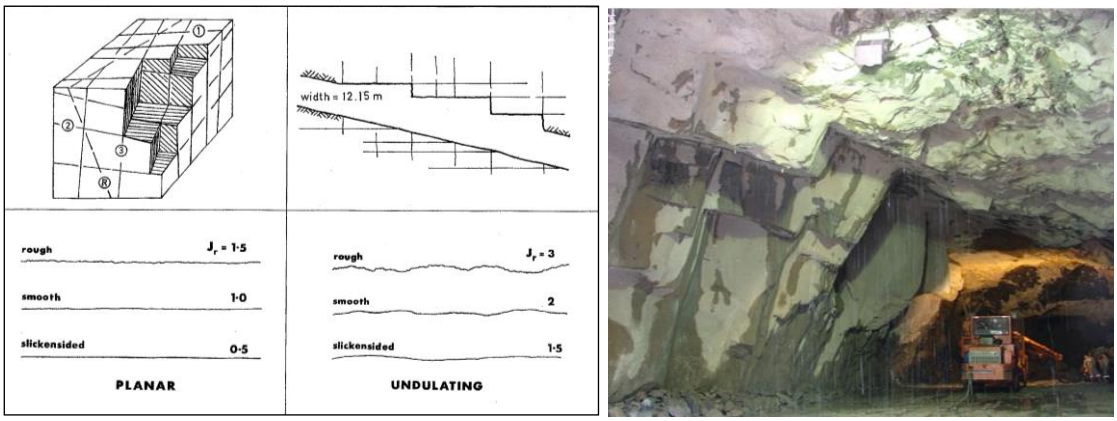

Fig. 9. When there are sufficient joint sets to define blocks, together with the presence of the excavation for stress release, then the roughness of the principal joint sets will play an important role in determining if over-break occurs, or does not, because Jr = 3. A recent case: Jn =15 (four sets), three of them steeply dipping, and Jr = 1.5. Over-break was inevitable.

In Norway, extensive studies of the Manning number (M) and/or the Darcy-Weisbach friction factor (f) have been performed for rough unlined tunnels, also for partly shotcrete lined tunnels, and for asphalted floors and for TBM tunnels, both with and without some amounts of overbreak. The Norwegian Hydrotechnical Laboratory, under the guidance of senior research engineer Øivind Solvik was particularly active in these studies. A brief summary follows of some of their key findings.

In the more than 4,000 km of Norwegian hydropower tunnels, the choice over the last 40 years or so has been between drill-and-blast tunnels of larger cross-section, or TBM tunnels of smaller cross-section. Solvik (1993) has given the following comparison of alternative areas and methods of excavation (with or without their lining) that give equal head loss per km of tunnel:

unlined (drill-and-blast) 30 m^2	TBM tunnel (unlined) 18 m^2
shotcrete lined (all around) 20 m^2	concrete lined (all around) 15 m^2

On average, only 4 to 5% of Norwegian hydropower tunnels have local areas of lining [S(fr), cast concrete, in response to low Q-values]. Often these linings are only for lengths of 10 to 20 m, basically covering fault zones. Other areas have partial shotcreting and bolting, applied for reasons of stability, not for reducing roughness. The unlined water tunnel, even with its 50% (or more) increase in area is still much less expensive than a lined one with equal head loss. Consequently, much is known about the friction factor and Manning's number of these rough-walled, drill-and-blasted tunnels.

The head loss measurements in a large number of D+B unlined tunnels indicated a Manning number M varying from 26 to 38, with reduced effect of roughness as size was increased, up to the point where larger overbreak in the largest tunnels tended to reduce M (i.e. more head-loss) due to the greater absolute roughness. Head loss (h_l) according to Manning and Darcy-Weisbach were as follows, using M and f respectively. (V = velocity of flow – often 1 to 1.5 m/s, L = length of tunnel):

Table 1. Manning and Darcy-Weisbach equations for head-loss, and their inter-relationship.

$h_l = \frac{V^2 \times L}{M^2 R_h^{4/3}}$	$h_l = f \times \frac{L}{4R_h} \times \frac{V^2}{2g}$	Conversion between M and f is given by the equations on the right-hand side	$M = \left(\frac{89}{f \times R_h^{1/3}}\right)^{1/2}$	$f = \frac{89}{M^2 R_h^{1/3}}$

For standard horseshoe-shaped tunnels, R_h, the hydraulic radius ≈ 0.26 $A^{1/2}$, where R_h = area/wetted perimeter. The practical Manning range was about 26 to 66 for D+B (extreme roughness) and TBM, respectively. The Manning's factor for asphalt-paved tunnel inverts was not known before its introduction , and it had to be determined by experimentation. A reasonable range of the Manning's number M for asphalted inverts was found to be 65–75, and it depended on aggregate size and machine or manual rolling. That meant 25% head loss reduction if the alternative was to leave the invert unlined and completely cleaned. (The corresponding increased power production could exceed the total cost of the asphalt). If Manning's formula is used to calculate the head loss, the following formula should be used to calculate the *resulting* Manning's number M_R (=1/n) for a composite area. (Solvik, 1993).

Table 2. Composite Manning's number estimation when an asphalted invert is combined with unlined rock (Solvik 1993).

$M_R = \left(\frac{P}{\frac{P_a}{M_a^{3/2}} + \frac{P - P_a}{M_u^{3/2}}}\right)^{2/3}$	P = total periphery P_a= asphalt periphery M_a = Manning's number for asphalt M_u= Manning's number for unlined rock

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