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# NMT-NATM hybrid method for sustainability and risk reduction

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## Abstract

In 1992 a group of authors from Norwegian companies proposed the name NMT (the Norwegian Method of Tunnelling) as a distinctive alternative to the well-known NATM of Austria. In a nut-shell one can call NMT 'single-shell' while NATM can be called 'double-shell'. NMT is most appropriate for tunnel and cavern construction in medium to hard jointed rock. Support and reinforcement are guided by Q-system application. NATM with its final cast concrete lining can be considered most appropriate for lower quality, weathered and faulted rock.

Temporary support and reinforcement methods: shotcrete with mesh or fibre reinforcement, bolting and lattice girders (LG) are typically associated with NATM, prior to the drainage fleece, membrane and cast concrete. In the case of NMT the permanent bolting and shotcrete are deliberately chosen to be of high quality and corrosion protected and are both applied up to the tunnel face, with the final layer of shotcrete perhaps making a total thickness of 10cm also applied only a round or two behind the face. In case of poor rock conditions such as faulted rock, a more robust alternative to LG is used in NMT, called RRS (rib-reinforced shotcrete arches).These consist of systematically bolted arches of reinforcing bars typically 6 x 16mm diameter, welded to cross-bars and sprayed into the form of a thick local arch that may be repeated at 1.5 to 3m intervals. NMT may also have a final concrete lining ifthe rock mass quality is very poor such as $Q < 0.01$. In such cases cost and time involved will have approached that of NATM. Systematic high-pressure pre-injection is a frequent first stage of NMT. This radically reduces risk and actually also reduces support and reinforcement needs.

In this article a tentative NATM-NMT-NATM hybrid approach is suggested for a 'typical hard rock tunnel through a weathered hillside' as so frequently encountered for motorways in tropically weathered regions. Both ends of the tunnel would be NATM while a longer central NMT construction would save cost and time overall. Two case records will be described in which, due to unaccounted conditions, the NATM portions were over-loaded and LG folded due to high vertical loads. Risk reduction measures will be suggested. Due to overbreak effects on concrete volumes in NATM, greater lengths of NMT means greater sustainability as overbreak is not filled in the case of NMT.

## Keywords

Tunnelling, NMT, NATM, Lattice girders, Overbreak, RRS.

# 1 Introduction

## 1.1 NMT and the Gjøvik cavern connection

The six-companies and seven-authors article that suggested the letters 'NMT' in 1992, was based on synthesising what was increasingly being used in tunnels in Norway following 1974 Q-system development and later acceptance by our road and rail authorities in Norway. It was a two-part publication: Barton, N., Grimstad, E., Aas, G., Opsahl, O.A., Bakken, A., Pedersen, L. & Johansen, E.D. 1992. Norwegian Method of Tunnelling. WT Focus on Norway, World Tunnelling. The Norwegian companies involved were NGI, Selmer A/S, A/S Veidekke, Entreprenørservice A/S, NoTeBy and Statkraft. Some in Norway who were left out complained about NMT and Q for several more years. In fact the prominent contractors and designers of the ongoing Gjøvik cavern were our co-authors. The 62m span cavern is shown in Figure 1 in two contexts.

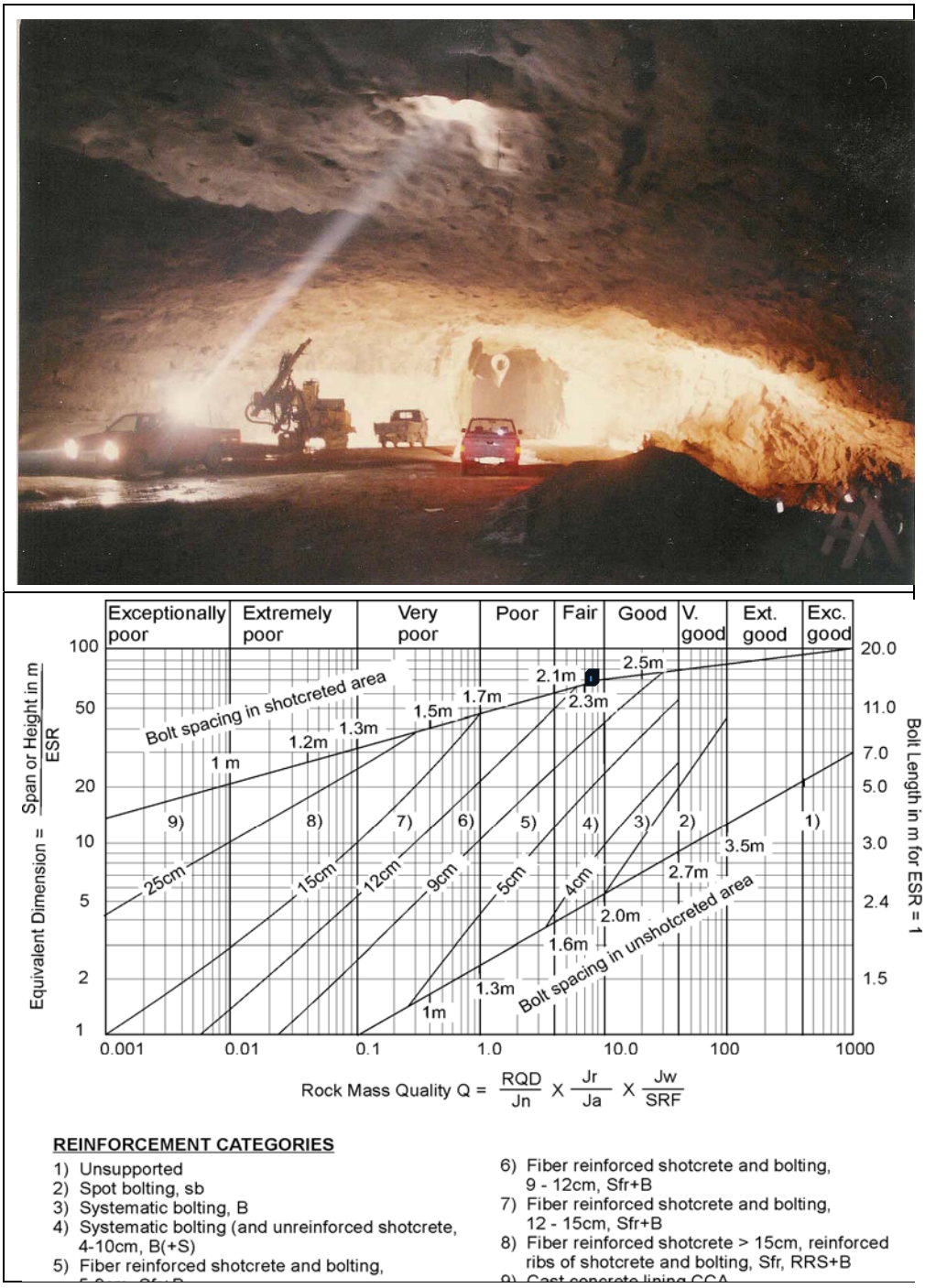



Fig. 1. The top-heading of the 62m span Gjøvik cavern with a mean Q-value of approx. 10, range 2 to 30. The dog-leg kink in the Grimstad and Barton (1993) Q-based support diagram was due to Gjøvik and its extreme span in rock of fair to good quality.

## 2 NMT components B plus S(fr) plus RRS

The components of NMT can be most conveniently illustrated by photographs and diagrams, as in Figure 2. On the left: S(fr), S(mr) – not favoured, and CT-bolts. RRS examples are shown on the right.

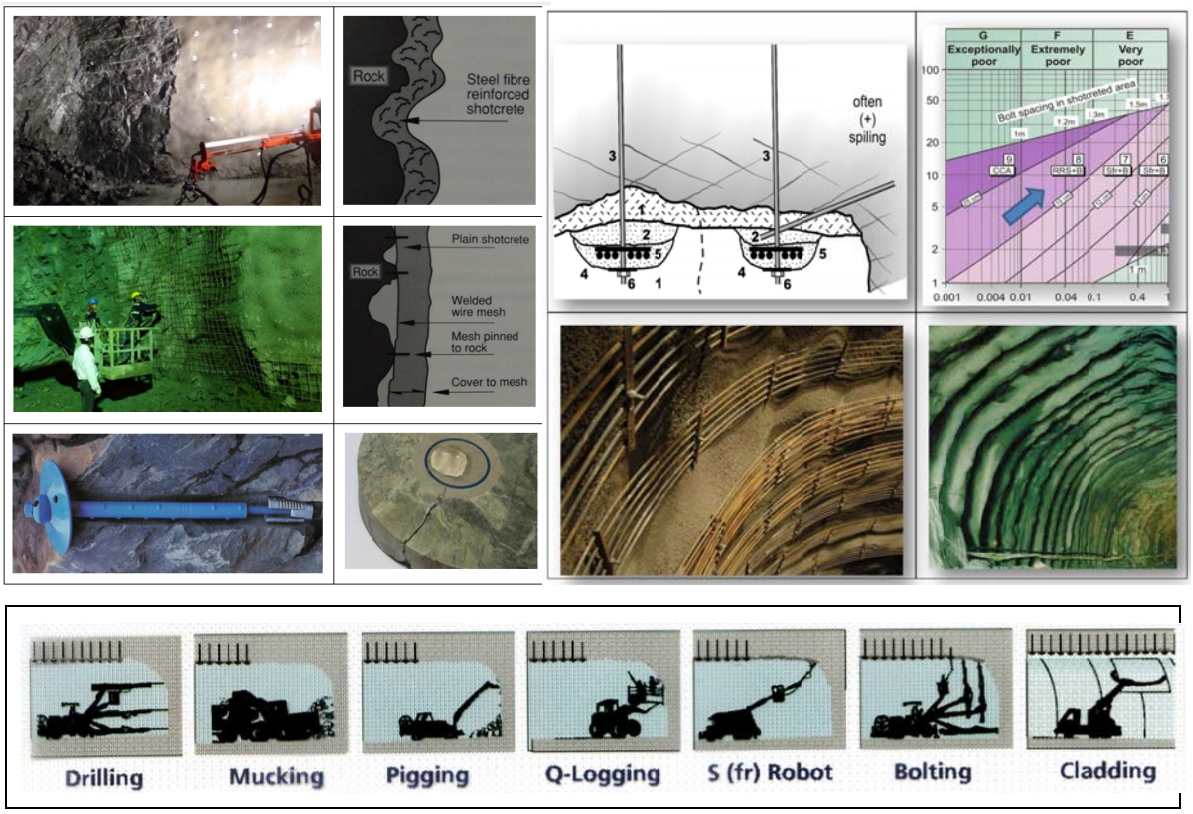



Fig.2 In the NMT method the clear preference since about 1980 has been S(fr) rather than S(mr) as this increases longevity. The graphic drawings of S(fr) and S(mr) (mid-left) are from Vandevall (1990). The longevity argument also applies to the CT-bolt, with the short blue demo photographed bottom-left. In the right-hand panels RRS is illustrated. Their design is detailed in Figure 3. Note the bending for architectural reasons. (National Theatre Station, Oslo). This 28m station cavern was constructed beneath a deep clay-filled depression. For convenience no overbreak is shown. In reality if there had been overbreak both bolting and shotcrete would follow the overbreak, which is not filled.

The PVC-sleeved CT-bolts are typically 3m up to 6m length. They have four layers of corrosion protection even after joint deformation: galvanizing, epoxy paint, outer annulus of grout (may be cracked due to joint deformation during the next tunnel advance), PVC sleeve, inner annulus of grout. The drilling, mucking, scaling, followed by Q-logging ('the owner's half hour'), S(fr) application by robot, and CT-bolting are illustrated in the lower diagram.

In the case of roads with heavy traffic PC-element cladding with outer membrane may be used. One may then be looking at a total of 10cm of shotcrete and 25cm of concrete, compared with obviously more in the case of NATM. This is because with NMT overbreak is not filled. The permanent reinforcement effect of bolting (CT-type) in NMT means that cast concrete lining is only used in exceptionally poor conditions (e.g. $Q < 0.01$).

In a later section of this paper the security of systematically bolted RRS arches in bad rock driven by NMT principles will be compared with the obviously faster application of lattice girders in NATM. The convenience of LG is appreciated but they are not recommended in the case of NMT for reasons that will become clear when considering their relative deformability. NMT principles are based on early application of permanent support and therefore small deformations. Only in major cavern construction will deformation monitoring be considered necessary. There were 7 pre-installed MPBX in the case of the Gjøvik cavern, and three shorter MBX installed in the top-heading.

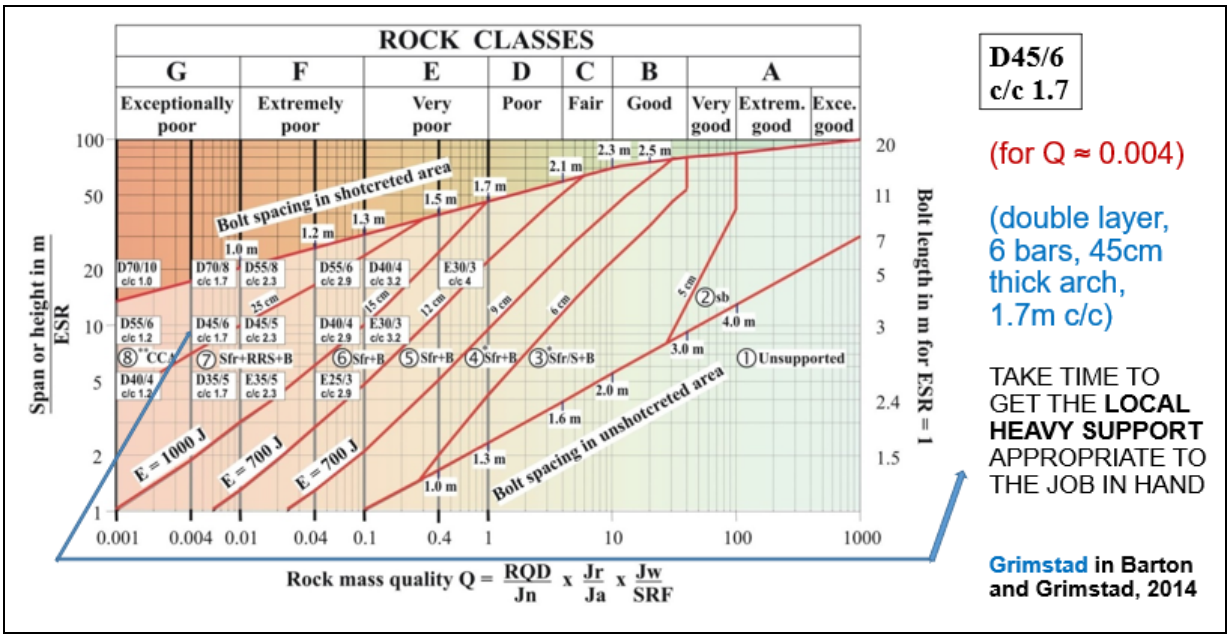



Fig. 3 The Q-system based design chart details, especially of RRS, from Grimstad.

## 3 NATM components

Figure 4 illustrates the various stages of NATM tunnel construction, with typically a top-heading and benching approach. It may be sequential split face in the case of soil-like conditions.

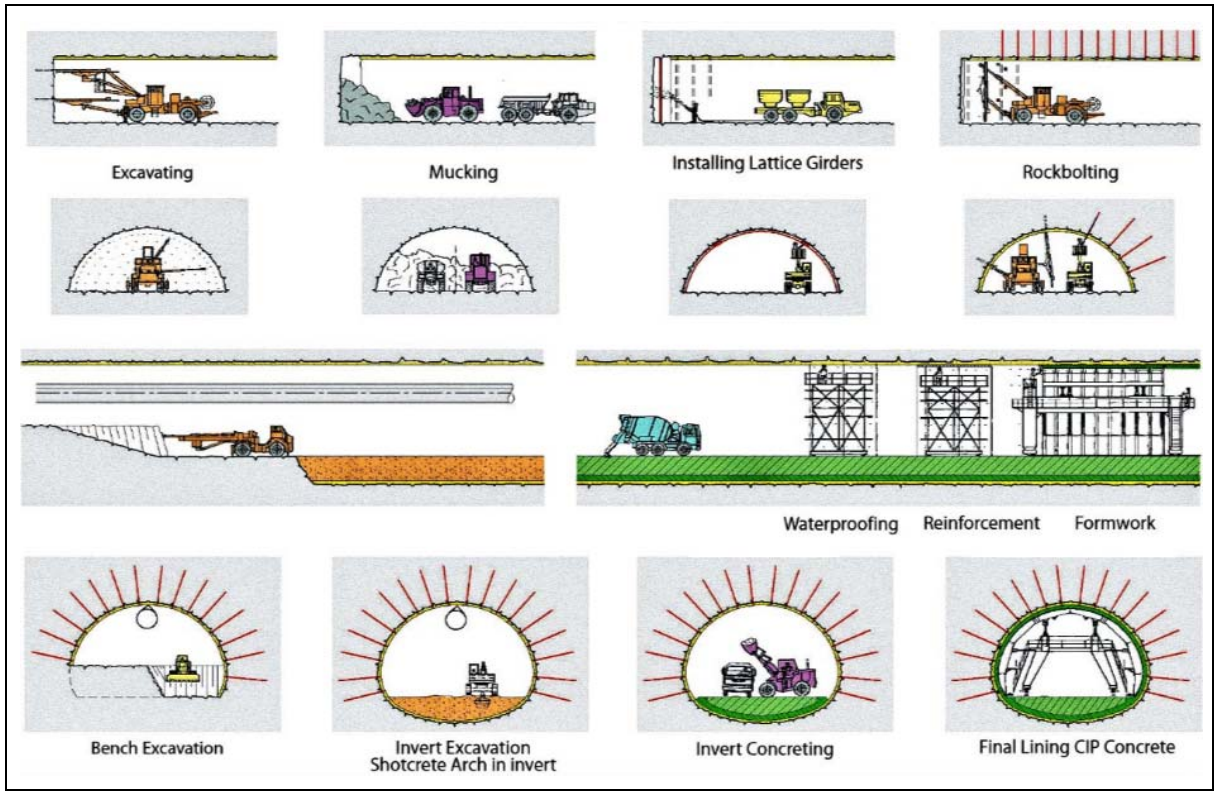



Fig. 4 Eight of the activities occurring with NATM tunnel construction when top-heading followed by benching are utilized. This is a coloured version of the official ASG (2010) report. Note no overbreak.

Note that overbreak has been ignored in the drawings but may severely affect shotcrete and concrete volumes in practice. In Figure 5, potential major overbreak has been sketched, and actual overbreak photographed. Readers may assume it has been exaggerated. However, 0.5m, 1m, 2m and even 3m are occasionally experienced in jointed rock. It depends on the ratio of Jn/Jr from the Q-system (Fig. 6).

# 4 Overbreak challenges

As with all tunnelling in jointed rock there is the problem of overbreak, and as will be clear from Figure 5 this is a minor problem with NMT as overbreak does not need to be filled, while for NATM it results in a potentially large volume of 'extra' shotcrete and concrete as the over-break has to be filled. Overbreak is also problematic for LG contact with the rock and may also present a '3D' challenge for drainage fleece and membrane installation prior to the concrete lining.

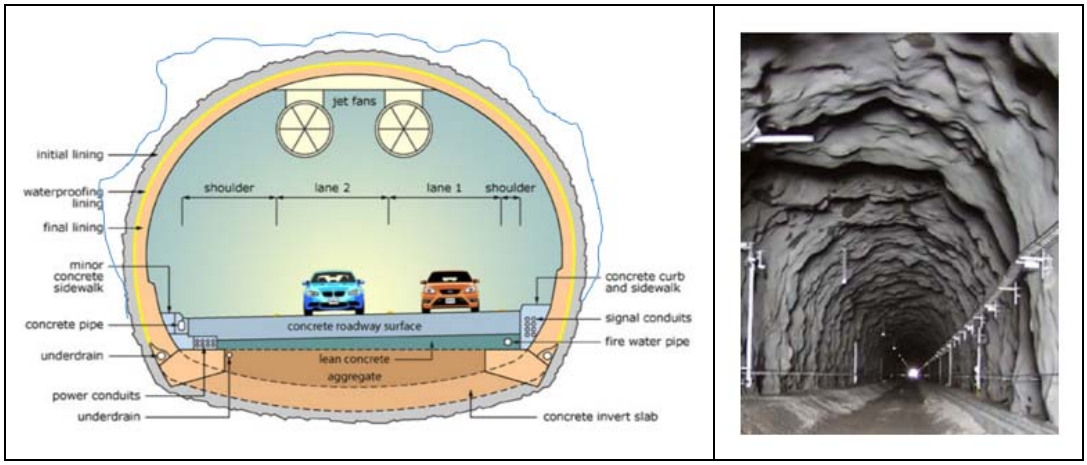



Fig. 5 A perhaps exaggerated addition of overbreak to an NATM tunnel (left), while overbreak >1m is clearly accepted in this Swedish rail tunnel where Q + S(fr) + B (= 'NMT' in practice) was used.

Figure 6 illustrates by means of a drawing (top-left), by means of joint roughness profiles (top-right) and by means of two photographs of significant overbreak with ratings for Jn/Jr = 9/1.5 why overbreak occurs when sufficient numbers of joint sets and sufficiently planar-surfaced jointing are present.

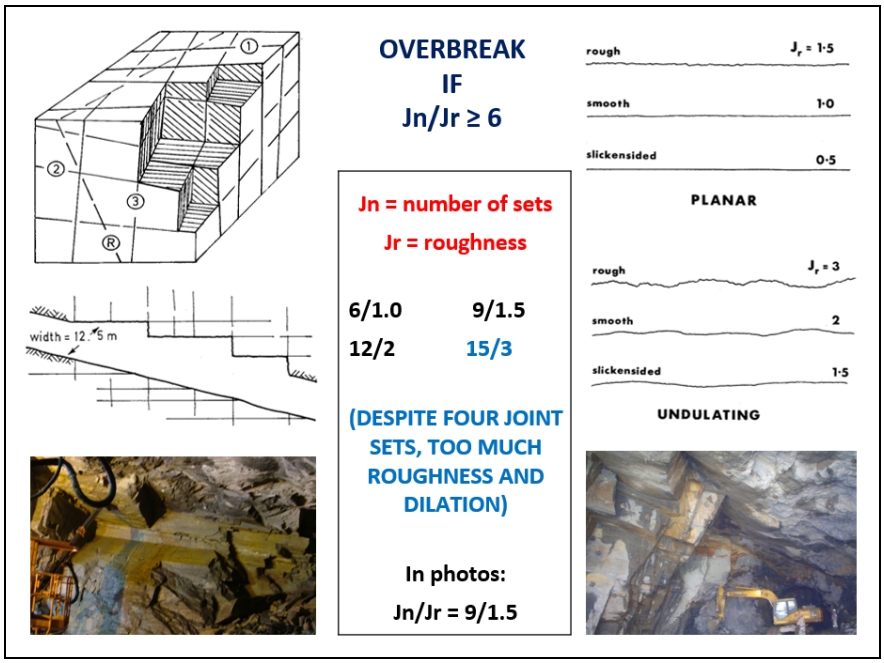



Fig. 6 A simple criterium for the likelihood of overbreak occurring in tunnelling. This influences cost (more shotcrete in NMT, more concrete in NATM) and therefore sustainability. The black ratios of Jn/Jr will be likely to suffer overbreak, but not the blue example as < 6.

In the case of mining, overbreak will affect drift and roadway stability with possible increases of shotcrete support, and in the context of underground stoping perhaps the threat of not only increased dilution but also blockage of draw-points. Figure 7 illustrates Q-histogram logging at the Kiruna Oscar project where long-hole drilling drifts were severely affected by overbreak and near failure as sketched in the central panel.

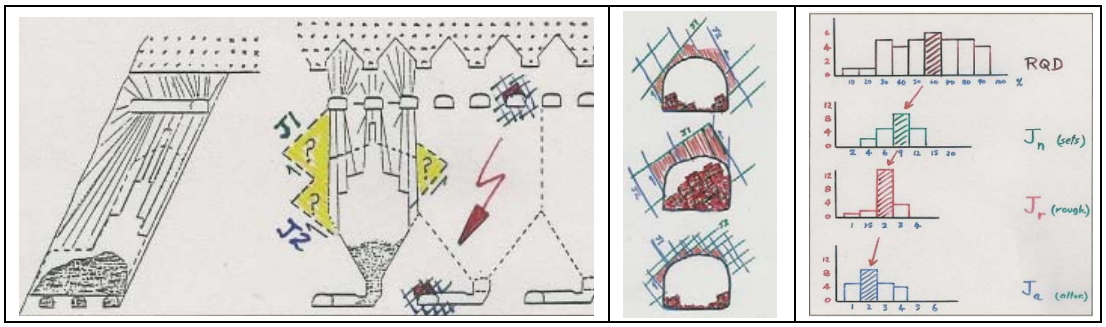



Fig. 7 Overbreak effects at Kiruna LKAB Oscar long-hole drilling Project. On this occasion in 1987 the author used Q-parameter histogram logging to characterize the likelihood of overbreak or stability problems. Most-frequent ratings for RQD, Jn, Jr and Ja of 60, 9, 2 and 2 did not explain the strong overbreak. However, with extreme values like Jn = 12 and Jr = 1 or 1.5, also assisted by Ja = 3 or 4, excessive over-break and near drift collapse in some cases was explained in a logical manner.

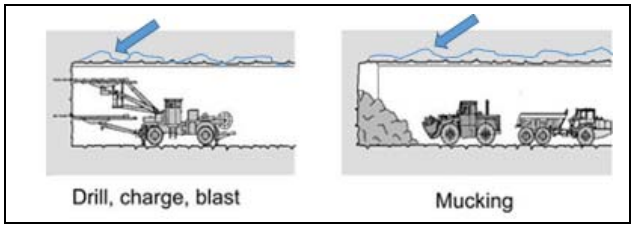



Fig.8. Overbreak affects NMT and NATM in different ways due to the final concrete of NATM.

## 5 Shear strength determines tunnel reinforcement and support needs

The ratio Jr/Ja seen in Figure 9 is remarkably close to being a friction coefficient. It is obviously also a potential parameter for indicating the increased (or decreased) likelihood of overbreak and increased (or decreased) amounts of shotcrete in NMT (periphery length increased, or not) and of increased volumes of concrete (or not) in the case of NATM. Jr/Ja therefore also affects sustainability.

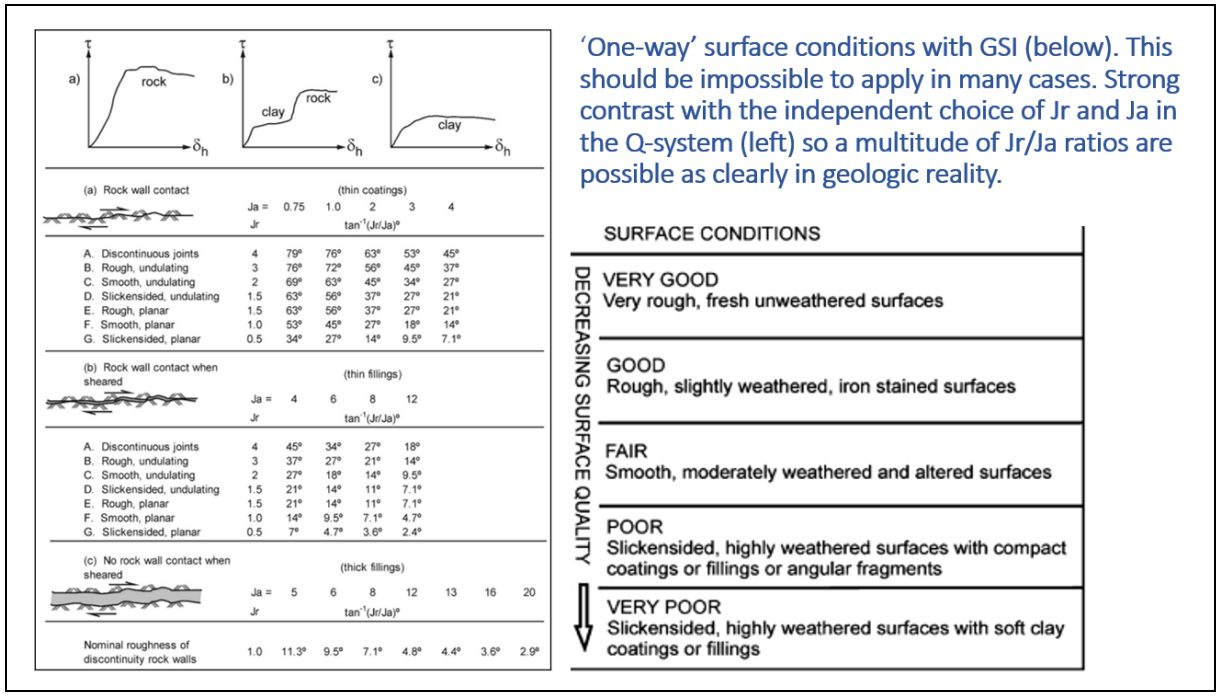



Fig. 9 The Q-parameter pair Jr and Ja are also important indicators of overbreak potential as well as the ratio of Jn/Jr. Note that the 'joint condition' ratio in Q is in another class of reality compared to the 'x-axis' in the GSI diagram of Hoek (or Hoek and Marinos). As seen on the right rough joints cannot be clay filled or be in weathered rock while smooth-planar joints have to be in weathered rock and also be clay filled. It is not clear to the author how users of GSI get around this severe limitation (basically an elimination of much of engineering geology) in practice.

Readers may be aware that the formula for the Q-value can be generalized by normalizing Q with UCS/100. $Q_C$ is defined as Q x $\sigma_c$/100 with compression strength expressed in MPa. It was found (Barton, 1995 and Barton, 2006) that Qc correlated much better than Q with P-wave velocity and with the stress or depth-dependent deformation modulus. The seven-parameter equation forQc = RQD/Jn x Jr/Ja x Jw/SRFx$\sigma_c$/100 can be 'cut in half' and appears to approximately lly represent the cohesive component (CC) and the frictional component (FC) of rock masses as suggested in Table 1 and Figure 10.

Table 1. Left: The Qc-based equation 'cut in half' to represent the cohesive and frictional strength of rock masses, contrasted with the Hoek-Brown equations for the supposed 'c' and 'ϕ' of rock masses. Right: These three supporting equations mean that GSI is repeated 16x and 12x in the H-B equation of shear strength. However if two further supporting equations (e.g. for $\sigma'_{3n}$) are included, we see a remarkable 78x repetition of GSI, which is already a limited descriptor of rock mass quality.

| Expression | Origin |
|---|---|
| $\varphi \approx \tan^{-1}\left(\frac{J_r}{J_a}\times\frac{J_w}{1}\right)$ | FC from Q |
| $\varphi' = a\sin\left[\frac{6am_b(s+m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a)+6am_b(s+m_b\sigma'_{3n})^{a-1}}\right]$ | From GSI |
| $c \approx \left(\frac{RQD}{J_n}\times\frac{1}{SRF}\times\frac{\sigma_c}{100}\right)$ | CC from Q |
| $c' = \frac{\sigma_{ci}\left[(1+2a)s+(1-a)m_b\sigma'_{3n}\right](s+m_b\sigma'_{3n})^{a-1}}{(1+u)(2+a)\sqrt{1+[6am_b(s+m_b\sigma'_{3n})^{a-1}]/[(1+a)(2+a)]}}$ | From GSI |

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

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

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

Figure 10 shows four potential rock masses with successively decreasing rock mass quality and rock strength due to strongly decreasing Qc. The suggested equations for FC and CC are evaluated and given tentative units of degrees ° and MPa. It is interesting to note that the classic Mohr-Coulomb equation of 'c + $\sigma_n$ tan ϕ' (which actually fails to describe the strength of rock masses as cohesion has to be broken before friction can be mobilized) is in the case of Qc replaced by 'CC x tan FC' – so the potential cohesive strength is modified (reduced, or sometimes increased) by the frictional strength – a not illogical criterium. The components 'c' and $\sigma_n$ tan 'ϕ' are not added as in M-C and H-B.

| RQD | $J_n$ | $J_r$ | $J_a$ | $J_w$ | SRF | Q | $\sigma_c$ | $Q_c$ | FC° | CC MPa | $V_p$ km/s | $E_{mass}$ GPa |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 100 | 2 | 2 | 1 | 1 | 1 | 100 | 100 | 100 | 63° | 50 | 5.5 | 46 |
| 90 | 9 | 1 | 1 | 1 | 1 | 10 | 100 | 10 | 45° | 10 | 4.5 | 22 |
| 60 | 12 | 1.5 | 2 | 0.66 | 1 | 2.5 | 50 | 1.2 | 26° | 2.5 | 3.6 | 10.7 |
| 30 | 15 | 1 | 4 | 0.66 | 2.5 | 0.13 | 33 | 0.04 | 9° | 0.26 | 2.1 | 3.5 |

**Four rock masses with successively reducing character: more joints, more weathering, lower UCS, more clay.**

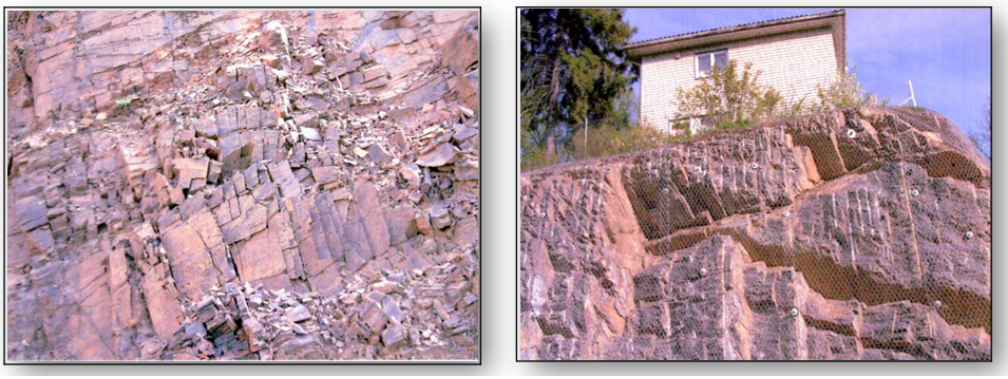


Fig. 11 Top: Four example rock masses with decreasing quality and decreasing rock strength and the apparent realism of the FC and CC estimates of frictional and cohesive strength. Rock masses with low CC need relatively more S(fr) (such as urgently needed for the shale in the left photograph) and rock masses with low FC need relatively more bolting (such asthe massive limestone in the right photograph which has only bolting).The FC and CC components of Qc correspond closely to the original 'conditional factors' for choosing more or less shotcrete and more or less bolting in the original Barton et al. 1974 Q-system tunnel support tables.

# 6 Choice of tunneling depth as potentially serious consequences

Some years ago the author presented a paper in Hong Kong with the title: '*The Shallow Metro Syndrome'*. This was in response to several years following a new metro line in São Paulo, Brazil: Linha 4-Amarela on behalf of the contractor consortium. Despite warnings by several other experts, the designers and owners had apparently prioritized 'short escalators' perhaps having concern for passengers' experiences and future maintenance costs. They apparently failed to understand the time and cost consequences of mixed-face tunnelling through saprolite, clay, granite or gneiss, choosing the adverse elevation/shallow depth where each could be met in the same tunnel face. The project suffered years of delays and cost overruns. In London, long escalators are needed to reach London clay, the cheapest and most stable tunnelling medium. The long escalator rides are entertaining due to adverts.

One of the São Paulo Linha 4 station caverns in granite suffered 2 to 4m over-break (Figure 12). The other end of the station was in clay. The shallow location of the tunnels and stations had enormous consequences on cost and delay and indirectly cost seven lives at the next station about 1km away. Lattice girders and shotcrete were overcome by the vertical loading from an unanticipated 10m high ridge of 'differentially *less-weathered*' gneiss. It was not discovered despite five nearby boreholes on either side of the planned station cavern, and one even on the axis of the future cavern. In retrospect cross-hole seismic tomography should have been attempted. Refraction seismic failed due the very noisy 24/7 environment next to five lanes of traffic. Figures 13 and 14 show the tragic consequences.




Fig. 12 Shallow metro consequences: mixedface, overbreak, potential collapse through to the surface.

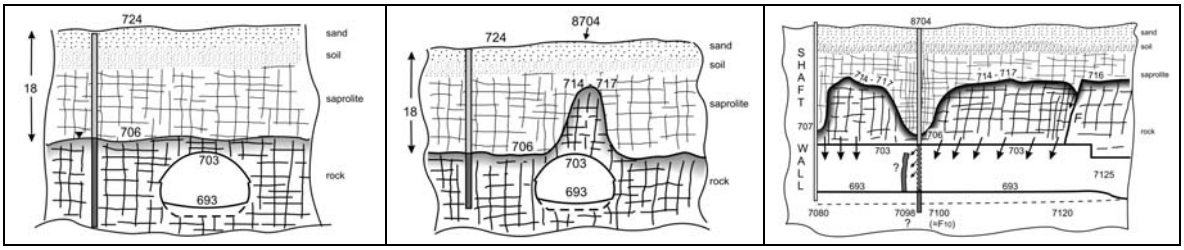



Fig. 13 a, b, c. Five boreholes surrounding or within the planned station cavern failed to reveal the presence of a ridge-of-rock, which had a 'fatal' fracture zone along its axis. All five boreholes suggested just 3m of rock cover. The undiscovered 10m high ridge-of-rock was the result of differential weathering. 'Robust' LG of 32mm x 25mm x 25mm embedded in 30-40cm of S(fr) suddenly folded, allowing the less weathered ridge-of-rock to fall 10m.

The massive collapse seen in Figure 14 caused five people driving in a minibus to be sucked into the sudden failure. Their vehicle was not found for 12 days as it had been sucked to a lower level than most of the surrounding sand and clay. The falling ridge-of-rock had a piston-like effect. A tunnel worker was blown over some distance from the sudden cavern collapse. Two pedestrians were also sucked into the collapse.



Fig. 14 The collapse of the Pinheiros Station cavern during construction. The fatalities occurred along the small road (Rua Capri) seen on the right. Five boreholes had failed to detect the 10m high ridge of less weathered rock that was hidden beneath the cavern axis drawn on the left photograph.

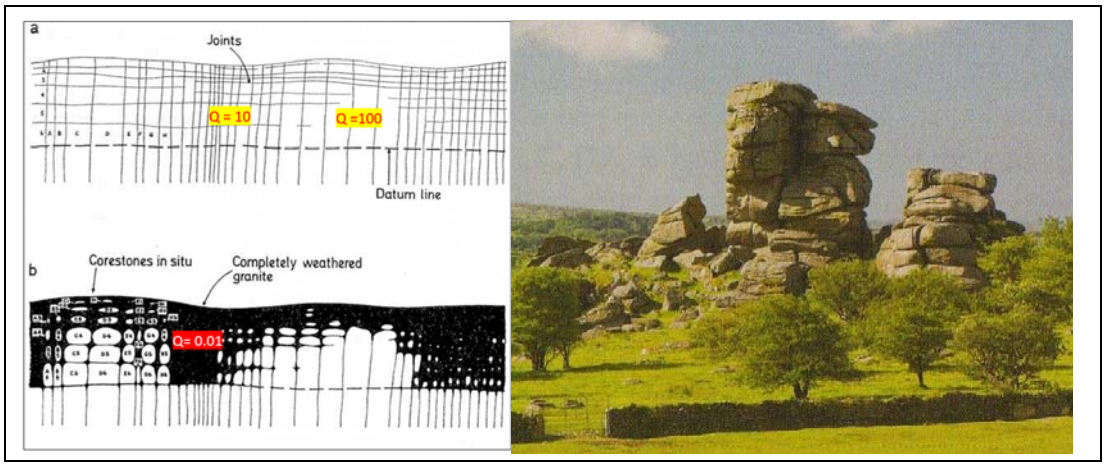



Fig. 15 Two drawings from 'The problem of tors' by Linton (1955) have been given possible Q-values to demonstrate the possible morphological progression to core stones, in this case to dangerous ridges of less weathered rock that can load tunnel or cavern support if too shallow excavations are planned. The tor photograph from S.W. England with a chance 'central' fracture is a potential model for the tragically missed ridge-of-rock (Figure 13c).



Fig. 16 This shows the top of the loosened 'ridge-of-rock' *after* it had fallen 10m to the cavern floor.



Fig. 17 Before and after. The same lattice girders of 32mm x 25mm x 25mm dimensions appear to have limited strength when confronted with a large concentrated vertical load. Designers are usually assuming a rare evenly distributed load. In this case a very high vertical loading was experienced due to an extreme 'ridge-of-rock' core stone phenomenon. This was not discovered in the police-led investigations.

On another occasion contractors tunneling for a motorway city bypass through presumed jointed granites had to tackle deeply weathered (saprolitic) materials at each end. So in principle NATM solutions were chosen for the first 150-200m at each end of the tunnel, and NMT-style Q-based B + S(fr) in the central hard rock kilometers in jointed granite. The unfortunately over-simplified design for the 'NATM' sections is illustrated in Figure 18. Regrettably both the sloping ground and vertically oriented phyllite and a dike (neither were discovered) were not modelled. Approximately 140m of the right tube collapsed on a Saturday afternoon when all the tunnelers were at a God-given barbeque so a tragedy was avoided. Three months later 140m of the parallel tunnel also collapsed. There were no casualties as the site was closed pending a hearing. The more realistic UDEC model by Bandis is illustrated in Figure 19.

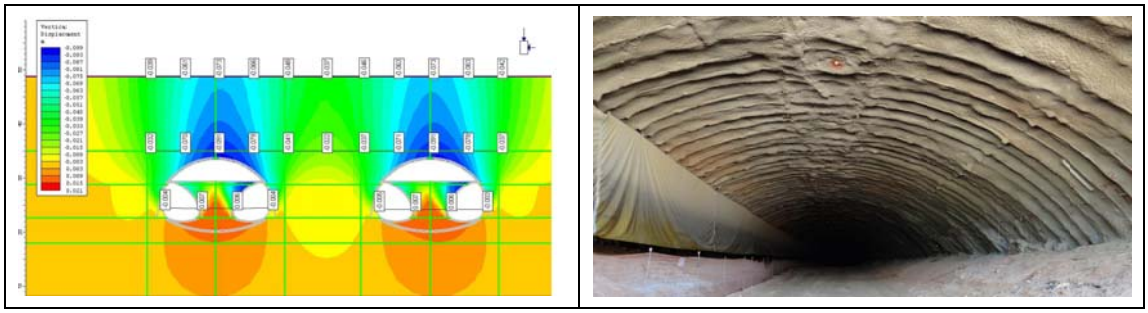


Fig. 18 Optimistic symmetric design, when actually sloping ground. Light lattice girders. No bolts due to saprolite. Much more than specified shotcrete thickness due to 30 to 80cm of over-break. This is the second tube. 140m collapsed three months after the 1st 140m collapse. Again, lattice girders folded.

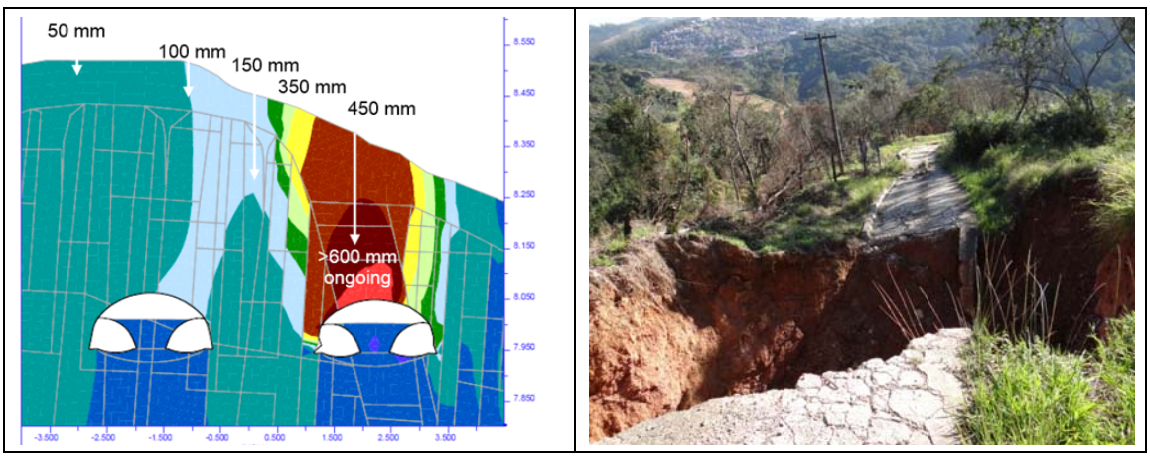



Fig. 19 When the vertical structure (and sloping ground) was modelled the imminent failure is understood. The photograph is after the 2nd tube collapse. UDEC modelling by Bandis.

# 7 Steel sets and lattice girders invite and need deformation to function

Why lattice girders may allow significant deformation is suggested in the instrumented tunnel results from Ward et al. 1983 shown in the left diagram of Figure 20. Firstly, reinforcing any rock mass with systematic bolting and holding it in place with shotcrete (solving the frictional strength/weakness and cohesive strength/weakness) is obviously positive, as proved by the top experimental results of 2-3mm.The circular steel ribs result show how poorly steel sets are working. Rockmass deformation and loosening and potentially reduced rock mass strength is needed before the tangential strain builds in the LG so that it can finally resist radially. Elephant feet must also resist the axial load in the LG.

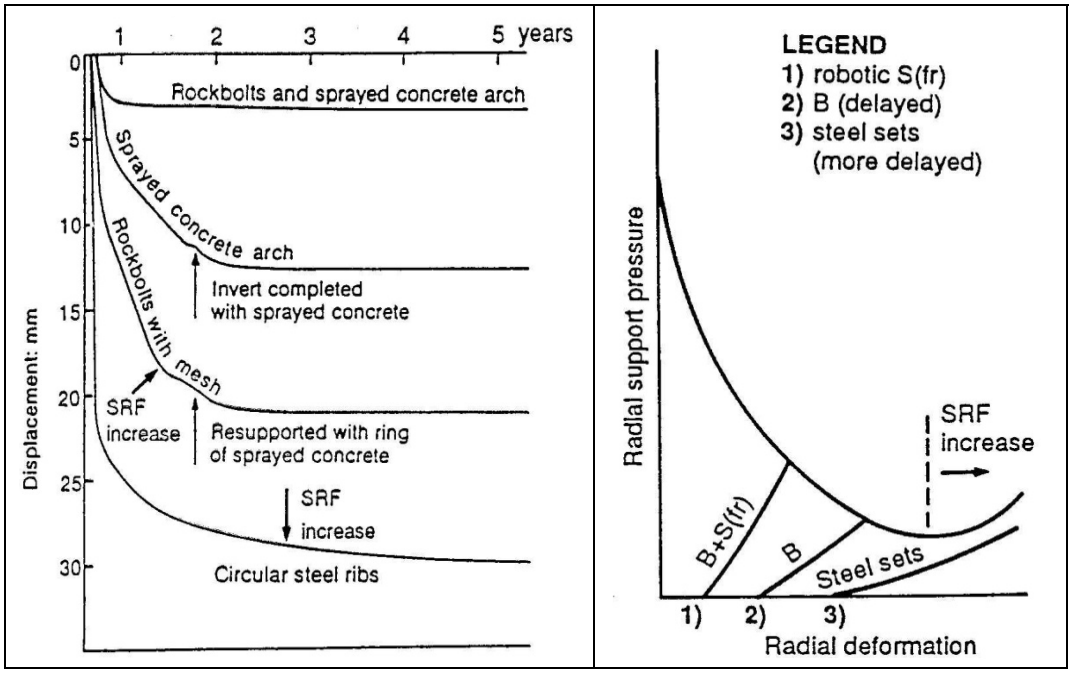



Fig. 20*Left:* Avoidance of steel sets (or lattice girders) remains an important advisory for NMT Q-system users. The instrumented experimental tunnel was driven in mudstones. Figure from Ward et al. 1983. *Right:* Steel sets and lattice girders allow and even require loosening of the rock mass to make better contact, i.e. an increase of SRF in the Q-value. Barton and Grimstad, 1994. The use of steel sets with constant spacing so not allowing for faulted rock, i.e. increased loading, can cause tragic collapses as seen in Figure 21.



Fig. 21 Collapse in partly completed down-stream surge chamber arch in Bhutan. Tragically, six workers were caught in the collapse. First collapse ≈ 35,000 $m^3$. There was an attempt to remove fallen rock (approx. 15,000 $m^3$ was removed). Note the extra destruction of steel sets in the 'left' arch due to the fault. A larger collapse subsequently occurred.

The total volume of collapsed rock mass finally reached approx. 70,000$m^3$, basically initiated due to constant steel set spacing despite changed conditions. As a point of difference when using NMT there is a specific 'owner's half-hour' when engineering geologists representing both sides (owner and contractor) map conditions together and try to agree on the Q-based 'support class'. This occurs after every blast.

In this section a potential source of weakness of NATM has been identified by means of the exceptional design challenges illustrated. It surely must be assumed that lattice girders leave some room for improvement as they do not seem to be stiff enough. Most frequently a 'triangular' format with e.g. 32, 25, 25 mm bars, or a much lighter version: 20, 16, 16mm bars are supposed to add

stability to the shotcrete they reinforce once sprayed in. Figures 17 and 18 show initial deficiencies here with presently insufficient shotcrete. An obviously delaying but much more robust solution is to use a 'rectangular' bundle of four equal bars or 2 + 2 of different diameter *and bolt them into the arch.*

The key structural change would be the ease of mounting / welding a cross-piece with a central bolt hole. In other words bolting the unconventional 4-bar LG in a similar way to RRS (Figure 2: mid-right). This would take longer but would be 'an order of magnitude' safer. The bolting would be an integral part of the bolt reinforcement of the tunnel or cavern arch, as it is in the case of the RRS which is used with NMT if rock mass conditions are very unstable, for instance due to faulting.

## 8An example of failed NMT due to water velocity

On occasion there are of course collapses with NMT.A Canadian tunnelling consultant recently suggested that Norwegians should not consult outside Norway. In 'reply' it was suggested that he should stay in Canada also. (Barton, Grimstad and Panthi, 2024). Collapses have been experienced in major fault zones where failure to recognize the greatly reduced quality has been the cause. Even failure to identify clay in discontinuities in a sub-sea tunnel. There have also been cases where the design company has not understood the functions of SRF, resulting in multiple collapses of a 'so-called' NMT. An Australian author once criticised Q/NMT because of a cavern collapse, most likely due to the fact that the bolts were ungrouted. Ungrouted bolts and failure to recognise swelling clay resulted in major collapses on another continent. In several projects where asked to give independent opinions, 'painting with shotcrete' (1mm to 5mm?) has been the obvious cause of local collapses. This is a very serious corner-cutting operation made by occasional sub-contractors in several countries.

The case to be described here is due to failure of the designer to recognise that diversion tunnel water velocities as high as 10m/s and even 12m/s are far too high without at least invert erosion protection. Insult was added to injury by the 'design' (or poorly-considered drawing) of a sharp bend in the tunnel close to a fault zone. Obviously water in a 12 x 14m diversion tunnel will exert a huge force on the walls when travelling at 36-40km/hr and forced round a sharp bend. The original Q-system case records from hydropower projects were generally based on 1.5 to 2.5m/s headrace tunnel velocities and no sharp bends. The low velocity is to prevent debris transport, so no 'rock in the turbines'!




Fig. 22 A cascade of tunnel failures occurred at Hidroituango, Colombia, starting with the conical feature and ending in flow of rock debris through the powerhouse and transformer hall. A 2.5 billion dollar claim and 4 years delay were the final result of the simple design error. Note sharp bend.

## 9 Conclusions

1. Single-shell NMT and double-shell NATM can provide valid solutions to the three principal parts of a typical tunnelling project: the two portal areas in weathered rock and the usually much longer central section of a tunnel that is generally in higher quality rock.

2. By its very nature NATM is more expensive and less sustainable in terms of the volume of concrete needed in the portal areas. This is because overbreak needs to be filled in the case of NATM but not in the case of NMT.

3. Lattice girders are considered to be deformation enhancing because radial support pressure is not immediately developed. Stiff footings (elephant feet) are needed, contact with the rock arch is necessary through diligent shotcreting, followed by build-up of tangential strain along the structure and finally the desired radial but mostly passive support.

4. The design of lattice girders can be improved if a rectangular 4-bars system is used with a welded plate and bolt hole. Systematic bolting of lattice girders would be 'a delaying factor' as it is with RRS in the case of NMT, but the support and reinforcement effects would be enormously improved, and the effects of non-uniform loading much reduced.

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