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# Q-SYSTEM APPLICATION IN NMT AND NATM AND THE CONSEQUENCES OF OVERBREAK

Nick Barton and Eystein Grimstad
NB&A, Høvik, Norway and Geolog Eystein Grimstad, Oslo, Norway
nickrbarton@hotmail.com and eystein.grimstad@vikenfiber.no

## SUMMARY

The Q-system of rock mass classification for assisting in support and reinforcement selection for rock tunnels and caverns has now been in use for 40 years. During the last 20 years it has been used in order to assist in the choice of permanent single-shell fiber-reinforced S(fr) support and systematic corrosion protected rock bolt reinforcement. Twenty years ago the original S(mr) mesh reinforced recommendations were updated to fiber-reinforced shotcrete, in order to reflect the by then more than ten years of experience of wet process, robotically-applied S(fr) in Norway. This revolutionary product now has a 35 years track record. The Q-system is also used to select rib-reinforced shotcrete arches (RRS) which are superior to steel arches and lattice girders, because intimate contact with the tunnel arch and wall, and systematic bolting of these arches are integral and essential components of the method. The bolted RRS arches therefore help to prevent further deformation instead of allowing it as in NATM, which is a labour-intensive method which does not address these two problems adequately. In this paper some of the other differences between single-shell and double-shell tunnelling will be emphasised, including the frequent use of Q to select only the *temporary support and reinforcement* in double-shell tunnelling, using the 5Q and 1.5 ESR rule-of-thumb. Hong Kong road and metro authorities have applied this method in the last 25 years in hundreds of kilometres of tunnels and in station caverns. The B+S(fr) applied in such cases as the first stage of double-shell NATM, is considered as temporary support and reinforcement, prior to casting the final concrete lining with its drainage fleece and membrane. The temporary support is ignored in the final lining design. This of course is wasteful and adds to the cost. This paper also briefly addresses some of the useful Q-correlations to rock engineering parameters such as P-wave velocity, deformation modulus, and tunnel and cavern deformation. All are depth or stress-dependent.

## INTRODUCTION

The first wet-process fiber-reinforced shotcrete applied in Norway was in a hydropower cavern in 1979 and in a main road tunnel in 1981. Mesh-reinforced shotcrete S(mr) ceased to be used by about 1983. The Q-system development in 1974 [1], which was first based on B+S(mr), was updated 'late' in Norway [2] by Grimstad and Barton, but obviously 'early' for many other countries. In Austria, B+*S(mr)*+lattice girders are seemingly still favoured as temporary support for transport tunnels. The writers have been surprised to see Austrian consultants continued recommendation of S(mr) in good quality but over-breaking rock in Asia. However the very strange and diametrically-incorrect instructions are given to accept the use of S(fr) when there is *no significant overbreak,* but to use S(mr) *where there is overbreak*. As we shall see in a moment, overbreak is of course a fundamental geometric factor that increases the volume of S(fr) in both single-shell and double-shell tunnelling, and can have quite adverse influence in the latter concerning concrete volumes and the need to construct a more time-consuming three-dimensional membrane and drainage fleece. In many Scandinavian tunnels and in many of the world's much larger hydropower caverns, *single-shell* and Q-system based methods are preferred due to their economy. However, thorough jet-cleaning of the rock surface prior to shotcreting, and use of good quality and corrosion protected B+S(fr) are obviously essential.

## THE ADVANTAGE OF A LOGARITHMIC QUALITY SCALE

Unlike RMR or GSI (= RMR-5) and the Austrian F1 to F7 rock mass quality scale, the Q-value resembles a logarithmic scale of quality with its six orders of magnitude from approximately $10^{-3}$ to $10^{3}$. With the normalization $Q_c = Q \times UCS/100$ described in [3], the Qc scale can reach almost eight orders of magnitude, and then approaches the actual variability found in nature. One only needs to consider the range of deformation moduli and shear strengths depicted in the deliberately contrasted photographs in Figure 1 to realise that not only these parameters, but also the need for support and the *loading of the support* (i.e. Sfr) can cover an extremely big range: from zero up to 100 t/m$^2$. As will be seen later, there is a clear inverse proportionality between support pressure/capacity needs, and the simply estimated deformation modulus, which itself can vary by a factor of 100, or even 1000, between the extremes of saprolite/soil and hard unjointed rock. The non-linearity of nature does not link in a simple way to linear (RMR or GSI) qualities.

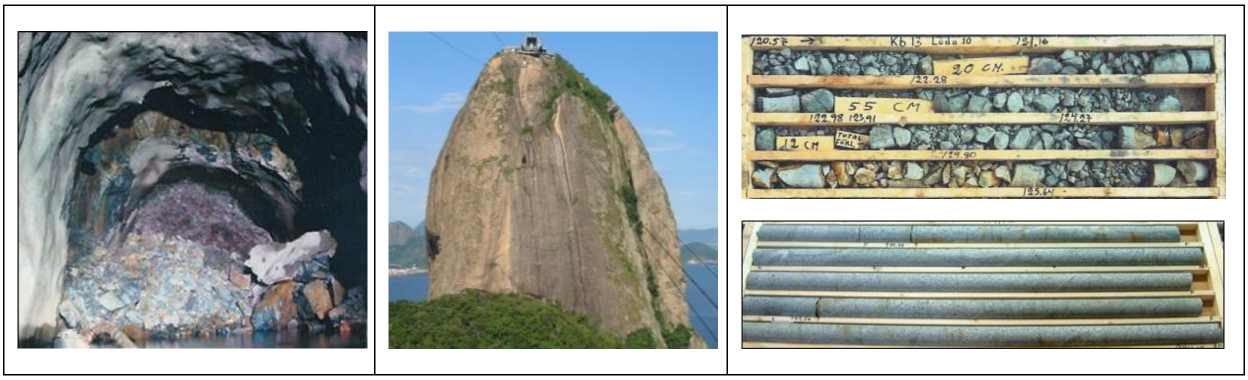


*Figure 1 Contrasting worst (Q ≈ 0.001) and best (Q ≈ 1000) rock mass qualities. The logarithmic appearance of the Q-value scale, stretching over six orders of magnitude, has proved to be a great advantage, and results in simple empirical equations for relating Q or $Q_c$ to velocity, modulus, and deformation. The large numerical ranges of Q and especially $Q_c$ appear realistic when considering the extreme variation of shear strength and deformation modulus in these examples. A huge variation in construction time is also implied: i.e. 15 years to 1 year. This of course depends on tunnel length. The core box with core loss is from Hallandsås.*

## OVERBREAK ESTIMATION USING THE RATIO Jn/Jr

An unusual combination of Q parameters: the ratio Jn/Jr, specifically involving the number of joint sets, indicates that a ratio Jn/Jr ≥ 6 automatically means a strong likelihood of 'natural' overbreak, for which a contractor cannot be blamed for harsh blasting practices. Figure 2 illustrates the combined importance of Jn and Jr. The overbreak-facilitating ratio Jn/Jr ≥ applies over a range of Jn = 15, 12, 9, 6, 4 and over a range of Jr of 1, 1.5, 2 and 3. If blocks are not formed due to insufficient joint sets, or when joint roughness is significant, a typically massive rock mass with for instance Jn/Jr = 4/3, would mean virtually all half-rounds are visible. However, a contractor will have great difficulty to produce half-rounds when Jn/Jr > 6.

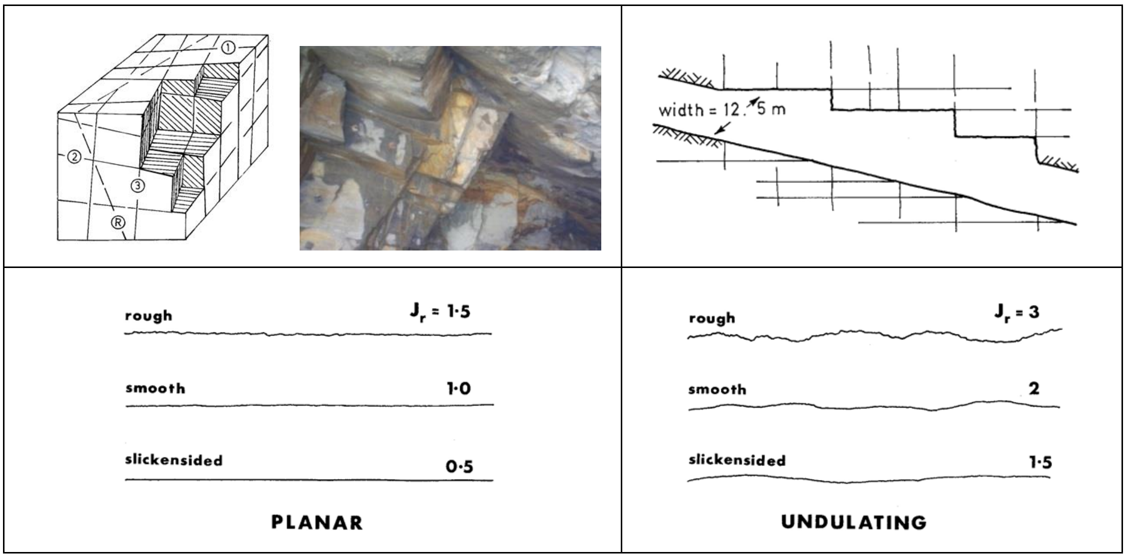



*Figure 2 Overbreak is extremely likely to occur despite a contractor's efforts with careful blasting, if the most frequent value of the ratio Jn/Jr ≥ 6, i.e. 6/1, 9/1.5, 9/1.0, 12/2, 12/1.5, 12/1.0, 15/1.5. Visible half-rounds and lack of overbreak will tend to be found when Jn/Jr < 6, such as 3/1, 4/1, 6/1.5, 9/2, 9/3, 12/3, 15/3 since the lack of block structures combined with dilatant joint roughness or discontinuous joints, prevent its occurrence. All half-rounds (and virtually no overbreak) will appear with Jn/Jr = 2/3, or 2/4 which would be typical of a massive granite with discontinuous jointing.*

Overbreak is a key 'ingredient' in the thickness and especially the volume of S(fr) required in both NMT (*as permanent support*) and in NATM (*as temporary support*), and if time-consuming NATM is to be used, membrane fixing and final concrete volumes (or increased volumes of overbreak-smoothing shotcrete) will be greatly affected by excessive over-break. There are in addition about 12 to 15 km of welds to be guaranteed in each 1 km of NATM tunneling requiring a drained membrane, so over-break is a complicating issue here too, as a '3D membrane' is needed where overbreak is severe, unless large volumes of 'smoothing' shotcrete are used. Cost and time both increase due to overbreak, whatever solution is used to 'fix it'.

## ILLUSTRATIONS OF THE CONTROL OF WATER IN NATM AND NMT

The problem of excessive overbreak due to joint set structure and joint roughness (or lack of roughness) easily doubles or triples the volume of shotcrete, and causes even larger increases in the volume of concrete, if double-shell (NATM) is to be the final stage of rock tunnel or rock cavern development. Since a dry tunnel or (metro-station) cavern is obviously required, excessive overbreak is also a problem that affects the fixing and welding of the drainage fleece and membrane in the case of double-shell (NATM-style) tunnels and (metro-station) caverns.

If a rock mass has characteristics such as Jn = 9 (three joint sets) and Jr = 1 or 1.5 (smooth planar or rough planar joints), there may be sufficient risk of overbreak that the economy of the project is affected. The membrane is also more difficult to construct, because damage from concrete fluid pressures during pouring, may cause leakage if the membrane is not formed as a sufficiently 3D surface, as illustrated in Figure 3. This extra work is more time-consuming.

Concrete volumes will frequently be far higher than the 30 or 40 cm uniform thickness that was designed (and unrealistically drawn) in designer's offices. In fact thermal stresses resulting from widely different thicknesses of concrete (for instance 30 cm to 130 cm) may cause cracking, if there is no reinforcement of the concrete. Unrealistic numerical models with an assumed

*Figure 3 Only one of these photographs can be considered as 'unrepresentative' of the norm, and is clearly the too-shallow metro station (top-left) which shows overbreak as much as 4 meters. In fact it is photographed in the pre-concreting / post-shotcreting stage of the Jn/Jr >6, photograph in Figiure 2. The other photograph of overbreak (bottom left) shows 'normal overbreak' which was (insufficiently?) shotcreted in a Norwegian tunnel, and where the white membrane (lower-right) is about to be applied. The yellow membrane (top-right) is from a metro tunnel in Hong Kong. In both membrane photos, a degree of 3D adjustment is evident, allowing the concrete to increase to at least 1 m thickness, instead of the minimum 35 cm as designed.*

uniformly thin shell all in compression would change, if widely varying concrete thicknesses had been introduced. The optimistic assumption of 'no need for steel reinforcement' would also change. The potential for cracking can lead to subsequent problems in a cold climate such as in Norway, because of moisture and water brought into the tunnel by wet cars, lorries or trains, which subsequently freezes, thereby gradually changing an assumed maintenance free tunnel into one that is probably less capable of resisting the effects of aging than a single-shell NMT S(fr)-lined tunnel, since the poured concrete may be unreinforced 'to save time and money'.

An additional problem with membrane construction for the case of double-shell NATM-style tunnelling, are all these 'extra' kilometres of membrane welds needed for each kilometre of tunnel. In a large twin-track rail tunnel with for instance an excavation area of 110 $m^2$, the membrane will need to cover some 25 m of the tunnel arch and walls. With welded seams each 1.8 to 2.0 m, one is looking at 12.5 to 13.9 km of welds for each kilometre of tunnel: a severe test of quality control. Many unplanned 'rubber-boat' type repairs can also be noted in such membranes. The process seems to be 'unnatural' when reliable alternatives are available.

*Figure 4 Two tunnels where single-shell construction with sprayed-membrane for water control has been the successful solution. The tunnel on the left is from the Lausanne metro (photo: Karl Gunnar Holter), and the tunnel on the right is the Hindhead main road tunnel in England during final shotcreting. (Photo:Shani Wallace, Tunnel Talk, July 2011).*

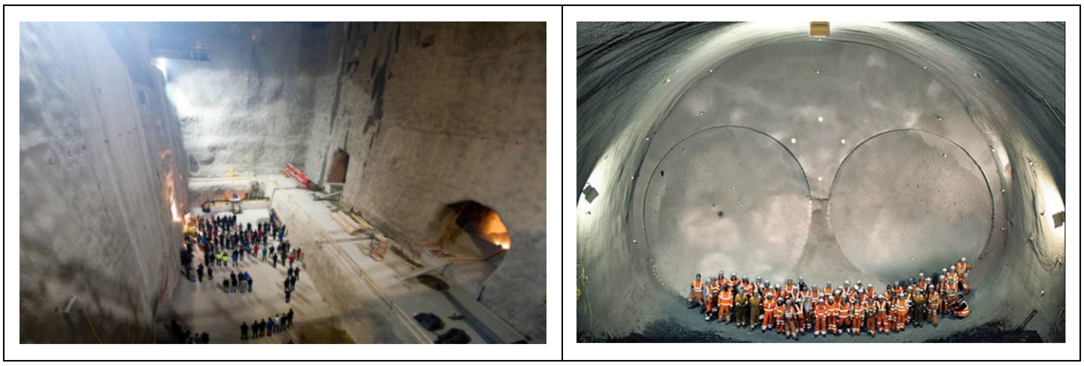


*Figure 5 The pumped storage cavern on the left demonstrates that even in Austria, single-shell construction may be used, even though obviously not called NMT. The tunnel on the right shows a recent dry station cavern at 40 meters depth in London clay, with multiple layers of S(fr) as the final support. In the UK this logical method is called SCL. For contrast to NMT see [4].*

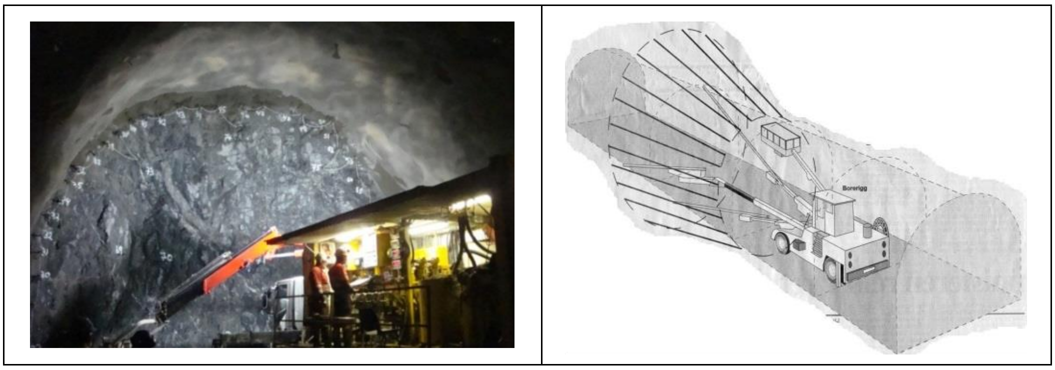


*Figure 6 High pressure pre-injection as often practiced in recent rail tunnels in Norway is found to seal effectively, and the usual 1 to 6 litres of grout per m3 of rock mass, injected into an assumed 6 m thick annulus (very few bolt holes leak) usually ensures only 2 to 4 litres/min/100m of water inflow [5].*

Although seldom used in Norway so far, though now under-going serious field trials, a final local application of e.g. BASF 345 sprayed membrane in occasional humid areas is all that is required for ensuring an economic, dry and easily inspected tunnel lining. Even with pre-injection, and a sprayed membrane, NMT costs a fraction of NATM.

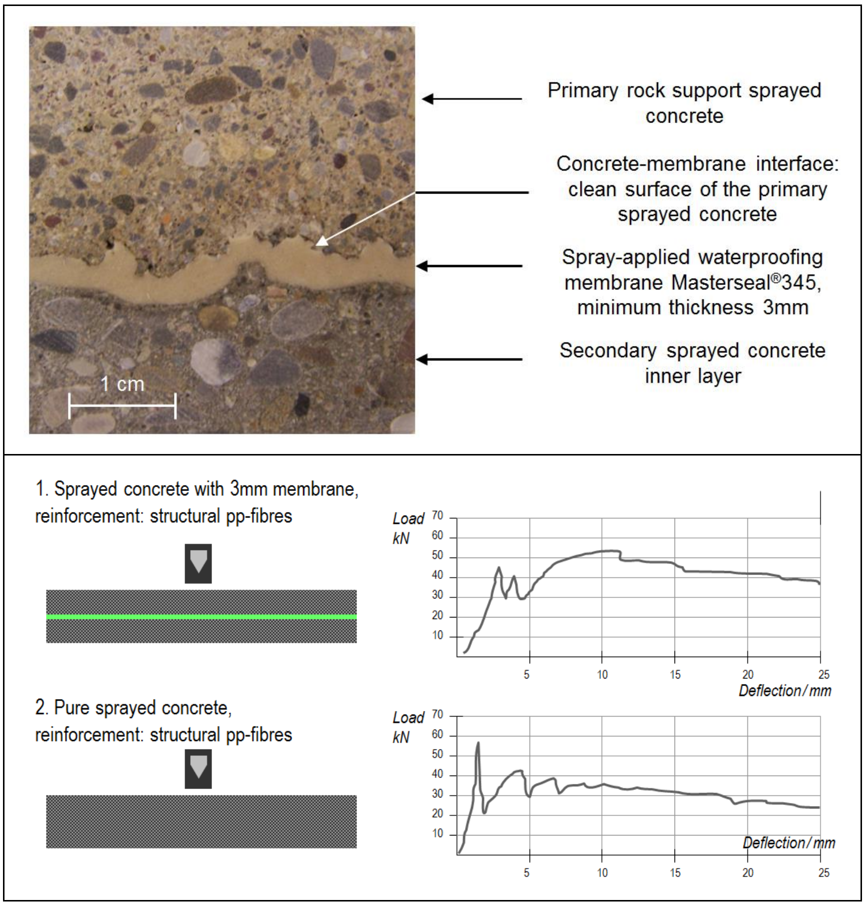



*Figure 7 Sprayed membrane in a sandwich as illustrated using BASF 345 technology [6]. Central-loading tests of 10 cm thick S(fr) polypropylene-fiber reinforced circular plates on a circular base, show inferior load-deformation characteristics compared to 5cm + 5 cm + membrane sandwich, using BASF 345 sprayed membrane of a few millimetres thickness as illustrated. The presence of the membrane is therefore positive for two reasons.*

## TEMPORARY SUPPORT ESTIMATE FROM Q IN NATM TUNNELS

The updated Q-support chart from 1993 [2] is often referenced in relation to single-shell NMT tunneling. In Figure 8 (top), the ‘coordinates’ of the cube, representing a portion of a 20 m span cavern with local Q = 3, would require B+S(fr) of 2.0 x 2.0m c/c + S(fr) of 9 cm for *permanent NMT-style support*. Each would be of high quality, meaning multi-layer corrosion-protected (CT) bolts, and e.g. C45 MPa S(fr) with stainless steel (or pp) fibers. However with the rule-of-thumb of *1.5 x ESR and 5 x Q* for temporary support, which was actually intended as guidance to contractors (i.e. not a temporary support procedure for consultants planning a concrete lining), the largest ‘cube’ would reduce to ‘coordinates’ of *B + S(fr) = 2.4 x 2.4 m c/c + S(fr) = 4 cm. (SPAN / (1.5 x ESR) = 13.3m, and 5Q = 15*, as shown by the larger arrow-head in Figure 8a).

Some 25 years of experience using this officially approved method, in hundreds of kilometers of metro, road and rail tunnels in Hong Kong alone, has proved its reliability in ensuring sufficient *temporary support*, pending the construction of the permanent reinforced concrete lining (with drainage fleece and membrane). The authors much prefer NMT to NATM, since it is 1/4 to 1/5 as expensive and the tunnel is completed much faster. However the reality is that many countries find the budget for NATM and permanent concrete linings: they have fewer tunnels than, for instance Norway. In which case a Q-based *‘5Q+1.5 ESR’* can be used to select the temporary support. If RRS (see later) instead of lattice girders are also used, deformation may be reduced.

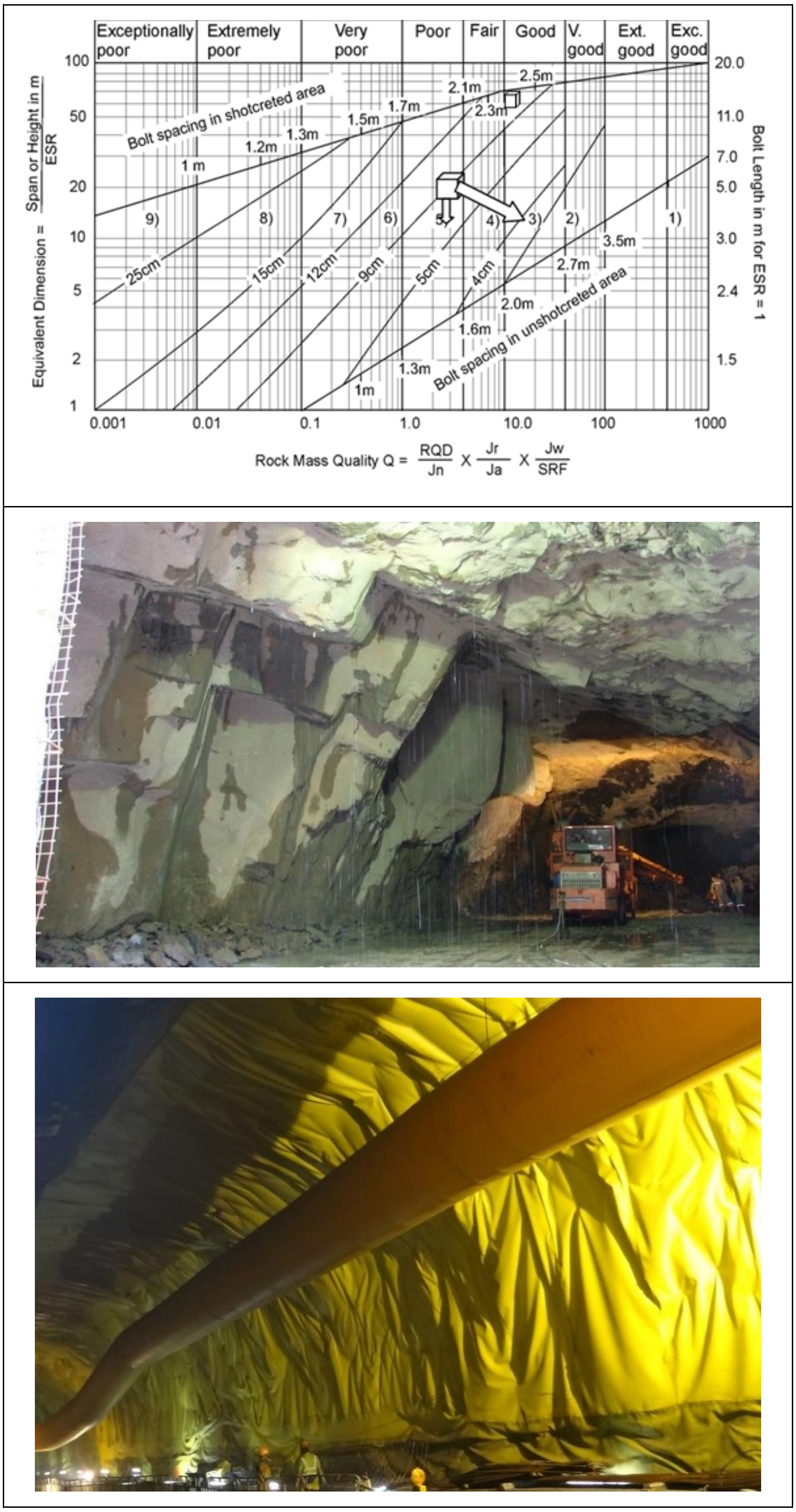
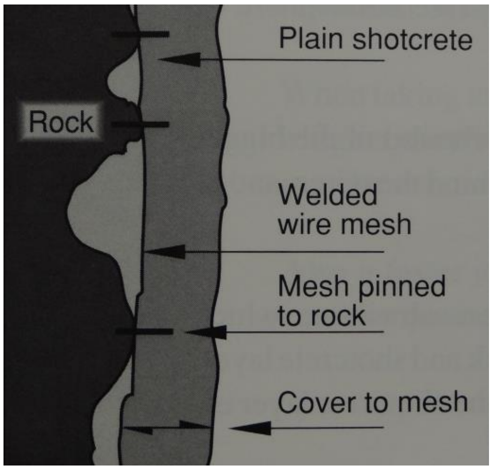



*Figure 8 Using the 1993 updated Q-support chart [2], the rule-of-thumb [1] of 5Q and 1.5 ESR for temporary B+S(fr) support is demonstrated for a SPAN = 20m, ESR =1.0, Q =3 portion of an imaginary station cavern (an updated ESR table is given later). If this rule-of-thumb is used, as in Hong Kong, for selecting temporary support prior to final concrete lining works, then the problem of overbreak (see exaggerated Brazilian example) and sometimes extensive smoothing and 3D membrane construction still needs to be solved. This can be costly. (Note the small cube representing the single-shell NMT Gjøvik cavern plotted at 60m span. This is described later).*

## THE COMPONENTS OF Q-SYSTEM BASED SUPPORT : S(FR), CT BOLTS, RRS

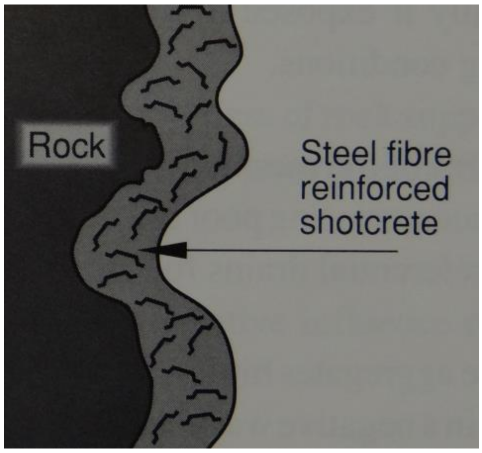
This section consists of illustrations of some key items of the NMT-based Q-support recommendations, including a poor example of yesterday's S(mr), to illustrate what we have left behind, compared to the last thirty five years of S(fr), as dimensioned for instance using [2].

Because the Q-system was developed in 1973, the single-shell case records had permanent shotcrete support and bolting reinforcement of lower quality than that available in the decades that followed. This is an example of S(mr) from Peru, with all the potential disadvantages of S(mr) well illustrated.



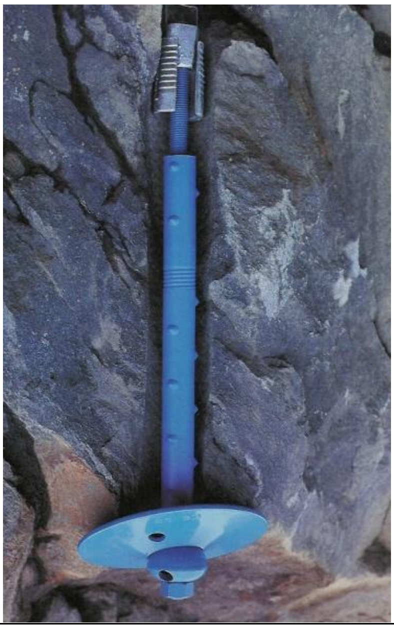

Vandevall [6] illustration of the pitfalls with mesh reinforced shotcrete: three processes, risk of 'shadow' and /or some rebound, corrosion of the mesh due to electrolytic currents, and delayed installation.

Wet process steel-fiber reinforced shotcrete, applied after thorough washing, and use of corrosion-protected rock bolts (e.g. CT-type) are the most important components of the updated Q-system support/reinforcement recommendations.




Vandevall, [6] illustration of the obvious advantages of S(fr): better bonding, no shadow, less corrosion, much lower permeability, faster, cheaper per meter.

*Figure 9 The advantages of S(fr) compared to S(mr) are easily appreciated in these contrasting examples. The sketches from [6] 'Tunnelling the World' are not-exaggerated. The reality of single-shell NMT-style tunneling, in comparison to double-shell NATM-style tunneling is that each component of support has to be permanently relied upon. In NMT there is no neglect of the contribution of temporary shotcrete, temporary rock bolts, and temporary steel sets, and reliance on a final concrete lining, as in NATM. Thus more care is taken in the choice and quality of the support and reinforcement components B+S(fr) + (eventual) RRS. Figure 10 illustrates (in the form of a shortened demo sample) the workings of the CT bolt, and Figure 11 illustrates some of the internal reinforcement details and final appearance of RRS (rib reinforced shotcrete) which are robust and stiff compared to yielding un-bolted lattice girders.*



Over-cored CT bolt: Joint/crack deformation next to the bolt (an expected mechanism when installing close to the tunnel face) does not initiate a potential process of corrosion which it might in the case of a conventional bolt without the PVC sleeve.

*Figure 10  Because single-shell (NMT) relies on high quality S(fr) and long-life rock bolts, the multi-layer corrosion protection methods developed by Ørsta Stål in the mid-nineties, became an important part of NMT. The left photo shows a blue-coloured PVC sleeve: there are also CT-bolts with black, and white, PVC sleeves.*

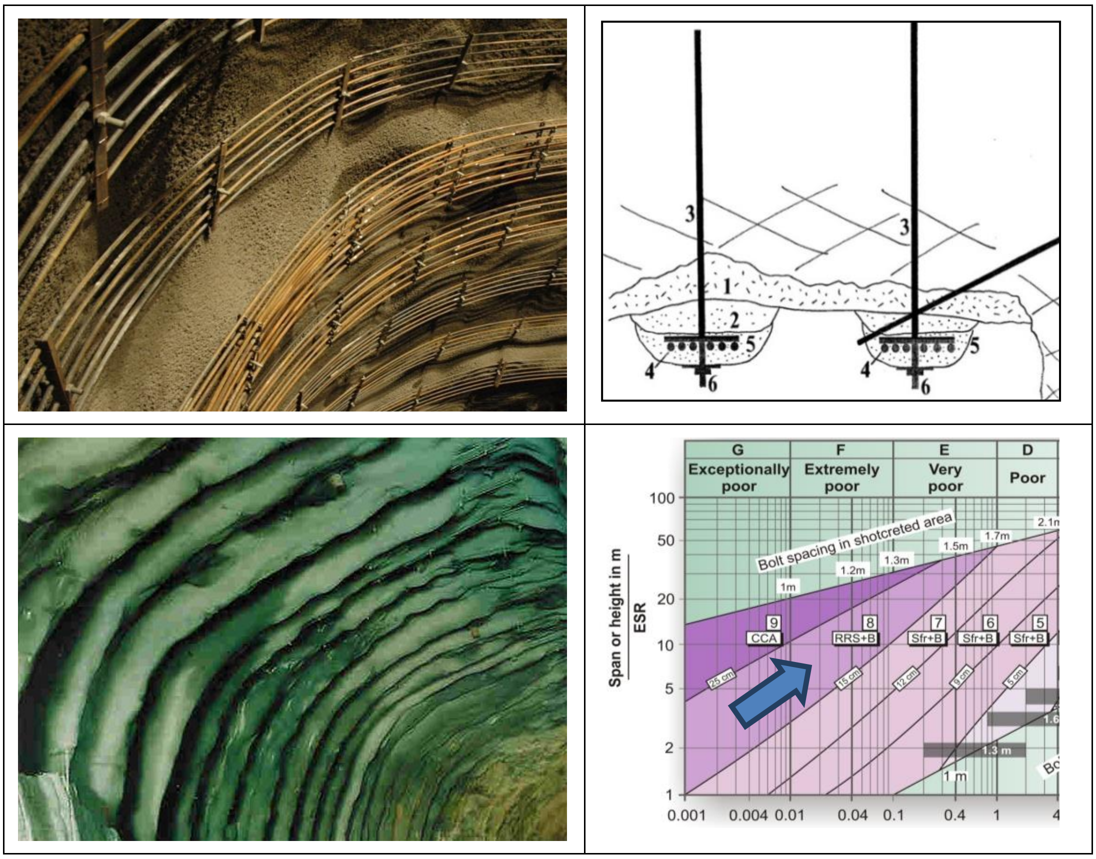
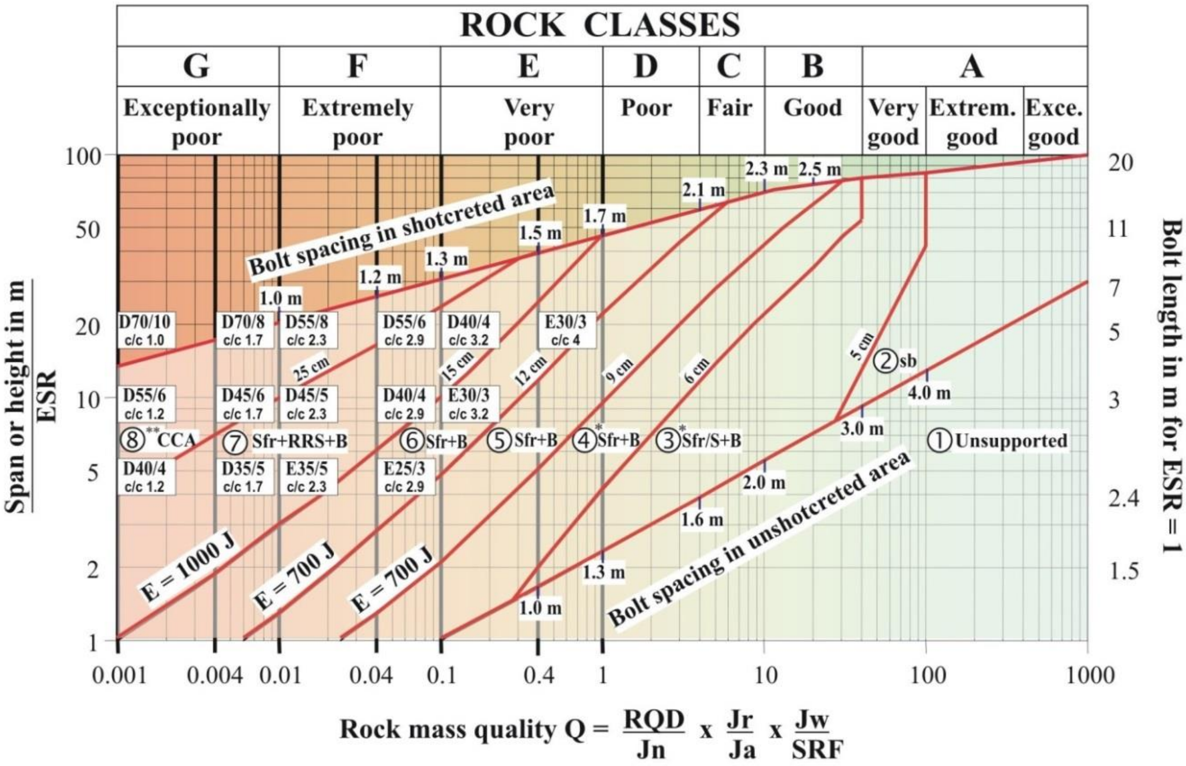



*Figure 11  Some details which illustrate the principle of rib reinforced shotcrete RRS, which is an important component of the Q-system recommendations for stabilizing very poor rock mass conditions. The top left photograph is from an LNS lecture published by NFF, and the design sketch is from [8]. The blue arrow shows where RRS is located. See details in Figure 12.*

The photograph of completed RRS ribs (bottom-left, Figure 11) is from one side of the National Theater station in downtown Oslo, prior to pillar removal beneath only 5m of rock cover and 15m of sand and clay. Final concrete lining followed the RRS for obvious architectural reasons.




**REINFORCEMENT CATEGORIES**

1) Unsupported
2) Spot bolting, **sb**
3) Systematic bolting, and unreinforced or fibre reinforced shotcrete, 5-6 cm), **Sfr/B+S**
4) Fibre reinforced shotcrete and bolting, 6-9 cm, **Sfr+B**
5) Fibre reinforced shotcrete and bolting, 9-12 cm, **Sfr (E700) +B**
6) Fibre reinforced shotcrete and bolting, 12-15 cm, **Sfr (E700) +B**
7) Fibre reinforced shotcrete > 15 cm + reinforced ribs of shotcrete and bolting, **Sfr (E1000) +RRS+B**
8) Cast concrete lining, **CCA** or **Sfr (E1000) +RRS+B**

The bolts are 20 or 25 mm in diameter

E) Energy absorbtion in fibre reinforced shotcrete at 25 mm bending during plate testing

D45/6 c/c 1.7 = RRS with totally 6 reinforcement bars in double layer in 45 cm thick ribs with centre to centre (c/c) spacing 1.7 m. Each box corresponds to Q-values on the left hand side of the box

*) Up to 10 cm in large spans
**) Or **Sfr+RRS+B**

*Figure 12 The most recent updated Q-support chart published by Grimstad, [9].The details of RRS dimensioning given in the 'boxes' in the left-hand-side of the Q-support diagram were derived by a combination of empiricism and some specific numerical modelling by a small team at NGI. Details of this modelling are given in [10] and [11]. Note that each 'box' contains a letter 'D' (double) or a letter 'E' (single) concerning the number of layers of reinforcing bars. (Figure 11a shows both varieties). Following the 'D' or 'E' the 'boxes' show maximum (ridge) thickness in cm (range 30 to 70 cm), and the number of bars in each layer (3 up to 10). The second line in each 'box' shows the c/c spacing of each S(fr) rib (range 4m down to 1m). The 'boxes' are positioned in the Q-support diagram such that the left side corresponds to the relevant Q-value (range 0.4 down to 0.001). Note energy absorption classes E=1000 Joules (for highest tolerance of deformation),700 Joules, and 500 Joules in remainder (for when there is lower expected deformation). Note: S(fr) rib below steel bars, then S to minimize rebound.*

It should be noted that the 1993 Q-support chart (shown earlier in Figure 8a) suggested the use (at that time) of only 4-5 cm of *unreinforced* sprayed concrete in category 4. The application of *unreinforced* sprayed concrete came to an end during the 1990's, at least in Norway. Furthermore, thickness down to 4 cm is not used any longer in Norway, due to the already appreciated risk of the shotcrete drying out too fast when it is curing. The Q-chart from 1993 (Figure 8a) and also an updated 2002/2003 version, indicated a very narrow category 3 consisting of only bolts in a 10m wide tunnel when Q was as high as 10-20. This 'bolt-only' practice is not accepted any longer in Norway, at least for the case of transport tunnels.

The category 3 in 1993 and 2002/2003 has been taken away in this newest 2007 chart (Figure 12) which was fine-tuned by Grimstad in 2006. However for less important tunnels with ESR = 1.6 and higher, only spot bolts are still valid. Hence we may distinguish between transport tunnels (road and rail) and head race tunnels, water supply etc. 2014 is an appropriate 40 year milestone for updating the ESR table published in 1993 [2].

*Table 1 The ESR values recommended in 1993 [2] are given in the table on the left. In a recent Q-manual [12] some updated ESR values tabulated on the right have been suggested, in order to reflect the increased conservatism in some sectors of civil engineering, when applying single-shell NMT.*

| Type of Excavation | | ESR | **ESR recommended** |
|---|---|---|---|
| A | Temporary mine openings, *etc.* | *ca* 2-5 | **ca. 2 to 5 (unchanged)** |
| B | Permanent mine openings, water tunnels for hydropower (exclude high pressure penstocks), pilot tunnels, drifts and headings for large openings, surge chambers | 1.6-2.0 | **1.6 to 2.0 (unchanged)** |
| C | Storage caverns, water treatment plants, minor road and railway tunnels, access tunnels | 1.2-1.3 | **0.9 to 1.1**<br>**Storage caverns 1.2-1.3 (unchanged)** |
| D | Power stations, major road and railway tunnels, civil defence chambers, portals, intersections | 0.9-1.1 | **Major road and rail tunnels**<br>**0.5 to 0.8** |
| E | Underground nuclear power stations, railway stations, sports and public facilities, factories, major gas pipeline tunnels | 0.5-0.8 | **0.5 to 0.8 (unchanged)** |

## CONTRASTING SINGLE-SHELL NMT AND DOUBLE-SHELL NATM

The use of steel sets or lattice girders is avoided in the practice of single-shell NMT, due to the potential loosening of insufficiently supported rock in the periphery of the excavation. It is difficult to 'make contact' between the steel sets or lattice girders and the rock, especially when there is over-break. The results of experiments using different support methods are illustrated in Figure 13. The left-hand diagram shows the results of trial tunnel sections in mudstone [13].The five years of monitoring clearly demonstrate the widely different performance of the four different support and reinforcement measures.

In the right-hand diagram, from [14], the contrasting stiffness of B+S(fr) and steel sets is illustrated in a 'confinement-convergence' diagram, with the implication that an elevated SRF (loosening variety) may occur when using steel sets. It should be clear that the early application of S(fr) by shotcrete robot, and the installation of permanent corrosion protected rock bolts from the start, as in single-shell NMT, is likely to give a quite different result from that achieved when using NATM.

In the latter, the commonly used steel sets or lattice girders, and mesh-reinforced shotcrete and rock bolts, are all considered just as temporary support, and are not 'taken credit for' in the design of the final concrete lining. These temporary support measures are assumed to eventually corrode. It is then perhaps not surprising that convergence monitoring is such an important part of NATM, as a degree of loosening seems to be likely when so often using steel sets or lattice girders as part of the temporary support. Both are very deformable in relation to systematically bolted RRS arches. If lattice girders could be bolted there would be some improvement, but *intimate* contact with a tunnel wall and arch exhibiting overbreak remains a problem.

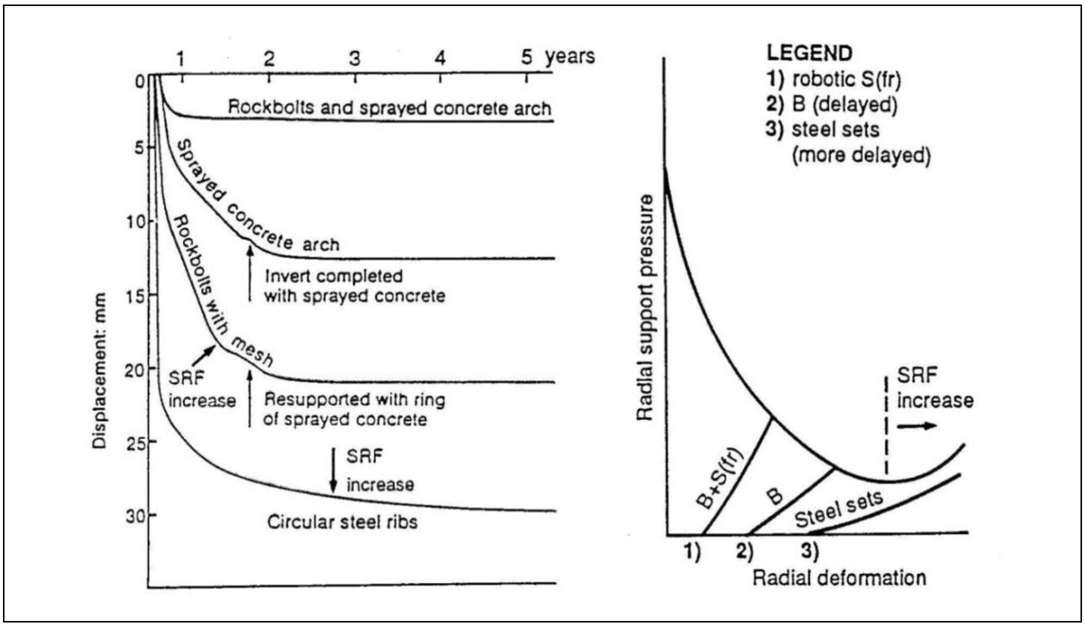
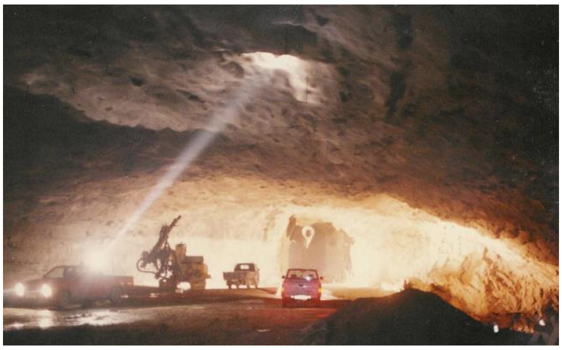



*Figure 13 Left: Results of five years of monitoring test-tunnel sections in mudstone, using four different support and reinforcement measures [13]. The obvious superiority of B+S in relation to steel sets is clear. The last 35 years of B+S(fr), as practiced in Norway would presumably give an even better result. Right: Representation of the relative stiffness of different support measures, from [14]. SRF may increase due to loosening in the case of steel sets. See Figure 14, which illustrates the implicit difficulty of controlling deformation with steel sets/lattice girders.*

*Figure 14 Left: An illustration of the challenge of making contact between the excavation periphery and the steel sets, even for the case of limited over-break. In NATM the 'sprayed-in' steel sets, and S(mr) and bolting are considered temporary, and are not included in the design of the final concrete lining. Right: Steel sets are actually a very deformable type of tunnel support. However in squeezing rock as illustrated, the application of RRS might also be a challenge, unless self-boring rock bolts were used to bolt the RRS ribs in the incompetent (over-stressed) rock that is likely to surround the tunnel in such cases.*

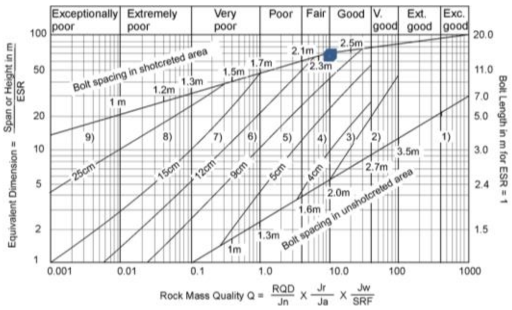
**GJØVIK CAVERN Q-LOGGING AND NMT SINGLE-SHELL B+S(fr) SUPPORT**

The Gjøvik Olympic cavern was a milestone event in Norwegian rock engineering and rock mechanics practice, combining as it did the experience of several of Norway's leading consulting, research institutes and contracting companies. The Q-system was well utilized.

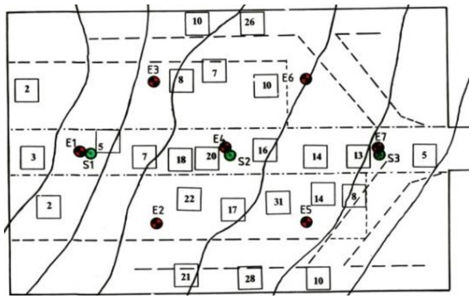


World's largest top-heading before benching-down.

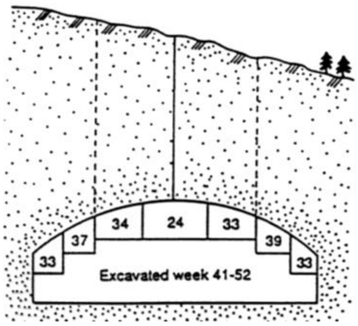
Mean 10 cm of S(fr). Note some over-break > 1 m.



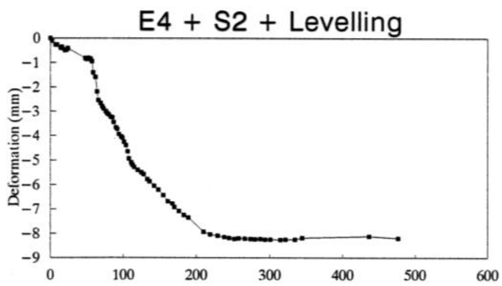

$Q_{mean} \approx 10$, span 62m, B 2.5m c/c, S(fr)10cm + cables.



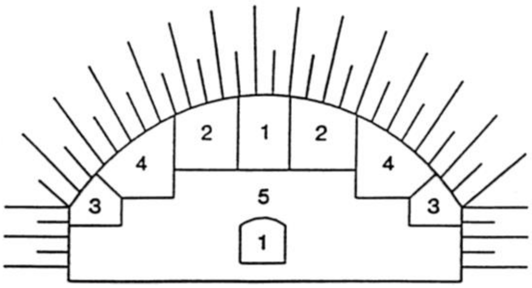

Q-values of arch (boxes): long external MPBX: red




MPBX locations (three rows), excavation week 1991.




Central deformation 8 mm, ends: 7.0 mm , 7.5 mm.



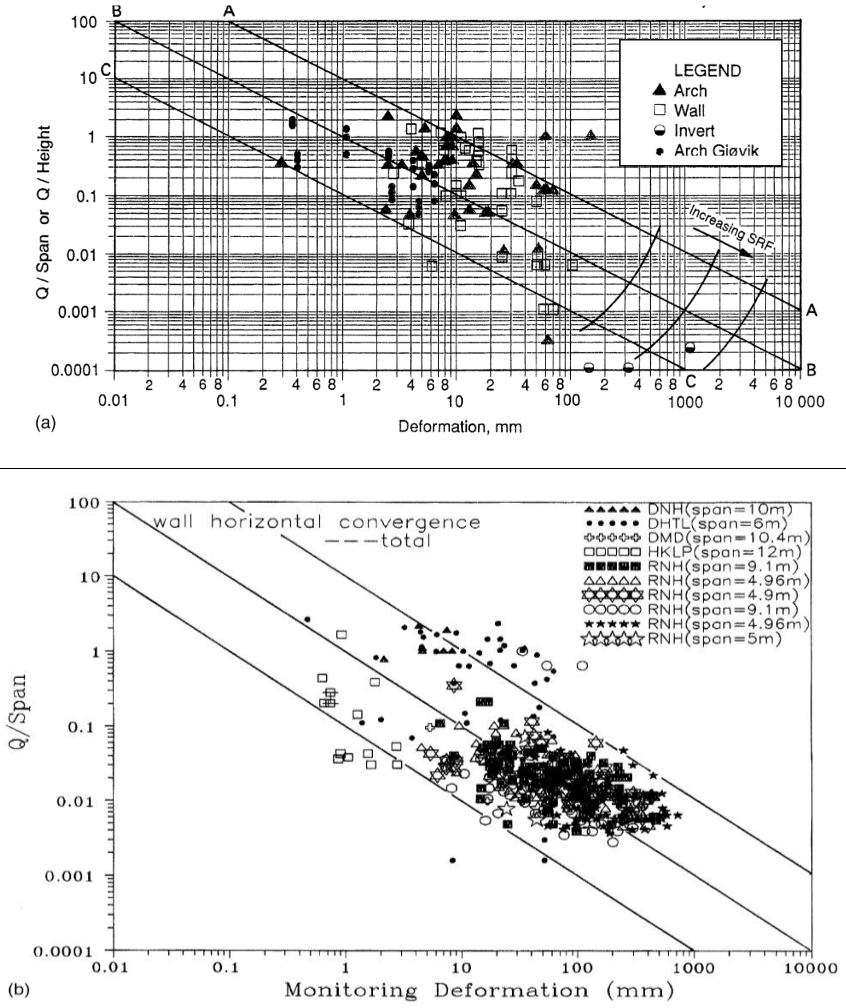

B 2.5m c/c L= 6m, twin-strand anchors 12m, c/c 5m.

Early game: ice hockey in Winter Olympics, 1994.

*Figure 15 Some details of the Gjøvik Olympic cavern. Concept from Jan Rygh, design studies by Fortifikasjon and NoTeBy, design check modelling, external MPBX, seismic tomography, stress measurements and Q-logging by NGI [15], internal MPBX, bolt and cable loads, modelling, research aspects by SINTEF-NTNU. Construction in 6 months using double-access tunnels, by the Veidekke-Selmer JV. The cavern is an example of a drained NMT excavation.*

## ESTIMATING TUNNEL OR CAVERN DEFORMATION IN RELATION TO Q

It appears that the large numerical range of Q (0.001 to 1000 approx.) referred to in the introduction, helps to allow very simple formulæ for relating the Q-value to parameters of interest to rock engineering performance. See Figure 16 for tunnel (and cavern) deformation.




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

| **Nathpa Jakri power station cavern** | **Gjøvik Olympic cavern** |
|---|---|
| $\Delta_v = \dfrac{20{,}000}{100 \times 3} \times (6/35)^{1/2} = 28$ mm<br>$\Delta_h = \dfrac{50{,}000}{100 \times 3} \times (4/35)^{1/2} = 56$ mm<br>(SPAN = 20m, HEIGHT = 50m, Q = 3, $\sigma_V$=4 MPa, $\sigma_h$ = 6 MPa, $\sigma_C$ = 35 MPa).<br>(In the middle of the range of MPBX measurements for the arch and walls). | $\Delta_v = \dfrac{60{,}000}{100 \times 10} \times (1/75)^{1/2} = 6.9$ mm<br>(SPAN = 60m, $Q_{mean}$ = 10, $\sigma_V$ = 1 MPa at 40 m depth, $\sigma_C$ = 75 MPa)<br>(Almost identical to that measured with nine MPBX, and almost identical to UDEC-BB modelling results). |

*Figure 16 Top: The log-log plotting of Q/span versus deformation was published in [15], with fresh data from the MPBX instrumentation of the top-heading and full 60 m span of the Gjøvik Olympic cavern. Shen and Guo (priv. comm.) later provided similar data for numerous tunnels from Taiwan. When investigated, the central trend of data was simply Δ(mm) ≈ SPAN(m)/Q. An empirical improvement is shown, by employing the 'competence factor' format of SRF (i.e. stress/strength).These formulæ should be tested when checking reality in numerical modelling.*

## CORRELATING Q WITH VELOCITY AND MODULUS OF DEFORMATION

An empirically-based correlation between the Q-value and the P-wave velocity derived from shallow refraction seismic measurements was developed [3] from trial-and-error lasting several years (Figures 17 and 18). The velocities were based on a large body of experimental data from hard rock sites in Norway and Sweden, thanks to extensive documentation by Sjøgren and colleagues [16], using seismic profiles (totaling 113 km) and local profile-oriented core logging results (totaling 2.85 km of core). The initial $V_P$-Q correlation had the following simple form, and was relevant for *hard rocks with low porosity*, and specifically applied to shallow refraction seismic, i.e. 20 to 30m depth, as suggested by Sjøgren.

$V_P \approx 3.5 + \log Q$ (units of velocity: km/s) (1)

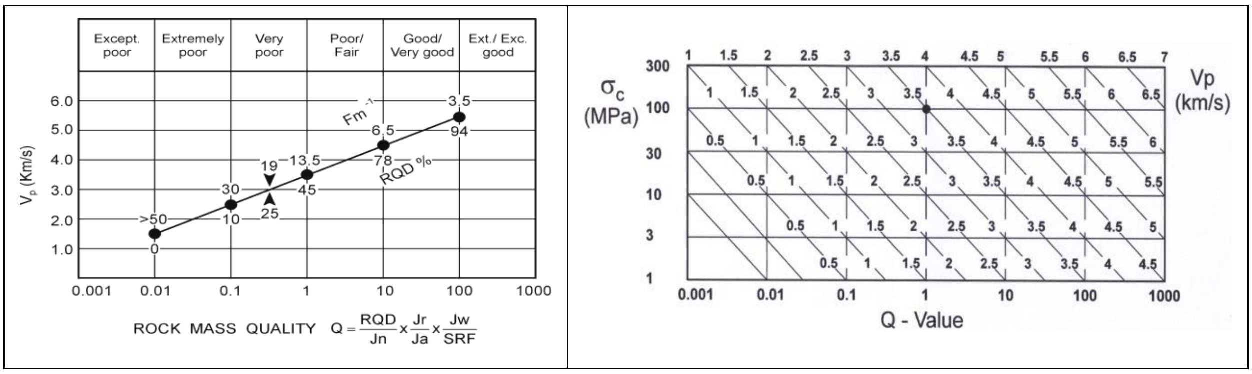



*Figure 17 Left: Hard rock, shallow seismic refraction mean trends from Sjøgren and colleagues [16]. The Q-scale was added by Barton [3] using the hard rock correlation $V_p \approx 3.5 + \log Q$. By remembering Q = 1: Vp ≈ 3.5 km/s, the Q-Vp approximation to a wide range of qualities is at one's fingertips (e.g. for hard, massive rock: Q = 100: Vp ≈ 5.5 km/s). Right: Generalization to include rock with different $\sigma_c$ values.The results still apply to shallow refraction seismic.*

A more general form of the relation between the Q-value and P-wave velocity is obtained by normalizing the Q-value with the multiplier UCS/100 or $\sigma_c/100$, where the uniaxial compressive strength is expressed in MPa ($Q_c = Q \times \sigma_c/100$). The $Q_c$ form has more general application, as weaker and weathered rock can be included, with a (-ve) correction for porosity. For a more detailed treatment of seismic velocity, such as the effects of anisotropy which are accentuated when the rock is dry or above the water table, refer to the numerous cases illustrated in [17].

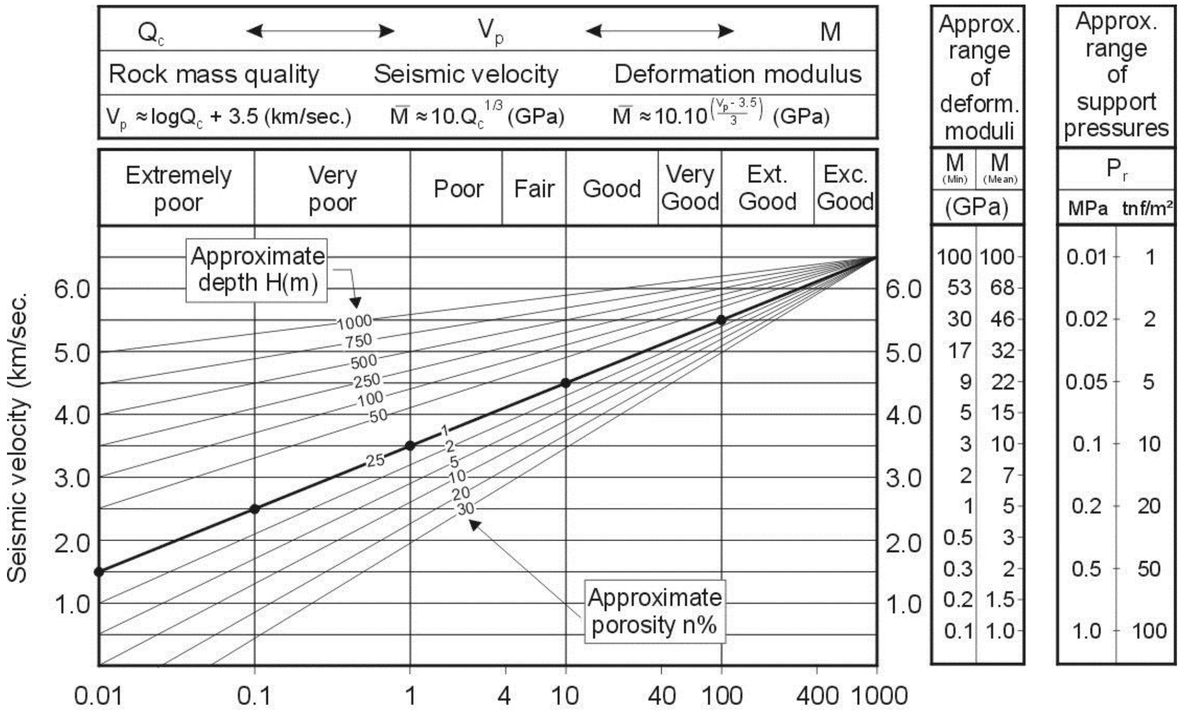
$V_P \approx 3.5 + \log Q_c$ (units of velocity: km/s) (2)

The derivation of the empirical equations for support pressure, published in [1], and for the static deformation modulus, which were derived independently in [3], suggest an approximately inverse relationship between *support pressure needs* and rock mass *deformation moduli*. This surprising simplicity is not illogical. It specifically applies with mid-range Jr = 2 joint roughness.

Further useful equations derived from $Q_c$ concern the deformation modulus $E_{mass}$. There are several possible equivalent forms [18], and $V_P$ can be used in place of Q or $Q_c$ if need be.

$E_{mass} \approx 10 Q_c^{1/3}$ or $E_{mass} \approx 10^{(Vp - 0.5)/3}$ or $E_{mass} \approx 10^{(Vp - 2.5 + \log \sigma_c)/3}$ (3a, 3b, 3c)

where Vp is in km/s, $E_{mass}$ is in GPa, and $\sigma_c$ is in MPa. For instance with Q = 10 and $\sigma_c$ = 100 MPa and Vp = 4.5 km/s, one obtains $E_{mass} \approx$ 22 GPa using all three equations (3a, 3b, 3c). This corresponds to the nominal 25 m depth (shallow seismic refraction) 'central diagonal' in Figure




$$\mathbf{Q}_c = \left[\frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}\right] \frac{\sigma_c}{100}$$

*Figure 18 The thick 'central diagonal' line is the same as the sloping line given in Figure 17a, and this applies to nominal 25-30m depth shallow seismic refraction results. In practice the nominal 1% (typical hard rock) porosity would be replaced by increased porosity if rock was deeply weathered, and the more steeply sloping lines (below the 'central diagonal') would then suggest the approximate (-ve) correction to $V_P$. Note that very jointed rock with open joints may have lower velocity than saturated soil. The less inclined lines above the 'central diagonal' represent greater depth (50, 100, 250m etc), and these lines correct $V_P$ for documented stress or depth effects (+ve). These depth-lines were derived from several sets of deep cross-hole seismic tomography, with independent Q-logging of the respective cores by NGI and Atkins. [3].*

18. If Q was unknown, a higher Vp of say 5.5 km/s (because of a deeper location) suggests $E_{mass} \approx 46$ GPa. In the Gjøvik cavern modelling with UDEC-BB [15], deformation moduli of 20, 30 and 40 GPa were modelled at increasing depth due to the increased velocity with depth. The Q-value, RQD and joints/meter had shown no improvement with depth. The measured vertical cavern deformation of 7 to 8 mm was numerically modelled very accurately, and was also confirmed empirically, which is always an important reality check (Figure 16, right inset).

## CONCLUSIONS

1. The Q-system appears to have weathered the test of time and has had application in thousands of civil and mining engineering projects in a large number of countries during the last four decades. It is most strongly linked to single-shell permanent tunnel support.
2. In relation to 'competing methods' such as RMR it appears to have the advantage of a logarithmic quality scale, and has some important parameters like number of joint sets (Jn) and inter-block friction (Jr/Ja) and ways to evaluate the stress/strength ratio (SRF).
3. The Q-value and its statistical variation has important roles to play during site investigation, core-logging, seismic velocity interpretation, support and reinforcement design

assistance, and for deciding on ‘support class’ and therefore suitable support during tunnelling. This cannot be done by ‘finite element modelling’ at 80 m/week.

4. Q has simple direct links to deformation and deformation modulus, each of which are depth dependent. It can also be linked to the realistic estimation of a depth-dependent permeability via a modified parameter $Q_{H2O}$, and to TBM prognosis via $Q_{TBM}$.

## REFERENCES


1. Barton, N., Lien, R. & Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics*. 6: 4: 189-236.
2. Grimstad, E. & Barton, N. 1993. Updating of the Q-System for NMT. *Proc. of Int. Symp. on Sprayed Concrete - Modern Use of Wet Mix Sprayed Concrete for Underground Support*, Fagernes, 1993, (Eds Kompen, Opsahl and Berg. Norwegian Concrete Association, Oslo.
3. Barton, N. 2002. Some new Q-value correlations to assist in site characterization and tunnel design. *Int. J. Rock Mech. & Min. Sci.* Vol. 39/2:185-216.
4. Barton, N. 2012. Defining NMT as part of the NATM SCL debate. *Tunnel Talk*, Sept. 2012, 4 p.
5. Barton, N. 2011. An Engineering Assessment of Pre-Injection in Tunnelling. Keynote lecture. *Indian Conf. on Rock Mechanics and Tunnelling*. Roorkee, India, 2011.
6. Holter, K.G. and Nymoen, B.2009. Permanent waterproof tunnel lining based on sprayed concrete and spray-applied double-bonded membrane. *Proc. of ITA Congress*, Helsinki,
7. Vandevall, M. 1990. *Dramix - Tunnelling the World.* NV Bækert S.A, 1991 edition.
8. Barton, N. 1996. Investigation, design and support of major road tunnels in jointed rock using NMT principles. Keynote Lecture, *IX Australian Tunnelling Conf*. Sydney, 145-159.
9. Grimstad, E. 2007, The Norwegian method of tunnelling – a challenge for support design. *XIV European Conference on Soil Machanics and Geotechnical Engineering.* Madrid.
10. Grimstad, E., K. Kankes, R. Bhasin, A. W. Magnussen and A . Kaynia, 2002. Rock Mass Q used in designing Reinforced Ribs of Sprayed Concrete and Energy Absorption” *4th Int. Symp. on Sprayed Concrete*, Davos, Switzerland.
11. Grimstad, E., K.Kankes, R. Bhasin, A. W. Magnussen and A.Kaynia, 2003.Updating the Q-system for Designing Reinforced Ribs of Sprayed Concrete and General Support. Proceedings, *Underground Construction*, London.
12. Barton, N. and Grimstad, E. 2014. An illustrated guide to the Q-system following 40 years use in tunnelling. *In press*.
13. Ward, W.H., P. Todd and N.S.M.Berry. 1983. The Kielder Experimental Tunnel: Final Results. *Geotechnique* 33, 3, 275-291.
14. Barton, N. & Grimstad, E. 1994. The Q-system following twenty years of application in NMT support selection. *43rd Geomechanics Colloquy*, Salzburg. Felsbau, 6/94. pp. 428-436.
15. Barton, N., By, T.L., Chryssanthakis, P., Tunbridge, L., Kristiansen, J., Løset, F., Bhasin, R.K., Westerdahl, H. & Vik, G.. 1994. 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. Pergamon.
16. Sjøgren, B., Øfsthus, A. & Sandberg, J. 1979. Seismic classification of rock mass qualities. *Geophys. Prospect.*, 27: 409-442.
17. Barton, N. 2006. *Rock Quality, Seismic Velocity, Attenuation and Anisotropy*. Taylor & Francis, UK & Netherlands, 729 p.
18. Barton, N. 2007. Near-surface gradients of rock quality, deformation modulus, Vp and Qp to 1km depth. *First Break*, *EAGE*, October, 2007, Vol. 25, 53-60.