

## UPDATING OF THE Q-SYSTEM FOR NMT

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### ABSTRACT

Since the early 1980s, wet mix, steel fibre reinforced sprayed concrete (S(fr)) together with rock bolts have been the main components of permanent rock support in underground openings in Norway. The concrete technology and the experience with this concept of rock support has improved considerably in this decade. Based on studies of 1,050 case records, an empirical connection has been established between the thickness of sprayed concrete and bolt spacing on the one hand and the rock mass quality, Q, on the other hand. In extremely poor rock mass quality, a concept using rebar steel reinforced sprayed concrete ribs in addition to S(fr) and rock bolts has been developed which has actually been replacing cast concrete lining during the last few years. The thickness, width and spacing between the ribs depend on the rock mass quality, Q. Rock support by means of S(fr) has also been widely used in order to prevent spalling and slabbing under high rock stresses. Use of the Q-system together with S(fr) and rock bolting as final tunnel support constitute the most important components of NMT, the Norwegian Method of Tunnelling. The article provides a detailed discussion of some improvements that have been made to the stress term SRF in the Q-system. Onset of stress slabbing in massive rock and squeezing in soft fractured rocks are more closely defined. Finally, the ability of early S(fr) support to minimise the SRF (loosening) term is noted, in marked contrast to the adverse effect of using steel sets which tend to increase the SRF value of the rock mass. Ground reaction concepts in Q-NMT support design are discussed.

### INTRODUCTION

Sprayed concrete is a product which has mainly been developed by practical application. It is one of several tunnel support techniques, and is often combined with other types of support such as rock bolts, steel straps, wire mesh, steel arches and reinforced sprayed ribs. In recent years, use of additives and an increased knowledge of concrete have made it possible to vary the properties of sprayed concrete in desired directions, in response to the planned application (Opsahl, 1982).

It is important to understand and predict the behaviour of the rock mass before deciding on the design and application of rock support. A way to approach this goal is to use a rock mass classification system. In the last two or three decades, many attempts have been made to find a mathematical solution for predicting the thickness of the sprayed concrete necessary for underground support. So far few have succeeded. The parameters controlling the behaviour of the rock mass in connection with sprayed concrete are not

sufficiently well known. Mathematical solutions generally consider only the properties of the concrete in the calculations, which to begin with are based on a uniform, circular opening which is seldom achieved in the case of a blasted tunnel. Large scale testing of sprayed concrete and a thorough investigation of all components is necessary in order to solve this problem.

There is some indication that discrete element modelling using UDEC or UDEC-BB (Cundall, 1980; Makurat *et al.*, 1990) with thin structural elements to represent the shotcrete, may be capable of answering some of the problems of shotcrete support in tunnels with overbreak. However, the special properties of the shotcrete do present certain problems of simulation.

As an alternative to idealised analyses, the Q-system of rock mass classification can be used. This system has recently been updated. It has been used to correlate the rock support with the rock mass quality. The study has been based on 1,050 new cases from main road tunnels constructed during the last 10 years, and includes widely distributed rock mass qualities between Q-values of 0.003 (*exceptionally poor*) to 200 (*extremely good*).

The poorest rock mass qualities almost always cause appreciable deformation of the tunnel periphery. In these cases, it is important to provide a temporary support which is flexible, but strong enough to increase stand-up time and prevent collapse, while allowing the rock mass to gain a new stress distribution. The final support can be installed based on observations. In Norway, the temporary support is almost always a part of the final support. Applying sprayed concrete and rock bolts gives great flexibility with respect to the amount of support since the sprayed concrete thickness, the spacing between rock bolts and the spacing and thickness of sprayed concrete ribs can be varied with the greatest of ease to suit rock conditions.

#### NUMERICAL CLASSIFICATION OF THE ROCK MASS

In order to estimate (*i.e.*, perform forward modelling of) rock support requirements, the rock mass has to be numerically described. This concept differs from NATM where descriptive rock class estimation and monitoring form the basis of final support selection. Specific differences between NMT and NATM are discussed by Barton *et al.*, 1992a.

According to the Q-system, the rock mass quality may be expressed by:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad (1)$$

The numerical value of Q ranges from 0.001 (*exceptionally poor*) to 1000 (*exceptionally good*) quality rock. The six parameters can be estimated from surface mapping and from core logging, and can later be verified or corrected during excavation. The parameters represent:

RQD = degree of jointing	}	$\frac{RQD}{J_n}$	is a measure of block size
$J_n$ = number of joint sets			
$J_r$ = joint roughness	}	$\frac{J_r}{J_a}$	is a measure of inter-block friction angle
$J_a$ = joint alteration or filling			
$J_w$ = joint water leakage or pressure	}	$\frac{J_w}{SRF}$	is a measure of the active stresses
SRF = rock stress conditions			

The rock mass quality,  $Q$ , is correlated to installed support, the result of which is given in detailed tables, or simplified as in Figure 1. The derivation of these reinforcement classes and support quantities are described in the following pages.

It may be noted here that the  $Q$ -value can also be roughly estimated from seismic velocity measurements using the equation:

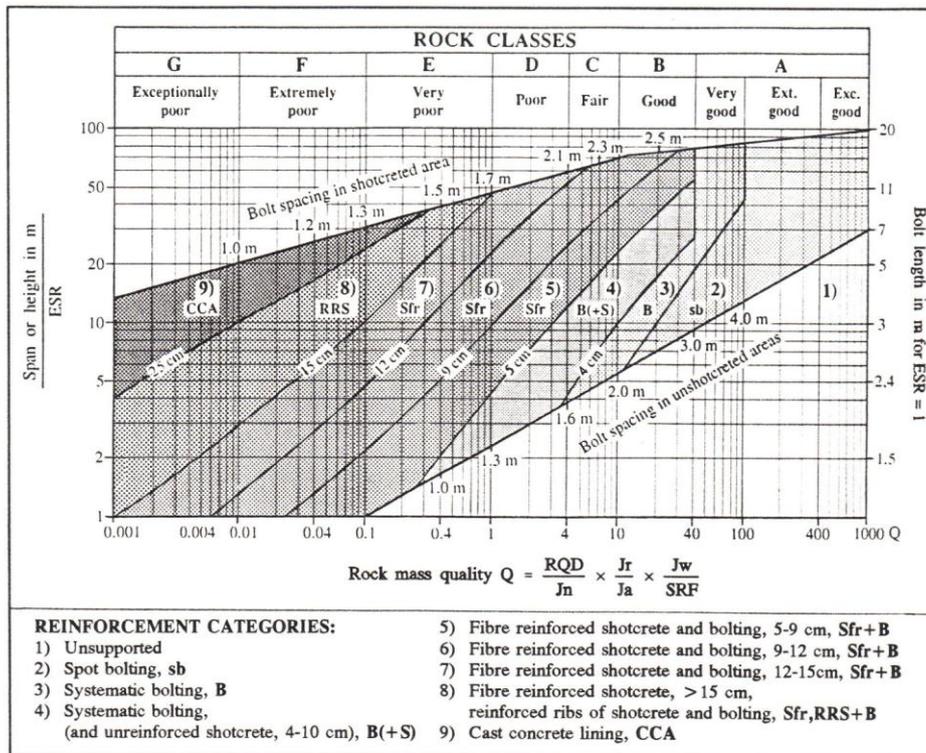
$$Q = 10^{\left(\frac{V_p - 3500}{1000}\right)} \quad (2)$$

where  $V_p$  = P-wave velocity in metres per second. For example:  $V_p = 4500\text{m/s}$  implies  $Q \approx 1$ ;  $V_p = 5500\text{m/s}$  implies  $Q \approx 10$ . For the case of fair to good quality granites and gneisses, an even better fit can be obtained using the relation  $Q = (V_p - 3600)/50$  (Barton, 1991) for velocities above 3600m/s.

#### The Stress Factor SRF in Hard Rocks

Updating of the 1974  $Q$ -system has taken place on several occasions during the last few years, and is now based on 1,050 new cases where the installed rock support has been correlated to observed  $Q$ -values. The original parameters of the  $Q$ -system have not been changed, but some of the ratings for the stress factor SRF have been altered. This was done because hard massive rock under high stress requires far more support than recommended by the corresponding  $Q$ -value. In the original 1974  $Q$ -system, this problem was addressed in a supplementary note instructing how to support spalling or rockburst zones with closely spaced, end-anchored rock bolts and triangular steel plates. Recent experience from tunnels under high stresses in hard rock now includes less bolting, but extensive use of S(fr): an unknown product when the  $Q$ -system was first published in 1974.

If the stress conditions and the compressive strength of the rock mass are known, it is theoretically possible to predict stress slabbing and the likelihood of rockburst in hard rock or squeezing in soft rock. The updating of the  $Q$ -system has shown that in the most extreme cases of high stress and hard massive (unjointed) rock, the maximum SRF-value has to be increased from 20 to 400 in order to give a  $Q$ -value which correlates with the modern rock support shown in Figure 1. Figure 2 is a histogram which shows the number of cases analysed under various stress conditions. This is the basis for the recommended new SRF-values now ranging from 0.5 to 400.



**Figure 1** Rock mass classification - permanent support recommendation based on  $Q$  and NMT. (Note extensive use of S(fr) as permanent support.)

Data from eight tunnel projects, in which stress measurements and laboratory testing of rock properties were carried out, have been used as a basis for correlating the relation between the maximum tangential stresses ( $\sigma_\theta$ ), the compressive strength ( $\sigma_c$ ), the virgin, maximum principal stress level ( $\sigma_1$ ) and the support carried out. It was found that reducing ratios of  $\sigma_c/\sigma_1$  correspond well with increases in  $\sigma_\theta/\sigma_c$ . These 8 cases were described in more detail by Grimstad (1984). Some of the tunnels referred to here are among the 1,050 cases of tunnels on which the updating of the Q-system is based.

The rating of SRF in rock affected by high stress is difficult to estimate by visual observations. If possible SRF should be estimated by the ratio between unconfined compressive strength and the major principal stress, or by the ratio between the tangential stress and the compressive strength as shown in Table 1. In countries where the stress level is seldom measured at tunnelling projects, SRF has to be classified by means of the observed behaviour of the rock and by good engineering judgement.

As can be seen from the descriptive terms in Table 1, some of the main features of visual observation can be used in order to stipulate the stress level. Furthermore, the shape of the slabs released from the rock, and whether the slabbing is affected by the orientation of the schistosity and joints or not, will tell the observer about the level and, to some extent, the orientation of the stress.

In order to further facilitate the determination of SRF, a survey has been performed with respect to the support installed in areas affected by high stresses. This field data has been plotted against the ratio between RQD and  $J_n$  as shown in Figure 3. As can be seen, no real stress problems were observed in areas with heavily jointed or crushed rock (*i.e.*, low ratios of RQD/ $J_n$ ).

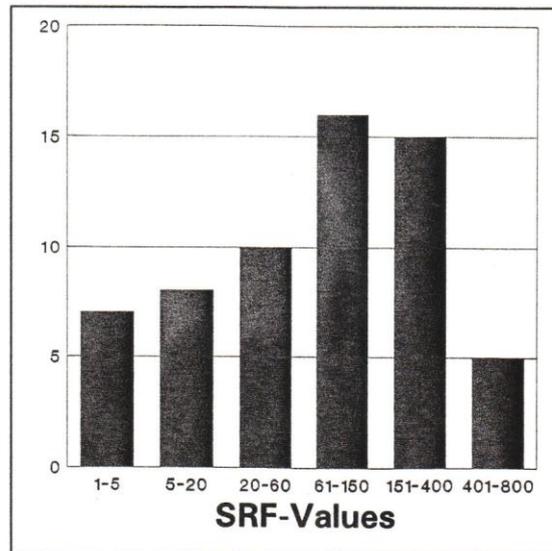
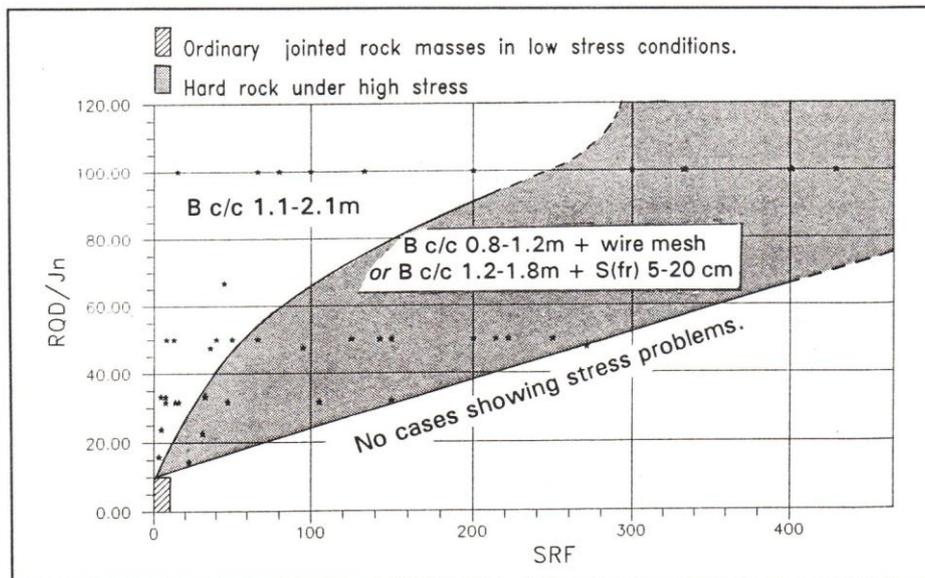


Figure 2 Frequency of estimated SRF-values from eight tunnelling projects with some extreme stress conditions.

The installed support types were differentiated according to the stress level. In the shaded area in Figure 3, wire mesh and rock bolts were mainly used in tunnels earlier than 1980, and S(fr) and rock bolts in tunnels later than 1980. In cases with low and moderate stress and little and moderate jointing, only rock bolting has traditionally been used. However, the use of S(fr) is now becoming more widespread, even in areas with minor slabbing or spalling. In all cases the thickness of the sprayed concrete has to be increased, and the spacing between the rock bolts decreased when the stress increases.

Table 1 Approximate values of SRF in relation to stress-strength ratios.

STRESS LEVEL	$\sigma_s/\sigma_1$	$\sigma_0/\sigma_c$	SRF (old)	SRF (new)
Low stress, near surface, open joints	> 200	< 0,01	2.5	2.5
Medium stress, favourable stress condition	200-10	0.01-0.3	1	1
High stress, very tight structure. Usually favourable to stability, maybe unfavourable to wall stability	10-5	0.3-0.4	0.5-2	0.5-2
Moderate slabbing after > 1 hour in massive rock	5-3	0.5-0.65	5-9	5-50
Slabbing and rockburst after minutes in massive rock	3-2	0.65-1.0	9-15	50-200
Heavy rockburst (strain-burst) and immediate dynamic deformations in massive rock	< 2	> 1.0	15-20	200-400



**Figure 3** Relationship between the ratio  $RQD/J_n$ , SRF and rock support in hard rock under high stresses.

### SRF For Squeezing Rock Conditions

Squeezing rock is a well known problem in soft rock areas. Singh (1993) has confirmed that squeezing may occur when the overburden  $H$  (in metres) exceeds  $350 Q^{1/4}$ . He has also proposed that the compressive strength of the rock mass can be expressed as  $7 \times \gamma \times Q^{1/4}$  (MPa), when  $\gamma$  is the rock density in gm/cc. If we assume  $Q$  as low as 0.01, and a rock density ( $\gamma$ ) of 2.5 gm/cc, squeezing conditions can be assumed for a tunnel depth of only 75 metres, where the effective rock mass compressive strength is about 4 MPa. At this depth the tangential stress may also be about 4 MPa, *i.e.*, the onset of squeezing conditions occurs with  $\sigma_\theta > \sigma_c$ . Values of  $\sigma_\theta/\sigma_c$  which are larger than 1 will in most cases only be found in soft rock conditions. Higher values of  $Q$  (*i.e.*, 0.1 and 1) imply onset of squeezing conditions at about 160m and 350m depth, respectively, due to corresponding predicted increases in the rock mass strength (8 and 17.5 MPa).

As in the case of massive hard rocks, very high values of SRF may be applicable in the special cases of soft rocks which have high ratios of  $RQD/J_n$  (*i.e.*, massive but soft rocks). Further case records are required before provision of specific SRF values can be made here. However, the above approach appears promising for establishing relationships between SRF and the ratio  $\sigma_\theta/\sigma_c$ .

Since crushed, squeeze-prone rock masses will tend to have low  $Q'$ -values (first five parameters) even before high values of SRF are applied, it is unlikely that the "new-SRF"

values just derived for slabbing and bursting in hard massive rocks will apply. It is probable that for squeeze-prone rock masses, SRF values in the range 5 to 10 will apply when  $\sigma_\theta/\sigma_c$  ratios are in the range 1 to 5, while SRF values as high as 10 to 20 will be applicable when the  $\sigma_\theta/\sigma_c$  ratio exceeds 5. The large deformations that occur in extreme squeezing ground (sometimes in the 1 to 3m range) mean that the highest stress concentrations are well behind the tunnel periphery. The rock mass that is most heavily stressed is in a confined state and therefore tolerates higher ratios of  $\sigma_\theta/\sigma_c$  than is the case for the "elastic" hard rock case, where  $\sigma_\theta/\sigma_c > 1$  already implies extreme difficulties due to the closeness of the highest stress to the periphery of the tunnel.

As a point of interest, borehole stability studies performed at NGI in weak porous rock simulants have shown tolerance of stress levels from 4 to 8 times higher than would be predicted by elastic theory and use of unconfined compression strength. This was due to stress redistribution and fracturing effects; in continuum terms - effective changes of deformation modulus (Addis *et al.*, 1990).

#### DETAILS OF THE NEW 1,050 CASE RECORDS

The Q-system has now been updated on the basis of 1,050 new cases from main road tunnels, and to some extent, on the basis of 440 new cases from hydropower tunnels. The data from the hydropower tunnels are, however, not very detailed with respect to sprayed concrete thickness and rock bolt spacing. In most of the cases, the support has been selected and applied by experienced engineers and contractors.

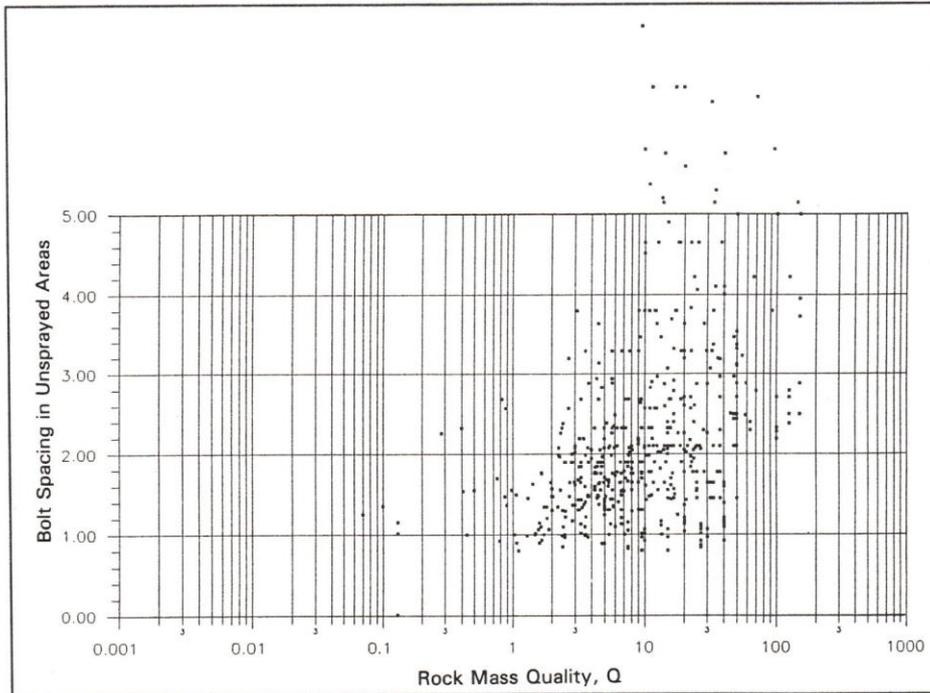
On the basis of these new case records, Figure 4 shows bolt spacing in areas **not** lined with sprayed concrete, correlated to the rock mass quality, Q. There is a large spread in the data. Some of the points (upper left) represent cave-in's or down fall of rock blocks. Some of the cases (lower right) obviously represent cases of over-support.

Figure 5 shows bolt spacing in areas **with** sprayed concrete, correlated to the rock mass quality, Q. There are fewer cases here than in unsprayed areas, because it often was impossible to count the rock bolts, and reports were not always available about the number of rock bolts in each section of the tunnel.

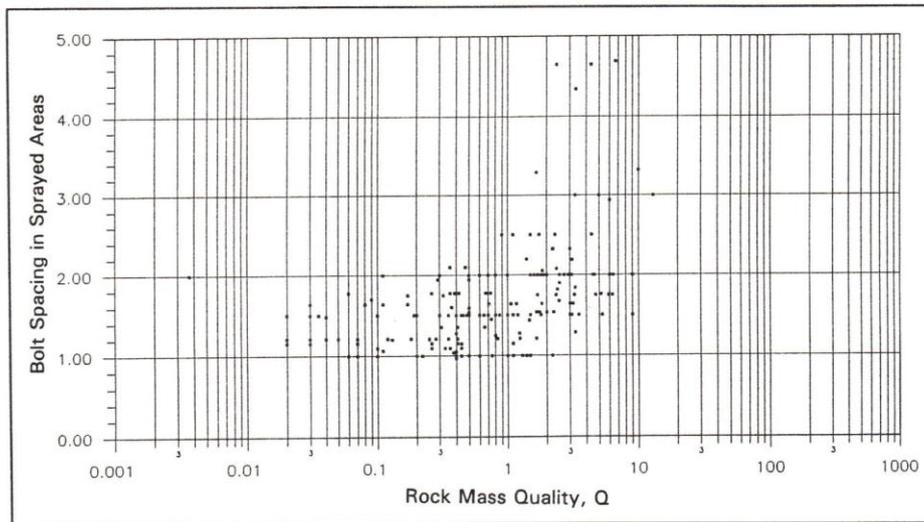
Figure 6 shows the plots of observed cases where S(fr) was used, in relation to their position in the rock mass quality diagram.

Figure 7 gives the plots of observed cases where sprayed concrete ribs, cast concrete arches (cast with steel shuttering) and cast concrete arches combined with freezing of the ground have been used, in relation to their position in the rock mass quality diagram.

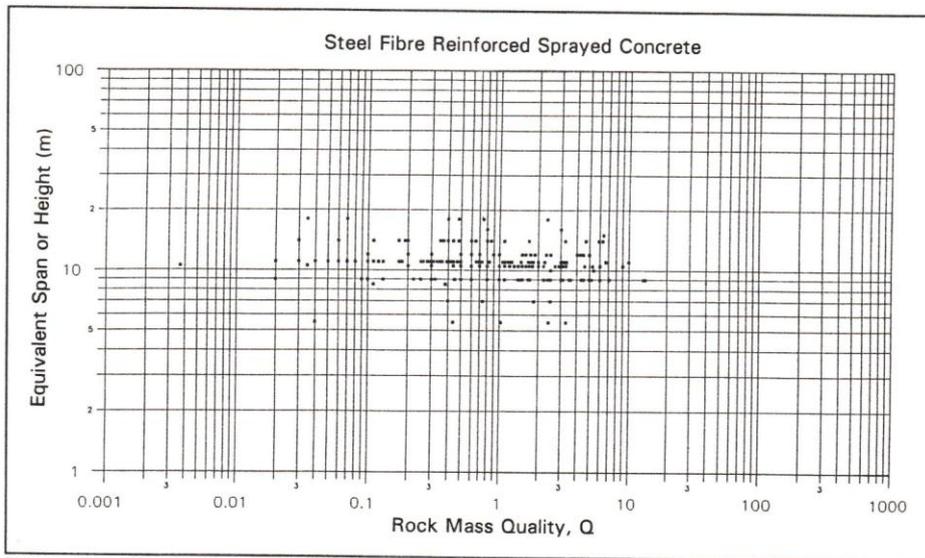
Figure 8 shows the range of S(fr) thickness (in centimetres) as a function of the rock mass quality Q. A third variable not shown in the figure is the size of the tunnels. Figure 9 gives an approximate fit to the data trend. The span-thickness-Q relation is further refined in the Q-NMT design chart shown in Figure 1, where S(fr) thicknesses are increased successively as either Q reduces or as tunnel span increases.



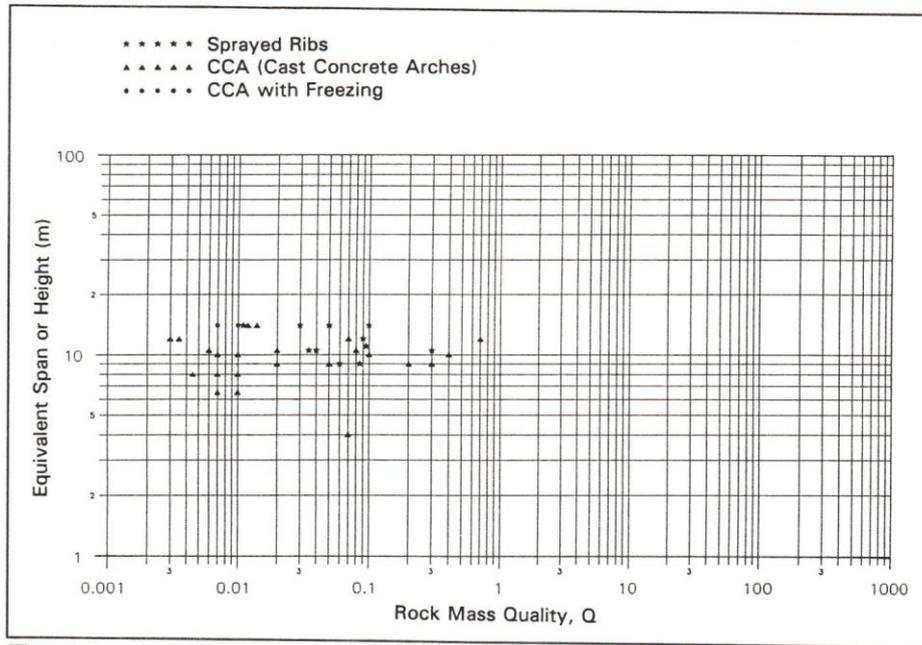
**Figure 4** Bolt spacing related to  $Q$ -value in unsprayed areas.



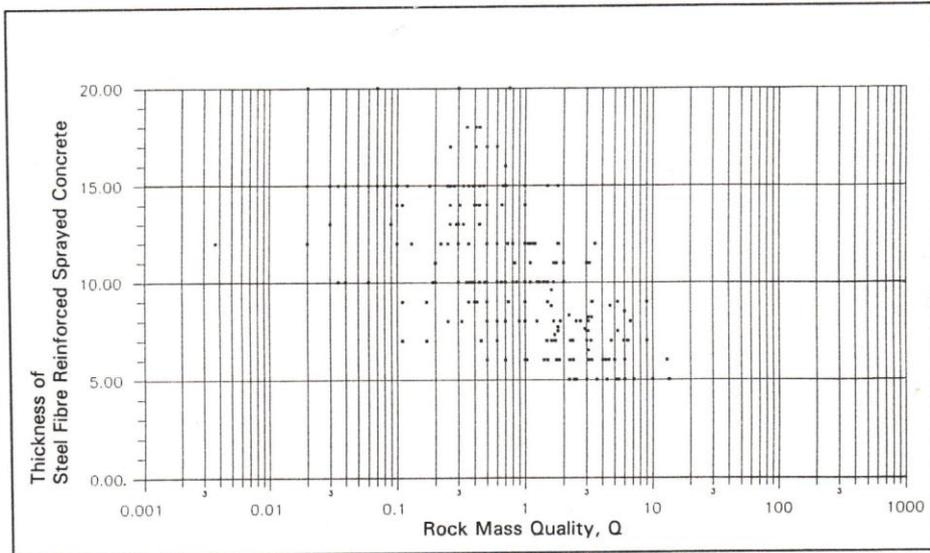
**Figure 5** Bolt spacing related to  $Q$  in sprayed areas.



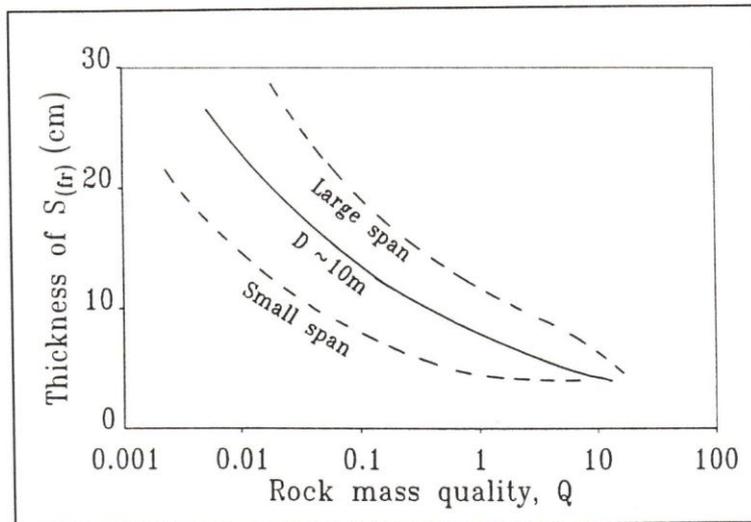
**Figure 6** Cases where steel fibre reinforced sprayed concrete has been used, in relation to the rock mass quality diagram.



**Figure 7** Cases where sprayed concrete ribs, cast concrete arches, or cast concrete arches combined with freezing have been used, in relation to the rock mass quality diagram.



**Figure 8** Case records of  $S(fr)$  showing thickness in centimetres as a function of the  $Q$ -value.



**Figure 9**  $S(fr)$  thickness as a function of tunnel span and  $Q$ -value

### Effect of Excavation Method

The distinction between a tunnel excavated by drill-and-blast and one excavated by a tunnelling machine is important. A tunnel excavated by drill-and-blast will almost always be irregular and overbreak will be controlled by joints and/or bedding planes, while a machine excavated tunnel is usually very even and smooth with a circular shape (when excavated with a TBM) or semicircular shape (when excavated by road header). However, overbreak will of course also occur in machine excavated tunnels if  $J_n$  and  $J_r/J_s$  are sufficiently adverse, *i.e.*, too many joint sets with low frictional strength.

In the drill-and-blast excavated tunnel, an ordinary thickness of sprayed concrete (*i.e.*, 5 to 10 cm) will not provide structural support like an arch, and must be supplemented by systematic rock bolting, or sprayed ribs. In machine excavated tunnels on the other hand, a rather thin layer of sprayed concrete will act as a ring or an arch, and will be able to support large forces from the surrounding rock mass. This was thoroughly tested and proven in the Kielder Water Scheme Experimental Tunnel (see Ward and Hills, 1976; Ward *et al.*, 1983).

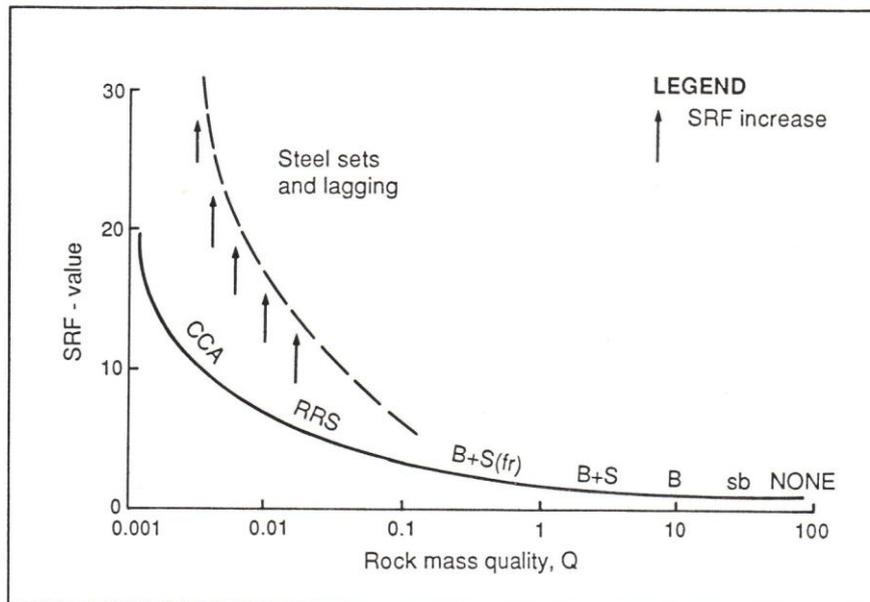
In a Norwegian road tunnel which was first excavated by TBM and later widened both downwards and towards one side by drill-and-blast, the need for rock bolts for rock support increased by 77%, and the predicted amount of sprayed concrete increased by 64% (Løset, 1992).

When evaluating the Q-value of the rock mass exposed by TBM or road header excavation, the reduced need for support in relation to drill-and-blast will be reflected in automatically higher values of Q in the mid-range of rock qualities (*i.e.*,  $Q \approx 3$  to 30). Below this range, and above this range, the rock mass will react to excavation by drill-and-blast and by TBM in a similar fashion, *i.e.*, with overbreak on the one hand, or with lack of overbreak on the other hand, and Q-value assessment will be little affected by the mode of excavation (Løset, 1992).

### GROUND REACTION CONCEPTS IN Q-NMT SUPPORT DESIGN

Steel fibre reinforced sprayed concrete (S(fr)) in combination with rock bolts offers the modern tunneller the greatest possible flexibility and control of stand-up time in difficult ground. In fact, he has the ability to *control the SRF value* of the rock mass as regards loosening or squeezing ground!

The gradually increasing support measures shown in Figure 1 as Q reduces from rock Class A to Class G also reflect a potentially increasing value of SRF. This increase in SRF is an inevitable consequence of low Q-values, however, the level of SRF increase in poor ground can be limited by suitable temporary reinforcement. This general concept is illustrated in Figure 10, and emphasises the negative consequences of steel sets for tunnel support, due to the loosening and lack of ground control that may result.



**Figure 10** SRF as a function of  $Q$ -value and rock support method in jointed rock.

The comprehensive series of tunnelling experiments in the Four Fathome mudstone (a carboniferous shale) reported by Ward *et al.*, 1976 and 1983, demonstrated very clearly the positive influence of rock bolting and shotcrete on tunnel closure in weak, highly laminated rocks. The sketches in Figure 11, which show Ward's results for the case of drill-and-blast excavation, indicate the greatly reduced deformations achievable with shotcrete and rock bolts. Obviously the advent of robotically applied S(fr) since these Kielder Tunnel experiments further emphasise the advantages of avoiding the use of steel sets where possible. However, heavy water bearing ground which cannot be drained and pre-grouted effectively may be an obvious example of a valid and necessary application of steel sets, since drainage (piping) of the worst leakage points prior to shotcreting is not always successful.

The ground reaction-support load sketch shown in Figure 11a also emphasises the potential for SRF increase (loss of ground control) with too flexible support. Clearly there will be advantages in designing the S(fr) with a variety of toughness indices and flexural strengths for tackling ground with different expected levels of deformation. When only small deformations are expected, the compressive strength of the concrete is of course more important than toughness. Some guidance in this respect can be gained from the data presented in Figure 12, which shows  $Q/\text{SPAN}$  (units of  $\text{m}^{-1}$ ) versus measured deformation (mm).

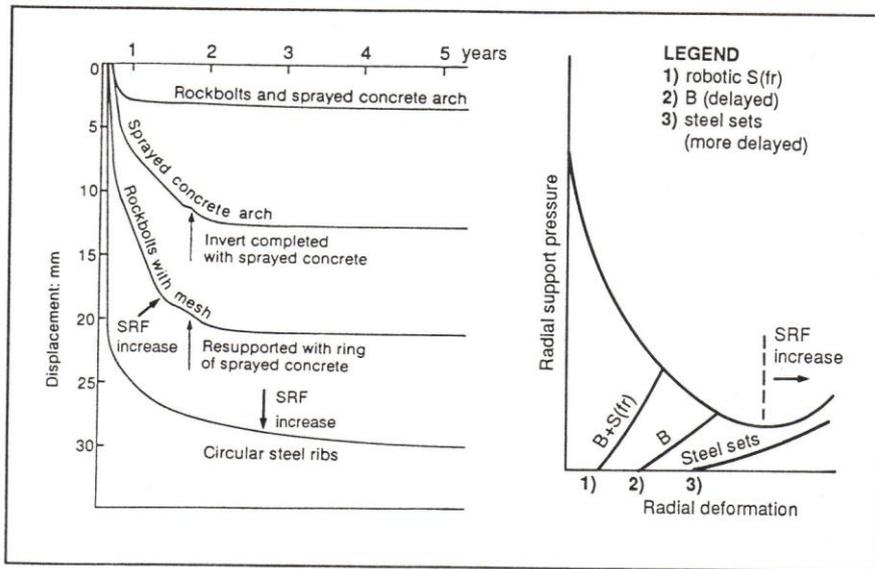


Figure 11 Left: Ward et al. (1983) experimental results in mudstones.  
 Right: Conceptual effects of early and late placement of support on SRF.

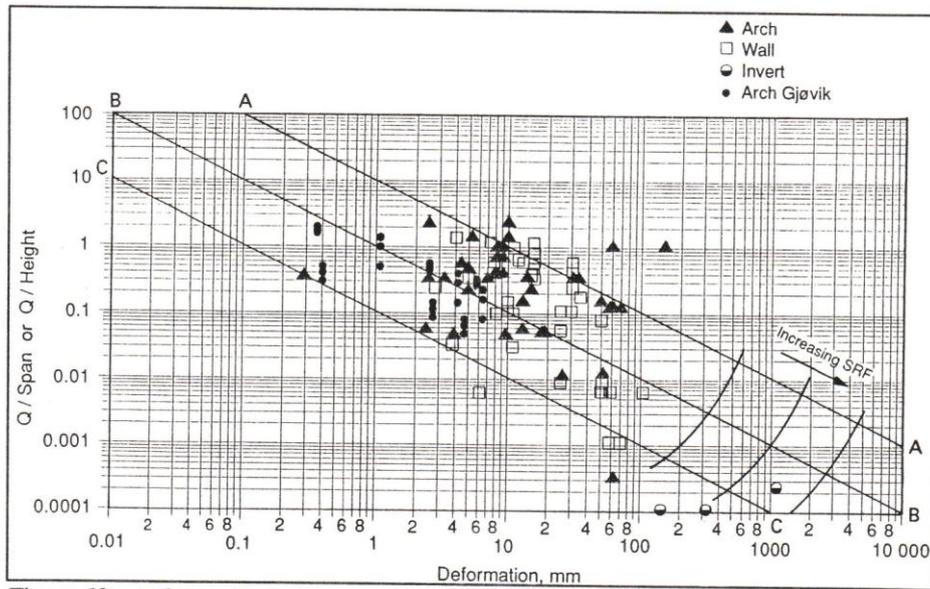


Figure 12 Relationship between measured deformation, SPAN (in metres) and Q-value.

### Deformation and Modulus of Deformation

Deformation data plotting between lines AA and BB in Figure 12 represents the most typical data which are also in general agreement with numerical analyses. Results plotting between lines B and C are an order of magnitude lower and presumably represent results for tunnels that were either instrumented rather late in the deformation process, or perhaps were over-supported. Stress conditions could also have been unusually favourable (*i.e.*, producing an arching effect near-surface in the case of high horizontal stresses). There is justification for expecting that in general terms, increasing values of SRF will be operative as the lower right hand corner of the figure is approached.

Estimation of modulus of deformation (E) (in units of GPa) can be made using the relation for  $Q > 1$ :

$$E_{(mean)} = 25 \log_{10} Q \quad (3)$$

(See Barton *et al.*, 1980.) This relation gives good agreement with measured deformations when used in numerical analyses (see for example Barton *et al.*, 1992b). However, rock mass moduli vary considerably and a range from  $10 \log_{10} Q$  to  $40 \log_{10} Q$  should be expected.

### SPRAYED CONCRETE IN Q-NMT SUPPORT DESIGN

The extensive case record data presented in Figures 3 to 9 has been synthesised and organised in a suitable form for support design in Figure 1.

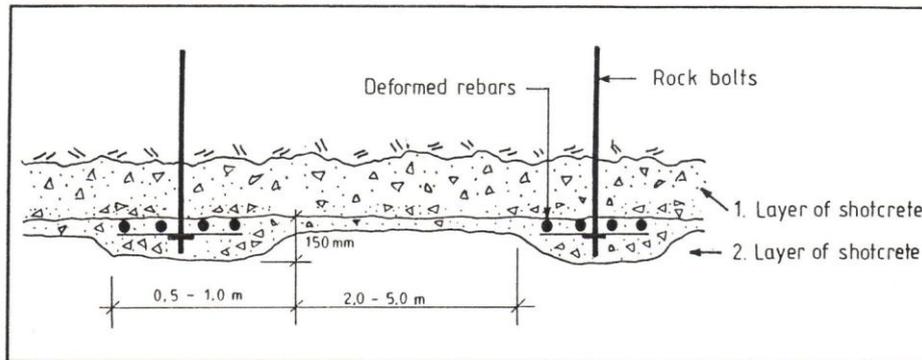
#### Spacing Between Rock Bolts

It will be noted that the spacing between rock bolts is some 20 to 40% greater when sprayed concrete is utilised than when only the rock bolts are used. The bridging effect of the sprayed concrete, particularly when fibre reinforced, is obvious.

#### Thickness of Sprayed Concrete

The reinforcement class 4 shown in Figure 1 consists of rock bolts and unreinforced sprayed concrete when the block size is small ( $RQD/J_n < 10$ ). Typical thicknesses of (S) will be 4 to 6 cm in smaller tunnels where block size ( $RQD/J_n$ ) is limited. However, in large excavations with significant wall height, it is customary to use up to 10 cm thickness, even when the rock quality Q is as high as 30.

The reinforcement classes 5, 6 and 7 consist of S(fr) varying in thickness from 5 to 15 cm, combined with systematic rock bolts. The bolt spacings given on the upper diagonal will apply in these cases. In these classes of rock mass involving significant deformation, the advantage of selecting the appropriate toughness index for the S(fr) to suit the problem needs emphasis. The same applies to the next class of support: RRS.



**Figure 13** Reinforced ribs of sprayed concrete (RRS).

### Reinforced Ribs of Sprayed Concrete

The reinforced ribs of sprayed concrete (RRS) shown as Class 8 reinforcement will be necessary when the usual thickness of S(fr) is insufficient for bearing the load, or if the shape of the blasted opening is very irregular and a more circular shape has to be built up in order to support the rock. As shown in Figure 13, RRS is an extremely flexible method in which the thickness and spacing of the ribs can be varied according to needs. The use of spiling ahead of the face and monitoring of closure will generally be an advantage in these extremely poor quality rock masses which typically have Q-values in the range 0.001 to 0.1. The contrast in ground control when using RRS instead of regular shaped steel sets and blocking is fairly clear, and the total thickness of concrete is of course potentially greatly reduced.

### Cast Concrete Arches

In exceptionally poor rock (swelling or squeezing conditions) and in larger excavations it will be necessary to use multiple drifting, spiling, pre-injection and drainage measures, and supplement the temporary RRS (or its equivalent) with full profile cast concrete arches (CCA) using steel shuttering. Depending on the amount of overbreak that has occurred prior to placement of the temporary B+S(fr), the CCA thickness is likely to vary from an average 30 cm to 1m or more locally. A stiff invert, preferably with a convex form, will be essential in this type of squeezing or swelling ground. Monitoring of the B+S(fr) or RRS temporary support before placement of the cast lining is essential.

### Support Pressure Estimation

Selection of *type* of support, and general *thickness* of support, was given in the Q-NMT design chart (Figure 1) in the beginning of this paper. Additional information on expected levels of support loads is of course prone to uncertainty, and in view of the behaviour in Figure 11, should be regarded more as the "art of tunnelling".

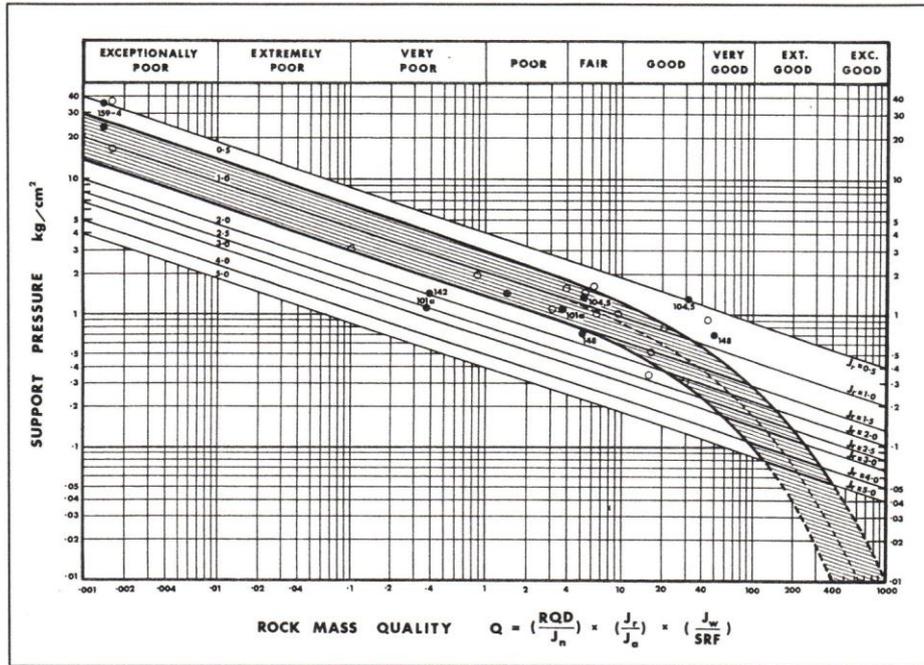


Figure 14 Support pressure estimation (Barton et al., 1974)

However, some guidance concerning support pressures is provided in Figure 14. The empirical equation which fitted the available case records in 1974 is as follows:

$$P_{arch} = \left( \frac{2.0}{J_r} \right) Q^{-\frac{1}{3}} \quad (4)$$

where  $P_{arch}$  is the support pressure in  $\text{kg}/\text{cm}^2$ . As an example, when  $Q = 1$ , and  $J_r$  (joint roughness number) is equal to 1.5, the typically designed support capacity will be 13  $\text{tons}/\text{m}^2$ . A recommendation for a given bolt spacing in Figure 1 can therefore be converted to selection of the appropriate working load for the bolts. The choice of bolt diameters is frequently 20, 25 or 32mm, with corresponding *yield* strengths of approximately 13, 20 and 32 tons respectively, when steel quality of 500  $\text{N}/\text{mm}^2$  is used.

In the case of thick RRS or CCA linings, where structural support is provided by the consistently positive (*i.e.*, non-negative) radius of the arch, then the theory of thin walled cylinders can be applied, at least in theory to help check the required thickness, assuming only compressive loading. An appropriate working stress for the concrete is needed.

An improved empirical fit to case records is obtained by a minor modification to the above equation, incorporating separate weighting for the number of joint sets ( $J_n$ ).

$$P_{arch} \leq \frac{2J_n^{\frac{1}{2}}(Q)^{-\frac{1}{3}}}{3J_r} \quad (5)$$

Taking our previous example of  $Q = 1$  and  $J_r = 1.5$ , the required support pressure will vary from **9** to **13** to **17** tons/m<sup>2</sup> as the number of joint sets increases from 2 to 3 to 4 ( $J_n = 4, 9$  and  $15$ ). This  $J_n$  correction demonstrates the importance of the degree of freedom for block and wedge fall-out: three sets marking the division between markedly reduced and markedly increased degrees of freedom for block fall-out.

An important point to note with the above equations, and with the data shown in Figure 14, is that the *size of opening* does not figure in the support pressure prediction. This supposition was made by Barton *et al.* (1974), and contrasted to the rock load estimates of Terzaghi (1946) who suggested doubling the support pressures when doubling the tunnel span (in the 5m to 10m range).

According to Singh (1993) and co-workers, convincing evidence for the independence of support pressure and tunnel size is now available from numerous cases with tunnel spans ranging from 2 to 22m in size. Singh (1993) has also suggested that the *short term support pressure* for the case of tunnels supported by steel arches can be approximated by:

$$P_{arch} = \frac{2}{J_r} (5Q)^{-\frac{1}{3}} \times f \quad (\text{kg/cm}^2) \quad (6)$$

$$\text{where } f = 1 + \frac{(H-320)}{800} \quad (\text{for } f \geq 1.0)$$

where H is the overburden in metres.

Our tunnel in ground with  $Q = 1.0$  and  $J_r = 1.5$  (as before) will therefore be subjected to short term loads of approximately 10 and 14 tons/m<sup>2</sup> for tunnel depths of **500** and **1000m** respectively. *Ultimate loads* 1.75 times higher than *short term loads* are predicted by Singh (1993) in the case of tunnels supported by steel sets and concrete lagging (*i.e.*, 17 and 25 tons/m<sup>2</sup>). This increase may well be the "SRF effect" alluded to in Figures 10, 11 and 12, *i.e.*, the use of too flexible steel sets allows the inherent SRF value of the ground to increase adversely.

As a point of interest, a five-fold increase in SRF due to unnecessary ground loosening will indeed cause the predicted support pressure to rise by a factor of about 1.7 for the case of *initial* Q-values of 0.1 and 1.0 in equation 6. Calculated *initial* support pressures of 28.7 and 13.3 tons/m<sup>2</sup> increase to 49.1 tons/m<sup>2</sup> and 22.8 tons/m<sup>2</sup> respectively as a result of an assumed five-fold increase in SRF due to unnecessary loosening. The reason that inadequately stiff steel sets sometimes buckle is clear. Use of immediate S(fr) and RRS and B could sometimes prevent such problems.

## TUNNEL DRIVING RATE WITH THE NMT

Very great advances in tunnel driving rates have been achieved in countries that have access to hydraulic bolting jumbos and robotic shotcreting rigs. A further increase in advance rate has also been noted in cases where S(fr) and B or RRS and B are used in place of cast concrete arches (Grimstad, 1981).

Tunnel advance rates appear to vary enormously from country to country, with slowest rates achieved where steel sets predominate as temporary support and where nominal concrete liners are the norm rather than the exception. A country such as Norway with more than 100 km of new tunnels driven each year, has seen some of the greatest increases in advance rates for the case of drill-and-blast tunnels, and of course some hard rock TBM's have achieved dramatic rates of advance in Norway. Drill-and-blast tunnel driving rates of 90-100m per week are not unusual for road tunnels with a 50m<sup>2</sup> cross-section. More than 400m have been driven in the best weeks of TBM advance at the Svartisen Hydroelectric project and at the Meråker Hydroelectric Project in 3.5m and 4.3m diameter tunnels.

Attempts to suggest achievable rates of advance with drill-and-blast tunnels for varying Q-values and NMT support methods are nevertheless fraught with uncertainty. The principal variable is of course the tunnel cross-section, followed by the availability of special equipment, the form of contract, manpower skill and general management of the contract.

Figure 15 shows the approximate range of driving rates that have been regularly achieved in NMT projects, with tunnels of 60 to 90m<sup>2</sup> in cross-section. A tentative range of advance rates that are apparently achieved in countries not yet having access to these tunnel support methods is also given. Again, the implication of increased SRF values when using less efficient support methods is present at the lower end of the Q-values range.

### Tunnel Support for Weakness Zones

While "design-as-you-go" is a fundamental principle for using the Q-system and NMT support methods, there are of course practical limits to the frequency of changing support. A case in point is the securing of narrow weakness zones containing crushed rock and clay.

To achieve adequate support of narrow weakness zones it is necessary to take account of the Q-value both of the zone itself (which might be as low as  $Q = 0.001$ ) and of the adjacent rock mass (which might be as good as  $Q = 1$  or more). Furthermore, the thickness of the zone, and its angle to the tunnel axis will be important. Note that in general terms, the value of  $J_r/J_a$  chosen for the zone will already have taken some account of the favourable or unfavourable orientation.

The following empirical equation has been developed by Løset (1990) for assisting the tunnel designer in choosing a practical mean Q-value for the zone and adjacent rock mass, for which support can be chosen in the usual way (using Figure 1):

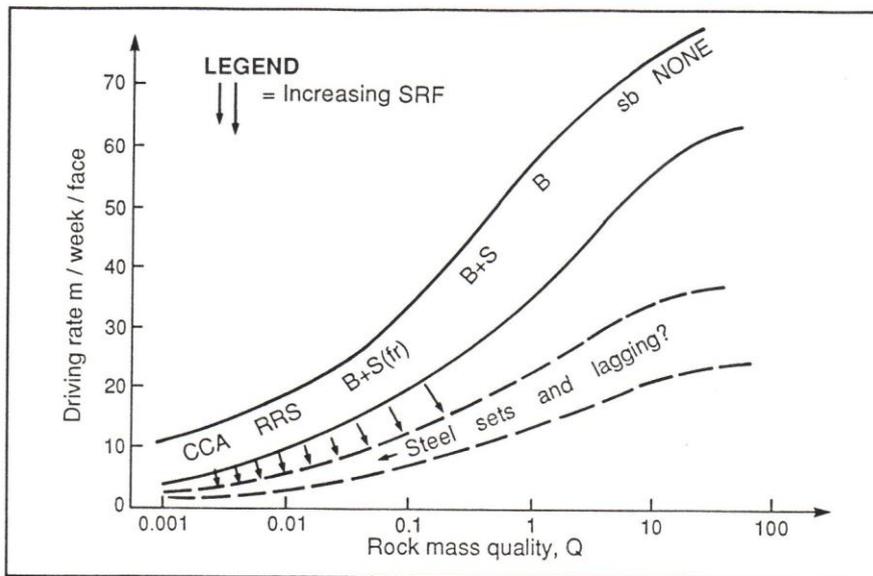


Figure 15 Approximate tunnel advance rates in metres per week per face for 70 to 90m<sup>2</sup> tunnels driven by NMT (modified from Grimstad, 1981). Curves for steel sets and lagging are tentative.

$$\log Q_m = \frac{b \log_{10} Q_z + \log_{10} Q_r}{b + 1}$$

where  $Q_m$  = mean Q-value for zone and side rock (7)  
 $Q_z$  = Q-value of weakness zone  
 $Q_r$  = Q-value of adjacent rock  
 $b$  = breadth of weakness zone (in metres) \*

- \* Use 1  $b$  for zone/tunnel axis intersection angles 90°-45°
- Use 2  $b$  for zone/tunnel axis intersection angles 45°-20°
- Use 3  $b$  for zone/tunnel axis intersection angles 20°-10°
- Use 4  $b$  for zone/tunnel axis intersection angles < 10°

Let us suppose  $Q_z = 0.01$  for the 2m wide zone and  $Q_r = 1.0$  or 10 for the adjacent rock mass. An intersection angle of 30° with the tunnel axis will be assumed. We therefore have the following two cases:

$$1. \log Q_m = \frac{4 \log_{10}(0.01) + \log_{10}(1.0)}{4 + 1}$$

$$Q_m = 0.025 \quad (\text{poor quality adjacent rock})$$

$$2. \log Q_m = \frac{4 \log_{10}(0.01) + \log_{10}(10)}{4 + 1}$$

$$Q_m = 0.040 \quad (\text{good quality adjacent rock})$$

As will be seen from this example, the extremely poor character of the assumed weakness zone causes a high level of support to be retained for the 4 metre length of tunnel to be treated with B+S(fr), or with RRS for larger spans than 5-6m. Bolting should be designed so that anchoring across the zone can be achieved in addition to reinforcement of the zone itself. "Stitching" is perhaps the terminology that best suits this requirement.

### CONCLUSIONS

1. An extensive updating of the Q-system support recommendations has been undertaken, to bring the Q-recommendations of 1974 (and 1986) in line with modern Norwegian Method of Tunnelling (NMT) support techniques.
2. Development of wet process, steel fibre reinforced sprayed concrete S(fr) in the late 1970s that could be applied by a hydraulic robot arm twenty metres ahead of the operator (over the muck-pile) has revolutionised the tunnelling environment and advance rates achieved in Norway.
3. Some changes to the sixth Q-parameter SRF have been made for the case of hard massive rocks under high stress (slabbing and rockburst categories). The choice of SRF has generally been put on a more scientific basis where possible. Loss of ground control through inappropriate or late placement of support has been demonstrated to increase the inherent SRF value through loosening and deformation effects.

### REFERENCES

- Addis, M.A., N. Barton, S.C. Bandis and J.P. Henry, 1990, "Laboratory studies on the stability of vertical and deviated boreholes", 65th Annual Technical Conference and Exhibition of the Society of Petroleum Engineers, New Orleans, September 23-26, 1990.
- Barton, N., 1991, "Geotechnical Design", World Tunnelling, November 1991, pp. 410-416.
- Barton, N., R. Lien and J. Lunde, 1974, "Engineering Classification of Rock Mass for the Design of Tunnel Support." NGI Publication 106, Oslo 1974. Rock Mechanics 6: No 4: 189-236 (1974).
- Barton, N., F. Løset, R. Lien and J. Lunde, 1980, "Application of Q-system in Design Decisions Concerning Dimensions and Appropriate Support for Underground Installations." Subsurface Space, Pergamon, pp. 553-561.

- Barton, N., E. Grimstad, G. Aas, O.A. Opsahl, A. Bakken, L. Pedersen and E.D. Johansen, 1992a, "Norwegian Method of Tunnelling", WT Focus on Norway, World Tunnelling, June/August 1992.
- Barton, N., T.L. By, P. Chryssanthakis, L. Tunbridge, J. Kristiansen, F. Løset, R.K. Bhasin, H. Westerdahl and G. Vik, 1992b, "Comparison of prediction and performance for a 62m span sports hall in jointed gneiss". 4th Int. Rock Mechanics and Rock Engineering Conf., Torino, Italy, Ed. G. Barla, pp. 17.1-17.15.
- Cundall, P., 1980, "A generalized distinct element program for modelling jointed rock", Report PCAR-1-80, Contract DAJA37-79-C-0548, European Research Office, US Army, Peter Cundall Associates.
- Grimstad, E., 1981, "Engineering-geology at the Holmestrand Tunnel" (in Norwegian) Fjellsprengningsteknikk/Bergmekanikk/Geoteknikk, 30.1-30.8, Tapir Press.
- Grimstad, E., 1984, "Rockburst Problems in Road Tunnels." Low Cost Road Tunnels. Proceedings. Tapir, Trondheim, Norway.
- Løset, F., 1990, "Bruk av Q-metoden ved sikring av smale svakhetssoner og arbeidssikring" "Use of the Q-method for securing small weakness zones and worker security", NGI internal report No. 548140-1.
- Løset, F., 1992, "Support Needs compared at the Svartisen Road Tunnel", Tunnels and Tunnelling, June 1992.
- Makurat, A., N. Barton, G. Vik, P. Chryssanthakis and K. Monsen, 1990, "Jointed rock mass modelling", International Symposium on Rock Joints. Loen 1990. Proceedings, pp. 647-656, 1990.
- Opsahl, O.A., 1982, "Steel Fibre reinforced Shotcrete for Rock support". Royal Norwegian Council for Scientific and Industrial Research (NTNF) project 1053.09511.
- Singh, B., 1993, Workshop on Norwegian Method of Tunnelling, CSMRS, New Dehli.
- Terzaghi, K., 1946, "Rock defects and loads on tunnel supports", Proctor, R.V. and T.L. White: *Rock tunneling with steel supports*, Youngstown, Ohio, Commercial Shearing and Stamping Co., 17-99. Harvard University. Graduate School of Engineering. Publication 418 - Soil mechanics series 25.
- Ward, W. H. and Hills, D. L., 1976, "Sprayed Concrete: Tunnel Support Requirements and the dry Mix Process". Shotcrete for Ground support. Proceedings. An Engineering Foundation Conference, October 1976.
- Ward, W.H., P. Tedd and N.S.M. Berry, 1983, "The Kielder Experimental Tunnel: Final Results", Geotechnique 33, no. 3, p. 275-291.