

The Q-System following Twenty Years of Application in NMT Support Selection

By Nick Barton, Ph. D. and Eystein Grimstad M. Sc.

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1994 marks a twenty-year milestone since the publication of the Q-system* (Barton et al., 1974). During this time the method has been used for the design of more than 1,000 km of tunnels in Norway alone. It has obviously matured in this time, been improved and updated, and is today being used more and more frequently as a quantitative measure of tunnelling conditions and support needs in an increasing number of countries around the world.

At the time of the Q-system development in the early 1970s, mesh reinforced shotcrete and bolting were increasingly being used as a replacement for steel sets and concrete, as a means of cutting costs, improving safety and completing tunnels more efficiently. S(mr) + B was becoming accepted in several countries even outside Scandinavia as a valid permanent support method. Of course, when no longer relying on the cure-all (but expensive) final cast concrete liner, there was a need to be sure of the adequacy of this seemingly „light“ support method. The need to describe rock mass conditions in an appropriate manner (in case concrete was needed) was the reason that the Q-system could be developed, thanks to NGI's and other people's excellent case records.

In our experience, tunnel projects where reliance was placed on steel sets for temporary support and cast concrete for final support (virtually independent of rock conditions) were poorly described in an engineering geological sense. This state of affairs is probably still true today, and tunnelling costs are still very high in many countries, partly for these reasons.

Corrosion Worries Over

A tunnelling revolution has occurred in the last 15 to 20 years with the development of wet process shotcrete and the ability to spray stainless steel fibre reinforcement S(fr) in dense, low permeability concrete of C35 to C45 MPa in situ quality. Since steel fibres are non-continuous, they do not suffer anodic/cathodic corrosion like steel mesh or steel reinforced concretes or shotcretes.

In the area of bolting, another revolution has occurred with the development of epoxy-coated and PVC-sleeved triple corrosion protection rock bolts. These can be end anchored and tensioned as temporary support, and later (after shotcreting), can be fully grouted in one simple operation both along the inside and outside of the PVC liner.

No longer can critics claim that final support consisting of S(fr) + B (steel fibre reinforced shotcrete and systematic

bolting) has limited life. Of course, as in 1974, many Owners and Consultants are still nervous of the longevity and assumed maintenance costs of „light“ permanent support methods such as S(mr) or S(fr) and bolting. However, if they follow the recommendations and methods outlined in the remainder of this paper, they will acquire many kilometres of tunnels at a fraction of the present cost!

The Norwegian Public Roads Administration, in their 450 km of main road tunnels have stretches totalling some 160 km where final support consists only of shotcrete (S) or fibre reinforced shotcrete S(fr) and bolting (Grimstad et al., 1993). Critics and conservatives may assume that this is due to the predominantly harder jointed rocks in Norway. However, S(fr) + B is not used unless rock conditions are poor or very poor (i. e., Q-values from about 4 down to 0.01). Such conditions usually involve heavy jointing, clay bearing joints and marked overbreak.

Concrete lining is only used where exceptional conditions prevail. However, it is steadily losing ground to RRS (rib reinforced shotcrete) supplemented by S(fr) + B (Grimstad and Barton, 1993). This is a flexible (easy to apply) method of building steel reinforced shotcrete ribs that are in immediate and complete contact with the whole tunnel profile. Their thickness and spacing can be varied as dictated by the ground and by convergence measurements.

Q-System Classification

Following an extensive period of trial and error in 1973, a final total of six Q-system parameters and ratings were developed as shown in equation 1 and in Table 1. According to the Q-system, the rock mass quality may be expressed by:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \dots\dots\dots [1]$$

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
- J_a = joint alteration or filling } of inter-block friction angle
- J_w = joint water leakage } $\frac{J_w}{SRF}$ is a measure
- or pressure } of the active stresses
- SRF = rock stress conditions

The large range of Q-values (six orders of magnitude) is a very important feature of the Q-system and reflects rock quality variation probably more readily than the linear

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* See Glossary of Terms at the end of this article.

Table 1 Ratings for the six Q-system parameters (updated).

1. Rock Quality Designation		RQD
A	Very poor	0 - 25
B	Poor	25 - 50
C	Fair	50 - 75
D	Good	75 - 90
E	Excellent	90 - 100

Note: i) Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.
 ii) RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate.

2. Joint Set Number		J_n
A	Massive, no or few joints	0.5 - 1.0
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four or more joint sets, random, heavily jointed, "sugar cube", etc.	15
J	Crushed rock, earthlike	20

Note: i) For intersections, use $(3.0 \times J_n)$
 ii) For portals, use $2.0 \times J_n$

3. Joint Roughness Number		J_r
<i>a) Rock-wall contact, and b) rock-wall contact before 10 cm shear</i>		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
Note: i) Descriptions refer to small scale features and intermediate scale features, in that order.		
<i>c) No rock-wall contact when sheared</i>		
H	Zone containing clay minerals thick enough to prevent rock-wall contact	1.0
J	Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact	1.0
Note: i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3m. ii) $J_r = 0.5$ can be used for planar slickensided joints having lineations, provided the lineations are oriented for minimum strength.		

4. Joint Alteration Number		ϕ_r approx.	J_a
<i>a) Rock-wall contact (no mineral fillings, only coatings)</i>			
A	Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote	-	0.75
B	Unaltered joint walls, surface staining only	25-35°	1.0
C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	25-30°	2.0
D	Silty- or sandy-clay coatings, small clay fraction (non-softening)	20-25°	3.0
E	Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays.	8-16°	4.0
<i>b) Rock-wall contact before 10 cm shear (thin mineral fillings)</i>			
F	Sandy particles, clay-free disintegrated rock, etc.	25-30°	4.0
G	Strongly over-consolidated non-softening clay mineral fillings (continuous, but <5mm thickness)	16-24°	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but <5mm thickness)	12-16°	8.0
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but <5mm thickness). Value of J_a depends on percent of swelling clay-size particles, and access to water, etc.	6-12°	8-12
<i>c) No rock-wall contact when sheared (thick mineral fillings)</i>			
KL	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6-24°	6, 8, or 8-12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	-	5.0
OP	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	6-24°	10, 13, or 13-20

5. Joint Water Reduction Factor		approx water pres. (kg/cm ²)	J_w
A	Dry excavations or minor inflow, i.e., <5 l/min locally	< 1	1.0
B	Medium inflow or pressure, occasional outwash of joint fillings	1-2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	2.5-10	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	2.5-10	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2-0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	> 10	0.1-0.05

Note: i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
 ii) Special problems caused by ice formation are not considered.

6. Stress Reduction Factor		SRF		
<i>a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated</i>				
A	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)		10	
B	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation $\leq 50m$)		5	
C	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50m)		2.5	
D	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)		7.5	
E	Single shear zones in competent rock (clay-free) (depth of excavation $\leq 50m$)		5.0	
F	Single shear zones in competent rock (clay-free) (depth of excavation > 50m)		2.5	
G	Loose, open joints, heavily jointed or "sugar cube", etc. (any depth)		5.0	
Note: i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.				
<i>b) Competent rock, rock stress problems</i>				
H	Low stress, near surface, open joints	$\sigma_c / \sigma_1 > 200$	$\sigma_\theta / \sigma_c < 0.01$	2.5
J	Medium stress, favourable stress condition	200-10	0.01-0.3	1
K	High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability.	10-5	0.3-0.4	0.5-2
L	Moderate slabbing after > 1 hour in massive rock	5-3	0.5-0.65	5-50
M	Slabbing and rock burst after a few minutes in massive rock	3-2	0.65-1	50-200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in massive rock	< 2	> 1	200-400
Note: ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1 / \sigma_3 \leq 10$, reduce σ_c to $0.75\sigma_c$. When $\sigma_1 / \sigma_3 > 10$, reduce σ_c to $0.5\sigma_c$, where σ_c = unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_θ = maximum tangential stress (estimated from elastic theory). iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).				

c) Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure		σ_g / σ_c	SRF
O	Mild squeezing rock pressure	1-5	5-10
P	Heavy squeezing rock pressure	> 5	10-20
Note: iv) Cases of squeezing rock may occur for depth $H > 350 Q^{1/3}$ (Singh et al., 1992). Rock mass compression strength can be estimated from $q = \gamma \times Q^{1/3}$ (MPa) where γ = rock density in gm/cc (Singh, 1993).			
<i>d) Swelling rock: chemical swelling activity depending on presence of water</i>			
R	Mild swelling rock pressure		5-10
S	Heavy swelling rock pressure		10-15

Note: J_r and J_a classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance (where $\tau = \sigma_n \tan^{-1}(J_r / J_a)$).

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

RMR scale. Correlation with physical parameters is probably easier to achieve because of this.

Careful study of the parameter ratings given in Table 1 will show that the only changes that have been made since 1974 are in the SRF term (Table 1, part 6b: rock stress problems, cases L, M and N and the σ_v/σ_c column. See also Note iv in Table 1, part 6c - squeezing conditions). These corrections and additions have been made so that slabbing and rock bursting cases (usually experienced in massive rocks) can also be accommodated in the Q-system support diagram (by using higher SRF numbers). Support consisting of S(fr) + B is now used for support of such conditions, which was not the case in 1974 before S(fr) was developed.

Q-Logging in Practice

A convenient way of recording the Q-parameters when logging in the field, in a tunnel, or when assessing the Q-value of drill core, is shown in Figure 1. This logging chart contains the basic ratings given in Table 1. Histograms plotting furthest to the right represent the best qualities, and to the

left, worst conditions. This chart is convenient for summarising field data. It can also be used for recording conditions core box by core box or location-by-location (1, 2, tc.) by filling in the six histograms with the location numbers within individual boxes (i. e., 1,1,1,1,1,1/2,2,2,2,2,2 etc.). Statistical trends appear quite rapidly and support the particular choice of tunnelling support method, i. e., NATM or NMT or other alternatives.

Design-as-you-drive, which is the recommended way of applying the Q-system within the Norwegian Method of Tunnelling (NMT), requires an engineering geological record of tunnelling conditions prior to shotcreting (or local concreting). In this case it is convenient to use a graphic log such as those shown in Figure 2. Note that the permanent support recommendations given in the left-hand figure are for S(mr) + B. These date from pre-1980. Fibre reinforced shotcrete S(fr) was not used commercially in Norway until 1978. The two tunnels shown were (left) a 10 m x 16.7 m headrace tunnel and (right) a 4 m span sub-sea outfall tunnel.

Q-System Support Recommendations (Updated)

The original 1974 Q-system support recommendations were arranged in the form of tables, and support was selected from one of 38 support categories, after plotting the Q-value and the equivalent span of the tunnel in a span-versus-Q diagram. Small variations in support (within a given category) were seen to be a function of the conditional factors RQD/ J_n (relative block size) and J_r/J_a (inter-block frictional strength). These factors are also worth checking when using the updated Q-system design charts shown in Figure 3.

Table 2 Summary of recommended ESR values (update) for selecting safety level.

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

The uppermost chart in Figure 3 is a very convenient introduction to Q-system support design, giving as it does the approximate distribution of final support methods as a function of tunnel span and rock quality. Note that ESR (for modifying the span) is a user's method of modifying the level of safety required. A headrace tunnel to a distant power house can tolerate occasional falls of stones (use ESR ≈ 1.6-2.0), likewise, a non-entry mining stope (use ESR ≈ 2-5). On the other hand, a very important excavation such as a main road or rail tunnel, or a power station will require „absolute“ guarantees against stone falls (use ESR ≈ 0.9-1.1). Occasionally, still lower ESR values will be used, i. e. ESR = 0.8 in the case of a major public sports hall, or ESR = 0.5 in the case of a critical sub-sea gas pipe-

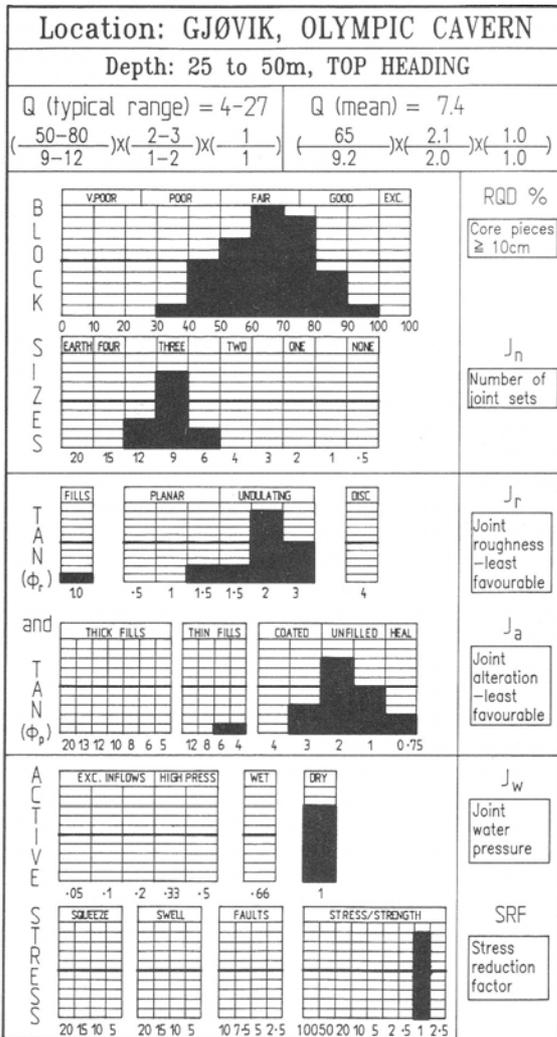


Figure 1 Logging chart for assembling Q-parameter statistics.

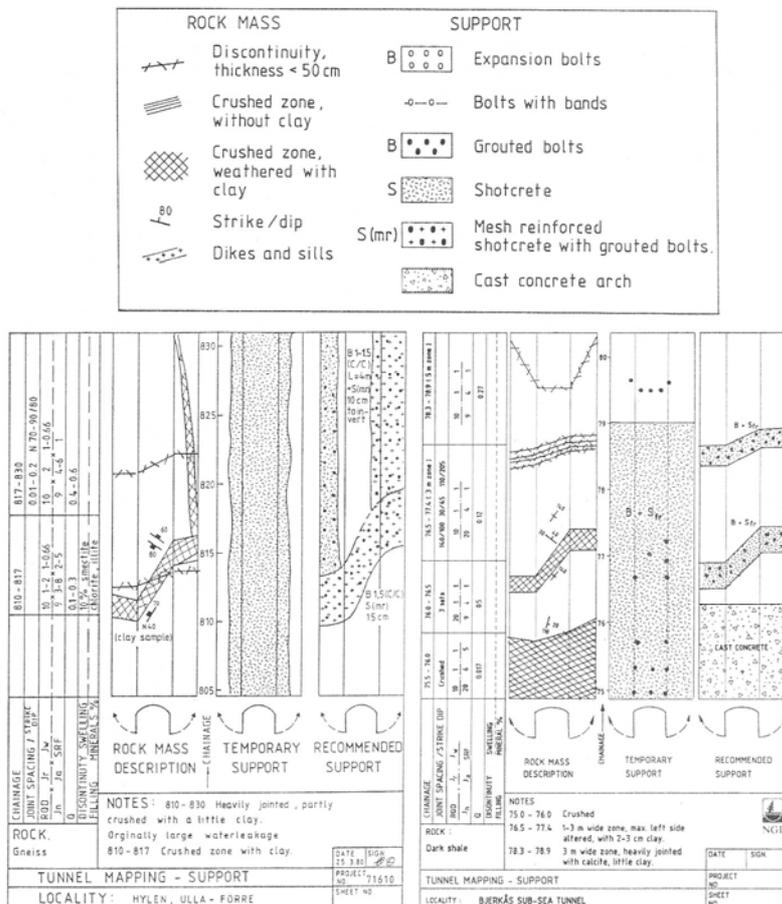


Figure 2 Graphic tunnel log for recording principal structures, Q-parameters, temporary support and the permanent support recommendation (Barton et al., 1980).

line tunnel through which a significant proportion of the national earnings flows. Values less than $ESR = 1.0$ may be regarded as very conservative and result in significant (and some would say unnecessary) cost increases.

The lower chart shown in Figure 3 is the most recent update of the Q-system, and shows the final support recommendations, following synthesis of more than 1,050 new case records from main road tunnels in Norway. Note the close specification of shotcrete thickness, bolt spacing and bolt length, which are based on a wealth of case record data and engineering experience. Although specifically developed for arch support (and from case records of such),

Table 3 Methods for selecting approximate temporary support and cavern wall support using observed Q-values.

1. Temporary Support	a) increase ESR to $1.5 \times ESR$ b) increase Q to $5Q$ (arch) c) increase Q_w to $5Q_w$
2. Wall Support (based on modified quality Q_w for walls)	a) select $Q_w = 5Q$ (when $Q > 10$) b) select $Q_w = 2.5Q$ (when $Q < 10$) c) select $Q_w = 1.0Q$ (when $Q < 0.1$)

Note 1 Use total excavation height (H) for wall support design.
Note 2 Q is the general rock quality observed when inspecting the arch or walls of a tunnel. For local variations of rock quality (arch or wall), map locally and change support as appropriate. (Q_w is not the observed value of Q in a cavern wall.)

the Q-system chart can also be used for guidance concerning suitable temporary support and for wall support.

The following factors were given by Barton et al. (1977) for selecting temporary support and wall support.

Contrasts between S(mr) and S(fr)

The revolution in the use of wet process, steel reinforced shotcrete in Norway in the last 16 years has culminated in the spraying of some 60 to 70,000 m³ per year of S(fr) in the 100 km or so of new tunnels that are presently constructed every year in this mountainous country. Modern robots can spray 10 to 25 m³/hr of S(fr) onto tunnel surfaces 15 to 20 m to the side of, ahead of, or above the operator. For obvious reasons S(mr) was no longer used after about 1984 following such a development.

Besides ease of application, the product S(fr) has several important advantages over the traditional mesh reinforced S(mr) and mostly dry process shotcreting still used in many countries. It's low rebound (5-10%), safer robotic application for support of unstable rock, and significant cost and time savings when steel mesh is no longer used means that concrete volumes are greatly reduced, making S(fr) a „must“ for economic, fast, safe and environmentally acceptable tunnel driving. Tunnel costs have actually not increased in the last

ten to twelve years in Norway for these reasons.

The enormous attraction of the NMT support method is that it is infinitely flexible in application, even more so than S(mr). Either moderately yielding or very stiff combinations of S(fr) + B can be selected for temporary or permanent support according to expected deformation levels. These can be estimated from Q/SPAN versus deformation data given by Barton et al. (1994) for numerous case records. Concrete qualities and fibre lengths determine the ability of S(fr) to „ride“ with big deformations close to the face advance, or to be stiffened as final support back from the face. Rock classification (performed pre-design and during design-as-you-drive) is the basis for a potentially conflict-free choice of final support.

The essential features of NMT and its contrasts to NATM are listed in Table 4. S(fr) is especially good compared to S(mr) when extensive jointing or clay-bearing discontinuities cause overbreak and potential instability. The tunnel profile here is often uneven, and mesh fixing is not only hazardous but causes „shadow“ when spraying, and general over-use of concrete (and time) since it cannot be „fitted“ very well to the tunnel profile.

Contrasts between NATM and NMT

When correctly used by experienced contractors and consultants, NATM must of course take credit for some re-

Table 4 Essential features of NMT (after Barton et al., 1992).

- 1. Areas of usual application:**
 Jointed rock giving overbreak; harder end of uniaxial strength scale $\sigma_c = 3$ to 300 MPa)
 Clay bearing zones, stress slabbing
 Q = 0.001 to 10 or more
- 2. Usual methods of excavation:**
 Drill and blast, hard rock TBM, machine excavation in clay zones
- 3. Temporary rock reinforcement and permanent tunnel support may be any of following:**
 CCA, S(fr)+RRS+B, B+S(fr), B+S, B, S(fr), S, sb, (NONE) (see key below and their distribution in Figure 3, bottom).
 - ▶ temporary reinforcement forms part of permanent support
 - ▶ mesh reinforced shotcrete not used
 - ▶ dry process shotcrete not used
 - ▶ steel sets or lattice girders not used; RRS and S(fr) are used in clay zones and in weak, squeezing rock masses
 - ▶ Contractor chooses temporary support
 - ▶ Owner/Consultant chooses permanent support
 - ▶ final concrete linings are less frequently used; i. e., B+S(fr) is usually the final support
- 4. Rock mass characterisation for:**
 - ▶ predicting rock mass quality
 - ▶ predicting support needs
 - ▶ updating of both during tunnelling (monitoring in critical cases only)
- 5. The NMT gives low costs and**
 - ▶ rapid advance rates in drill and blast tunnels
 - ▶ improved safety
 - ▶ improved environment

CCA = cast concrete arches, S(fr) = steel fibre shotcrete, RRS = reinforced ribs of shotcrete, B = systematic bolting, S = shotcrete, sb = spot bolts, NONE = no support needed.

markable successes (and it's share of occasional failures) when used in extremely poor soft ground, requiring a load-bearing closed ring for support. Here we are talking of soft rock or squeezing rock masses probably in the range of Q-values of 0.001 to 0.01 (i. e., exceptionally poor). In better ground than this, the typical NATM use of lattice girders, S(mr) and extensive monitoring may represent an unnecessary use of time, concrete volumes and resources; and NMT is suggested as an appropriate alternative.

For the case of tunnels showing extensive overbreak at blasting, use of S(fr) as opposed to S(mr) may cut support costs by at least 50 % and produce a more stable tunnel. However, when a smooth profile is created by machine excavation in soft rock, the use of S(mr) and the formation of a closed, load-bearing ring, is obviously still appropriate. Initial deformation might nevertheless be reduced by use of robotically applied S(fr), since the reinforcement effect is achieved earlier than with S(mr).

When NATM design is followed by a membrane and water pressure resistant cast concrete, the cost differences compared to a drained but internally dry NMT tunnel are of course greatly accentuated. There are many cases where membrane and cast concrete are more a product of conservatism than necessity, for there are a good many ways of keeping road and rail tunnels dry by combinations of pre-injection, drainage, frost-insulated water panelling, and light free-standing liner elements.

Control of Loosening Ground

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 degree of loosening (or SRF value) of the rock mass in squeezing ground. The gradually increasing support measures shown in Figure 4 as Q reduces from 1 to 0.001 also reflect a potentially increasing value of SRF. This increase in SRF is an inevitable cause of the low Q-values. However, the level of SRF increase in poor ground can be limited by suitable temporary reinforcement such as S(fr) and bolting.

The negative consequences of traditional steel sets and lagging for tunnel support are evident from the classic results of Ward et al. (1983) from the Kielder experimental tunnel. These are reproduced in Figure 5. An attempt to explain such dramatic contrasts in tunnel deformation (in the same mudstone) between modern S + B support (deformation = 3mm) and steel sets and lagging (deformation = 30mm) is given on the right hand side of Figure 5.

It is possible that the squeezing conditions frequently observed in some hydropower developments in the Himalayas, may sometimes be a function of the rock mass loosening that occurs due to less than ideal support methods. At present, relatively few of the tunnelling contractors operating in the Himalayas have shotcrete robots and modern drill jumbos. Typical temporary support consisting of steel sets and lagging may in some cases induce an earlier state of squeezing (or occurrence at shallower depth) as compared to the tunnel behaviour if S(fr) and efficient bolting could be applied.

It is of interest to note the following ranges of tunnel depths (H) reportedly required for squeezing to occur, and to see the apparent rock mass strengths (q) predicted when using Singh et al. (1992) and Singh (1993) equations.

Critical depth

$$H \geq 350 Q^{\frac{1}{3}} \dots \dots \dots [2]$$

Apparent compressive strength (of the rock mass)

$$q \approx 7 \gamma Q^{\frac{1}{3}} \dots \dots \dots [3]$$

Our interpretation of predicted conditions given in the right hand side of the table distinguishes clastic from plastic phenomena.

The updated Q-system support chart (Figure 3, bottom) used in combination with Table 1, indicates the need to use high SRF values (Table 1, part 6b) when massive, elastic rocks are under the influence of very high stresses. In such rocks, the onset of stress-slabbing occurs when σ_p/σ_c exceeds about 0.5 or 0.6.

Table 5 Prediction of depth (H) for squeezing ground, and prediction of effective rock mass strength (q) from Q-values (from Singh's equations).

Rock Class	Q range	H (m)	q (MPa)	Likely phenomenon
A	40-1000	1196-3492	62-182	strain bursting
B	10-40	754-1196	39-62	rock slabbing
C	4-10	555-754	29-39	block yielding
D	1-4	350-555	18-29	block yielding
E	0.1-1	162-350	8.4-18	crush and squeeze
F	0.01-0.1	76-162	3.9-8.4	squeezing
G	0.001-0.01	35-76	1.8-3.9	squeezing

Assume γ = density = 2.6 gm/cc in equation 3.

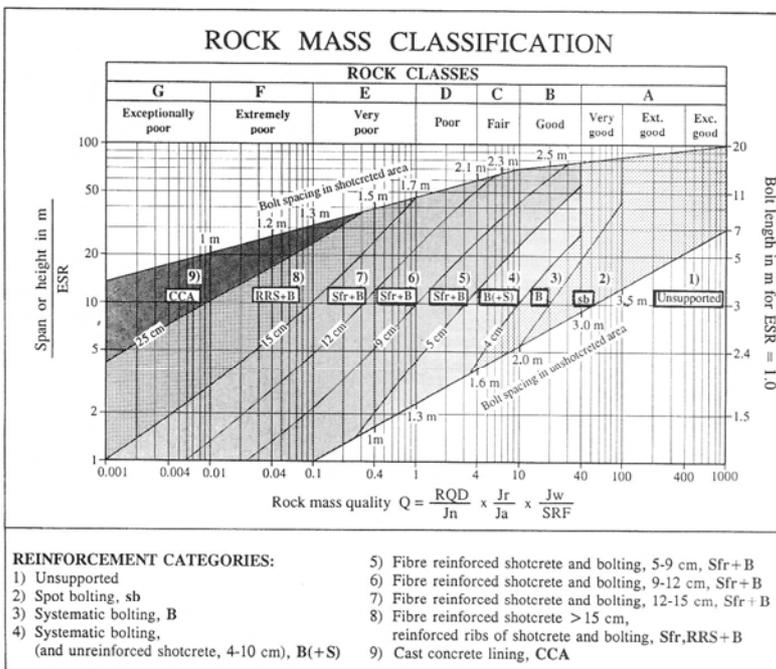
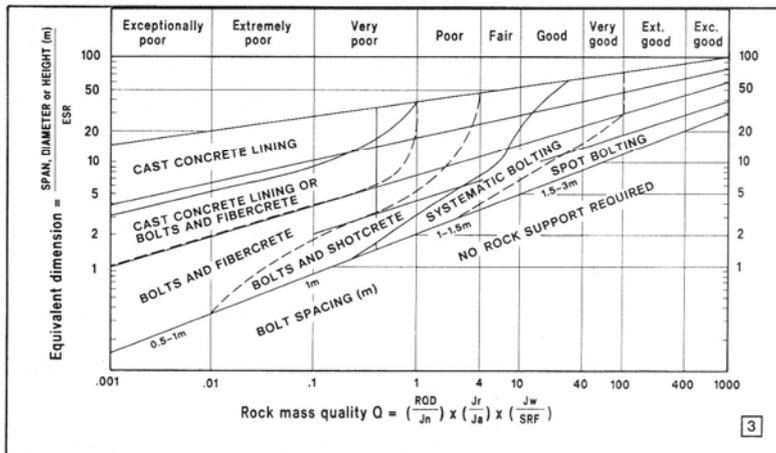


Figure 3 1986 and 1993 updates of the Q-system tunnel and cavern design charts, based on NMT permanent reinforcement principles (Grimstad et al., 1986; Grimstad and Barton, 1993).

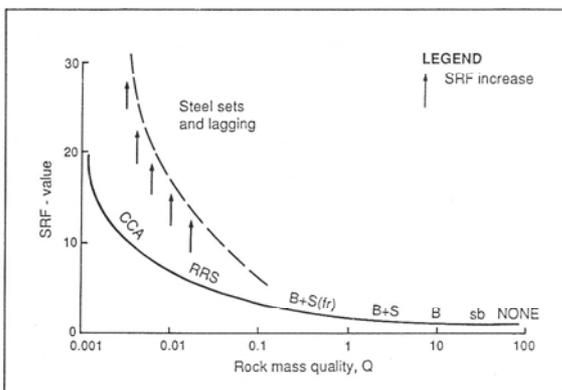


Figure 4 SRF as a function of Q-value and rock support method in jointed rock.

Since crushed, squeeze-prone rock masses will tend to have low Q'-values (first five Q-parameters) even before high values of SRF are applied, the „new-SRF“ values (Table 1, part 6b) for slabbing and bursting in hard massive rocks will not apply. It is probable that for squeeze-prone rock masses, SRF values in the range 5 to 10 will apply when σ_θ/σ_c ratios are in the range 1 to 5, while SRF values as high as 10 to 20 will be applicable when the σ_θ/σ_c ratio exceeds 5. The large deformations that can occur in extreme squeezing ground (sometimes in the 1 to 3 m 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 σ_θ/σ_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 deep-seated, log-spiral fracturing effects; in continuum terms – effective changes of deformation modulus with radius (Addis et al., 1990).

Q-Values from Seismic Surveys

As a result of cross-hole seismic testing in several countries and use of tomography for two-dimensional presentation of the seismic velocities between boreholes, it has been possible to develop an approximate correlation between the P-wave velocity V_p and the Q-value. Field data (mostly shallow, some deep) has been obtained from projects in Norway, Sweden, England, Hong Kong and China and includes fault zone breccia, clay interbeds in sandstones, siltstones, thin and thickly bedded sandstones, moderately and heavily jointed gneiss, granites and tuffs. The approximate correlation for non-porous rocks is as follows (at this stage uncorrected for depth):

$$V_p = 1000 \log_{10} Q + 3500 \text{ (m/s)} \dots \dots \dots [4]$$

$$Q = 10 \left(\frac{V_p - 3500}{1000} \right) \dots \dots \dots [5]$$

The simple, easy-to-remember form of these results is shown in Table 6.

For non-porous rocks, the deformation modulus (M) is given approximately by $25 \log_{10} Q$ (in GPa), for values of

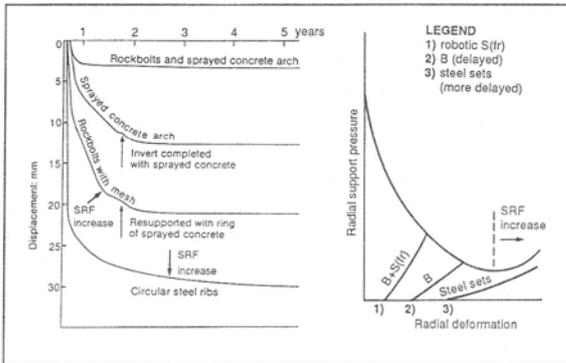


Figure 5 Left: Experimental tunnelling results in mudstones. (Ward et al., 1983). Right: Conceptual effects of early and late placement of support on SRF.

$Q > 1$. Therefore M is equal to approximately $(V_p - 3500)/40$ GPa for values of $V_p > 3500$ m/s. However, the effect of high confining pressures on M is uncertain, and of course is difficult to test at large scale.

Table 6 Approximate correlation between Q and V_p for non-porous rocks at shallow sites.

V_p (m/s)	1500	2500	3500	4500	5500	6500
Q	0.01	0.1	1	10	100	1000

Sjøgren et al. (1979) found extensive correlations between V_p and such measures of jointing as RQD (%) and F (m^{-1}) (frequency of joints per metre in drill core). His correlation between RQD and V_p for hard rocks (based on over 100 km of seismic surveys and nearly 3 km of drill core) is shown in Figure 6. In the same diagram we have indicated the $V_p - Q$ correlation and the suggestion of rock or support class divisions for a specific tunnelling project. Class 2 and 3 rock happen to be separated by a Q -value of 1.0 or a V_p value of 3.5 km/s.

The $Q - V_p$ correlation given in Table 6 is shown in more complete graphical form in Figure 7. An attempt has also been made to correct the relationship for depth (or stress) effects and for rock porosity, both of which can have major influence. More data is required to refine the trends shown in Figure 7. However, it may represent a useful starting point for relating rock quality (in its broadest tunnelling sense) to seismic velocity. The effect of high confining pressures on V_p is clear. A marked effect on deformation modulus (M) is expected but needs more data.

Application of Q-System for TBM Projects

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., $Q = 3$ 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).

Overbreak occurs in a TBM tunnel when, for example, J_n is too high (too many joint sets) or when J_r/J_n is too low (too low friction), or when water pressure is also acting together with these factors. This was typically the case in the Channel Tunnel driven in chalk marl, in some of the early kilometres driven from the UK side. Of course if overbreak can be controlled (by bolting through trailing fingers immediately behind the TBM, or by pre-cast concrete ring building within a tailshield) then a thin circular liner will be able to support large forces, whether this is shotcrete or concrete elements.

Recent developments of „open“ hard rock TBM's with bolting and shotcreting facilities at two or more locations behind the drill head makes the use of rock classification methods and probe drilling extremely relevant for optimising the tunnel support. Some of the deep base TBM tunnels planned beneath the Alps may not be supportable with concrete elements due to the non-uniform and unnecessarily high peripheral loading attracted by such elements. Very large rock loads may need to be distributed further into the rock mass by controlled deformation and heavy rock bolting.

In such cases the use of probe drilling and seismic velocity sondes as illustrated in Figure 8 may be an alternative method of predicting rock qualities (and of course, water problems) 100 m or more ahead of the machines (i. e., drilled during the maintenance shift). The measured distribution of seismic velocities after due correction for depth/stress effects could be converted to Q -classes (as in Figure 6).

The proposed method of applying the V_p - Q -NMT design-as-you-drive principles is illustrated (in idealized form) in Figure 9. Several large open-mode, hard rock machines with bolting and shotcreting stations are now available. In the example we have assumed that „Class 4“ rock (i. e., $Q = 0.1$ - 0.4) is currently being drilled through. The TBM-modified NMT support recommendation is $S(fr) = 120$ mm + B 1.5 m c/c for „Class 4“ rock. This is obtained by taking the lowest values from a general support recommendation range of $S(fr)$ 120 → 150 mm + B 1.3 → 1.5 m c/c for the „Class 4“ range of 0.1-0.4 for a 10 m diameter tunnel. The „lightest“ support is taken out of respect for the fundamentally positive, circular TBM tunnel profile.

The recommended final support is applied in convenient stages having regard both for the number of holes that can be drilled during one stroke of the TBM and the preference

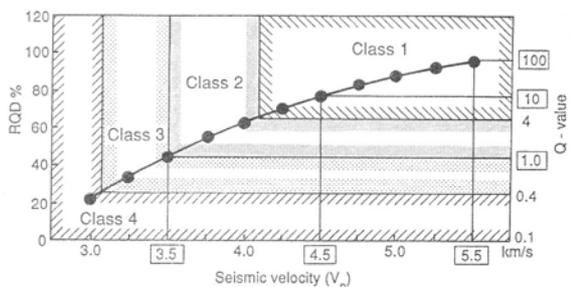


Figure 6 Relating support class to seismic velocity via the Q -value. The RQD - V_p relationship is from Sjøgren et al. (1979).

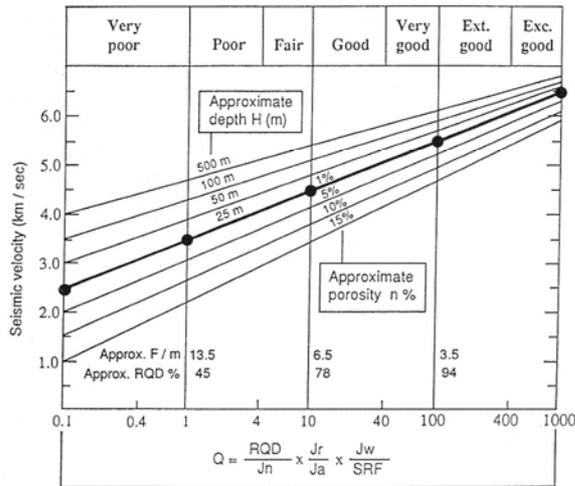


Figure 7 V_p - Q trends corrected for depth and porosity. Approximate RQD and F values are from Sjøgren et al. (1979).

for shotcreting (if possible) somewhat back from the drill head, i. e., at station B. Monitoring of convergence is designed to confirm the rock class, or to suggest correction to an adjacent rock class with application of more (or less) support at the next B + S(fr) station.

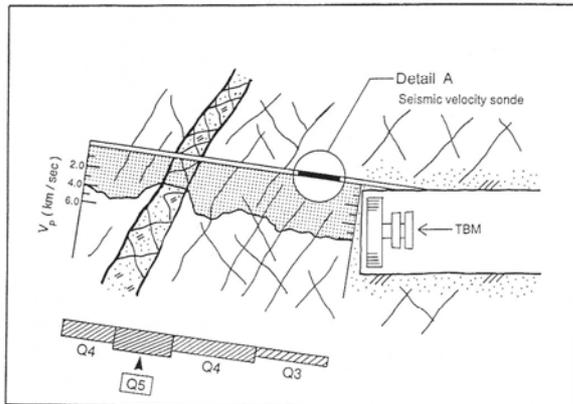


Figure 8 Sonic logging of TBM probe hole for preliminary estimate of Q-value from V_p .

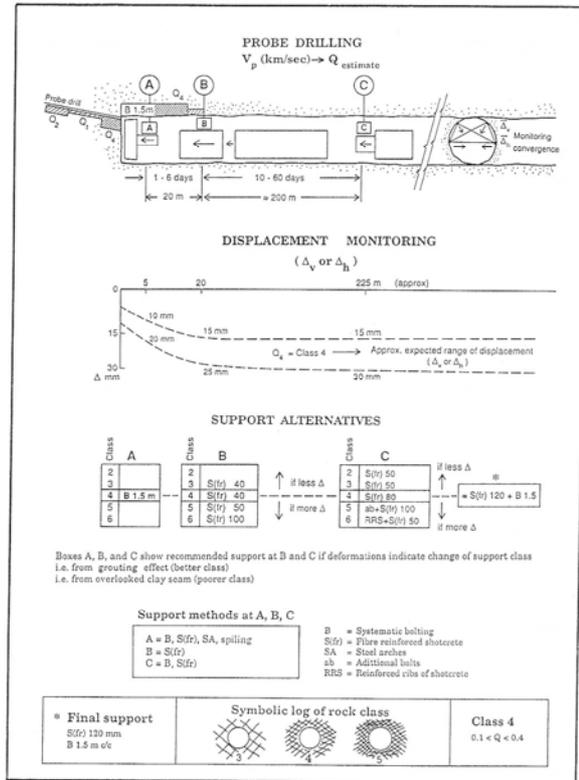


Figure 9 Design-as-you-drive support estimation for a hard rock TBM based on V_p probing, Q estimation, NMT support and monitoring of convergence.

Conclusions

1. The original Q-system tunnel support recommendations of 20 years ago were mostly based on mesh reinforced shotcrete and bolting as final support. The recently updated support chart, which is based on 1,050 new case records from main road tunnels, is based on the Norwegian Method of Tunneling (NMT) support principle where wet process steel fibre reinforced shotcrete S(fr) and fully grouted, corrosion protected bolting are the principal components of final support.
2. The combined use of a quantitative classification system (Q) and the infinitely flexible NMT support techniques are considered to be superior to NATM for the case of jointed

and clay bearing rock masses that give marked overbreak upon excavation. The use of S(mr) and lattice girders that nicely conform to a uniform machine excavated profile in soft rocks, will not often be appropriate in drill and blasted tunnels, due to the greater use of concrete they will cause where overbreak is present. In NMT, structural support can be built up more efficiently by rib reinforced shotcrete (RRS), which is readily fitted to an uneven profile by appropriate spraying of shotcrete ribs at any desired spacing.

3. The use of seismic velocity probing ahead of tunnels, in particular TBM tunnels, is considered to be a promising method for predicting rock mass quality, following development of a correlation between V_p and Q. Design-as-you-drive with the Q-system (with convergence monitoring when $Q < 0.01$) is likely to provide the tunnel with the support most likely to match the rock conditions, and therefore represent minimum cost to the Owner.

Summary

Quantitative description of rock mass quality and selection of rock reinforcement using the Q-system has been an important component of the Norwegian Method of Tunnelling (NMT) for many years. It contrasts with the descriptive classification used in the New Austrian Tunnelling Method (NATM) and the requirement for monitoring that are integral parts of the final support used in soft ground, machine driven tunnels. The Q-system has recently been updated with some 1,050 new case records mainly from main road tunnels, which adds to the reliability of the updated recommendations for permanent bolting and fibre reinforced shotcrete lining. The combination of Q-logging and NMT reinforcement techniques is ideally suited to tunnels driven in a wide range of jointed and faulted rocks with compressive strengths ranging from about 3-300 MPa. Applying the Q-system as a basis for selecting NMT results in optimal tunnel design, with the permanent support probably more closely related to the tunnel's reinforcement needs than other methods.

Glossary of Terms

Q	= rock mass quality number
Q-system	= support design method based on Q-value
NMT	= Norwegian Method of Tunnelling
S(mr)	= mesh reinforced shotcrete
S(fr)	= steel fiber reinforced shotcrete
B	= systematic bolting
PVC	= poly vinyl chloride (bolt sleeve)
RRS	= rib reinforced shotcrete
RQD, J_n , J_r , J_b , J_w , SRF	(see definitions in text)
σ_θ	= tangential stress
σ_c	= uniaxial compression strength
ESR	= excavation support ratio (see Table 2)
Q_w	= wall quality (modified Q-value)

(All terms are defined in the text or in relevant tables)

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