

Investigation, Design and Support of Major Road Tunnels in Jointed Rock using NMT Principles

Nick Barton, NGI, Oslo, Norway

Summary When fibre reinforced shotcrete, S(fr), and rock bolts form the key components of permanent rock reinforcement and tunnel support and are not followed by concrete lining, then the investigation, design and tunnel support phases have each to be relied upon to a greater extent than is the case with typical NATM tunnelling. The Norwegian Method of Tunnelling (NMT) which can be used for a very wide range of jointed and faulted rock, places reliance on rock mass classification, on empirical design of permanent support, on numerical verification of special cases and on the knowledge or assumption that a flexible approach to rock support variation will be possible within the contract. "Design as you drive" or "in situ selection of support" presupposes anticipation and designs for the full range of rock conditions, and unit prices for all the tunnelling and support methods likely to be used. Tunnelling and support costs in the range of US\$4,000 to US\$8,000 per metre are normal in Norway for two-to-three lane highway tunnels using these NMT principles. The article demonstrates the use of the Q-system and correlations with seismic investigation methods for anticipation of the likely range of tunnelling conditions. A look at the Sydney basin sandstones is used to demonstrate this method. Numerical verification of empirical support designs is demonstrated with UDEC-BB and UDEC-S(fr). Finally, some details of NMT permanent support components are illustrated including corrosion protected rock bolts, almost rebound-free fibre reinforced shotcrete and economic frost and water insulation methods.

1 INTRODUCTION

In the context of road and rail tunnels, NMT (Barton et al., 1992) is a collection of practices that produce dry, drained, permanently supported and "lined" (fully cladded) tunnels for approximately US\$4,000 to US\$8,000 per metre. These low-cost, high-tech Norwegian tunnels may range in cross-section from about 45 m² to 110 m². The following list gives the essential components of NMT.

1.1 Design

- Preliminary design is based on field mapping, drill core logging and seismic interpretation.
- Rock mass quality is usually described by the Q-value (Barton et al., 1974; Barton and Grimstad, 1994).
- Final support is selected during tunnel construction based on tunnel logging and use of the Q-system support recommendations.
- Numerical verification of the various permanent support classes may be performed with the distinct element (jointed) two-dimensional UDEC-BB or three-dimensional 3DEC computer codes.

- A basic NMT designed tunnel is drained, with insulated, pre-cast concrete panels for water and frost control when needed. These can be assembled at approximately 1 km per month.

1.2 Contractual

- The Owner pays for technically correct support.
- The Contractor is compensated *via* the unit prices quoted in the tender document.
- The Owner bears more risk than the Contractor, thereby reducing prices.
- Needed support is based on the agreed Q-value, and may vary frequently.

1.3 Excavation and Support

- Excavation, usually by drill and blast, is tailored to the rock conditions.
- The temporary support such as sb, B or B+S(fr)¹ is approved as part of the permanent support.

¹ sb = spot bolts; B = systematic, fully grouted bolting; S(fr) = wet-mix, steel fibre reinforced shotcrete

- The permanent support class is chosen during tunnel advance.
- The permanent support usually consists of high quality wet process, fibre reinforced shotcrete applied by high capacity robot, and fully grouted, corrosion protected rock bolts. These may be supplemented by rib-reinforced shotcrete (RRS) when very poor conditions are encountered. Concrete lined sections through fault zones and swelling clay are infrequent.
- Verification of shotcrete and bolting using numerical modelling.

2 ROCK MASS CHARACTERISATION

The Q-system of rock mass characterisation was developed in the early 1970s (Barton et al., 1974) before the advent of fibre reinforced shotcrete. Tunnel reinforcement and support recommendations given by the Q-system have been updated in the last twenty years (Grimstad and Barton, 1993) in order to incorporate the last sixteen years of S(fr) technology and experience. The basic rock mass characterisation for obtaining the rock quality Q-value is almost identical to that used twenty years ago as shown in Table 1. Only in the area of rock stress problems have improvements been made.

In relation to the well known and sometimes misused terminology NATM (New Austrian Tunnelling Method):

- NMT uses a predictive classification for support design. (NATM uses monitoring for support class selection.)
- NMT gives the permanent support which *is not* followed by concrete lining. (NATM gives the temporary support, which *is* followed by concrete lining.)
- NMT uses high capacity (10 - 25 m³/hr) robotically applied wet-mix, steel fibre reinforced shotcrete. (NATM uses hand placed steel mesh, and usually dry mix shotcrete which is often applied by hand held equipment.)

Due to the low rebound of wet-mix S(fr) (usually 4 to 6%), and the lack of a water pressure resistant concrete lining, the amounts of concrete used in NMT and NATM tunnel differs greatly. Consequently, the cost differences may be dramatic (NMT ≈ 1/2 to 1/5 times NATM) and construction times are also affected significantly. Furthermore, due to the mechanisation used in the NMT (robot shotcrete rigs, and nowadays also computer steered drill jumbos) the number of personnel regularly working at or near the face is usually only three in any given shift.

Figure 1 summarises some of the essential features of NMT. In this introductory article on NMT we will take a brief look at the items listed in the upper four drawers in the NMT design desk (Figure 1):

- Rock mass characterisation.
- Choice of suitable reinforcement using the Q-system.
- Relationships between rock quality and seismic velocity.

The rock mass quality number, or Q-value, ranges over six orders of magnitude (0.001 to 1000) and is a fair reflection of the huge range of tunnelling conditions found in nature (i.e., from swelling or squeezing rock and clay to massive, unjointed rock). Rock mass deformation moduli (M) may vary by as many as three orders of magnitude, i.e., 0.05 to 50 GPa, and shear strengths (S) of controlling joints or discontinuities by at least two orders of magnitude, i.e., 0.1 to 20 MPa at relevant tunnel depths. When multiplied, these parameters M and S give a rough measure of the rock quality which actually resembles the Q-value. Since both these components contribute, in a cumulative manner, to tunnel stability problems, the great numerical range of Q-values is considered realistic and advantageous.

To illustrate this we could consider the product of the minimum values (0.005) for a hard fissured clay (Q ≈ 0.01) and the product of the maximum values (1000) for a massive, sparsely jointed rock mass (Q ≈ 500).

The Q-value is formulated by the six parameters (and their empirical ratings) given in equation 1.

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

These six items and their ratings are given in Figure 2. This figure shows the numerical range of each parameter (i.e., $J_n = 0.5$ to 20) and also gives a brief description of each category (i.e., one, two or three joint sets have J_n values of 2, 4 or 9, respectively). Histograms plotting to the right hand side represent best qualities in each case.

Since rock mass conditions are rarely if ever constant from place to place, even within the same lithology, it

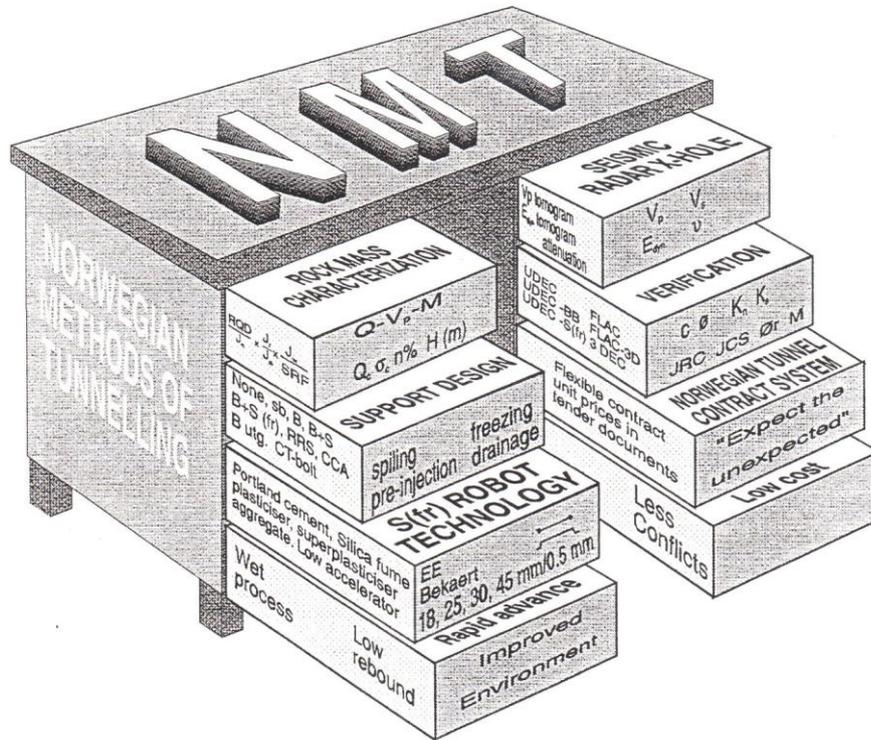


Figure 1 NMT Design and Execution

is convenient and correct to record the statistical variation. The example in Figure 2 is from the 10 m span top heading of the 62 m span Norwegian Olympic cavern in Gjøvik. It shows a weighted mean Q -value of 7.4 (fair quality) and a typical range of about 4 to 27 (fair to good quality).

In addition to providing the statistics of rock quality for a given tunnel, rock type or structural domain, the Q -value is also used for describing the quality along drill core, as a supplement to the conventional RQD term. When logging in a tunnel that is under construction, other methods of logging the six, spatially varying Q -

parameters of the rock mass are used (Barton et al., 1980).

3 EMPIRICAL SUPPORT DESIGN

The second, left-hand drawer of our NMT design desk (Figure 1) tells us how to utilise the Q -value for permanent rock reinforcement and support selection.

Support recommendations are now based on 1,250 case records of permanent tunnel and cavern support (Grimstad and Barton, 1993). The empirical support design technique is illustrated in Figure 3. The x -axis

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_r .
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	
a) Rock-wall contact, and b) rock-wall contact before 10 cm shear			
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.
ii) $J_r = 0.5$ can be used for planar slickensided joints having lineations, provided the lineations are oriented for minimum strength.

c) No rock-wall contact when sheared			
H	Zone containing clay minerals thick enough to prevent rock-wall contact	1.0	
J	Sandy, gravely 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	
a) Rock-wall contact (no mineral fillings, only coatings)					
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	

b) Rock-wall contact before 10 cm shear (thin mineral fillings)			
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

c) No rock-wall contact when sheared (thick mineral fillings)			
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
OPR	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		J_w	
A	Dry excavations or minor inflow, i.e., < 5 l/min locally	1.0	
B	Medium inflow or pressure, occasional outwash of joint fillings	0.66	
C	Large inflow or high pressure in competent rock with unfilled joints	0.5	
D	Large inflow or high pressure, considerable outwash of joint fillings	0.33	
E	Exceptionally high inflow or water pressure at blasting, decaying with time	0.2-0.1	
F	Exceptionally high inflow or water pressure continuing without noticeable decay	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	
a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated			
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.

b) Competent rock, rock stress problems		σ_1/σ_3	σ_2/σ_3	SRF
H	Low stress, near surface, open joints	> 200	< 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_1 to $0.75\sigma_1$. When $\sigma_1/\sigma_3 > 10$, reduce σ_1 to $0.5\sigma_1$, where σ_1 = unconfined compression strength, σ_2 and σ_3 are the major and minor principal stresses, and σ_2 = 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_2/σ_3	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 $Q = 0.7 \gamma Q^{1/3}$ (MPa) where γ = rock density in kN/m³ (Singh, 1993).

d) Swelling rock: chemical swelling activity depending on presence of water			
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, τ (where $\tau = \sigma_2 \tan^2(\psi_r/J_r)$). Choose the most likely feature to allow failure to initiate.

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

Table 1 Q-Ratings Showing Some Updated SRF Values (Barton and Grimstad, 1994)

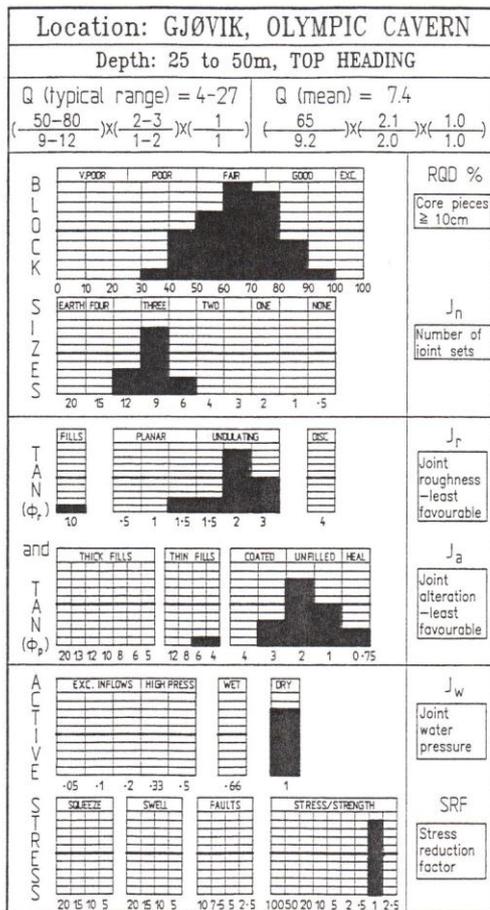


Figure 2 Logging Chart for Defining and Assembling Q-Parameter Statistics

shows the numerical range of Q-values, and rock classes are described along the top of the diagram. These descriptions are based on Norwegian opinions of what is poor, fair or good. It is possible that some other countries would choose these descriptive terms from one category to the right, i.e., Q = 1 to 4 might be described as "fair" rather than "poor", etc.

The y-axis shown in Figure 3 gives the span or height of the tunnel or cavern in metres, and is divided by a factor ESR describing the excavation safety requirement. For main road and rail tunnels (and hydropower caverns), ESR = 0.9 to 1.1. For water tunnels, ESR = 1.6 to 2.0.

The ESR factor modifies the degree of support, and therefore also the cost and safety level.

A specific example will demonstrate how the chart is used. The black circle shown in Figure 3 shows a hypothetical Q-value of 2.0 for a main road tunnel of 18 m span.

Permanent arch support can be provided by:

$$B \ 1.9m + S(fr) \ 10cm$$

where B = systematic bolting at 1.9 m c/c, and
 S(fr) = wet-mix, steel fibre reinforced shotcrete of 10 cm thickness.

A mean bolt length of 5 m is recommended from the right-hand side of the chart for this example.

3.1 Bolt Design

It is often wise to vary the bolt length about the mean, for example 4 m and 6 m bolts could with advantage be alternated in larger openings, so that the bolts do not all "stop" at the same internal surface within the rock mass. As shown later, numerical modelling can be performed to verify an empirical bolt design. An optimum design will consider the locations where bolting is most heavily loaded, which might consistently be in the walls. It will also consider which cross-sections are most appropriate for steel bolts, i.e., 20, 25 or 32 mm, or whether, due to lower modulus ground, more deformable epoxy grouted fibre reinforced plastic bolts would be more suitable. The influence of the various choices of bolt capacities and stiffnesses can be seen in the forces developed in the modelled shotcrete, and of course vice versa.

Acceptance of the NMT principle that B+S(fr) can permanently stabilise a tunnel and does not need a nominal (or water pressure resisting) concrete liner, means that not only the steel fibre resistant shotcrete, but also the bolting must be as corrosion resistant as possible. As shown later, this implies the use of multiple corrosion protection (i.e., galvanised, epoxy-coated, PVC-sleeved steel bolts) or fibre glass types. Further comments on optimal choices are given later.

3.2 Fibre Reinforced Shotcrete

The Q-system support recommendations of 1974 include only plain or steel mesh reinforced shotcrete S(mr). Already from 1978, S(fr) was used commercially in Norway in hydroelectric power projects, and since about 1984, use of S(mr) has been discontinued. The reason for this dramatic change of techniques was the successful development of high capacity (10 to 25 m³/hr) shotcrete robots and high quality, low rebound

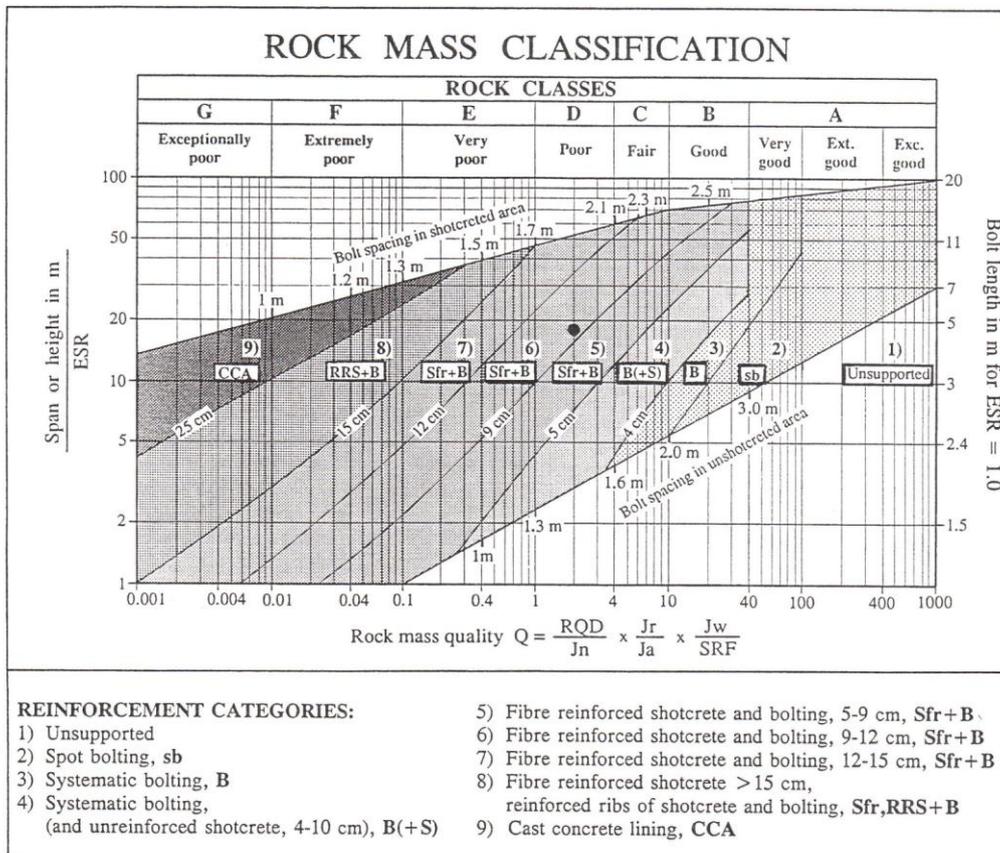


Figure 3 The Q-System Tunnel Reinforcement Design Chart (Grimstad and Barton, 1993)

wet process shotcrete. Figure 1 (third drawer on left) summarises the key components of modern S(fr), which represents a revolution in safe, fast and economic tunnelling.

Key properties of S(fr) include the potentially very low permeability (except where water is leaking during initial hardening) and the protection of the non-continuous fibres from corrosion when good quality, low water content shotcretes are used (i.e., C45, W/C+F = 0.4 to 0.45). The fracture energy absorbed when deforming S(fr) is some 30 to 40 times that for unreinforced shotcretes, and in fact two layers of steel mesh reinforcement are needed to exceed the performance of S(fr) when using S(mr). Another advantage of S(fr) is the "closeness of fit" to the uneven rock surface (where mesh might be impossible to bend,

resulting in shadow and increased potential for corrosion). Because S(fr) contacts all the exposed rock surface, it gives the highest possible bond, cohesion and frictional strength. The three operations, S, mr, S, needed to give one good quality layer of S(fr), are provided in only one operation by S(fr) and all can be applied remotely if conditions are unsafe, prior to bolting. Hand-held operatives in space suits belong to the past.

4 ROCK QUALITY FROM SEISMIC SURVEYS

When investigating rock mass qualities for a future tunnel project, there are great advantages in performing refraction seismic surveys for locating low velocity zones that may be partly hidden by soil cover in non-

glaciated regions. The problem remains, however, in estimating the velocity and rock quality at tunnel depth, which may exceed the 20 to 40 m depth typical of such seismic data. Ideally the seismic survey should be used for locating deeper, inclined drill holes, or for locating specific pairs of holes at say 50 m spacing, so that cross-hole tomography can be performed. Velocities can then be correlated to the adjacent, logged drill core, at critical locations. This technique was used with success at the Gjøvik cavern, as reported by Barton et al., 1994.

Based mainly on hard rock experience and relatively shallow seismic refraction measurements, a general relationship between the field P-wave velocity V_p and the Q-value was suggested by Barton et al., 1992:

$$V_p \approx \log Q + 3.5 \text{ km/s} \quad (2)$$

The approximate joint frequency (F per metre) and RQD value typical for a seismic velocity of 3.5 km/s (when $Q \approx 1.0$) are 13.5 joints per metre and 45%, as reported by Sjøgren et al. (1979) for a variety of generally hard, jointed rocks measured at shallow depth. When $V_p = 4.5$ km/s ($Q = 10$) these values increase to approximately

6.5 joints per metre and $RQD = 78\%$ according to Sjøgren et al. data.

Figure 4 shows the above central relationship between Q and V_p for jointed hard rocks, and a recent development for also accounting for the depth, rock matrix porosity and unconfined compression strength (σ_c) of the rock.

To demonstrate the use of the correlations shown in Figure 5, we can test them against the Triassic, Sydney Basin, Hawkesbury Sandstone, whose properties have been conveniently assembled by Pells (1985). We can take as a starting point the typical range of *in situ* seismic velocities of about 1.9 to 2.5 km/s (see bar at left side of Figure 4). Looking at the above Q- V_p relationship (equation 2, which is the solid line in Figure 4, representing approximately 25 m depth and approximately 1% porosity) we can expect Q_c values ranging from 0.025 to 0.1. The value Q_c is a conventional Q-value normalised by $\sigma_c/100$, where the unconfined compression strength is designed to modify the Q-value when σ_c is different from its typical hard rock value of approximately 100 Mpa.

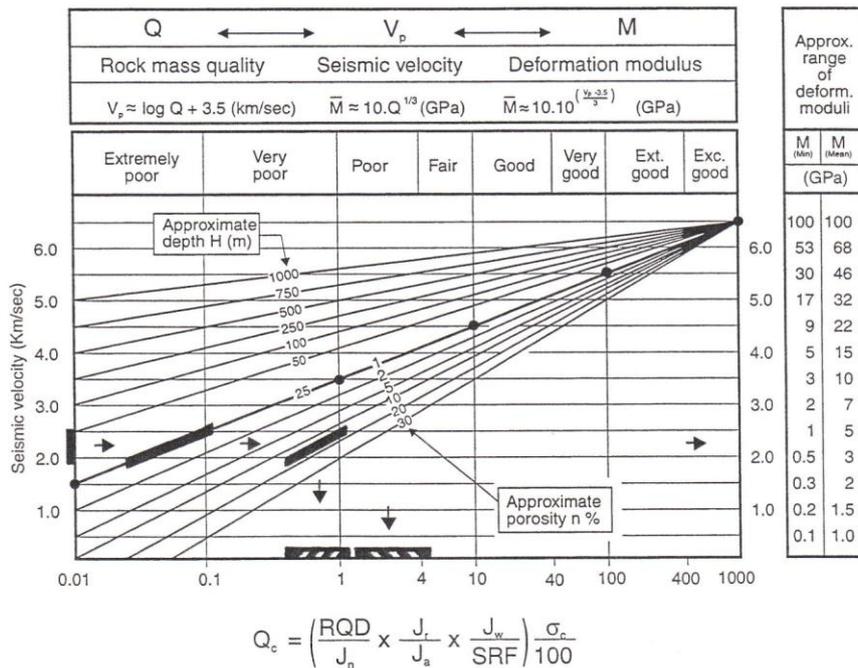


Figure 4 Relationships Between Seismic Velocity, Rock Quality and In Situ Static Deformation Modulus used for Numerical Modelling (Barton, 1995)

We now have to invoke the correct σ_c value (mean for Hawkesbury Sandstone = approx. 30 MPa for saturated samples) and also apply the relevant matrix porosity (mean approximately 15%). The porosity of 15% brings us to a range of Q_c values of approximately 0.4 to 1.5 for the same reference depth of 25 metres. Conversion to conventional Q -values using the relation:

$$Q_c = Q \times \frac{\sigma_c}{100} \quad (3)$$

gives predicted Q -values of 1.3 to 5.0, and predicted in situ, mean deformation moduli (M) in the range 2.5 to 5 GPa (approx.) at 25 m depth, with disturbed sample $M(\min)$ values of about 0.5 to 1 GPa. Pells (1985) gives

a range of moduli of 2 to 4 GPa for Class 1 rock, and 1 to 2 GPa for Class 2 rock, which are satisfactorily close to the above.

At greater depth, if it was possible to measure the velocity with cross-hole seismic tomography, higher velocities would be seen due to more tightly closed jointing. However, the tunnelling quality (Q -value) might not be improved. In fact, in some cases, a stress penalty could arise in which squeezing, crushing or stress slabbing might occur at depth. This would increase the SRF value (see Figure 2) and reduce the Q -value, requiring heavier support (see Figure 3). Care must be taken to fully evaluate the circumstances when applying the Q - V_p relationship.

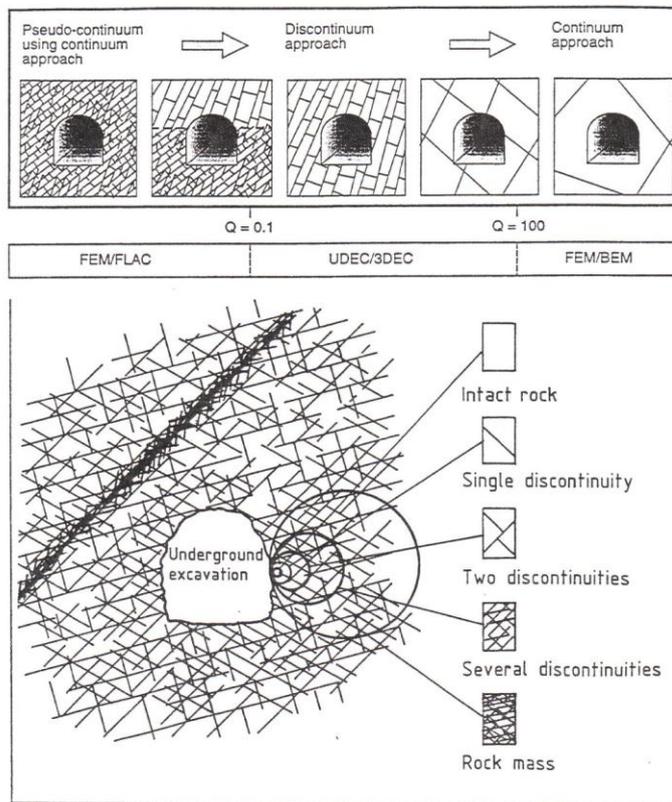


Figure 5 Top: Approximate Q -Value Limits for Realistic Distinct Element Modelling
 Bottom: UDEC-BB Could be used for Modelling all the Scales of Jointing
 Illustrated here (modified from Hoek, 1983)

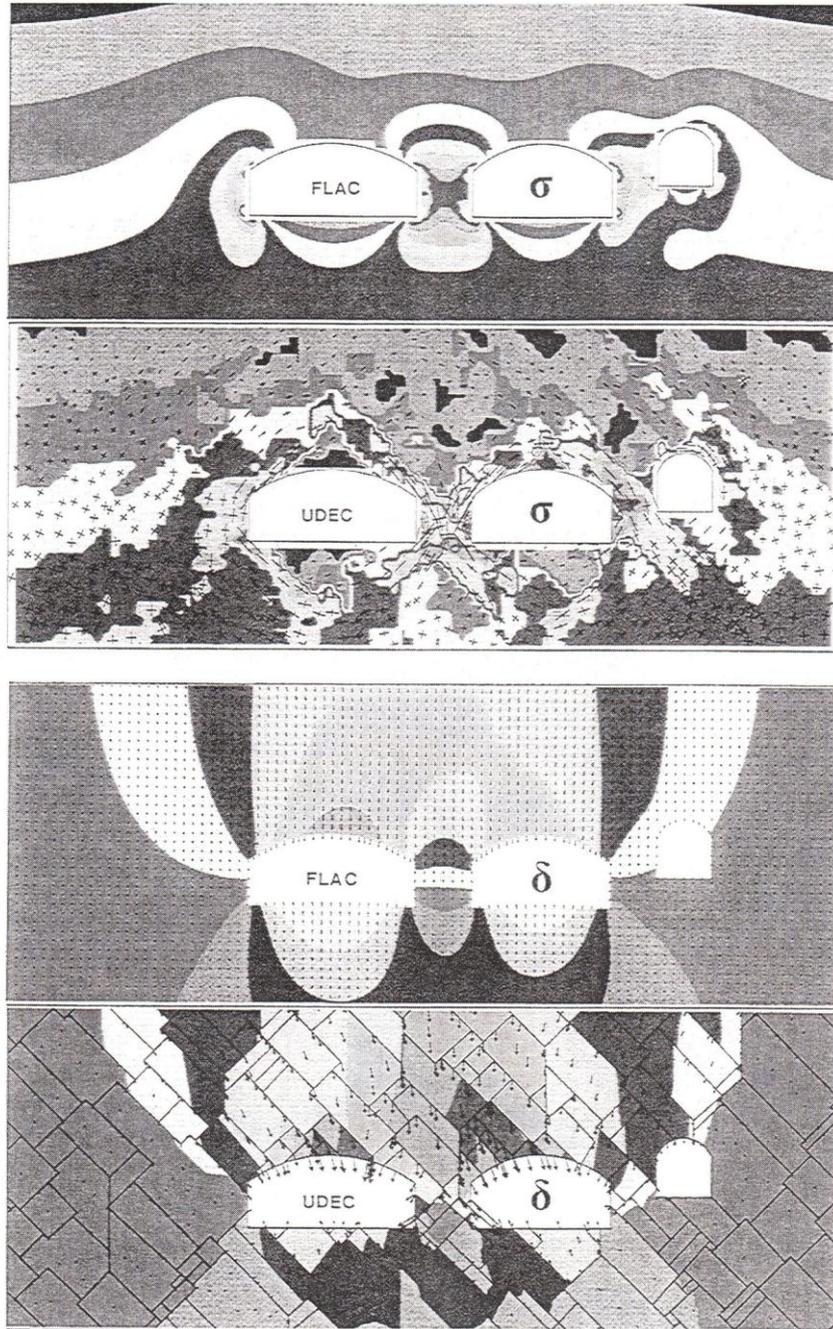


Figure 6 Comparison of Continuum (FLAC) and Discontinuum (UDEC-BB) Modelling of Tunnels in Jointed Rock (Backer, NGI, 1995)

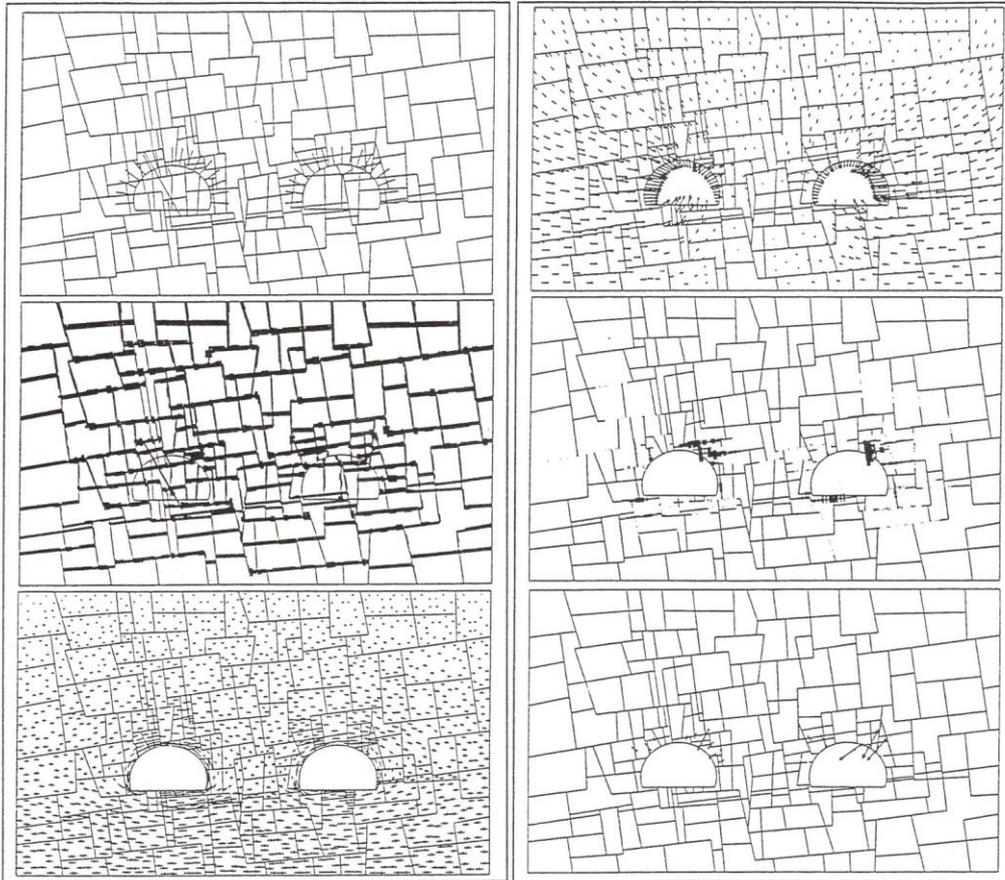


Figure 7 UDEC-BB Analysis of Twin Motorway Tunnels in Granite for Design Verification (Backer, NGI, 1993)

Japanese data on laboratory uniaxial compression strength (σ_c) and elastic wave velocity (V_p lab.) for weak and squeeze-prone rocks reported by Aydan et al., 1992 and Sato et al., 1995 also emphasise the importance of using σ_c in the Q_c estimation given in Figure 4. Roughly speaking, the following laboratory V_p values can be expected for given σ_c values, the scatter being largely a function of porosity and density differences.

σ_c (MPa)	V_p lab. (km/s)
1	1.5
5	2.0
10	2.5
20	3.0

Table 2 Approximate Mean Relationship Between Laboratory σ_c and V_p lab. Values for Weak Rocks, based on Aydan et al., 1992 and Sato et al., 1995.

In the case of the Hawkesbury sandstone, Pells (1985) shows somewhat lower velocities for σ_c values of 10

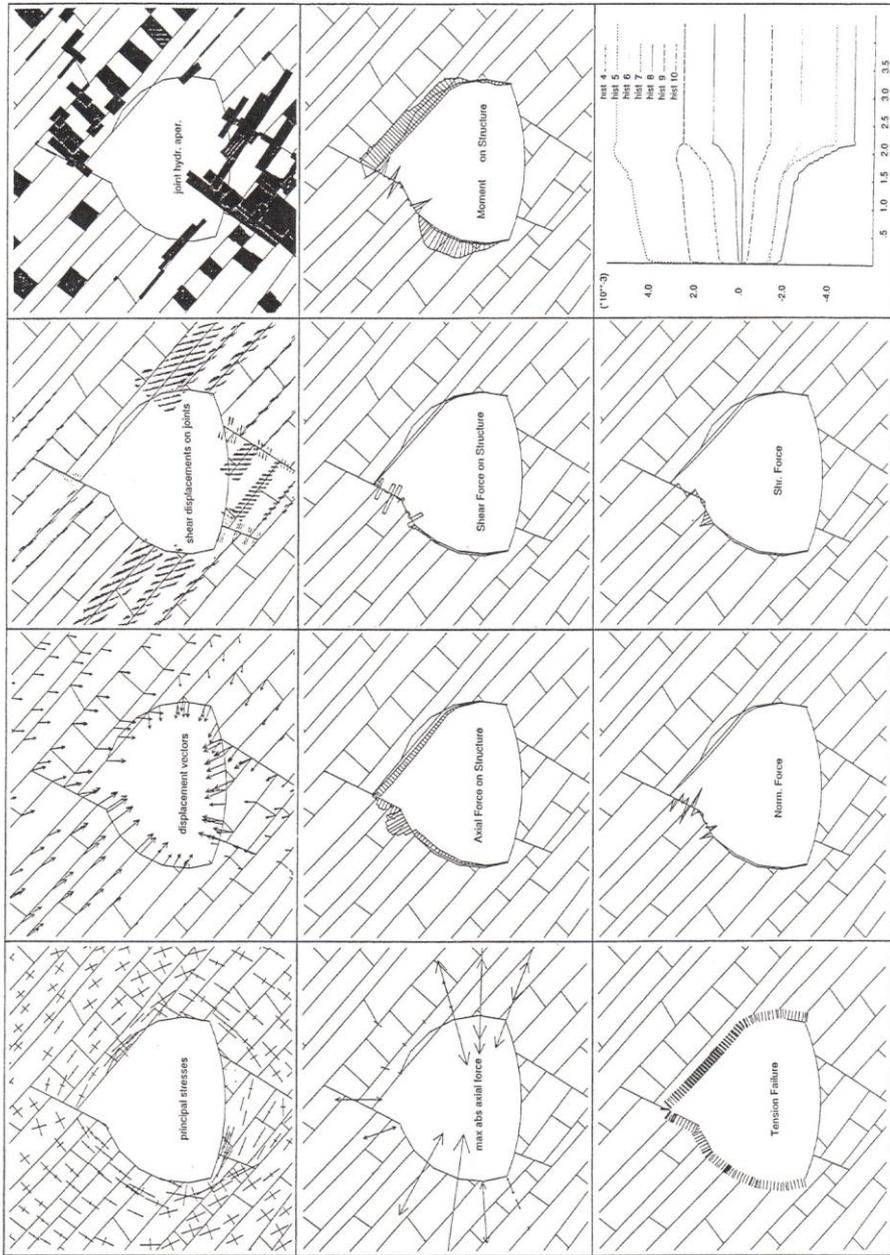


Figure 8 Examples of UDEC-S(fr) Modelling (Chryssanthakis, NGI, 1996)

and 20 MPa, *i.e.*, V_p values nearer to 1.5 and 2.0 km/s would be typical, presumably due to their higher porosities.

5 NUMERICAL VERIFICATION

The Q-system of tunnel support design (summarised in Figure 3) provides recommendations for rock bolt spacing and thickness of fibre reinforced (or in some cases, unreinforced) shotcrete. In poor rock conditions, rib reinforced shotcrete (RRS) or cast concrete arches (CCA) are recommended. RRS is described later in this paper. These designs are each based on empirical correlations. Investigations still need to be made concerning bolt capacity and stiffness, shotcrete loading levels and concrete arch thickness in special cases of very poor rock conditions.

Numerical modelling is therefore often utilised in NMT designs made by NGI for helping to understand the potential failure modes (or anisotropic loading of support) thereby improving on the basic empirical design. As illustrated in Figure 5, for Q-values below about 0.1, it is probably appropriate to utilise continuum models in view of the heavily jointed nature of the ground, while for Q-values between about 0.1 and 100, the jointing can generally be represented in two or three dimensional UDEC or 3DEC studies.

When continuum modelling is used to try to represent the behaviour of distinctly jointed rock, reality is not achieved as shown by the FLAC models in Figure 6. For example, the zone of joint related tension predicted in the arch by the jointed UDEC-BB model of each large tunnel is not shown in the FLAC results. However, for exceptionally massive rock, continuum modelling for assisting in the prediction of rock failure (rock bursting) would again be most relevant, if tunnelling under very high overburdens.

Utilisation of Cundall's UDEC code (or the more recent UDEC-BB Itasca-NGI version with NGI's non-linear joint model) requires careful description of the roughness and strength characteristics of principal joint sets and clay-filled discontinuities, as detailed by Barton (1995). Figure 7 can be used as an example of the type of numerical verification that is required to check NMT designs. This UDEC-BB model was performed for three-lane tunnels in jointed granite. Approximate, but sufficient input data was readily obtained from field logging and core characterisation performed by NGI for the contractor.

The six diagrams in the figure represent the following:

Left-Hand Figures		Right-Hand Figures	
Top	joint pattern and bolting	Top:	displacements (max. = 3.9mm)
Middle:	hydraulic apertures (max. = 44 μ m)	Middle:	joint shear (max. = 2.6mm)
Bottom:	principal stresses (max. = 8 MPa)	Bottom:	bolt forces (max. = 6.9 tnf)

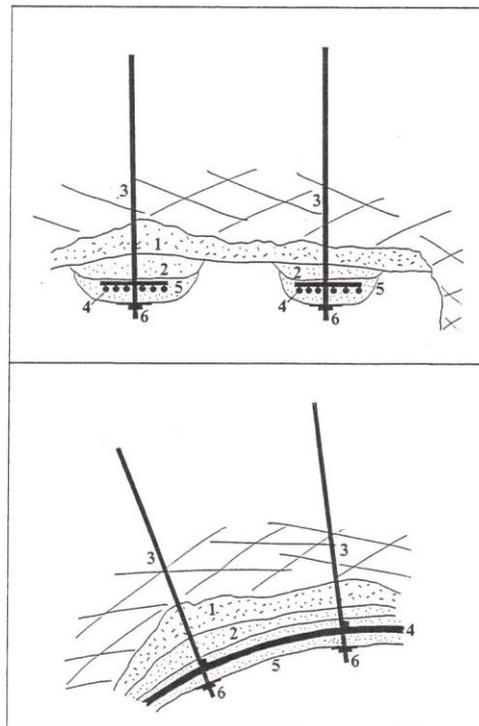


Figure 9 Cross-Section and Longitudinal-Section through RRS Arches.

Note:

- (1) S(fr)
- (2) S
- (3) bolts with welded cross-piece
- (4) six ribs
- (5) S
- (6) washers and nuts.

Each layer of S(fr) or S should exceed 100 mm and should be built up in 4 to 6 cm layers.

In cases where bolt capacity is exceeded based on the UDEC-BB analysis of the Q-system design, greater bolt capacity would be designed (*i.e.*, increasing bolt

diameters from 20 to 25 mm). Some potential failure modes or anisotropic deformation mechanisms might also justify locally increased bolt lengths. Recently NGI has numerically investigated the different performance of steel bolts (mortar cemented) and fibre reinforced plastic bolts (epoxy cemented) which have widely different axial stiffness characteristics. It is found, as one might expect, that the steel bolts are more suited to harder ground. They will be more heavily loaded than FRP bolts in softer ground, and differences in the shotcrete loading are observed as a consequence.

Recently, the ability to model fibre reinforced shotcrete in addition to the fully grouted steel or fibre bolting has been developed for UDEC and UDEC-BB. The code has been christened UDEC-S(fr). This Itasca-NGI development is illustrated in Figure 8 by an example with deliberate overbreak to a low friction bedding plane and joint surface. In order to demonstrate the code's capabilities, reduced bond strength ($JTENS = 0.25 \text{ MPa}$) and reduced shear strength ($JCOH = 0.18 \text{ MPa}$ and $JFRIC = 20.3^\circ$) along the rock-shotcrete interface were modelled. As can be seen in Figure 8, this resulted in extensive detachment of the shotcrete. Conventional development of forces in the shotcrete especially at joint crossings, is only seen where the shotcrete is still bonded in the left hand side of the arch.

When normal ranges of these parameters are used, the modelled values of axial and shear forces in the shotcrete and of the normal and shear forces along the rock-shotcrete interfaces can each be compared with design assumptions, and adjustments made to designs if need be.

6 DRY DRAINED NMT TUNNELS

As a conclusion to this introduction to NMT, reference will be made to typical NMT tunnels in the outskirts of Oslo. These had permanent rock support designed using the Q-system (Figure 3), and two of the four tunnels included RRS (rib reinforced shotcrete) arches as the permanent structural support where very poor rock was encountered with swelling clay zones, and also beneath a river bed with a minimum cover of only 2 m.

The principle of RRS support is illustrated in Figure 9. It represents great advantages to steel sets due to its intimate contact with the ground from the start. In other words, the rock does not have to deform significantly, and thereby possibly lose strength. Deformation is resisted from the start by a structural member that is bolted into the surrounding rock, with a normal pattern of rock bolts around the RRS arch.

The basic methods of water and frost insulation (or cladding) of these NMT tunnels are illustrated in Figure 10. Four important points of this design method are emphasised:

- The tunnel is drained but dry. (Pre-injection is used for limiting inflow).
- There are large savings of concrete volumes. (An air gap replaces much of the usual nominal concrete lining).
- There is a need for reliable NMT rock reinforcement. (Since this is permanent tunnel support).
- There is a need for low maintenance, easily assembled cladding. (The cast concrete elements can be assembled at a rate of 1 km/month).

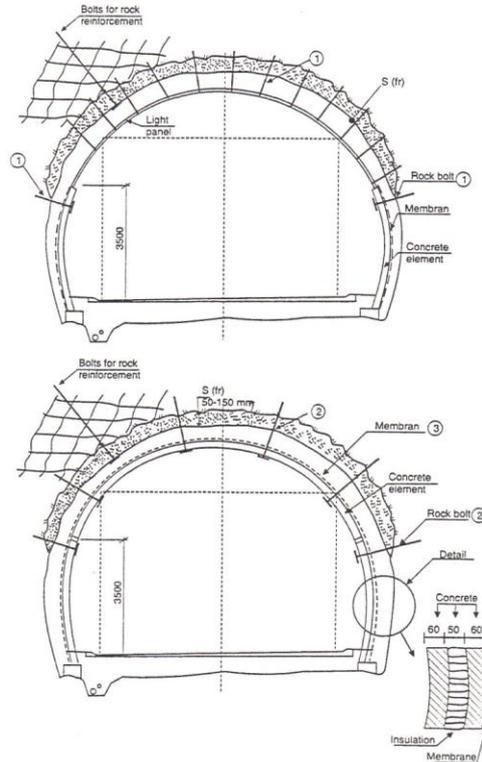


Figure 10 NMT Tunnel Linings are Drained but the Carriageway is Dry. (After Kvelsvik and Karlsrud, 1995)

7 CORROSION PROTECTED ROCK BOLTS

Reliance on B+S(fr) as permanent tunnel support places an extra burden on the designer, in that bolt corrosion has to be minimised or removed as a threat to the assumed 50 or 100 year life of the tunnel.

This problem has been solved by the development of the CT (Combi-Tube) bolt by Ørsta Stålindustri in Norway. Figure 11 shows that a PVC sleeve separates the two layers of grout, and additional galvanising and epoxy coating virtually guarantees unlimited life. (The temporary mechanical anchor will not of course survive longer than is usual for a conventionally grouted bolt.) The full length of the bolt can be grouted either before or after shotcreting, by using extension tubes projecting beyond the planned thickness of the shotcrete.

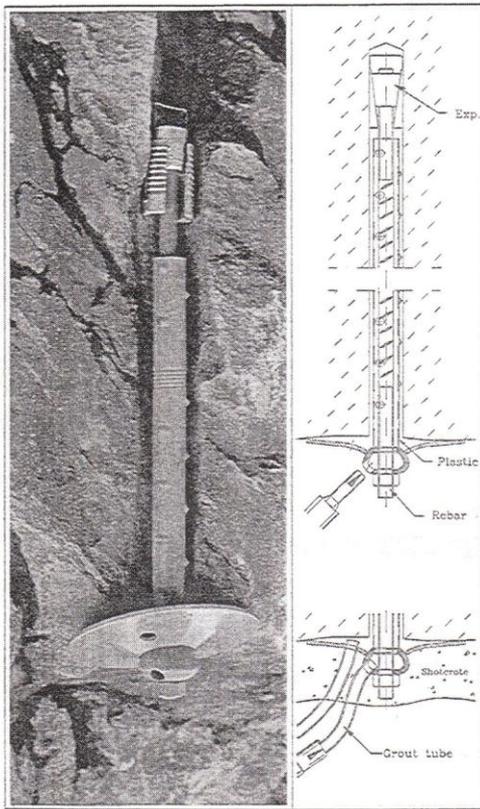


Figure 11 The Norwegian CT Bolt for Temporary and Multi-Corrosion Protected Permanent Bolting (Ørsta Stålindustri A/S)

Glass fibre reinforced plastic bolts (FRP) with epoxy or polyester grout are also another good solution to corrosion-free performance, in cases where the rock mass has a lower modulus and stiffer steel bolts would be incompatible with the deformation of the ground. FRP bolts have advantages of lightness and extremely high axial strengths, but have less shear resistance than steel bolts, in case this should be a pronounced form of loading in a given rock mass.

8 CONCLUSIONS

- NMT is an abbreviation for several measures that together make Norwegian tunnelling extremely cost effective by world standards. Key aspects are: contractual flexibility, drained tunnels, rock mass classification for support design, robotic application of S(fr), and experienced tunnellers.
- NMT is the most appropriate method available for tunnels in jointed rock that tends to overbreak, whether this rock is of 5 MPa or 250 MPa compression strength. Great savings in concrete volumes, time and cost are achievable compared to conventional or NATM tunnelling.
- Empirical rock reinforcement design via the Q-system, and re-evaluation during tunnel driving results in the most cost-effective tunnel support. The owner pays only for the operations and materials that are needed, and the contractor is paid via the unit prices given in his tender.
- Robotically applied, wet-mix, steel fibre reinforced shotcrete is one of the secrets of NMT efficiency, which gives significant speed and cost savings, especially in countries with high labour costs. Concrete savings start with this product, and continue right through to the final drained tunnel, which has rapidly assembled pre-cast concrete panels for water and frost insulation.

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