

# NORWEGIAN METHOD OF TUNNELLING

For a country with only 4 million inhabitants, Norway has quite an unusual level of tunnelling activity. Tunnel construction in the civil sector has been especially high in the last 15 years, with 8 of these years seeing more than 4 million m<sup>3</sup> of tunnel and cavern excavation. Several years have seen more than 3 million m<sup>3</sup> of water tunnel construction for hydropower per year (mostly in the late 1970s), and many years with more than 1 million m<sup>3</sup> of road tunnels (2 million m<sup>3</sup> in 1990). In 1990, 32 km of hard rock TBM tunnels were driven, mostly at Statkraft's Svartisen hydroelectric project. One of the largest Norwegian tunnelling contractors, Selmer A/S has constructed more than 20 km of tunnels per year during five of the last seven years, with a total of 31 km in 1991. A/S Veidekke, who in a joint venture with Selmer A/S were responsible for the construction of the 62m span Olympic ice hockey cavern, also have a very impressive record of tunnelling and underground construction. The Veidekke Group excavated a total of 95 km of tunnels in 1987 and 1988 and have averaged more than 30 km per year since 1987. It has been estimated that 4,500 km of tunnels have been constructed in Norway since 1970.

Norwegian tunnelling is also blessed with several very experienced Consultants whose role in pre-investigations, design and tender document preparation are significant in many ways. Norconsult, Norpower, Berdal Strømme, Grøner, Noteby, Fortifikasjon, Geoteam, SINTEF and NGI are notable examples of organisations with extensive tunnelling and underground construction experience, built around numerous hydropower, petroleum storage and road tunnel projects both in Norway and abroad.

A major Norwegian tunnelling development over the past 12 to 15 years has been high capacity wet process shotcreting equipment which allows steel fibre reinforcement (typically 30 mm x 0.5 mm) to be applied by robot at the tunnel face 15 to 20 m ahead of the rig, in volumes of 15 to 25 m<sup>3</sup> per hour. At present, some 50 to 60,000m<sup>3</sup> of fibre reinforced shotcrete are sprayed each year in Norway; one company, Entreprenørservice A/S, being responsible for 30,000m<sup>3</sup> in 1991.

Another major player in the shotcreting field, besides Selmer and Veidekke, is Robocon International who did pioneering work in the development and application of fibre reinforced shotcrete in the early 80s, and are currently heavily involved in the Middle East.

The three largest tunnelling organisations,

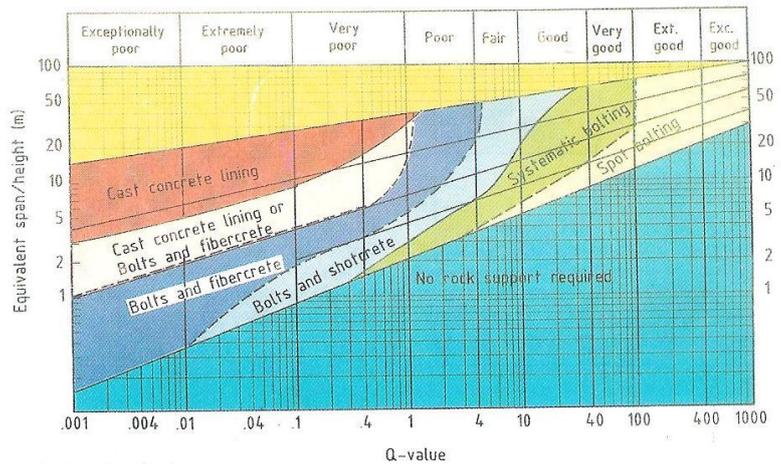


Figure 1. Simplified diagram for design of rock support based on the Q-system (Grimstad et al., 1986)<sup>2</sup>.

Selmer, Veidekke and Statkraft, have between them constructed one half of Norway's 200 underground hydroelectric power stations, or more than one quarter of the world's total of approximately 400. In a current hydroelectric project in Northern Norway (Statkraft's Svartisen Project), more than 40 km of hard rock TBM tunnels have been constructed in record time in hard gneisses, diorite, quartzite, marbles and schists with compressive strengths of 120 to 300 MPa. High cutter loads of 32 tons and high cutter head power (3150 HP in 4.3 m and 5.0 m diameter Robbins machines) have given average weekly advance rates of 119 to 200 m in 100 hour weeks for the five machines. Best results of 61.2 m in a shift, 90.2 m in a day, 415 m in a week and 1176 m in a month were achieved in the 36 km driven during a 2 year period from 1989 to 1992. At the Meråker hydroelectric project the joint venture Veidekke/Eeg Henriksen has recently set a new world record of 426 m in one week, using a Robbins HP TBM of 3.5 m diameter.

This article describes key aspects of Norwegian tunnelling technology to assist potential users of these methods in deciding between the so-called New Austrian Tunnel Method (NATM) and the Norwegian Method of Tunnelling (NMT). A comparison of methods and typical areas of application is given. In assembling key aspects of NMT, the authors have included geotechnical investigation techniques, numerical modelling, tunnelling equipment and

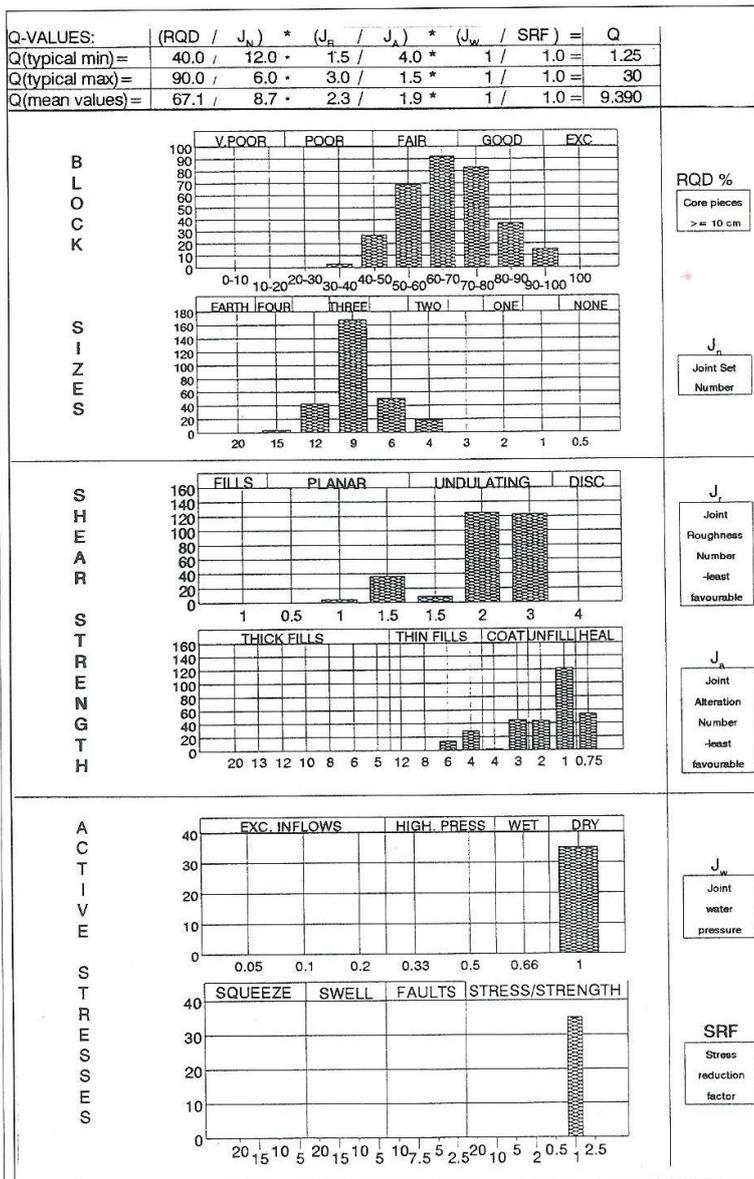
materials, and contractual aspects to give readers a glimpse of the level of technology available. A case record of NMT used in difficult tunnelling conditions is given at the end of the article.

## NMT AND NATM - WHAT ARE THE DIFFERENCES?

Despite the comment by an experienced NATM pioneer that "it is not usually necessary to provide support in hard rocks", Norwegian tunnels require more than 50,000m<sup>3</sup> of fibre reinforced shotcrete and more than 100,000 rock bolts each year. Two major tunnelling nations, Norway and Austria, have in fact long traditions in using shotcrete and rock bolts for tunnel support, yet there are significant differences in philosophy and areas of application for NATM and NMT. To start this brief review, it may be pertinent to first state what appear to be the major differences between NATM and NMT.

NATM appears most suitable for soft ground which can be machine or hand excavated, where jointing and overbreak are not dominant, where a smooth profile can often be formed and where a complete load bearing ring can (and often should) be established. Monitoring appears to play a significant part in deciding on the timing and extent of secondary support. (Using the surrounding ground as the main loading component is not an exclusive NATM philosophy. It is essential practice and is often inevitable)

NMT appears most suitable for harder ground, where jointing and overbreak are dominant, and where drill and blasting or hard rock TBM's are



the most usual methods of excavation. Bolting is the dominant form of rock support since it mobilises the strength of the surrounding rock mass in the best possible way. Rigid steel sets or lattice girders are inappropriate in Norway's harder rocks due to the potential overbreak. Potentially unstable rock masses with clay-filled joints and discontinuities will increasingly need shotcrete and fibre reinforced shotcrete [S(fr)] to supplement the systematic bolting (B). It can be stated with some certainty that B+S(fr) are the two most versatile tunnel support methods yet devised, because they can be applied to any profile as temporary or as permanent support, just by changing thickness and bolt spacing. A thick load bearing ring (reinforced rib of shotcrete = RRS) can be formed as needed, and matches an uneven profile better than lattice

Figure 2. Systematic recording of Q-system data in the arch of the 62 m span Olympic Ice Hockey cavern (Løset and Bhasin, 1991).

girders or steel sets.

The following table emphasises some of the major features of NMT. Some of the differences to NATM will be apparent in the listing of "not used" features.

### GEOLOGICAL MAPPING AND CLASSIFICATION

In the Norwegian Method of tunnelling, great emphasis is placed on thorough descriptions of geological and geotechnical aspects of the project. The owner and his consultant have as

their objective the preparation of tender documents that reflect as closely as possible the likely equipment, tunnelling methods and tunnel support materials for successfully tunnelling through the investigated rock. Since the Norwegian Geotechnical Institute (NGI) has been responsible for the geological mapping and classification of at least one quarter of the 4,500 km of civil engineering tunnels constructed in Norway in the last 20 years, some emphasis will be given to the methods developed and used by this consulting group.

Table 1. Essential features of NMT.

- Areas of usual application:**  
 Jointed rock; harder end of scale (XXX, = 3 to 300 MPa)  
 Clay bearing zones, stress slabbing (Q - 0.001 to 10)
- Usual methods of excavation**  
 Drill and blast, hard rock TBM, hand excavation in clay zones.
- Temporary support and permanent 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)
  - temporary support forms part of permanent support
  - mesh reinforcement not used
  - dry process shotcrete not used
  - steel sets or lattice girders not used; RRS used in clay zones
  - 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
- Rock mass characterisation for:**
  - predicting rock mass quality
  - predicting support needs
  - updating of both during tunnelling (monitoring in critical cases only)
- The NMT gives low costs and**
  - rapid advance rates in drill and blast tunnels
  - improved safety
  - improved environment

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

A key requirement for ensuring consistent mapping quality, good tender documents and good records of actual conditions is a method that describes the rock mass in quantitative rather than just qualitative terms. Although the high level of experience in the Norwegian tunnelling community has allowed "rules-of-thumb" and much "previous experience" to dictate a lot of the support estimates, more and more companies are realising the value of a documentation method such as the Q-system for regulating the description of rock mass conditions and support recommendations (Barton et al., 1980).

The Q-system is an empirical method based on the RQD method of describing drill core

(Deere et al., 1967)<sup>2</sup> and five additional parameters, which modify the RQD-value for the number of joint sets, joint roughness and alteration (filling), the amount of water, and various adverse features associated with loosening, high stress, squeezing and swelling. The rock mass classification is associated with support recommendations based originally on 212 case records. (More than 1000 new case records are presently being processed at NGI).

**The Q-value is expressed by**

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

The numerical value of Q ranges from 0.001 for exceptionally poor quality squeezing ground up to 1000 for exceptionally good quality rock which is practically unjointed. The six parameters, each of which has a rating of importance, can be estimated from surface mapping and from core logging and can be verified during excavation. In combination they represent:

1. The block size, given as the quotient  $J_n = \frac{\text{degree of jointing}}{\text{number of joint sets}}$
2. The inter-block shear strength  $J_r = \frac{\text{joint roughness}}{J_a \text{ joint alteration or filling}}$
3. The active stress  $J_w = \frac{\text{water pressure or leakage}}{SRF \text{ rock stress conditions}}$

The Q-system, as represented in Figures 1 and 2, is a forward predictive method and therefore differs significantly from NATM methods, which apparently depend on monitoring to decide on the timing and amount of additional support to finally "place the rock in the correct class". It has been said elsewhere, perhaps unfairly, that "when experienced, use the Q-system; when uncertain, use NATM" (uncertainty here due to the weak ground commonly associated with NATM).

The important point is that forward prediction of conditions and agreed modifications for unexpected conditions should each be done as early and as accurately as possible, so that on the one hand tender documents are a fair reflection of revealed conditions, and unexpected conditions are agreed upon and tackled without delay by all parties concerned. This minimises disputes and also minimises tunnel instability! Legal action is in fact virtually non-existent in Norwegian tunnelling.

Although the Q-system of rock mass classification has been used for many years, improvements have taken shape rather slowly. The significant simplification made by Grimstad et al. (1986)<sup>3</sup> shown in Figure 1 also shows an updating to incorporate an essential element of the Norwegian Method (NMT), namely, wet process steel fibre reinforced shotcrete.

Figure 2 shows an improvement in the method of recording the six Q-system

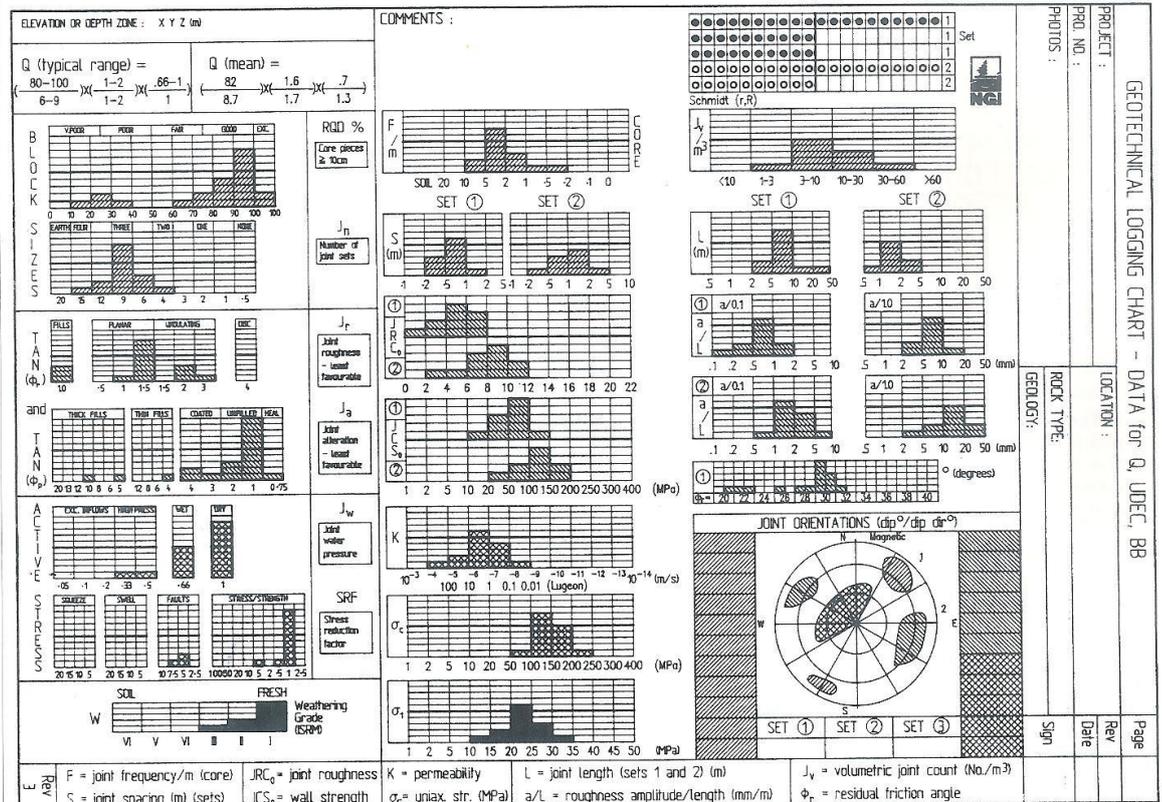
parameters. The example shown gives all the Q-system observations made in the arch of the 62m span Olympic ice hockey cavern. The histograms are filled in rectangle by rectangle in field, core, or tunnel mapping and subsequently incorporated in a PC-based spread sheet for easier data manipulation.

Figure 3 shows a further improvement in the systematic recording of field- or core-logging data made by NGI in 1990. Note that the Q-system occupies the left hand side of the Geotechnical Logging Chart. Other parameters are filled in as they become available, and if relevant to the project in hand. The chart contains information for setting up input data files for numerical modelling of critical sections of the tunnel or cavern as the case may be.

NGI increasingly uses the discrete element method UDEC developed by Cundall (1980)<sup>5</sup> for checking the performance of Q-system designed rock reinforcement in critical projects. The Barton and Bandis (1990)<sup>6</sup> BB joint descriptive terms JRC, JCS and  $\phi_r$  feature in the Geotechnical Logging Chart, and are integrated with Q-system logging when UDEC-BB computer models are required by the Owner.

The parameters presented in the geotechnical

Figure 3. Geotechnical logging chart for describing rock masses and rock joints for subsequent use in Q-system and UDEC-BB tunnel and cavern design.



logging chart are as follows:

I ROCK MASS STRUCTURE

1. RQD (Deere et al., 1967)<sup>(Q)</sup>
2.  $J_n$  = joint set number(Q)
3.  $F$  = joint frequency (per metre)
4.  $J_v$  = volumetric joint count (Palmström, 1982)<sup>(Q)</sup>
5.  $S$  = joint spacing (in metres)
6.  $L$  = joint spacing (in metres)
7.  $w$  = weathering
8.  $\sigma/\beta$  = dip/dip direction of joints

II JOINT CHARACTER

9.  $J_r$  = joint roughness number(Q)
10.  $J_a$  = joint alteration number(Q)
11. JRC = joint roughness coefficient
12.  $a/L$  = roughness amplitude of asperities per unit length (mm/m)
13. JCS = joint wall compressive strength
14.  $\phi_r$  = residual friction angle

III WATER, STRESS, STRENGTH

15.  $J_w$  = joint water reduction factor(Q)
16. SRF = stress reduction factor(Q)
17.  $K$  = rock mass permeability (m/s)
18.  $c$  = compressive strength
19.  $l$  = major principal stress

Further details of these parameters are given by Barton et al. (1992)<sup>(Q)</sup>.

The method of recording field mapping data, core logging data or tunnel logging data shown in Figure 3 gives the engineering geologist a convenient format (and check list) of essential data. These charts can subsequently be used for developing statistics for tender documents, for drill-core description, for parameter depth logs and for documenting the progressing or completed tunnel.

CROSS-HOLE TOMOGRAPHY

Urban tunnelling through difficult fault zones with low cover, or the approach of a major fault zone mid-way beneath a deep fjord are two typical tunnelling scenarios that call for more information on the rock mass. With good warning well ahead of the face, a tunnel contractor can plan his strategy, mobilise equipment and minimize risk. In other cases he may avoid costly over-reaction and unnecessary delays. Cross-hole seismic tomography and tunnel radar are invaluable aids in this respect (Kong et al., 1992<sup>(Q)</sup>; Westerdahl and By, 1991<sup>(Q)</sup>).

In the case of large caverns where choice of location also exists, improved knowledge of the internal structure of the rock mass can save considerable sums in rock support if a location within higher quality rock can be found. A perfect example of this was the cross-hole seismic investigations performed for the 62 m span Olympic Ice Hockey Cavern at Gjøvik, one example of which is shown in Figure 4.

In the case of Fjellinjen twin motorway tunnels beneath Oslo, cross-hole seismic tomography was performed for a total of ten profiles, first using pairs of boreholes drilled from the surface, and subsequently using pairs of probe holes ahead of the tunnel face as the 13 m span tunnels approached a major fault

containing crushed alum shale overlain by soft clays. The contractor elected ground freezing for one of the tunnels as a result of this geophysical information.

In the case of the Hvaler sub-sea tunnel, a string of hydrophones was placed on the sea bed as receivers. A pilot borehole was drilled 75 m ahead of the tunnel face for successive positioning of the signal source. Tomographic presentation of the results gave the contractor a graphic picture of the gradual narrowing of the vertical fault zone with increasing depth from the sea bed. Surface refraction surveys and probe drilling from a drilling ship had given a false impression of the width of the feature.

Promising correlations between seismic velocity ( $V_p$ ) and  $Q$ -values have recently been obtained, as described in *World Tunnelling* by Barton (1991)<sup>(Q)</sup>. Table 1 gives the approximate, easy to remember correlation between these parameters which can no doubt be refined upon in due course.

Table 2. Approximate correlation between  $Q$  and  $P$ -wave velocity.

$V_p$ (m/sec)	500	1500	2500	3500	4500	5500	6500
$Q$	0.001	0.01	0.1	1	10	100	1000

In equation form, this is as follows.

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

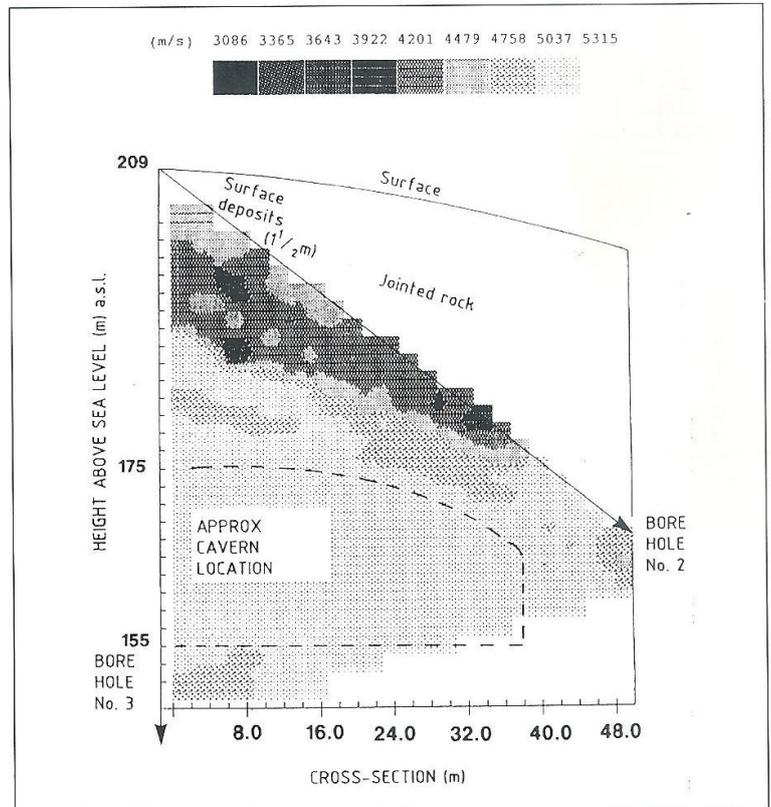
The implication here is that by use of such correlations in the future, cross-hole seismic tomography may be used in a more direct manner for specifying likely rock qualities and potential rock support needs in tender documents.

ROCK SUPPORT PREDICTION

In the geological mapping phase, rock mass classification plays an important part in the recording and graphical presentation of data. In some tunnelling projects in Norway, due to minimal soil deposits above significant lengths of tunnel, it is possible to set up a log of recorded  $Q$ -values along these planned sections of tunnel. The engineering geologist uses his previous knowledge of diverse projects to predict the improved quality likely to be found at depth in the same rock formation. Relevant values of RQD and number of joint sets ( $J_n$ ) will be somewhat affected, likewise the general anticipation of reduced amounts of clay fillings in discontinuities at tunnel depth (i.e., potentially reduced  $J_a$  and SRF values).

Where geological mapping of the bedrock is prevented by soil cover, interpretation of seismic surveys (for example using the correlations

Figure 4. An example of cross-hole seismic tomography for extrapolating data between boreholes: Gjøvik Olympic cavern.



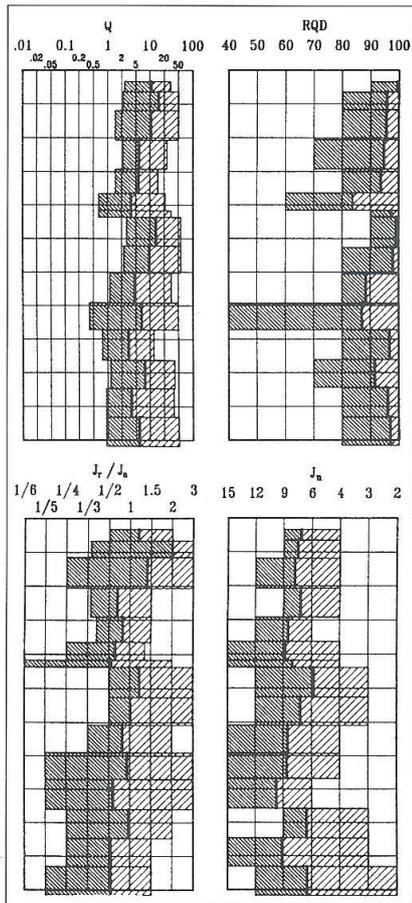


Figure 5. Examples of depth logs for key Q-system parameters from a section of a Sellafield borehole.

given in Table I) can be used to extrapolate data. Selected drill core may also be available for calibrating the extrapolation from the surface down to the tunnel depth, and for probing below the soil cover.

Extensive use of drill core is presently being made by UK Nirex Ltd. in England, for investigating the geological and hydrogeological conditions at depth for possible siting of a repository for solid low-level and intermediate-level radioactive waste. NGI and WS Atkins are utilising the Q-system (and other rock mass parameters given in Figure 3) for presenting likely rock qualities as a function of depth for assisting Nirex in planning the repository layout and access method.

Figure 5 shows depth logs for Q-values and for some key Q-parameters for one of the boreholes at Sellafield. Dark shading shows typical poorest quality rock and light shading: typical best quality rock. Direct use of Q-system support tables and the data in Figure 1 provide the preliminary rock support requirements for access tunnels and caverns.

When a tunnelling project is under way it is very convenient to map conditions using the tunnel logging chart shown in Figure 6. This gives Q-parameter observations on the left hand side, while principal geologic structure,

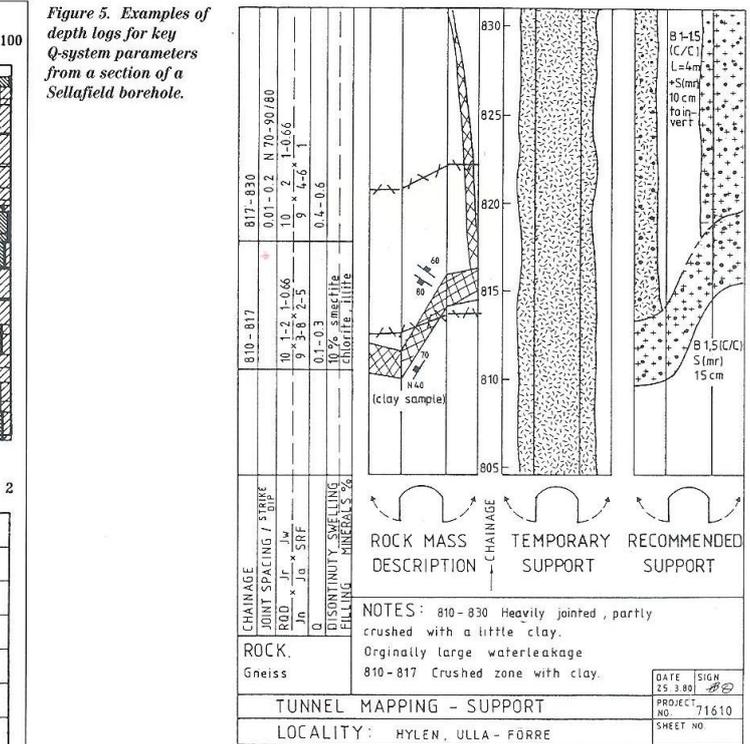


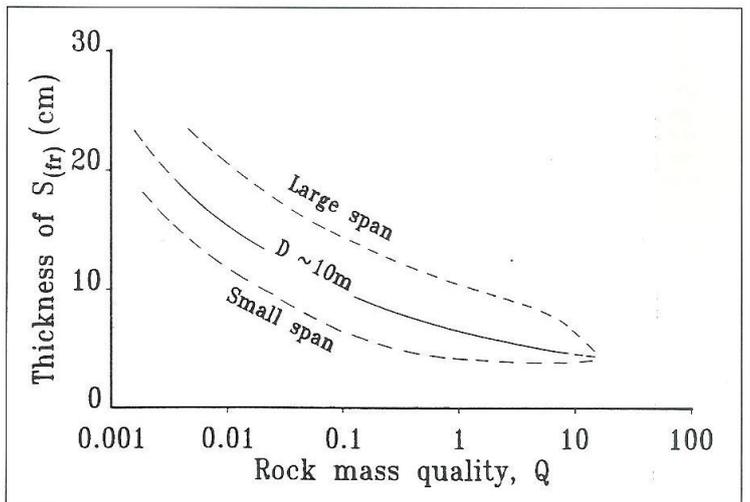
Figure 6. Example of a tunnel logging chart for recording temporary and permanent support: Ulla Førré (Barton et al., 1980)<sup>1</sup>.

temporary support and final Q-based support recommendations are given as symbolic logs. The tunnel shown is a 17m high by 10m span headrace tunnel to Hylene hydropower station at

the 2100 MW Ulla Førré hydroelectric project in western Norway (Barton et al., 1980).

The recommendations for rock reinforcement provided by the Q-system are derived in several stages. Following evaluation of the Q-value and the span of the tunnel, the user must choose a

Figure 7. Typical thickness of fibre reinforced shotcrete used at Norwegian tunnel sites in the late 1980s (Grimstad et al., 1991)<sup>15</sup>.



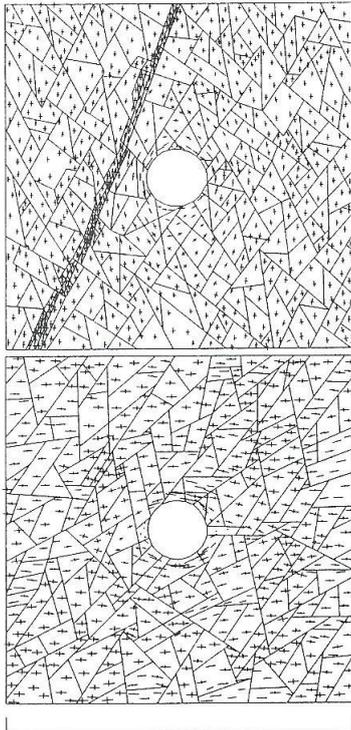


Figure 8. Sections of a TBM tunnel at two locations and depths around a planned spiral access tunnel. UDEC-BB results showing principal stresses. Bolted cases. (Barton et al., 1992)<sup>8</sup>.

suitable value of ESR for modifying the span or height of the excavation. This ESR number is a means of choosing different levels of safety; a lesser degree of safety may be acceptable in a non-entry tunnel such as a water tunnel (where ESR = 1.6), as compared to a road tunnel (where ESR = 1.0). The equivalent span given in Figure 1 is equal to span/ESR (expressed in metres).

To take an example: a recommendation for systematic bolting and fibre-reinforced shotcrete would be obtained with  $Q = 0.4$  (very poor) and span/ESR = 10m (Figure 1). Required bolt spacing and shotcrete thickness would then be estimated from Q-system support tables (Barton et al., 1980)<sup>1</sup> or from Figure 7.

A choice of the bolt diameter must also be made, based on support pressure estimates. Bolts of 20, 25 and 32mm diameter generally have yield limits of approximately 15, 25 and 40 t, and spacing should therefore be integrated with the chosen diameter, taking into consideration other factors such as block size, etc.

### INTEGRATION OF Q-SYSTEM AND NUMERICAL MODELLING

In important excavations and on projects where the Owner's documentation requirements are quite high, it will be usual practice to check Q-

system derived bolting recommendations with numerical models. This has been done for example for a representative section of the Oslo motorway tunnels of 13m span (Chryssanthakis et al., 1991)<sup>2</sup>, and for the bolts and untensioned cables used to reinforce the 62m span Olympic Ice Hockey cavern at Gjøvik (to be used for the 1994 Winter Games). Examples of the numerical modelling using the discrete element code UDEC-BB for this purpose were given in the November 1991 issue of *World Tunnelling* (Barton, 1991)<sup>11</sup>.

In preliminary Q-system design checks performed for UK Nirex Ltd, NGI have utilised UDEC-BB for checking the stability of specific sections of the planned spiral tunnels needed to access the planned radioactive waste repository at approximately 800 metres depth at Sellafield.

Figures 8 and 9 show the stresses and displacements derived for TBM driven tunnels of 8.4m span which were numerically bolted a certain period after excavation with bolts of 1.5 m c/c, length 3 m and diameter 20 mm, as derived by the Q-system. The final forces developed in each of the bolts at joint crossings could then be compared with the yield levels of 25 t to decide if increased capacity was needed.

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The foregoing constitutes the first part of a two part feature. Part 2 dealing with contract systems, support methods and equipment will appear in the next issue of WT.

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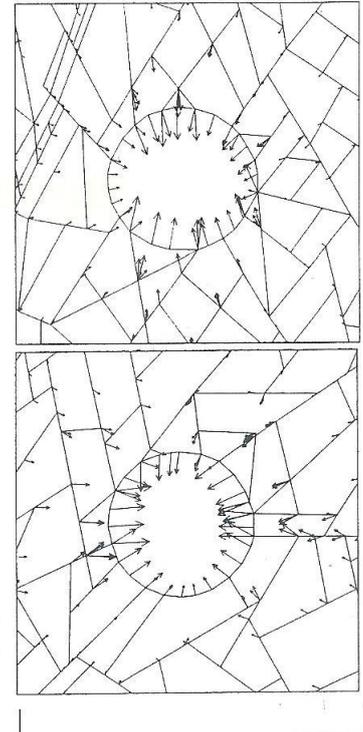


Figure 9. UDEC-BB results showing displacement vectors for the two cases shown in Figure 8. Both models are bolted according to Q-system logging results. (Barton et al., 1992)<sup>8</sup>.

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