

Rock mass conditions dictate choice between NMT and NATM

The Norwegian Method of Tunnelling is most appropriate for drill+blast tunnels in jointed rock which tends to overbreak. Nick Barton and Eystein Grimstad, Norwegian Geotechnical Institute, discuss the different applications of NMT and NATM, usually employed in driving soft ground tunnels.

The NATM support design philosophy has been employed on numerous occasions for soft ground tunnelling¹⁰. In general, it has been used with great success. The soundness of an active design approach, sometimes called design-as-you-go (more correctly design-as-you-monitor), has been demonstrated by major cost savings compared to conventional, inflexible design approaches. However, it would be unfair to the NATM concept, and also incorrect, to refer to all tunnels that incorporate shotcrete and rockbolting in their method of construction as being 'driven by NATM', which appears to be occurring in some quarters.

NATM clearly cannot be the best or cheapest method for tunnels in extensively jointed, harder rock masses that are drill+blasted as opposed to machine excavated. Extensive overbreak (i.e. negative radii) frequently causes mesh reinforced shotcrete (S(mr)) and lattice girders to be impractical, time consuming and possibly unsafe. Such methods may also cause unnecessarily large concrete consumptions. For this reason, Norwegian tunnelers were only too ready to stop using mesh reinforcement and steel ribs within a few

years of their developing wet process, steel fibre reinforced shotcrete (S(fr)) in place of the earlier S(mr) method. Commercial application of wet process S(fr) in Norway by 1978 caused S(mr) to fall out of use by about 1984⁹.

Fibre reinforced shotcrete

Use of this revolutionary permanent reinforcement and final support method for jointed ground with overbreak since 1978 has increased from 60 000 to 70 000m³/year in Norway, close to the highest use in the world at present, despite Norway's small population. Robotic application 10 to 20m above, to the side of or in front of the operator, production rates of 10 to 25m³/h, low dust levels (rebound 5 to 10 per cent), secured rock bolting conditions in unstable ground, and no problems with uneven profiles and overbreak, have caused a revolution in driving rates and tunnelling costs.

Cast concrete lined sections for permanent support of fault zones and clay-bearing rock are virtually disappearing from use due to their cost and time constraints as compared to S(fr). Rib (rebar) reinforced shotcrete (RRS) with S(fr),

unreinforced shotcrete (S) and bolting (B) are now used as permanent support in such zones at approximately half the cost of cast concrete. Similar advantages can be expected when S(fr) and RRS are used as permanent support in tunnels or caverns in soft jointed rocks and in over-consolidated fissured clays, such as London Clay.

The no-nonsense B+S(fr) Norwegian Method of Tunnelling (NMT) allows drill+blast driving rates of up to 40 and 70m a week in 75 hours and 110 hours a week tunnelling. These figures are achieved even when significant amounts of shotcreting and bolting are performed. Maximum rates of about 60 and 100m a week are achieved when there is only minor rock reinforcement.

Surprisingly, the use of shotcrete as the final lining of major tunnels in Scandinavia has gone relatively unnoticed. A recent survey of major tunnels with S(fr) as final support overlooked both Norway and Sweden in this respect, due no doubt to the commonplace use of these methods in Scandinavia which goes largely unreported¹¹.

There are in fact some 460km of main road tunnels in Norway which have stretches totalling 160km with S or S(fr) as approved final support, some of them subsea tunnels. An important point to remember is that the Norwegian Public Roads Administration is just as interested in maintenance free tunnels as its international counterparts.

No significant fibre corrosion

A common misconception is that S(fr) is unsuitable for long life, maintenance free tunnels, due to possible fibre corrosion. This is proving to be an unfounded worry, even in salt-water environments, provided that sensible precautions are taken. The key to success is good quality concretes.

High grade concretes with plasticisers, super-plasticisers, silica fume, slump killers and hydration control have extremely low water contents, permeabilities and porosities. Since the fibre is non-continuous, it does not suffer galvanic cell type corrosion as may occur with mesh reinforcement. Even the medium grade concretes such as C35 that were common with S(fr) application ten years ago do not show fibre corrosion in ten year old subsea tunnels. Convincing information on the environmental effects in such tunnels was

1) Areas of usual application:

Jointed rock giving overbreak; harder end of uniaxial strength scale ($\sigma_c = 3$ to 300MPa)
Clay bearing zones, stress slabbing
Q = 0.001 to 10 or more

2) Usual methods of excavation:

Drill+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 Fig 1)
- 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 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:

- Rapid advance rates in drill+blast tunnels
- Improved safety
- Improved environment

CCA = cast 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.

Table 1. Essential features of NMT (after Barton et al, 1992b).

SUPPORT

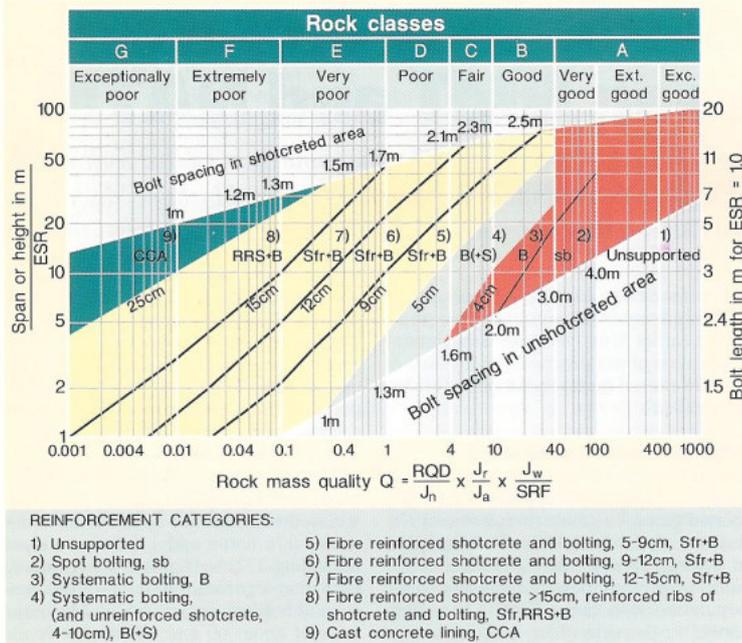
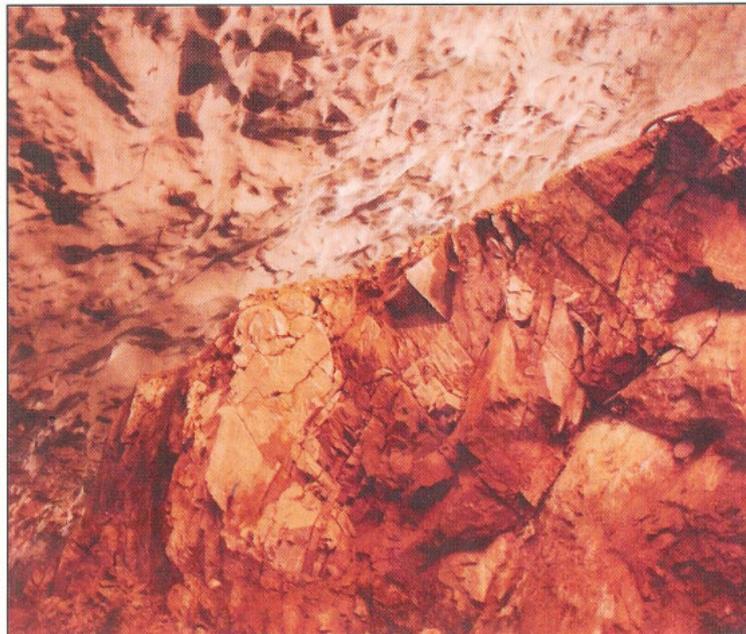


Fig 1. Updated Q-system tunnel and cavern design chart, based on NMT permanent reinforcement principles (Grimstad and Barton, 1993).

provided at the International Symposium on Sprayed Concrete at Fagernes, in October 1993¹².

The Norwegian Public Roads Administration recommend strength class C45 and environmental class MMA (water to binder mass ratio not higher than 0.40) for

S(fr) that is to be used in saline water zones. Not less than 70mm thickness of S(fr) is generally recommended in subsea tunnels where S(fr) is to be used. Recent guidelines have been given in the Norwegian Concrete Association's publication No. 7⁸.



Typical jointing and overbreak at the Gjøvik cavern: view along the north side of the 62m span.

Fibre lengths of between 20 and 45mm, fibre dosages of 40 to 90 kg/m³ (for high and low aspect ratios) and different classes of concrete, i.e., C25 to C45, can each be used to tailor the shotcrete to either 'ride with' or severely limit the deformation during the periods between temporary support, further tunnel advance and final support.

The speed of application, flexural strength and exceptional toughness index of S(fr) represent revolutions in tunnelling that are perhaps of equal importance to the revolution caused by rockbolting and blasthole drilling with hydraulic as opposed to pneumatic drilling jumbos. The Norwegian B+S(fr) method of tunnelling has proved its worth over and over again in the 1600km of tunnels driven through Norway in the last 15 years.

NATM versus NMT

NATM is most appropriate for soft ground, machine excavated tunnelling. It is based on a descriptive ground classification (often about six classes), appropriate selection of temporary support based on these ground classes, monitoring of deformation, and application of additional support such as mesh reinforced shotcrete and lattice girders in order to satisfy the best principles of ground reaction curves. A uniform, load bearing structure is usually the end product.

Due to the soft nature of the ground and the type of the tunnel or cavern, NATM is frequently followed by cast concrete linings, perhaps with a waterproofing membrane. A closed invert is used in very weak ground.

NMT, which is most appropriate for drill+blast tunnels in jointed rock which tends to overbreak, is frequently based on a quantitative rock mass classification such as the Q-system¹, appropriate use of temporary reinforcement such as bolting and fibre reinforced shotcrete, and supplementary reinforcement and support according to the engineering geologist's Q-based permanent support design. The tunnel span and the purpose of the excavation also figure in this selection of final support. Essential features of NMT are summarised in Table 1.

The final tunnel lining is most likely to consist of B + S(fr) when a lining is required. Monitoring will generally not be performed unless the rock mass is of extremely poor quality ($Q < 0.01$), or unless the span of the excavation is exceptionally large. The Gjøvik Olympic cavern of 62m span was, of course, extensively monitored despite Q-values in the range of 1 to 30, i.e., poor to fair to good quality⁷.

Q-system update for NMT

An update of the Q-system, specifically for selecting NMT support, was recently published, based on more than 1000 new case records from main road tunnels⁷. The new design chart shown in Fig 1 indicates

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: iii) 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	
a) Rock-wall contact, and b) rock-wall contact before 10cm 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.

c) No rock-wall contact when sheared			
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		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 10cm 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)			
K	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
O	Thick, continuous zones or bands of clay	6-24°	10.13, or 13-20
P	(See G, H, J for description of clay condition)		

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 pressure in competent rock with unfilled joints	2.5-10		0.5	
D	Large inflow or high pressure, considerable outwash of joints 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			
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, stress problems					
		σ_c / σ_1	σ_θ / σ_c		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_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_θ / σ_c			SRF
O	Mild squeezing rock pressure	15			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 $q \approx 0.7 Y Q^{1/3}$ (MPa) where Y = rock density in kN/m^3 (Singh, 1993).

d) Swelling rock: chemical swelling activity depending on pressure of water					
R	Mild swelling rock pressure				5-10
S	Heavy swelling rock pressure				10-20

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 $\tau = \sigma_n \tan (J_r / J_a)$).

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

Table 2. Ratings for the six Q-system parameters (SRF updates by Grimsdall and Barton, 1993).

increased use of S(fr) since 1986 (the previous support method update). Table 2 gives the ratings of the six Q-system parameters for describing the rock mass at any particular site. This table has been updated and improved in the SRF ratings only, due to the direct application of modern NMT, i.e., B+S(fr), in rock bursting and stress slabbing ground.

The generally increased bolt spacing and reduced thickness of S(fr) shotcrete as compared to early case records with S(mr) will be noted by previous users of the Q-system. Bolt spacing is also larger when S(fr) is used than when no shotcrete is applied. A cheaper and safer tunnel construction is generally achieved with S(fr) than with S(mr) because of problems caused by overbreak and the risk of fixing mesh reinforcement and bolting beneath unreinforced shotcrete.

Reinforcement categories 8 and 9 shown in Fig 1 are perhaps the closest that NATM and NMT ever come to overlapping one another, since monitoring would also tend to be used in the NMT design due to uncertainty and the probable benefit of adjusting the design according to monitored behaviour.

Combining NMT and NATM

In the past, specific attempts to combine the Q-system with NATM have been reported. Certainly, a more quantitative description of the six or seven NATM rock classes using the Q-system or the RMR method of Bieniawski¹ is inherently attractive. However, the '1/2 Q-NATM' version suggested some years ago in South Africa is not recommended due to the excessive (50 per cent) cut in recommended Q-reinforcement for use as temporary support prior to NATM-style monitoring and final support. This might allow the ground to yield excessively before the NATM

phase could ensure stability.

An attractive combination of NMT and NATM principles has recently been proposed for a major tunnel in partly soft and partly hard rock. Up-front prediction of support needs using the Q-system, temporary support close to the face with B + S(fr), monitoring of resulting performance, and adjustment of support class (if necessary) for the application of final support well behind the advancing face appears to be an ideal combination of three well tried techniques, namely Q, NMT and NATM, which have been successfully used for numerous tunnels over the past 15 to 25 years respectively.

NMT with monitoring

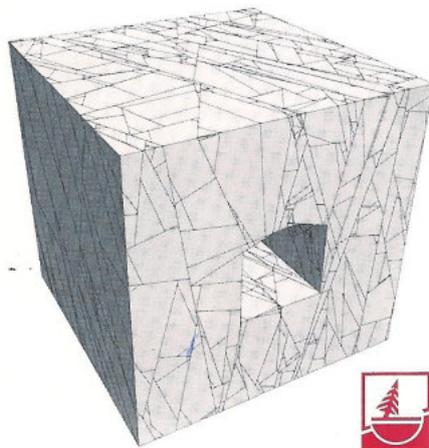
The photograph on p40 shows the rock mass conditions and typical overbreak encountered at the 62m span Olympic cavern at Gjøvik. This was excavated in jointed gneiss of 60 to 90MPa uniaxial compressive strength. The ability of the 100mm of S(fr) and bolting to reinforce the uneven surface of this enormous rock arch is apparent from the small deformations recorded by MPBX; values ranged from 6 to 8mm. The gneiss had an average RQD of about 70 per cent, a mean Q-value of about 9 (range 1 to 30), and three to four joint sets, causing marked overbreak. The cavern is designed to house 5400 spectators during the Winter Olympics of 1994. Permanent support is by NMT. ■

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