

NORWEGIAN METHOD OF TUNNELLING

Continuation from June issue

In June *WT* we introduced the idea of The Norwegian Tunnelling (NMT) as a hard rock rival to NATM. In this second instalment, Dr Nick Barton and his colleagues further explore its application against the background of Norway's contract system.

NORWEGIAN CONTRACTS

An inexperienced tunnel Owner who describes the desired final product with its concrete lining and leaves all risks to the Contractor in a Turnkey or Lump Sum, Fixed Price contract invites high costs, disputes and legal actions. Another extreme, also inviting high costs which this time maximises the Owner's risk and minimises the Contractor's risk is the Cost Reimbursement type of contract. Figure 10 illustrates where Norwegian practice lies (Kleivan, 1988)¹³

The contract system used in Norway which has a 20 year track record of low costs and few disputes is based on tender documents that reflect the unit prices for the equipment, methods and materials most likely to be needed for tunnelling through the investigated rock. An integral part of this system is a tender document that thoroughly describes the geological and geotechnical investigations, giving the Contractor a fair idea of likely rock conditions, rock support needs and details of all investigations performed. The Owner utilises engineering geologists from his own or from his Consultant's organisation who are experienced in tunnelling work, for this important task.

The tender documents therefore represent the best judgement of the Owner and his Consultants on most likely conditions. They list the different types and amounts of support work that are to be included in the tender sum, and request alternative unit prices for instance for driving a pilot heading, for probe-drilling, and for various pre-grouting strategies in case these are needed for parts of the tunnel. The required support work is divided into two main categories: that to be executed at the tunnel face, and that to be executed behind the tunnel face that does not delay the progress of the tunnel. Unit prices are also given for all delays and idle time for workers and for equipment, and of course for running costs for the workers' camp and for administration.

When rock problems are encountered, the Owner, Contractor and Consultant together choose the most suitable and practical methods for coping with the problems. The Contractor who has correctly priced different types of support will be able to give his best advice

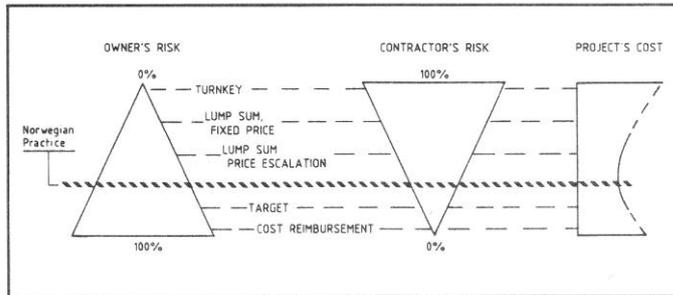


Figure 10. Risk sharing according to type of contract and assumed influence on project cost (Kleivan, 1988)¹³

without concern for "tactical motives". The Owner pays for the technically correct solution, no more and no less. (Aas, 1986)¹⁴

The idea of the Norwegian contract system is to help create a cooperative attitude among parties involved when difficult and unexpected conditions are encountered, yet still maintain

the advantage of a full competition at the tender stage since all eventualities have been priced. The need for the Owner to employ consultants and engineering geologists who have significant experience is fundamental to the proven success of the Norwegian tunnel contract system.

Great emphasis is laid on avoiding unnecessary damage to the rock mass due to careless blasting by the Contractor. In the tender documents the Owner asks for alternative unit prices concerning restricted

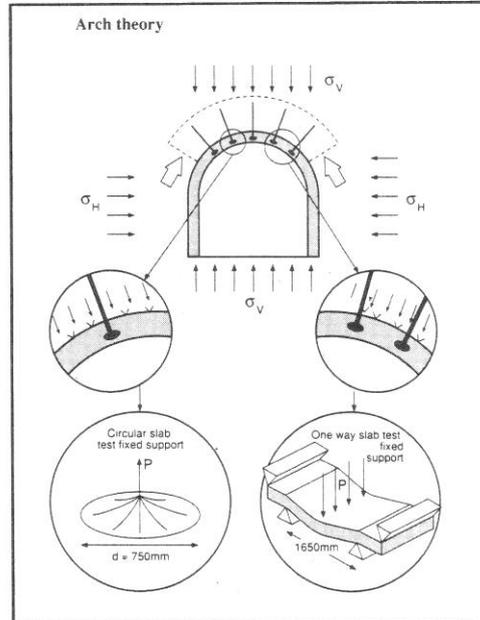


Figure 11. Analogy between a bolted shotcrete lining and the large scale load simulation rigs; the one-way slab test and the circular slab test. (Torsteinsen and Kompen, 1986)¹⁷

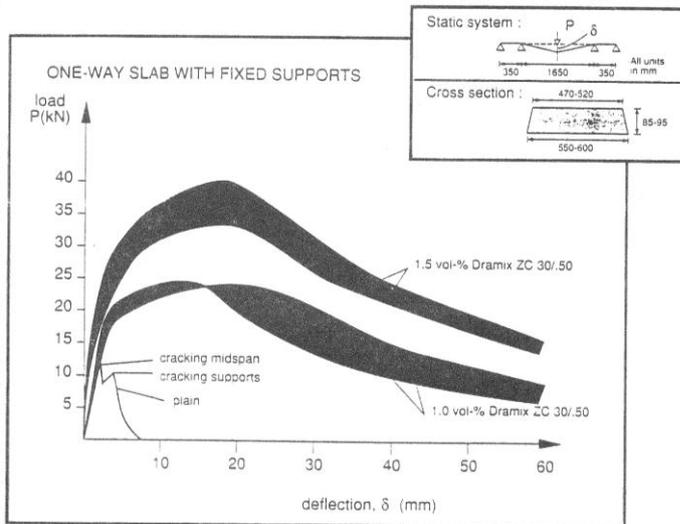


Figure 12. Load-deflection curves for the one-way slabs. One curve corresponds to one sample. (Shading between two samples of the same fibre dosage.)

lengths of rounds for certain rock qualities, modified drill patterns, spacing and changing of contour holes.

Finally, when evaluating the different tenders, the Owner uses skilled and experienced personnel to compare unit prices. The possible influence of changed rock conditions on ranking of tenders must be considered, and "tactical pricing" based on a Contractor's gamble on influencing the quantity of a given support method must be evaluated. It is obvious that an agreed rock mass quality documentation method such as the Q-system can play an important role in helping all three parties in this tendering and tender evaluation process, and of course in later disputes if such should arise.

TUNNEL SUPPORT

In spite of predominantly hardrock tunnelling in Norway, many fault zones, intense tectonic jointing, hydrothermal alteration zones and rock burst areas require rock support. The support used in our tunnels and large rock caverns varies to a large extent with the purpose of the excavation and the intended working life of the constructions. These aspects are addressed by the ESR number in the Q-system.

It is of great advantage to select a temporary support which can act as a permanent support later, or act together with other permanent support methods. The most commonly used support methods are: rock bolts (sometimes combined with steel straps), shotcrete (usually steel fibre reinforced), and cast concrete using steel shuttering. The length and spacing between the rock bolts, and the strength and thickness of the shotcrete can be designed in accordance with the Q-system.

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In recent years it has become more and more common in very poor rock conditions to replace the traditional cast concrete by steel fibre reinforced shotcrete combined with rock bolts and steel bar reinforced ribs of shotcrete (RRS).

ROCK BOLTS

A wide variety of rock bolt types are in use in Norway. Here we have to distinguish between temporary support and permanent support. For temporary support there is a need for immediate effect on the rock stability and for quick and simple installation. For permanent support the most important requirement is long term reliability. In many cases, efforts are made to combine all these demands in one single type of rock bolt in order to save money.

In hard rock conditions, point anchored rock bolts are widely used for temporary support. For this purpose rebar bolts and tube bolts with expansion shells, and rebar bolts end anchored with polyester resin cartridges are the most widespread types. All these types of rock bolts may be grouted later (at some distance from the face) for permanent support purposes.

In rock burst or squeezing rock conditions, end anchored rock bolts equipped with large triangular steel plates are usually used. Under such conditions, the steel has to act together with the rock mass and grouting has to be avoided.

Other bolt types such as the Split Set bolt, the wedge bolt and the Swellex bolt are not in common use in Norway.

STEEL FIBRE REINFORCED SHOTCRETE

When the rock mass quality is poor, or when the spacing between the rock bolts has to be decreased excessively, application of shotcrete is usually recommended. Since about 1984, the use

of mesh has greatly decreased in Norway, though steel straps are often used as temporary "bridging" between bolts.

Steel fibre reinforced shotcrete is used in order to stabilise the rock surface, or to carry larger forces from swelling clay, rock burst or squeezing ground. The thickness of the shotcrete and the need for combination with rock bolts varies with the rock mass quality. The thickness is decided using modified Q-system tables or Figure 7.

When significant deformation of the rock mass is expected, the flexural strength and the toughness index of the shotcrete are very important parameters. When only small deformations are expected, the compressive strength is more important. In both cases, the adhesion has to be taken into account. However, the adhesion varies to a large extent with the rock type and the mineralogy of the rock mass. Hence it may be inappropriate to specify the adhesion between the rock and the shotcrete in tender documents. In the case of a reinforced shotcrete arch, the thickness has to be calculated based on the support pressure, which may be decided empirically from the Q-value.

In the last three years, approximately 1000 new cases from hydropower tunnels and about 40 recently completed road tunnels have been mapped and classified according to the Q-system. The support measures carried out have been plotted against the recorded Q-values. The spacing between rock bolts in areas with and without shotcrete have been controlled.

In areas without shotcrete, the average bolt spacing varies from 1.2m when the Q-value is 0.1 to 3.4m when the Q-value is 30. In areas covered by shotcrete the bolt spacing varies from 1.3m when the Q-value is 0.1 to 3.0m when the Q-value is 10 (see Table 2).

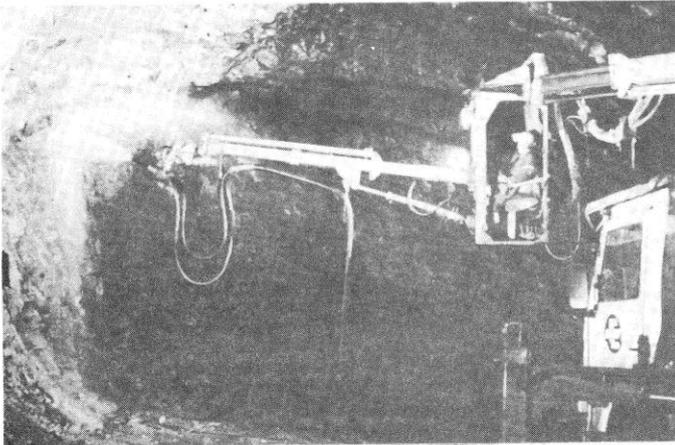
	Spacing between rock bolts in metres			
	Q = 0.1	Q = 1.0	Q = 10	Q = 30
Areas without shotcrete	1.2	1.4	2.0	3.4
Areas with shotcrete	1.3	1.6	3.0	4 to 5

Table 2 Bolt spacing as a function of Q-value with and without shotcrete (Grimstad et al., 1991)¹⁵

WET PROCESS SHOTCRETE

The early 1970s saw the introduction of wet process shotcreting in Norway. However, concrete quality problems initially arose due to the high w/c ratio needed for yielding pumpable concrete. For this reason, the dry method remained popular. However, in the middle of the 70s, silica and plasticizers were introduced and this meant an entirely new era for wet shotcreting in Norway. High capacity combined with high concrete quality at low cost was the result.

The wet shotcreting concrete is composed of Standard Portland Cement (in some cases quick setting cement), sand of maximum diameter



8-10mm, micro-silica, plasticising and super-plasticising agents and accelerator. The concrete is mixed nominally of 50 to 60 MPa strength depending on the specified end quality. For tunnel linings a quality of C35-45 is normally specified, for rehabilitation up to C55 is required. The applied accelerator is of a sodium silicate type that results in a relatively small reduction in concrete strength.

Sodium silicate addition is in the order of 15 to 25 litres per m³ of concrete with an average of 20 litres per m³. The ratio of water to combined cement and micro-silica is 0.40 to 0.45, yielding a slump of 15 to 20 cms. With the use of micro-silica (approximately 8-10% by weight of cement) the result is a highly cohesive concrete with good adhesion to the rock surface. The concrete is supplied to the work site by Ready-Mixers and the accelerator is added at the nozzle. The rebound is of the order of 10% though often as low as 4 to 6%.

Prior to shotcreting, the rock surface is cleaned with a mixture of water and compressed air using the robot equipment and starting at the highest level.

The shotcrete is most commonly applied in thicknesses ranging between 5 and 10 cms.

NORWEGIAN WET PROCESS

Incorporation of steel fibre reinforcement in shotcrete was commercially introduced in Norway in 1978. Its introduction led to a rapid change to the remote controlled application of sprayed reinforced concrete for rock support. By 1984, steel fibres had more or less replaced the use of wire mesh as reinforcement in Norway, and offered remarkable advantages over the earlier S(mr) technique still used in NATM. (Opsahl, 1982¹⁶; Torsteinsen and Kompen, 1986¹⁷)

The method has numerous references from hard rock tunnelling projects. While typical lining thicknesses for these applications are 5 to 10 cm, the technique has been steadily developed and steel fibre reinforced shotcrete

Figure 13. Robotic application of fibre reinforced shotcrete S(fr). (A/S Veidekke)

has gradually been used as an alternative to cast concrete linings in weakness zones, also those filled with clay. Under such conditions S(fr) may be combined with steel reinforced ribs of shotcrete (RRS).

Currently up to 40mm long x 0.5mm thick steel fibres are used in S(fr). These latest developments have set new standards for achievable ductility or toughness. A supreme advantage of the wet process application of fibre reinforced shotcrete is also the high capacity achievable.

Wet process steel fibre sprayed concrete S(fr) has the following main advantages:

- high application capacity, up to at least 25m³ per hour with piston pumps;
- efficient reinforcement can be applied at once;
- rebound in the range of 5 to 10%, i.e., much lower than dry process;

- no extra rebound of steel fibres: even with the 40mm fibres, the rebound can be controlled by a proper mix design combined with the "new generation" of admixtures for sprayed concrete;

- Uniform and high quality of sprayed concrete: normally up to 55 MPa characteristic cube strength, in special jobs, up to 100 MPa compressive strength;

- healthier working environment, i.e., less dust than for dry process shotcreting which causes much higher nozzle speeds;

- safe working conditions: application with a remote controlled nozzle manipulator (robot) without delay in bad ground, i.e., no mesh fixing in unstable areas;

- low permeability due to low w/c ratio;

- no continuous corroddible reinforcement as with S(mr) or reinforced concrete; avoids cathodic reaction and accelerated corrosion;

The method is most cost effective for application in large volumes for jobs requiring high capacities. This is due to the advanced and expensive equipment. The equipment also calls for skilled and educated operators, both with respect to machinery and concrete technology.

Figure 13 shows the robot arm and nozzle end of the technology.

Extensive test programmes on steel fibre reinforced shotcrete have been carried out and reported (Opsahl, 1982¹⁶; Torsteinsen and Kompen, 1986¹⁷; Vandervalle, 1991¹⁸). The main conclusions are:

- Fracture toughness increases considerably and is the property most positively influenced by steel fibre addition: longer fibres give better ductility.

- Other properties such as flexural strength, uniaxial tensile strength, bond strength and permeability are improved by steel fibre addition.

Figure 14. Typical Norwegian wet process shotcrete rig for application of up to 25m³ per hour with or without steel fibre reinforcement. (Entreprenørservice A/S)



- For specimens and structural members subject to realistic or adverse curing conditions, the improvements due to fibre addition are much more significant both for mechanical and durability properties. This is also verified by in situ test programmes in the North Sea and in sub-sea tunnels.

For tunnel support, the increased ductility of the applied S(fr) lining can be utilised especially well when in combination with systematic rock bolting. Large scale tests have been performed using S(fr) slabs and loading them as if loaded by rock bolts that are "too well" anchored into a yielding rock mass. Examples of such tests are given in Figures 11 and 12 showing point loading of beams and slabs to simulate the bolt loading. The shaded curves of load-deformation are for S(fr) while the small curve near the axis is for plain unreinforced shotcrete. The area under the curves is 30 to 40 times larger for the S(fr) samples, signifying both the high capacity and ductility, ideal for application close to a deforming tunnel face.

This "new" product (actually rather old now after some 13 years of use in Norway) has seen new admixtures and longer fibres (including stainless steel fibres) and more and more applications as an alternative to cast concrete linings, even for Q-values down to 0.001 (swelling clay problems and major fault zones, refer to Figure 1).

SHOTCRETING TRUCKS

Stabilising tunnel roofs during tunnel advancement requires high capacity and short rigging times. To meet these requirements, several major contractors in Norway (see Introduction) have mounted all necessary equipment on trucks to obtain completely self-contained mobile units certified for road travelling as well. A photograph of a modern Norwegian shotcreting rig is shown in Figure 14. Such rigs are used to deliver wet process shotcrete with or without the addition of the steel fibres described earlier.



Figure 15. The Rennfast project

A complete mobile rig typically contains the following units:

- concrete pump (mono pump or piston pump)
- shotcrete robot with mobile operator chair
- air compressor (electric or diesel powered)
- accelerator tank
- cable drum for 100m electric cable
- transformer
- high pressure cleaner for cleaning of equipment
- equipment for application of membrane hardener

The composition of the units is adapted to the fact that compressed air has been replaced by 660 volt electric power in Norwegian tunnels. The rig is operable on site once the electric power connection is made. Necessary time from arrival on site until 10-12 m³ of concrete have been placed is approximately 2 hours. The continuous capacity is 15 to 20 m³ per hour with top rates of 25 m³ per hour, making it particularly effective in excavations of large cross-sectional area.

Shotcrete has been successfully used for rock stabilisation in Norway for the past 25 years. The development of the wet shotcreting method in the 1970s has resulted in use on an industrial scale, with some 50-60,000 m³ of fibre reinforced shotcrete sprayed each year. Today, all tendering documents specify the use of this method and state the requirements as to quality, fibre addition, maximum rebound, etc.

RENNFAST SUB-SEA

This final section presents a case study describing how NMT enables one to drill through soft rock and long weakness zones at low cost and high excavation rate.

The Rennfast link consists of two 75 m², 3-lane sub-sea road tunnels, 30 km of roads, two bridges and two ferry ports, see Figure 15. The Byfjord tunnel, with its maximum depth of 223 m sub-sea and total length of 5830m, is the world's longest and deepest sub-sea road tunnel. The Mastrafjord tunnel reaches a maximum depth of 133m sub-sea, and has a total length of 4390m. Conventional drilling and blasting has been used

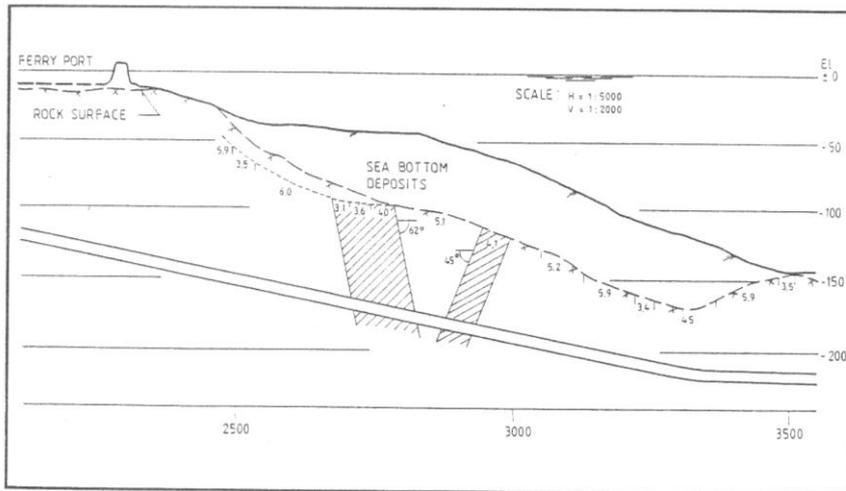


Figure 16. Major weakness zones at the Byfjord tunnel

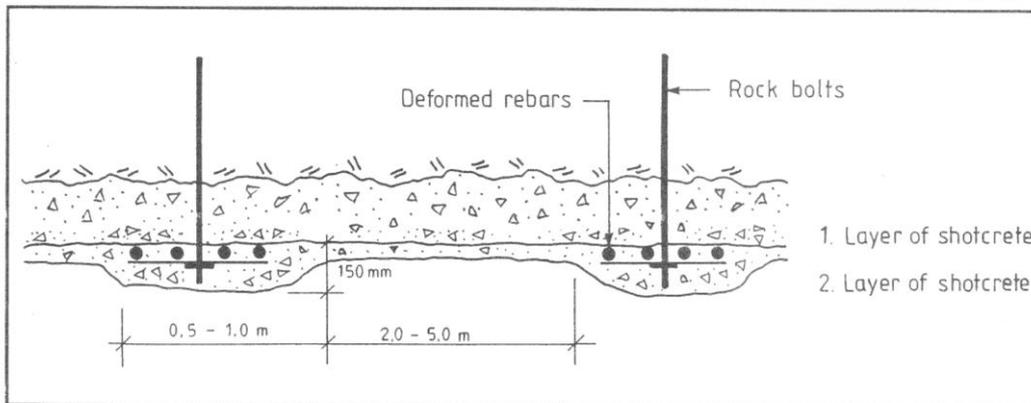


Figure 17. Support with ribs of reinforced shotcrete (RRS)

for the excavation. (Espedal and Nærum, 1991)¹⁹.

The cost of the two sub-sea road tunnels has been estimated at USD 75 million (NOK 410 million, USD 7,500 or NOK 41,000 per metre tunnel), with the total project costing an estimated USD 136 million (NOK 750 million, 1991 estimate). The project will be fully financed through a toll system, with the Public Road Administration as Owner. Construction of the Byfjord tunnel started in July, 1990, and successfully made the breakthrough in March 1992. The Mastrafjord tunnel started construction in August 1990 and achieved breakthrough in November 1991. The article presents a case study from the Byfjord tunnel.

Rock conditions have been mapped and observed in tandem with the excavation, in order to assign the proper rock support to each section of the tunnel. This was achieved by referring to the pre-established rock support specifications in the tender documents.

Preliminary studies including seismic refraction revealed a 130 m long weakness zone in the sub-sea part of the tunnel. Figure 16 shows how this zone was encountered in the tunnel and that it nearly intersects with another zone; both zones were sub-perpendicular to the tunnel axis.

The Byfjord tunnel runs mainly in phyllite which has two predominant sets of joints. The foliation joints have a dip sub-parallel to the tunnel axis. The foliation joints have a spacing of less than 100 mm and have a very smooth and undulating surface, occasionally slickensided. Filling material within the zones normally consisted of non-swelling quartz or calcareous filling less than 20 mm thick.

The main instability problem resulted from the foliation joint system combined with the medium to completely altered rock mass. The degree of alteration of the rock mass within the zones gave a stand up time of between 1 and 5 hours at the most. The rate of convergence (tunnel closure) was measured in 6 positions along a 400 m section of the tunnel. The maximum measured convergence was 300 mm. Rock mass classification (in accordance with the Q-system) gave Q-values ranging from 0.08 to 0.002 within the zones and values between 0.4 and 2.5 elsewhere in the Byfjord tunnel.

In the weakness zones, pre-reinforcement

with spiling bolts was carried out ahead of the excavation. Fully grouted bolts 25 mm in diameter were installed with a length of twice the blasting round length. Minimum spacing between the bolts was 300 mm. After the blasting and mucking, S(fr) was used to cover the entire section including the face with a maximum thickness of 25 cm. Depending upon rock conditions, either end-anchored rock bolts in a systematic pattern or cast, unreinforced or reinforced, concrete lining was installed as temporary support. Convergence was observed and, in areas with excessive deformation, additional support was installed after most of the deformation had taken place. The application of rock bolts in poor rock conditions was always used in combination with S(fr) to allow controlled deformation of the tunnel. The rock bolts, named combi-bolts, were then grouted through a system of grouting and air evacuation tubes.

The additional and permanent support consisted of a variety of additional bolts, S(fr) and ribs of reinforced shotcrete (RRS), see Figure 17.

These ribs were constructed of four 12mm diameter deformed rebars fixed by bolts to the initial layer of shotcrete. An additional layer of shotcrete, 15 cm thick, was used to cover the rebars. Each rib section was between 0.5 and 1.0m in length and the rib spacing ranged between 2 and 5 m.

While waiting for the deformation to take place, the excavation continued. The permanent support could be installed 10-25 m behind the face, without interfering with the excavation rate. At the Rennfast link, such a support system in very poor rock conditions enabled up to 25 m of weekly excavation rate without tunnel collapse.

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