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NATM, NMT and Hybrid of the Two Approaches for Tunnelling

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ABSTRACT

The NATM approach for tunnel design has been employed successfully on numerous occasions for soft ground tunnelling. The NMT, on the other hand, appears suitable for rock masses where jointing and overbreak are dominant. The paper describes key aspects of NATM and NMT to assist potential users of these methods in deciding between the two. A comparison of the two approaches has also been given. In the end, it is emphasized to use the hybrid approach, i.e. the combination of the best of NATM and NMT approaches.

Keywords: NATM; NMT; Instrumentation; Tunnel supports; Contract practices; Hybrid of NMT and NATM

1. INTRODUCTION

The New Austrian Tunnelling Method (NATM) and the Norwegian Method of Tunnelling (NMT) are the two approaches of tunnel design which are mainly being used in various tunnelling projects world-over. The concept of these two approaches is different. While NATM advocates for primary supports followed by secondary lining, the NMT suggests single shell of support.

The New Austrian Tunnelling method (NATM) was developed between 1957 and 1965 in Austria. Earlier, when rock bolt and shotcrete supports were not popular in India, people use to consider rock bolt and shotcrete supports as NATM. But, using rock bolt and shotcrete support only is not the NATM. In fact the NATM is a tunnel design approach based on the principal of 'build as you go' wherein the ground/rock mass strength is to be used as much as possible. Therefore, the monitoring is an essential component of NATM to verify and finalize the support design during construction.

The Norwegian Method of Tunnelling (NMT), as evident from the name, has an origin from Norway. Numerous case records, eventually more than 1250, have been used to develop NMT which is based on the Q-system of rock mass classification and tunnel support designs (Grimstad and Barton, 1993). NATM and NMT are discussed in detail by Singh and Goel (2006).

Q-system based approach, updated for the NMT supports, has been used for the tunnel design in India. NATM is also being adopted in the road and rail tunnel projects mainly in the Himalaya. Thus both the approaches, NATM and NMT, are being extensively used in India. For the benefit of field engineers, the two approaches have been discussed briefly in the paper highlighting their merits and demerits. A comparison of the two approaches has also been given at the end.

2. NEW AUSTRIAN TUNELLING METHOD (NATM)

The nNew Austrian Tunnelling Method (NATM) is a method of tunnel design and tunnelling. The New Austrian Tunnelling Method (NATM) was originally developed for weak ground, i.e. where the materials/rocks around the tunnel require supports because they are overstressed and unstable. The NATM is based on the philosophy of "*Build-as-you-go*" (strengthening of supports as the tunnel progresses) with the caution - *"Not too stiff, Nor too flexible; Not too early, Nor too late"* (supports).

In NATM the pre-construction phase tunnel support design concepts are based on rock mass behaviour types (RMBT) derived from rock mass type (RMT) and other influencing factors. The system behavior, subsequently, describes the rock mass-tunnel support interaction which is based on previous experience (including data base knowledge), analytical and numerical simulations. During construction, geological face mapping, geotechnical monitoring and observations allow the design of support and excavation methods to be checked and completed. The observed and predicted behaviours are compared by evaluating monitored deformations, support utilization, and overbreak volume. Deviations between the observed and predicted behaviour lead to a re-evaluation of the design process resulting in modifications to the support and excavation methods (Schubert, 2003).

The NATM accomplishes tunnel stabilization by controlled stress release. The surrounding rock is thereby transformed from a complex load system to a self-supporting structure together with the installed support elements, provided that the detrimental loosening, resulting in a substantial loss of strength, is avoided. The self-stabilisation by controlled stress release is achieved by the introduction of the so called "semi-rigid lining", i.e., systematic rock bolting with the application of a shotcrete layer. This offers immediate support and the flexibility to allow stress release through radial deformation. The development of shear stresses in the shotcrete lining in arched roof is thus reduced to a minimum (Singh and Goel, 2006). Later, when the rate of displacement is less than a specified limit, the permanent rock support mainly comprising of concrete lining is installed. *Thus, NATM has dual-lining support, primary supports followed by secondary lining*.

First time in India the NATM was used in Loktak hydroelectric project in 1980s. Since then, because of its in-built feature of managing the inhomogeneity of the rocks mass, NATM is being used in quite a number of tunnelling projects in the Himalaya in India.

2.1 Important Features and Principles of NATM

(a) NATM is based on the principle that utmost advantage of the strength of the rock mass should be taken to support itself by carefully controlling the forces in the re-distribution process which takes place in the surrounding rock mass when a cavity is made. The main feature is that the rock mass in the immediate vicinity of the tunnel excavation is made to act as a load bearing member, together with the supporting system. The outer rock mass ring is activated by means of systematic rock bolting together with shotcrete. The main load carrying member of the NATM is not only the shotcrete but also the systematically anchored rock arch.

(b) The installation of systematic rock bolting with shotcrete lining allows limited deformations but prevents loosening of the rock mass. *Whatever support system is used, it is essential that it is placed and remains in intimate contact with the surrounding ground and deform with it.* In the initial stage it requires very small forces to prevent rock mass from moving in, but once movement has started, large forces are required. Therefore, NATM advocates installation of supports within stand-up time to prevent movements/deformations.

(c) In static consideration a tunnel should be treated as a thick wall tube, consisting of a bearing ring of rock arch and supporting lining. Since a tube can act as a tube only if it is closed, the closing of the ring becomes of paramount importance, especially where the foundation rock is not capable of withstanding high support pressure such as in squeezing ground condition.

Thus, the basic principles of NATM are summarized as,

- Mobilisation of rock mass strength (achieved by allowing rock mass to deform in controlled manner). *(Using the surrounding ground as the main loading component is not an exclusive NATM philosophy. It is essential practice and is often inevitable, Barton et al.,1992)*.
- Shotcrete protection and other support to preserve the load-carrying capacity of the rock mass (the support must have suitable load-deformation characteristics and be installed at the right time. *This requires knowledge of inter-relation between ground deformation and load, support deformation and load, and time*),
- Monitoring the deformation of the excavated rock mass and action taken accordingly,
- Providing flexible but active supports, and
- Closing of invert to form a load-bearing support ring to control deformation of the rock mass.

2.2 NATM Rock Mass Classes

Various rock mass classes, sub-classes and expected rock mass behavior as per NATM are listed in Table 1, which is prepared using the data from various researchers (Onorm, 1994; Geoconsult, 1993; Karahan, 2010). The NATM classification is qualitative without a numerical rating. However, the NATM rock mass classes have been correlated with the values of Q and RMR (after Karahan, 2010) as shown in Table 1. The values of Q and RMR given in Table 1 are just an indication.

Table 1 - NATM rock mass classes with approximate Q and RMR values and expected rock mass behavior (After Onorm, 1994; Geoconsult, 1993; Karahan, 2010)

Rock Mass Class	Sub-Class of Rock Mass and RMR & Q Range	Expected Rock Mass Behaviour and Description
A	A1 RMR > 80; Q>70	Stable: The rock mass behaves elastically. Deformations are small and decrease rapidly. There is no tendency of overbreaking after scaling of the rock portions disturbed by blasting. The rock mass is permanently stable without support.
	A2 RMR = 65 to 80; Q =10 to 70	Slightly Overbreaking: The rock mass behaves elastically. Deformations are small and decrease rapidly. A slight tendency of shallow overbreaks in the tunnel roof and in the upper portions of the sidewalls caused by the discontinuities and the dead weight of the rock mass exists.
B	B1 RMR = 58 to 65; Q = 5 to 10	Friable: Major part of the rock mass behaves elastically. Deformations are small and decrease rapidly. Low rock mass strength and limited stand-up time related to the prevailing discontinuity pattern, yield overbreaks and loosening of the rock strata in tunnel roof and the side walls if no support is installed in time.
	B2 RMR = 47 to58; Q = 1.5 to 5	Very Friable: This type of rock mass is characterized by large areas of non-elastic zones extending far into the surrounding rock mass. Immediate installation of the tunnel support will ensure controlled deformations, which can be kept small and cease rapidly. In case of a delayed installation or an insufficient quantity of support elements, the low strength of the rock mass yields deep loosening and loading of the initial support. Stand-up time and unsupported span are short. The potential of deep and sudden failure from roof, side walls and face is high.
	B3 RMR = 29 to 4; Q = 0.1 to 1.5	

C	C1	Rock Bursting: C1 is characterized by brittle and strong rock mass under high in situ stresses. The overstressed zone of rock mass fails suddenly leading to failure mechanisms such as spalling and rock bursting.
	C2 RMR = 20 to 29; Q = 0.03 to 0.1 (UCS<25 to 30 MPa)	Squeezing: This is characterized by weaker rock mass under high in situ stresses. The surrounding overstressed zone of rock mass fails and deform inside the tunnel due to shearing. Rock mass shows a moderate but distinct time dependent squeezing behavior; deformations calm down slowly. Magnitude and velocity of deformations at the tunnel boundary are moderate.
	C3 RMR = 10 to 20; Q = 0.008 to 0.03 (UCS<20)	Heavily Squeezing: C3 is characterized by the development of deep failure zones and a rapid and significant movement of the rock mass into the cavity. Deformations are of high order and decrease very slowly. Support elements may frequently be overstressed.
L	L1, Short-term-stable with high cohesion	Due to limitation of the unsupported spans at arch and face, the rock mass remains stable for a limited time.
	L2, Short-term-stable with low cohesion	No stand-up time without support by prior installation of forepoling or forepiling and shotcrete sealing of faces simultaneously with excavation. The low cohesion requires a number of subdivisions.

<u>Notation:</u> RMR = Bieniawski's rock mass rating and Q = Barton's rock mass quality

During tunnel excavation, the rock mass classes and sub-classes are obtained as per the actual geology encountered on the tunnel face. Accordingly, as per the expected behavior the supports are designed. To avoid the individual bias, it is generally suggested to engage different person for the design work and during the construction work.

2.3 Two Layers of Lining in NATM

Two layers of supports (lining) have been emphasized in NATM as follows:

Initial or first layer of support: The support is often carried out as an outer lining and designed to stabilize the rocks during excavation. It mainly consists of (i) shotcrete with or without mesh reinforcement, (ii) systematic rock bolting, and (iii) lattice girders or steel ribs as per the rock and/or support class. In addition, an invert support is also installed to close the support ring in very weak ground.

Final or second layer of support: This is often comprises of a concrete lining. The concrete lining may be RCC or PCC as per the ground conditions and the tunnel deformations. It is generally advised to place the concrete lining only after the deformations of the initial or primary support have stabilized or showing decreasing trend to an acceptable limit.

The initial support can partly or completely represent the total support requirements. The second final lining over the initial layer of support is advocated to be necessary for long-term stability and for structural reasons, such as (i) when the initial lining is stressed beyond its elastic limit or (ii) when squeezing or swelling from time-dependent loads will exceed its bearing capacity, (iii) to take care of deterioration (corrosion etc.) of initial support in long-term and (iv) second lining is required for waterproofing.

The dimensioning of the final support is based on the results of systematic measurements of stress in the primary support element and/or deformations of the tunnel surface and the ground surrounding the tunnel. In order to investigate the real behaviour of the rock mass when the excavation is completed, the NATM is based on systematic in-situ measurements primarily of deformations and of stresses. Looking into the trend of deformations, it is possible to recognize an unacceptable trend and to act accordingly as discussed below.

2.4 Instrumentation

Monitoring plays a significant role in deciding the timing and the extent of secondary support. Timely strengthening of primary supports is important otherwise with large deformations, in some cases, the roof and walls need to be trimmed to get the space for secondary lining. This may again create instability problems. Thus, it is important to have a good instrumentation programme and immediate implementation of instrumentation results to strengthen the primary supports.

In a highway tunnel in J&K state, India the convergence has crossed the alarm limit which called for the countermeasures in terms of longer rock bolts and additional shotcrete layer (Fig. 1). After countermeasures, further monitoring shows that even the countermeasures were not adequate. Therefore, again additional supports were applied to make the tunnel stable. This is a good example of NATM showing how the supports were strengthened with time after excavation.

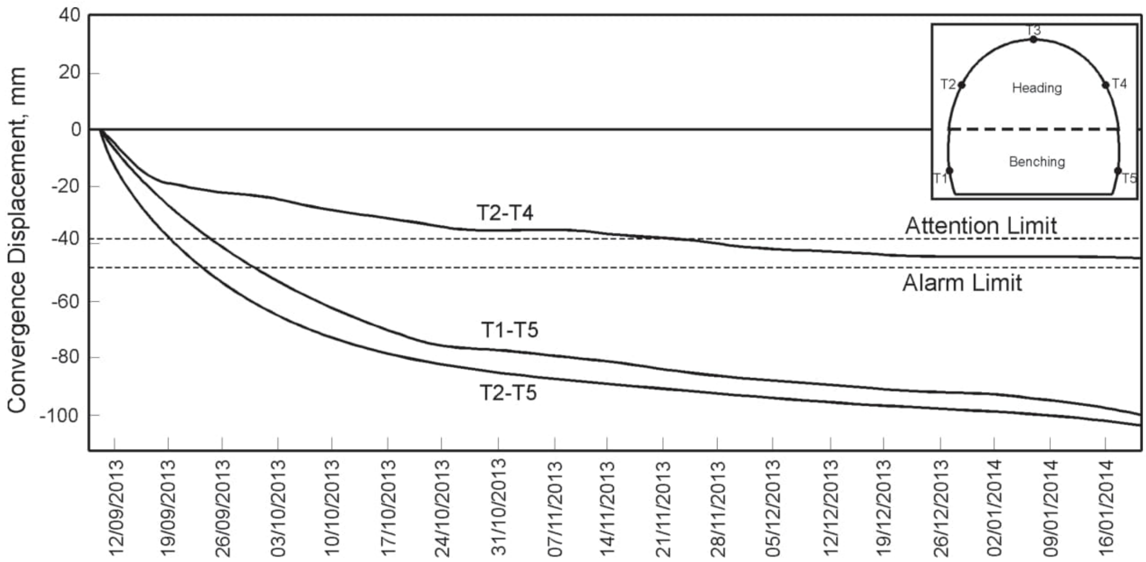


Fig. 1 - Additional rock bolt and shotcrete support installed after the convergence crossed the designed alarm limit in a highway tunnel constructed using NATM [T1, T2,…,T5 = Bireflex targets; T1-T5, T2-T4, T2-T5 = convergence between respective targets]

Of late, the use NATM in India has increased many folds. For its success it is important to have flexibility in the contract document. Some of the issues are given below.

2.5 Provisions in Contract for NATM

The NATM method of tunnelling is further improved where appropriate contractual arrangements are made (Palmstrom, 1993).

- The contract document should have greater sharing of responsibility and risk between owner, designer, and contractor. The use of NATM requires that all parties involved in the design and execution of project must understand and accept the implementation of NATM and adopt a co-operative attitude to decision making and the resolution of problems.

- Possibility of changes in support and construction method as per the ground condition: The monitoring presupposes that it is possible to make changes in support and construction methods as per the ground conditions encountered. This, however, is only possible if there are provisions in the contract that are open for changes during construction. (Austrian contractual practice does contain a certain amount of valuable features in terms of sharing risk and decision making, encouraging flexibility in construction methods, and providing simple and equitable arrangements for settling disputes. This is another important feature for using NATM in tunnel construction.)

- The monitoring program is included in the specifications and bill of quantities.

- Provision of payment as per actual site conditions: The level of detail in the pre-construction description depends on the information available from site exploration and experience from earlier tunnels in the vicinity. Brosch (1986) reports that Austrian engineers believe that a qualitative ground classification and contract conditions are inseparable. Clearly, such principles could lead to disputes, but since the contractor is paid on the basis of 'as found' conditions, possible disagreements are minimized (Palmstrom, 1993). An expert consultant shall be engaged in the project who shall visit the tunnel site at frequent interval and is available at short notice to look in to any disagreement.

- Payment for support is based on the rock mass classification made during construction. In some countries this is not acceptable contractually, and this is why the method has received limited attention (Palmstrom, 1993).

2.6 Precaution with NATM

- Different limit of tunnel deformations to be allowed shall be known to engineers and followed.
- Some time, where high deformations are observed, the deformed roof and walls encroaches the space available for secondary lining. Trimming the primary support and applying the secondary lining is not only costly and time consuming but also disturb the stabilized rock mass around the opening. Monitoring shall again be carried out to study the tunnel stability and then only the secondary lining support shall be applied.
- NATM shall be used cautiously in (i) inhomogeneous rock masses where stress concentrations on the lining can cause sudden outfalls and collapse (important to use the radial deformation of targets for deciding about the support strengthening and the time for final lining), (ii) some tunnels with high anisotropic rock stresses or loads where the arching effect do not develop, (iii) urban areas where the ground is not allowed to deform in order to avoid building settlements (Palmstrom, 1993).

3. NORWEGIAN METHOD OF TUNNELLING (NMT)

NMT is based on the Q-system of Barton et al. (1974). With more number of case histories and development of new supports with time, the support design approach and the supports proposed in 1974 has been modified to propose the Norwegian Method of Tunnelling by Grimstad and Barton (1993).

3.1 Features of NMT

NMT appears most suitable for rock masses where jointing and overbreak are dominant, and where drill and blasting method or hard rock TBM's are the most usual methods of excavation. The essential features of the NMT are summarized in Table 2 (Barton et al., 1992).

Table 2 - Essential features of NMT (Barton et al., 1992)

S.No.	Features
1.	Areas of usual application
	Jointed rock, harder end of scale (q_c = 3 to 300 MPa) Clay bearing zones, stress slabbing (Q - 0.001 to 10)
2.	Usual methods of excavation
	Drill and blast hard rock, TBM, hand excavation in clay zones
3.	Temporary support and permanent support may be any of the following
	CCA, S(fr)+RRS+B, B+S(fr), B+S, B, S(fr), S, sb, (NONE) * temporary support forms part of permanent support * mesh reinforcement not used * dry process shotcrete not used * steel sets or lattice girder not used, RRS used in clay zones * contractor chooses temporary support * owner/consultant chooses permanent support * final concrete lining are less frequently used, i.e., B+S(fr) is usually the final support
4.	Rock mass characterization for
	* predicting rock mass quality
	* predicting support needs
	* updating of both during tunnelling (monitoring in critical cases only)
5.	The NMT gives low costs and
	* rapid advance rates in drill and blast tunnels
	* improved safety
	* improved environment

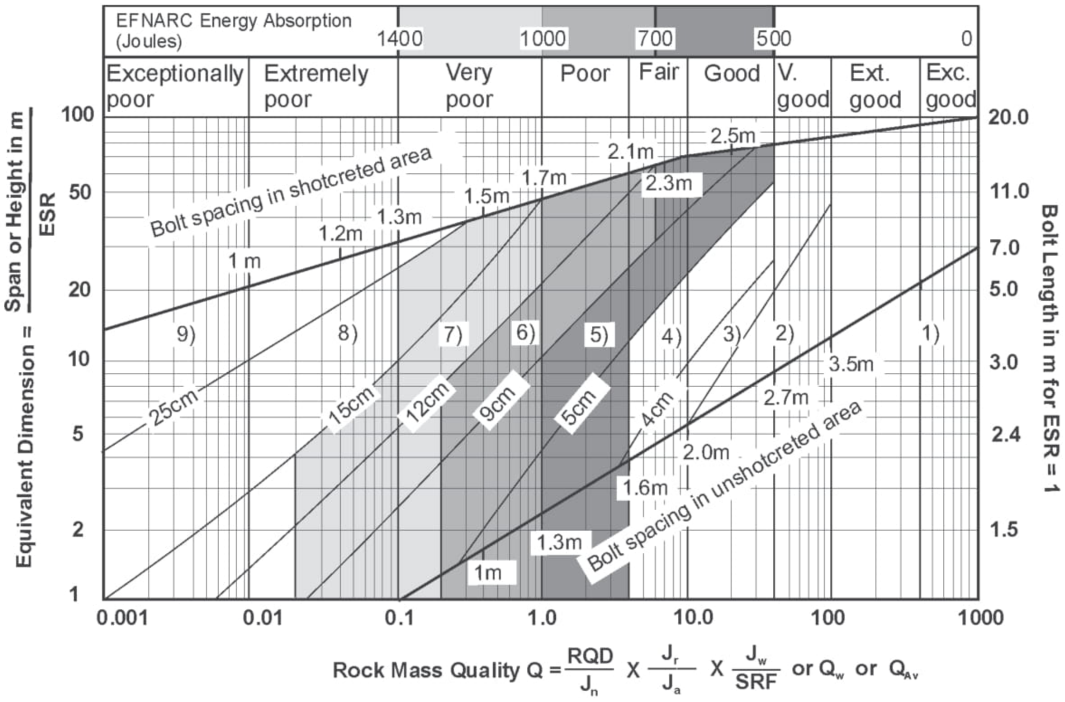
Notations: CCA = cast concrete arches; S(fr) = steel fibre reinforced shotcrete; RRS = reinforced steel ribs in shotcrete; B = systematic bolting; S = conventional shotcrete; sb = spot bolting; NONE = no support needed

In NMT great emphasis is placed on thorough description of geological and geotechnical aspects of the project (for forward prediction of conditions). The tender documents shall reflect as closely as possible the likely equipment, tunnel excavation methods and tunnel support materials for successful tunnelling through the investigated rock. The important feature of NMT is the forward prediction of conditions and agreed modifications for unexpected conditions. This is done as early and as accurately as possible. This minimizes disputes and also minimizes tunnel instability.

When unstable rock blocks and wedges with low shear strength are present (i.e. rock mass conditions $Jr/Ja \leq 1$ and $Jn/Jr \geq 6$), overbreak may be inevitable. In RMR (and in GSI) the number of joint sets is ignored, so there it may be difficult to understand the potential for overbreak, and the consequent advantages of B+S(fr) over concrete.

3.2 Design of Tunnel Supports

The support requirements based on the Q - system in case of NMT are shown in Fig. 2 where the Q-value is related to tunnel support requirements with the equivalent dimensions of the excavation. The relationship between Q and the equivalent dimension of an excavation determines the appropriate support measures.



REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting, sb
3) Systematic bolting, B
4) Systematic bolting (and unreinforced shotcrete, 4 to10cm, B(+S)
5) Fiber reinforced shotcrete and bolting, 5 to 9cm, S(fr)+B
6) Fiber reinforced shotcrete and bolting, 9 to 12cm, S(fr)+B
7) Fiber reinforced shotcrete and bolting, 12 to 15cm, S(fr)+B
8) Fiber reinforced shotcrete > 15cm, reinforced ribs of shotcrete and bolting, S(fr), RRS+B
9) Cast concrete lining,CCA

Fig. 2 - Grimstad and Barton (1993) chart for the design of support including the required energy absorption capacity of SFRS suggested by Papworth (2002) [ESR in Figure is Barton's Excavation Support Ratio]

Figure 2 is recommended for tunnelling in poor rock conditions also provided that more steel fibres are added in shotcrete so that energy absorption or the toughness is increased as shown in top row of Fig.2. In case the fly-ash is being used as the admixture in shotcrete/SFRS, Kadkade (2009) suggested using the fly-ash obtained from electrostatic precipitator.

Proposers of NMT advocate that with NMT the tunnel is not filled with concrete. As-needed support and reinforcement is selected on the basis of an empirical method, which was derived and updated from a very large number of successful case records. NMT is a cheap and fast tunnelling method. In case of fault zones, which are generally not much wide, the correctly designed and applied S(fr) is superior to many concrete linings: shrinkage cracks are avoided, damage by frost is avoided. Most importantly substantial construction time and cost can be saved (Barton et al., 1992). Barton (2009) has highlighted the use of pre-grouting in NMT and shown that for poor rock masses (Q <1) and in water charged rock masses pre-grouting by penetration into one or more of the joint sets can improve the tunnelling condition by improving most or all of the six Q-parameters.

More recent water 'control' has consisted of a sprayed membrane in an S(fr) sandwich. This has been used as a final measure to remove any remaining damp patches from the final layer of shotcrete. This can be used in pre-injected tunnels if needed in limited areas with remaining damp shotcrete. Sprayed membrane has been used in recent UK and Swiss tunnels, apparently even without pre-injection (Barton et al., 1992).

3.2.1 Shotcrete and rock bolting

It is understood that in NMT rock bolt and fibre reinforced shotcrete [B+S(fr)] are the two most used tunnel support elements because they can be applied to any profile as temporary or as a permanent support, just by changing thickness of shotcrete and bolt spacing. Bolting helps in mobilising the strength of the surrounding rock mass. Potentially unstable rock masses with clay-filled joints and discontinuities would increasingly need shotcrete and steel fibre reinforced shotcrete SFRS [S(fr)] to supplement systematic bolting (B).

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 preferred. All these types of rock bolts may be grouted later (at some distance from the face) for permanent support purposes. In rock burst and squeezing rock conditions, end anchored rock bolts equipped with large triangular steel plated are generally used. Under such conditions, the steel has to act together with the rock mass and grouting has to be avoided (Barton et al., 1992).

Wet process steel fiber reinforced shotcrete is suggested in NMT. Thorough air/water jet cleaning of the rock surface is also highlighted in NMT prior to shotcreting. Water cleaning is avoided in case of erodible or such soft rocks. Micro silica is needed in shotcrete (as also in pre-excavation grouting) as fibres need good anchorage in shotcrete of sufficient quality. This is achieved with the resulting lower water/(cement plus filler) ratios of 0.4 to 0.45. Thus strong bond between shotcrete and rock mass is ensured, and ensures less bending stresses in the lining and good performance during earthquake vibrations.

Good quality stainless steel fibres and the best bar chip polypropylene (surface-roughened) fibres are important for fracture energy enhancement of the S(fr). In the case of NMT the obvious choice is S(fr), which is considered essential for actively supporting rock with over-break, instead of leaving inevitable voids and 'shadows' when reinforcement is applied in the form of mesh instead of fibres (Fig.3).

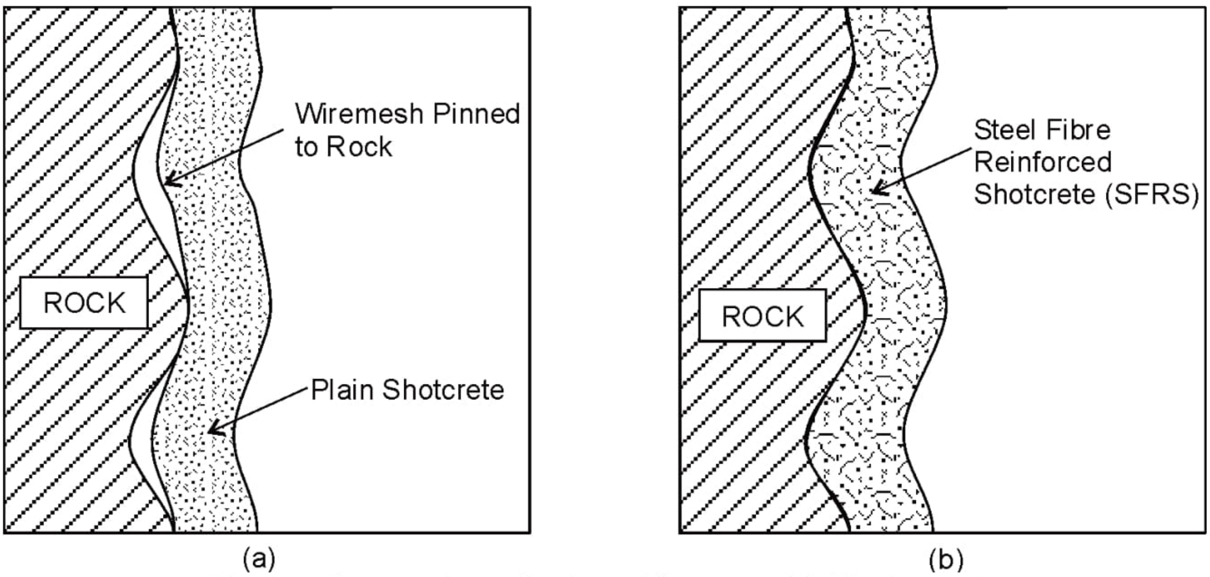


Fig. 3 - Difference in application of shotcrete with (a) wire mesh and (b) steel fibre (Vandevall, 1990)

Use of S(mr) also attracts deformation, and can even be dangerous, as it is an inefficient, multi-process and therefore delayed support measure (Barton et al., 1992). In unstable rock, arches of S(fr) can be built rapidly by robot application (with non-alkali accelerator).

3.2.2 Reinforced rib of shotcrete (RRS)

A thick load bearing ring of reinforced rib in shotcrete (RRS) can be formed as needed in extremely poor rock conditions (Fig. 4). RRS matches an uneven profile better than lattice girders or steel sets. In fact RRS is suggested in NMT as a replacement of lattice girder or steel rib.

Lattice girders are not used in NMT, as they attract extra deformation and have caused failures when applied 'as part of the Q-system'. Lattice girders have not been recommended in the Q-system because of potential overbreaks in harder rocks (Barton et al., 1992). These composite RRS (reinforced rib in shotcrete) arches are found to be superior to lattice girders. By contrast, lattice girders (and steel sets) depend on deformation before they start to apply 'support pressure'. With RRS one is fortunately not depending on footing stiffness, which is also an advantage.

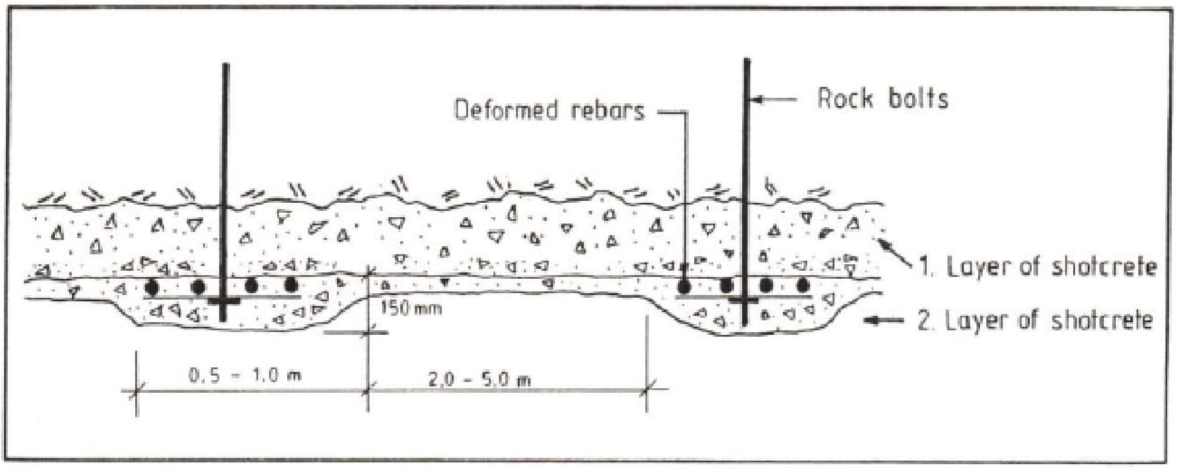


Fig. 4 - Reinforced rib of sprayed shotcrete (RRS) (Barton et al., 1992)

3.3 Contract Practices in NMT

- The tender documents reflect the unit prices for the equipment, methods and materials most likely to be needed for tunnelling through the investigated rock. Thus the tender document 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.
- To avoid the delay in the progress of tunnel construction, the required support work is divided into two main categories: supports to be executed at the tunnel face, and supports to be executed behind the tunnel face.
- Unit prices are also given for all delays and idle time for workers and for equipment and of course for running cost for the workers' camp and for administration.
- 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 lengths of rounds for certain rock qualities, modified drill patterns, spacing and charging of contour holes.
- Using the ratios of Q parameters, different aspects of tunnelling can also be checked. For example, the issue of overbreaks can be controlled by looking into the ratio of J_r/J_a and J_n/J_r.

- Owner must have consultants and engineering geologists having significant experience. Their services are taken for evaluation of tender documents and at all other stages of the project. This is fundamental to the success of NMT contract system.

4. COMPARISON OF NATM AND NMT

Using the above explanations from various researchers and author's experience Table 3 has been prepared showing the comparison of NATM and NMT.

Table 3 - Comparison between NATM and NMT

S.No.	NATM	NMT
1.	Most suitable for soft ground which can be machine or hand excavated where jointing and overbreak are not dominant. May be applied to other grounds as well excavated by drill & blast. Suitable to tackle uncertainty due to weak and unexplored or less explored ground.	Most suitable for harder ground where jointing and overbreak are dominant and where drill & blast and hard rock TBMs are the most usual method of excavation.
2	Qualitatively classify the rock mass on the basis of its expected behavior.	Quantitatively classify the rock mass using the Q-system.
3	For the expected rock mass behavior, different supports are designed. This is further evaluated during construction on the basis of actual geological, and geotechnical observations and deformation monitoring.	The supports are designed as per the actual Q-value obtained after excavation. The supports are evaluated by estimating the support pressure as per the actual geology encountered and comparing it with the predicted value of support pressure.
4	Depends on monitoring to decide on the amount of additional support and timing for secondary lining.	Based on Q system, it is a forward predictive method emphasizing fair prediction of unexpected ground condition and modifications accordingly as early as possible.
5	Dry or wet shotcrete with wire mesh; rock bolts; lattice girders, steel ribs.	Wet fibre reinforced shotcrete; corrosion resistant rock bolts; reinforced rib in shotcrete (RRS) [see Table 2 & Fig. 2]. The split set bolt and swellex bolts are not common in NMT.
6	Use of ground reaction and support reaction concepts and numerical/analytical analysis to evaluate the system behavior.	Application of numerical modelling to check Q-system derived bolting recommendations.
7	Dual lining; primary support plus final cast in situ concrete lining.	Primary support having number of layers of shotcrete as per the ground/rock mass. Cast concrete lining is suggested only in case of exceptionally poor rock mass (see Fig. 2) .
8	Comparatively longer time of construction.	Shorter time of construction .
9	An expensive approach in overbreak conditions.	A comparatively economical approach.

5. HYBRID OF NATM AND NMT

Barton and Grimstad (1994) have suggested combining the best of NATM and NMT. Hybrid NATM-NMT is the obvious solution for tunnels through rock masses having both poor and good conditions. Tunnel construction costs can be reduced using such hybrid measures, for example by adopting single shell lining sufficient quantity of concrete may be saved which will additionally help in reducing CO_2.

A combination of NMT and NATM principles can be used for a 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 a ideal combination of NMT and NATM. This is incidentally being followed in some of the Indian tunnels as well.

Barton and Grimstad (2014) have also highlighted as to how the Q-system can be used in the selection of temporary supports (for NATM), if desired, using Fig. 2 by taking 1.5ESR and short-term Q value, i.e. 5Q. They have used this method in Hong Kong metro and rail tunnels.

6. CONCLUSIONS

The type of rock mass/ground which best suits for the applicability of NATM and NMT has been highlighted. Using the NATM in hard jointed rock mass having overbreaks might not be appropriate and costly. Hence, one should have detailed information about the ground to be tunnelled through before taking the decision between the use of NATM and NMT.

The discussions in the above paragraphs on NATM and NMT also highlights the merits and demerits of different supports suggested in the two approaches. The designers must take the advantage of this while deciding the methodology. An approach having the combination of NATM and NMT, i.e. hybrid approach seems to be the future approach which still needs some detailing and research.

'Build-as-you-go' (or design-as-you-monitor) though is said to be the philosophy of NATM, but is applicable to both NATM and NMT in modern day tunnelling. Therefore, monitoring shall be the essential part of tunnelling to overcome the inhomogeneity of the rock mass and to back-analyse the design parameters.

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