Defining NMT as part of the NATM SCL debate Sep 2012

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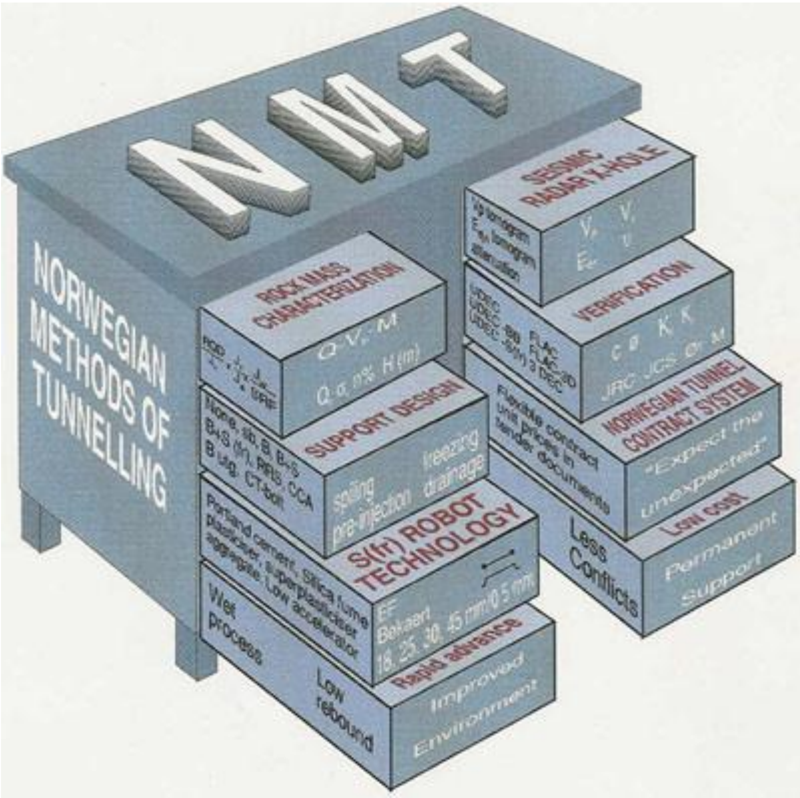
In response to Feedback to the *TunnelTalk* NATM and SCL article earlier this month, I suggested the addition of NMT to the pool of tunnelling method names. If we are seeking definitions, as per the longer Feedback definition of SCL contributed to the original article (Rekindled NATM debate - SCL debate opens - *TunnelTalk*, Aug 2012), then let me try defining and describing NMT a bit more thoroughly, as it is very different from NATM and quite different from SCL.



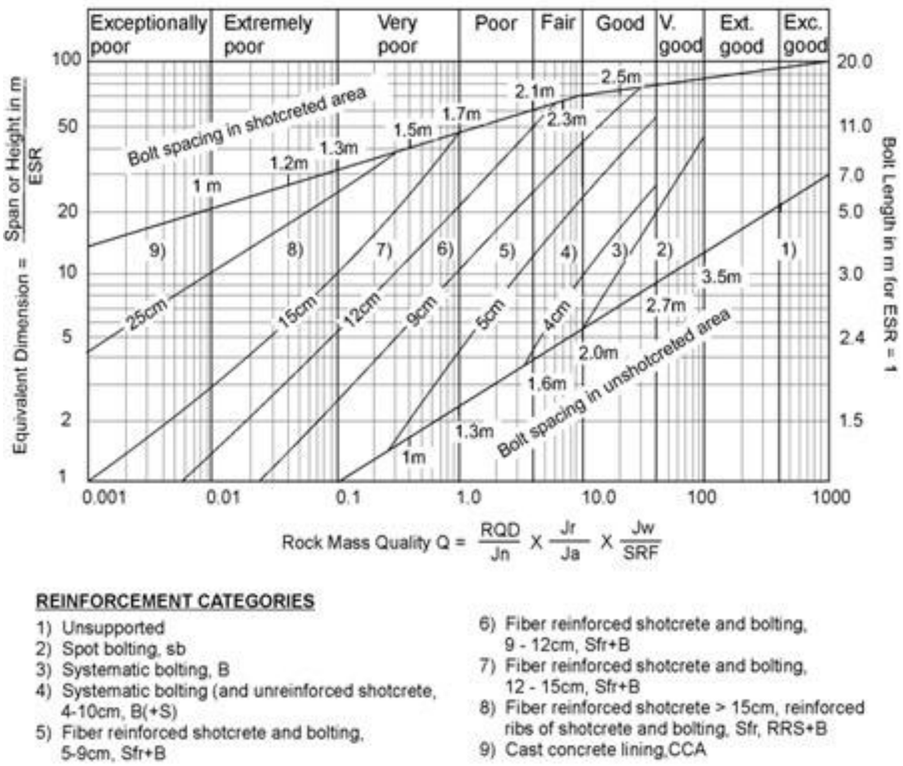
The desk of NMT development

The Norwegian Method of Tunnelling (NMT) has as one might expect from the name, an origin mostly from Norway. Numerous case records, eventually more than 1,250, were also finally mostly from Norway, but many of the early cases were from Sweden. It is this (and numerous single-shell caverns from many other countries too) that stimulated the original development of the Q-system of rock mass classification and tunnel support class definition. Q was developed in response to a State owner's question - 'why so variable deformations in Norwegian powerhouse caverns'?

The Q-system was always based on economic 'single shell' tunnel and cavern reinforcement and support concepts, for mostly hard jointed rock, which however can often be faulted and have numerous clay-bearing joints and major clay-filled discontinuities. Sometimes solutions are needed for swelling clays as well. All of the above explains why the combination B+S(fr) (rockbolting and fibre reinforced shotcrete) is needed, as both the internal friction and the cohesive strength of the rock mass may be inadequate. Maybe this also applies to London Clay with its 'greasy backs'.

There are some 5,000km of single-shell tunnels in Norway, and of these, 3,500km are for hydropower. Many of the latter are nominally 'unlined', where the Q-value is high enough in relation to the span and the tunnel's use as a water conduit, sometimes with high internal pressure. The early (mostly pre-1980s) method of B+S(mr) using systematic bolting and *mesh reinforced* shotcrete was gradually replaced, starting from about 1978 in Norway. Mesh may have been replaced at about the same time in Sweden, as contractors there also performed large-scale panel tests to demonstrate the superiority of the new fibre reinforced S(fr) product. Norway’s first Ph.D. from this era dates from 1981, long before UK studies of S(fr).

Wet process steel fiber reinforced shotcrete marked a revolution in tunneling progress and safety, and in Norway the subsequent development of multiple-layer corrosion-protected rock bolts (galvanized, epoxy painted 'combicoat', inner grout layer, PVC sleeve, outer grout layer) known as CT bolts, while essential, added to the confidence of building 'single shell' tunnels and caverns, including the 62m span and 140,000m3 Gjøvik Olympic Sports Hall Cavern, where Q was only from 2 to 30 (poor/fair/good), and 10cm of S(fr) with the bolting and (temporary) anchoring was sufficient for permanent support. Numerical verification of designs with UDEC-BB (Fig 1) predicted the 7mm to 9mm of vertical deformation that was eventually measured (Barton et al. 1994). Excavation and support (Veidekke/Selmer) took six months after access was established.



Fundamentals of the rock mass quality Q-system

Contrary to NATM, where the primary B+S(mr) seems always to be discounted in the final concrete lining design, sometimes due to the expected squeezing and deformation-induced damage, NMT usually relies on the contribution of each layer of good quality S(fr), except in the case of damaged primary layers when there is stress fracturing due to high cover in hard rock. For instance, three 30m span caverns in the 24.5km long Lærdal Tunnel, constructed at depths of between 1,000m and 1,400m, required extra layers of S(fr) and initially end-anchored bolting to accommodate such stress-fracturing. Grouting of the bolts was performed later. In the case of bolting, corrosion protection is needed if the considerable time and cost benefits of NMT are to be realized in the long term.

Thorough air/water jet cleaning of the rock surface is also a necessity in NMT prior to shotcreting. This procedure (of course avoided if too erodible rock) is ignored in too many countries. Needless to say good quality S(fr) is also needed: both the concrete and the fibres need to be of good quality. In too many countries there is still resistance to accepting 'the high unit cost' of additives and good quality components. Long experience in the same countries suggests that the tunnelling speed and total cost suffers as a result. Microsilica is needed in shotcrete (as also in pre-excavation grouting) as fibres need good anchorage in shotcrete of sufficient quality. This is readily achieved with the resulting lower water/(cement plus filler) ratios.

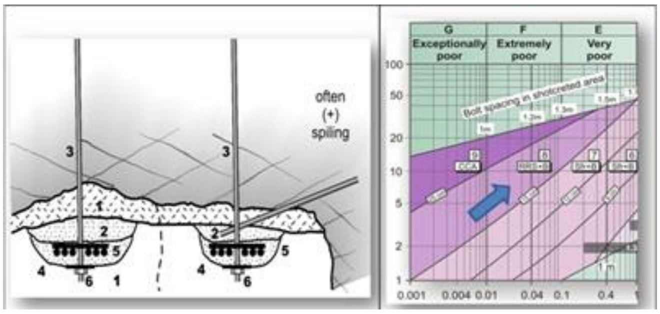
Dramix stainless steel fibres and the best BarChip polypropylene (surface-roughened) fibres are equally acceptable for their fracture energy enhancement of the S(fr). Smooth floor-slab fiber qualities should not be used in tunnels. So, unlike planned and reported NATM cases, which still recommend and apply primary S(mr), in the case of NMT the obvious choice for the last 30-35 years 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 as fibres. (See example photo, and the Vandevall 'Tunnelling the World' drawing, contrasting S(fr) with S(mr). Differences are not exaggerated).

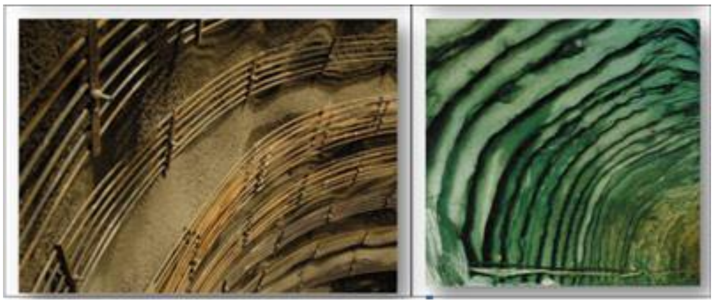
Lattice girders are not and should not be used in NMT, as they attract extra deformation and have caused failures when applied 'as part of the Q-system'. They have never belonged in the Q-system recommendations. Use of S(mr) also attracts deformation, and can even be dangerous, as it is an inefficient, multi-process and therefore delayed support measure. In unstable rock, arches of S(fr) can be built rapidly by robot application (with non-alkali accelerator). The arches are then systematically bolted and internally reinforced with steel bars. These composite RRS (rib reinforced shotcrete) arches are inevitably far superior to lattice girders. Due to their bolted construction (see photos and drawings), there is a reduction in the total number of bolts required. 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 obviously an advantage. The reinforced rock arch, if it can be achieved, is the resisting element. Self-drilling bolts might be needed on occasion. RRS can be bent into (and bolted into) extreme over-break, meaning early active support, instead of trying to 'connect' a rigid lattice girder to a distant tunnel wall or arch, due to sometimes unavoidable overbreak.

Shadows and voids behind S(mr)

In the last ten years or so, high pressure (5 to 10 MPa) pre-injection has become a standard method of water control in NMT, and rock mass conditions in general are greatly improved (five or six of the Q-parameters are assumed to improve), resulting in much reduced over-break, and actually reduced permanent support requirements. This has been experienced in three recent rail tunnels, but care is needed to consider both the tunnel environment as well as ground-water level maintenance. We know that seismic velocity and deformation modulus must each increase in the minimum 5m to 6m thick pre-injected annulus. The pre-injection cycle (long-hole drilling and grouting) takes 20 to 30 hours, and is an excellent investment in overall faster and cheaper tunnelling. When the tunnel breaks through it is 'complete'. No waiting for concrete lining.

In traditional NMT which is still much used in Norway, the tunnel by contrast, remains drained, and a rapidly built (1km per month) free-standing PC-element liner with an outer membrane is often used. The rock support and reinforcement behind this must be permanent (including CT bolts) and diligently selected through correct rock mass characterization and Q-system application. There have been two local failures in road tunnels in Norway in the last two decades, one of them due to failing to detect clay, and managers also electing not to use the Q-system. Not following a well-tested 'check-list' is a potential source of support-selection error, and therefore a source of risk to future tunnel users. Do we really believe that numerical modeling is a substitute here?

- 



RRS is a flexible (until bolted) 'lattice girder

3D effect of S(fr) arches

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. In the Bærum Tunnel (see photos), with all 5 km systematically pre-injected, using typical high pressures of 5 to 10 MPa, there were vanishingly few damp patches due to the excellent quality of the work. Sprayed membrane has been used in recent UK and Swiss tunnels, apparently even without pre-injection.

Finally, for those who are wedded to numerical modelling as a basis for tunnel design (NATM, SCL or NMT) as opposed to simpler empirically-based design: they were given good advice in the original article by David Hindle that stimulated the TunnelTalk feedback. Turn off the computer sometimes. Many seek longer rock bolts and predict larger deformations than the subsequently measured reality. This is due to their reliance on numerically-produced and very colourful 'plastic zones'. A widely used continuum-based method was in fact proved to give false predictions of rock mass behaviour and support needs, exactly because of grossly exaggerated ‘plastic zones’.

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Examples of NMT tunnelling in action

NMT excavation heading and equipment

We should remember that a posteriori design (based on the experience of successful solutions) is fundamentally sounder than a priori design (based on assumptions and questionable non-empirical algebra). In the case of rock masses we now finally know that it is not correct to add c and $\sigma n' \tan \phi$, as in linear Mohr Coulomb or non-linear Hoek Brown shear strength criteria, as each strength component is mobilized at widely different strains. See for instance Barton and Pandey, 2011, where such problems are discussed and a possible solution is found.

Numerical modelling is inevitably an inadequate method of design, when tunnelling is likely to be progressing at 50m to 100m per week with full-face drill-and-blast excavation. There are NMT records of 160m and even 170m per week per face, and more than 100m per week per face for a whole tunnelling project. Empirical methods for rock mass characterization and support selection have to be used at that speed, and at much slower speeds too. Certainly the assumed performance of different rock mass classes can be modelled before-hand, and updated and improved during construction. Preferably this should be with due recognition of the fact that rock masses are usually anisotropic and jointed media. Continuum modelling usually over-simplifies the reality, therefore potentially giving incorrect conclusions and recommendations, seen too often in many countries.

In hard rock tunnels that only require NMT methods, one may see another contortion of reality. This is the uniform-thickness numerical modeling of concrete linings for the alternative and more expensive NATM designs. Highly idealized modeling is too often used as a basis for selecting concrete thickness. Clearly theoretical concrete linings can be 'engineered' (and optimistically costed) apparently showing limited requirements for steel reinforcement. But try the reality of 25cm to 75cm of variable thickness, sometimes 25cm to 125cm due to unavoidable overbreak, plus anisotropic loading, and a different truth (and lack of 'economy') may emerge. When unstable rock blocks and wedges with low shear strength are present (i.e. rock mass conditions $Jr/Ja \leq 1$ and $Jn \geq 6$), overbreak may be inevitable. In RMR (and in the numerical modeller's GSI) the number of joint sets is ignored, so there is no help in understanding the potential for overbreak, and the consequent advantages of B+S(fr) over concrete.

To conclude: with NMT the tunnel is not filled with concrete, so the above contortion of reality when designing linings does not arise. 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, because it proved to be so long ago, and can be designed to be so in the future, with so many thousands of kilometres built and remaining to be built in the world's hydropower and other tunnelling developments in jointed rock - which is an all too common medium of tunnel construction.

Actually long ago (in Barton and Grimstad, 1994), it was suggested that we should combine the best of NATM and NMT. Hybrid NATM/NMT/NATM is the obvious solution for tunnels with bad conditions (soil and saprolite) in (both) portal and deeply weathered areas, and good conditions in (most of) the central hilly or mountainous kilometres, which is quite a common situation. Indeed it is often an inevitable consequence of deep weathering, with its usual absence at greater depth. Costs can be at least halved by such hybrid measures, and less CO_2 from saved concrete production may be an added advantage for all of us. However, even the Q-system resorts to concrete lining on occasion, but only where absolutely needed, never for the whole length of tunnel.

Fault zones are never four or fourteen kilometers wide. Correctly designed and applied S(fr) is superior to many concrete linings: shrinkage cracks are avoided, damage by frost is avoided. Most important: tens of billions of dollars and hundreds of years in construction time can be saved in any given tunnel construction year, considering just a few dozen countries.

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