Norwegian Method of Tunnelling

Prezentacja została przygotowana w roku akademickim 2003/4 przy udziale niżej wymienionych studentów kierunku Budownictwo:

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Norwegian Method of Tunnelling (NMT)

➢ The development of road tunnel construction has been enormous in Norway over the past 40 years. While there were few tunnels in 1960, the total length had reached 752 km (620 tunnels) by 2002.

➢ In 1990, 32 km of hard rock TBM tunnels were driven, mostly at Statkraft's Svartisen hydroelectric project.

➢ It has been estimated that 4,500 km of tunnels have been constructed in Norway since 1970 to 1992

➢ At present, some 50 to 60,000m^3 of fibre reinforced shotcrete are sprayed each year in Norway

➢ hard rock TBM tunnels have been constructed in record time in hard gneisses, diorite, quartzite, marbles and schists with compressive strengths of 120 to 300 MPa

➢ Best results of 61.2 m in a shift, 90.2 m in a day, 415 m in a week and 1176 m in a month were achieved in the 36 km driven during a 2 year period from 1989 to 1992.

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NMT and NAMT – what are the differences?

➢ Despite the comment by an experienced NATM pioneer that "it is not usually necessary to provide support in hard rocks", Norwegian tunnels require more than 50,000m³ of fibre reinforced shotcrete and more than 100,000 rock bolts each year.

➢ Two major tunnelling nations, Norway and Austria, have in fact long traditions in using shotcrete and rock bolts for tunnel support, yet there are significant differences in philosophy and areas of application for NATM and NMT, To start this brief review, it may be pertinent to first state what appear to be the major differences between NATM and NMT.

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

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NMT and NAMT – what are the differences?

Essential features of NMT	
Areas of usual application	Jointed rock; harder end of scale (XXX, = 3 to 300 MPa) Clay bearing zones, stress slabbing (Q-0.001 to 10)
Usual methods of excavation	Drill and blast, hard rock TBM, hand excavation in clay zones.
Temporary support and permanent support may be any of following	CCA,S(fr)+RRS+B,B + S(fr); B + S,B,S(fr),S,sb,(NONE); temporary support forms part of permanent support; mesh reinforcement not used; dry process shotcrete not used steel sets or lattice girders not used; RRS used in clay zones; Contractor chooses temporary support; Owner or Consultant chooses permanent support; final concrete linings are less frequently used, i.e., B + S(fr) is usually the final support
Rock mass characterization for	predicting rock mass quality and support needs; updating of both during tunnelling (monitoring in critical cases only)
The NMT gives low costs and	rapid advance rates in drill and blast tunnels; improved safety; improved environment

NMT and NAMT – what are the differences?

➢ NATM appears most suitable for soft ground which can be machine or hand excavated, where jointing and overbreak are not dominant, where a smooth profile can often be formed and where a complete load bearing ring can (and often should) be established.

➢ Monitoring appears to play a significant part in deciding on the timing and extent of secondary support. (Using the surrounding ground as the main loading component is not an exclusive NATM philosophy. It is essential practice and is often inevitable!)

➢ NMT appears most suitable for harder ground, where jointing and overbreak are dominant, and where drill and blasting or hard rock TBM's are the most usual methods of excavation.

➢ Bolting is the dominant form of rock support since it mobilises the strength of the surrounding rock mass in the best possible way.

NMT and NAMT – what are the differences?

➢ Rigid steel sets or lattice girders are inappropriate in Norway's harder rocks due to the potential overbreak.

➢ Potentially unstable rock masses with clay-filled joints and discontinuities will increasingly need shotcrete and fibre reinforced shoterete [S(fr)] to supplement the systematic bolting (B).

➢ It can be stated with some certainty that B+S(fr) are the two most versatile tunnel support methods yet devised, because they can be applied to any profile as temporary or as permanent support, just by changing thickness and bolt spacing.

➢ A thick load bearing ring (reinforced rib of shotcrete = RRS) can be formed as needed, and matches an uneven profile better than lattice girders or steel sets.

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➢ A key requirement for ensuring consistent mapping quality,good tender documents and good records of actual conditions is a method that describes the rock mass in quantitative rather than just qualitative terms.

➢ Although the high level of experience in the Norwegian tunnelling community has allowed "rules-of thumb" and much "previous experience" to dictate a lot of the support estimates, more and more companies are realising the value of a documentation method such as the Q-system for regulating the description of rock mass conditions and support recommendations

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

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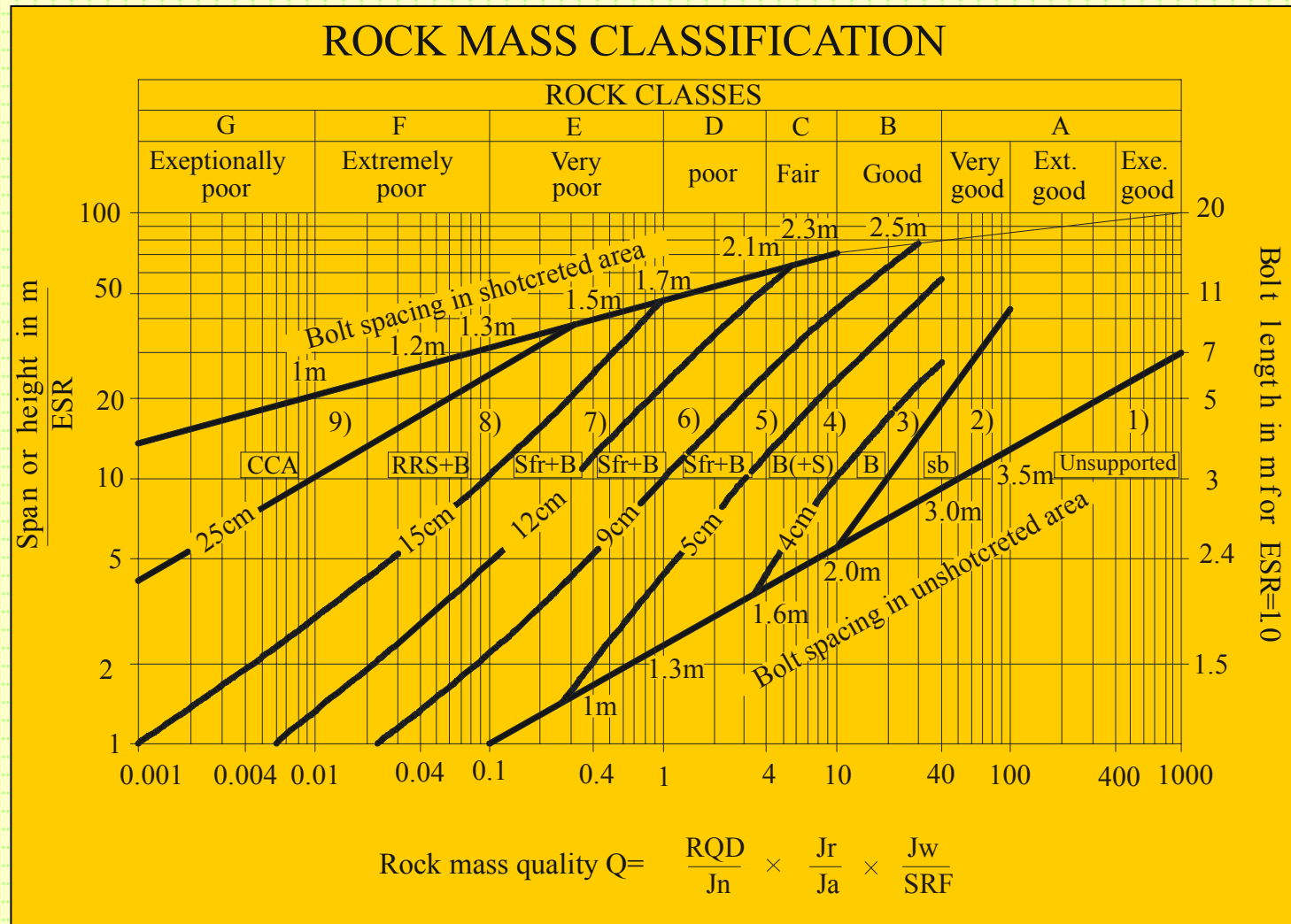
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➢ It has been said elsewhere, perhaps unfairly, that "when experienced, use the Q-system; when uncertain, use NATM" (uncertainty here due to the weak ground commonly associated with NATM).

➢ The important point is that forward prediction of conditions and agreed modifications for unexpected conditions should each be done as early and as accurately as possible, so that on" the one hand tender documents are a fair reflection of revealed conditions, and unexpected coaditions are agreed upon and tackled without delay by all parties concerned. This minimises disputes and also minimises tunnel instability!

➢ Legal action is in fact virtually non-existent in Norwegian tunnelling.Although the Q-system of rock mass classification has been used for many years, improvements have taken shape rather slowly.

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ROCK MASS CLASSIFICATION

Rock mass quality $Q = \dfrac{RQD}{Jn} \times \dfrac{Jr}{Ja} \times \dfrac{Jw}{SRF}$

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> With good warning well ahead of the face, a tunnel contractor can plan his strategy, mobilise equipment and minimize risk. In other cases he may avoid costly over-reaction and unnecessary delays. Cross-hole seismic tomography and tunnel radar are invaluable aids in this respect

> A perfect example of this was the cross-hole seismic investigations performed for the 62 m span Olympic Ice Hockey Cavern at Gjovik, one example of which is shown here.

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

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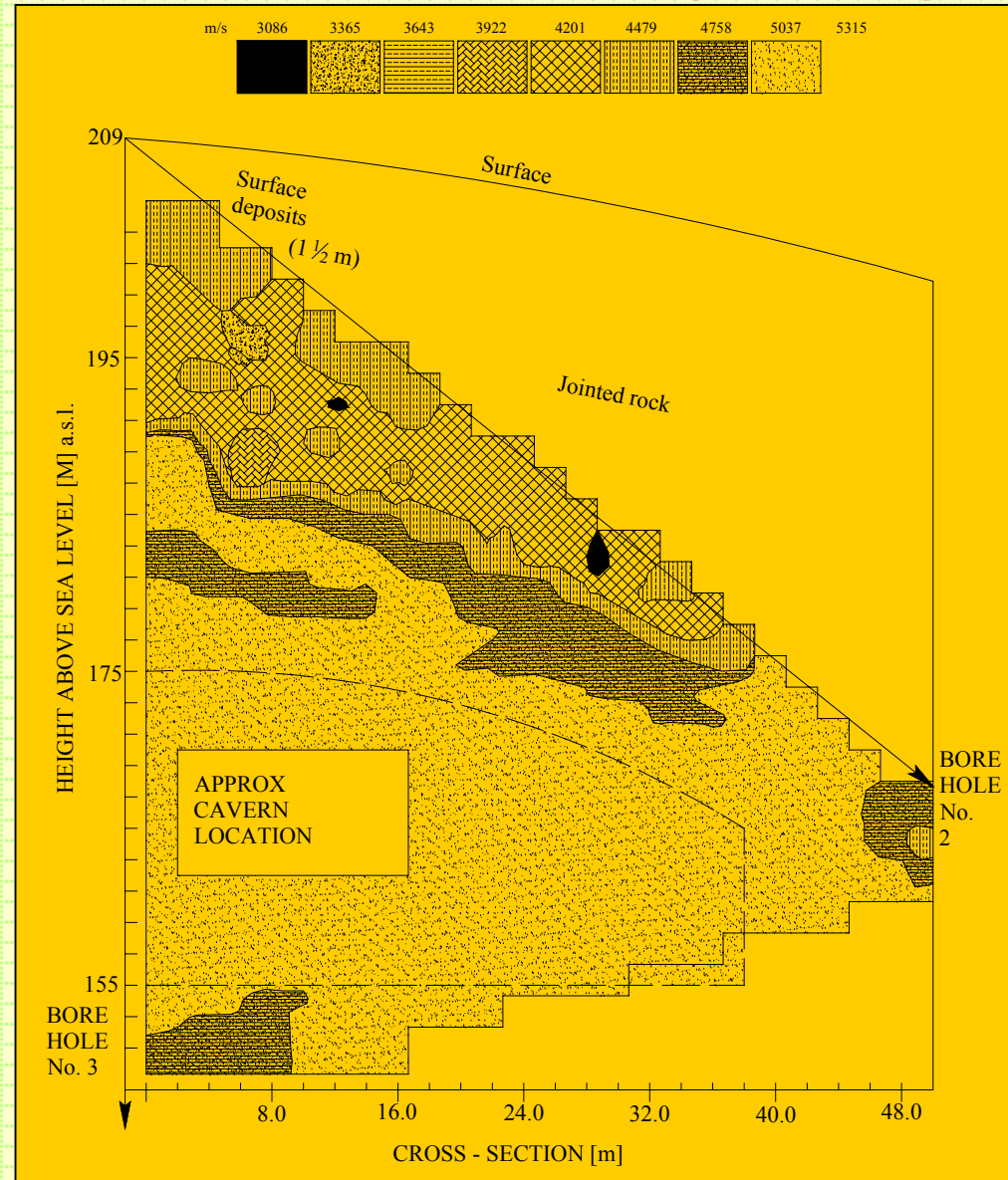
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➤ When a tunnelling project is under way it is very convenient to map conditions using the tunnel logging chart. This gives Q-parameter observations on the left hand side, while principal geologic structure, temporary support and final Q-based support recommendations are given as symbolic logs

➤ The tunnel shown is a 17m high by 10m span
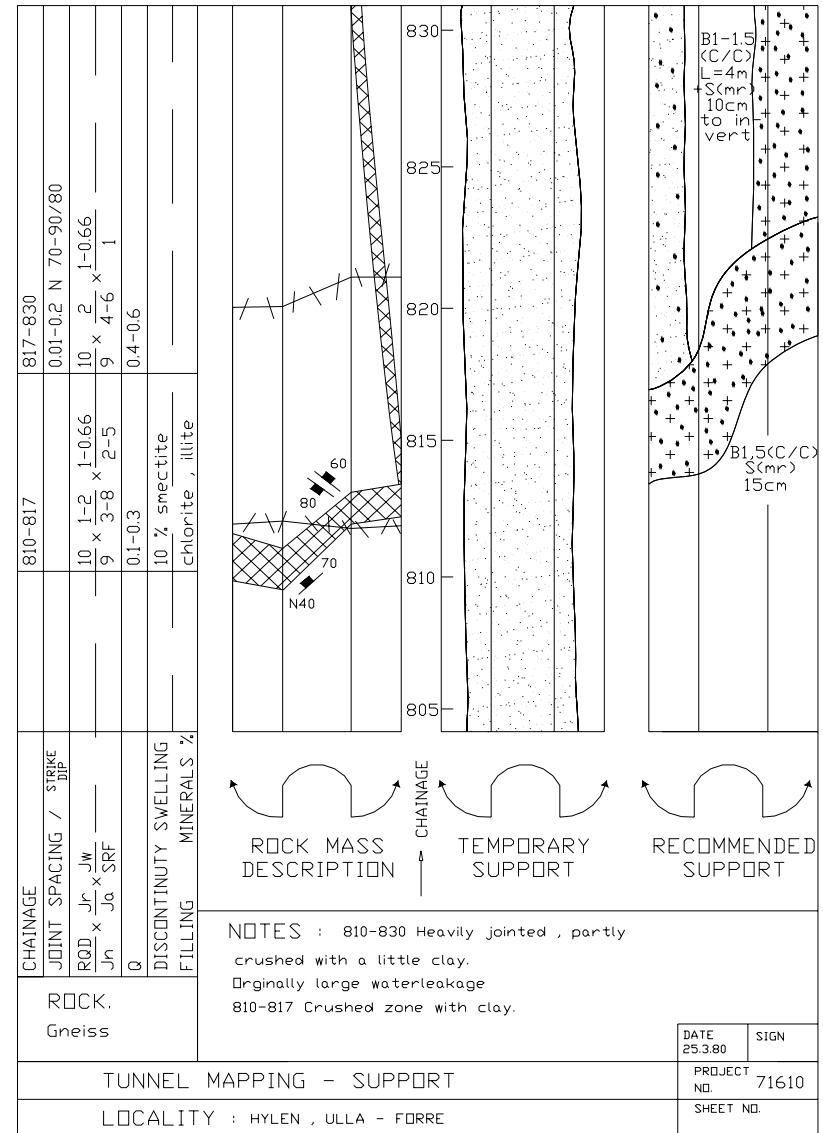
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➢ In spite of predominantly hardrock tunneling in Norway, many fault zones, intense tectonic jointing, hydrothermal alteration zones and rock burst areas require rock support. The support used in our tunnels and large rock caverns varies to a large 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.
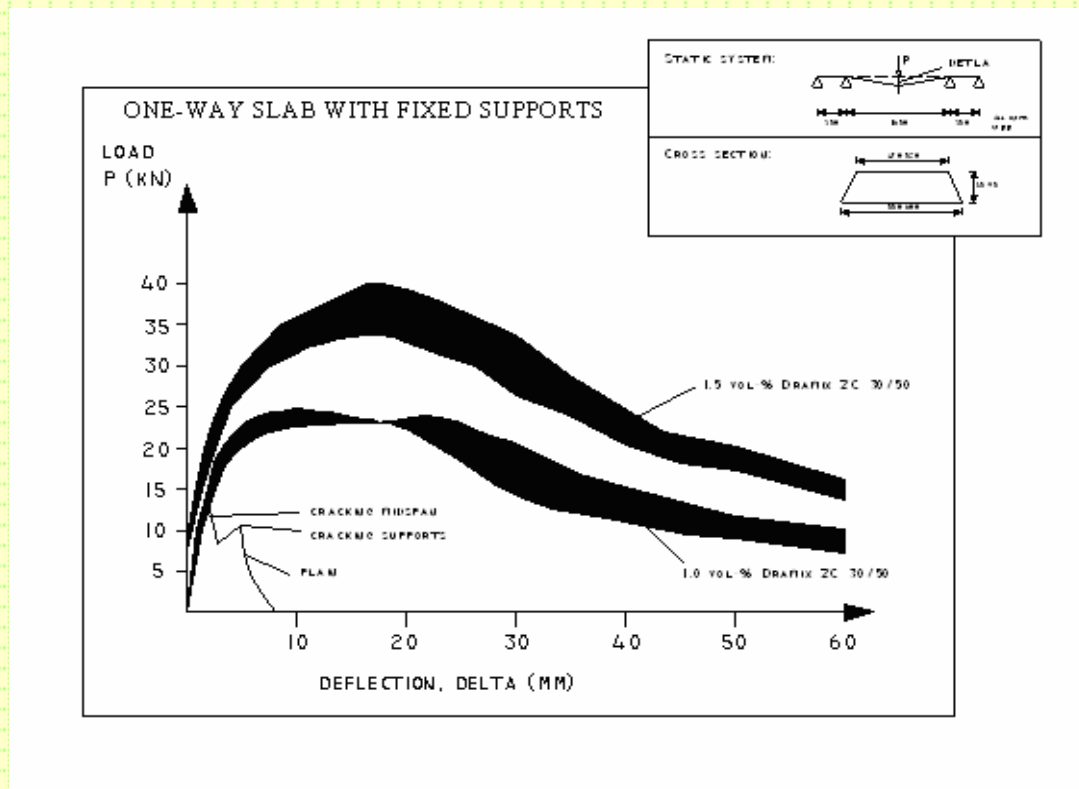
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➢ Incorporation of steel fibre reinforcement in shotcrete was commercially introduced in Norway in 1978. It's introduction led to a rapid change to the remote controlled application of sprayed reinforced concrete for rock support.

➢ 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.

➢ 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 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).

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➢ 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.

➢ 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.

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Deformed rebars

Rock bolts

1. Layer of shotcrete

2. Layer of shotcrete

150 mm

0,5 - 1,0 m   2,0 - 5,0 m
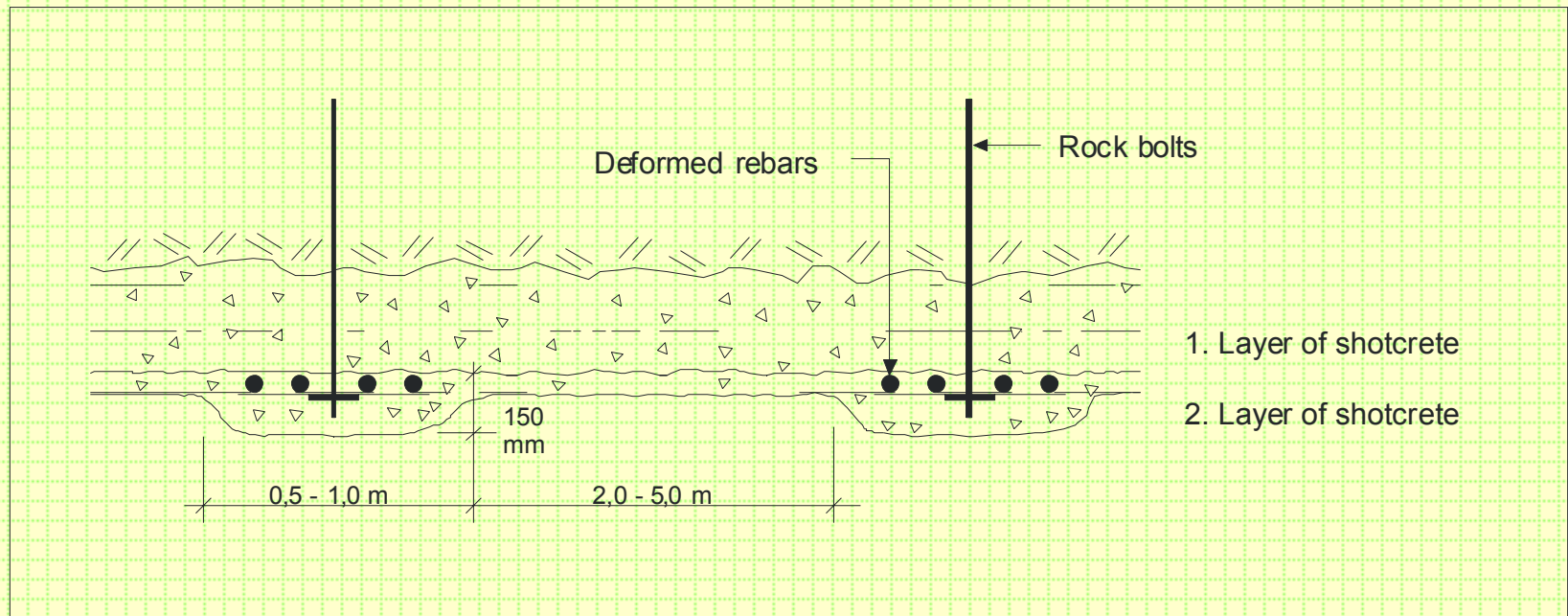
➢ The additional and permanent support may consist of a variety of additional bolts, S(fr) and ribs of reinforced shotcrete (RRS).

➢ 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 1.0 m in length and the rib spacing ranged between 2 and 5 m.

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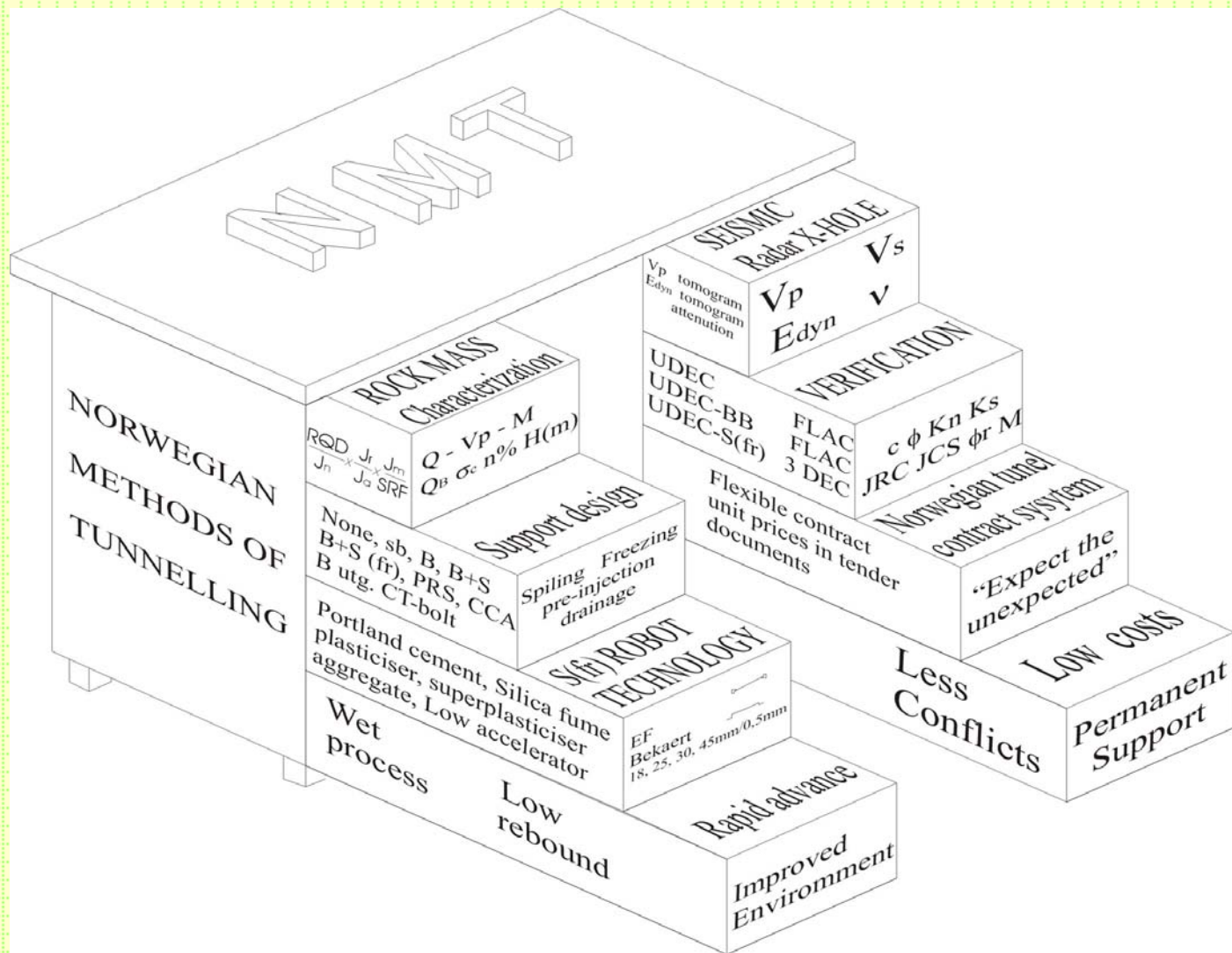
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➢ Although the Q-System of rock mass classification was developed in Norway and is supposedly much influenced by typical hard rock experiences, it is nevertheless true to say that hundreds of the case records on which it is based are for section of poor, very poor and extremely poor rock. Here, the intensity of jointing, weathering or alteration, clay fillings and problems with stability and overbreak are common to many countries.

➢ The Q-system is in fact used more frequently in softer rocks than in hard rocks on a global basis, so it is appropriate to address the problems of concern to the many users of the method.

➢ In the context of road and rail tunnels NMT (Barton et al., 1992) is a collection of practices that produce dry, drained, permanently supported and "lined" (fully cladded) tunnels for approximately US$ 5.000 to US$ 10.000 per metre. These low-cost, high-tech Norwegian tunnels may range in cross-section from about 45 m² to 110 m².
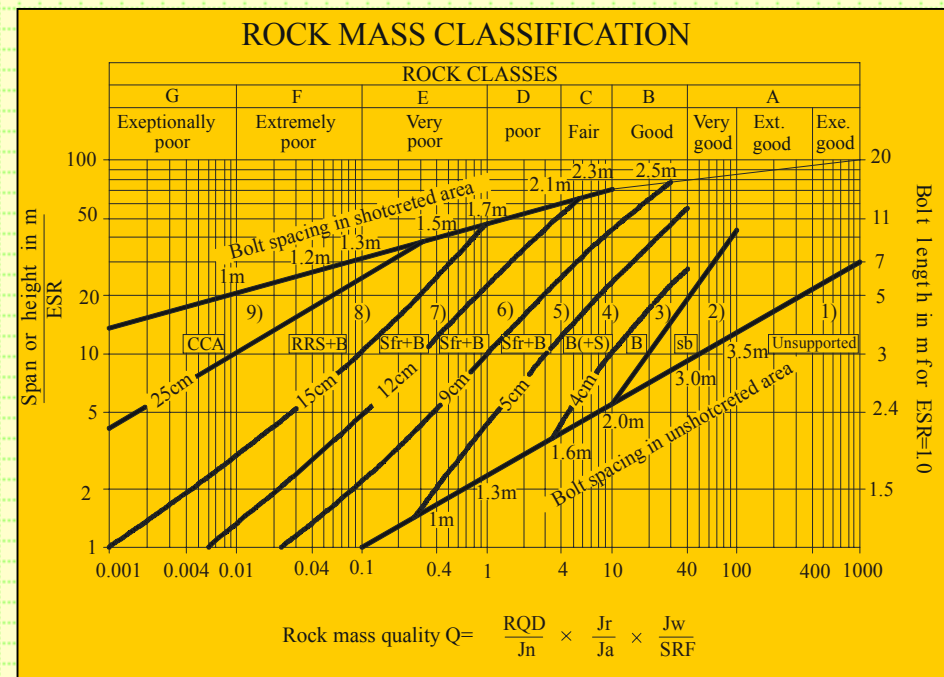
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> ➤ Figure shows some of the essential components of NMT in the form of an "NMT design desk".

Norwegian Method of Tunnelling (NMT)

➢ Preliminary design is based on field mapping, drill core logging and seismic interpretation using newly developed Vp-Q relationships.

➢ Rock mass quality is described by the Q-value (Barton et al. 1974; Grimstad and Barton, 1993; Barton and Grimstad, 1994).

➢ Final support is selected during tunnel construction based on tunnel logging and use of the Q-system support recommendations.

➢ Numerical verification of one or more of the various permanent support classes is performed in special cases, using the distinct element (jointed) two-dimensional UDEC-BB or three-dimensional 3DEC computer codes.

ROCK MASS CLASSIFICATION

Rock mass quality $Q= \dfrac{RQD}{Jn} \times \dfrac{Jr}{Ja} \times \dfrac{Jw}{SRF}$

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Norwegian Method of Tunnelling (NMT)

CONTRACTUAL

➢ The Owner pays in principle for technically correct support.

➢ The Contractor is compensated *via* the unit prices quoted in the tender document.

➢ The Owner bears more risk than the Contractor thereby reducing prices.

➢ Needed support is based on the agreed Q-value, and may vary frequently

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➢ Excavation, usually by drill and blast, is tailored to the rock conditions. NMT is also applied in tunnels excavated by road header or hydraulic breaker outside Norway.

➢ The temporary support such as B or B+S(fr) is approved as part of the permanent support. In poor conditions, pre-grouting, spiling and use of rib reinforced shotcrete arches up to the face may be used. Cast concrete may also be needed as temporary support in some cases, cast against an articulated shield.

➢ The permanent support class is chosen during tunnel advance, and will depend on the rock conditions which are systematically logged. Deformation measurements will usually be used in very and extremely poor rock as confirmation of the support class. However, it may be dangerous to assume that reducing rates of deformation signal stable conditions.

➢ In general, in poor to fair conditions an NMT designed tunnel is drained, with insulated, pre-cast concrete panels for water (and frost) control when needed. These can be assembled at approximately 1 km per month.

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➢ The permanent rock support usually consists of high quality wet process, S(fr) applied by high capacity robot, and fully grouted, corrosion protected rock bolts. These may be supplemented by RRS in very poor conditions.

➢ Concrete lined section will be used through fault zones, swelling clay and very weak rock that may squeeze. When the overburden and rock conditions combine to give high SRF estimates, the final concrete lining will obviously need careful design.

➢ The use of nominal thickness, final cast concrete linings for appearance or due to tradition is discouraged due to cost, scheduling and lack of loading when Q-system designed B+S(fr) for assumed loading levels is already in place.

➢ "Design as you drive" or "in situ selection of support", presupposes anticipation and designs for the full range of rock conditions, and unit prices for all the tunnelling and support costs in the range of US$ 5000 to US$ 10000 per metre are normal in Norway for two-to-three lane highway tunnels using these NMT principles. Consistently poor conditions with tunnelling progress delayed by necessary heavy support will obviously cause these prices to be exceeded.

Norwegian Method of Tunnelling (NMT)

➢ This figure shows tunnel deformation data plotted as a function of Q-value and span. The approximate cross-hatched zone actually envelopes some 1000 measurements from unpublished weak rock cases, which mostly lie between the envelopes AA and CC.
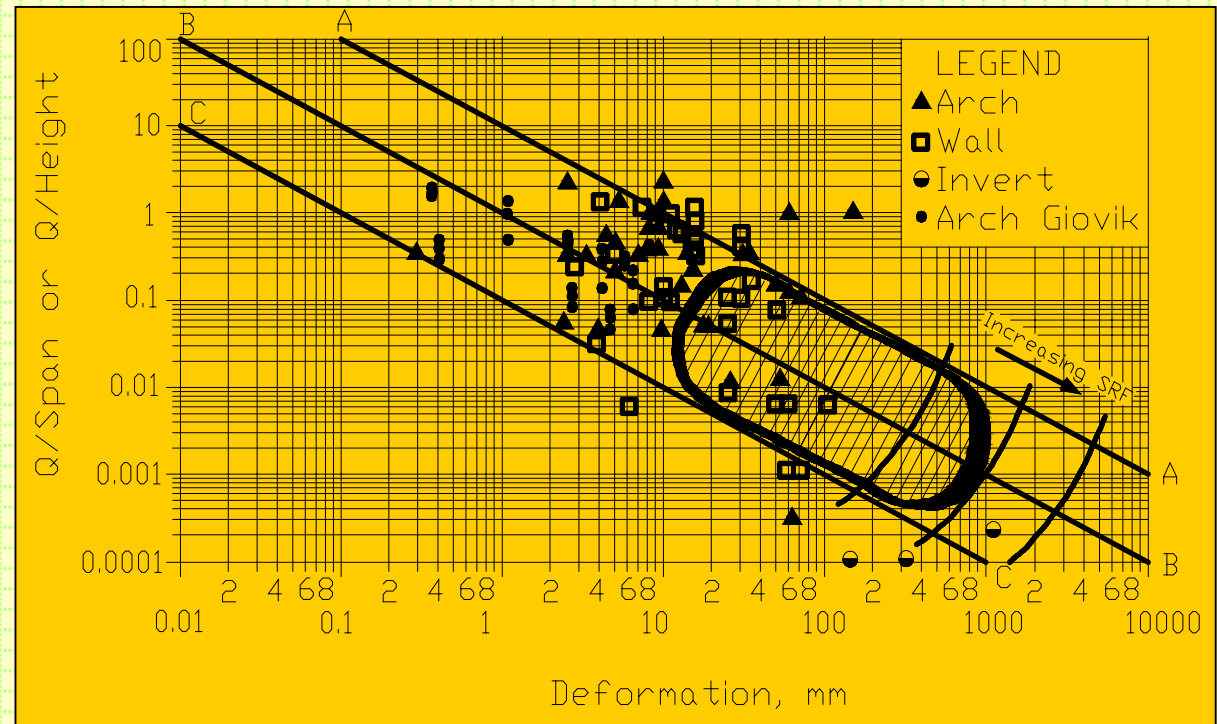
Deformation from Q-values and span width

The central trend line (BB) for considerably more than 1000 measurements of wall and arch deformations, ranging all the way from 0.3 to 1000 mm, is given by the simple equation:

$$\Delta(mm) \cong \frac{SPAN(m)}{Q}$$

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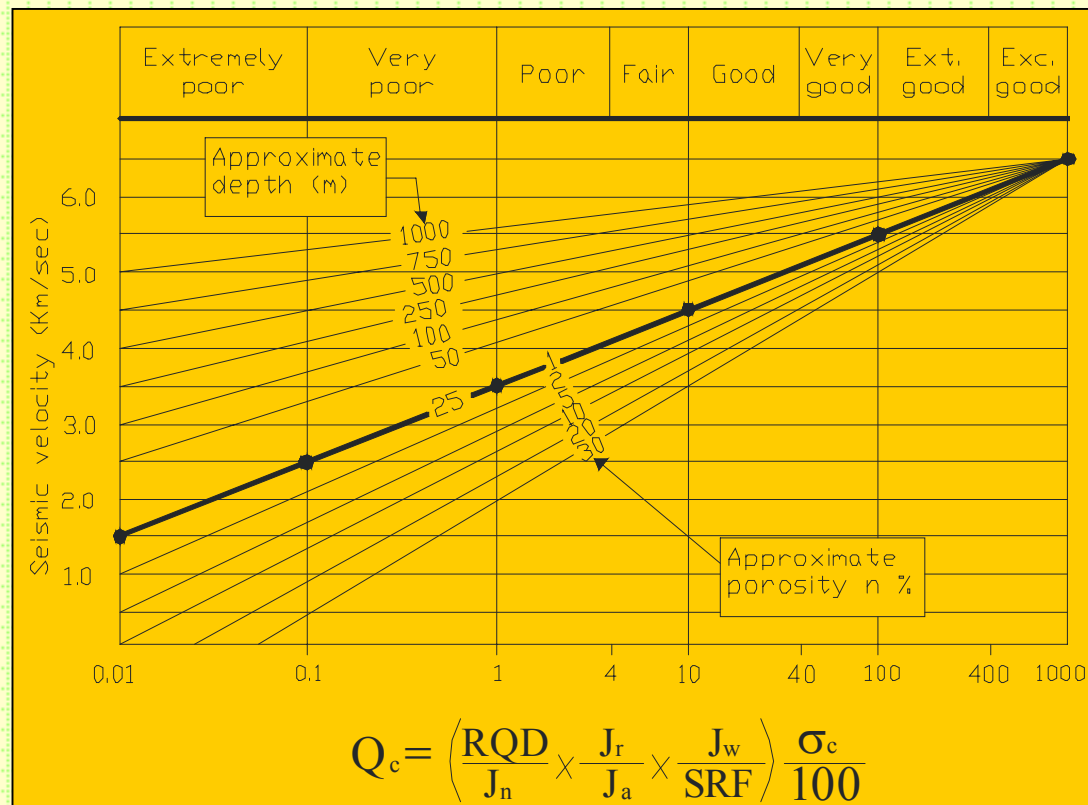
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➢ The scatter is nevertheless quite large and improved fit may be obtained by normalizing with depth and uniaxial strength. The following approximation appears reasonable for explaining the wide scatter of data, which is undoubtedly related to these additional direct variables of depth (i.e. stress) and uniaxial strength (i.e. modulus):

$$\Delta(mm) \cong \frac{SPAN(m)}{Q}\left(\frac{H}{\sigma_c}\right)^{\frac{1}{2}}$$

➢ For example with Q=0.01 (extremely poor), H=100 m, σ_c=10 MPa and SPAN=10 m we obtain the estimate: ~(10/0.01)(100/10)$^{1/2}$ ~316 mm, which appears very reasonable according to the measured data. Under more favourable rock conditions (Q=2) but with SPAN increased to 20 m and depth increased to 200 m and σ_c=50 MPa, we obtain the estimate: Δ = (20/2)(200/50)$^{1/2}$ ~ 20 mm which corresponds to relevant cavern experience quite well.

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➢ Seismic refraction measurements are an invaluable aid in site investigation and can be used with some confidence in locating low velocity zones near the surface, provided that these are not masked by overlying layers of higher velocity. Hard rock sites (nominal $\sigma_c \sim 100$ MPa) with the usual limited penetration of seismic (nominal H = 25 m) have shown the following correlation between Q-values and Vp:

$$Q_c = \left(\frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \right) \frac{\sigma_c}{100}$$

$$V_p \cong 3.5 + \log_{10} Q_c$$

This relationship is shown by the bold central line

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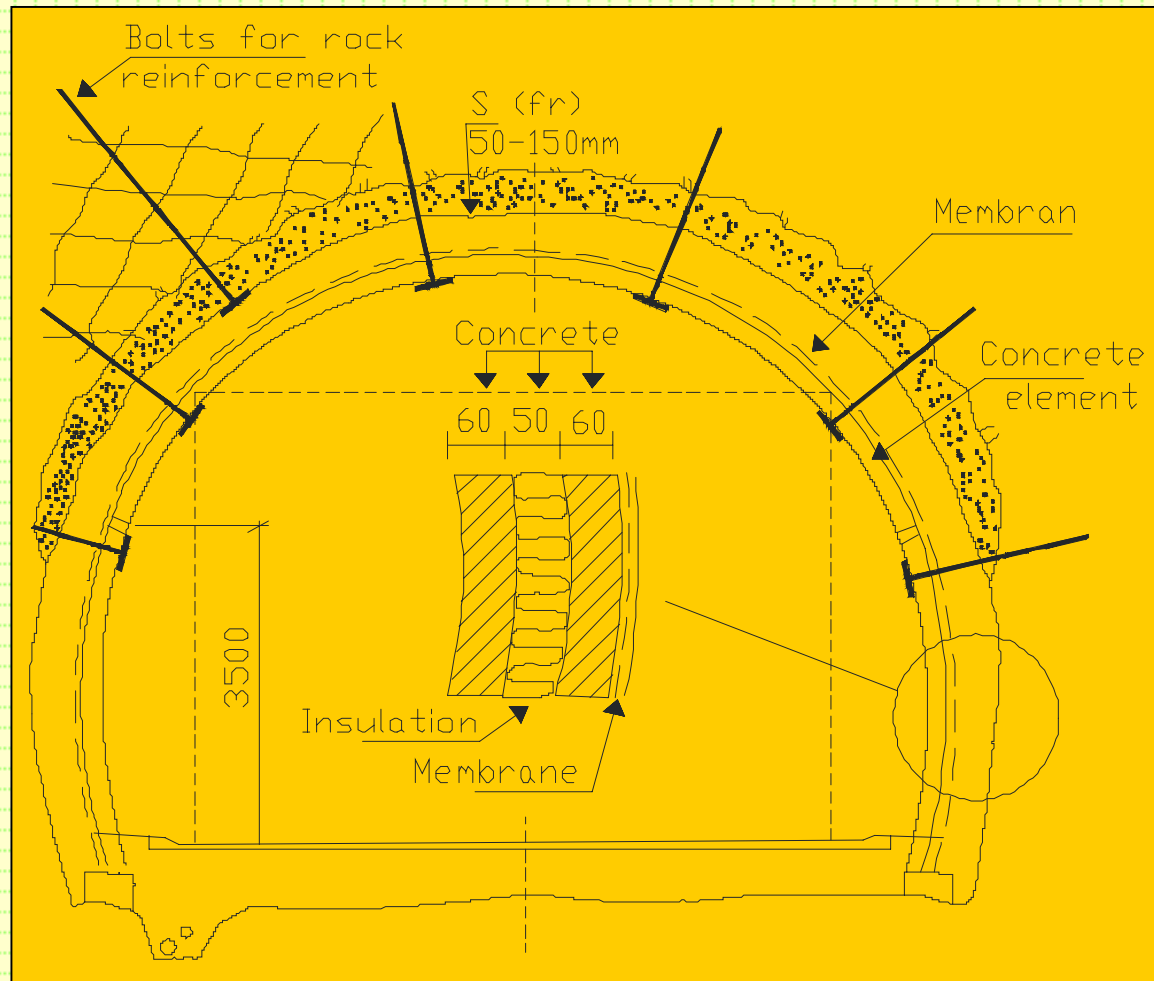
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➢ It will be noted that the Q-value which was originally developed for estimating rock support needs, has to be normalized by uniaxial strengths that are different from 100 MPa, which is the typical hard rock value. The normalized value Q_c is given by:

$$Q_c = Q \cdot \frac{\sigma_c}{100}$$

➢ Further adjustments are made for porosities that are larger then the nominal 1% for hard rock, and adjustments are also made for depths greater than the nominal 25 m depth of penetration for conventional surface seismic. It is also possible to estimate deformation modulus (M, in units of GPa) using the Q_c concept:

$$M \cong 10 Q_c^{\frac{1}{3}}$$
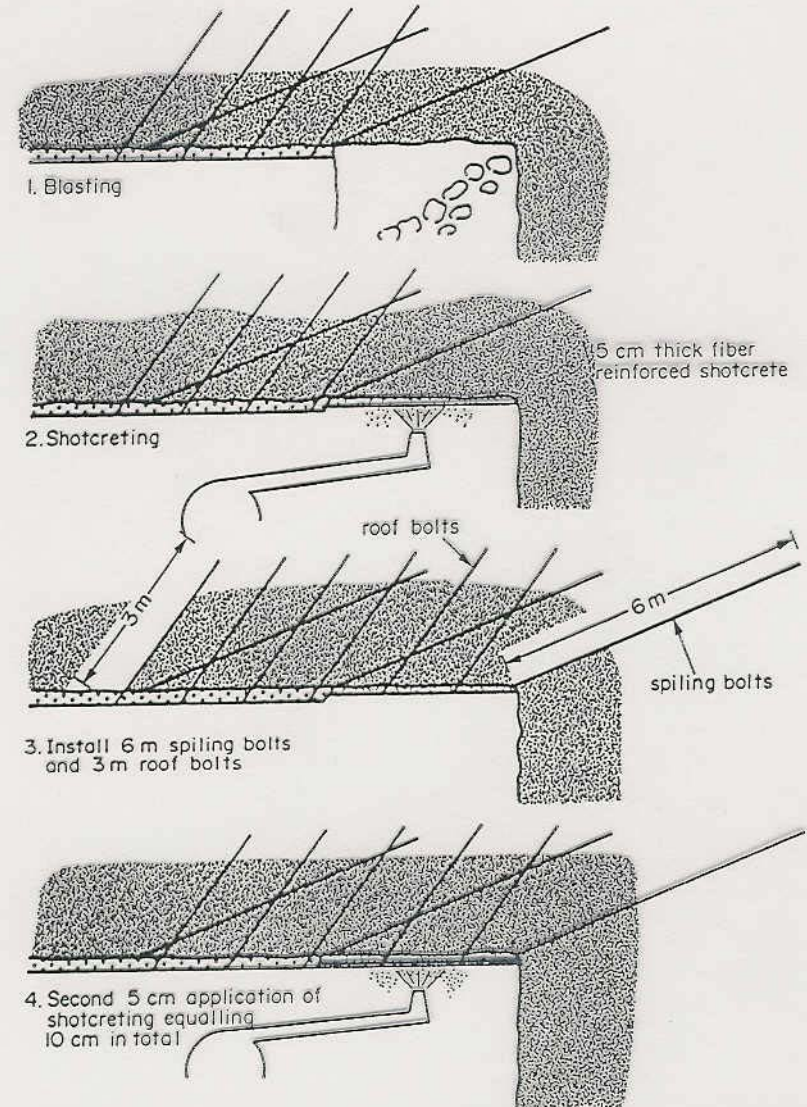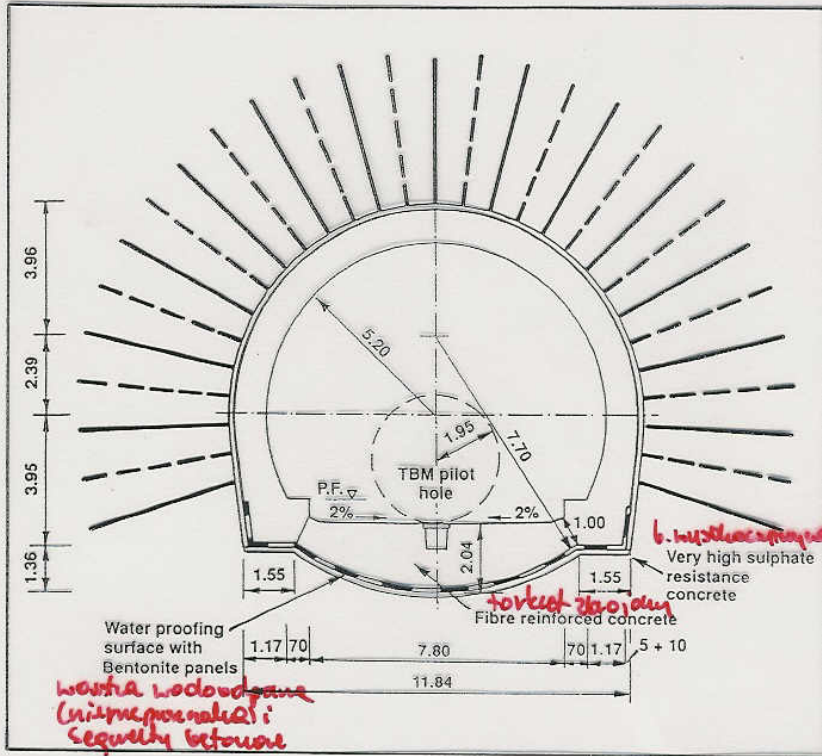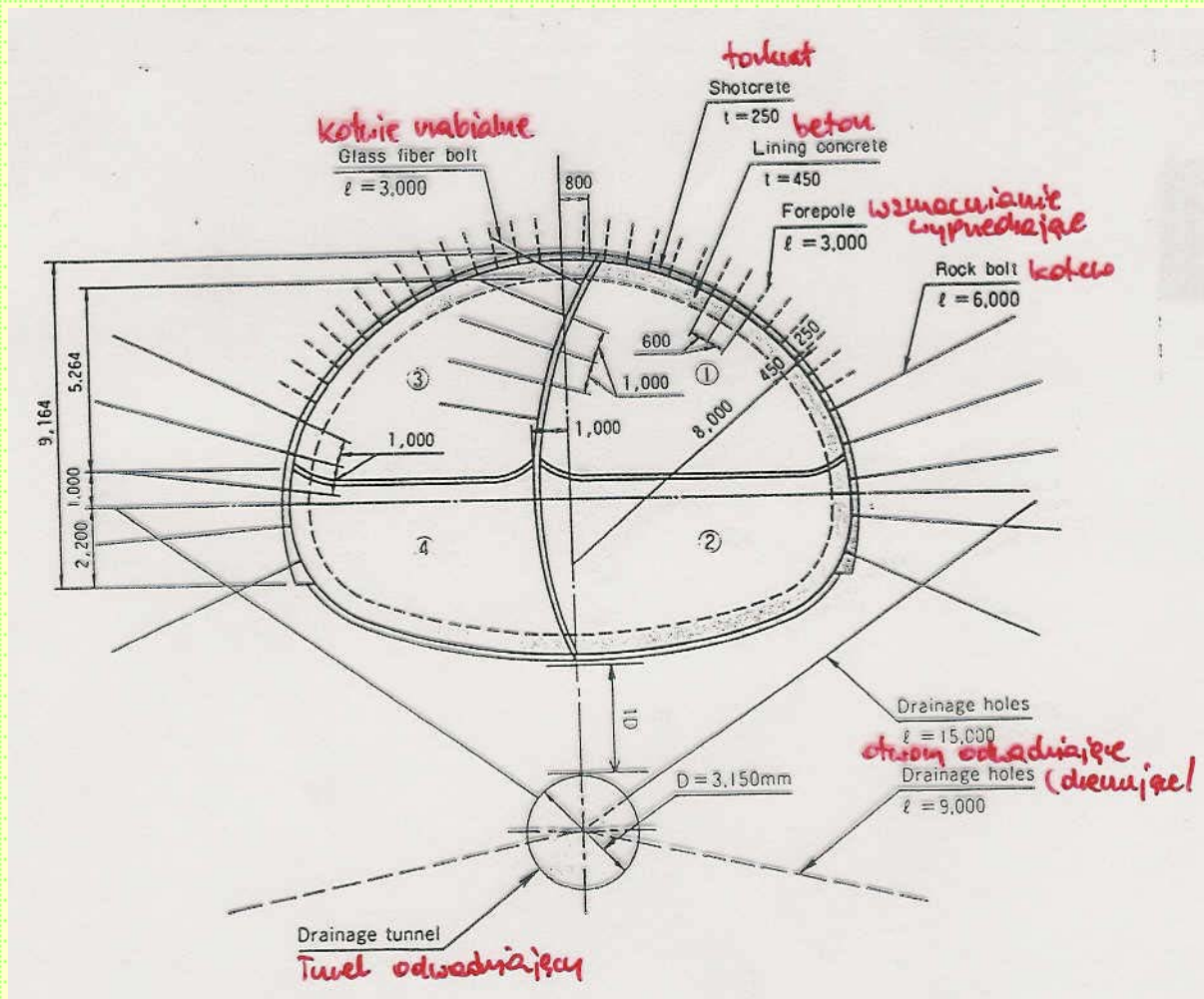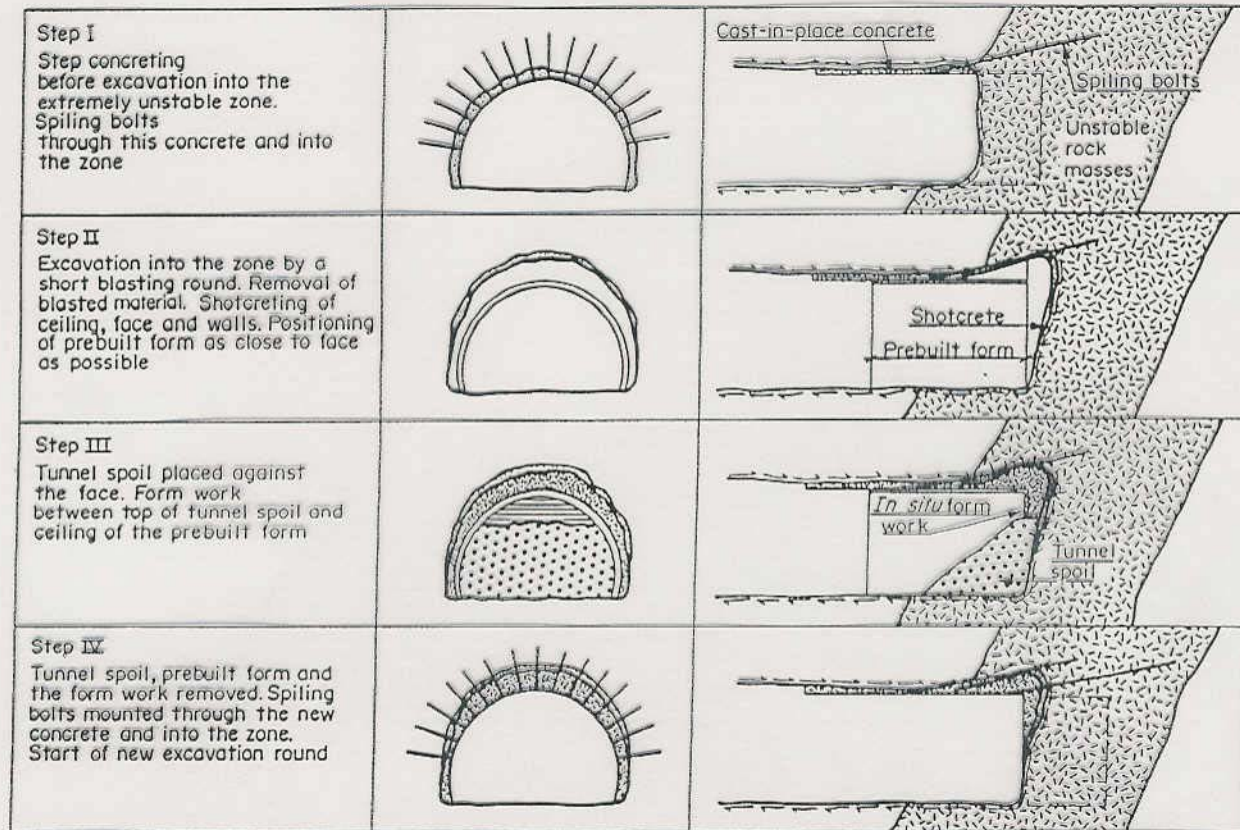
Norwegian Method of Tunnelling (NMT)

Typical support for NMT tunnel

Norwegian Method of Tunnelling (NMT)

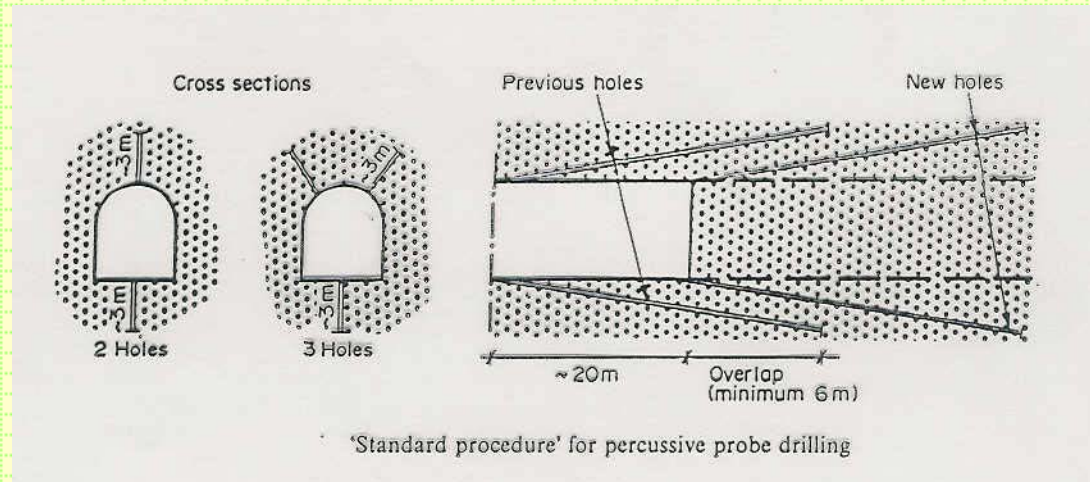# Norwegian Method of Tunnelling (NMT)

Norwegian Method of Tunnelling (NMT)

Step I
Step concreting before excavation into the extremely unstable zone. Spiling bolts through this concrete and into the zone

Step II
Excavation into the zone by a short blasting round. Removal of blasted material. Shotcreting of ceiling, face and walls. Positioning of prebuilt form as close to face as possible

Step III
Tunnel spoil placed against the face. Form work between top of tunnel spoil and ceiling of the prebuilt form

Step IV
Tunnel spoil, prebuilt form and the form work removed. Spiling bolts mounted through the new concrete and into the zone. Start of new excavation round

Cast-in-place concrete
Spiling bolts
Unstable rock masses
Shotcrete
Prebuilt form
In situ form work
Tunnel spoil

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'Standard procedure' for percussive probe drilling

References:

1. Barton N., Grimstad E., Aas G., Opsahl O.A., Bakken A., Pedersen L., Johansen E.D.1992. Norwegian Method of Tunnelling. *World Tunnelling*. June 1992 and continuation in August 1992.

2. Barton N. 1998. NMT support concepts for tunnels in weak rocks. *Tunnels and Metropolises*. Negro & Ferreira (eds). Balkema. Rotterdam.

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