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Rock Mass Characterisation and Seismic Measurements to Assist in the Design and Execution of TBM Projects

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Abstract

The choice of TBM excavation *contra* drill-and-blast excavation of major tunnels is a decision that can have major economic and schedule implications. Unfortunately, both economy and schedule on occasions go in the opposite direction to those expected, and greater costs and more time are incurred than would be likely with the drill and blast alternative. This paper is an attempt to reduce the risk in TBM tunnel driving, so that costly TBM machines are less likely to be stopped for long periods. The proposed method is based on systematic collection and utilisation of data from the rock ahead of the TBM. The principles involved are based on correlations between seismic P-wave velocity and rock quality. Rock quality is described by a Q-value that has been normalised by the uniaxial rock compression strength. An estimate of the rock matrix porosity and the approximate stress level or depth is also used in the analysis of tunnel support needs. Convergence monitoring is used to verify that the prediction of support class and execution of support is in accordance with expected behaviour.

Introduction

Large TBM machines represent a very big financial investment and they are a commitment to a method of tunnelling that includes standard solutions to what may be a very wide range of ground problems. It probably has to be admitted that the range of human and technical ingenuity in tackling tough ground problems is limited by the TBM "tunnel production" method. Problems incurred can, on occasion, be correspondingly exaggerated due to the reduced flexibility when a large tunnel is filled with perhaps 200 metres of heavy machinery and associated equipment. More room for human ingenuity and a wider range of solutions are available in the drill-and-blast method, including multi-drift excavation and more thorough pre-injection or pre-treatment methods.

Learning Curve or Geological Delays?

On some TBM projects, there are extended early periods of low productivity due to mistakes in expected ground conditions (Barton and Warren, 1996). These are usually followed by impressively inclined learning curves once redesign of TBM details and crew experience are optimised. A classic example of this is the Channel Tunnel project between England and France, where English contractors eventually broke world records for soft ground (chalk marl) tunnelling after making some design changes that were needed because of blocky, overbreaking, water-bearing ground in the early kilometres.

An incomplete but illustrative set of net penetration rates (metres per hour) and weekly, monthly and yearly progress rates (also expressed as metres per hour) are given in Table 1 and Figure 1. The data are obtained from a handful of hard rock projects in Norway (3.5 to 8.5m in diameter) and from some recent hard rock, large diameter TBM projects in the USA.

Table 1. Some typical experiences from hard rock TBM projects in Norway and the USA.

Net penetration rate m/hr	Daily progress m/hr	Weekly progress m/hr	Monthly progress m/hr	Yearly progress m/hr (extrapolated)
5	3	2	1.5	1.0
3	1.5	1.0	0.7	0.5
2	1.0	0.7	0.5	0.3

The trend exhibited in Figure 1 is a reminder to take a conservative view when developing prognoses, since geological phenomena (major inflows, face collapses, and stuck machines) may have a dramatic effect on actual average progress rates. If we could lift the projected yearly trends of the data shown in Figure 1 by avoidance (or suitable pre-treatment) of major

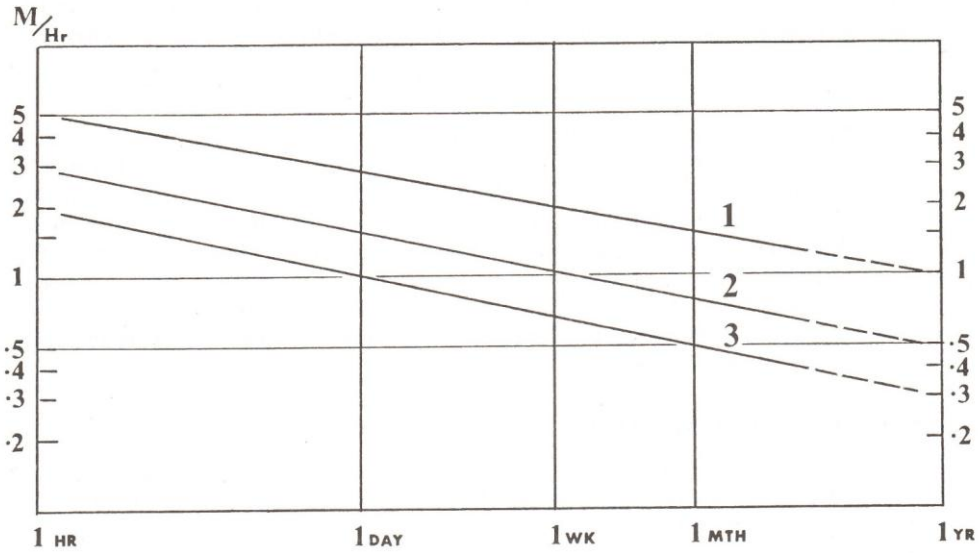


Figure 1. Net penetration rates, and declining TBM progress rates with time in some Norwegian and American hard rock TBM tunnels.

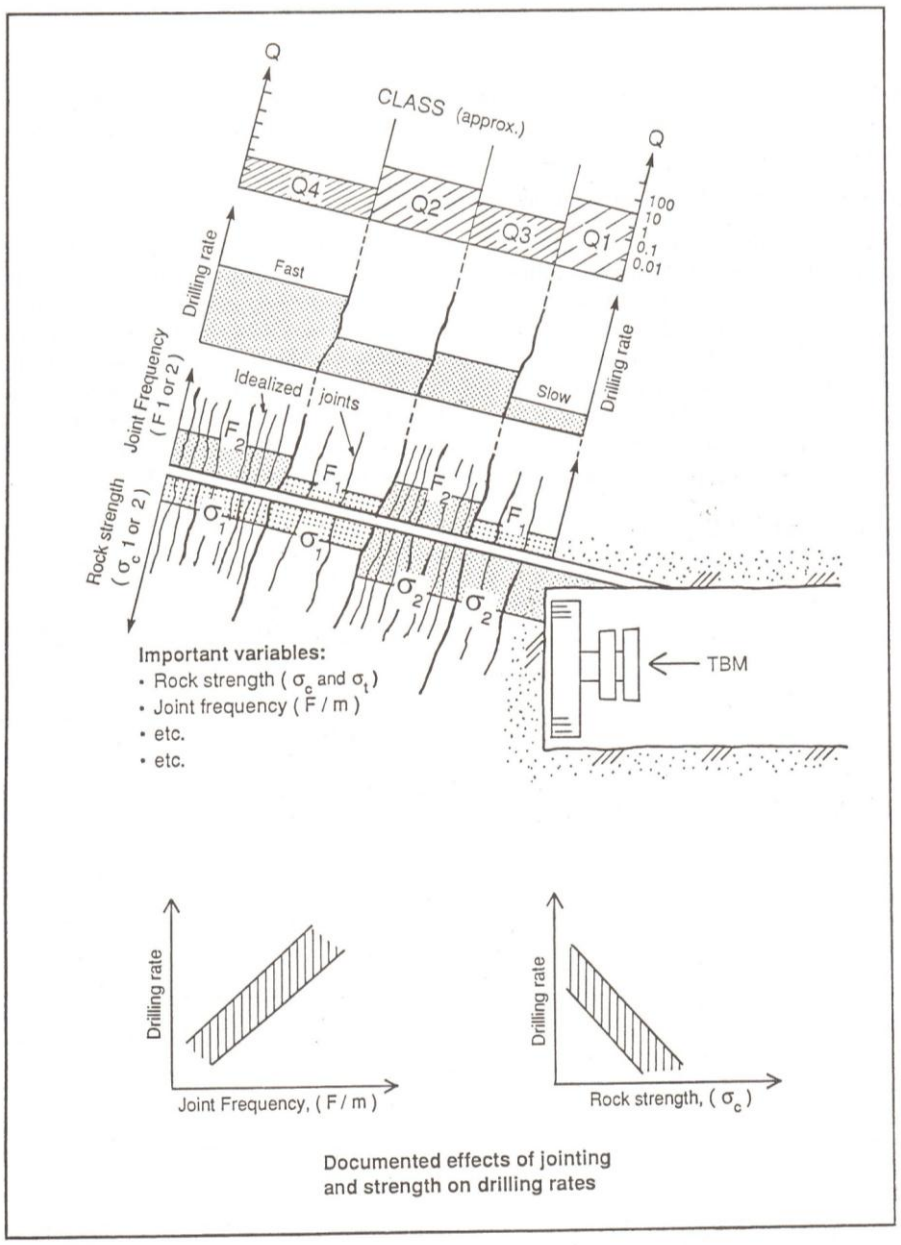


Figure 2. Simplified concept of the effect of joint frequency and rock strength on drilling rate and implied rock quality.

problem zones, then economies and schedules would be less in doubt. Other problems caused by longer tunnels and supply lines with increased time are of course part of the trend shown in Figure 1 and cannot readily be avoided.

Use of Information from Ahead of the Tunnel Face

When discussing how to obtain data from ahead of the face, we can consider the trends sketched in Figure 2. In principle, drilling rate is increased with a higher joint frequency, and reduced with higher rock strength. Increased joint frequency may eventually slow progress when the Q-value reaches some threshold if support measures delay boring or if conveyors become overloaded by "over-production" at the face. The second trend of reduced boring rate with high rock strength may of course be modified by the use of higher cutter loads. The data shown in Table 2 and Figure 3 has been collected from some of NGI's TBM consulting projects (Løset, personal communication, 1991) and shows the strong, direct influence of the ratio of compression strength and cutter load (MPa/tnf) on net penetration rates.

Table 2. Some Norwegian hard rock tunnel data.

Project		Rock	σ_c (MPa)	Q	diam (m)	m/hr	* (MPa/tnf)
(K)	Kobbelv	granite	200-250	1-40	6.5	1.2-1.5	9-12
(S_r)	Svartisen (r)	mica schist limestone sandstone	25-50 75-125	1-40	6.25	1.8	1.5-6
(S_h)	Svartisen (h)	mica schist limestone gneiss	25-75 100 200	10-30	3.5-4.3	3.5	2-5
(U)	Ulla-Førre	gneiss	200	20-30	3.5	1-2	10-11
(F)	Fløyfjell	gneiss quartzite h.b- schist	155-290	10-40	7.8	1.5-2.2	7.5-14
(V)	Veas	shale limestone sandstone	10-50	5-10	3.5	3-4	0.7-3.3
(J)	Jostedalen	gneiss	200-250	5-30	3.5-6.0	0.5-1.5	10-12.5

* Ratio of UCS and cutter load (MPa/tnf)

Not only UCS but also rock type and especially quartz content have of course a very large effect on cutter wear. At the Svartisen hydro-electric project (S_h, Figure 3), the following

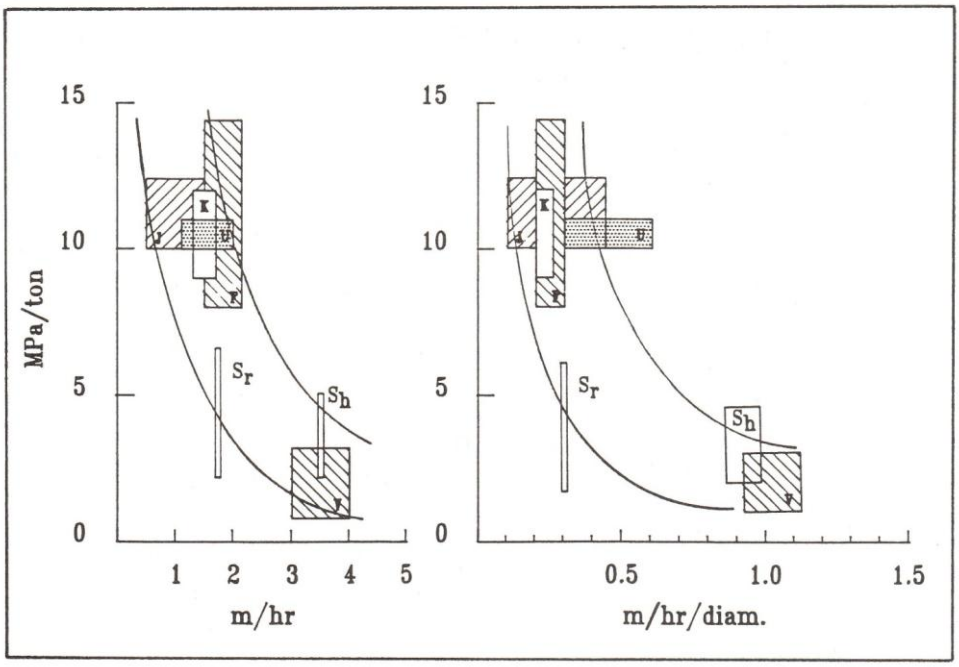


Figure 3. Net penetration rates for some Norwegian hard rock tunnels, versus rock UCS/cutter load in MPa/tnf.

production extremes per cutter were published (World Tunnelling, 1991).

limestone	215m^3 per cutter
granitic gneiss	60m^3 per cutter
quartzite	38m^3 per cutter

The mean for the 9 km tunnel, of which 60% was in mica gneiss, was 87m^3 per cutter.

Further ideas of the potential for use of rock data obtained from ahead of the face are illustrated in Figure 4. If the rock is probed on a continuous basis when rock quality is critical for advance rates, then an expectation of support needs and approximate convergence can be obtained. In the idealised example in Figure 4, it is assumed that the absolute final deformation ($\delta_A + \Delta_2$) has a direct relation with the Q-value. Barton *et al.* (1994) have shown that the ratio Q/span is indeed a determinant factor in numerous case records of convergence and extensometer (MPBX) measurements. The monitoring of convergence from a point as close to the cutter head as possible and at regular intervals behind the advancing TBM will allow some confirmation or correction of the predicted support class (*i.e.*, Q_4) to be made.

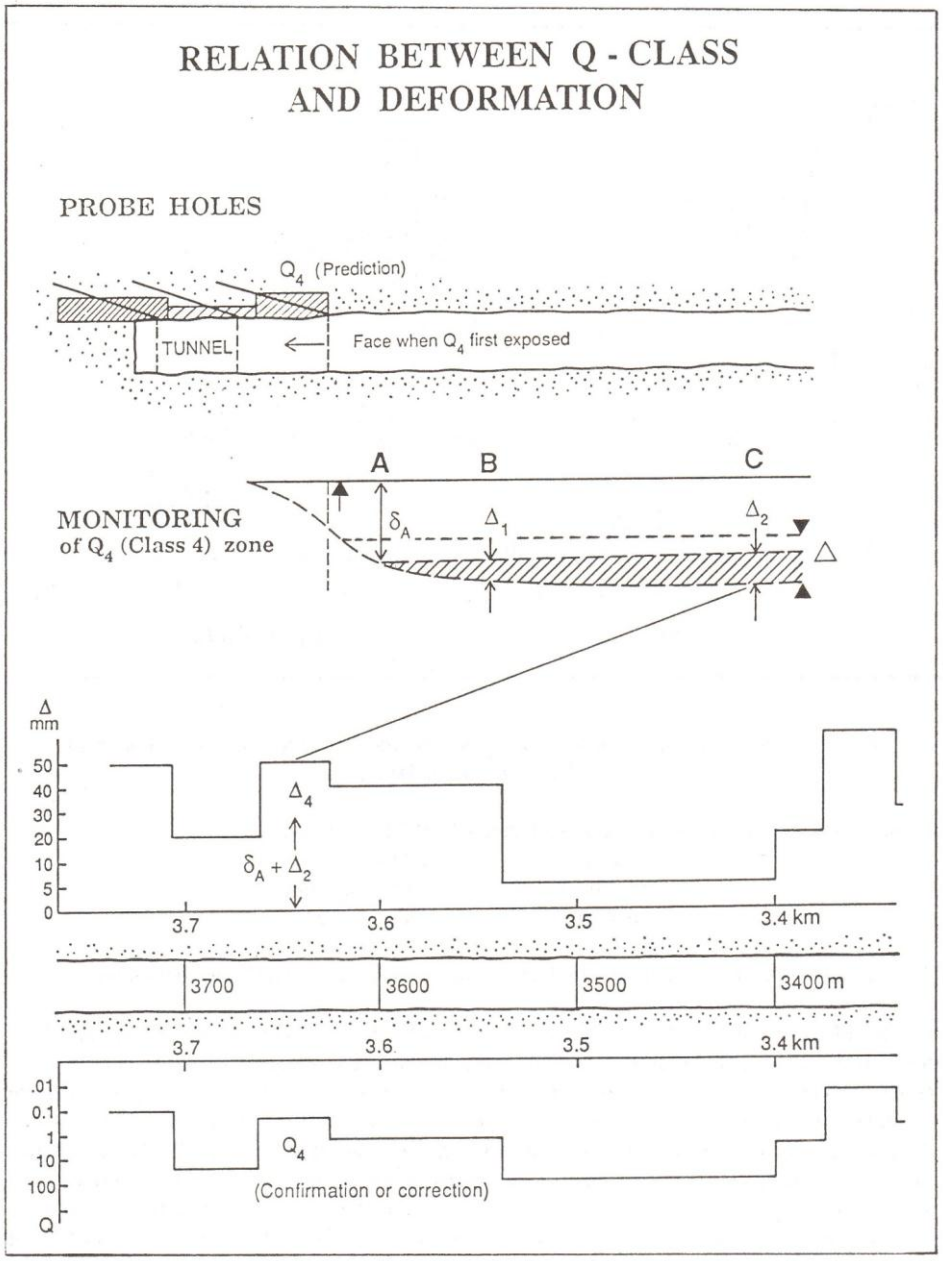


Figure 4. Idealised relation between Q class and deformation. δ_A will be unknown in most cases, but can be estimated.

The above concepts of Q-estimation, support selection, convergence monitoring and support class correction are illustrated in Figure 5. In this hypothetical example, probe drilling ahead of the face has (by means to be described later) given an indication of Q-class (in this case Q_4). The tunnel face is shown advancing through this quality and monitoring and Q-mapping ***after*** the cutter head is confirming (or correcting) this prognosis. In the actual real life large diameter hard rock machine envisaged there are four drills mounted at point A for pre-injection and bolting round most of the 360° profile. Different levels of bolting and shotcreting capabilities are available at A, B and C, as indicated at the bottom of Figure 5.

In the section titled "support alternatives" in Figure 5, the preliminary and final support measures are applied at locations A, B and C in accordance with rock qualities and final support needs. Quantities are adjusted at B and C in accordance with actual mapped Q-values and convergence monitoring results.

Seismic Probe Monitoring for Determining Support Class

The principles of utilisation of rock quality information from ahead of the tunnel face have been described in some detail in the foregoing. We have seen how indications of jointing frequency and rock strength (Figures 2 and 3) could be utilised in assessment of relative advance rates. If Q-values could be determined by some means ahead of the face, then support class changes and even advance rate adjustments could be anticipated ahead of the event.

This optimistic goal is illustrated in Figure 6. Measurement of seismic P-wave velocity profiles using rapidly drilled percussion holes ahead of (and usually above) the cutter head is the potential source of our Q-value estimates. This method is preferable to core drilling as a routine method, since it is relatively easy to drill 25 metres or more ahead of the face during each hour of cutter replacement time. Longer holes, even core drilled, could be made during a maintenance shift, if considered necessary.

A relationship between seismic velocity and Q-value for hard jointed rocks at shallow depth was established some years ago (Barton, 1991).

$$V_p \approx \log_{10} Q + 3.5 \qquad 1)$$

(where V_p is expressed in km/s)

This simple relationship (V_p = 3.5 km/s when Q = 1.0, V_p = 2.5 km/s when Q = 0.1, *etc.*) is illustrated in Figure 7.

The approximate indications of jointing frequency and RQD shown in the figure are average results reported by Sjøgren *et al.* (1979) in relation to velocity measurements from 115 km of seismic refraction measurements. The F (m^{-1}) and RQD (%) estimates were obtained from nearly 3 km of logged drill core at the same sites. Results were obtained from the following

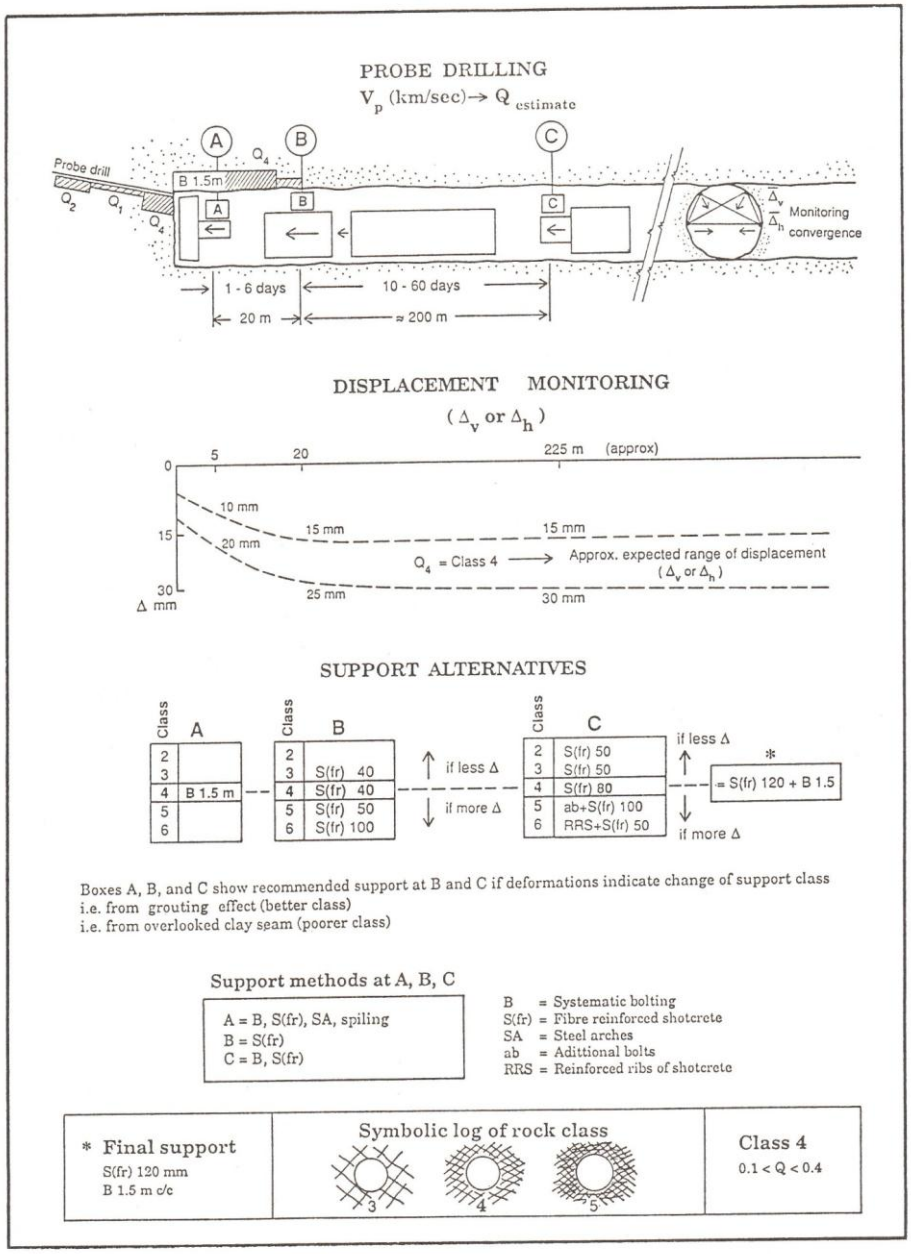


Figure 5. Probe drilling, rock class selection, monitoring convergence, correction of support measure if needed.

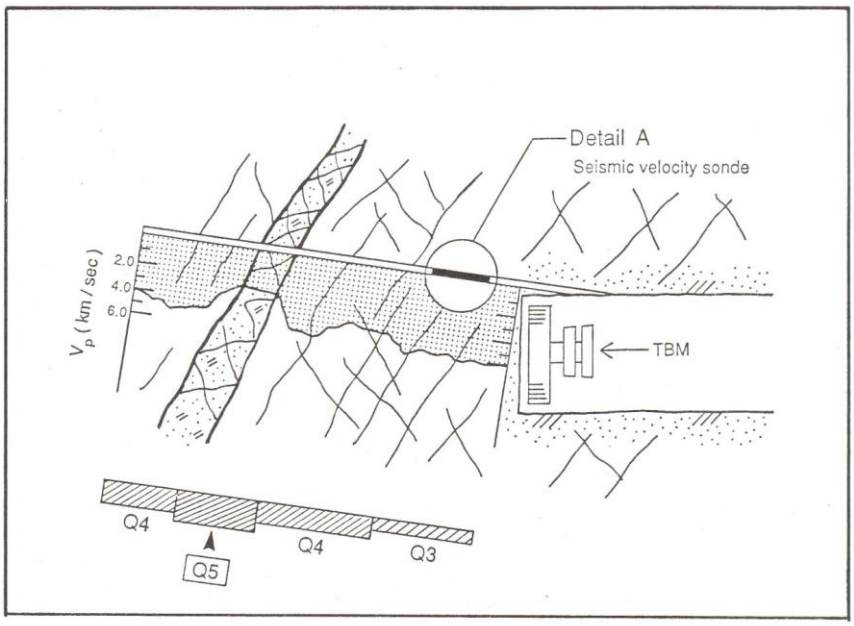


Figure 6. Conceptual support class estimation by converting seismic V_p measurement to Q-value.

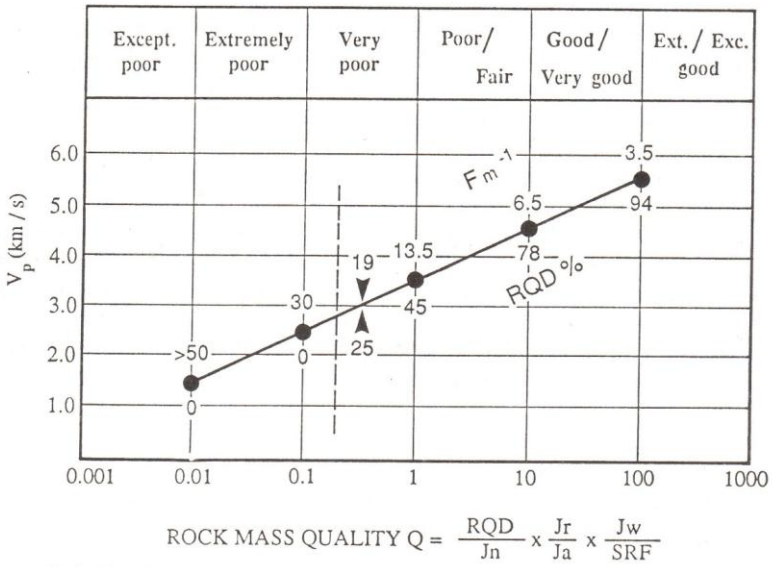


Figure 7. Q-V_p relationship for hard, jointed rock masses at shallow depth (Barton, 1991).

hard Norwegian rocks: granite, gneiss, amphibolite, pegmatite, meta-anorthosite, porphyry, quartzite and mylonite.

A subsequent survey of numerous V_p measurements from sites in chalks, chalk marl, sandstone, mudstones, shales, tuff, *etc.,* where Q-values were known and depths were variable made it clear that adjustments to equation 1 were required for rock compression strength and porosity for the case of softer rocks. Furthermore, depth was an important variable. The Q-scale in Figure 7 needed to be shifted to the right since a logged Q-value of say 10 might correspond not to 4.5 km/s as in equation 1 but to a velocity as high as 5.5 km/s based on cross-hole measurements at several hundreds of metres depth.

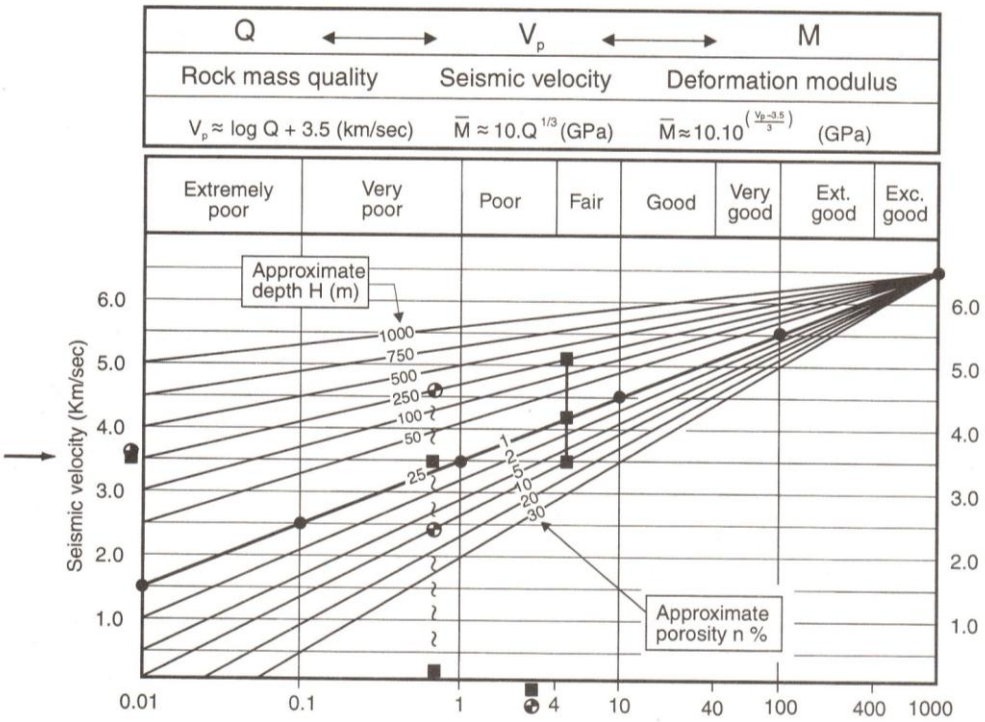
It has recently been shown in approximate empirical terms (Barton, 1996) how Q, V_p, σ_c, n%, M and depth are inter-related. Details are shown in Figure 8.

It will be noted that the conventional Q-value consisting of six parameters, has been "normalised" by uniaxial compression strength, based on the need for adjusting Q to Q_c when the uniaxial strength is markedly different from an assumed typical hard rock value of 100 MPa. Porosities different from an assumed nominal hard rock value of 1% are allowed to reduce the velocity estimates, while depths greater than the assumed nominal 25m (maximum) depth of normal refraction seismic surveys are allowed to increase the velocity estimates.

In the proposed seismic tunnel probing method sketched in Figure 6, the seismic design chart shown in Figure 8 is needed to interpret measurements. It will be utilised first from the left hand (V_p) side. A measured V_p value of say 3.5 km/s from a probe hole (see arrow) is followed horizontally to the right on the chart until it intersects the nominal low porosity (n = 1%), hard rock (σ_c = 100 MPa) shallow location (25m) result for Q_c (= Q) = 1.0. [The central line (with black dots) is the equation 1, Figure 7 prediction.]

The next step is to make an estimated adjustment for porosity. The rock is a sandstone with n = 10%. This means that a relatively higher Q_c value (about 5) will be needed to explain the V_p value of 3.5 km/s. Let us suppose, however, that the sonic probe measurement was made at a tunnel depth of 250m in these sandstones. This would imply an increment of V_p of about 0.9 km/s in relation to a shallow seismic refraction result. This more than "cancels" the V_p reduction of 0.7 km/s caused by 10% porosity. This means that a Q_c value less than 1 needs to be chosen, in order to give the necessary increment of 0.9 - 0.7 = 0.2 km/s above the nominal "shallow, hard rock, low porosity" reference line. Our estimated Q_c value is 0.7 (see black square).

The final step in using the rock mass seismic quality chart (Figure 8) is to correct the Q_c-value to the Q-value. If we assume in this example a uniaxial compression strength (σ_c) equal to 25 MPa, the estimated Q-value becomes $0.7 \times 100/25 = 2.8$, *i.e.,* a "poor" rock mass quality.



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

Figure 8. Rock mass seismic quality chart for estimating inter-relationship between Q, V_p, M, σ_c, n and H (σ_c = uniaxial compression strength of rock).

Of course one can use the diagram in the opposite order, starting with an observed Q-value and estimating what the likely seismic velocity will be at the same tunnel depth of 250m. In this case, we start with Q = 2.8, correct to $Q_c = 0.7$ using $\sigma_c = 25$ MPa and follow vertically up the chart to a preliminary "shallow, hard rock, low porosity" reference of about 3.3 km/s. We then have to correct for the conflicting influences of 10% porosity (V_p reduction = 1.0 km/s) and 250m depth (V_p increase of 1.3 km/s). Refer to the shaded circles in Figure 8. The predicted V_p value at tunnel depth is 3.3 - 1.0 + 1.3 = 3.6 km/s, almost identical to the first method (3.5 km/s). There will usually be small differences in these two methods of estimation which can be eliminated by averaging.

TBM Q-Value Compared to Drill-and-Blast Values

The attractiveness of the TBM method of excavation besides high excavation rates (when conditions are favourable) is that the reduced level of disturbance compared to drill-and-blast gives a reduced need of rock reinforcement (and an apparently higher Q-value) over a significant central range of rock qualities. When, however, the rock is of very poor quality, almost equal stability conditions (overbreak problems) will be experienced by the two methods of excavation and a common, lower Q-value will tend to be in operation.

In the Svartisen road tunnel, which is listed in Table 1, a rather unique opportunity was presented to map the 6.25m diameter TBM tunnel and later to repeat the mapping when the same tunnel was enlarged by drill-and-blast to a 7.5m diameter road tunnel (see Figure 9). NGI's senior geologist Løset (1992) found that on average the Q-values mapped in the TBM tunnel were 1.5 to 3.0 times higher than the values mapped in the drill-and-blasted version of the same tunnel. However, this mismatch was only experienced where the TBM Q-values were in the middle range of 4 to 30 (*i.e.*, in the drill-and-blast tunnel they were about 2 to 10). Both above and below these ranges the Q-values were similar with both types of tunnel excavation. Nearly a doubling of the quantities of rock bolts and shotcrete were specified for the drill-and-blasted version.

It appears from other experiences of mapping TBM tunnels that this "central threshold" of Q-values (where there are significantly different Q-value estimates) occurs over a lower range of rock qualities in small span tunnels, and the differences here may also be more marked. For example, drill core or drill-and-blast estimates of Q-values in the range 0.5 to 2.0 may be judged to be significantly higher (*i.e.*, as much as $Q \approx$ 2 to 10) in 2 to 3m size pilot tunnels driven by TBM. Conversely, the "central threshold" occurs at an even higher range of Q-values in the case of large 8-12m TBM tunnels. The Q-values encountered in the large diameter Channel Tunnel TBM UK drives, which were generally in the range 4 to 10 in the early kilometres of overbreaking rock, were below the "central threshold" (where Q_{TBM} becomes markedly higher than Q_{actual}) and are therefore comparable with drill-and-blast estimates.

These differences in judging Q-values are due to the maintenance of limited joint lengths in the case of TBM excavation, and the general impression of higher RQD, lower J_n and lower

(better quality) J_a-values. It is difficult to spot thin clay fillings until they are exposed by overbreak. Incipient joints tend to remain incipient in TBM tunnels, while they will tend to be propagated by drill-and-blast disturbance. A potential negative aspect of TBM excavation, however, is the maintenance of high tangential stresses close to (at) the tunnel wall when blasting is not used for excavation. On occasion (high overburden, massive rock) SRF values may be judged to be higher in the case of the TBM tunnel, due to less favourable stress/-strength ratios and experience of time-dependent stress-induced slabbing (Løset, 1992).

The support recommendations given in Figure 10 relate, in the first instance, to drill-and-blast case records of which there are now some 1,250 cases (Grimstad and Barton, 1993). When this support chart is used as guidance for TBM tunnel support, it is advisable to adjust the

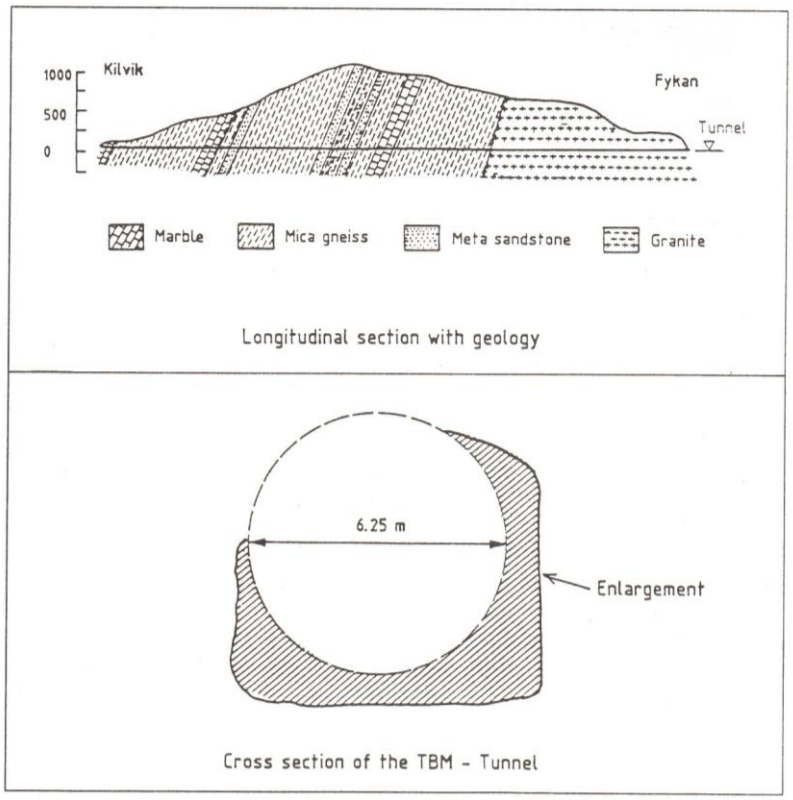
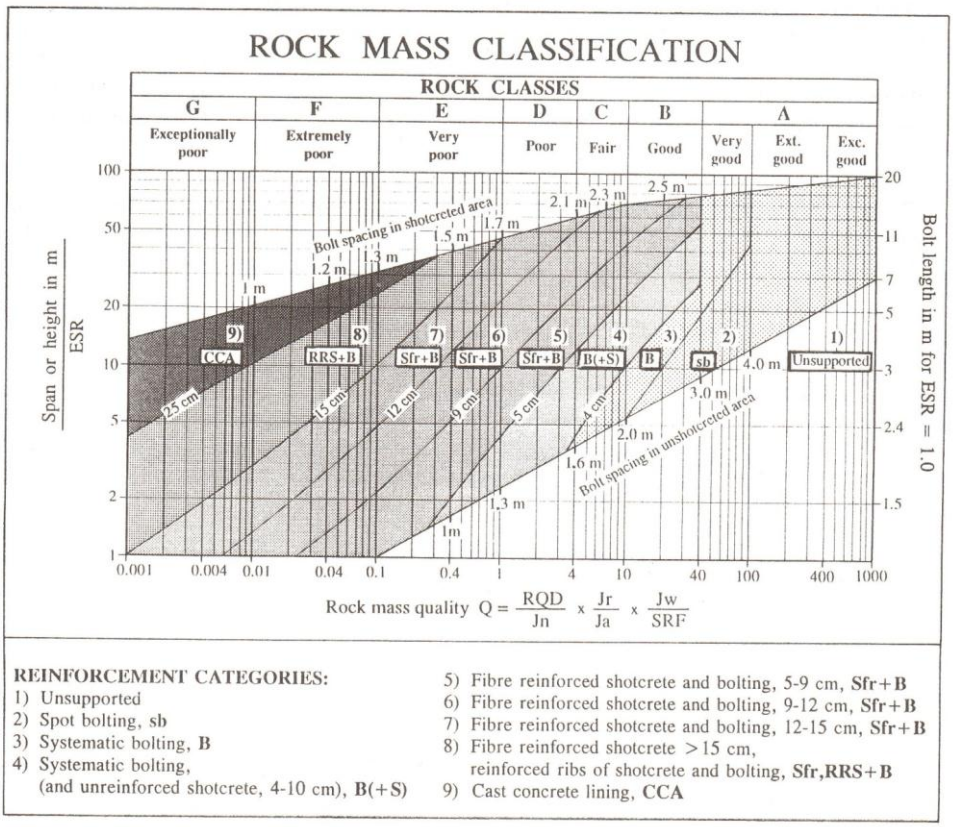


Figure 9. Svartisen road tunnel geology, overburden and final cross-section after drill-and-blast enlargement of a previous owner's TBM tunnel (Løset, 1992).



REINFORCEMENT CATEGORIES:

1) Unsupported
2) Spot bolting, **sb**
3) Systematic bolting, **B**
4) Systematic bolting, (and unreinforced shotcrete, 4-10 cm), **B(+S)**
5) Fibre reinforced shotcrete and bolting, 5-9 cm, **Sfr+B**
6) Fibre reinforced shotcrete and bolting, 9-12 cm, **Sfr+B**
7) Fibre reinforced shotcrete and bolting, 12-15 cm, **Sfr+B**
8) Fibre reinforced shotcrete >15 cm, reinforced ribs of shotcrete and bolting, **Sfr,RRS+B**
9) Cast concrete lining, **CCA**

Figure 10. Q-system support chart for drill-and-blast tunnels. When utilised for TBM support recommendations, adjust to Q_{TBM} in the "central threshold" range of qualities.

rock mass Q-value to a higher value within the "central threshold" range of rock qualities if TBM support quantities are to be estimated ahead of the tunnelling. The experiences of improved qualities registered in "central threshold" ranges of qualities referred to earlier are listed below:

span	6-7m	Q_{TBM} =	4-30	$Q_{actual} \approx$	1.5-10
span	2-3m	Q_{TBM} =	2-10	$Q_{actual} \approx$	0.5-2

In each case it can be noted from Figure 10 that the TBM range of Q_{TBM} values crosses the no-support boundary, *i.e.,* the above ranges are in the threshold of light-support, no-support. The rock mass quality judged from horizontal core or drill-and-blast excavation would require light systematic support, *i.e.,* systematic bolting or systematic bolting plus unreinforced shotcrete. The relatively reduced need of support for the TBM tunnel is partly real, joints remain incipient, disturbance is slight, arched beams of rock remain intact but would be fractured into several pieces by blasting. The danger is in failing to recognise clay bearing surfaces where kinematic release is possible once softening has progressed due to the radial unloading caused by tunnelling. Løset *et al.* (1996) describes the case of a 7m long wedge of rock falling onto the tail of a 3.3m TBM, due to failure in recognising the reduced quality.

On balance, and as a preliminary recommendation for support selection, we may estimate that TBM tunnel support needs are equivalent to selecting an ESR value of 2.0 (a halving of the equivalent dimension) in the "central threshold" area. Outside this range both above and below, the rock behaves as equally massive or equally overbreaking and requiring support, whether the tunnel is TBM excavated or drill-and-blasted. The massive rock will show "half-pipes" and give a "perfect" profile almost equivalent to that of a TBM tunnel. The overbreaking rock on the other hand will give a profile that is structurally (joint-plane) controlled and show great similarity between the TBM and drill-and-blast cases, and will require similar levels of support which can be readily selected from Figure 10 using correct values of ESR.

Conclusions

1. TBM excavation represents a big investment in an inflexible but potentially very fast method of excavating and supporting a tunnel. When extremely unfavourable conditions are encountered without warning, schedule and practical consequences are often far greater in the TBM driven tunnel than in the drill-and-blast tunnel.

2. A method of utilising sonic velocity probe data obtained from percussion holes drilled ahead of the face has been proposed in order to minimise risk and maximise the potential advantage of TBM drivage. The V_p data is converted to rock quality and support class estimation using a basic hard rock relationship between Q and V_p with systematic adjustments for uniaxial compression strength, porosity and tunnel depth. Convergence monitoring can be used for confirmation or correction of these prognoses.

3. The ranges of Q-values that are interpreted as likely tunnel face conditions with further TBM advance must be utilised with care, since TBM tunnelling causes a markedly reduced disturbance effect in a central (threshold) range of Q-values because of the reduced propagation of incipient joints and the favourable curved nature of quite intact "rock arches" in the periphery of the TBM tunnel. These will tend to be damaged in blasting and will not be effective in the same way due to the overbreak that may cut into these "rock arches".

4. In the central threshold region, Q-values will often be judged to be between 1½ to 5 times higher when logged in the TBM tunnel compared to drill core or drill-and-blast tunnel logging. In some of this region on the Q-support chart, the TBM tunnel will be judged correctly as not requiring systematic support, where the drill-and-blasted tunnel would require support. The central threshold range of Q-values is tunnel size dependent and requires careful assessment from case to case before reliable recommendations can be given.

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