ESTIMATING ROCK MASS DEFORMATION MODULUS FOR EXCAVATION DISTURBED ZONE STUDIES

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ABSTRACT
The full scale deformation modulus that needs to be used in numerical models of tunnels, caverns and geological disposal facilities (or related tunnel research sites) is of fundamental importance to the stresses, displacements and magnitude of the excavation disturbed zone or EDZ that is predicted. An alternative to direct measurement which has merit in scoping exercises and may be accurate enough for detailed design is outlined in the paper. The method is derived from rock mass characterisation methods and was initially based on correlation between Q and RMR to give access to additional case records. Key parameters in the new method include the seismic P-wave velocity obtained from seismic refraction surveys, or from cross-hole seismic tomography, the Q-value, the depth of the site and the physical properties of the matrix as described by its uniaxial compression strength and porosity. The method has been checked at hard rock sites with sparse or frequent jointing, and in weaker, porous rocks which have no relevance to waste disposal but which provide a large range of conditions for verification.

INTRODUCTION

Predicting the behaviour of excavations in rock masses is complicated by the huge number of interlocked pieces of rock that react with one another *via* non-linear stiffness and strength components. In one major school of rock mechanics the obviously discontinuous rock mass is simplified as if it were a continuum. Finite element, finite difference, or boundary element analyses are utilised with elastic or elasto-plastic constitutive models. There is an obvious need for a good estimate of the rock mass modulus which takes into account "all" the features of the rock mass that are otherwise ignored. Nevertheless, details of behaviour are sure to be missed in such analyses and it is often necessary to change the modulus close to the excavations to get better fit to observed rock displacements, *e.g.*, Barton and Bakhtar (1983).

Another school of rock mechanics which is expanding due to the needs for more detailed understanding of "real" behaviour, follows the argument that major sets of joints and discontinuities can be represented discretely in two- and three-dimensional distinct element models such as UDEC or 3DEC, *e.g.*, Cundall and Hart (1993). However, since the number of discrete blocks that can be modelled is still rather limited—usually no more than a few thousand blocks—it is inevitable that the detailed joint structure that is seen in a scale of a few metres is only represented in terms of a deformation modulus and Poison's ratio. Only the major joints, *i.e.*, those which are expected to affect performance most, are modelled discretely. An exaggerated example of major joints and their contrast to minor structure is shown in Figure 1. There is a general tendency for the minor structure to have higher shear strength and stiffness than the major structures, a feature that is deliberately exaggerated in Figure 1.

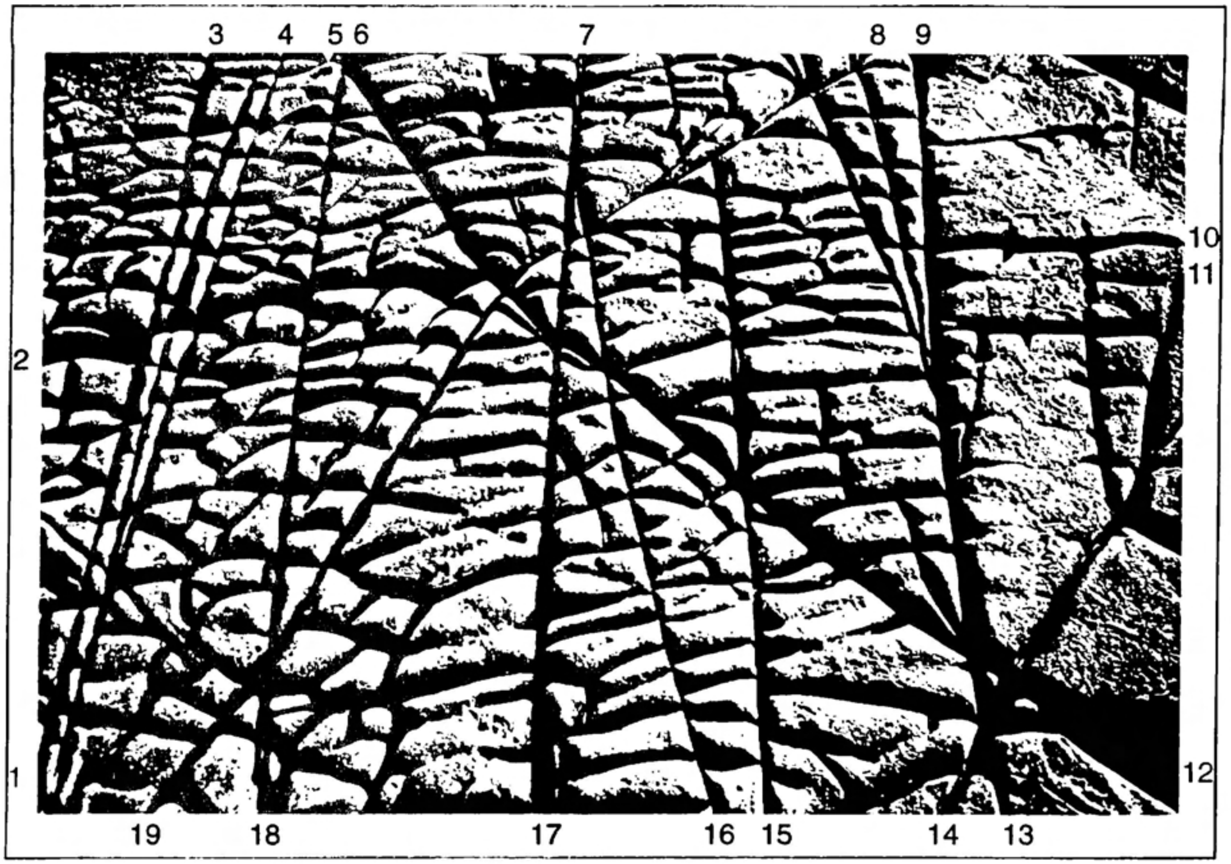


Fig. 1 *Distinct element models of such a rock mass would usually have to ignore details of structure and concentrate on the numbered features which have lower shear resistance and are more active in M-H coupling.*

During the development of this paper, a method will be derived for estimating a rock mass modulus that is sensitive to the general characteristics of the rock mass *via* the Q or RMR value, *e.g.*, Barton *et al.* (1980), Bieniawski (1989). This estimate of static modulus will also be linked to the P-wave seismic velocity, with adjustment for stress level, and for rock matrix porosity and uniaxial compression strength. The method has been developed from data ranging from soft porous rock masses to hard jointed rock masses. It is empirical in nature and appears to work well within the limits of our knowledge of these inter-related rock mass parameters.

THE EFFECT OF TEST SIZE ON MODULUS

A particularly well documented example of the problem of test size on modulus determination is that described by Heuzé (1981), for the Climax Mine-By experiment in jointed quartz monzonite. Lawrence Livermore National Laboratories (LLNL) utilised six methods for evaluating the deformation modulus, in an attempt to obtain better data to improve their modelling of the rock mass response to excavation of a central drift of 5m span that was driven along the pillar between existing tunnels. The results of the laboratory scale and field scale measurements are reproduced in Figure 2. The best field estimate of 26 GPa (from tunnel relaxation analysis) was very close to the estimates of modulus (M) obtained from RMR and from Q utilising the following equations from Bieniawski (1978) and Barton *et al.* (1980).

$$M \approx 2RMR - 100 \quad \textbf{(1)}$$

$$M \approx 25 \log_{10} Q \quad \textbf{(2)}$$

The mean values of RMR = 72.5 ($M \approx 25$ GPa) and Q = 10 ($M \approx 25$ GPa) give estimates of modulus that do not in general show such close agreement between these two equations, except over a limited range of RMR and Q-values, (*i.e.,* approximately RMR = 50, Q = 1 where both estimates of M vanish to zero).

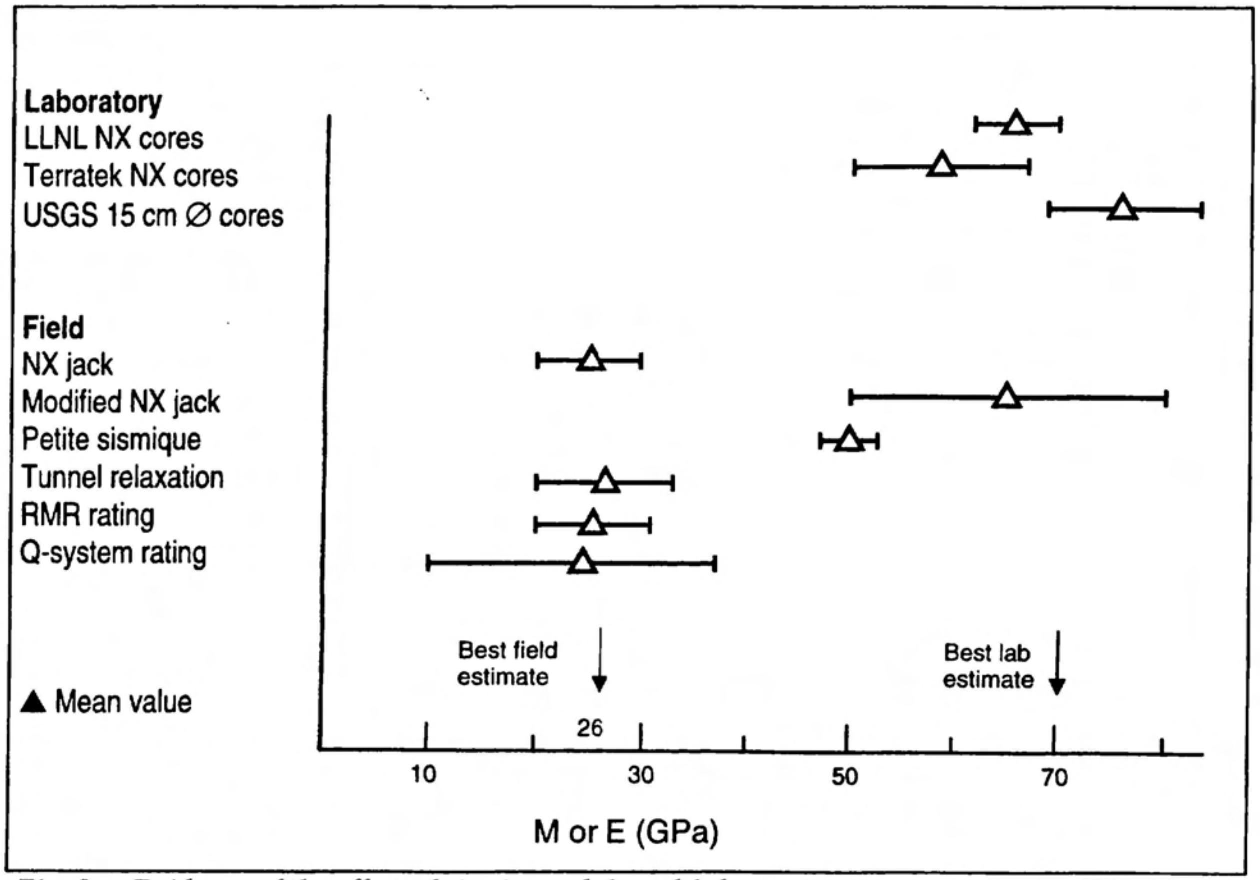


Fig. 2 Evidence of the effect of size in modulus of deformation measurements at Climax Mine-By Experiment (Heuzé, 1981).

THE EFFECT OF CONFINEMENT AND DISTURBANCE ON MODULUS

Some of the best documented measurements of disturbed zone effects around excavations are to be found from the URL studies by AECL and their contractors. Measurements of deformation modulus as a function of distance from rectangular and circular openings reported by Koopmans and Hughes (1988) leave one in no doubt about the importance of confining pressure. The results of CSM dilatometer measurements surrounding the rectangular (upper) section of the URL shaft are reproduced in Figure 3.

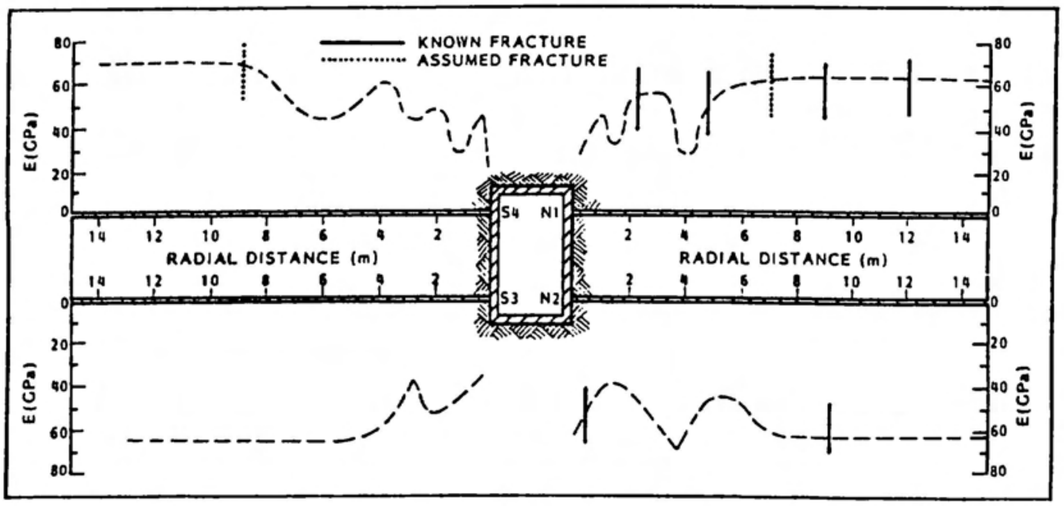


Fig. 3 Secant moduli of deformation measured by CSM dilatometer at the upper levels of the URL shaft (Koopmans and Hughes, 1988).

In the case of similar measurements surrounding the circular Room 209 at URL, the modulus decreased some 10 to 20 GPa in the last 2 to 3 metres from the drift wall in three out of four radial boreholes, increasing by about 15 GPa in the case of the fourth vertically upwards hole in the roof. Generally the zone of influence on modulus was only about one radius into the perimeter of the excavation.

Other studies reported by Koopmans and Hughes (1986) show modulus measurements in sandstone at a mine in Nova Scotia, where the effect of TBM excavation was compared with adjacent drill and blast excavation. Figure 4 shows a background modulus of about 10 GPa increasing erratically to between 15 and 27 GPa in the 3 metres closest to the 7.5m diameter TBM tunnel, while the increase in modulus (to a peak value of 35 GPa) is seen one diameter away from the drill and blast tunnel, with small decreases (below 10 GPa) in the innermost radius.

The competing effects of reduced radial stress close to the opening and increased tangential stress away from the excavation walls (in the case of the drill and blast tunnel) are no doubt responsible for these effects. It would appear that disturbed zone (*i.e.*, stress redistribution) effects can outweigh damage zone (*i.e.*, method of excavation) effects in terms of modulus magnitude changes. However, the drill and blast method may "physically displace" the location of a presumably stress-related modulus increase further into the rock mass, if blasting disturbance is heavy. This is presumably because the peak tangential stress is apparently itself transposed deeper into the rock mass.

THE EFFECT OF CONFINEMENT AND DISTURBANCE ON SEISMIC VELOCITY

Since deformation modulus and seismic velocity are each affected by stress level, and both have an intrinsic relation to the rock mass quality or Q-value, it is now of interest to record some measured effects of stress and disturbance on seismic velo-

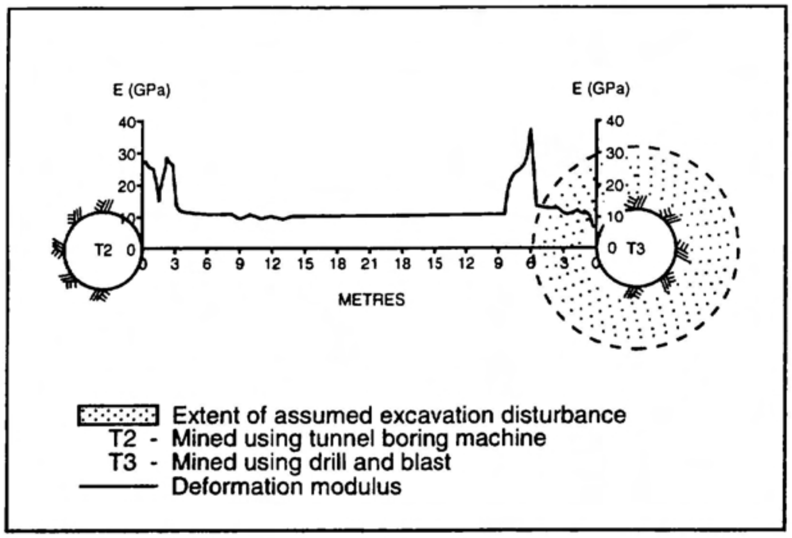


Fig. 4 Contrasting effects of TBM (T2) and D+B (T3)excavation on dilatometer measurements of modulus (Koopmans and Hughes, 1986).

city. A remote sensing method such as seismic is inherently attractive for mapping the extent and magnitude of a disturbed zone, as attempted at many project sites.

Kujundzic *et al.*(1970) have shown the results of cross-hole seismic measurements at 45° (1.2m) intervals around a circular pressure tunnel of 6m diameter driven by TBM. The average result for the eight radial directions showed a V_p value of 3.5 km/s at the wall, increasing to about 5 km/s 1m into the wall and declining to the far field value of about 4.5 km/s after some 2 to 3m. Cross-hole seismic measurements in the walls of a drill and blasted excavation in strongly jointed basalts reported by King *et al.*(1984) showed more dramatic results with up to 2 km/s reduction in velocity at the tunnel walls, from a background value of about 5.5 km/s.

At Grimsel, in massive granitic rocks Egger (1987) reported only 0.5 km/s reduction in velocity at the walls of the TBM test drift, from a background value of 5.2 km/s. In the absence of joints, presumably the ***damage*** zone together with the radial stress reduction causes the reduction in velocity, while the other ***disturbed*** zone component (*i.e.,* from the increase in tangential stress) has in this case no joints available to close, so only a very minor increase (0.1 km/s) is seen. There are some indications of similar behaviour in the rather sparsely jointed granite and dolerite at Äspö, within the SKB, Nirex and Andra ZEDEX project, which is reported elsewhere in this conference.

A more direct proof of the effect of stress increase on seismic velocity in jointed rock is that provided by the seismic cross-hole tomography (Figure 5) performed by NGI at the Gjøvik cavern in Norway. The velocities increased almost linearly down borehole No. 3 from about 3.8 km/s at 10m depth, through 4.6 km/s with 30m depth, to about 5.8 km/s at 60m depth. However, the joint frequency (from three to four mostly steeply dipping sets) remained in the general range of 4 to 8 per metre and RQD was generally 85 to 95% throughout this depth range. An important extra detail from this site was that the horizontal stress increased from about 3 MPa to about 6 MPa over this same depth range. In other words, the 2 km/s increase in velocity may have mainly occurred due to the 3 MPa increase in major horizontal stress, rather than from 50m depth increase.

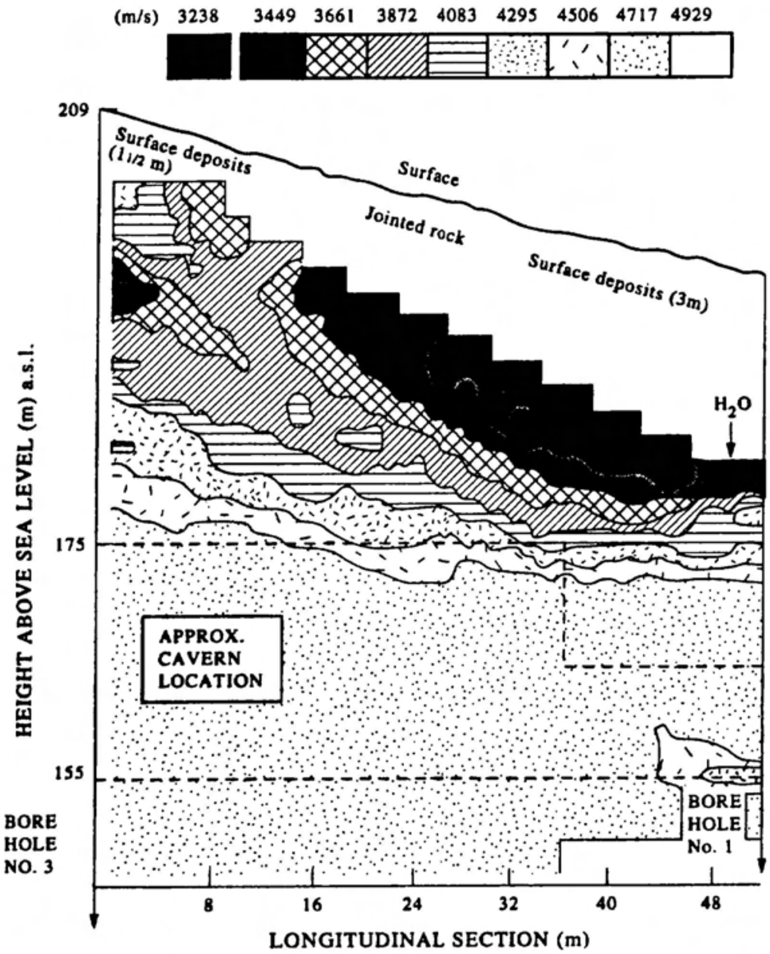


Fig. 5 Velocity increases down borehole No. 3 at the Gjøvik cavern site were not accompanied by reductions in joint frequency. Barton et al. (1994)

RELATIONSHIPS BETWEEN MODULUS (M) AND Q AND RMR

Attempts to correlate RMR and Q values have been common in the literature since Bieniawski (1976) suggested the relation:

$$RMR = 9 \ln Q + 44 \quad \text{(3)}$$

This was reinforced by further case records in Bieniawski (1989). However, the scatter has always been quite large and several hundred cases recently added by NGI have led to a reappraisal of the database and a suggestion of the following simple relation (Barton, 1995):

$$RMR = 15 \log Q + 50 \quad \text{(4)}$$

Equation 4 has been utilised here in order to be able to access earlier correlations between deformation modulus (M) and RMR (Bieniawski, 1978) and correlations between M and Q values for the same case records (Barton *et al.*, 1980).

The earlier correlations of modulus and RMR from Bieniawski (1978) (equation 1) and of modulus and Q-value from Barton *et al.* (1980) (equation 2) have each been improved, first by Serafim and Pereira (1983) to the non-linear form:

$$M = 10^{\frac{RMR-10}{40}} \tag{5}$$

and then by Barton (1995) to the non-linear form:

$$M = 10\ Q^{1/3} \tag{6}$$

The above equations are compared in Figure 6.

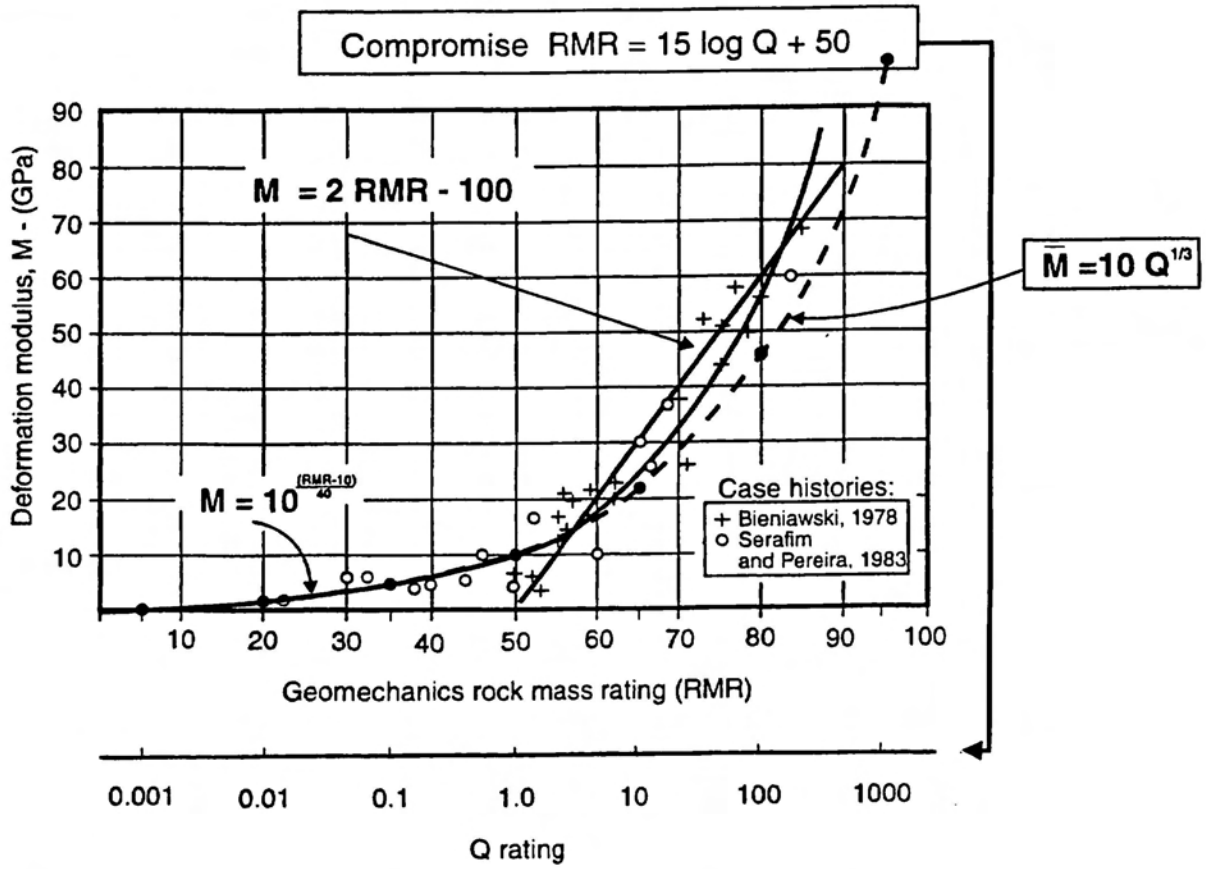


Fig. 6 Comparison of alternative correlations between deformation modulus (M) and Q and RMR. Other case records of Q versus M were given by Barton et al. (1980)

There is a considerable body of case records that plot below the curve represented by equation 6, especially in the range Q ≈ 2-20. The relevant moduli are in the approximate range 2 to 20 GPa, and were given by Barton *et al.* (1980). This additional data was the reason for the initial preference for equation 2 relating modulus and Q-value which worked well for moderately strong to hard jointed rocks and gave an extremely good fit between predicted and measured displacements at the Gjøvik cavern (Barton *et al.*, 1994).

The Q-system, however, was developed primarily for selecting appropriate rock support for tunnels and caverns and does not consequently have directly applied ratings for rock matrix compression strength (only the ratio σ_c/σ_1). Nor does it have a value for porosity. Both are needed for improved correlation with modulus and seismic velocity where softer rocks are concerned. A new seven parameter version of the Q-value for use when correlating Q with V_p and modulus M is written as follows:

$$Q_c = \left(\frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \right) \frac{\sigma_c}{100} \quad \textbf{(7)}$$

In effect, the Q-value is normalised to σ_c = 100 MPa (a typical hard rock value) and simply reduced or increased by rock uniaxial compressive strengths below or above 100 MPa.

INTER-RELATIONSHIPS BETWEEN MODULUS, SEISMIC VELOCITY AND Q

The first relationship between Q and V_p to be described here was developed as a result of the seismic tomography performed at the Gjøvik cavern project, and comparison of velocities with adjacent Q-logging of core from the same holes. In addition, the author performed a limited survey of cross-hole and seismic data from other sites particularly in the far east, and confirmed that for shallow depths (*i.e.*, 5 to 50m, typically 25m) and for moderately hard to hard jointed rocks, the relationship:

$$V_p = \log Q + 3.5 \text{ km/s} \quad \textbf{(8)}$$

gave a surprisingly good estimate between Q and V_p. The attractiveness of this equation is that it is especially easy to recall during field work, *i.e.*, when Q = 1, $V_p \approx$ 3.5 km/s, when Q = 10, $V_p \approx$ 4.5 km/s, *etc.*

A detailed study of jointing-velocity relations has been presented by Sjøgren *et al.* (1979), based on some 115 km of seismic refraction traces and logging of 2850m of adjacent drill core in the following Norwegian rocks: granite, gneiss, amphibolite, pegmatite, meta-anorthosite, porphyry, quartzite and mylonite.

Sjøgren *et al.* (1979) relate joint frequency F to V_p, and RQD to V_p. The relation given in equation 8 has been added to Sjøgren's data at the bottom of Table 1. The commonly experienced increase in velocity with increasing depth (see for example Figure 7) means that the scale of Q-values given in Table 1 needs to be shifted to the right. In other words, a velocity of 5.5 km/s will not imply a Q-value as high as 100 if, for example, the tunnel is at a depth of 100m or 500m.

Table 1 Approximate trends between V_p and rock quality parameters for the harder rock types at shallow depth (up to 25m). (Sjøgren *et al.*, 1979; Barton *et al.*, 1992).

V_p (km/s)	3.0	3.5	4.0	4.5	5.0	5.5
RQD (%)	24	46	61	76	87	94
F (m^{-1})	≈ 20	≈ 14	10.2	6.7	4.4	3.4
Q	0.32	1.0	3.2	10	32	100

The rock matrix properties such as uniaxial compressive strength and porosity (and therefore density) also have a significant effect on velocity, as shown by the following set of data from projects that the author has been involved in.

For the shallow tunnel depths given in Table 2, each rock type was strong enough to avoid overstress, and in principle similar tunnel support could be applied, despite larger expected deformations (several mm larger) in the weaker rocks. The need to utilise equation 7 (the seven parameter Q_c-value), together with corrections for depth and porosity to explain such results is evident.

Table 2 Examples of mechanical and velocity data variations for the same range of Q-values as needed for rock support purposes.

	Q	V_p (km/s)	M (GPa)	σ_c (MPa)	n %	depth (m)
1. Jointed gneiss	2-30	3.5-5.0	20-40	60-90	≈ 1%	25-50
2. Jointed chalk	2-30	2.2-3.0	4-10	10-20	10-40%	25-50
3. Jointed chalk marl	2-30	2.0-2.4	0.4-1.2	2-7	28%	25-50

Considering equations 6 and 8, we can already increase their range of application by adopting the modification:

$$Q_c = Q \times \frac{\sigma_c}{100} \qquad \textbf{(9)}$$

Thus we have:

$$V_p \approx \log Q_c + 3.5 \qquad \textbf{(10)}$$

and

$$M \approx 10 \times Q_c^{1/3} \qquad \textbf{(11)}$$

Eliminating Q or Q_c between equations 6 and 8, or between equations 10 and 11, we obtain:

$$M \approx 10 \times 10^{\frac{V_p - 3.5}{3}} \qquad \textbf{(12)}$$

Graphic use of the above equations is shown in Figure 7, where, through trial and error, approximate corrections for depth and matrix porosity are also given. Note that equation 12 relates to the mean value of quoted modulus. There are indications that even lower ranges can sometimes be experienced due to sample unloading and excavation disturbance effects at the *in situ* test site. The equation:

$$M(\min) \approx 3\, Q_c^{1/2} \qquad \textbf{(13)}$$

is the source of the tabulated values of M(minimum) given on the right hand side of Figure 7.

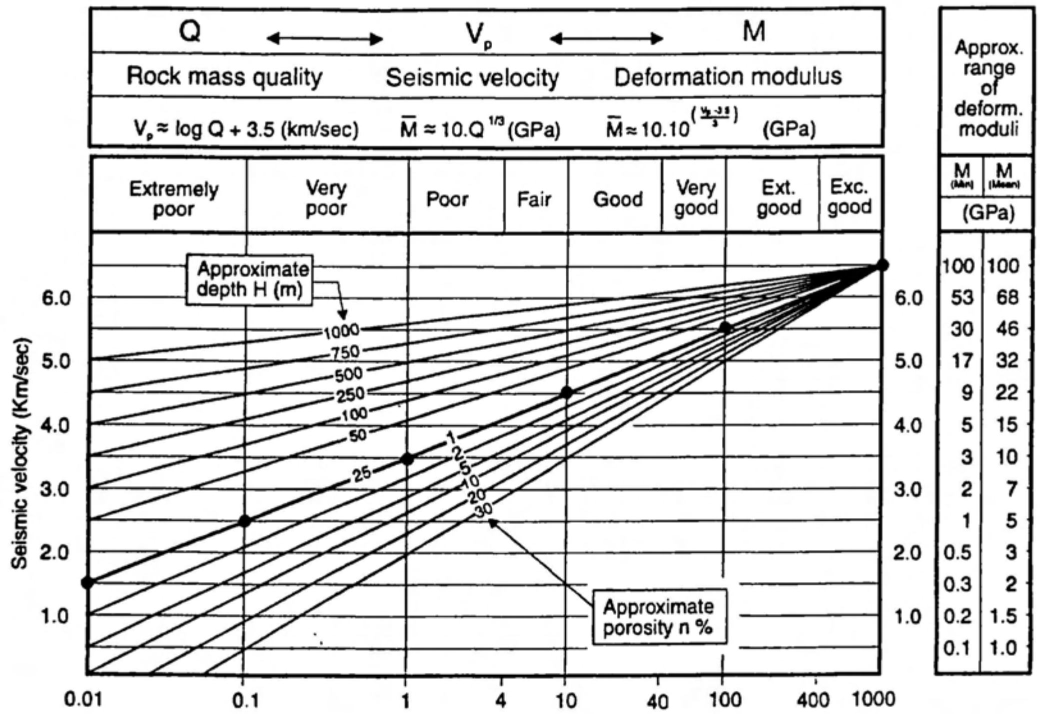
CASE RECORD EXAMPLES OF Q, V_p and M CORRELATIONS

1. **Japanese Tunnel** — **Rock type: Sandstone**

   Q = 2-4 (most frequent range) — σ_c = 50-75 MPa (most frequent range)

   Q_c = 1-3 (equations 7, 9) — Refer to Figure 7 for following:

Porosity (n%)	*Depth (m)*	*Correction (km/s)*		*Estimate of V_p (km/s)*	
If n = 1%	and H = 25m	none	∴	3.5-4.0 km/s	
Actual 5-10%	where 25m,	-0.7, -0.6	∴	2.8-3.4 km/s	(as measured)
Actual 5-10%	where 50m,	+0.6, +0.5	∴	3.4-3.9 km/s	(as measured)



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

Fig. 7 *Rock mass quality chart for estimating inter-relationships between Q, V_p, M, σ_c, n and H (σ_c = uniaxial compression strength of rock).*

Modulus range = 10-22 GPa (estimated) for 25m depth (Q = 2) and 50m depth (Q = 4) (see Figure 7 and equation 12).

2. Äspö Tunnel (Zedex Project) **Rock type: granite and diorite**

Q = 22-23 (mean for D+B and TBM tunnels σ_c (mean) = 195 MPa

Q_c = 43-44 (equations 7, 9) Refer to Figure 7 for following:

Porosity (n%)	*Depth (m)*	*Correction (km/s)*		*Estimate of V_p (km/s)*
If n = 1%	and H = 25m	none	∴	5.1-5.2 km/s
Actual 1% (est.)	where H = 450m,	+0.7, +0.8	∴	5.8-6.0 km/s
				(6.1, 6.3 km/s measured)

Modulus estimate = 58-68 GPa. A value of 60 GPa gave a good fit between measured and calculated deformations.

3. Channel Tunnel (UK) **Rock type: Chalk marl**

i Q = 8 (precedent study) (Barton and Warren, 1996)

Q = 9 (TML mean for all running tunnels km 20-24) σ_c (mean) = 6 MPa

Q_c = 0.48-0.54 (≈ 0.5) Refer to Figure 7 for following:

Porosity (n%)	*Depth (m)*	*Correction (km/s)*		*Estimate of V_p (km/s)*
If n = 1%	and H = 25m	none	∴	3.2 km/s
Actual 28%	if H = 25m,	-1.6	∴	1.6 km/s

Actual 28%	where H = 40m,	+0.4	∴	2.0 km/s	(as measured)

ii Q = 15 (TML mean for MST, km 24-30) σ_c (mean) = 6 MPa
Q_c = 0.9 Refer to Figure 7 for following:

Porosity (n%)	*Depth (m)*	*Correction (km/s)*		*Estimate of V_p (km/s)*	
If n = 1%	and H = 25m	none	∴	3.4 km/s	
Actual 28%	if H = 25m,	-1.4	∴	2.0 km/s	
Actual 28%	where H = 40m,	+0.5	∴	2.5 km/s	(as measured)

The range of 2.0-2.5 km/s shows excellent agreement with the offshore geophysics performed in the English Channel.

CONCLUSIONS

1. A literature review has demonstrated the difficulties involved in obtaining deformation modulus test data at sufficiently large scale to be comparable to the values obtained from back-analysis of tunnel measurements. Good estimates were however obtainable from earlier RMR and Q-system empirical equations, but these were limited to harder, jointed rocks.

2. A literature review of the effects of stress change and disturbance caused by tunnelling shows that similar trends exist between modulus change and seismic velocity change. The effect of depth, porosity and compression strength on seismic velocity and modulus are each demonstrated.

3. A method has been developed for relating a modified Q-value to deformation modulus and seismic velocity, with corrections for porosity and depth. The modified Q-value includes a seventh parameter. When the uniaxial compression strength is less than or more than 100 MPa, the Q-value is reduced or increased in proportion by the ratio of $\sigma_c/100$. Several examples of application of the new method with comparison to actual cases are given.

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