Integrated empirical methods for the design of tunnels, shafts and caverns in rock, based on the Q-system

Metodos empiricos integrados para el diseño de túneles, lumbreras y cavernas en roca, basados en el sistema Q

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RESUMEN: El sistema Q se ha convertido en un método ampliamente referenciado y utilizado para caracterizar las condiciones de los macizos rocosos y para ayudar en la selección del soporte temporal o permanente durante la construcción de túneles, lumbreras y cavernas. Su origen y uso fue inicialmente previsto para túneles y cavernas con revestimiento simple cuando barras de anclaje protegidos contra la corrosión y concreto lanzado de buena calidad son empleados en el soporte *permanente.* Esta es todavía una función ampliamente utilizada del sistema Q. El control del agua mediante el uso sistemático de inyecciones de alta presión es el método recomendado y más barato para asegurar excavaciones secas, en lugar del doble revestimiento incluyendo manta de drenaje, membrana y concreto final que se utiliza ampliamente en muchos países a pesar de su alto costo; incluso cuando se utiliza este ultimo método, el sistema Q todavía puede ser empleado para la caracterización de la roca y para la selección del soporte que se necesita para asegurar la estabilidad hasta que el revestimiento de concreto se ha completado. El sistema Q ha sido utilizado con éxito en esta forma durante al menos 20 años en Hong Kong en la red de túneles de carreteras y túneles urbanos excavados en tobas intensamente fracturadas y algunas veces granitos altamente meteorizados. Características adicionales de este método empírico han indicado una correlación de Q con velocidades sísmicas, módulo de deformación, deformación de túneles o cavernas, permeabilidad y estimaciones de la cohesión y resistencia a fricción de macizos rocosos. Esto indica que los seis órdenes de magnitud en la escala de calidad del sistema Q reflejan la variabilidad de la naturaleza de una manera realista, ya que las formulas empíricas son muy simples en comparación con las expresiones algébraicas muy largas utilizadas actualmente en la modelización del medio continuo.

ABSTRACT: The Q-system has become a widely referenced and widely used method for characterizing rock mass conditions, and for assisting in the selection of temporary support or permanent support for tunnels, shafts and caverns. Its origin and its originally intended use was for single-shell tunnels and caverns where corrosion protected rock bolts and, good quality fiber-reinforced shotcrete, form the final *permanent* support. This is still a widely used function of Q. Water control using high pressure systematic pre-injection is the preferred and cheaper method of ensuring dry excavations, in preference to cost-driving and schedule-driving double-shell concepts involving drainage fleece, membrane and final concrete. However this latter method is widely used in many countries, despite its high cost, and in this case the Q-system can be used for the rock mass characterization and for the selection of temporary support, which is needed to ensure stability until a concrete lining has been completed. Q has been used in this way with success for at least 20 years in Hong Kong's extensive metro and road tunnel network in intensely jointed tuffs and sometimes deeply weathered granites. Additional features of this empirical method have indicated correlation of Q with seismic velocity, deformation modulus, tunnel or cavern deformation, permeability, and estimates of the cohesive and frictional strength of rock masses. This suggests that this six orders of magnitude quality scale is reflecting the variability of nature in a realistic way, as the empirical formulæ are very simple compared to long algebraic expressions in use in current continuum modelling.

1 INTRODUCTION

From the outset the focus in this paper will be on sound, simple empiricism, that works because it reflects practice, that can be used because it can be remembered, and that does not require black-box software solutions.

With 3,500 km of hydro-power related tunneling, about 180 underground power houses, and some 1,500 km of road and rail tunnels, it has always been necessary to construct economic tunnels (and power-houses and storage caverns) in Norway. The Q-system development from 1974 always reflected this, and 50% of initial case records were from Norwegian and Swedish hydro power projects, with *fifty different rock types* in the first 212 case records. An update with 1,050 new (independent from Q) case records, was mostly from road tunnel projects, where higher levels of support were used.

Contrary to popular belief, few cases from the Pre-Cambrian and mostly high quality bedrock could be used as case records, unless they were challenging fault-zone or shear-zone cases. One cannot develop a rock mass classification system from cases of 'no support needed', since Q is often in the range 10 to 100 in these basement rocks. Yet some believe Q cannot be used 'in their country' due to all the granitic gneiss that they imagine accounts for the Q-system development. This misunderstanding is unfortunate.

The basic Q-system reinforcement and support components B + S(fr) meaning systematic rock bolting, and (since the 1993 update) fiber-reinforced shotcrete, were developed from challenging conditions, with weathered rock, clay-fillings, shear zones and fault zones,

sometimes with swelling clay, like montmorillonite, due to hydrothermal alteration. However, Norwegian (and world) records for drill-and-blast, single-face advance rates from the last decade, of > 160 m/week, and more recently > 170 m/week, and even >100 m/week as a whole-project average, for *single-face* drill-and-blast progress, of course give evidence of plenty of good rock, but in fact are evidence of well-proven methods by the contractors.

Cycle-times may be <5 hours in the best quality rock masses. However, the record 5.8 kilometers in 54 weeks with drill-and-blast from one face where a best week of 176 meters was achieved, was in coal-measure sedimentary rocks requiring significant bolting and shotcreting. The advantages of B+S(fr) compared to the much slower temporary support components of NATM, like steel-sets or lattice girders, bolts and mesh reinforced shotcrete S(mr) need to be emphasized, as they make tunneling an unnecessarily costly and slow process.

Steel sets and lattice girders provide psychological support. However they are the most deformable aspects of tunnel support until they begin to take load. They are seldom thought of in this way, but unfortunately it is true, and increased tunnel deformation is inadvertently 'encouraged'. In the end the presumed stiff support of a final concrete lining is actually needed.

2 THE WIDE RANGE OF Q IS REALISTIC

It is appropriate to start by illustrating contrasting rock mass qualities and their characterization. The classic Sugar Loaf mountain in Rio de Janeiro shown in Figure 1 is clearly at the high end of the rock mass quality scale. It requires a cable car for access, and contrasts greatly with the fault zone in Figure 2, also in Brazil, which required successive boat trips through flooded sections of the tunnel. There were five such collapsed zones.

Figure 3 (top) shows a core box from a regional fault zone at a project which took 15 years to complete. The massive core shown in the lower photograph is from a project which may not be *started* for at least 15 years. The first should already have been passing high-speed trains, the other accepting high-level nuclear waste, some indeterminate time in the future. They are both from the same country (Sweden) and may have six orders of magnitude contrast in Q-value.

Shear strength and deformation modulus in these two cases would also vary by orders of magnitude. In contrast, the quality descriptors RMR and GSI would suggest different ratings of about 5 and 95 in the illustrated *in situ* and core-box cases, which intuitively speaking do not seem to be adequate to represent the huge contrast.

When using the scale 0.001 to 1000 compared to 5 to 95 to correlate this range of qualities with engineering parameters of interest, it is found that a logarithmic scale is more in tune with nature's logarithmic variation.

Figure 1. Sugar Loaf mountain in Rio de Janeiro is clearly an example at the top end of the rock mass quality scale. (Q ≈ 100/0.5 x 4/0.75 x 1/1, i.e. >1000). Similar conditions have been known to slow TBM dramatically, yet for drill-and-blast it is ideal as no support is actually needed.(See ratings in Appendix).

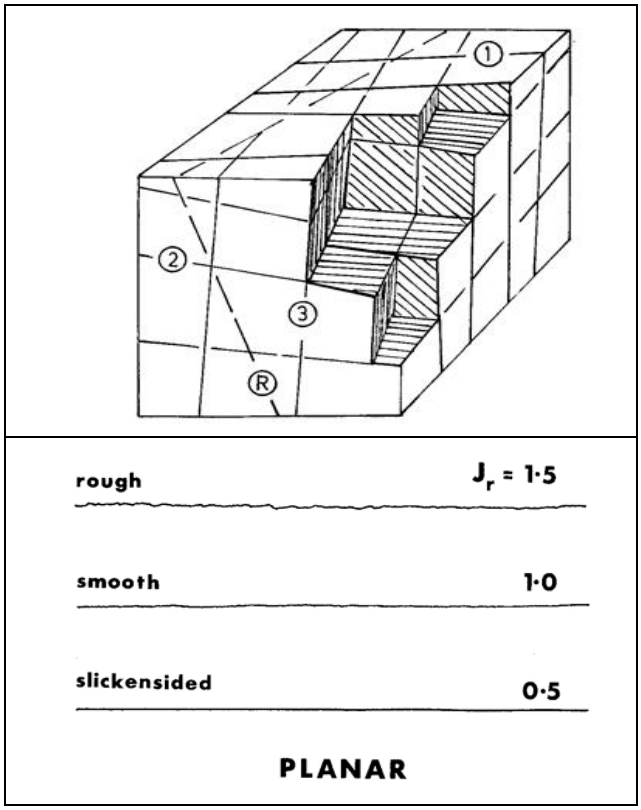
Figure 2. This fault zone in a tailrace tunnel is at the lower end of the rock mass quality scale. (Q ≈ 10/20 x 1/8 x 0.5/20, i.e. < 0.001). As may be noted, all the Q-parameters: RQD, Jn, Jr, Ja, Jw and SRF are adversely affected by the faulting, and there will be a tendency for greatly increased deformation (prior to failure).

3 OVERBREAK FROM THE Q PARAMETERS

A common dilemma for consultants and owners is whether a contractor's claim about overbreak being 'unavoidable' is true, or whether more effort in blast-hole layout and perimeter-hole charging, could help to solve excessive overbreak. In different circumstances both arguments hold.

Figure 3. The obvious contrasts of these two core boxes suggest orders of magnitude differences in quality, modulus and strength.

It is clear that there are elements of the structural geology that can be a genuine 'excuse' for a contractor, while in other cases the lack of 'half-rounds' seems to be blasting induced. A convenient way to distinguish the two is by reference to diagrams showing the number of joint sets (Jn in the Q calculation), and the joint roughness (Jr in the Q calculation). Figure 4 illustrates some of the logic.



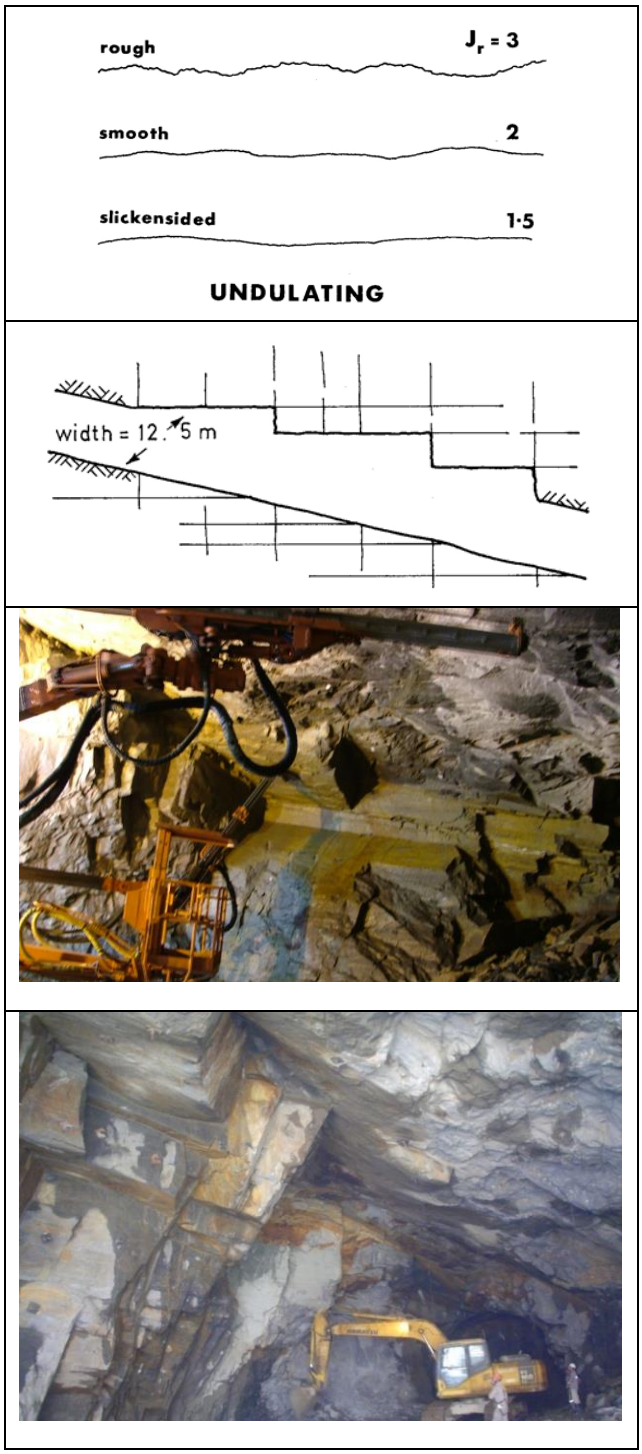


Figure 4. It has been found that overbreak is extremely likely to occur despite a contractor's efforts with careful blasting, if the most frequent value of the ratio Jn/Jr ≥ 6, i.e. 6/1, 9/1.5, 9/1.0, 12/2, 12/1.5, 12/1.0, 15/1.5. Visible half-rounds and lack of overbreak will tend to be found when Jn/Jr < 6, such as 3/1, 4/1, 6/1.5, 9/2, 9/3, 12/3, 15/3 since the lack of block structures or dilatant joint roughness prevent its occurence. All half-rounds (and virtually no overbreak) will appear with Jn/Jr = 2/3, or 2/4 which would be typical of a massive granite. (The definitions of the various ratings can be seen in the Appendix Q-histogram).

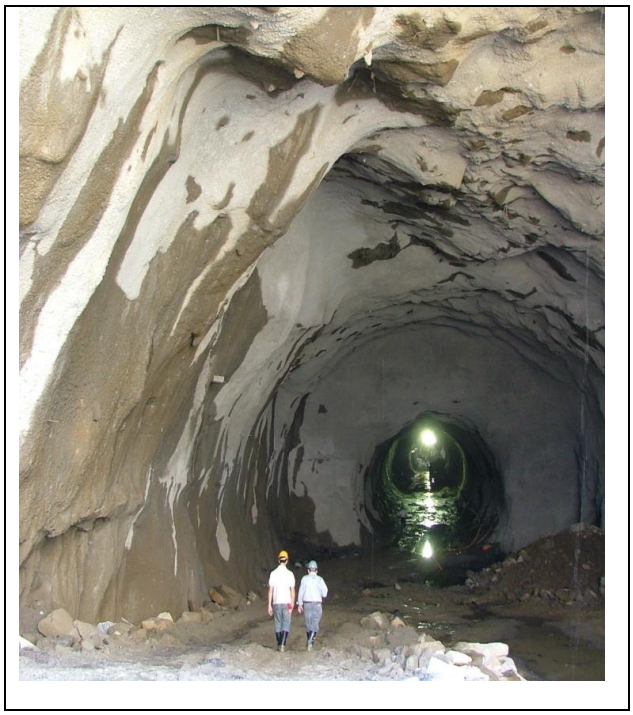


Figure 5. Here the excessive overbreak due to structure causes considerable increases in the volume of shotcrete, and even larger increases in the volume of concrete, if double-shell (NATM) is to be the final stage of station cavern development.

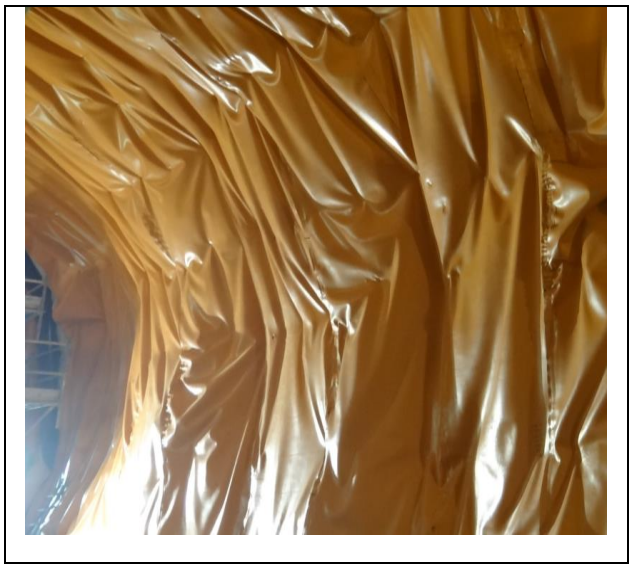


Figure 6. Excessive overbreak is also a problem that affects the fixing and welding of the drainage fleece and membrane in the case of double-shell (NATM-style) tunnels. If a rock mass has characteristics such as Jn = 9 (three joint sets) and Jr = 1 or 1.5 (smooth planar or rough planar joints), there may be a risk of overbreak that effects the economy of the project, as the membrane is also more difficult to construct, and damage from concrete pressure threatens subsequent leakage if not formed, as here, as a sufficiently 3D surface, which is more time-consuming. Concrete volumes will be far higher than 'designed' and thermal stresses may cause cracking, if no reinforcement.

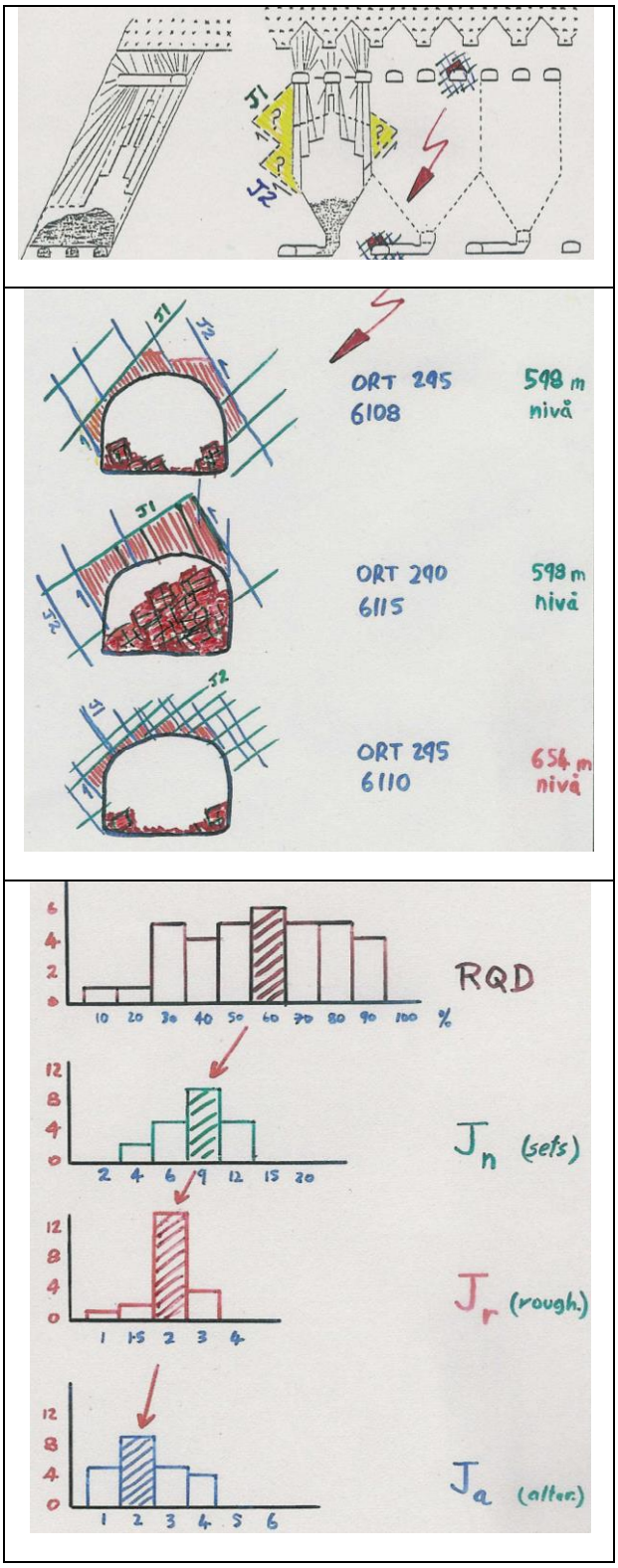


Figure 7. In the case of these long-hole drilling galleries which were used to develop a series of mining stopes, the excessive overbreak observed in some cases was due to adverse combinations of Jn/Jr. The most frequent ratio 9/2 was no problem, and was indeed < 6, as shown by these Q'-parameter histograms. Barton, 1987.

4 RELATING Q TO VELOCITY AND MODULUS

Since there is a limit to how many boreholes can be drilled, how many cores can be logged, and how many permeability tests can be performed, it is useful to have alternative ways of estimating and extrapolating these 'point sources' of information. The Q-value helps here.

We will start by looking at correlation between velocity and measures of quality, with Sjøgren et al. (1979) as a *very* useful starting point for the case of investigations in hard rock, using seismic profiles (totalling 113 km) and local profile-oriented core logging results (totalling 2.85 km of core).

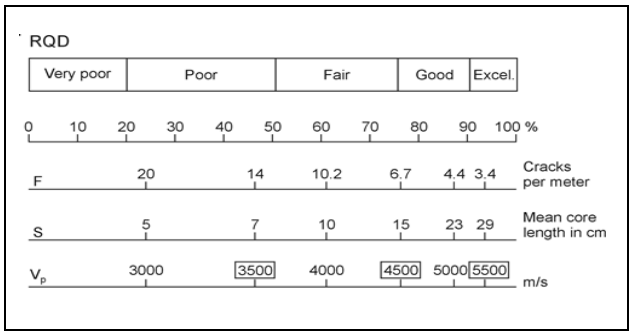


Figure 8. Hard rock, shallow refraction seismic. Sjøgren et al. (1979) combined 113 km of seismic profiles and 2.85 km of core logging to derive these mean trends for hard rocks like granite, gneiss, porphyry, quartzite.There was mostly limited weathering.

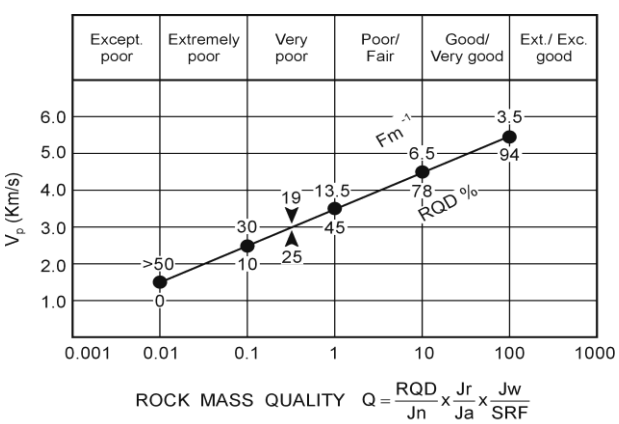


Figure 9. Hard rock, shallow seismic refraction mean trends from Sjøgren et al. (1979). The Q-scale was added by Barton (1995), using the hard rock correlation $V_p \approx 3.5 + \log Q$. By remembering Q = 1: Vp ≈ 3.5 km/s, and V_p = 3 km/s: Q ≈ 0.3, the Q-Vp approximation to a wide range of qualities is at one's fingertips (e.g. for hard, massive rock: Q = 100: Vp ≈ 5.5 km/s, and for Q ≈ 30 V_p = 5 km/s).

A more general form of the relation between the Q-value and P-wave velocity is obtained by normalising the Q-value with the multiplier UCS/100 or $\sigma_c/100$, where the uniaxial compressive strength is expressed in MPa. This is shown in Figures 10 and 11.

5 RELATING Q TO ROCK MASS PERMEABILITY

Here we move into more difficult Q-correlation territory, since there are potential problems of flow-channels that have suffered erosion or solution-effects, and there are also joint sets that may be clay-sealed, therefore having both low permeability *and* low Q-value.

For hard, low porosity, jointed rock masses without clay, the approximate Lugeon scales shown in Table 1 may have some practical merit, when 'out in the field' and also away from colleagues who make a living from permeability measurements.

Table 1. A set of inter-related approximations that are useful when assessing results in the field. Lack of similarity to these clay-free norms can indicate chanelling, or clay-filling of joints.

Q-value	0.1	1	10	100
Lugeon	10	1	0.1	0.01
K ≈ m/sec	10^{-6}	10^{-7}	10^{-8}	10^{-9}
V_P km/s	2.5	3.5	4.5	5.5

Table 2 The two versions of 'Q-permeability' estimation. It should not need to be emphasised that both are approximate. Both are presently based on limited test data.

Hard, jointed, clay-free rock masses $L \approx 1/Q_c$ (1 Lugeon ≈ 10^{-7} m/s ≈ 10^{-14} m^2) Q_c = RQD/Jn x *Jr/Ja* x Jw/SRF x $\sigma_c/100$
General case, with depth/stress allowance, and consideration of joint wall strength JCS $\mathbf{Q}_{\mathbf{H2O}}$ = RQD/Jn x *Ja/Jr* x Jw/SRF x 100/JCS $K \approx \dfrac{2}{1000 \times Q_{H_2O} \times 10^{5/3}}$

Table 3. Example of Q_{H2O} estimation. Note the more logical inversion of the ratio Jr/Ja to the form Ja/Jr in case of clay filling.

Clay-bearing, well-jointed rock at 100 m depth, with a low assumed JCS of 10 MPa due to low UCS of 15 MPa.
Regular Q-value = 50/9 x 1.5/4 x 0.66/1 = 1.4, i.e. 'poor'
$Q_{H_2o} = \frac{50}{9} \times \frac{4}{1.5} \times \frac{0.66}{1} \times \frac{100}{10} = 98$
The estimated result is K ≈10^{-8} m/s (at 100 m depth) (Quite low permeability due to clay coatings, and compressible joint walls, despite the well-jointed nature of this Q = 1.4 rock mass).

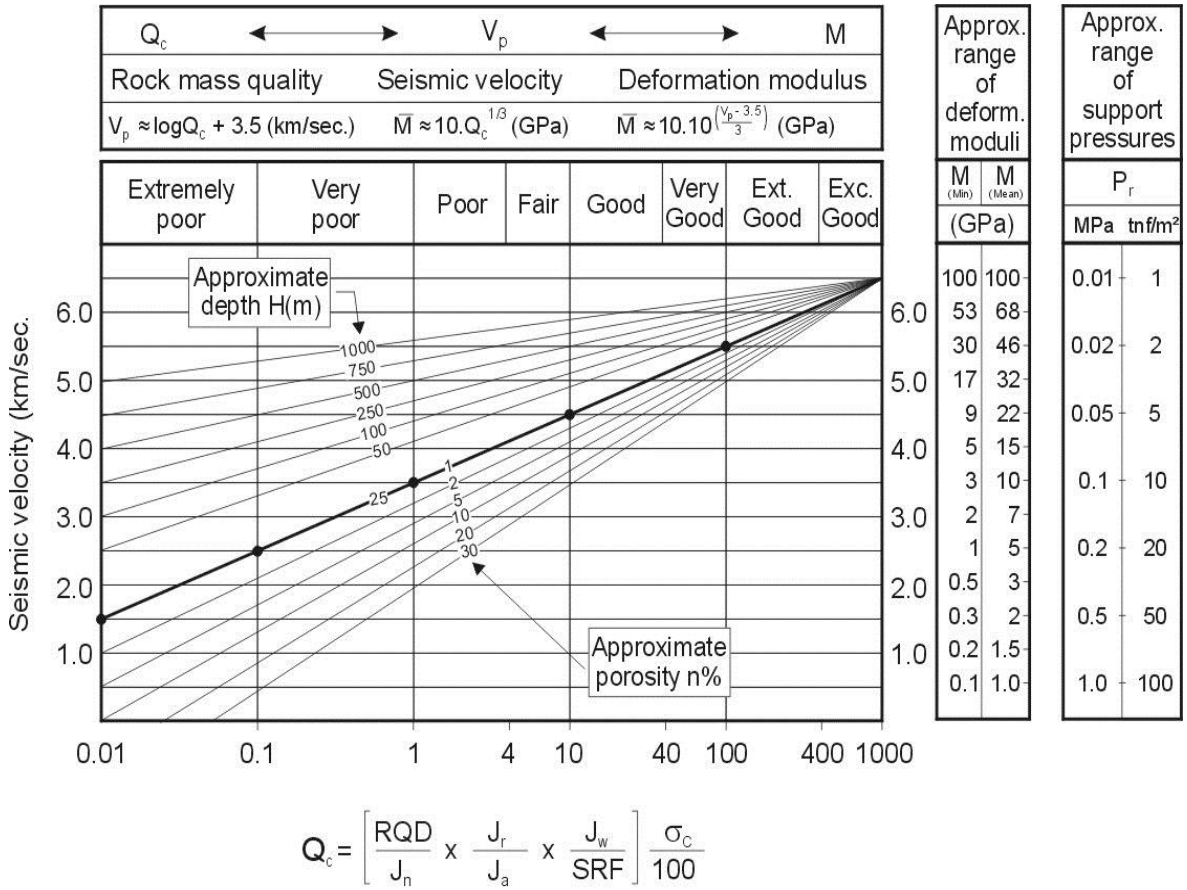


Figure 10. The 'central diagonal' is the same as the sloping line given in Figure 9, and this applies to nominal 25-30m depth shallow seismic refraction results. In practice the nominal 1% (typical hard rock) porosity would be replaced by increased porosity if deeply weathered, and the more sloping lines below the 'central diagonal' would apply if the Q-value had reduced. The less inclined lines representing greater depth (50, 100, 250m etc) were derived from deep cross-hole seismic with Q-logging of the respective cores (Barton, 2002). Note the inverse nature of (static) deformation moduli and support pressure shown in the right-hand columns. These derivations are described in Barton, 1995 and Barton, 2002.

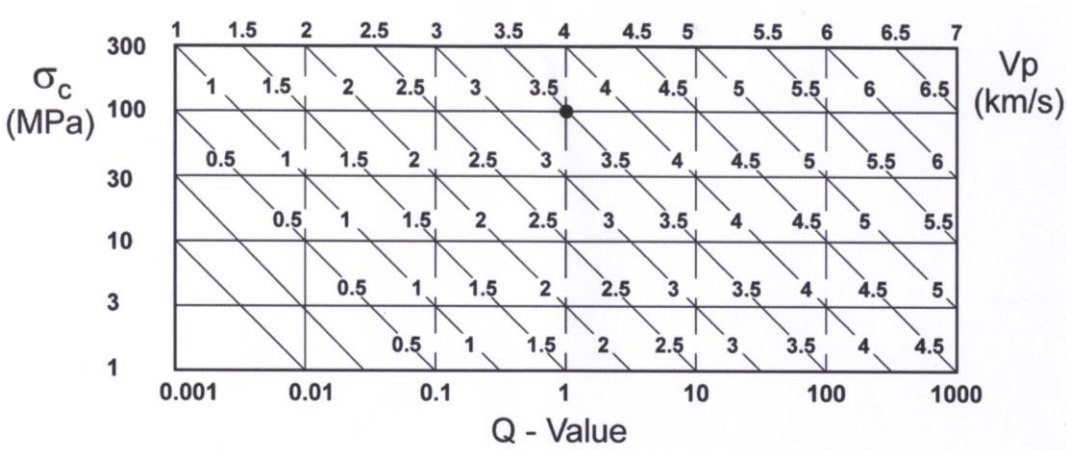


Figure 11. A nomogram representing the shallow refraction seismic 'central diagonal', with UCS or σ_c given instead on the left axis, and the regular Q-value given along the lower axis. Note: When Q = 1 and σ_c = 100 MPa, then V_P = 3.5 km/s. This is an easy to remember guide. When conditions are poor and Q = 0.1, and if σ_c = 30 MPa, then V_P = 2 km/s. In good quality massive rock, when Q = 100 and σ_c = 150 MPa, then V_P = 5.6 km/s. These are empirical but also intuitive results.

Note that the Barton et al. 1974 support pressure formulation, and the Barton, 1995 deformation modulus formulation (shown in the right-hand side columns of Figure 10) suggest inverse proportionality between support pressure and deformation modulus, This is logical, but the simplicity is nevertheless surprising. See Barton (2002) for further discussion, and note that the simplicity applies specifically to the mid-range case of Jr =2 (smooth undulating rock joints: refer to Figure 4). For other values of Jr this 'perfect' inverse proportionality will be modified somewhat, but the general trend remains, and has joint-dilation dependent origins.

6 RELATING Q (CC, FC) TO ROCK MASS FAILURE

As summarised in the series of figures below, the conventional way to analyse the stability and possible 'plastic zone' development around an overstressed tunnel have a lot of problems in linking the modelling to reality.

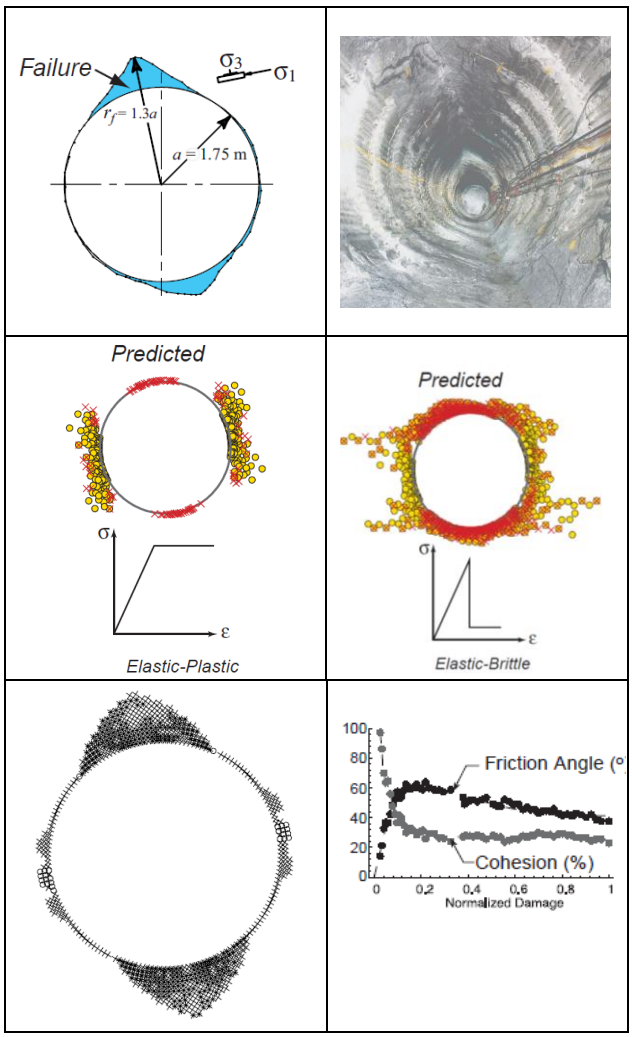


Figure 12. Pairs of diagrams which illustrate: 1) Reality, 2) Conventional continuum modelling with Mohr-Coulomb, 3) Degrading cohesion while mobilizing friction in FLAC.

This is because of the non-empirical *a priori* assumptions which lie behind Mohr Coulomb (and Hoek-Brown) constitutive 'modelling' of rock mass failure. Figure 12 shows surprisingly unrealistic distributions of 'shear' (yellow circles) and 'tensile' failure (red crosses), following important research reported by Hajiabdolmajid, Martin and Kaiser, 2000. The problem is similar when using Hoek-Brown. The error is due to the long-standing addition of cohesion (c) and frictional strength (σ_n tanφ) when trying to model rock mass failure. As already pointed out by Müller, 1966 long ago, the cohesive component (rock 'bridges') breaks first at smaller strain, followed by the mobilization of frictional strength, which fortunately remains for large displacements.

Attempts to model 'break-out' phenomena such as those illustrated in Figure 12 are not especially successful with standard Mohr Coulomb or Hoek Brown failure criteria, because the *actual phenomena* are not following our long-standing *a priori* assumption of *'c plus σ_n tan φ'*. The reality is degradation of cohesion at small strain and mobilization of friction (first towards peak, then towards residual) which occur at larger strain.

The demonstrated shortcomings of continuum modelling with *'c plus σ_n tan φ'* shear strength assumptions, should have alerted our profession for change already more than ten years ago, but deep-seated beliefs or habits are traditionally hard to change. Why are we *adding* 'c and σ_n tan φ' in 'continuum' models, making them even poorer representations of the strain-and-process-sensitive reality?

Input data obtained via Hoek and Brown and GSI formulations that obviously ignore such complexity, nevertheless consist of remarkably complex algebra (e.g. Table 4). Rock masses actually follow an even more complex progression to failure, as suggested in Barton and Pandey, 2011, who recently demonstrated the application of a similar *'c then tan φ'* modelling approach, but applied it in FLAC 3D, for investigating the behaviour of multiple mine-stopes in India.

A further break with convention was the application of peak 'c' and peak 'φ' estimates that were derived directly from mine-logged Q-parameters, using the CC and FC parameters suggested in Barton, 2002. For this method, an estimate of UCS is required, as CC (cohesive component) and FC (frictional component) are derived from separate 'halves' of the formula for $Q_c = Q \times \sigma_c / 100$.

The Q-value (or Qc) seems to consist of the product of the *cohesive strength* (the component of the rock mass requiring shotcrete or mesh or concrete support), and the *frictional strength* (the component of the rock mass requiring rock bolting to compensate for lower friction).

$$CC = \frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100} \qquad FC = \tan^{-1}\left(\frac{Jr}{Ja} \times Jw\right)$$

These two expressions are obtained by cutting the formula for Q_c into two halves. Low CC requires S(fr) while low FC requires bolting as illustrated in Figure 13.

RQD	J_n	J_r	J_a	J_w	SRF	Q	σ_c	Q_c	*FC°*	*CC* MPa	V_p km/s	E_{mass} GPa
100	2	2	1	1	1	100	100	100	63°	50	5.5	46
90	9	1	1	1	1	10	100	10	45°	10	4.5	22
60	12	1.5	2	0.66	1	2.5	50	1.2	26°	2.5	3.6	10.7
30	15	1	4	0.66	2.5	0.13	33	0.04	9°	0.26	2.1	3.5
10	20	1	6	0.5	5	0.008	10	0.0008	5°	0.01	0.4	0.9

Low CC –shotcrete preferred **Low FC – bolting preferred**

Figure 13. The division of the formula for Q_c (= Q x UCS/100) into two halves, symbolically cutting the formula with scissors as no inversion or changes are required, gives realistic-looking values of 'c' and 'φ' as seen in the above table for five successively decling rock mass qualities (massive rock declining to completely weathered rock). This perhaps suggests that original case records, which demanded successive adjustments of the ratings to link Q with shotcrete and/or bolting, were the result of a deliberate choice of mainly bolting (cases with low FC) or mainly shotcrete (cases with low CC). These options are seen in the 'conditional factors' given in Barton et al., 1974, where relative block size (RQD/Jn) or frictional strength (Jr/Ja) were the means of choosing one or the other dominant type of support. The usually combined 'B+S(fr)' solution seen in today's Q-support chart (Figure 14) represents a simplification in relation to the earlier choices of dominant B or S(mr) mesh (or both combined) reinforcement and support.

It is of interest, in view of the very poor result achieved with the current habit of using GSI and the algebra-rich Hoek-Brown criterion, to compare the CC and FC simplicity with HB complexity. If forced by the scale of the problem to do continuum modelling, then if chosen, CC and FC must be used in the form 'c *then* $\sigma_n \tan\varphi$'.

Table 4. Contrasts in simplicity and complexity: Q and H-B.

FC	$"\phi" \approx \tan^{-1}\left(\frac{J_r}{J_a} \times \frac{J_w}{1}\right)$
	$\phi' = a\sin\left[\frac{6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}{2(1+a)(2+a)+6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}}\right]$
CC	$"c" \approx \left(\frac{RQD}{J_n} \times \frac{1}{SRF} \times \frac{\sigma_c}{100}\right)$
	$c' = \frac{\sigma_{ci}\left[(1+2a)s+(1-a)m_b\sigma'_{3n}\right]\left(s+m_b\sigma'_{3n}\right)^{a-1}}{(1+u)(2+a)\sqrt{1+\left(6am_b\left(s+m_b\sigma'_{3n}\right)^{a-1}\right)}\Big/\left((1+a)(2+a)\right)}$

7 CONVERSION BETWEEN Q AND RMR

There is widespread use of Bieniawski's RMR in many countries, often in parallel with Q, so it is appropriate to address possible inter-relationship between the two methods of rock mass description. As shown in Figure 14, the use of the 'log' method as opposed to the more frequently quoted 'ln' method, gives a more tangible range of RMR in relation to the Q-scale, avoiding the negative values that occur below Q = 0.01.

While there are admitted pitfalls when attempting to utilize Q to RMR conversion, due to some serious differences in structure and parameter weightings between the two systems, it is nevertheless considered that the advantages may outweigh the disadvantages. For this reason, both stand-up time estimates from Bieniawski (1989), and deformation moduli trends with RMR have been utilized, in an attempt to add to the tools available. Since we are engineers, and not scientists, our craft is the ability to make realistic approximations, leaving decimal places on the calculator. The algebra in Table 4 does not seem to be in the spirit of rock engineering, and it is not based on data, so is *a priori* rather than *a posteriori*.

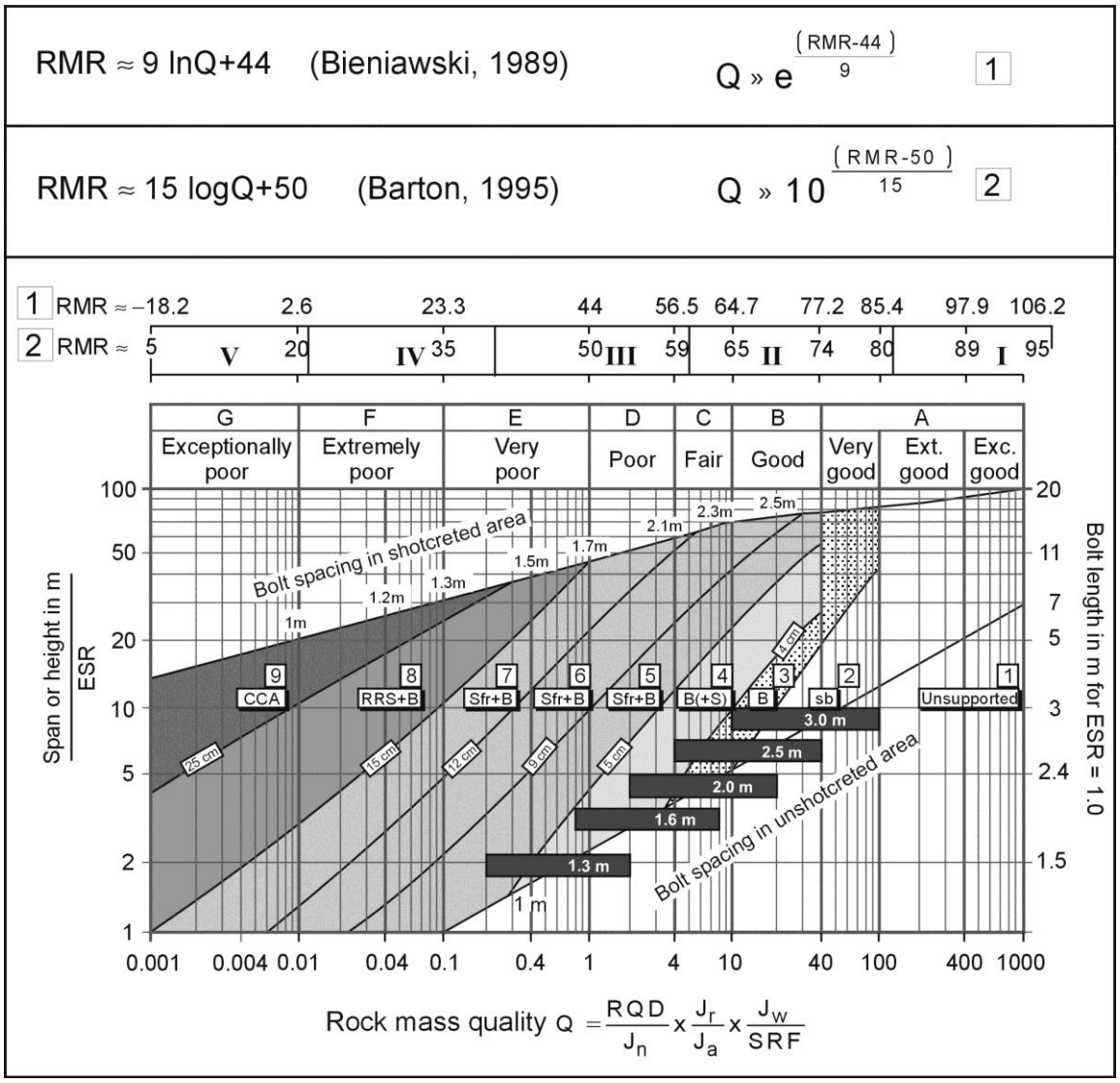


Figure 14 The Q-system tunnel (and cavern) support chart, which is presented here to show two possible methods of converting between Q and RMR. The $\log_{10}$ equation #2 avoids negative values of RMR when Q is very low. Nevertheles it is not 100% certain that this is desirable, as Q does cover a wider range of conditions than RMR, including high stress.

8 TEMPORARY SUPPORT SELECTION WITH Q

In Figure 15, the 'coordinates' of the cube, representing a portion of a 20 m span cavern with local Q = 3, would require B+S(fr) of 2.0 x2.0m c/c + S(fr) of 9 cm for *permanent support*. Each would be of high quality, meaning multi-layer corrosion protected (CT) bolts, and C45 MPa S(fr) with stainless steel fibers. But with the 1977 rule-of-thumb of *1.5 x ESR and 5 x Q*, which was actually intended as guidance to contractors (*i.e. not a temporary support procedure for consultants planning a concrete lining),* it would reduce to 'coordinates' of *B + S(fr) = 2.4 x 2.4 m c/c, L = 4 m long + S(fr) = 4 cm. (SPAN / (1.5 x ESR) = 13.3m, and 5Q = 15*, as shown by the larger arrow-head in Figure 15).

Some 25 years of practice using this method, for instance in hundreds of kilometers of metro, road and rail tunnels in Hong Kong, has proved its reliability in ensuring sufficient *temporary support*, pending the construction of the permanent concrete lining (with drainage fleece and membrane). While the writer prefers NMT to NATM, since it is ¼ to 1/5 as expensive, and faster, the reality is that many countries find the budget for permanent concrete linings. In which case a Q-based *'5Q+1.5ESR'* can be used to select the temporary support.

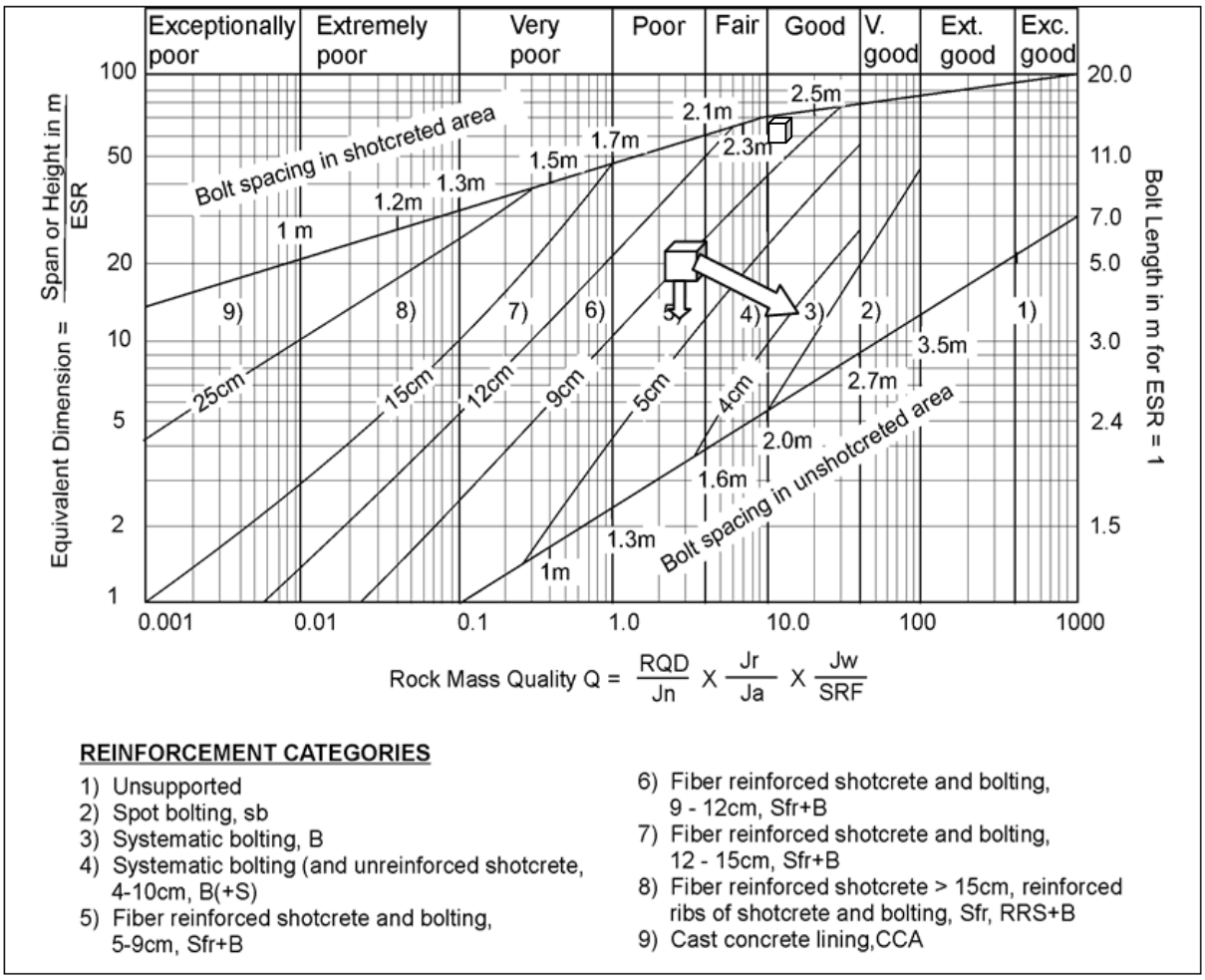


Figure 15. The Q-based tunnel and cavern support chart, developed from a total of 1,250 case records by Grimstad and Barton, 1993 (Norwegian conference) and Barton and Grimstad, 1994 (Austrian conference), was designed from the start for helping to select appropriate permanent support for single-shell NMT (Norwegian Method of Tunnelling) tunnels or caverns. This tunnelling philosophy was summarised in Barton et al., 1992 : a multiple-company description of engineering geological aspects, tunnel (permanent) support methods, contractual methods, and the equipment typically used. Table 6 on the next page summarises principal aspects. (Note single 'cube' at SPAN/ESR = 62m, Q = 11. This was Gjøvik cavern permanent support, but with added cable reinforcement, as recommended for large spans in Barton et al, 1974, 1977.

Table 5. The recommended ratings of ESR (for modifying effective SPAN) so that different degrees of safety (and cost) can be selected for different types of tunnels and caverns.

Type of Excavation		ESR
A	Temporary mine openings, *etc.*	*ca* 2-5
B	Permanent mine openings, water tunnels for hydropower (exclude high pressure penstocks), pilot tunnels, drifts and headings for large openings, surge chambers	1.6-2.0
C	Storage caverns, water treatment plants, minor road and railway tunnels, access tunnels	1.2-1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections	0.9-1.1
E	Underground nuclear power stations, railway stations, sports and public facilities, factories, major gas pipeline tunnels	0.5-0.8

Table 6. The guidelines for selecting temporary support and *differentiated arch and wall support* using the Q-system. These methods have been used extensivly for several decades, and have a successful and safe 'track record'. Barton et al., 1977.

1. Temporary Support	a) increase ESR to 1.5 × ESR b) increase Q to 5Q (arch) c) increase Q_W to $5Q_W$
2. Wall Support (based on modified quality Q_W for walls	a) select $Q_W = 5Q$ (when $Q > 10$ b) select $Q_W = 2.5Q$ (when $Q < 10$ c) select $Q_W = 1.0Q$ (when $Q < 0.1$

Note 1 Use total excavation height (H) for wall support design.
Note 2 Q is the general rock quality observed when inspecting the arch *or* walls of a tunnel. For local variations of rock quality (arch or wall), map locally and change support as appropriate. (Q_W is *not* the observed value of Q in a cavern wall.)

Table 7. The main characteristics of single-shell NMT (Norwegian Method of Tunnelling), as described by Barton et al., 1992, a report written with several colleagues from other Norwegian companies.

1. **Areas of usual application:**
   Jointed rock giving overbreak; harder end of uniaxial strength scale σ_c = 3 to 300 MPa)
   Clay bearing zones, stress slabbing
   Q = 0.001 to 10 or more
2. **Usual methods of excavation:**
   Drill and blast, hard rock TBM, machine excavation in clay zones
3. **Temporary rock reinforcement and permanent tunnel support may be any of following:**
   CCA, S(fr)+RRS+B, B+S(fr), B+S, B, S(fr), S, sb, (NONE) (see key below and their distribution in Figure 3, bottom).
   - temporary reinforcement forms part of permanent support
   - mesh reinforced shotcrete not used
   - dry process shotcrete not used
   - steel sets or lattice girders not used; RRS and S(fr) are used in clay zones and in weak, squeezing rock masses
   - Contractor chooses temporary support
   - Owner/Consultant chooses permanent support
   - final concrete linings are less frequently used; i.e., B+S(fr) is usually the final support
4. **Rock mass characterisation 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

CCA = cast concrete arches, S(fr) = steel fibre shotcrete,
RRS = reinforced ribs of shotcrete, B = systematic bolting,
S = shotcrete, sb = spot bolts, NONE = no support needed.

9 SHAFT SUPPORT SELECTION USING Q

The possibility of using the Q-system to help select appropriate support for shafts is a question raised frequently, in view of the huge data base and decades of successful practice behind Q-system selection of tunnel and cavern support. There is a relative lack of systematised case records concerning shafts in civil engineering. However the following has been advised for the case of shafts in civil engineering projects: as frequently used for *access* to drive tunnels, and subsequent use as *permanent ventilation shafts* of e.g. metro tunnels.

There are of course two principle dimensions to consider: the shaft diameter and the depth. (Sometimes shafts are roughly elliptical in the case of some metro ventilation shafts). Since a vertical wall is involved, as in the case of a high cavern wall in a hydroelectric machine hall (some are more than 80 m high), it is appropriate to select Q_W as a basis for support, following the guidelines in Table 5. The missing piece of information is the *shaft diameter*: the larger the diameter the less stable 'ring-effect' is achieved, so it is logical to use this dimension as 'H'. Temporary support can be based on $5Q_W$ providing that a minimum of safety protection (bolts and mesh) are applied in the case of stable rock masses with high Q-values. When Q is (locally) low as in the weathered top 20-30 m, heavier support will be automatic following either the temporary or permanent support guidelines.

10 VARIOUS EXAMPLES OF NMT and NONE-NMT

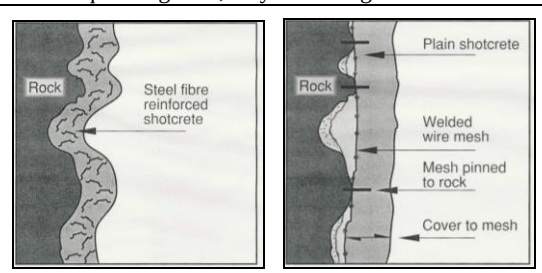
The following collection of photographs and diagrams may help to vizualise some typical features of NMT, in particular the choice of S(fr) – vastly superior and safer than S(mr) – and the choice of RRS (rib reinforced shotcrete arches) – greatly superior to steel sets or lattice girders, which allow too much deformation until they begin to take load. It is not good practice to combine elements of Q-system support with steel sets or lattice girders as these are initially not in *good* contact with the rock and subsequently they are too deformable in the early stages of rock deformation. In fact they enhance the need of a final concrete lining, since they will have allowed too much deformation. RRS on the other hand is a systematically bolted arch that is rapidly moulded to the existing tunnel profile, which may not be 'circular'. It requires no footing, though is firmly bolted at its base.

Pre-injected tunnel in shales. Robotic S(fr) up to face.

ROOF:	bolts	L = 2 + 0.15 B/ESR
	anchors	L = 0.40 B/ESR
WALLS:	bolts	L = 2 + 0.15 H/ESR
	anchors	L = 0.35 H/ESR

L = length in metres
B = span in metres
H = excavation height in metres
ESR = excavation support ratio

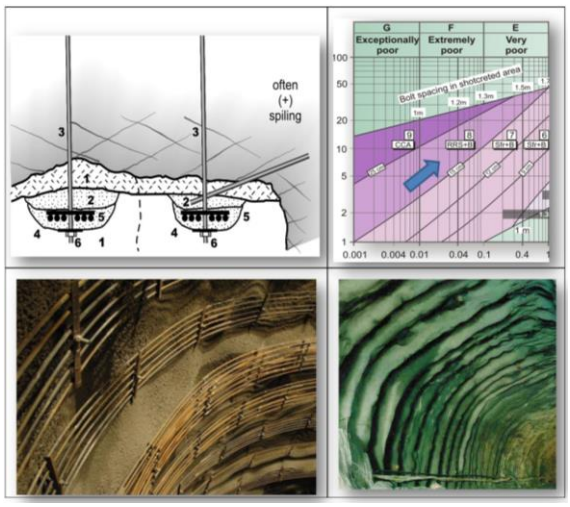
Recommended bolt and anchor lengths. In case of actually measured squeezing rock, may need longer bolts and anchors.



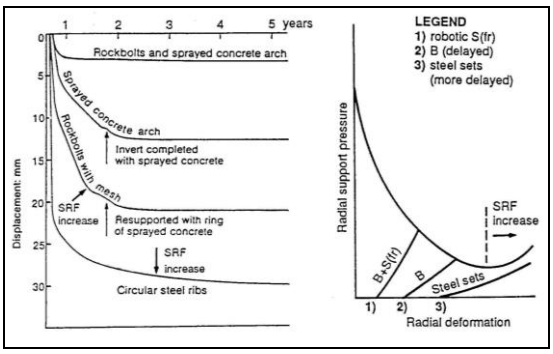
S(fr) prevents 'shadow' and does not corrode like S(mr).

Example of why S(mr) can be a dangerous support method.

Pre-injection using high pressures gives $k \leq 10^{-8}$ m/s.

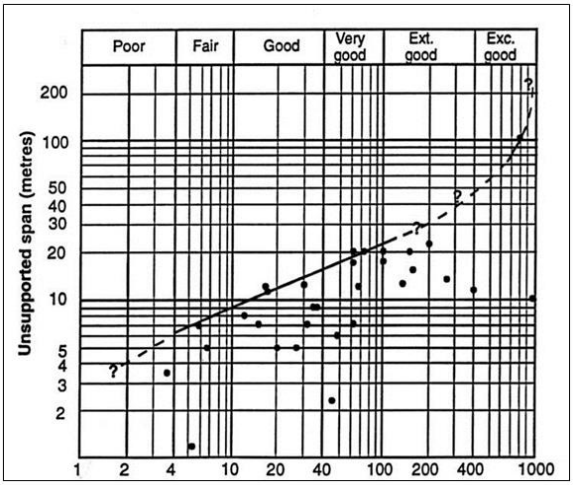


RRS for low Q is far superior to steel sets or lattice girders because it is in contact with the rock and is also bolted.



Experimental tunnel in mudstones demonstrating steel arch deformation in relation to minimal deformation for B+S.

Steel arches represent psychological but not real support.

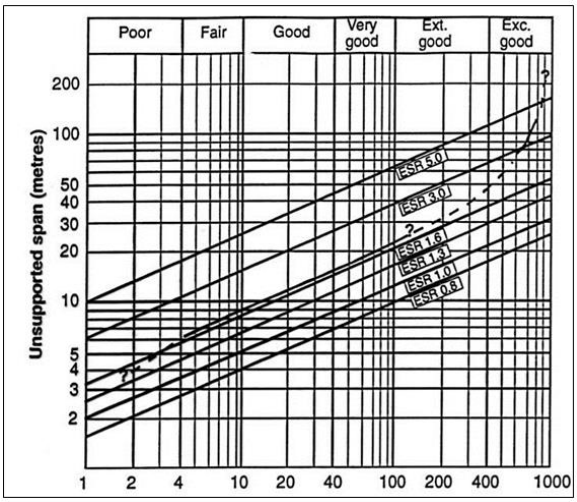


Permanently unsupported tunnels and caverns due to high Q.

PERMANENTLY UNSUPPORTED TUNNELS AND CAVERNS, AND THE WORKINGS OF THE ESR (degree-of-safety) NUMBER

General requirements for permanently unsupported openings

$$1.\ Jn \leq 9,\ Jr \geq 1.0,\ Ja \leq 1.0,\ Jw = 1.0,\ SRF \leq 2.5$$



Function of different ESR numbers for modifying safety.

If RQD ≤ 40, should have $J_n \leq 2$

If $J_n = 9$, should have $J_r \geq 1.5$ and RQD ≥ 90

If $J_r = 1$, should have $J_n < 4$

If SRF > 1, should have $J_r \geq 1.5$

If SPAN > 10 m, should have $J_n < 9$

If SPAN > 20 m, should have $J_n \leq 4$ and SRF ≤ 1

Additional conditional factors for permanent no support.

Figure 16 parts i to xii. Various characteristics of NMT, and items that are not NMT, are more efficiently illustrated by photographic examples and explanatory diagrams than by text. NMT = single-shell. NATM = double-shell. Cost difference usually 1:4 or 1:5, and reduced construction time.

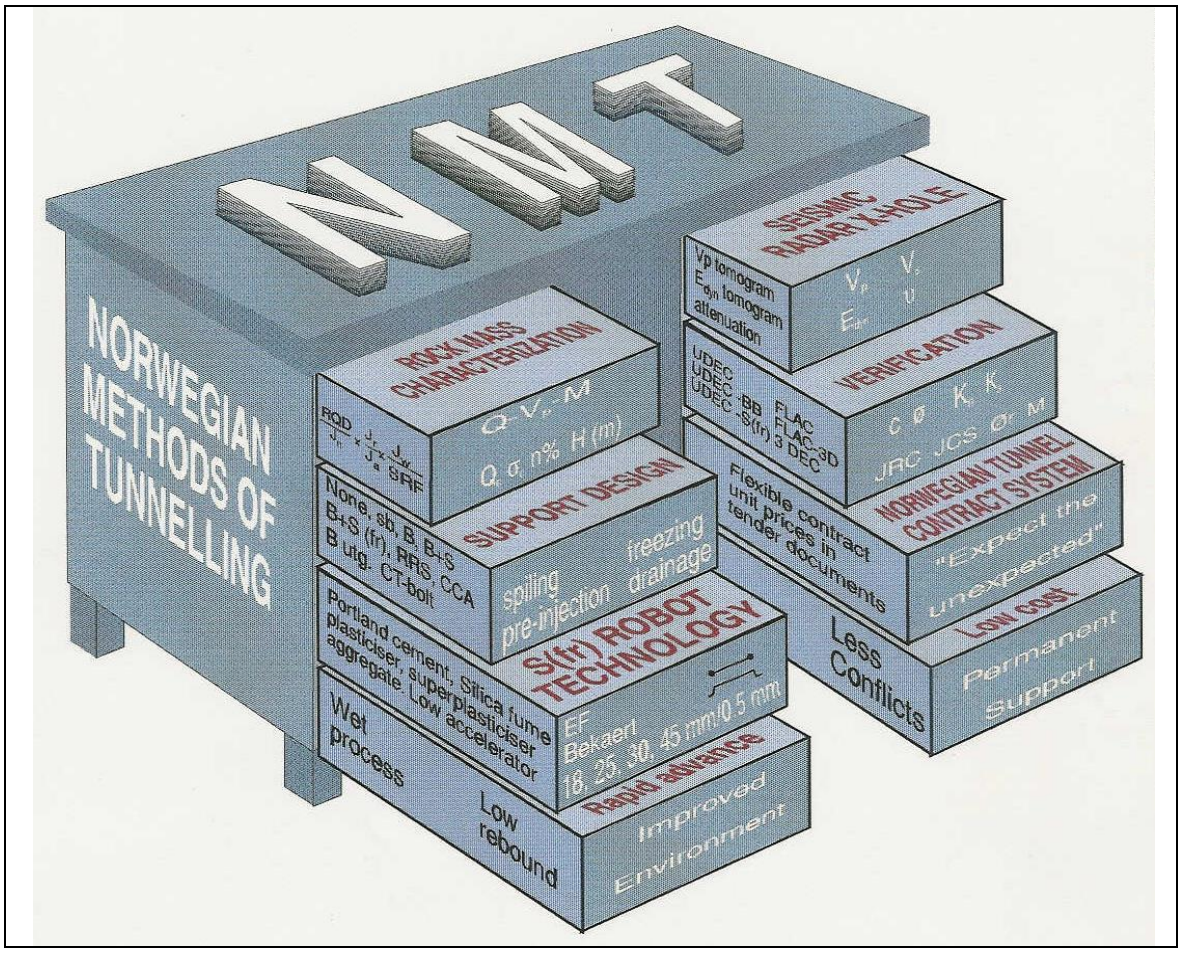


Figure 17. Exploration, characterization, design, and tunnel support selection, using single-shell NMT concepts, drawn in the form of an office desk. Note also the contractual aspects given in lower-right of diagram. From Barton and Itoh, 1994.

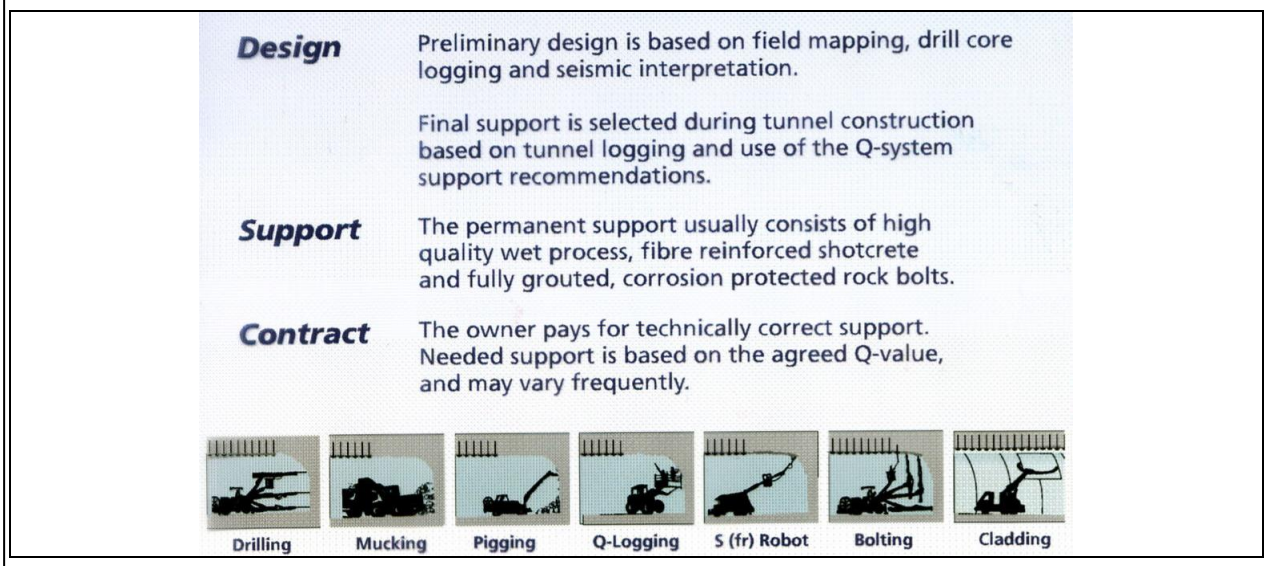


Figure 18. Summary of key elements of NMT, showing drilling, mucking, scaling, Q-logging, S(fr), B and cladding with PC-elements and outer membrane, if water inflow has not been prevented by high-pressure (5 to 10 MPa) pre-injection with MFC.

10 AN EXAMPLE OF CAVERN DESIGN AND DESIGN CHECKING: GJØVIK 62 m SPAN

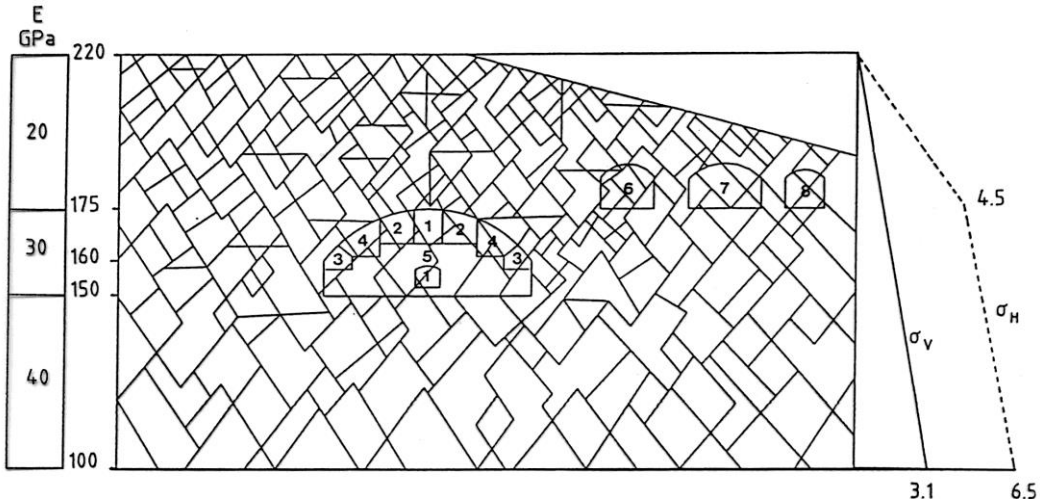
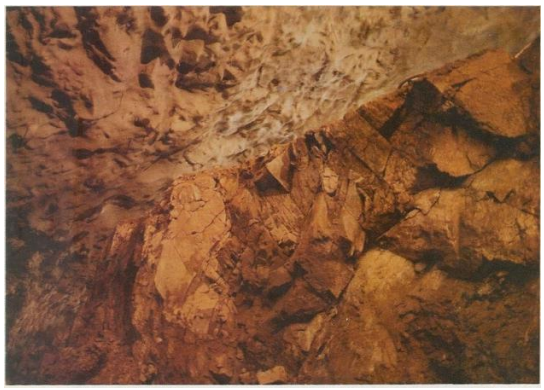
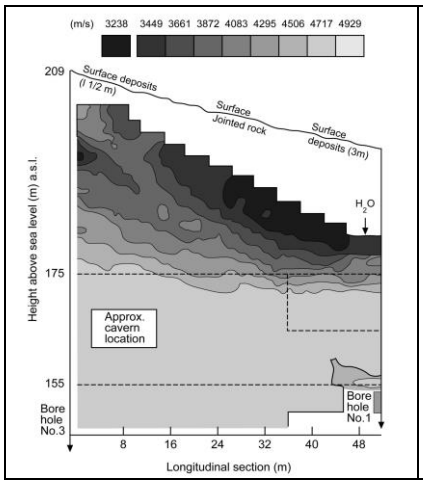
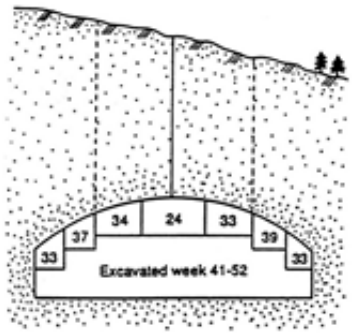


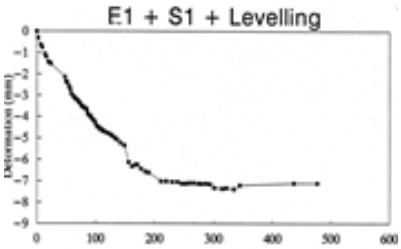
Figure 19. The assumed two-dimensional representation of jointing and boundary stresses (MPa) for the 62 m span Gjøvik Cavern, , and Q-value based deformation moduli increasing with depth as illustrated in Figure 10. (UDEC-BB input: JRC_0 = 7.5, JCS_0 = 75 MPa, φ_r = 27°, i = 6° (Patton larger-scale roughness). See Barton et al., 1994 for details.





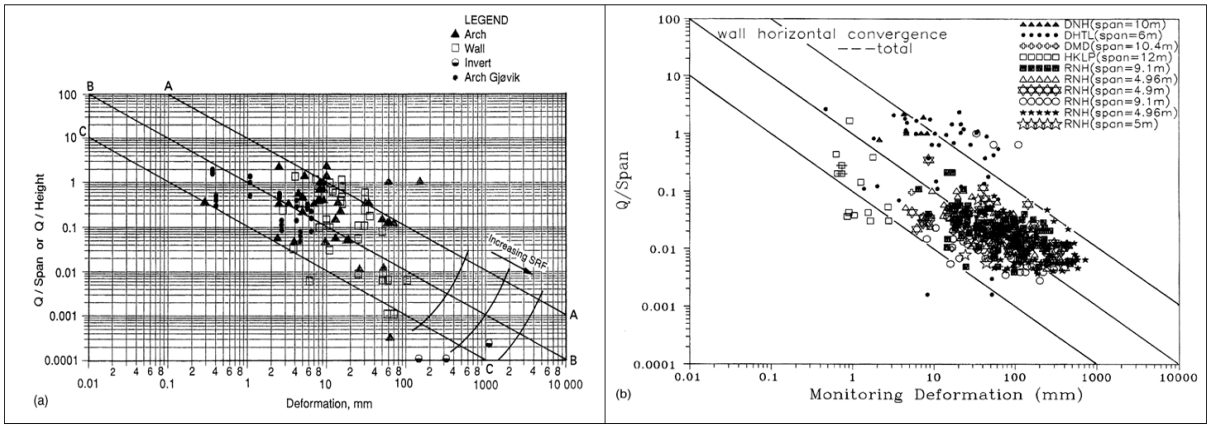


Seven MPBX from surface (E-series) and three MPBX from inside top heading (S-series)



This result for the central MPBX locations is typical, with total deformation stable at 7 to 8 mm, just as modelled with UDEC-BB

Figure 20. *Permanent support* was NMT single-shell (drained/drip-shield) and direct Q-system based: 10 cm S(fr) and 2.5 x 2.5 m c/c permanent (CT) bolts, L=6m. Due to the large span there was also a 5 x 5 m c/c spacing of twin-strand cable anchors, L=12m. Note maximum 1.0 to 1.5 m overbreak. (Jn/Jr was sometimes an adverse 12/2). Q-range in cavern arch was 2 to 30, $Q_{most\ frequent}$ = 10 to 12 (from four cores and from inside arch). RQD was most frequently 60 to 90 %. V_P from cross-hole seismic tomography was 3.6 – 4.9 km/s in the arch, consistent with $V_P \approx 3.5 + \log Q_C$. See Barton et al., 1994 for further details.



Original plotting method from Barton et al., 1994	Data from Chen and Guo (priv. comm.)

$\Delta = \frac{SPAN}{Q}$	(central trend of all data: approx)	$\Delta_v = \frac{SPAN}{100Q}\sqrt{\frac{\sigma_v}{\sigma_c}}$	(more accurate estimate)

MORE ACCURACY – BY CONSIDERING *'COMPETENCE FACTOR'*

STRESS/STRENGTH DIRECTLY, NOT JUST AS IN SRF

$$\Delta_v = \frac{SPAN}{100Q}\sqrt{\frac{\sigma_v}{\sigma_c}} \qquad \Delta_h = \frac{HEIGHT}{100Q}\sqrt{\frac{\sigma_h}{\sigma_c}} \qquad k_o = \left(\frac{SPAN}{HEIGHT}\right)^2\left(\frac{\Delta_h}{\Delta_v}\right)^2$$

(Units are as follows : SPAN, HEIGHT, Δ_v and Δ_h are each in millimetres, while rock stresses and rock strengths need consistent units such as MPa).

Figure 21. By plotting Q/SPAN versus tunnel or cavern deformation, as in Barton et al., 1994, and with the addition of hundreds of supplementary data from Chen and Guo in Taiwan (priv. comm.) it was possible to see the central trend of all data as Δ (mm) = SPAN (m) / Q. More accuracy is given using a stress/strength ratio. Accuracy has been checked in: instrumented powerhouse cavern in schists, instrumented Olympic cavern at Gjøvik in gneisses, and at instrumented station caverns in granites. The empirical predictions are much closer to measured values than unrealistically continuous-jointing UDEC models.

CONCLUSIONS

1. Q-system linkages to parameters useful for design are based on sound, simple empiricism, that works because it reflects practice, and that can be used because it can be remembered. It does not require black-box software loss-of-touch-with-reality.

2. The wide range of Q-values (0.001 to 1000) reflects to some degree the very wide range of structural-geological and hydro-geological conditions found when tunnelling,, and is probably responsible for the fact that empirical equations based on the Q-value or on Q_C (= Q x UCS/100) are particularly simple.

3. An integration of Q and Q_C with seismic data is useful because there is a limit to how many boreholes can be drilled, how many cores can be logged, and how many permeability tests can be performed. The ability to extrapolate these 'point sources' of information from core-logging helps to project rock quality classes along a tunnel, or to different parts of a large cavern.

4. Due to the effect of increased stress at greater tunnel or cavern depth, it must be expected that deformation modulus and seismic velocity will increase. Eventual sonic logging or cross-hole tomography ahead of a tunnel face may therefore give a higher velocity than the rock quality may suggest.

5. The most simple and least accurate approximation for permeability is that the number of Lugeon: $L \approx 1/Q_c$. This is strictly for the case of clay-free, jointed, low porosity rock masses. A more generally applicable approximation uses an inverted Ja/Jr term and 100/JCS to give a better link to permeability. A high value of Q_{H2O} implies low permeability, and a general reduction of permeability with depth is also modelled.

6. There are other surprisingly simple relationships that have their origin in empirical links to Q-values. Support pressure appears to be inversely proportional to deformation modulus, and a central trend for tunnel deformation is that Δ in millimetres is equal to span in meters divided by Q. An improved fit to the quite scattered deformation data incorporates the stress-to-strength ratio, with differentiation of vertical and horizontal stress, for estimating arch or wall deformation.

7. As expected from a system that has it's origin in tunnel and cavern support selection, there is a strong correlation of time for construction with Q, and cost of construction with Q. The strongest correlation, where the curves of time and cost are steepest, is where the Q-value is between 0.01 and 1.0. It is here that the greatest benefit of high pressure pre-injection may be obtained, with effective, apparent improvements in many of the Q-parameters, and therefore in correlated properties like increased velocity and modulus, reduced support needs, and increased round lengths.

8. Application of discontinuum codes like UDEC-BB gives much more understanding and more relevant behaviour predictions than continuum codes. The example of the Gjøvik Olympic cavern of 62 m span is given, where the blind prediction of displacements was remarkably accurate, despite the possibility of either upward or downward displacements, that depended upon the interaction of joint orientations, their strength and stiffness, and horizontal stress levels. However there is some unrealistic UDEC modelling seen, where too much joint-continuity is modelled. Deformations may be exaggerated by a factor of 10 compared to reality.

9. Strength criteria of the form 'c + tan φ' used in continuum codes, which have remarkable complexity requiring software for evaluation of their components, have in addition the problem that when supposedly simulating shear failure, the reality is cohesion reduction at small strain, and friction mobilization at larger strain. Shear strength criteria may therefore need to be of the form 'c *then* tan φ', different from those used by most designers worldwide.

REFERENCES

Barton, N., Lien, R. & Lunde, J. (1974). Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics*. 6: 4: 189-236.

Barton, N., Lien, R. & Lunde, J. (1977). Estimation of support requirements for underground excavations. *Proc. of 16th Symp. on Design Methods in Rock Mechanics*, Minnesota, 1975. pp. 163-177. ASCE, NY. Discussion pp. 234-241.

Barton, N., Grimstad, E., Aas, G., Opsahl, O.A., Bakken, A., Pedersen, L. & Johansen, E.D. (1992). Norwegian Method of Tunnelling. WT Focus on Norway, World Tunnelling, June/August 1992.

Barton, N. (1994). A Q-system case record of cavern design in faulted rock. *5th Int. Rock Mechanics and Rock Engineering Conf., Tunnelling in difficult conditions*, Torino, Italy, pp. 16.1-16.14.

Barton, N. & Grimstad, E. (1994). The Q-system following twenty years of application in NMT support selection. *43rd Geomechanic Colloquy, Salzburg. Felsbau*, 6/94. pp. 428-436.

Barton, N. & Grimstad, E. (1994). Rock mass conditions dictate choice between NMT and NATM. *Tunnels & Tunnelling*, October 1994, pp. 39-42.

Barton, N., By, T.L., Chryssanthakis, P., Tunbridge, L., Kristiansen, J., Løset, F., Bhasin, R.K., Westerdahl, H. & Vik, G.. (1994). Predicted and measured performance of the 62m span Norwegian Olympic Ice Hockey Cavern at Gjøvik. *Int. J. Rock Mech, Min. Sci. & Geomech. Abstr.* 31:6: 617-641. Pergamon.

Barton, N. (1995). The influence of joint properties in modelling jointed rock masses. Keynote Lecture, *8th ISRM Congress*, Tokyo, 3: 1023-1032, Balkema, Rotterdam.

Barton, N. (2000). *TBM tunnelling in jointed and faulted rock.* 173p. Balkema, Rotterdam.

Barton, N. (2002). Some new Q-value correlations to assist in site characterization and tunnel design. *Int. J. Rock Mech. & Min. Sci.* Vol. 39/2:185-216.

Barton, N. (2004). The theory behind high pressure grouting. *Tunnels & Tunnelling International*, Sept., 28-30, Oct., 33-35.

Barton, N. (2006). *Rock quality, seismic velocity, attenuation and anisotropy*. Textbook, in press. Taylor & Francis, The Netherlands, 800p.

Barton, N. and S.K.Pandey, (2011). Numerical modelling of two stoping methods in two Indian mines using degradation of c and mobilization of φ based on Q-parameters. *Int. J. Rock Mech. & Min. Sci.*, Vol. 48, No. 7, pp.1095-1112.

Barton, N. (2012). Assessing Pre-Injection in Tunnelling. *Tunnelling Journal*, Dec.2011/Jan. 2012, pp. 44-50.

Barton, N. (2012). Defining NMT as part of the NATM SCL debate. *TunnelTalk*, Ed. Shani Wallace. Sept. 2012, 4 p.

Bieniawski, Z.T. (1989). Engineering rock mass classifications: A complete manual for engineers and geologists in mining, civil and petroleum engineering. 251 p. J. Wiley.

Grimstad, E. & Barton, N. (1993). Updating of the Q-System for NMT. *Proc. of Int. Symp. on Sprayed Concrete - Modern Use of Wet Mix Sprayed Concrete for Underground Support*, Fagernes, 1993, (Eds Kompen, Opsahl and Berg. Norwegian Concrete Association, Oslo.

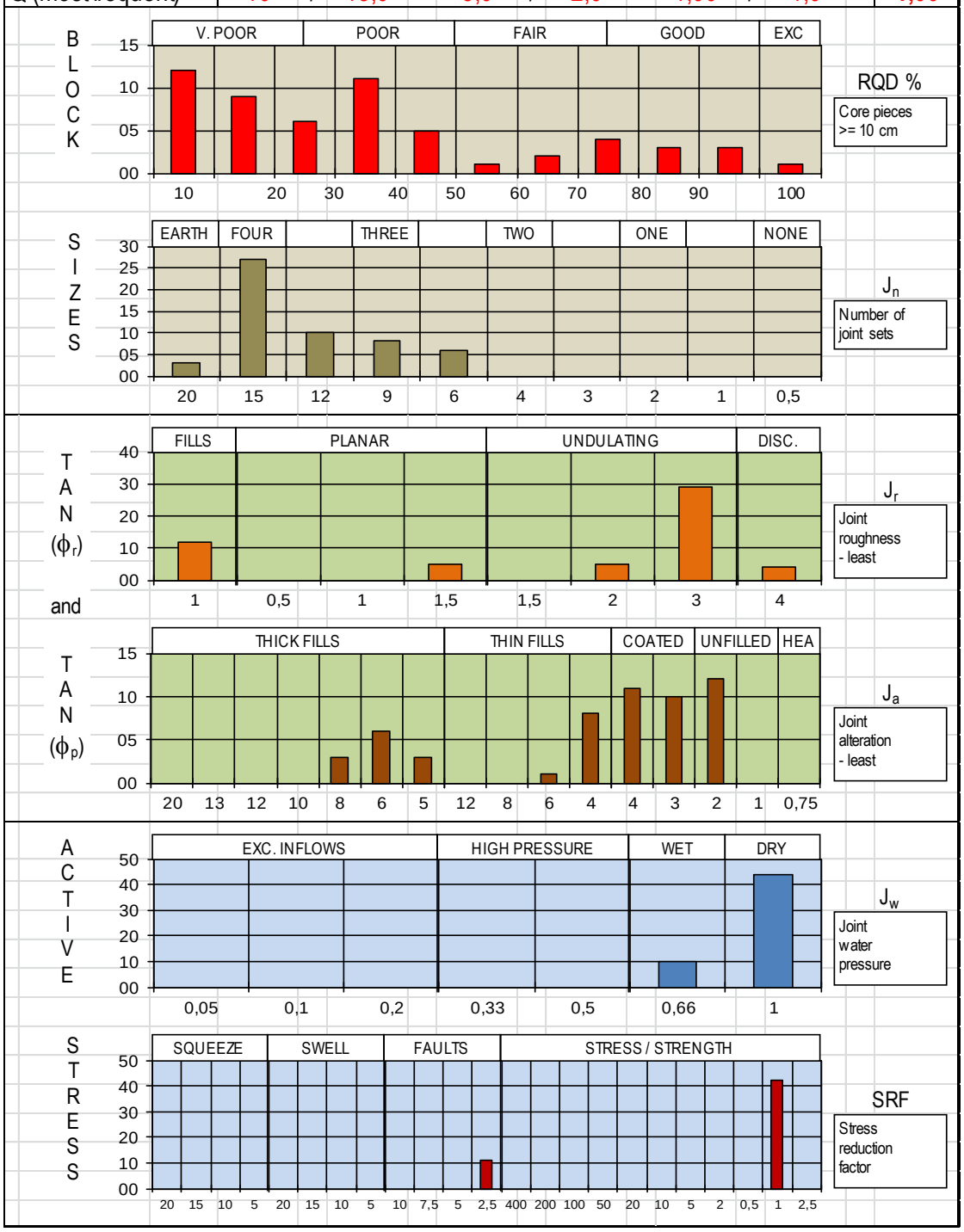
Hajiabdolmajid,V., C. D. Martin and P. K. Kaiser, (2000). Modelling brittle failure. *Proc. 4th North American Rock Mechanics Symposium, NARMS-2000* Seattle J. Girard, M. Liebman, C. Breeds and T. Doe (Eds), 991–998. A.A. Balkema, Rotterdam.

Müller, L. (1966). The progressive failure in jointed media. *Proc. of ISRM Cong*., Lisbon, 3.74, 679-686.

Roald, S., Barton, N. & Nomeland, T. (2001). Grouting – the third leg of underground construction. *Norwegian Tunnelling Society*, Publ. Nr. 12.

Sjøgren, B., Øfsthus, A. & Sandberg, J. (1979). Seismic classification of rock mass qualities. *Geophys. Prospect.*, 27: 409-442.

Q - VALUES:	(RQD	/	Jn)	*	(Jr	/	Ja)	*	(Jw	/	SRF)	=	Q
Q (typical min)=	10	/	15,0	*	1,0	/	6,0	*	0,66	/	2,5	=	**0,029**
Q (typical max)=	75	/	6,0	*	4,0	/	2,0	*	1,00	/	1,0	=	**25,0**
Q (mean value)=	38	/	12,8	*	2,4	/	3,9	*	0,94	/	1,3	=	**1,29**
Q (most frequent)=	10	/	15,0	*	3,0	/	2,0	*	1,00	/	1,0	=	**1,00**



APPENDIX: Q-HISTOGRAM LOGGING EXAMPLE IN HEAVILY JOINTED, CLAY-BEARING, FAULTED ROCK.
NOTE RATINGS AND BRIEF DESCRIPTIONS FOR EACH OF THE SIX PARAMETERS: RQD, Jn, Jr, Ja, Jw and SRF.