Forty Years with the Q-system – Lessons and Developments

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ABSTRACT

This paper describes some of the lessons learned during four decades of application of the Q-system, both in Norway and abroad. Personal application in more than 30 countries, and widespread use by others in civil and mining engineering around the world, has provided rich experiences, stimulated numerous discussions and critique, and probably provided a simpler means of communication for geologists, for rock engineers, for mining engineers and also for lawyers in court cases. In reality the Q-system is far more than six parameters, as the geology has to be understood before application can be optimal. A new combination of parameters, simply Jn/Jr, is found to have surprisingly useful properties for tunnels and mines. The paper shows some comparisons between NMT (single-shell) and NATM (double-shell), emphasises the safety of RRS compared to lattice girders and S(fr) compared to S(mr). Latest Q-based dimensioning are given, plus some links between Q and useful parameters when modelling, specifically stress-dependent modulus, velocity, and tunnel or cavern deformation. During the 40 years of Q-application, the last 15 years has seen application of Q_{TBM} with added machine-rock interaction parameters. Recently Q_{slope} was developed, to guide the choice of maintenance-free slope angles.

1 INTRODUCTION

Development of the Q-system occurred during six hectic months in 1973 (not three years as stated in a recent NGI report), and it started specifically as the result of a question from NVE the Norwegian State Power Company (now Statkraft) to NGI. The challenging question: Why all the different deformation magnitudes in Norwegian hydropower caverns? The author could not adequately answer the question, so had to start developing a rock mass classification method, linked to support needs. Bieniawski (1973) developments were not yet known. So in the Q-system development, RQD/Jn came first, with successively added parameters, and a lot of trial-and-error and empiricism using more than 200 case records. Sixty percent of the first case records were from Norway and Sweden, and 50% concerned hydropower excavations, both caverns and tunnels. The development of Q finally enabled an answer to be provided to the question about powerhouse deformation magnitudes. Since some deep caverns and road tunnels were included, the onset of stress-fracturing was important, hence the sixth parameter SRF, involving the ratio rock uniaxial strength/stress. Grimstad and Barton (1993) added the experiences from 1,050 more case records, including extensive deep road tunneling in Norway, with eventually a depth as much as 1,400 m at Lærdal (and a world record length of 24.5 km). In this project there were three lorry-turning caverns of 30 m span at > 1,000 m depth (and 6 km intervals). These challenge even mining experiences.

From the start not only rock mass quality, but excavation dimensions, purpose and rock reinforcement and support needs were integral parts of the method. The *number of joint sets*, which was suggested as an addition to RQD by Don Deere's Ph.D. student Cecil (1970), has remained an important part of Q, but is remarkably absent from Bieniawski's RMR and is therefore also absent from GSI, which is the basis of the Hoek-Brown estimation of shear strength, used by so many optimistic continuum modellers. Since rock mass classification ‘*is not possible, and can therefore never be developed’* (roughly the brief Norwegian opinion expressed in NTH/NTNU Norwegian Technical University engineering geology course notes for several decades), it can be imagined the enthusiasm from Trondheim and other quarters, when Q was launched and also used by others.

Besides geological engineering colleagues at NGI (principally Lien and Løset in the first years), the Swedish contractor Skanska was the first outside Norway to test the method in the ensuing months, in the Majes project in Peru and in a hydropower project in Kenya. Application in Norwegian hydropower and road and rail tunnels gradually developed to impressive levels in the seventies and following decades, despite natural early suspicion from those outside NGI. Today Q is used, often in combination with RMR, in thousands of projects around the world, and all principal mining countries use Q’ (= RQD/Jn x Jr/Ja) for non-entry stope design, in order to help find the transition between stable and over-breaking or caving ore-bodies or surrounding waste rock. Stope wall areas sometimes in excess of 10,000 m^2 present challenges in minimizing dilution. In Hong Kong, the speciality for the last 25 years is to use Q for the selection of *temporary support*, using the correction ‘5Q and 1.5 ESR’, since the more expensive NATM-style concrete-lined tunnels and caverns, with drainage-fleece and membrane, is the preferred method.

2 ADVANTAGE OF A LOGARITHMIC QUALITY SCALE

Unlike RMR or GSI (≈RMR-5) and the Austrian F1 to F7 rock mass quality scale, the Q-value resembles a logarithmic scale of quality with its six orders of magnitude from approximately 10^{-3} to 10^{3}. With the normalization Q_c = Q x UCS/100 introduced by Barton (1995), the Qc scale can reach almost eight orders of magnitude, and then approaches the *actual variability found in nature*. One only needs to consider the range of deformation moduli and shear strengths depicted in the deliberately contrasted photographs in Figure 1 to realise that not only these parameters, but also the need for support and the *loading of the support* can cover an extremely big range: from zero up to even 200 t/m^2. As will be seen later, there is a clear inverse proportionality between support pressure/capacity needs, and the simply estimated deformation modulus, which itself can vary by a factor of 100, or even 1000, between the extremes of saprolite/soil and hard unjointed rock. The extreme non-linearity and anisotropy of nature does not link in a simple way to linear (RMR or GSI) rock quality scales, so formulae (such as those behind the Hoek-Brown shear strength criterion) are unnecessarily complex, as discussed at length in Barton (2014).

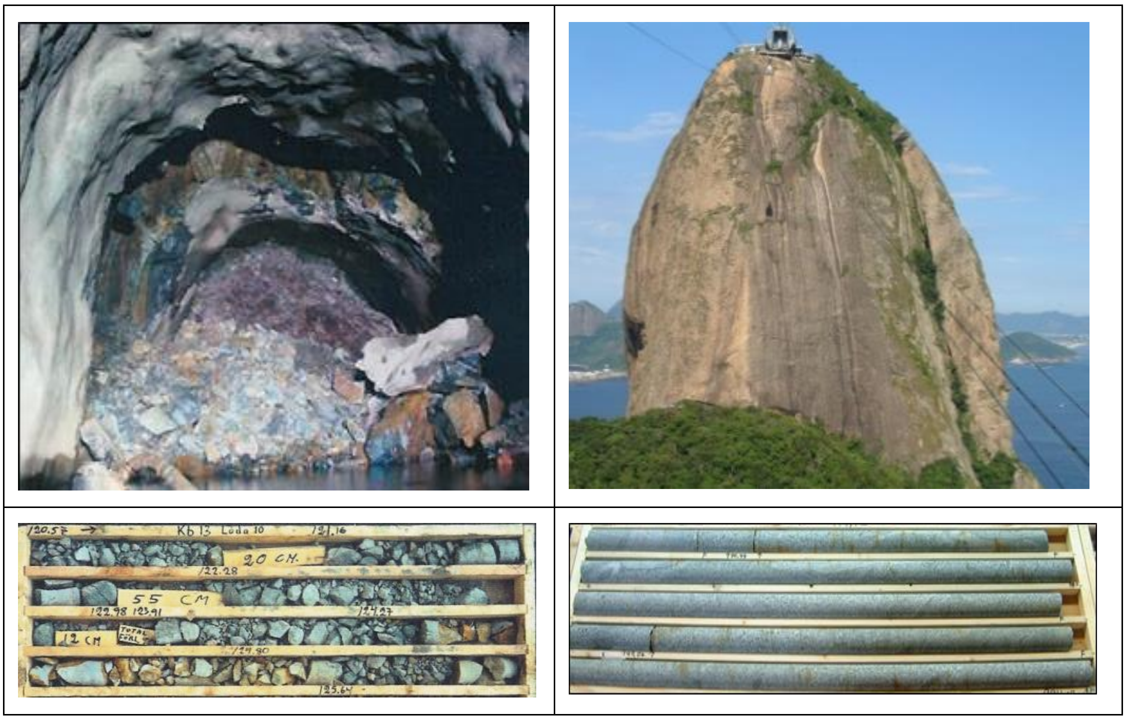


Figure 1: Contrasting worst (Q ≈ 0.001) and best (Q ≈ 1000) rock mass qualities from Brazil, Sweden and HK.

The contrasting cases illustrated in Figure 1 are from Brazil (two cases), and the core boxes are from Sweden (Hallandsås rail tunnels) and Hong Kong (the latter due to an unrepresentative vertical hole in granite). The core box with core loss is from a wide regional shear zone in S.W. Sweden, which eventually required freezing. The logarithmic appearance of the Q-value scale, stretching over six orders of magnitude, has proved to be a great *advantage*, and results in simple empirical equations for relating Q or Q_c to velocity, deformation modulus, and deformation. There appear also to be believable links to c and φ (using a 'split' Q-equation), if needing to perform continuum analyses: see Barton and Pandey (2011), and a modified (inverted Ja/Jr) version called Q_{H2O} appears to link with permeability. These will be briefly mentioned if space remains.

3 OVERBREAK ESTIMATION USING THE RATIO Jn/Jr

An unusual combination of Q parameters: the ratio Jn/Jr, involving the number of joint sets and the dominant joint roughness, indicates that a ratio Jn/Jr ≥ 6 automatically means a strong likelihood of 'natural' overbreak, for which a contractor cannot be blamed for harsh blasting practices. Figure 2 illustrates the combined importance of Jn and Jr. The overbreak-facilitating ratio Jn/Jr ≥ 6 applies over a range of Jn = 15, 12, 9, 6, 4 and over a range of Jr of 1, 1.5, 2 and 3. If blocks are not formed due to insufficient joint sets, or when joint roughness is significant, a typically massive rock mass with for instance Jn/Jr = 4/3, would mean virtually all half-rounds are visible. However, a contractor will have great difficulty to produce half-rounds when Jn/Jr > 6 due to the naturally over-breaking rock mass.

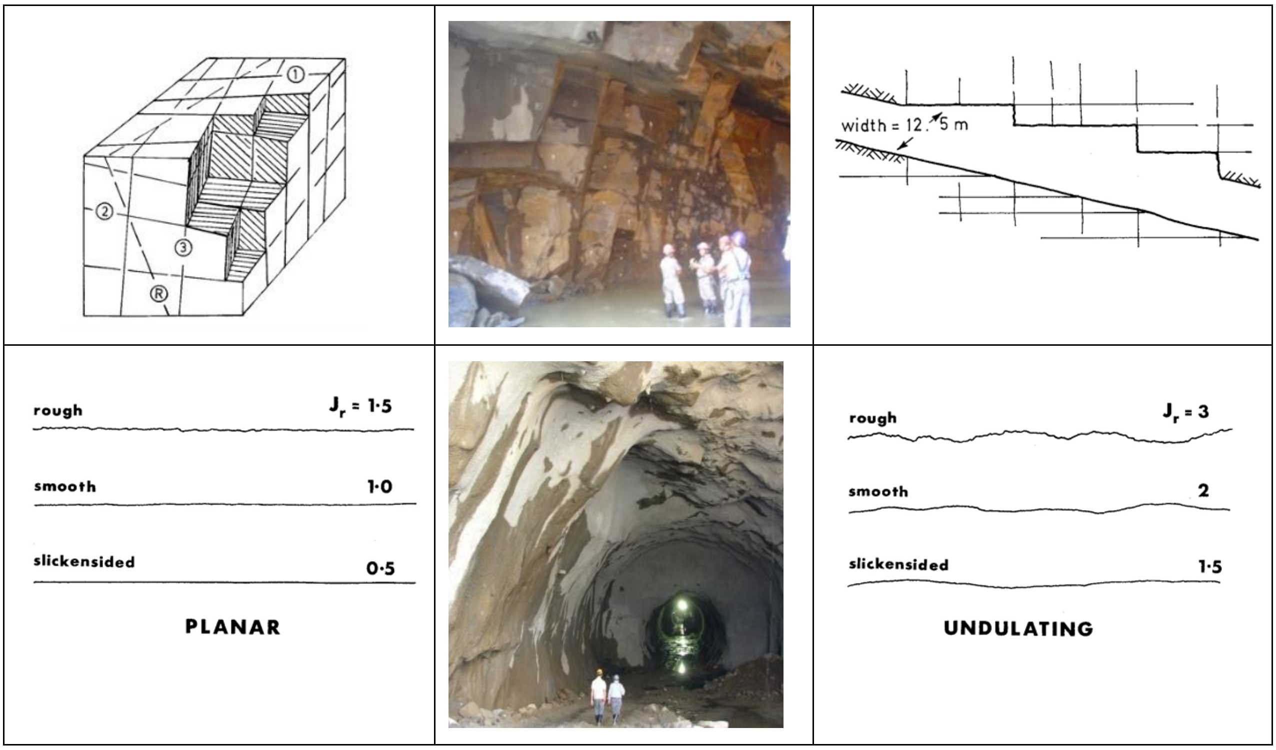


Figure 2 : Overbreak is extremely likely to occur despite a contractor's efforts with careful blasting, if the most frequent value of the ratio Jn/Jr ≥ 6, i.e. 6/1, 9/1.5, 9/1.0, 12/2, 12/1.5, 12/1.0, 15/1.5. Visible half-rounds or at least lack of overbreak will tend to be found when Jn/Jr < 6, such as 3/3, 4/2, 6/1.5, 9/2, 9/3, 12/3, 15/3. Lack of block structures combined with dilatant joint roughness or discontinuous joints prevent its occurrence. 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 with discontinuous jointing. We can assume 0.5 to 1 m scales when comparing the roughness of exposed (over-broken) joint planes with these Jr.

The ratio of Jn/Jr, applying as it does to a wide range of Jn values and to a wide range of Jr values, becomes a useful tool for assessing whether a contractor has blasted 'carelessly', or whether the over-break is inevitable, unless artificially short rounds were blasted, thereby compromising tunnel (or mine roadway / access ramp) production schedules too much.

4 Q-PARAMETER ZONING IN MINING AND CIVIL PROJECTS

The writer has noted the frequent use of the Q-system in various roles in mining. These include the use of Q' (or N) for open-stope behaviour prediction, Q-system support and reinforcement guidelines for permanent mine roadways, and Q-value based 'geotechnical zoning' for future or present mining resources. In the latter it is important to use all six parameters. A point to remember when logging Q-parameters is that, although Q alone may form a helpful number with which to communicate an impression of rock quality (or lack of quality), there is important information 'coded' in the six individual Q-parameters. In this context it is useful to collect, and present, the statistical spread of data, as in the form of Q-parameter histograms, as illustrated in Figure 3. This is a mining project with > 300 km of core-logging-based Q-value statistics. Here the visual (non-EXCEL spread-sheet) visualization of data is important, and assists auditing of the huge body of data.

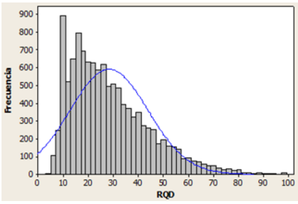
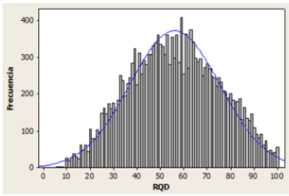
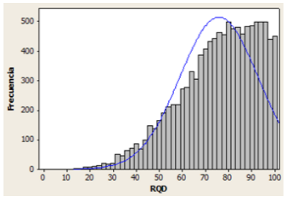
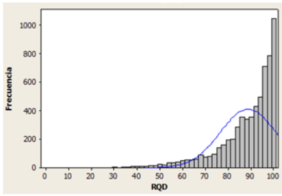
Q = 0.1 to 1.0	*Q = 1.0 to 4*	*Q = 4 to 10*	*Q =10 to 40*
			

Figure 3 : Q-classes 2, 3, 4 and 5 with respective Q-ranges as follows: 40-10, 10-4, 4-1, 1-0.1, and respective numbers of observations of RQD, from approximately 6,000, 10,500, 18,000, 10,400 m of core logging, demonstrate the central role played by RQD in commonly experienced rock mass conditions. Critique of RQD by Palmström and others, who make incorrect assumptions of its theoretically limited 'range' (the '9 cm, 11 cm problem') needs to be silenced, as RQD has useful anisotropic and three-dimensional recording abilities. Consistent high values such as 90 and 100 often cause problems with TBM penetration rates in hard rock, but obviously limit tunnel support needs. RQD = 90 and 100 very seldom means '11 cm block sizes' as implied by the critics. Note the 'bell curve' for RQD in the (almost) central Q = 1.0 to 4 rock mass class.

RQD is a particularly sensitive parameter for rock engineering problem areas, (but not for quarried dimension stones), and it has survived 50 years of use because of this. RQD is particularly sensitive to the general rock class – and it partly 'sets the scene' for the overall Q-value – despite obviously missing some important details if used as a stand-alone parameter.

Figure 4 : Evaluation of Q-parameters is not an exact science, and there is room for limited differences of opinion. Here are photo-examples of Jr stretching from 1.0 (smooth-planar) to 1.5, 2.0 (smooth-undulating), 2.5 and even 3.5 (extremely rough-undulating). There is no 'law' against using Jr = 2.5 or 3.5 when it seems to be needed. It gives greater 'accuracy' if one can talk of this on a 10^{-3} to 10^{3} logarithmic-looking scale. Users should avoid decimal places for Q > 1.

When evaluating the use of Q' (where Q' = RQD/Jn x Jr/Ja), for the first part of open stope design in mining, one can suggest that use of the largest stoping sizes (such as > 100m wall heights), may introduce a *scale effect* in the Q-estimation. This is because the larger-scale structural features, often also needing evaluation of SRF because they are shear zones or faulting, may in practice affect the dilution or unwanted over-break of waste rock, even if not affecting the much smaller tunnel-scale excavations. The truncation of Q suggested in the early eighties by mining consultants in Canada, has undoubtedly had some benefits. Nevertheless the 'loss' of SRF had immediately to be replaced by a strength/stress ratio. Some subsequent users have lamented the lack of fault zone effects in Q', and for faulted ore-bodies under deep (and therefore wet) river valleys, the 'absence' of Jw is also unfortunate, as experienced in a South American project.

I	**ROCK MASS STRUCTURE**		
1	RQD	Deere et al., 1967)	block size { Q
2	J_n	= joint set number	Q
3	F	= joint frequency (per metre)	
4	J_v	= volumetric joint count (Palmström, 1982)	
5	S	= joint spacing (in metres)	
6	L	= joint length (in metres)	
7	w	= weathering grade (ISRM, 1978)	
8	α/β	= dip/dip direction of joints (Schmidt diagram)	
II	**JOINT CHARACTER**		
9	J_r	= joint roughness number	shear strength { Q
10	J_a	= joint alteration number	Q
11	JRC	= joint roughness coefficient	
12	a/L	= roughness amplitude of asperities per unit length (mm/m)	
13	JCS	= joint wall compressive strength	
14	ϕ_r	= residual friction angle	
15	r,R	= Schmidt rebound values for joint and rock surfaces	
III	**WATER, STRESS, STRENGTH**		
16	J_w	= joint water reduction factor	active stress { Q
17	SRF	= stress reduction factor	Q
18	K	= rock mass permeability (m/s)	
19	σ_c	= compressive strength	
20	σ_1	= major principal stress	

Figure 5 : During NGI's six years of work with the UK Nirex nuclear waste disposal project, during which some 11 km of core was Q-logged, shear tested and tilt-tested at intervals, all the parameters shown in this table were included in our histogram-based spread-sheets. The above list serves as a useful reminder that the six Q-parameters are just a part of the description of any rock mass. See Barton et al. (1992) and Bhasin et al. (1999).

More parameters are needed when e.g. UDEC-BB modelling is to be performed in order to help verify the empirically-based solutions of e.g. cavern support. Note that a current commercial continuum model used by many younger-generation engineers, exaggerates deformation and 'plastic zones' (this has been proved in a formal court case), and some invalid suggestions to increase empirically-verified bolt lengths have been seen at various times. This is a case of *'a priori'* contra *'a posteriori'*. The latter has contributed much more to human development. Barton (2011).

It is always important to be aware that the six parameters are only an abbreviated description of the rock mass, as can be seen in Figure 5. The 'relative block size' (RQD/Jn) is just part of the *rock mass structure*. The shear strength (Jr/Ja) which is actually like a friction coefficient, is a relatively simple but very important part of the *joint character*. Histogram presentation of parameters in the third group of Figure 5 (III) is also beneficial, as the potential variation of most (all?) rock mass parameters is important. A very deep tunnel with *variable rock stress and variable UCS* may suffer from the fact that the 'tails' of the σ_{max} and UCS distributions may coincide, and cause damaging rock bursts. There are several examples, including numerous fatalities in a recent TBM tunneling project, where the highest σ_θ tangential stresses were higher than UCS_{min}. Probably due to scale effects, stress induced failure starts when this ratio exceeds 0.4 to 0.5. Barton (2012).

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**

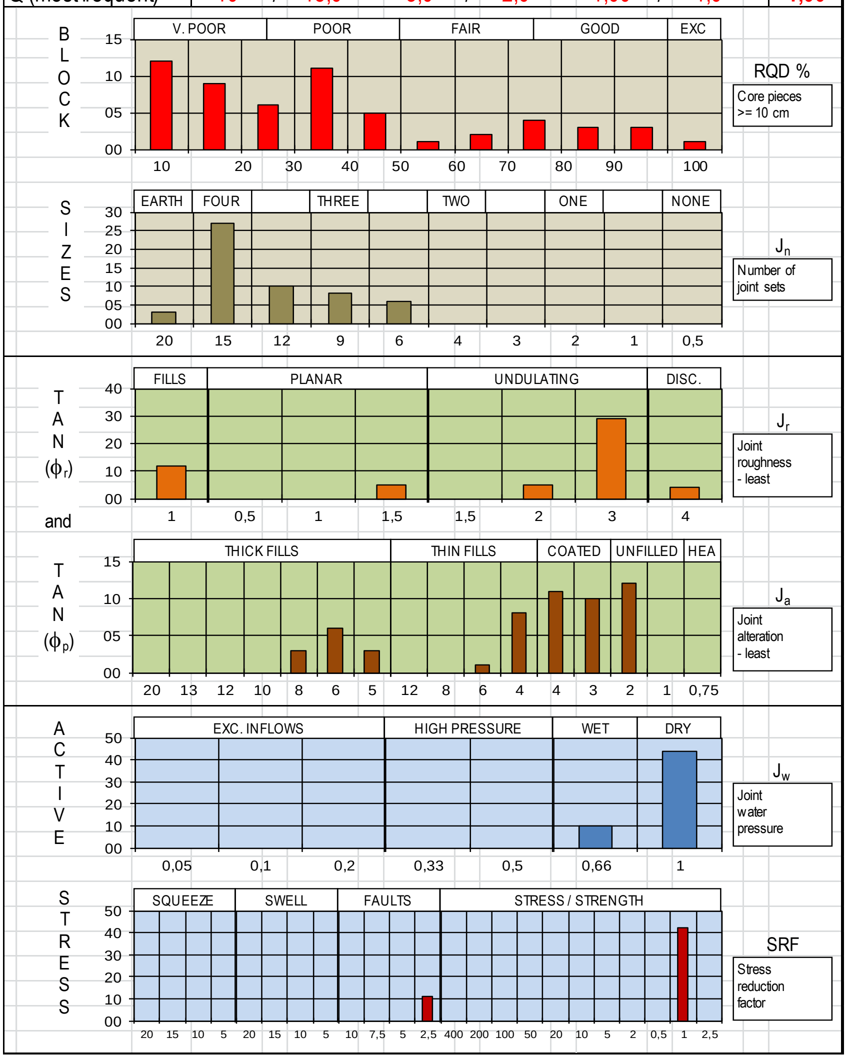


Figure 6: An example of Q-histogram representation of the actual variation of parameters.

Because of the variable nature of rock masses, it is difficult, sometimes even impossible, to give single representative values of Q. This is the nature of rock engineering for mining, tunneling, and dam site description. Compared to the conveniently small test samples of steel, and the manageable cube tests of concrete, we need by comparison a *lecture-theater (or city block) size sample* of the rock mass, in order to come close to representing, and allow the recording, of the locally variable properties. Hence the histogram logging illustrated in Figure 6.

Practical use of Q-histogram logging requires a scheme of labelling, in which each e.g. 5m of core (or tunnel wall, or outcrop) is numbered, and rock types or structural domains are labelled, also using footnotes. Footnotes can also be used to distinguish the most unfavourable Jr/Ja combination for tunnel support selection exercises. In the case of Q-histogram logging for TBM projects (e.g. Barton, 2000, 2013), all the significant jointing is recorded, as cutters sample a three-dimensional advancing slice of the rock mass. The orientation of optimal or non-optimal jointing needs to be noted, and the (usually) horizontal nature of the tunneling needs to influence the recording of RQD, Jr and Ja in particular.

With the present conference date of 2015 representing a 40-years anniversary since Q-system application began in earnest, Barton and Grimstad (2014) recently produced an extensively illustrated 43 pages documentation of the recommended use of the Q-system, with numerous core- and tunnel-logged examples, and an extensive discussion of characterization pitfalls, and support and reinforcement principles. The article also documents important parametric linkages to Q and Q_c. It can be obtained from www.nickbarton.com p4 of downloads. It is a little too long and too coloured to be published in regular rock engineering journals. Colour is the essence when recording the logging of core and rock outcrops for rock engineering projects.

5 TEMPORARY SUPPORT ESTIMATE FROM Q IN *NATM* TUNNELS AND CAVERNS

The updated Q-support chart from Grimstad and Barton, 1993 is often referenced in relation to single-shell NMT tunneling. In Figure 7, 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 x 2.0m c/c + S(fr) of 9 cm for *permanent NMT-style support*. Each would be of high quality, meaning multi-layer corrosion-protected (CT) bolts, and e.g. C45 MPa S(fr) with stainless steel (or PP polypropylene) fibers, such as the best (roughest-surface) Barchip fibers. The rule-of-thumb of 1.5 x ESR and 5 x Q for *temporary support* was actually intended (Barton et al. 1974) as guidance to contractors (i.e. not a temporary support procedure for consultants planning a concrete lining). The largest ‘cube’ shown in the figure would reduce to ‘coordinates’ of *B + S(fr) = 2.4 x 2.4 m c/c + S(fr) = 4 cm. (The coordinate SPAN /* 1.5 x ESR = 13.3m, and 5Q = 15, is shown by the larger arrow-head in Figure 7). As we will see shortly, S(fr) thicknesses of only 4 cm are no longer advised, due to curing problems, but for temporary support, to be followed by concrete lining, it might be accepted.

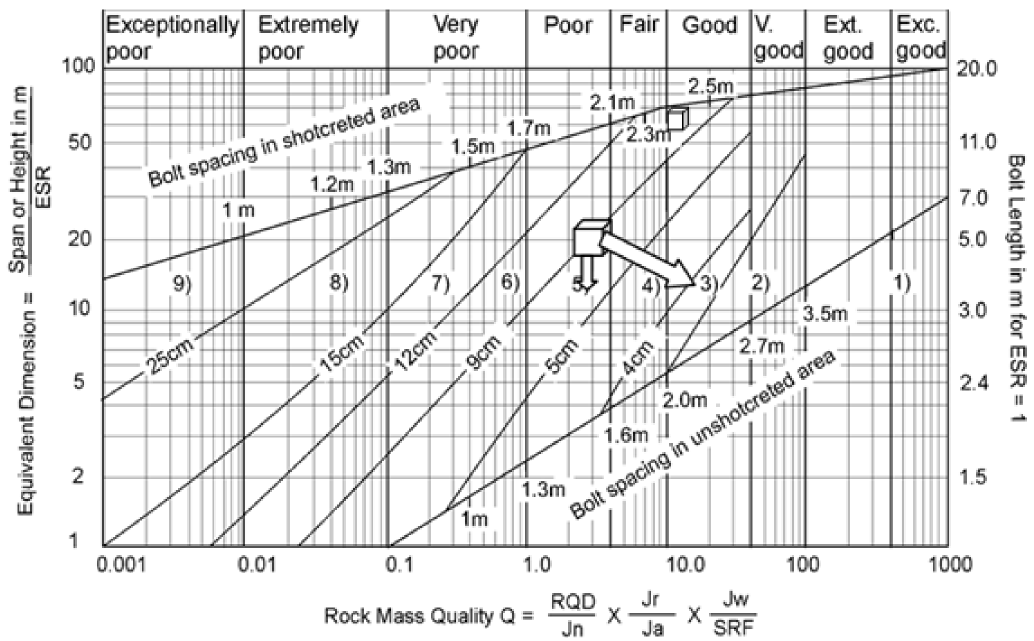


Figure 7: Using the Smr to Sfr updated Q-support chart of Grimstad and Barton (1993), the rule-of-thumb of 5Q and 1.5 ESR for temporary support is demonstrated for the SPAN = 20m, ESR =1.0, and Q = 3 portion of an imaginary station cavern. (Updated ESR table given later). The small cube represents the *single-shell*, drained, 62m span Gjøvik cavern.

Some 25 years of experience using this officially approved Q-based method, in hundreds of kilometers of metro, road and rail tunnels in Hong Kong alone, has proved its reliability in ensuring sufficient *temporary support*, pending the construction of the permanent reinforced concrete lining (with drainage fleece and membrane). A ‘delay’ of 1 to 2 years is frequent. Personally, the author prefers single-shell NMT to double-shell NATM, since it is 1/4 to 1/5 as expensive, and the tunnel or cavern is completed much faster. However the reality is that many countries find the budget for NATM and permanent concrete linings: they have fewer tunnels than, for instance Norway, with its > 5,000 km, 3,500 km of which are hydropower related. With a good budget, Q-based *‘5Q+1.5 ESR’* can be used to select temporary support, while waiting for the final liner.

6 THE COMPONENTS OF Q-SYSTEM BASED SINGLE-SHELL *NMT* SUPPORT

This section consists of illustrations of the three key items of the NMT-based Q-support recommendations, including a poor example of *yesterday’s* S(mr), to illustrate what we should have left behind, compared to the last thirty five years of much safer and faster S(fr). As clearly illustrated, both by sketches and by reality, S(fr) has huge advantages. The continued use of S(mr) in some (e.g. NATM-practicing) countries is remarkable.

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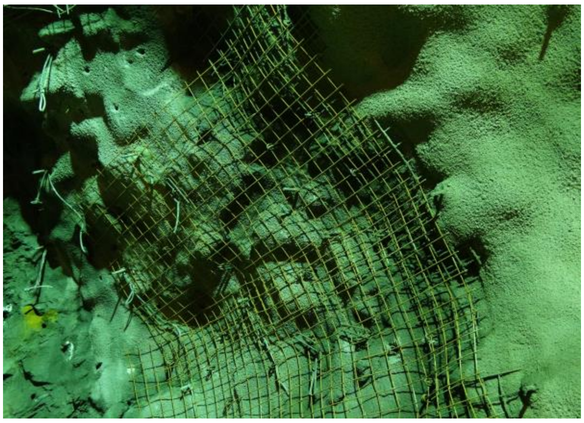
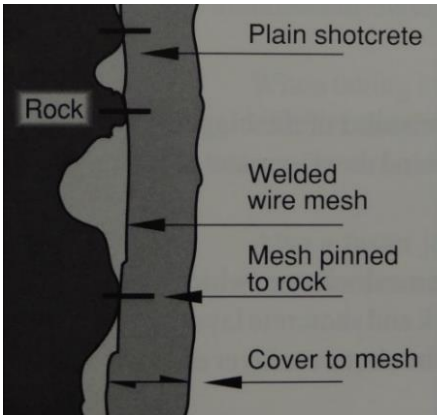
<td></td>
<td>
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<td>When the Q-system was developed in 1973, the single-shell case records had permanent shotcrete support and bolting reinforcement of lower quality than that available in the decades that followed. This is an example of S(mr) from a major project in Colombia, with all the potential difficulties and ineffectiveness of S(mr) well illustrated.</td>
<td>Vandevall (1990) illustration of the pitfalls with mesh reinforced S(mr) shotcrete. It involves three processes, risk of ‘shadow’ and/or some rebound, corrosion of the mesh due to electrolytic currents, and delayed, sometimes unsafe installation.</td>
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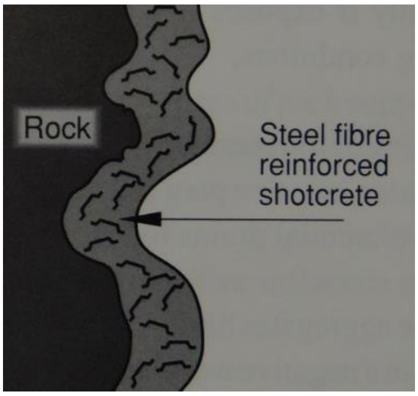
<td>
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<td>Wet process steel-fiber reinforced shotcrete, applied after thorough washing, and use of corrosion-protected rock bolts (e.g. CT-type) are the most important components of the updated Q-system support and reinforcement for single-shell NMT. One more layer of S(fr) is needed.</td>
<td>Vandevall (1990) illustration of the obvious advantages of S(fr): better bonding, no shadow, gets into over-break, less corrosion, much lower permeability, faster, safer, cheaper per tunnel-meter.</td>
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Figure 8: The advantages of S(fr) compared to S(mr) are easily appreciated in these contrasting examples. The sketches from Vandevall (1990) are not-exaggerated. With over-break (Jn/Jr>6), S(fr) must be the logical method.

The reality of *single-shell NMT*-style tunneling, in comparison to *double-shell NATM*-style tunneling is that each component of support has to be permanently relied upon. In NMT there is no long-term neglect of the contribution of temporary shotcrete, temporary rock bolts, and temporary steel sets (not used in NMT anyway), nor long-term reliance on a final concrete lining, as there is in NATM. Thus more care needs to be taken in the choice and quality of the support and reinforcement components B+S(fr) + (eventual) RRS.

There have been so many failures, sometimes dramatic, in the 'temporary support phase' of NATM projects, that it is surprising that the industry tries to cut corners, by for instance not reinforcing the shotcrete, except for the inadequate lattice girders, or leaving out rock bolts. Later, an approved sequence of NATM components will be shown, suggesting that 'temporary support phase' failures could be reduced if Austrian recommended methods were followed. Of course this comes at a much higher price than the simpler NMT.

Figure 9 illustrates (in the form of a shortened demo sample) the workings of the CT bolt, for those wishing to use NMT outside Norway, for instance for a large cavern, who may not have used this remarkable, permanent, tunnel reinforcement component. The figure text emphasizes the multiple-layer corrosion protection. Deformation of a bolt-intersected joint (the purpose of rock bolts is to limit this), may typically crack the outer annulus of grout. In normal circumstances the onset of corrosion could be at such locations.

Figure 9: Because single-shell (NMT) relies on high quality S(fr) and long-life rock bolts, the multi-layer corrosion protection methods developed by Ørsta Stål in the mid-nineties, became an important part of NMT. The left photo shows a blue-coloured PVC sleeve: there are also CT-bolts with black, or white, PVC sleeves, as seen in Figure 8. The over-cored CT bolt shown on the right, has a joint/crack deformation next to the bolt (an expected mechanism when installing close to the tunnel face) which does not in this case, initiate a potential process of corrosion. Such cracking might lead to the onset of corrosion of a more conventional singly-grouted bolt without the PVC sleeve.

Figure 10 illustrates some of the internal reinforcement details and final appearance of RRS (rib reinforced shotcrete) which are robust and stiff NMT-based alternatives to the yielding, deformation-inviting un-bolted lattice girders so commonly used in NATM tunnels as primary (and temporary) support. See discussion in Barton (2012a). The dimensioning of RRS is illustrated in a *Q-value versus tunnel dimension* diagram shortly.

If RRS (steel-rib reinforced and bolted) arches of S(fr) instead of lattice girders are used, deformation may be reduced in relation to conventional NATM. Lattice girders invite (in fact require) deformation, followed by

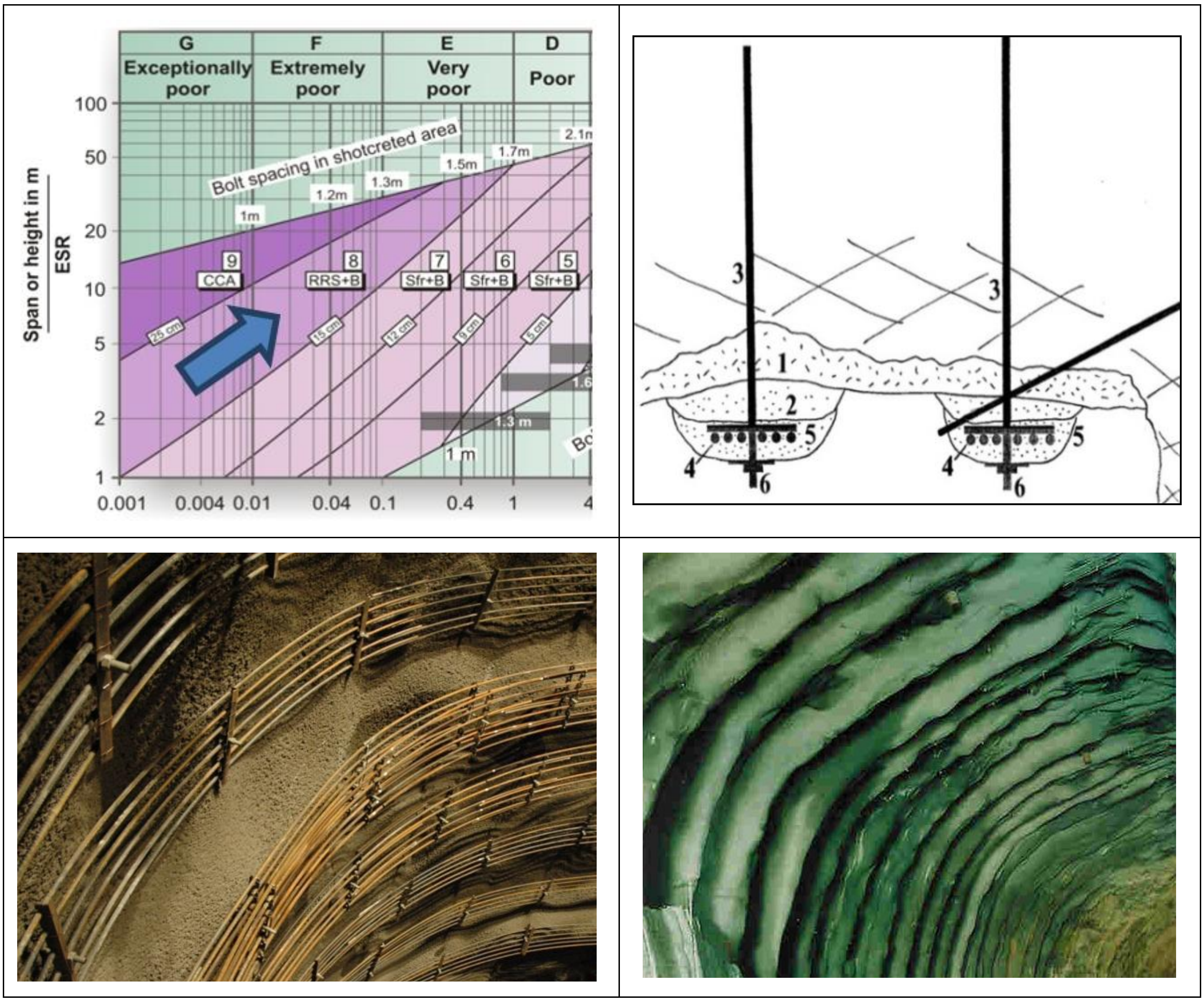


Figure 10: Some details which illustrate the principle of rib reinforced shotcrete arches (RRS), which are important components of the Q-system recommendations for stabilizing very poor rock mass conditions. The lower-left photograph is from VVS, and the design sketch is from Barton (1996). The blue arrow shows where RRS is located in the Q-support chart. See details of arch thickness and spacing in an updated Figure 11.

possible loosening of the rock mass, and will not resist deformation effectively until all blocking, all bolted joints, all steel (and shotcrete) have ceased to deform 'elastically', and the 'elephant-footing' has ceased to subside. This is truly a soft support component, and has been responsible for some big NATM tunnel failures.

The 1990's photograph of completed RRS ribs (bottom-right, Figure 10) was from one side of the 28 m span National Theater station in downtown Oslo, prior to pillar removal beneath only 5m of rock cover and 15m of sand and clay. Final concrete lining followed the RRS for obvious architectural reasons. The RRS shown here are 'partial', as pillar removal was needed for their completion as continuous arches. Hence the unexpected 'bend'(see the top-right corner of the photograph).

It should be noted that the 1993 Q-support chart (shown earlier in Figure 7) suggested the use (at that time) of only 4-5 cm of *unreinforced* sprayed concrete in support category 4. The application of *unreinforced* sprayed concrete came to an end during the 1990's, at least in Norway. Furthermore, thickness down to 4 cm is not used any longer, due to the already appreciated risk of the shotcrete drying out too fast when it is curing. The Q-chart from 1993, which was based on 1,050 additional case records collected by Grimstad (Figure 7), and also an updated 2002/2003 version, based on information from 800 new case records, indicated a very narrow category 3 consisting of only bolts in a 10m wide tunnel when Q was as high as 10-20. This 'bolt-only' practice is not accepted any longer in Norway, at least for the case of transport tunnels.

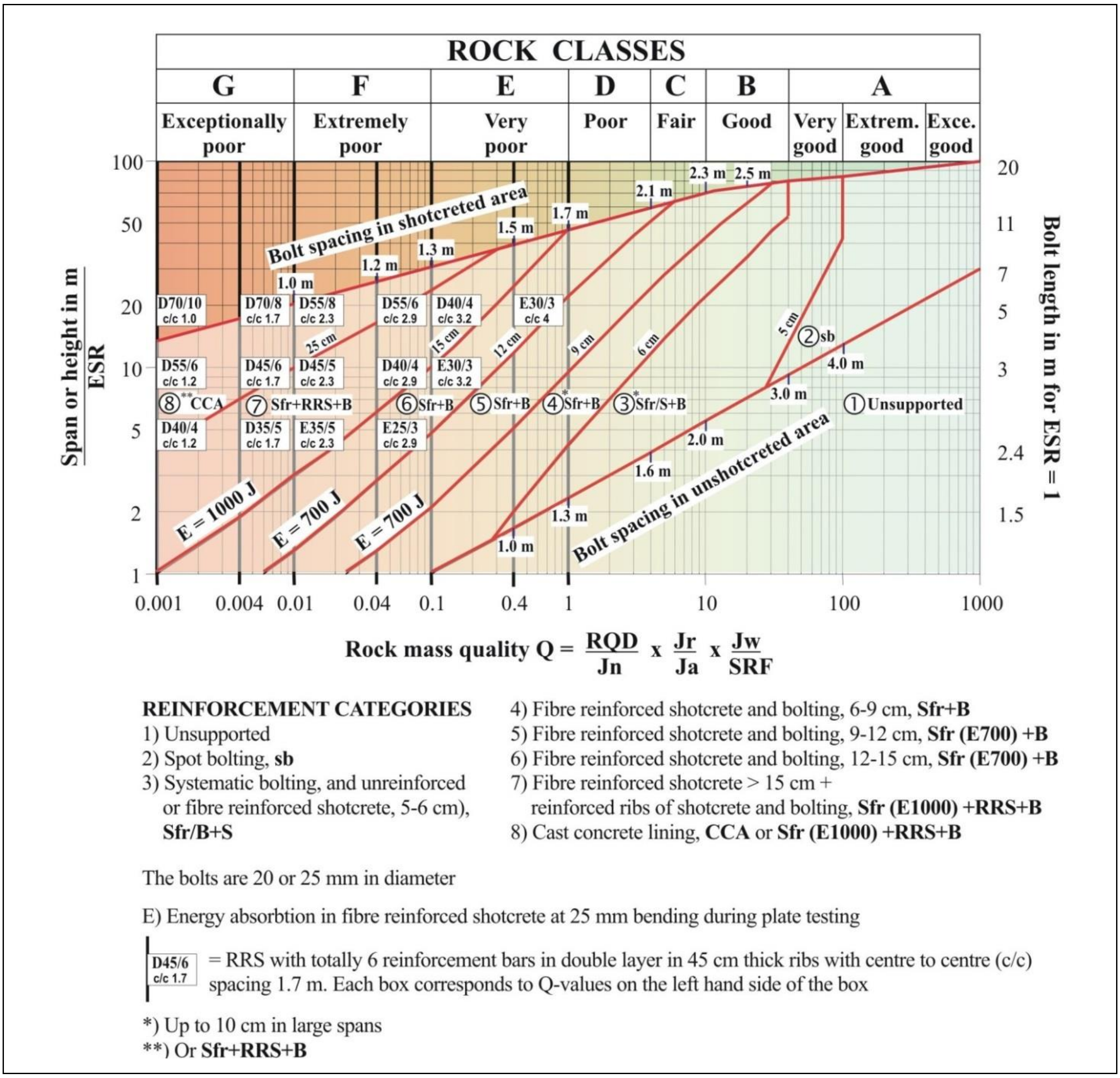


Figure 11: The most recent updated Q-support chart published by Grimstad (2007). The details of RRS (rib reinforced shotcrete arches) which are used as temporary support in the worst conditions are given in the ‘boxes’ in the left-hand-side of the Q-support diagram. Appropriate dimensions were derived by a combination of empiricism and some specific numerical modelling by a small team at NGI under the direction of Grimstad. Details of this modelling are given in Grimstad et al. (2002, 2003). Note that rapidly sprayed and non-alkali-accelerated adjacent ribs of S(fr) act as ‘smoothed’ foundations for the bent steel bars (typically 16 mm). These are fixed to rock bolts with standard spacing (i.e. 1 m to 1.5 m c/c around the S(fr) arch, depending on how low the Q-value is. Unreinforced shotcrete is used instead of final S(fr) to cover the bars, in order to minimize voids due to rebound of the fibres.

The support category 3 in 1993 and 2002/2003 has been taken away in the newest 2007 chart (Figure 11). However for less important tunnels with ESR = 1.6 and higher, only spot bolts are still valid. Hence we may distinguish between transport tunnels (road and rail) and head-race tunnels, or water supply tunnels, using the appropriate value of ESR: (see Table 1 which follows).

Note that each ‘box’ contains a letter ‘D’ (double) or a letter ‘E’ (enkel/single) concerning the number of layers of reinforcing bars. (The photo of unsprayed bars in Figure 10 shows both varieties). The ‘D’ or ‘E’ ‘boxes’ show maximum local (arch) thickness in the range 30cm to 70cm, and the number of bars in each layer (3 up to 10). The second line in each ‘box’ shows the c/c spacing of each S(fr) rib (range 4m down to

1m). The ‘boxes’ are positioned in the Q-support diagram such that the left side corresponds to the relevant Q-value (range 0.4 down to 0.001). Note energy absorption classes E=1000 Joules (for highest tolerance of deformation), 700 Joules, and 500 Joules in remainder (for when there is lower expected deformation).

Table 1: The year of 2014 was an appropriate 40-year milestone for again updating the ESR table published in 1993/1994. The ESR values recommended in 1993 are given in the table on the left. In a recent illustrated Q-manual (Barton and Grimstad, 2014) some updated ESR values tabulated on the right have been suggested, in order to reflect the increased conservatism in some sectors of civil engineering, when applying single-shell NMT. Note that most values of ESR are unchanged.

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

ESR recommended
ca. 2 to 5 (unchanged)
1.6 to 2.0 (unchanged)
0.9 to 1.1 **Storage caverns 1.2-1.3 (unchanged)**
Major road and rail tunnels **0.5 to 0.8**
0.5 to 0.8 (unchanged)

A final point for consideration in relation to all the experience-based case records behind Figure 11, is to note in wonder at the occasional attempts by Ph.D. students and their professors, who perform numerical analyses with rock mass strength criteria that do not describe rock mass strength correctly. Yet such authors make pronouncements that the empirically-derived rock bolt lengths must be increased because of their newly modelled ‘plastic zones’. Does a single numerical model, based on umpteen assumptions, really carry the weight to justify such changes, when there are some 2,000 case records behind Figure 11?

7 USE OF STEEL IN *SINGLE-SHELL NMT* AND *DOUBLE-SHELL NATM*

The use of steel sets or lattice girders should be avoided in the practice of single-shell NMT, due to the potential loosening of insufficiently supported rock in the periphery of the excavation. It is not part of the Q-system single-shell case-record data base, and has led to failures, occasionally very large failures. An unbolted lattice girder is a very poor substitute for RRS. Furthermore, it is difficult to make a *stiff contact* between the steel sets or lattice girders and the tunnel arch and spring-line, especially when there is over-break. The results of experiments using different support methods in the same rock quality are illustrated in Figure 12a. The left-hand diagram shows the results of trial tunnel sections in mudstone, reported by Ward et al. (1983). The five years of monitoring clearly demonstrate the widely different performance of the four different support and reinforcement measures.

In Figure 12b, from Barton and Grimstad (1994), the contrasting stiffness of B+S(fr) and steel sets is illustrated in a ‘convergence-confinement’ diagram, with the implication (and reality) that an elevated SRF (loosening variety) may occur when using steel sets. It should be clear that the early application of S(fr) by shotcrete robot, and the installation of permanent corrosion protected rock bolts from the start, as in single-shell NMT, is likely to give a quite different result from that achieved when using NATM principles and lattice girders.

In the latter, the commonly used steel sets or lattice girders, and mesh-reinforced shotcrete and rock bolts, are all considered just as temporary support, and are not ‘taken credit for’ in the design of the final concrete lining. These initial temporary support measures are assumed to eventually corrode. It is then perhaps not surprising that convergence monitoring is such an important part of NATM, as a degree of loosening seems to be likely when so often using steel sets or lattice girders as part of the temporary support. Both are very

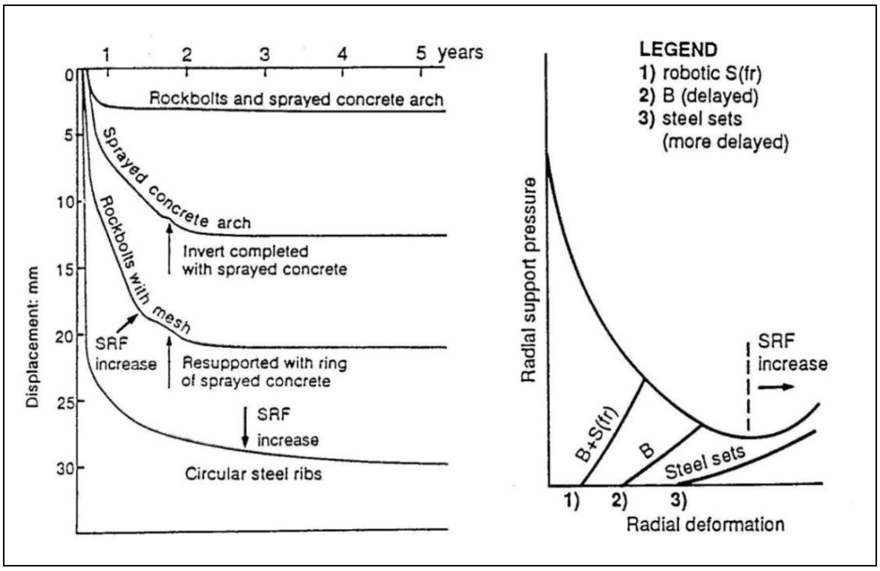


Figure 12 Left: Results of five years of monitoring test-tunnel sections in mudstone, using four different support and reinforcement measures, from Ward et al. (1983). The obvious superiority of B+S in relation to steel sets is clear. The last 35 years of B+S(fr), as practiced in Scandinavia, would presumably give an even better result. Right: Representation of the relative stiffness of different support measures, from Barton and Grimstad (1994). SRF is likely to increase due to loosening in the case of steel sets. However, in squeezing rock the application of RRS might also be a challenge, unless self-boring rock bolts were used to bolt the RRS ribs in the incompetent (over-stressed) rock that is likely to surround the tunnel in such cases.

deformable in relation to systematically bolted RRS arches. If lattice girders could be bolted there would be some improvement, but *intimate* contact with a tunnel arch exhibiting over-break remains a problem, even in the case of rectangular lattice girders with possible bolting plates. Standard NATM is shown in Figure 13.

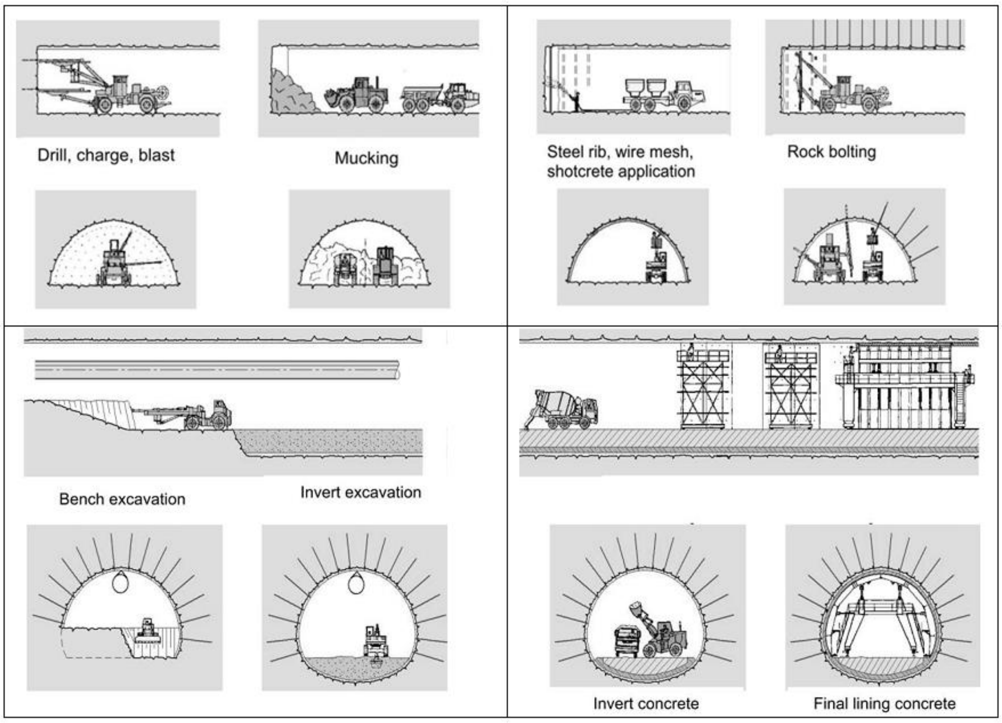


Figure 13 Schematic construction sequence of a typical NATM tunnel, apparently used in both softer and harder rock, from "Austrian Society for Geomechanics (2010). NATM, 'The Austrian Practice of Conventional Tunnelling'. This method has been observed in many countries when Q is 'poor', 'fair', 'good' i.e. Q = 1 to 40, where NMT would be eminently suitable and much faster and cheaper, and without the risk of weakening the rock mass with lattice girders.

8 CONTRASTING WATER CONTROL IN NATM AND NMT

Over-break is a key 'ingredient' in the thickness and especially the volume of S(fr) required in both NMT (*as permanent support*) and in NATM (*as temporary support*). Over-break is especially problematic *if* the time-consuming NATM tunneling method is to be used. This is because *water-proofing membrane* fixing and final concrete volumes (or increased volumes of over-break smoothing shotcrete) will be greatly affected by excessive over-break. There are in addition about 12 to 15 km of welds to be guaranteed in each 1 km of NATM tunneling requiring a drained membrane, so over-break is a complicating issue here too, as a '3D membrane' is needed where over-break is severe, unless large volumes of 'smoothing' shotcrete are used. Figure 14 illustrates the need of a local '3D' membrane in both cases.

The problem of excessive over-break due to joint set structure and joint roughness (or lack of roughness) easily doubles or triples the volume of shotcrete, and causes even larger increases in the volume of concrete, if double-shell (NATM) is to be the final stage of rock tunnel or metro-station cavern development. Concrete volumes will frequently be far higher than the 35 or 40 cm uniform thickness, which was designed and unrealistically drawn in designer's offices. Inevitably, over-break is of much less consequence with single-shell NMT tunneling, and indeed systematic pre-injection will tend to control it anyway, so the S(fr) volumes to ensure permanent stability remain moderate. The potential for cracking of the concrete may lead to subsequent problems in a cold climate such as in Norway. This is because of moisture and water frequently brought into the tunnel by wet cars, lorries or trains during bad weather. The water may subsequently freeze if the rain is followed by cold weather. This may gradually change an optimistically assumed 'maintenance-free' tunnel into one that is probably less capable of resisting the effects of aging than a single-shell NMT S(fr)-lined tunnel, since the poured concrete may be unreinforced, to save time and money – in the short term.

Figure 14: A degree of 3D adjustment is evident in these NATM-style membrane photographs, allowing the concrete to increase to at least 1 m thickness, instead of the 35 or 40 cm as typically designed in inevitably unrealistically uniform numerical models. An extreme challenge is shown in Figure 15. NATM would require a big budget for smoothing shotcrete, plus delays due to a 3D membrane, and finally a lot more concrete than in the numerical design drawing.

Thermal stresses resulting from widely different thicknesses of concrete (for instance when varying from a 'designed' 35 cm to 70 cm and even > 1 m) may cause cracking, if there is no reinforcement of the concrete. Unrealistic thinly-lined numerical models with an assumed uniformly thin shell all in compression will change considerably, if widely varying concrete thicknesses had been introduced at the modelling stage. This has yet to be seen. The consequences are cracking of the concrete, which is a long-term threat when ice can form.

In the case of *single-shell NMT tunneling*, the need for water control and an essentially dry tunnel is of course also a fundamental requirement. Various methods have been used, consisting among other things,

Figure 15: When tunnels have to link points A and B under a mountain, and the principal jointing and foliation have an adverse similar strike, sub-parallel to the tunnel axis, the best efforts of the contractor may not allow over-break to be reduced, if also fighting a $Jn/Jr \geq 6$ situation, as illustrated here with $Jn/Jr = 9/1$. What to do if a concrete-lined NATM tunnel was expected, and how to waterproof such a tunnel makes for interesting discussions. Of course pre-injection (to improve the effective Jn/Jr ratio (among other improvements), and use of single-shell NMT methods, especially S(fr) and never S(mr), is something to be seriously considered, since this alternative is much faster and also 75% cheaper.

of a free-draining shotcrete lining (with corrosion-protected bolting), combined with a membrane placed on the outside (the rock-side) of four-per-section bolted-and-sealed PC-element concrete panels. However in recent high-speed rail tunnels, the systematic use of *high-pressure pre-injection* has been the preferred method of trying to control the water. When this is carried out with due regard to stringent inflow limits (rather than just protection of the environment), and when stringent inflow limits like < 2 to 4 litres per min/100m are achieved, as in the case of the Bærum Tunnel of 5 km length, then an essentially dry tunnel is produced. The required injection pressures for successful hard-rock pre-injection are typically 5 to 10 MPa. Barton (2012b).

Humid patches of shotcrete could be observed in an extremely small percentage (0.0001%?) of the periphery of the Bærum Tunnel, despite the shale, limestone and dozens of permeable igneous dykes. With logical design strategies, eventual limited humid patches could be treated with easily inspected local ‘panels’ of sprayed membrane. Far lower tunnelling costs are associated with well-designed NMT than NATM.

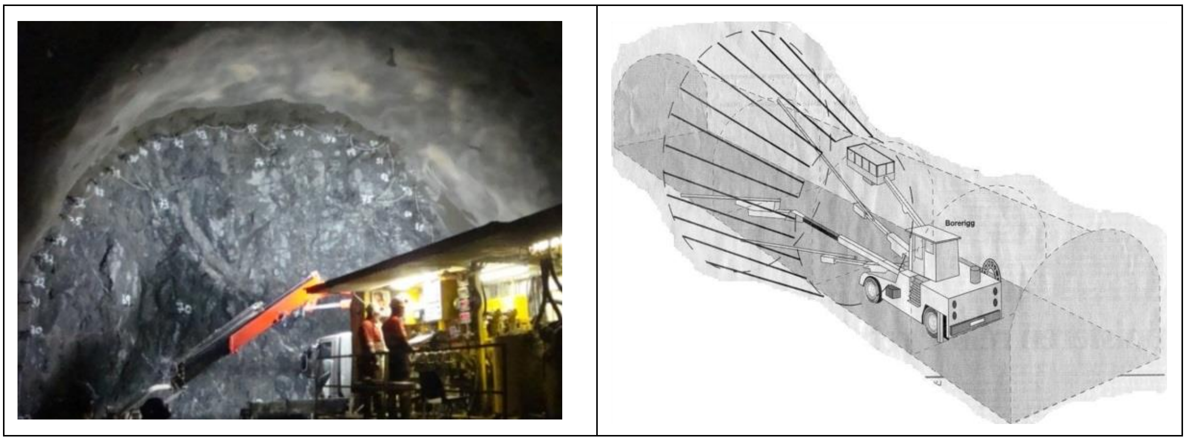


Figure 16: High pressure (5 to 10 MPa) pre-injection as often practiced in recent rail tunnels in Norway is found to seal effectively, if the appropriate grouting materials are used. The usual 1 to 6 litres of grout per m^3 of rock mass, injected into an approx. 6 m thick annulus, usually ensures only 2 to 4 litres/min/100m of water inflow, sometimes less. (The mean annulus thickness assumption is based on 4 to 5 m deep bolt holes seldom leaking, as opposed to numerous holes leaking, and pending court cases, when incorrect (low-pressure) methods are used).

Figure 17 Two tunnels where single-shell construction with sprayed-membrane for water control has been the successful solution. The tunnel on the left is from the Lausanne metro (photo: Karl Gunnar Holter), and the tunnel on the right is the Hindhead main road tunnel in England during final shotcreting. (Photo:Shani Wallace, Tunnel Talk, July 2011).

If some damp/humid patches remain, despite best pre-injection technology, then an attractive final solution is the local use of a sprayed membrane. Figure 17 illustrates two examples of systematically applied sprayed membrane, from a metro tunnel in Switzerland and a road tunnel in England.

It is interesting to note, and also a significant advantage, that the sprayed membrane which makes a shotcrete ‘sandwich’, has superior load-deformation characteristics. Samples with the membrane perform better in circular loading tests than the same thickness of S(fr) without the membrane. This is illustrated in Figure 18, modified from Holter and Nymoen (2009*)*. Although seldom used in Norway so far, though now undergoing seriously scientific field trials, a final local application of e.g. BASF 345 sprayed membrane in occasional humid areas, is all that is required for ensuring an economic, dry and easily inspected tunnel lining. (When/if an NATM liner leaks, the actual location of the membrane breach cannot normally be found).

The combination of single-shell NMT principles (Q-system logging, S(fr) + B support) with pre-injection and possible sprayed membrane for local humid zones, is attractive because of its relatively low cost and speed. Even with systematic pre-injection, and local sprayed membrane, NMT will still cost a fraction of NATM, typically 75% less (i.e. ¼) in countries where high labour costs are typical.

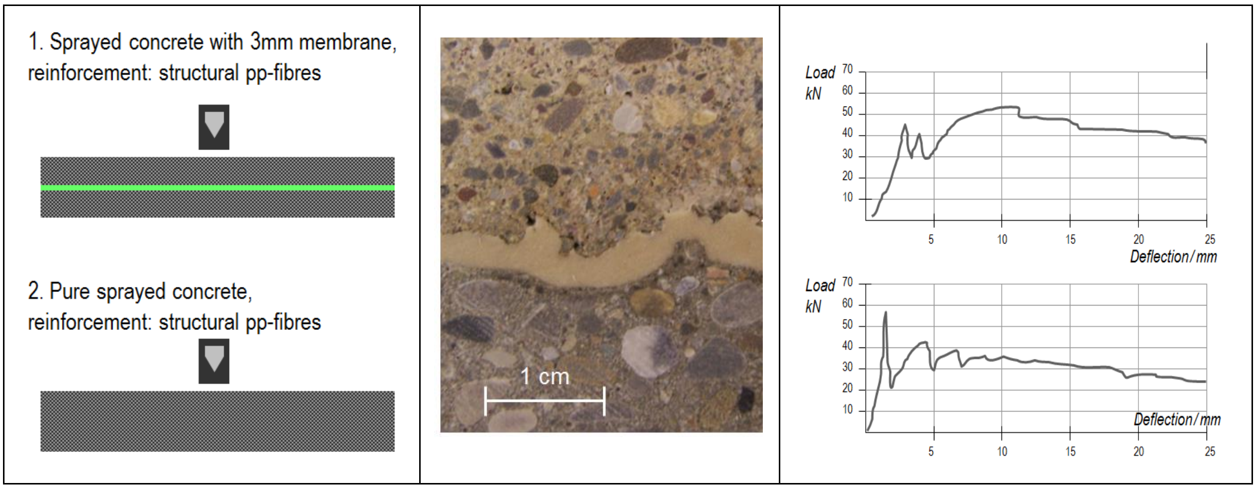


Figure 18 Sprayed membrane in a sandwich, using BASF 345 technology, from Holter and Nymoen, (2009). Note the use (in this case) of polypropylene fibres. The membrane is formed from powder and water, and can be sprayed with the same robotic equipment as the S(fr). The loading test results indicate that the presence of the membrane is positive for two reasons: improved S(fr) toughness (fracture energy) and obviously the expected water-proofing.

By chance and not as originally intended, the six Q-system parameters can each 'benefit' from pre-injection. This was proposed in Barton et al. (2001), and Barton (2002*)* and detailed more thoroughly, in Barton (2012b). For instance there can be increases in effective RQD and reduced effective Jn as the most permeable and/or least stressed joint sets are successively sealed. This mechanism is 'seen' in the form of pressure plateaux. The least favourable Jr and Ja may then apply to a different *less unfavourable* joint set. Of course Jw will tend towards a value 1.0. Even SRF will sometimes benefit from pre-injection.

The net result is a large apparent increase in Q, usually from 20-times to 50-times, e.g. Q = 0.1 to 2, or Q = 0.1 to 5. An exceptional example would be a "sugar cube rock" (Jn=15 and low RQD) with little clay, which may act as more or less massive rock after pre-injection, with effective RQD/Jn increasing from 10/15 to perhaps as much as 90/3. The significance of these internal grouting-induced changes in properties such as velocity, deformability, and inflow (obviously) are starting to be documented also by others, in publications from several countries, in mining and civil (dam) engineering projects.

9 GJØVIK CAVERN Q-LOGGING AND NMT SINGLE-SHELL B+S(fr)

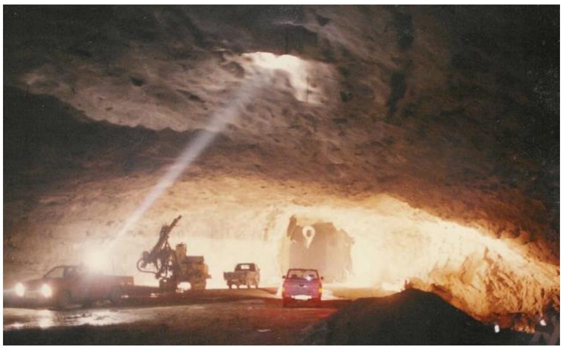
The Gjøvik Olympic cavern was a milestone event in Norwegian rock engineering and in rock mechanics practice, combining as it did the experience of several of Norway's leading consulting companies, research institutes and contracting companies. The Q-system was well utilized throughout the process, with Q-logging of the four drill-core, and also Q-logging of rock which was visible in existing caverns in the same small 50m high hillside. The large top-heading and cavern arch were of course extensively Q-logged.

The average Q-quality was 10 to 12, with a range of about 2 to 30 (poor to good). Details of all three Q-applications, and numerical modelling predictions of the measured deformations were given in the multi-author publication of Barton et al. (1994). The use of seismic cross-hole tomography was also demonstrated, including the effect of stress on increased velocity and deformation modulus. Figure 19 is a concentrated summary of some of the Gjøvik cavern experiences: over-break with RQD = 60-90, Jn/Jr = 12/2, Q-values, instrumentation, support, preliminary rounds of (dry) Olympic ice-hockey under a drained arch of 60 x 90 m.

10 ESTIMATING TUNNEL OR CAVERN DEFORMATION

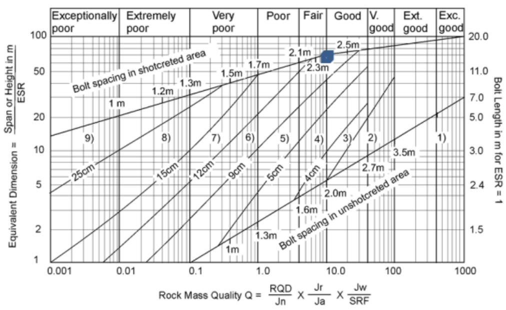
It appears that the large numerical range of Q (0.001 to 1000 approx.) referred to in the introduction, helps to allow very simple formulæ for relating the Q-value to parameters of interest in rock engineering performance assessment. Tunnel or cavern deformation is an example. The following method, illustrated with two 20-year-old examples has also been tested in some more recent Hong Kong caverns, and it out-performed UDEC analyses, because of exaggerated joint continuity in various consultants' models. Deformations up to 10-times those measured were modelled: the empirical method based on numerous core-logged Q-histograms was almost correct, and showed much smaller deformations: in fact about ½ to 1/3 those of the 3-times larger Gjøvik cavern.

Empirical improvements to the preliminary and simplistic ($\Delta \approx$ SPAN/Q*)* model shown in Figure 20, were made by employing the 'competence factor' format of SRF (i.e. stress/strength). The alternative and slightly more sophisticated formulæ which are also shown, should be tested when checking the reality of numerical modelling, as there are many examples of unrealistic modelling proffered by young authors as supposedly superior to empirical methods. Some of the errors made by them are use of too continuous (pseudo-) jointing, and deformation moduli without correction for depth. Both such errors, and indeed the reliance on adding 'c' and 'σ_n tan φ' in linear Mohr-Coulomb and non-linear Hoek-Brown formulations, are strong reasons for modelling 'bigger effects' (larger deformations, deeper 'plastic zones') and their presumed 'disqualification' of empirical methods. Reality may be (is usually) different from what is seen in colourful models, especially today's continuum models.

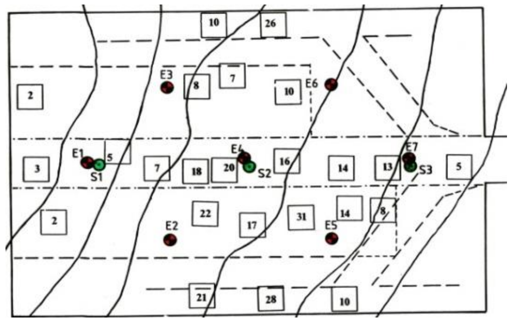


World’s largest top-heading waiting for benching-down.

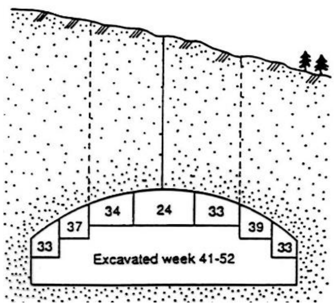
Mean 10 cm of S(fr). Note some over-break > 1 m.



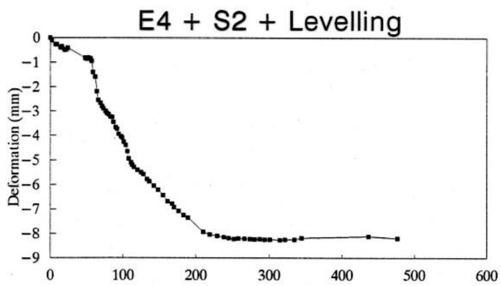
$Q_{mean} \approx 10$, span 62m. B 2.5m c/c + S(fr)10cm + cables.



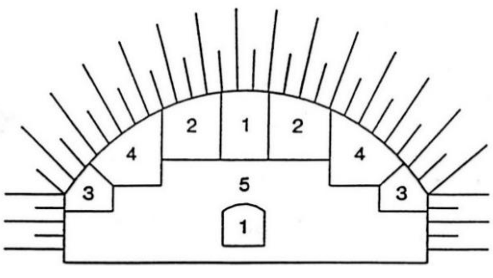
Q-values of arch (boxes): long external MPBX: in red



MPBX locations (three rows), excavation week in 1991.



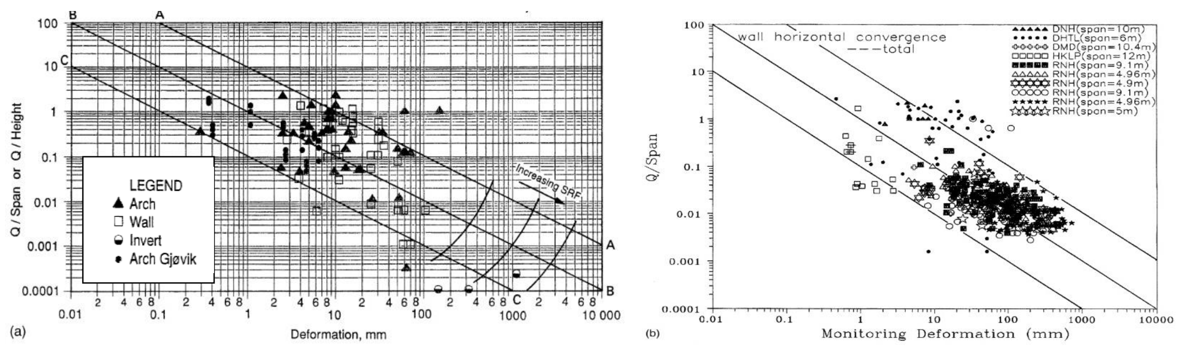
Central deformation 8 mm, ends: 7.0 mm and 7.5 mm



B 2.5m c/c L= 6m, + twin-strand anchors L=12m, c/c 5m

Early games: ice hockey in Winter Olympics of 1994.

Figure 19: Some details of the Gjøvik cavern top-heading, degree of over-break (therefore of course S(fr), position on Q-support chart, Q-value range (2 to 30, mean 10 to 12 from cavern and core-logging, MPBX example (7 to 8 mm after 200 days, as in Type-A prediction modelling with UDEC-BB). Horizontal stress levels were 3 to 5 MPa (favourably high) and P-wave velocities 3.5 to 5.5 km/s increase with depth as a result, despite no Q value improvement with depth.



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

Nathpa Jakri power station cavern

$\Delta_v = \dfrac{20{,}000}{100 \times 3} \times (6/35)^{1/2} = 28$ mm

$\Delta_h = \dfrac{50{,}000}{100 \times 3} \times (4/35)^{1/2} = 56$ mm

(SPAN = 20m, HEIGHT = 50m, Q = 3, σ_V=4 MPa, σ_h = 6 MPa, σ_C = 35 MPa). (In the middle of the range of MPBX measurements for the arch and walls).

Gjøvik Olympic cavern

$\Delta_v = \dfrac{60{,}000}{100 \times 10} \times (1/75)^{1/2} = 6.9$ mm

(SPAN = 60m, Q_{mean} = 10, σ_V = 1 MPa at 40 m depth, σ_C = 75 MPa) (Almost identical to that measured with nine MPBX, and almost identical to UDEC-BB modelling results).

Figure 20: Top: The log-log plotting of Q/span versus deformation was published in Barton et al. (1994), with fresh data from the MPBX instrumentation of the top-heading and full 62 m span of the Gjøvik Olympic cavern. Shen and Guo (priv. comm.) later provided similar data for numerous tunnels from Taiwan. When investigated, the central trend of data was simply Δ(mm) ≈ SPAN(m)/Q.

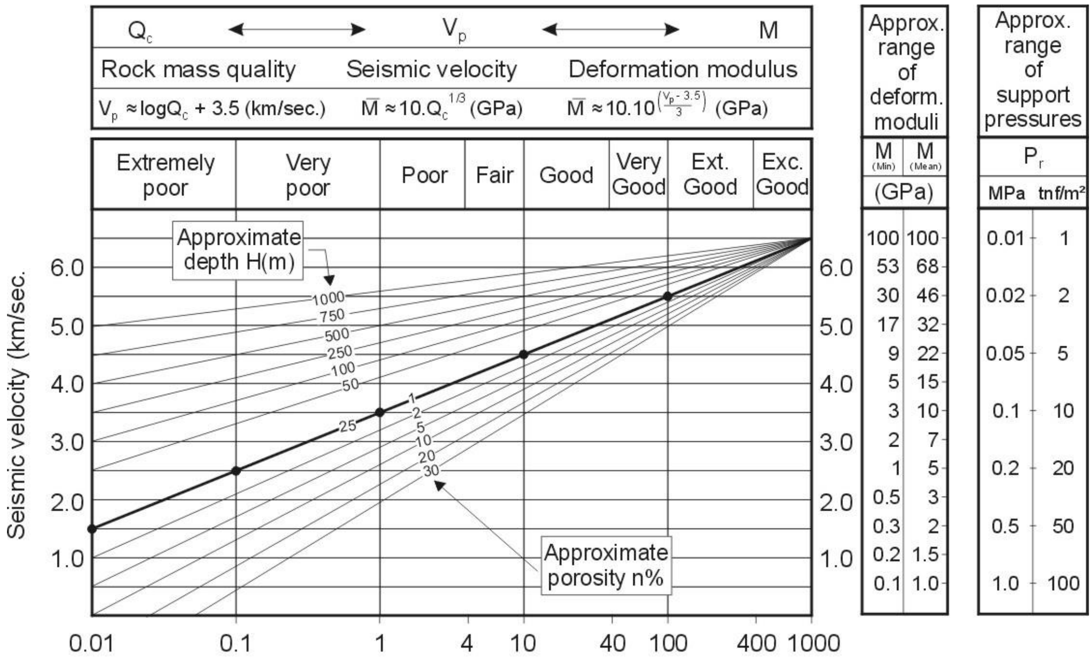
11 CORRELATING Q WITH P-WAVE VELOCITY AND MODULUS OF DEFORMATION

An empirically-based correlation between the Q-value and the P-wave velocity derived from shallow refraction seismic measurements was developed by Barton (1995) from trial-and-error lasting several years. The velocities were based on a large body of experimental data from hard rock sites in Norway and Sweden, thanks to extensive documentation by Sjøgren et al. (1979*)*, using seismic profiles (totaling 113 km) and local profile-oriented core logging results (totaling 2.85 km of core). The initial V_P-Q correlation had the following simple form, and was relevant for *hard rocks with low porosity*, and specifically applied to shallow refraction seismic, i.e. 20 to 30m depth, as suggested by Sjøgren.

$$V_P \approx 3.5 + \log Q \quad \text{(units of velocity: km/s)} \qquad (1)$$

A more general form of the relation between the Q-value and P-wave velocity, shown in Figure 21, is obtained by normalizing the Q-value with the multiplier UCS/100 or σ_c/100, where the uniaxial compressive strength is expressed in MPa (Q_c = Q x σ_c/100). The Q_c form has more general application, as weaker and weathered rock can be included, with a (-ve) correction for porosity. For a more detailed treatment of seismic velocity, such as the effects of anisotropy which are accentuated when the rock is dry or above the water table, refer to the numerous cases from numerous authors reviewed and summarized by Barton (2006).

$$V_P \approx 3.5 + \log Q_c \quad \text{(units of velocity: km/s)} \qquad (2)$$



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

Figure 21 The thick 'central diagonal' line applies to nominal 25-30 m depth shallow seismic refraction results. In practice the nominal 1% (typical hard rock) porosity would be replaced by increased porosity if rock was deeply weathered, and the more steeply sloping lines (below the 'central diagonal') would then suggest the approximate (-ve) correction to V_P. Note that very jointed rock with open joints may have even lower velocity than saturated soil. (A 'loosening' SRF as high as 5 might then be appropriate, though might be insufficient adjustment of Q for something that is more than just low stress). The less inclined lines above the 'central diagonal' represent greater depth (50, 100, 250m etc), and these lines correct V_P for documented stress or depth effects (+ve). These depth-corrected lines were derived from several sets of deep cross-hole seismic tomography, with independent Q-logging of the respective 11 km of drill-cores by former NGI colleagues, at the UK Nirex site in NW England.

The simplest empirical equation for support pressure (P_{arch}) from Barton et al. (1974) was as follows, when converted from units of kg/cm^2 to MPa. (Note that there was an incorrect inversion of this equation in Barton (2002). A more accurate equation involving Jn as a separate parameter does not need inclusion here.

$$P_{arch} \approx 0.2\ Jr^{-1}\ Q^{-1/3} \qquad (3)$$

The fact that no scale effect is implied in equation 3, contrary to e.g. Terzaghi's USA (and Indian hydropower) experiences with steel sets, was deliberate. We only need to look at the 60 m span Gjøvik cavern, and the 10 cm of S(fr) and B 2.4 to 2.5m c/c permanent support used there, to realize that modern tunnel support methods do not invite deformation (and therefore scale effects) in the same adverse way as steel arches or lattice girders. An independent approximation to the static deformation modulus (M: see Figure 21, the left-hand table), suggest an approximately inverse relationship between *support pressure needs* and rock mass *deformation moduli*. This surprising simplicity is logical. It specifically applies with mid-range Jr = 2 joint roughness.

Useful equations derived from Q_c concerning deformation modulus (M or) E_{mass} are given below. There are several possible equivalent forms, and V_p can be used in place of Q or Q_c if need be or if available.

$$E_{mass} \approx 10Q_c^{1/3} \text{ or } E_{mass} \approx 10^{(Vp - 0.5)/3} \text{ or } E_{mass} \approx 10^{(Vp - 2.5 + \log \sigma c)/3} \qquad (4a, 4b, 4c)$$

where Vp is in km/s, E_{mass} is in GPa, and σ_c is in MPa. For instance with Q = 10 and σ_c = 100 MPa and Vp = 4.5 km/s, one obtains $E_{mass} \approx 22$ GPa using all three equations (4a, 4b, 4c). This corresponds to the nominal 25 m depth (shallow seismic refraction) 'central diagonal' in Figure 20.

If Q was unknown, a higher Vp of say 5.5 km/s (because of a deeper more highly stressed location) suggests $E_{mass} \approx 46$ GPa. In the Gjøvik cavern modelling with UDEC-BB, deformation moduli of 20, 30 and 40 GPa were modelled at increasing depth due to the measured increased velocity with depth. The Q-value, RQD and joints/meter had shown no improvement with depth. The measured vertical cavern deformation of 7 to 8 mm was numerically modelled very accurately, and was also confirmed empirically, which is always an important reality check (see Figure 20, bottom-right box-insert).

12 THE Q-VALUE APPLIED TO TBM PROGNOSIS VIA Q_{TBM}

This Hong Kong conference on Underground Design and Construction has two eminent keynote speakers from competing TBM manufacturers Robbins and Herrenknecht. As the third keynote lecturer with frequent drill-and-blast *and* TBM duties as a consultant, one should perhaps hesitate to touch on the subject of TBM. Nevertheless, out of curiosity it is interesting to present TBM performance of the very best quality – namely Robbins' numerous world records – in a format that one does not yet see in TBM journal reporting.

To do this a log-log-log ('three-way' guarantee of linearity) plot is employed, as used in Barton (2000). The fact that two performance characteristics (penetration rate PR and advance rate AR) *and* time are each plotted on a $\log_{10}$ scale may have discouraged general use. But the chart is simplicity itself – and very useful. It also may help to separate open-gripper and double-shield ('push-off-liner') TBM performance, at least if the latter is a favourable choice for the particular rock mass. When changing cutters every 1 to 3 m (i.e at least 10 in every maintenance shift), in exceptionally massive (high-Q) hard abrasive rock, the expected improved efficiency of the double-shield machine may not be so obvious, and even drill-and-blast (if of best quality) becomes a viable alternative. Key factors to investigate are the amounts of very high *and* very low Q-values.

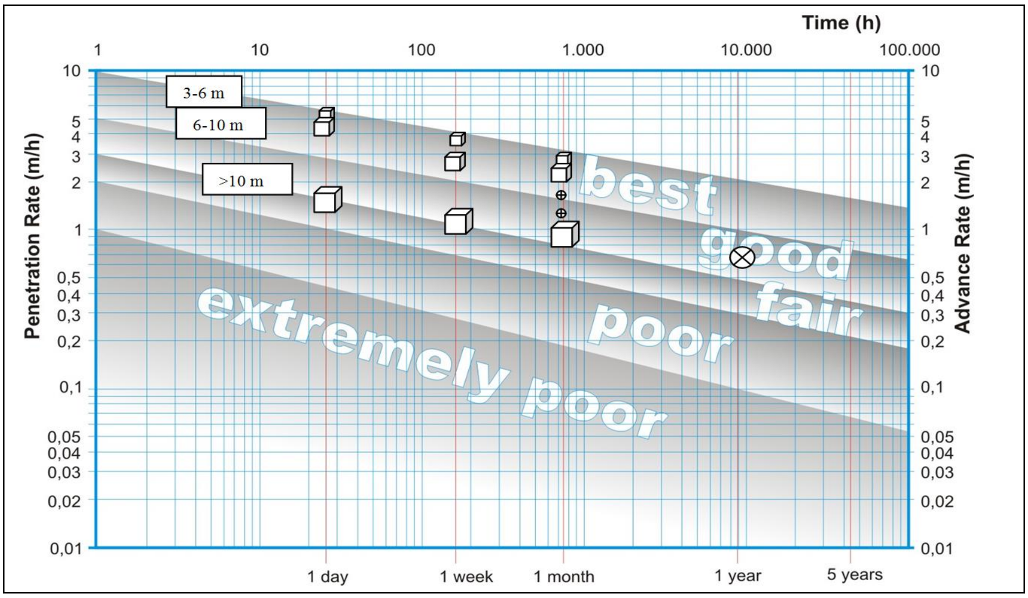


Figure 22: Using a log-log-log plot of PR (penetration rate, left axis only) and AR (advance rate in remainder of plotted area) and time T (total hours), the synthesized present world-record data for different sizes of TBM is shown, based on data provided by Robbins, for most sizes of TBM. The writer has converted day, week and month records (given in meters) to the form AR (m/hr) by dividing by assumed 24, 168 and 720 hours. Data from 8 countries are represented, chiefly USA and China. The record mean monthly data plots at AR = 1.7 m/hr for the 3 to 6m class, and at AR = 1.1 m/hr for the 6 to 10m class, and this is shown with two small circles. The larger crossed-circle to the right is 54 weeks for 5.8 km at Svea Tunnel, achieved during the Norwegian LNS drill-and-blast world record. This was driven in coal-measure rocks and obviously required significant shotcreting and bolting, due to varied Q.

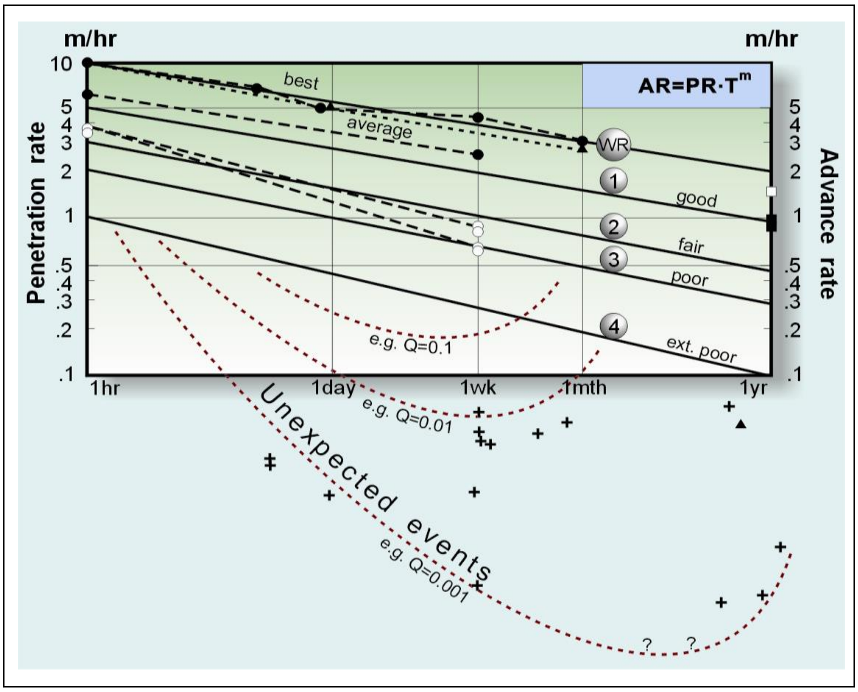


Figure 23: Trends from open-gripper case records representing 145 cases and approximately 1000km of TBM tunneling. The five typical 'lines' of performance are the same as shown in Figure 22, but here only extended to 1 year. The source of this smoothed data is given in Barton (2000, 2014). Note substitution of AR = PR x U with AR = PR x T^m where T is hours. In other words, due to the time-deceleration trends always seen, utilization is (obviously) time-dependent.

Where the Q-value comes into the prognosis, at least in many older TBM case records, is in the 'unexpected events' area of Figure 23. The three distant crosses (on the far right) are now permanently buried TBM. Perhaps 'today' with cross-over i.e. better equipped TBM, they would have avoided burial? But they were very tough cases, with the complication of high-pressure water and faulted crushed rock and clay.

So the key question for the TBM manufacturers is whether the best present-day TBM have really removed such cases (the *crosses* in Figure 23). The gradual increase in the number of holes in the forward shield for pre-injection has been noted.with satisfaction, during the last 10 to 15 years, Advanced 'cross-over' TBM apparently now have many of the facilities available with multi-boom jumbos. In fact one TBM manufacturer (at least) seems now to be claiming that TBM should be used both 'because the rock is so bad' *and* 'because the tunnel is so long'. This is a challenging viewpoint, and is discussed in detail in Barton 2012c. The question that remains is whether/how much updating is needed in the two following Q_{TBM} related Figures 24 and 25?

Table 2 Deceleration gradients (-m) for the five trends-of-performance lines in Figures 22 and 23. A specific 56 km of double-shield performance (two Wirth TBM, two Herrenknecht TBM at the twin Guadarrama Tunnels, Spain) is also indicated, but this halving of gradient (-m) may not apply in tough double-shield cases, if there is need for frequent cutter-change. An obviously slower EPB machine may double these gradients (Barton, 2013).

PERFORMANCE LINE #	**DECELERATION Gradient (-) m (units of LT^{-2})**
WR (world records) **1, 2, (good, fair)** **3, 4** **(poor, extremely poor)** **(trends from 145 cases)**	**-0.13 to -0.17** **-0.17, -0.19** **-0.21, -0.25** **(ca. 1000 km of mostly** **OPEN-GRIPPER cases)**
DOUBLE-SHIELD (at Guadarrama)	**-0.08 to -0.12 (4 x 14 km)**

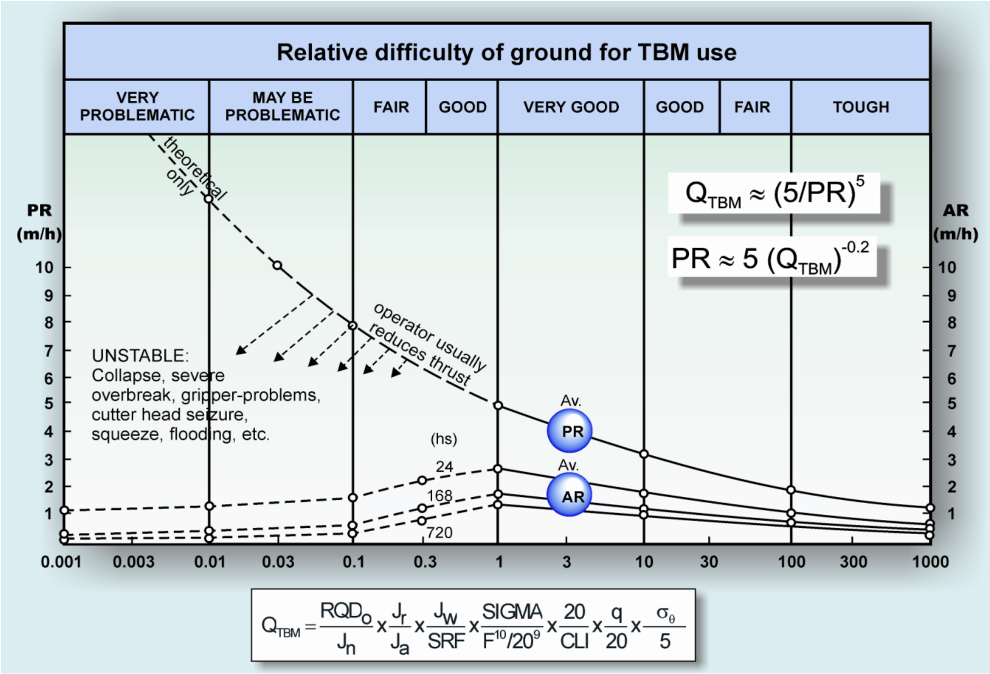


Figure 24 The Q $_{TBM}$ model for TBM prognosis involves an oriented Q_o-value and machine-rock interaction parameters given in normalized form. The Q_{TBM} value is (adversely) increased if CLI (cutter life index) is <20, if q (quartz content %) is >20, and if the estimated σ_θ (biaxial stress state on tunnel face) is more than 5 MPa (the estimated value at 100 m tunnel depth). Note that curves representing AR estimation for 24 hrs, 1 week, 1 month are separated, because of declining utilization (as in Figures 22 and 23). Note the new ‘adjectives’ specifically for TBM. It is clear that central Q $_{TBM}$ values of approx. 0.3 to 30 would be ideal for fast progress. One of the most important normalized parameters is mean cutter thrust (F, tons) which is normalized by 20 tons. Greater or lesser applied thrust is then compared with SIGMA (rock mass strength estimate = $5\gamma Q_c^{1/3}$) where Q_c = Q_0 xUCS/100. A cutter-force comparison to rock mass compression strength is not made in the NTNU prognosis, and can result in surprises when rock is very hard (γ = density in gm/cm^3).

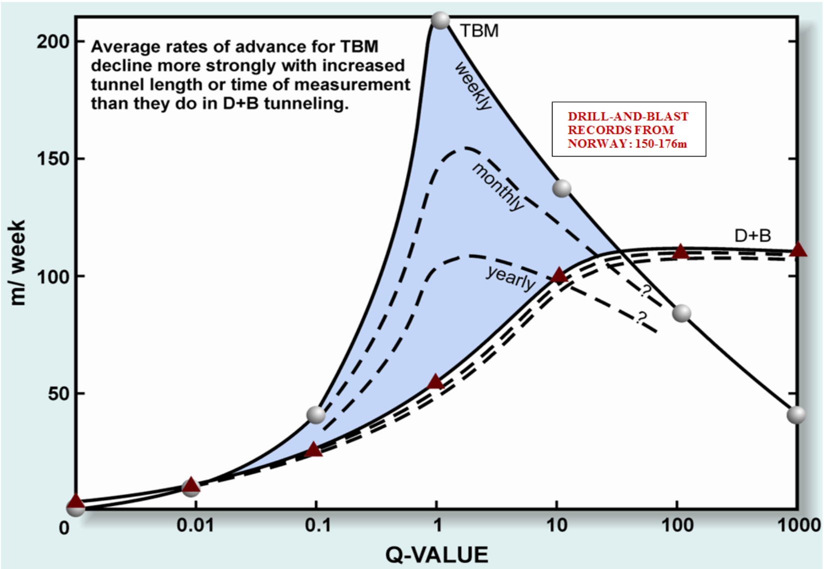


Figure 25 This comparison of TBM and drill-and-blast rates of advance, based on a moderate (6 km in one year) TBM prognosis, and comparison with Norwegian drill-and-blast cycle-times per Q-value-class measurements, suggests two important things. The longer the tunnel the more the need for central (well jointed but not faulted) rock qualities, if the TBM is going to remain much faster than drill-and-blast. Q-values consistently higher than 100-200 suggest drill-and-blast superiority, bearing in mind recent records of 150, 165 and 176m in best weeks, shown in the red box. LNS, a contractor based in Northern Norway, achieved a remarkable 104 m/week (mean) for a mine access tunnel of 5.8 km length in coal-measure rocks. Q-values were probably mostly 1 to 10, but sometimes 0.1 to 1 and needing some support.

13 CONCLUSIONS

1. The Q-system has weathered the test of time and has had application in tens of thousands of civil and mining engineering projects in a large number of countries during the last four decades. Besides its extensive use in rock mass characterization and 'rock class' determination, it is traditionally most strongly linked to single-shell NMT permanent *tunnel and cavern* support and reinforcement selection. This is due to the large number of (mostly Norwegian) case records on which it is now based.

2. The selection of support and reinforcement 'class' cannot be done by 'finite element modelling' as suggested by a prominent international consultant, when advancing at an average 60 to 80 m/week. At the occasional world record speed for drill-and-blast of 160 or 170 m/week, Q will obviously not be needed either, as such rates usually imply stable rock masses requiring only sporadic support.

3. The Q-system can also be used in the selection of *temporary support* (for NATM) if desired. This is a method used for several decades in Hong Kong metro and road tunnels where Q is used exclusively. For *temporary support*, 5Q and 1.5 ESR are used. Road and rail authorities in Norway however, require Q to be used for selecting *permanent* single-shell support, whatever water-proofing method is used.

4. When a truncated 'mining version' of Q is used in helping to design dimensions for non-entry stopes and minimize dilution of the ore, the form Q' = RQD/Jn x Jr/Ja is immediately in need of the strength/stress ratio. What has not yet been 'replaced' is the ability to describe water and faulting.

5. In relation to 'competing classification methods' such as RMR (and the numerically closely related GSI of Hoek) the Q-system appears to have the advantage of a logarithmic type of quality scale. In addition it has some important parameters like number of joint sets (Jn) and inter-block friction (Jr/Ja) and ways to evaluate the stress/strength ratio (SRF) seemingly absent in these other methods.

6. The less common ratio Jn/Jr is closely related to the amount of over-break, which is very likely to occur when Jn/Jr $\geq$ 6. Shotcrete (or concrete) volumes, and related difficulties with 3D membrane construction in the case of NATM will therefore also be linked to Jn/Jr when it is adverse (> 6).

7. The Q-values and their statistical variation have important roles to play during site investigation, using core-logging and rock-exposure logging, such as new road cuts. Since Q is correlated with seismic P-wave velocity, interpolation of quality can be made between boreholes if seismic refraction profiling is performed, or utilizing cross-hole seismic tomography in critical zones like faults or low rock cover. The effect of depth or stress on V_P must not be ignored.

8. Q also has simple direct links to deformation and to deformation modulus, and each are depth and/or stress dependent. The support capacity equation based on Q suggests an inverse relation to the deformation modulus. Q can also be linked to the estimation of a depth-dependent permeability via a modified parameter Q_{H2O} which has Jr/Ja inverted to the form Ja/Jr.

9. Q has been used on numerous occasions as a basis for realistic TBM prognosis via Q_{TBM}. This empirical model includes some important rock-cutter interaction parameters, like NTNU's CLI. When predicting *penetration rate*, care is needed to match cutter force to an estimate of rock mass strength, which varies from about 1 to 100 MPa, depending on Q and UCS. Without this comparison, unexpected errors in prognosis of PR may occur in very hard rock. This fact seems to be unknown.

10. Unexpected delays, and occasional permanent burial of TBM (or the need for more 'cross-over' facilities on the TBM) are strongly linked to adverse Q-values. Steep deceleration gradients and slower *advance rates* in 'bad ground' link directly to low Q-values. The writer is aware of impressive improvements in TBM performance capabilities for bad ground in the last decade, including many more holes for pre-injection. This particularly necessary improvement has been a long time arriving.

14 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., Løset, F., Smallwood, A., Vik, G., Rawlings, C., Chryssanthakis, P., Hansteen, H. & Ireland, T. 1992. Geotechnical core characterisation for the UK radioactive waste repository design. Proc. of ISRM Symp. EUROCK, Chester, UK.

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

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. 1996. Investigation, design and support of major road tunnels in jointed rock using NMT principles. Keynote Lecture, *IX Australian Tunnelling Conf*. Sydney, 145-159.

Barton, N., Buen, B. & Roald, S. 2001. Strengthening the case for grouting. Tunnels & Tunnelling International, Dec. 2001: 34-36, and Jan. 2002: 37-39.

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. 2006. *Rock Quality, Seismic Velocity, Attenuation and Anisotropy*. Taylor & Francis, UK & Netherlands, 729.

Barton, N. 2007. Near-surface gradients of rock quality, deformation modulus, Vp and Qp to 1km depth. *First Break, EAGE*, October, 2007, Vol. 25, 53-60.

Barton, N. 2012a. Defining NMT as part of the NATM SCL debate. *Tunnel Talk*, Sept. 2012, 1-4.

Barton, N. 2012b. Assessing Pre-Injection in Tunnelling. Tunnelling Journal, Dec.2011/Jan. 2012, 44-50.

Barton, N. 2012c. Reducing risk in long deep tunnels by using TBM and drill-and-blast methods in the same project – the hybrid solution. Keynote lecture. Risk in Underground Construction. J. Rock Mech. and Geotech. Eng., 4(2): 115-126.

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, 1095-1112.

Barton, N. 2013. TBM prognoses for open-gripper and double-shield machines: challenges and solutions for weakness zones and water. Fjellsprengningsteknikk, Bergmekanikk, Geoteknikk, Oslo, 21.1-21.17, Nov. 2013.

Barton, N. and Grimstad, E. 2014. An illustrated guide to the Q-system following 40 years use in tunnelling. Page 4 of downloads*: www.nickbarton.com. 43p.*

Barton, N. 2014. Lessons learned using empirical methods applied in mining. Keynote lecture. 1st. Int. Conf. on Applied Empirical Methods in Mining. Lima, Peru, 24p.

Bhasin, R.K., Barton, N., Makurat, A., Davies, N. & Cooper, A. 1999. Rock joint and rock mass characterization at Sellafield. ITA Congress, Oslo. Challenges for the 21st Century. Eds. Alten et al. NFF, Balkema, Rotterdam, 35-41.

Cecil, O. S. 1970. Correlation of rock bolt and shotcrete support and rock quality parameters in Scandinavian tunnels. Ph.D. Thesis, Univ. of Illinois, Urbana, 414p.

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.

Grimstad, E., K. Kankes, R. Bhasin, A. W. Magnussen and A . Kaynia, 2002. Rock Mass Q used in designing Reinforced Ribs of Sprayed Concrete and Energy Absorption" *4th Int. Symp. on Sprayed Concrete*, Davos, Switzerland.

Grimstad, E., K.Kankes, R. Bhasin, A. W. Magnussen and A.Kaynia, 2003. Updating the Q-system for Designing Reinforced Ribs of Sprayed Concrete and General Support. Proceedings, *Underground Construction*, London.

Grimstad, E. 2007. The Norwegian method of tunnelling – a challenge for support design. *XIV European Conference on Soil Machanics and Geotechnical Engineering.* Madrid.

Holter, K.G. and Nymoen, B. 2009. Permanent waterproof tunnel lining based on sprayed concrete and spray-applied double-bonded membrane. *Proc. of ITA Congress*, Helsinki.

Vandevall, M. 1990. *Dramix - Tunnelling the World.* NV Bækert S.A, 1991 edition.

Ward, W.H., P. Todd., N.S.M.Berry. 1983. The Kielder Experimental Tunnel: Final Results. *Geotechnique* 33.3, 275-291.

Sjøgren, B., Øfsthus, A. & Sandberg, J. 1979. Seismic classification of rock mass qualities. *Geophysical Prospecting,* 27: 409-442.