FORTY YEARS WITH THE Q-SYSTEM IN NORWAY AND ABROAD
FØRTI ÅR MED Q-SYSTEMET I NORGE OG I UTLANDET

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SUMMARY
This paper describes some of the lessons learned during four decades of application of the Q-system. It was first used in hydropower projects in Norway and in a water transfer project in Peru in 1974. Some years afterwards, application in Norwegian road tunnels followed. Personal application by the two main developers of the method 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 numerous 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.

1. INTRODUCTION

Development of the Q-system during six hectic months in 1973 started as the result of a question from NVE (Statkraft) to NGI. The first author could not answer the question, so started developing a rock mass classification method, linked to support needs. RQD/Jn came first, with successively added parameters, and a lot of trial-and-error and empiricism using more than 200 case records. This finally enabled an answer to be given to the challenging question from Statkraft (NVE): Why all the different deformation magnitudes in Norwegian hydropower machine halls? So 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 non-empirical failure criterion, used by so many optimistic continuum modellers. Since rock mass classification was ‘not supposed to be possible, and can therefore never be developed’ (roughly the opinion expressed in NTH/NTNU Norwegian engineering
geology course notes for several decades), it can be imagined the enthusiasm from Trondheim
and other quarters, when Q was launched in 1974.
Besides engineering colleagues at NGI (principally Lien and Løset in the first years), the
Swedish contractor Skanska was among the first outside Norway to test the method in the
ensuing months, in the Majes project in Peru. 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 in excess of 10 to 20,000 m² present challenges in minimizing dilution.

2. THE 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 $Q_c = Q \times UCS/100$ introduced by Barton
(1995), the $Q_c$ 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/m². 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 (Barton, 2014).

Figure 1 Contrasting worst ($Q \approx 0.001$) and best ($Q \approx 1000$) rock mass qualities. Examples from
Brazil (2) Sweden and Hong Kong (the latter due to an unrepresentative vertical hole). The core
box with core loss is from the wide regional shear zone at Hallandsås, in S.W. Sweden.
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 Qc to velocity, deformation modulus, and deformation.

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.

\[ \text{Figure 2 Overbreak is extremely likely to occur despite a contractor’s efforts with careful blasting, if the most frequent value of the ratio Jn/Jr ≥ 6, i.e. 6/1, 9/1.5, 9/1.0, 12/2, 12/1.5, 12/1.0, 15/1.5. Visible half-rounds or at least lack of overbreak will tend to be found when Jn/Jr < 6, such as 3/3, 4/2, 6/1.5, 9/2, 9/3, 12/3, 15/3. Lack of block structures combined with dilatant joint roughness or discontinuous joints prevent its occurrence. All half-rounds (and virtually no overbreak) will appear with Jn/Jr = 2/3, or 2/4 which would be typical of a massive granite with discontinuous jointing.} \]

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.

The writers have both worked mainly in civil engineering projects. Nevertheless on occasion there has been the opportunity to apply ‘civil engineering’ methods to mining problems. The open-stope case shown in Figure 3 was sketched on over-heads (25 years ago), to represent over-break situations in long-hole drilling drifts in the LKAB Oscar Project. A scale effect is anticipated. It shows one of the first applications of Q-parameter histogram logging (in 1987).
Figure 3 Observations of excessive over-break and even failure in some long-hole drilling drifts. Note that the most frequent Q-parameter statistics (Jn/Jr = 9/2) correctly suggest few problems. It is the tails of the distributions (e.g. Jn/Jr = 12/1.5) well assisted by locally too high Ja (clay-filling), which cause the worst cases of over-break. (The photograph lower-right is a disused 120 year-old limestone mine with ‘stable’ blocks in the corner of the room-and-pillar drift, due to Jn/Jr = 9/3 (i.e. despite three sets of joints, Jr is sufficiently rough and dilatant to have prevented fall-out all these years).

4. Q-PARAMETER ZONING IN MINING AND CIVIL PROJECTS

The writers have 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 Figures 3 and 4. Even the limited number of observations in Figure 3 are useful. However, we know of a project with > 300 km of core-logging-based Q-value statistics in a new gold mine. Here the visual (non-EXCEL spreadsheet) visualization of data is important, and assists auditing of the huge body of data.
<table>
<thead>
<tr>
<th>$Q$ = 0.1 to 1.0</th>
<th>$Q$ = 1.0 to 4</th>
<th>$Q$ = 4 to 10</th>
<th>$Q$ = 10 to 40</th>
</tr>
</thead>
</table>

Figure 4  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, numbering approximately 6,000, 10,500, 18,000, 10,400, demonstrate the central role played by RQD in commonly experienced rock mass conditions. The occasional critique of RQD and incorrect assumption 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 80, 90 and 100 often causing problems with TBM penetration rates in hard rock but limited tunnel support needs, usually correspond to quite massive conditions. $RQD = 90$ and 100 very seldom means ‘11 cm block sizes’ as implied by some critics of RQD.

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 5 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 > 3.

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 river valleys, the ‘absence’ of Jrw is also unfortunate, as personally experienced.
It is always important to be aware that the six Q-parameters are only an abbreviated description of the rock mass, as can be seen in Figure 6. 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 6 (III) is also beneficial, as the potential variation of most (all?) parameters is important. A very deep tunnel with variable rock stress and variable UCS may suffer from the fact that the ‘tails’ of the $\sigma_{\text{max}}$ and UCS distributions may coincide, and cause damaging rock bursts. There are several examples.

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 Figures 7 and 8.

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**Table: Rock Mass Structure**

<table>
<thead>
<tr>
<th></th>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>RQD</td>
<td>Deere et al., 1967)</td>
</tr>
<tr>
<td>2</td>
<td>$J_n$</td>
<td>joint set number</td>
</tr>
<tr>
<td>3</td>
<td>$F$</td>
<td>joint frequency (per metre)</td>
</tr>
<tr>
<td>4</td>
<td>$J_v$</td>
<td>volumetric joint count (Palmström, 1982)</td>
</tr>
<tr>
<td>5</td>
<td>$S$</td>
<td>joint spacing (in metres)</td>
</tr>
<tr>
<td>6</td>
<td>$L$</td>
<td>joint length (in metres)</td>
</tr>
<tr>
<td>7</td>
<td>$w$</td>
<td>weathering grade (ISRM, 1978)</td>
</tr>
<tr>
<td>8</td>
<td>$\alpha/\beta$</td>
<td>dip/dip direction of joints (Schmidt diagram)</td>
</tr>
</tbody>
</table>

**Table: Joint Character**

<table>
<thead>
<tr>
<th></th>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>$J_r$</td>
<td>joint roughness number</td>
</tr>
<tr>
<td>10</td>
<td>$J_a$</td>
<td>joint alteration number</td>
</tr>
<tr>
<td>11</td>
<td>$J_{RC}$</td>
<td>joint roughness coefficient</td>
</tr>
<tr>
<td>12</td>
<td>$a/L$</td>
<td>roughness amplitude of asperities per unit length (mm/m)</td>
</tr>
<tr>
<td>13</td>
<td>$JCS$</td>
<td>joint wall compressive strength</td>
</tr>
<tr>
<td>14</td>
<td>$\phi_r$</td>
<td>residual friction angle</td>
</tr>
<tr>
<td>15</td>
<td>$\sigma_{R}$</td>
<td>Schmidt rebound values for joint and rock surfaces</td>
</tr>
</tbody>
</table>

**Table: Water, Stress, Strength**

<table>
<thead>
<tr>
<th></th>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>$J_w$</td>
<td>joint water reduction factor</td>
</tr>
<tr>
<td>17</td>
<td>SRF</td>
<td>stress reduction factor</td>
</tr>
<tr>
<td>18</td>
<td>$K$</td>
<td>rock mass permeability (m/s)</td>
</tr>
<tr>
<td>19</td>
<td>$\sigma_c$</td>
<td>compressive strength</td>
</tr>
<tr>
<td>20</td>
<td>$\sigma_1$</td>
<td>major principal stress</td>
</tr>
</tbody>
</table>

Figure 6 During NGI’s six years of work with the UK Nirex nuclear 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 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. Barton et al. (1992).
Figure 7 Practical use of Q-histogram logging requires a scheme of labelling, as in Figure 8, in which each 5 m 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.

Q - VALUES:

\[
Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}
\]

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

![Q-histogram log of rock containing the Mascota ore-body DDH-12 NB 22.04.13](image-url)
Figure 8 Core-logging for a rail tunnel project near Oslo. Note the mostly 6 m core box nomenclature (1, 2, 3 etc), and the Q-parameter recording line-by-line, so that rock type changes, or new structural domains, can be marked or footnoted. In this particular case a colour-coded rock type was added in the almost empty left-hand (worse conditions) column. Note the five or six opinions allowed per core box of 5 or 6 m. Almost the same recording scheme would be used in tunnel logging, utilizing the 3, 4 or 5 m advance. Although perhaps appearing ‘complicated’, the method of recording is extremely fast, as few difficult decisions (‘what is the most representative rating?’) have to be made. What one sees one records. The statistics (and occasional footnotes) take care of the ‘representativeness’.
With the present conference date of 2014 representing a 40-years anniversary since Q-system development, 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 Qc. It can be obtained from www.nickbarton.com: see p3 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 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 9, the ‘coordinates’ of the cube, representing a portion of a 20 m span cavern with local Q = 3, would require \( B + S(\text{fr}) \) of 2.0 \( \times \) 2.0 m c/c + \( S(\text{fr}) \) 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 \( S(\text{fr}) \) with stainless steel (or pp) fibers. However with the rule-of-thumb of 1.5 \( \times \) ESR and 5 \( \times \) Q for temporary support, which 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’ would reduce to ‘coordinates’ of \( B + S(\text{fr}) = 2.4 \times 2.4 \) m c/c + \( S(\text{fr}) = 4 \) cm. (SPAN / (The coordinate 1.5 \( \times \) ESR = 13.3m, and 5Q = 15, is shown by the larger arrow-head in Figure 9). As we will see shortly, \( S(\text{fr}) \) thicknesses of only 4 cm are no longer advised, due to curing problems.

![Figure 9](image_url)  

*Figure 9  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 a SPAN = 20m, ESR =1.0, and Q = 3 portion of an imaginary station cavern (an updated ESR table is given later). The problem of over-break and sometimes extensive smoothing and 3D membrane construction still needs to be solved. This can be costly in the case of NATM, due to the final concrete lining (Note the small cube representing the single-shell NMT Gjøvik cavern plotted at 62m span. This ‘boundary-pushing’ project is described later).*

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 authors (and apparently 99% of Norwegian designers) prefer NMT to NATM, since it is 1/4 to 1/5 as expensive, and the tunnel 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. In which case a Q-based ‘$Q+1.5\ ESR$’ can be used to select a reliable temporary support. If RRS (see later) instead of lattice girders are also used, deformation may be reduced in relation to NATM.

6. THE COMPONENTS OF Q-SYSTEM BASED 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 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.

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 Peru, with all the potential disadvantages of S(mr) well illustrated.

\[\text{Vandevall (1990)}\] illustration of the pitfalls with mesh reinforced shotcrete. It involves three processes, risk of ‘shadow’ and/or some rebound, corrosion of the mesh due to electrolytic currents, and delayed, unsafe installation.

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.

\[\text{Vandevall (1990)}\] illustration of the obvious advantages of S(fr): better bonding, no shadow, less corrosion, much lower permeability, faster, cheaper per tunnel-meter.

\text{Figure 10} The advantages of S(fr) compared to S(mr) are easily appreciated in these contrasting examples. The sketches from Vandevall (1990) are not-exaggerated.
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 such thing as in NATM, as neglect of the contribution of temporary shotcrete, temporary rock bolts, and temporary steel sets (not used), and long-term reliance on a final concrete lining (not used). Thus more care needs to be (and is) taken in the choice and quality of the support and reinforcement components B+S(fr) + (eventual) RRS.

Figure 11 illustrates (in the form of a shortened demo sample) the workings of the CT bolt, for those outside Norway who may not have used this remarkable, permanent, tunnel reinforcement component. The figure text emphasizes the multiple-layer corrosion protection, even after movement on an intersected joint has typically cracked the outer annulus of grout.

Figure 12 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. The dimensioning of RRS is illustrated in a Q-value versus tunnel dimension diagram shortly.

The photograph of completed RRS ribs (bottom-right, Figure 12) was from one side of the 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 top-right corner).
Figure 12 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 13.

It should be noted that the 1993 Q-support chart (shown earlier in Figure 9) suggested the use (at that time) of only 4-5 cm of unreinforced sprayed concrete in 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 1050 additional case records collected by Grimstad (Figure 9), 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.

The category 3 in 1993 and 2002/2003 has been taken away in the newest 2007 chart (Figure 13) which was fine-tuned in Grimstad (2006). 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).
Figure 13 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 accelerated (and potentially drained) 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 the low Q-value). Unreinforced shotcrete is used instead of final S(fr) to cover the bars, in order to minimize voids due to rebound of the fibres.

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 12 shows both varieties). Following the ‘D’ or ‘E’ the ‘boxes’ show maximum local (arch) thickness in cm (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 4cm 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 is 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 is unchanged.

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

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 single-shell case-record data base, and has led to 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 14a. 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 14b, from Barton and Grimstad (1994), the contrasting stiffness of B+S(fr) and steel sets is illustrated in a ‘confinement-convergence’ 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 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 wall and arch exhibiting over-break remains a problem, even in the case of rectangular lattice girders with possible bolting plates.
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 15 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.
4.16

**Figure 15** 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.

Thermal stresses resulting from widely different thicknesses of concrete (for instance 30 cm to 120 cm) 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. The consequences are cracking of the concrete, which is a long-term threat when ice can form.

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.

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, 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 stipulated, as in the case of the Bærum Tunnel of 5 km length, then an essentially dry tunnel is achieved. The required injection pressures are typically 5 to 10 MPa.

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 numerous igneous dykes. With logical design strategies, these limited humid patches could be treated with easily inspected local ‘panels’ of sprayed membrane. Far lower tunnelling costs are associated with well-designed NMT.
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 m³ 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 when incorrect methods are used).

If damp 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.

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).

Although seldom used in Norway so far, though now undergoing serious 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.

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).

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. \( \frac{1}{4} \)), in countries where high labour costs are typical.

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 (2011). 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 RQD/Jn increasing from 10/15 to 90/3. The significance of these internal grouting-induced changes on 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 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 arch of the main cavern was of course logged in detail. 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 shared Norwegian Gjøvik cavern experiences.

![Figure 18](image-url)
World's largest top-heading before benching down. The final span was 62m, height 24 m. Mean 10 cm of S(fr) in distant arch. Note some over-break > 1 m and RQD = 60-90.

$Q_{\text{mean}} \approx 10$, span 62m, B 2.5m c/c (L = 6m) and S(fr)10cm + cable anchors (L = 12m).

Q-values of arch (boxes): long external MPBX: red, internal MPBX: green.

MPBX locations (three rows), excavation week 1991. Note (90m long) initial pillars.

Central deformation 8 mm, ends: 7.0 mm, 7.5 mm. Side locations less than these.

Corrosion-protected B 2.5m c/c L= 6m, twin-strand anchors 12m, c/c 5m for wedges.

Figure 19  Some details of the Gjøvik Olympic cavern. Concept from Jan Rygh, design studies by Fortifikasjon and NoTeBy, design check modelling, external MPBX, seismic tomography, stress measurements and Q-logging by NGI, internal MPBX, bolt and cable load measurements, modelling, and other research aspects by SINTEF-NTNU. Construction in 6 months using double-access tunnels, by the Veidekke-Selmer JV. The cavern is an example of a drained NMT excavation. For details including UDEC-BB modelling, and actual very similar deformations: see Barton et al.(1994).

10. ESTIMATING TUNNEL OR CAVERNS 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. We can start with deformation.

\[
\Delta_v = \frac{\text{SPAN}}{Q} \sqrt{\frac{\sigma_v}{\sigma_c}}
\]

\[
\Delta_h = \frac{\text{HEIGHT}}{100Q} \sqrt{\frac{\sigma_h}{\sigma_c}}
\]

\[
k_o = \left( \frac{\text{SPAN}}{\text{HEIGHT}} \right)^2 \left( \frac{\Delta_h}{\Delta_v} \right)^2
\]

<table>
<thead>
<tr>
<th>Nathpa Jakri power station cavern</th>
<th>Gjøvik Olympic cavern</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\Delta_v = 20,000 \times (6/35)^{1/2} = 28 \text{ mm})</td>
<td>(\Delta_v = 60,000 \times (1/75)^{1/2} = 6.9 \text{ mm})</td>
</tr>
<tr>
<td>(\Delta_h = 50,000 \times (4/35)^{1/2} = 56 \text{ mm})</td>
<td>(SPAN = 60m, (Q_{\text{Rock}} = 10), (\sigma_v = 1 \text{ MPa}) at 40 m depth, (\sigma_c = 75 \text{ MPa}))</td>
</tr>
<tr>
<td>(SPAN = 20m, HEIGHT = 50m, Q = 3, (\sigma_v = 4 \text{ MPa}), (\sigma_h = 6 \text{ MPa}), (\sigma_c = 35 \text{ MPa}). (In the middle of the range of MPBX measurements for the arch and walls).</td>
<td>(Almost identical to that measured with nine MPBX, and almost identical to UDEC-BB modelling results).</td>
</tr>
</tbody>
</table>
Empirical improvements to this preliminary and simplistic ($\Delta \approx \text{SPAN}/Q$) model were made by employing the ‘competence factor’ format of SRF (i.e. stress/strength). The alternative and slightly more sophisticated formulæ shown in Figure 20 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 ‘$\sigma_n \tan \varphi$’ in linear Mohr-Coulomb and non-linear Hoek-Brown formulations, are strong reasons for modelling ‘bigger effects’ (larger deformations, deeper ‘plastic zones’) and the presumed ‘disqualification’ of empirical methods. Their analyses (models) are based on so many a priori assumptions, that some a posteriori experience is strongly advised. Reality may be (is usually) different from what is seen in colourful models, especially continuum models.

11. CORRELATING Q WITH 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.

$$V_P \approx 3.5 + \log Q$$ (units of velocity: km/s)                                                                   (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 $\sigma_c/100$, where the uniaxial compressive strength is expressed in MPa ($Q_c = Q \times \sigma_c/100$). 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).

$$V_P \approx 3.5 + \log Q_c$$ (units of velocity: km/s)                                                                  (2)

The derivation of the empirical equations for support pressure, and for the static deformation modulus (see Figure 21, right-hand tables) which were derived independently, 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.

Further useful equations derived from $Q_c$ concern the deformation modulus $E_{\text{mass}}$. There are several possible equivalent forms, and $V_P$ can be used in place of Q or $Q_c$ if need be.
where $V_p$ is in km/s, $E_{\text{mass}}$ is in GPa, and $\sigma_c$ is in MPa. For instance with $Q = 10$ and $\sigma_c = 100$ MPa and $V_p = 4.5$ km/s, one obtains $E_{\text{mass}} \approx 22$ GPa using all three equations (3a, 3b, 3c). This corresponds to the nominal 25 m depth (shallow seismic refraction) ‘central diagonal’ in Figure 21.

If $Q$ was unknown, a higher $V_p$ of say 5.5 km/s (because of a deeper more highly stressed location) suggests $E_{\text{mass}} \approx 46$ 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 inset).

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 11km of drill-cores at the UK Nirex site in NW England.
CONCLUSIONS

1. The Q-system appears to have weathered the test of time and has had application in 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 support and reinforcement selection, due to the large number of (mostly Norwegian) case records on which it is now mostly based.

2. 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 always used exclusively, as also now expected by road and rail authorities in Norway, despite its limited acceptance among some people in Norway, particularly in the past.

3. In relation to ‘competing classification methods’ such as RMR (and the closely related GSI of Hoek) it appears to have the advantage of a logarithmic quality scale, and 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).

4. The less common ratio Jn/Jr (when ≥ 6) is closely related to the amount of overbreak, and therefore to shotcrete (or concrete) volumes, and related difficulties with 3D membrane construction in the case of NATM.

5. The Q-values and their statistical variation have important roles to play during site investigation, using core-logging, and seismic velocity interpolation between boreholes. The effect of depth or stress on VP must not be ignored. Subsequent application is usually for support and reinforcement design assistance, based on ‘support class’. This selection 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 Norwegian record speed of 160 or 170 m/week, Q will not be needed either).

6. Q has simple direct links to velocity, to deformation and to deformation modulus, and each are depth and/or stress dependent. Q can also be linked to the estimation of a depth-dependent permeability via a modified parameter Q_H2O though this needs more data at present.

7. Q has been used on numerous occasions as a basis for realistic TBM prognosis via Q_TBM. This empirical model includes some rock-cutter interaction parameters, like NTNU’s CLI. When predicting penetration rate, care is needed to match cutter force to a Q-based estimate of rock mass strength, which varies from about 1 to 100 MPa, depending on Q and UCS. Unexpected delays, and occasional permanent burial of TBM can be strongly linked to adverse Q-values. Steep deceleration gradients and slower advance rates in ‘bad ground’ are linked directly to low Q-values.

8. In recent years, several researchers in different countries have recognized that linear Mohr-Coulomb and non-linear Hoek-Brown are incorrect (continuum) models for the shear failure of rock masses surrounding tunnels or beneath rock slopes. More correct modelling requires degradation of cohesion followed by mobilization of friction (‘c then σn tan φ’), i.e. not their addition, because they do not get mobilized simultaneously.
9. Input data can be obtained from the two halves (CC and FC) of the $Q_c$ formula. These cohesive and frictional components are surprisingly close approximations to the (assumed) cohesive and frictional strength of rock masses. CC (approximating $c'$) and FC (approximating $\tan^{-1} \varphi'$) demand respectively more shotcrete or more rock bolts when of insufficient value: hence the link to case records. This is a new and long over-due area requiring progressive failure representation in numerical continuum modelling. It can already be done in the discontinuum model UDEC-BB, with prior failure of intact bridges.

REFERENCES


