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# USE AND MISUSE OF ROCK MASS CLASSIFICATION SYSTEMS WITH PARTICULAR REFERENCE TO THE Q-SYSTEM

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## SUMMARY

*Rock mass classification systems have gained wide attention and are frequently used in rock engineering and design. However, all of these systems have limitations, but applied appropriately and with care they are valuable tools.*

*The paper describes the history of the Q-system that was introduced in 1974, and its later development. The individual parameters are analysed, and their relevance for the natural geological features they seek to simulate, is discussed. This applies to both the original application for assessing rock mass quality to estimate the extent of rock support, and the later attempts to make the method into a kind of general rock mass classification with many applications. This also includes the recently introduced* $Q_{TBM}$*, which shall allow estimates of penetration and advance rate for TBM, and also the attempts to apply Q to express the effects of pre-grouting.*

*It is concluded that the Q-system, used with full awareness of its limitations, may be applied for classification of the stability and support estimates of tunnels and rock caverns, preferably in jointed rocks. Applied here, it may be used for planning purposes. It is less useful for prescription of rock support during construction. It is not likely that Q is suitable to express the effects of pre-grouting.* $Q_{TBM}$ *is complex and partly misleading and is not recommended for use in its present form.*

# 1 INTRODUCTION

## 1.1 About classification systems and tunnelling methods

Describing rock masses and ground conditions from a technical point of view is not an easy task. As engineers we feel more confident when we are working with numbers than with adjectives, as it is complicated to couple adjectives from different parameters when calculations are needed. Thus, at an early stage in the development of geological engineering and rock mechanics, several classification systems and so-called tunnelling methods were presented in different countries.

A well-known example is Terzaghi's classification system for support of tunnels. This descriptive system was developed in the U.S.A. and presented in a book with the title ”*Rock Tunneling with Steel Supports*” in 1946. It applies conservative estimations for loads on the support totally based on the use of steel (Terzaghi, 1946). Even with these original limitations, the system has later been used as a general classification system. Another example is the New Austrian Tunnelling Method (NATM), first time internationally described in 1963. Contrary to classification systems restricted to rock engineering, NATM includes most aspects of tunnel construction from field investigation and feasibility through contract documents, excavation, support and monitoring. It was originally developed for squeezing rocks in the Alps.

In addition to more or less general classification systems and methods, a number of special systems and methods have emerged, for instance for classification of the strength of rocks and rock masses, the degree of jointing (*RQD*), the degree of weathering, rock drillability and blastability, as well as performance of tunnel boring machines (TBM). A common platform for all these systems and methods is that they are based on more or less recordable observations and measurements made in the field or in the laboratory. Observations and measurements are expressed as numbers, which then in different ways are combined into a final sum or product. Quite commonly, these numbers are put in a table where they are transformed to an adjective, which again describes the quality of the rock mass. (As an example: *Q*-values between 1 and 4 describe ”poor” ground (with respect to tunnel stability).

During the more than thirty years the two authors have been involved in the field of rock engineering, both have experienced what has happened with some of these systems and methods. One observation is that, if and when a system or method obtains a certain acceptance, it does not take long before someone tries to expand its use.

This may be people who were involved in developing the system, but it may also be others. Some of the problems that have been reported from various tunnel excavations using the NATM may partly be explained by the extended use of this method, which was developed on the basis of experience from construction and support of tunnels in the Alps.

During the last 10 - 15 years many papers have been published on the *Q* system to extend its applications. This is a main reason why especially the *Q* system is being dealt with in this paper.

A first paper on the limitations of *Q* was presented in Norwegian by Palmstrom et al. (2002) at the annual Norwegian tunnelling conference in 2002 to trigger a debate in Norway about appropriate use of the *Q* system. A meeting was later arranged by the Norwegian Rock Mechanics Group with further discussions. Two of the authors now feel that it is due time that an evaluation on the use of the *Q* system is discussed on an international basis. The third author has presented a separate paper “A critique of $Q_{TBM}$” (Blindheim, 2005).

## 1.2 The structure of the *Q*-system

The *Q*-system was originally developed for classification of rock masses and ground with the aim of being a helpful tool for evaluating the need for support in tunnels and rock caverns. It was first time published in 1974 by N. Barton, R. Lien and J. Lunde of the Norwegian Geotechnical Institute (NGI), and has since undoubtedly been important in the development of rock engineering. Later, it was included as a basic factor in the ”Norwegian Method of Tunnelling” (NMT), which is a response or supplement to the NATM (New Austrian tunnelling method)

The *Q*-values are combined with the dimensions of the tunnel or cavern in a *Q*-support chart (see Figure 7). This chart is based on more than 1000 cases of rock support performed in tunnels and caverns. Using of a set of tables with a number of footnotes, the ratings for the different input parameters can be established based on engineering geological observations in the field, in tunnels, or by logging of rock cores.

The structure of the original Q system and the different input parameters will be discussed in some detail in the next main chapter.

## 1.3 Development of the *Q*-system since its introduction 30 years ago

During the 30 years since the *Q*-system was introduced it has received much attention worldwide. Through numerous papers, several improvements and/or adjustments of the system have been published, most of them by its originators or other people at the Norwegian Geotechnical Institute (NGI), as can be seen in Table 1.

As seen, *Q* has been "developed" to $Q_{TBM}$ for use in connection with TBM-tunnels; another application is the use of the *Q*-value in connection with grouting of tunnels. These aspects will also be discussed later in this paper. The authors are raising the question: Are the developers of additions to the *Q*-system going too far in trying to cover new fields for the use of this classification system?

Thus, the purpose of this paper is to carry out a critical review of the *Q*-system as such, and not least to discuss the many different ways the *Q*-value and the *Q*-system later have been applied by different authors. What is the original structure of the system, and what kinds of rock masses and ground conditions does it cover? What kinds of support is it supposed to cover: temporary, permanent or total support? Should it only be used at the planning stage, or can it also be used during construction? These are some of the questions dealt with in this paper.

Table 1. Main developments of the *Q*-system

| Year | Development | Author(s) and title of paper |
|---|---|---|
| 1974 | The *Q*-system is introduced | Barton*, Lien*, and Lunde*: Engineering classification of rock masses for the design of tunnel support. |
| 1977 | Estimate of rock support in tunnel walls<br>Estimate of temporary support | Barton*, Lien*, and Lunde*: Estimation of support requirements for underground excavation. |
| 1980 | *Q* system for estimate of input parameters to the Hoek-Brown failure criterion for rock masses | Hoek and Brown: Underground excavations in rock. |
| 1988 | New, simplified rock support chart | Grimstad* and Barton*: Design and methods of rock support. |
| 1990 | Rock support of small weakness zones | Löset*: Using the *Q*-system for support estimates of small weakness zones and for temporary support (in Norwegian) |
| 1991 | Estimate of *Q* values from refraction seismic velocities | Barton*: Geotechnical design. |
| 1992 | The application of the *Q*-system in the NMT ("Norwegian method of tunnelling") | Barton* et al.: Norwegian method of tunnelling. |
| 1992 | Estimate of squeezing using *Q* values | Bhawani Singh et al.: Correlation between observed support pressure and rock mass quality. |
| 1993 | Updating the *Q*-system with:<br>– adjustment of the SRF values<br>– application of new rock support methods<br>– *Q* estimated from seismic velocities<br>– estimate of deformation modulus for rock masses<br>– adjustment for narrow weakness zones | Grimstad* and Barton*: Updating of the *Q*-system for NMT. |
| 1995 | Introduction of *Qc* with application of compressive strength | Barton*: The influence of joint properties in modelling jointed rock masses. |
| 1997 | *Q*-system applied during excavation | Löset*: Practical application of the *Q*-system |
| 1999 | $Q_{TBM}$ is introduced | Barton*: TBM performance estimation in rock using $Q_{TBM}$. |
| 2001 | The *Q*-system is applied for estimating the effect of grouting | Barton* et al.: Strengthening the case for grouting. |
| 2002 | Further development of the *Q*-system | Barton*: Some new *Q*-value correlations to assist in site characterization and tunnel design. |
| * = NGI people | | |

## 2 ON THE INPUT PARAMETERS TO *Q*

The rock tunnelling quality *Q* has been considered a function of three important ground parameters, which are said (Barton et. al., 1974) to be crude measures of:

I. Relative block size *(RQD/Jn)*
II. Inter-block shear strength *(Jr/Ja)*
III. Active stresses *(Jw/SRF)*

These 6 parameters are combined to express the ground quality with respect to stability and rock support in underground openings in the following equation:

$$Q = RQD/Jn \times Jr/Ja \times Jw/SRF \qquad \text{Eq. (1)}$$

## 2.1 The relative block size *(RQD/Jn)*

The quotient *(RQD/Jn),* representing the structure of the rock mass, has the two extreme values (100/0.5 and 10/20) differing by a factor of 400. It consists of two parameters, one for characterization of the degree of jointing (*RQD*), the other the number of joint sets (*Jn*) occurring in the location in question.

### *2.1.1 The Rock Quality Designation, RQD*

*RQD* was introduced as a measure for the degree of jointing or block size. It is by its definition by Deere (1963), the length in percent of measured length of the unweathered drill core bits longer than 10cm. Originally, it was for use with NX-size core (54.7mm). The *RQD* is an easy and quick measurement, as only the core pieces longer than 10cm are included. Therefore, it is frequently applied in core logging and is often the only method used for characterization of block size. *RQD* can also be found from scanline measurements.

Over the years there have been several papers discussing this measurement, among others Grenon and Hadjigeorgiou (2003): "*RQD can often result in a sampling bias due to a preferential orientation distribution of discontinuities. A further limitation is that it cannot account for the size (length) of the considered discontinuities. Furthermore, RQD is insensitive when the total frequency is greater than* $3m^{-1}$*. Despite these limitations, RQD is used in its own rights and as an integral part of the more popular rock mass classification tools used in the mining industry (RMR and NGI)." "RQD is insensitive when the rock mass is moderately fractured. One has to keep in mind that RQD values are a function of the total frequency, which is highly sensitive to sampling line orientation."*

Analyses have shown that it is very difficult to relate *RQD* to other jointing measurements (Palmstrom, 2005), because *RQD* is a one-dimensional, averaged measurement based solely on core pieces longer than 0.1m. Simulations using blocks of the same size and shape penetrated by a line (i.e. borehole) at different angles have been used for such estimations. The first attempts when the volumetric joint count (*Jv*) was introduced, were made by Palmstrom (1974):

$$RQD = 115 - 3.3\,Jv \quad (RQD = 0 \text{ for } Jv > 35, \text{ and } RQD = 100 \text{ for } Jv < 4.5)\ ^{1} \qquad \text{Eq. (2)}$$

This expression was included in the introduction of the *Q* system by Barton et al. (1974). As was shown by Palmstrom (1974), it is a rather poor correlation between *RQD* and *Jv*, especially, where many of the core pieces have lengths around 0.1m. However, when *Jv* is the only joint data available (no borehole or scanline logging), Eq. (1) has been found to be an alternative transition for finding *RQD* from *Jv*.

In addition, the *RQD* covers only a limited part of the range of jointing, which reduces the applicability of *RQD* in characterizing the whole span of jointing. This is shown in Figure 1. It should, however, be mentioned that the range covered by *RQD* represents the most important part of blocky ground with respect to single rock falls, which is where classification systems generally work best.

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[1] In a recent paper, Palmstrom (2005) has found that $RQD = 110 – 2.5Jv$ (for $Jv = 4$ to 44) gives a better correlation, but still with several limitations.

Caused by the above, the application of *RQD* in rock engineering calculations may lead to inaccuracy or errors. *RQD* should, therefore, be applied with great care. Consequently, while *RQD* is a practical parameter for core logging, it is not sufficient on its own to provide an adequate description of a rock mass. (Bieniawski, 1984; Milne et al.,1998)

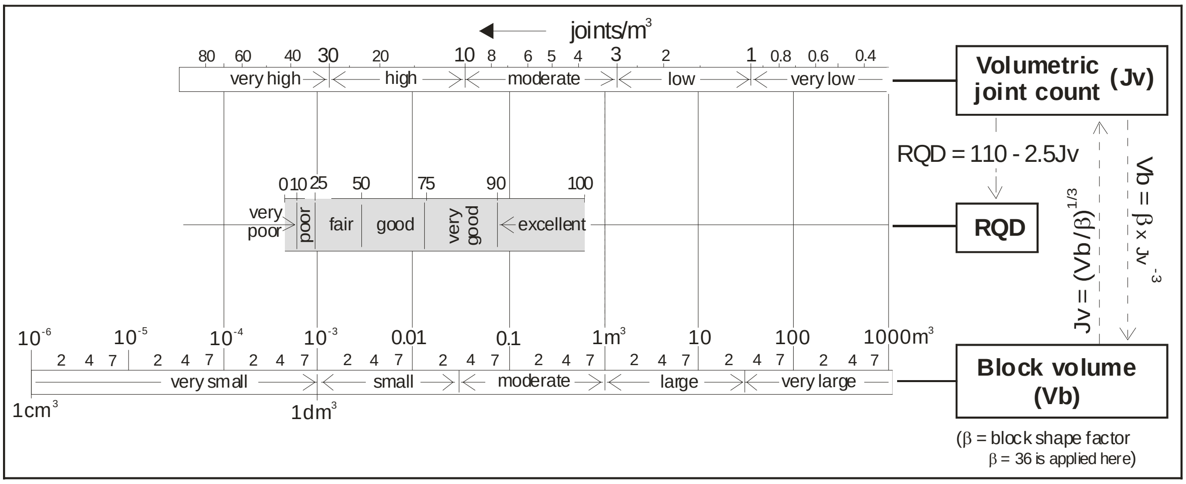



Figure 1. Correlations between various measurements of block size (revised from Palmstrom et al, 2002). The *Vb* and *Jv* cover a significantly larger interval of the jointing than the *RQD*. The best transitions are between *Jv* and *Vb*. Example: For block volume *Vb* = 0.1 m³ the *Jv* = 7 joints/m³ and *RQD* = 92

### 2.1.2 *The joint set number, Jn*

*Jn*, which is defined by the number of joint sets, is not necessary a reliable indicator of fracturing. A highly fractured rock may contain only a few discontinuity sets, while a widely spaced jointing may be described to consist of many joint sets by inexperienced users.

For drill and blast excavated tunnels Löset (1997) recommends that cracks formed by tunnel blasting are included in the *Jn* as random joints. Also very short joints (fissures) should be mapped in the same way. The latter feature is not described in the *Q* parameter for *Jn* or in any other input parameter to *Q*.

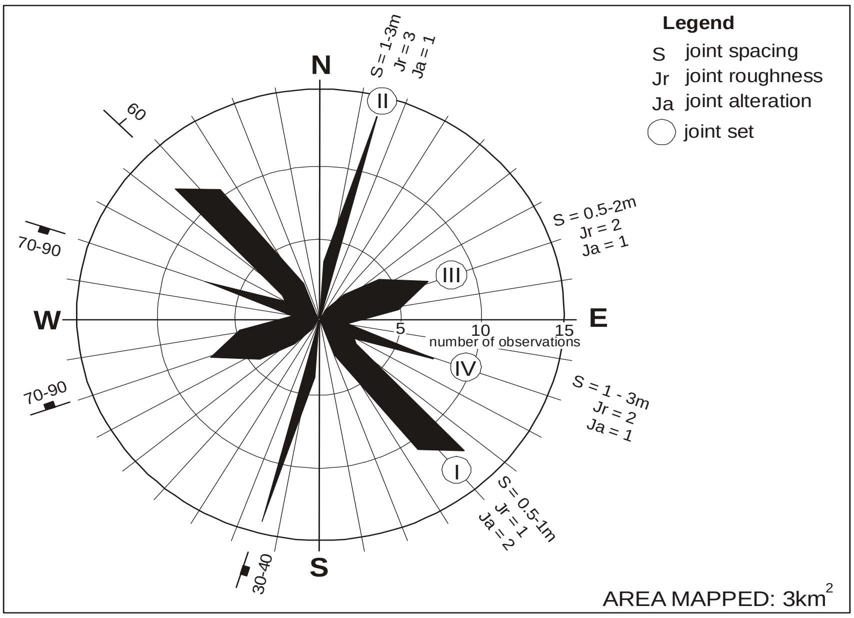



Figure 2. The joint pattern presented in a joint rosette with 4 joint sets in the whole area. In a single location only some of these may be present

A stereo-net or joint rosette for an area will generally show more joint sets than is the case for a single locality (Figure 2). Many users apply the number of joint sets occurring in the whole project area, and by this arrive at a higher (wrong) *Jn* value than for the actual location.

### *2.1.3 Connection between block volume and the ratio RQD/Jn*

The limited range of *RQD* is extended in the *Q*-system by dividing it with *Jn*. This quotient *(RQD/Jn)* is meant to represents the overall structure of the rock mass giving a crude measurement of the relative block size within the two extreme values 200 and 0.5. *"Despite claims that the non-dimensional ratio (RQD/Jn) is an indication of block size this is not obvious."* (Grenon and Hadjigeorgiou, 2003), which is also shown in Figure 3. This is not unexpected, remembering the inability of *RQD* to give accurate measurement of the density of joints (and hence the block size) (Palmstrom, 2005).

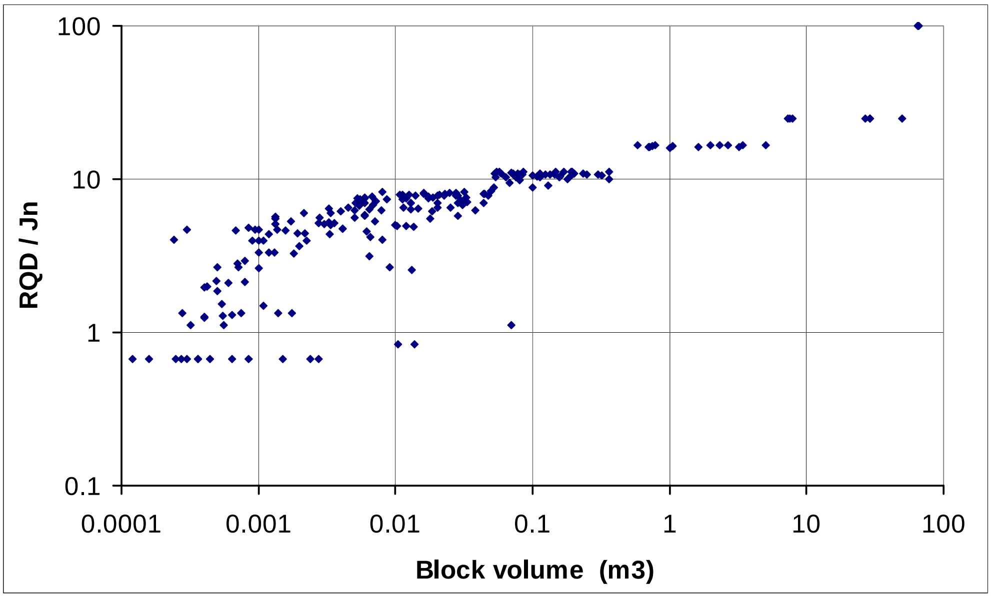



Figure 3. Correlation between block volume and *RQD/Jn* based on computer simulations with blocks of the same size and shape penetrated by a line (i.e. borehole) at different angles. Note that both axes are logarithmic (from Palmstrom et al., 2002).

## 2.2 The inter-block shear strength *(Jr/Ja)*

The second quotient *(Jr/Ja)* represents the roughness and frictional characteristics of the joint walls with or without filling materials. *"This quotient is weighted in favour of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favourable to tunnel stability."* (Hoek, 2004).

Rock wall contact after small shear displacements may be a very important factor for preserving the excavation from ultimate failure. Where no rock wall contact exists, the conditions are unfavourable to tunnel stability. The swelling pressure of montmorillonite may also be a factor to be incorporated here.

The *(Jr/Ja)* is a useful characterization of the condition of discontinuities. From numerous joint variations, limited, but easily recognisable features have been selected. In practice however, it might be difficult to observe or measure whether a discontinuity will have rock wall contact after 0.1m shear.

No division is given in the rating of filled joints whether the underground excavation is above or below the ground water table. As mentioned in Section 2.3.1 possible softening of clay with water should be given to this parameter.

## 2.3 The active stresses *(Jw/SRF)*

The quotient *(Jw/SRF)* is a complicated, empirical ratio consisting of two groups of stress parameters. *"It has proved impossible to combine these two parameters in terms of inter-block effective stress, because paradoxically a high value of effective normal stress may sometimes signify less stable conditions than a low value, despite the higher shear strength."* (Hoek, 2004).

### *2.3.1 The joint water reduction factor, Jw*

The description, division and input values used for *Jw* and the values are shown in Table 2.

Table 2. Description and ratings for the parameter *Jw* (Barton et al, 1974)

| | | |
|---|---|---|
| Dry excavations or minor inflow, i.e. < 5 l/min locally | $p_w$ < 1 kg/cm$^2$ | *Jw* = 1 |
| Medium inflow or pressure, occasional outwash of joint fillings | 1 - 2.5 | 0.66 |
| Large inflow or high pressure in competent rock with unfilled joints | 2.5 - 10 | 0.5 |
| Large inflow or high pressure, considerable outwash of joint fillings | 2.5 - 10 | 0.3 |
| Exceptionally high inflow or water pressure at blasting, decaying with time | > 10 | 0.2 - 0.1 |
| Exceptionally high inflow or water pressure continuing without noticeable decay | > 10 | 0.1 - 0.05 |
| ***Note:*** *(i) The last four factors are crude estimates. Increase Jw if drainage measures are installed* *(ii) Special problems caused by ice formation are not considered* | | |

The limits between medium, large and exceptionally high inflows in Table 3 are not given. Then it is up to each user to decide them in each case, resulting in inaccurate characterization.

*"As geotechnically trained engineers, we can visualise that the necessary addition of Jw as the last and 6$^{th}$ parameter of Q, was for 'fine tuning' this Jr/Ja ratio, adding something like a softening and an effective stress correction for when water was present. The parameter Jw was also designed to roughly account for stability problems due to combinations of high water pressures, high permeabilities and potentially high storativity."* (Barton, 2002).
There are, however, several other opinions of the effect of water on stability and rock support, and the possibility to generally include the wide range of inflow/water pressures in a support chart as is used in the *Q* system:

- How is the linking between inflow rate and water pressure? (E.g. in a permeable rock mass where high inflow may take place at low ground water pressure: which value for Jw should be applied?)
- How can rock support be performed when large and exceptionally high inflows take place? In such cases other measures have generally to be taken, not covered by the *Q* support chart.
- Water softening the clay fillings and reduce the friction should preferably have been included in *Ja*, as mentioned earlier.
- The influence of *Jw* on rock support, theoretically presented in Table 3, shows that water has a significant effect on the estimated amount of roof support. Is this the case?

It is obvious that small inflows of water (*Jw* ≥ 0.5) has influence on the conditions in a tunnel, but generally not as significant variation on the amount of rock support as shown in Table 3.

For water inflows where *Jw* < 0.5, the water may limit the use of shotcrete and cause solution with special support works.

Table 3. Influence of water on rock support in a 10m wide tunnel with *Q* = 4 (in dry conditions) and *ESR* = 1

| Water condition | *Q* value | Rock support in roof | | Comment |
|---|---|---|---|---|
| | | bolt spacing | shotcrete | |
| *Jw* = 1: Dry excavations or minor inflow, i.e. < 5 l/min locally | 4 | 2.1 × 2.1m | 45mm | |
| *Jw* = 0.3: Large inflow or high pressure, considerable outwash of joint fillings | 1.2 | 1.75 × 1.75m | 70mm | It is not possible to apply shotcrete where large and exceptionally large inflow of water occur |
| *Jw* = 0.1: Exceptionally high inflow or water pressure at blasting, decaying with time | 0.4 | 1.5 × 1.5m | 100mm | |

The natural joints and the cracks developed from blasting in drill and blast excavated tunnels will, together with redistribution of stresses, result in drainage around the tunnel and consequently reduction of the water pressures. This strongly limits the effect of groundwater on instability, which is the experience - with a few exceptions - in the large amount of Norwegian tunnels.

In the opinion of the authors, large water inflows are more an excavation problem than a stability problem during tunnel excavation, except for flowing ground and high water ingresses, where other measures are more important than rock support. The use of shotcrete, as indicated in the *Q* support chart, in flowing water is seldom appropriate in such conditions before sealing works and/or drainage (which is not indicated in the *Q*-system) are performed.

### *2.3.2 The stress reduction factor, SRF*

*SRF* is basically a measure of:

A. Loosening load in the case weakness zones/faults intersection the an excavation,
B. Rock stresses in competent rock, and rock overstressing problems in brittle rocks
C. Squeezing loads in plastic incompetent rocks
D. Swelling loads from chemical reaction caused by water

It is in the *Q*-system regarded as a total stress parameter. Table 4 shows the division of this parameter. Barton, (2002) has explained the background for this input parameter:
*"As in the frictional cases that needed 'fine tuning' and adjusting for effective stress with Jw, we may speculate that SRF was a necessary 'fine tuning' and adjustment for the effects of stress (and sometimes fragmentation) in the case of relative block size and 'cohesive strength'. We needed to account for the adverse effect of excavating an opening in an over-stressed (or sometimes under-stressed) rock mass. <u>In the case of competent rock, SRF is a measure of the stress/strength ratio, in anticipation of a stress-fractured EDZ (excavation damaged zone) in previously quite massive rock, requiring heavy, but yielding support.</u> When SRF applies to faulting, the idea of loosening due to previously fractured (i.e. faulted material) is also relevant. The less frequently used SRF categories of squeezing and swelling are also indicative of a shear-displacement-reduced or swelling-strain-reduced 'cohesive strength' (or rather, weakness), together with the presence of an unbalanced driving force or increased radial stress* $\sigma_r$*, in each case requiring heavier support to resist the effects of the tangentially strained EDZ."*

The sentence underlined (by the authors) gives no meaning to the authors. The normal rock mechanics definition of *competent rock* is a rock having higher strength than the stresses acting on it, and the need for a yielding support in a blasted tunnel where the EDZ is formed from the blasts, seems strange. The sentence also seems to indicate that the SRF is more used as a 'fine tuning' than an influence of stresses.

Table 4. Description and ratings for parameter *SRF* (stress reduction factor) (from Barton et al, 1974 and Grimstad and Barton, 1993)

| | | | | |
|---|---|---|---|---|
| A. Weakness zone intersecting excavation | Multiple weakness zones with clay or chemically disintegrated rock, very loose surrounding rock (any depth) | | | *SRF* = 10 |
| | Single weakness zones containing clay or chemically disintegrated rock (depth of excavation < 50 m) | | | 5 |
| | Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m) | | | 2.5 |
| | Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth) | | | 7.5 |
| | Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation < 50 m) | | | 5 |
| | Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation > 50 m) | | | 2.5 |
| | Loose, open joints, heavily jointed or "sugar-cube", etc. (any depth) | | | 5 |
| **Note:** (i) Reduce SRF by 25 - 50% if the actual shear zones only influence, but do not intersect the excavation | | $\sigma_c / \sigma_1$ | $\sigma_\theta / \sigma_c$ | |
| B. Competent rock, rock stress problems | Low stress, near surface, open joints | > 200 | < 0.01 | 2.5 |
| | Medium stress, favourable stress condition | 200-10 | 0.01-0.3 | 1 |
| | High stress, very tight structure. Usually favourable to stability, may be except for walls | 10-5 | 0.3-0.4 | 0.5-2 |
| | Moderate slabbing after > 1 hour in massive rock | 5-3 | 0.5-0.65 | 5-50 |
| | Slabbing and rock burst after a few minutes in massive rock | 3-2 | 0.65-1 | 50-200 |
| | Heavy rock burst (strain burst) and immediate dynamic deformation in massive rock | < 2 | > 1 | 200-400 |
| **Notes:** (ii) For strongly anisotropic stress: when $5 < \sigma_1/\sigma_3 < 10$, reduce $\sigma_c$ to $0.75\ \sigma_c$; when $\sigma1/\sigma3 > 10$, reduce $\sigma_c$ to $0.5\sigma_c$<br>(iii) Few cases found where overburden is less than span width. Suggest SRF increase from 2.5 to 5 for low stress cases | | | $\sigma_\theta / \sigma_c$ | |
| C. Squeezing rock | Plastic flow of incompetent rock under the influence of high pressure | Mild squeezing rock pressure | 1 - 5 | 5-10 |
| | | Heavy squeezing rock pressure | > 5 | 10-20 |
| D. Swelling rock | Chemical swelling activity depending on presence of water | Mild swelling rock pressure | | 5-10 |
| | | Heavy swelling rock pressure | | 10-15 |

### *A. Weakness zones intersecting excavation*

Weakness zone is by definition "*a part or zone in the ground in which the mechanical properties are significantly lower than those of the surrounding rock mass. Weakness zones can be faults, shears / shear zones, thrust zones, weak mineral layers, etc."* (Norwegian Rock Mechanics Group, 2000)

Basically, there are two main groups of weakness zones:

1) those, which are formed from tectonic events, and
2) those consisting of weak materials formed by other processes, such as weathering, hydrothermal activity and alteration.

In Table 4 the actual zone can be determined from 7 different features. Of the vast amount of weakness zones and faults occurring in the earth's crust, it is, of course, difficult to cover all these in a simple table. The *Q*-system applies the following features to meet all these variations:

- How the zone occurs; either as single zone, or as frequent occurrences of two or more zones.
- Content of clay materials or chemical alteration/weathering in the zone.
- At which depth the zone is located (in competent rocks) with a division between zones deeper or shallower than 50m.
- The conditions of the adjacent rock mass – given for single zones only.

- Various types of zones, such as unconsolidated, open joints/heavily jointed zones, sugar cube rock.

In fact, many weakness zones can be included in the classes. Zones in deformable (incompetent) rocks, such as phyllites and other schistose rocks seem, however, to be poorly covered. Further, the size (width) of the zones is not included. This is a very important feature: for example, a zone of 1m may have different instability in a tunnel than a zone of 10m or larger. Löset (1990) has, however, given a contribution for estimating *Q* values for small zones. This is to some help here, but the expression to be used is complicated with logarithmic values of the input parameters, and is probably not often included in the *Q* usage.

An important feature in connection with weakness zones is the stress variation in and near the zone (Löset, 1997). With more than one zone located close to each other, an extended section along the tunnel can be influenced as shown in Figure 4.

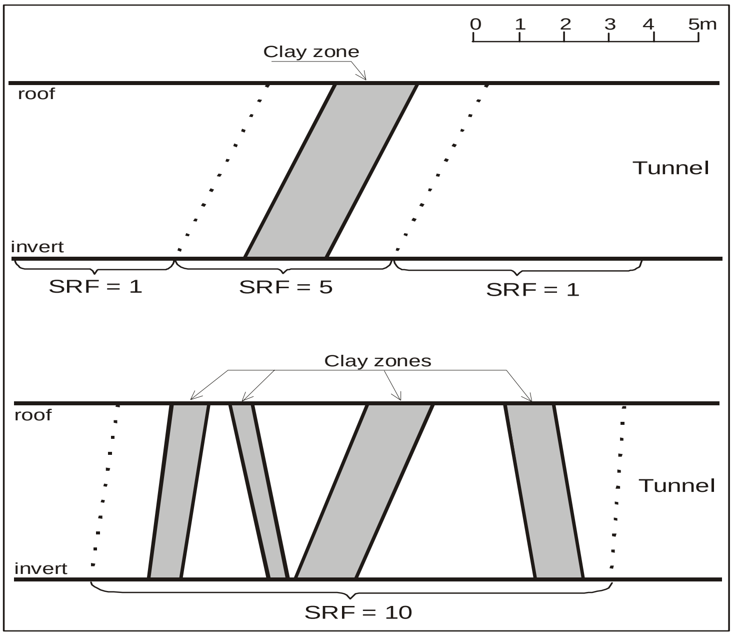



Figure 4. *SRF* in weakness zones, upper: single zone, lower: several, nearby zones (Löset, 1997).

It is, however, difficult to understand how the *SRF* for weakness zone represents the redistribution of stresses and can be called "active stresses" influencing on instability and rock support. What about geological formations having similar composition of block size (*RQD/Jn*) and friction properties (*Jr/Ja*) as a weakness zone? Figure 5 shows an example of this for a tunnel located at 40m depth:

- <u>In case A</u> (which is not a zone) with larger overbreak than case B, *SRF* = 1 belonging to group B in Table 4: Competent rock, rock stress problems, medium rock stress.
- <u>In case B,</u> the weakness zone with *SRF* = 5 belongs to group A: Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation < 50 m). For this case the amount of rock support according to the *Q*-system will be significantly higher than for case A. In practice, it is opposite, because in a weakness zone the arching effect will contribute to increase stability.

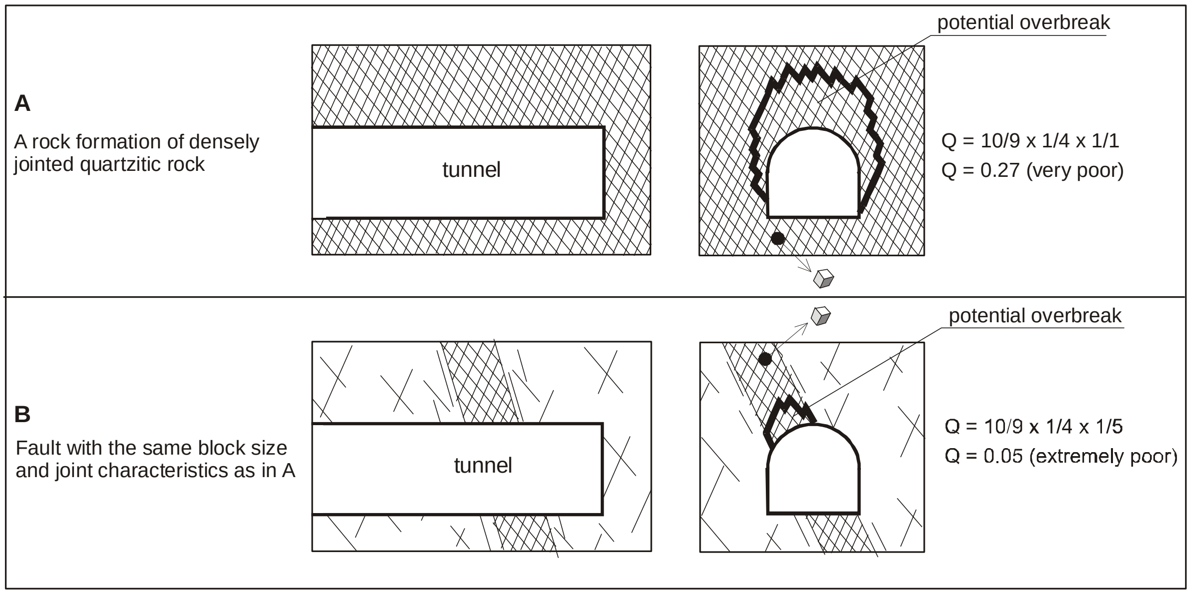



Figure 5. A large part of the North Cape subsea tunnel, Norway (A) is located in heavily broken meta-greywacke. The weakness zone (B) has a similar composition as the rock masses in A. It is a significant difference in overbreak in the heavily jointed rock masses (A) and a weakness zone (B) caused mainly by the arching effect in the weakness zone (from Palmstrom et al., 2002)

### *B. Competent rock, rock stress problems*

This group of *SRF* is generally the ratio between rock stresses and rock mass strength in an underground excavation. The group includes two main ground behaviours: 1) block falls, and 2) overstressing of brittle, massive rocks (rock burst, rock slabbing, spalling, etc.). The values for *SRF* in overstressed, massive brittle rocks were adjusted in 1993 with an increase of 500% to 2000% to better cover the large difference in support between these two. This is by the two authors understood as a 'fine tuning' of the Q system.

### *C. Squeezing rock. Plastic flow of incompetent rock under the influence of high pressure*

*Squeezing* is a time dependent deformation, essentially associated with creep caused by exceeding a limiting shear stress. Deformations may terminate during construction or continue over a long period. Squeezing rock slowly advances without perceptible volume increase and at almost constant water content. It takes place in overstressed, deformable rocks and rock masses.

It is a great challenge to plan and to excavate tunnels in squeezing ground conditions. The NATM has been developed mainly for this purpose, applying a whole system or method with description and analyses incorporating practical construction methods to cope with this complicated ground behaviour.

In the *Q*-system, the squeezing effect must first be evaluated. This requires insight, experience, measurements and analyses of the actual case. Bhawani Singh et al. (1992) have presented a simple, empiric method to estimate whether squeezing may take place, based on experience in the Himalayas using the *Q*-system. The critical overburden when squeezing may occur can be estimated as

$$H > 350\, Q^{1/3} \qquad \text{Eq. (3)}$$

The two authors have problems to find how the value of *Q* is determined in Eq. (3). Figure 6 shows that Eq. (3) in fact, is a loop, as it depends on the input value of *SRF* to be used for finding the *Q* value. This input depends on whether squeezing may take place (which is found from the value of *H*).

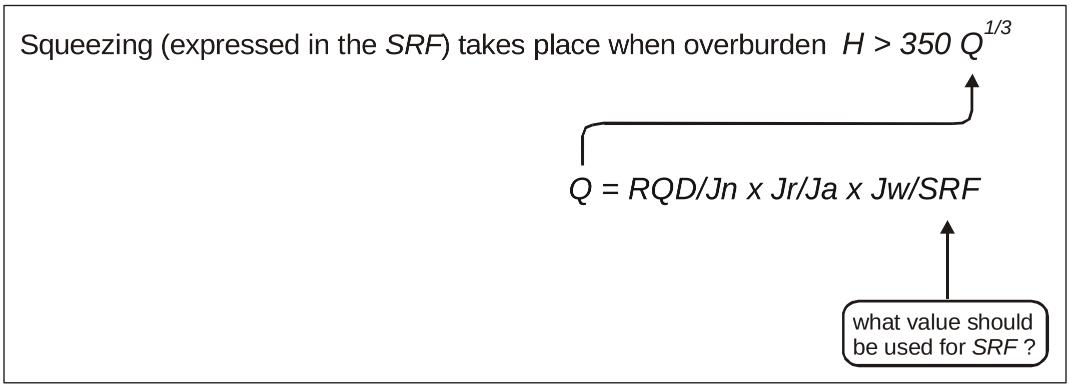



Figure 6. How is the value of *Q* in Eq. (3) found to estimate squeezing? (from Palmstrom et al., 2002)

In addition to the uncertainty in estimating the value of *Q* in Eq. (3) the two authors have not found what type of rocks it includes. Eq. (3) is presented uncritically in several papers on the *Q*-system without any attention to such limits.

A general comment: It is difficult to imagine how a single value of *Q* can be successfully used for calculation of rock support for such a complex case that squeezing is.

***D. Swelling rock. Chemical swelling activity depending on presence of water***

Swelling rock occurs often as complex, anisotrope structures and depends on local, geological and geometrical conditions. It is therefore difficult to characterize using simple, general tables for classification. Swelling rock may occur in weakness zones/faults and some altered rocks containing smectite/montmorillonite, as well as in rock containing anhydrite and gypsum.

In Table 4 the limit between mild and heavy swelling is not indicated. And it is not explained when swelling rock (in group D) should be used for weakness zones containing swelling clay.

The two authors are of the opinion that it is hardly possible to characterize or classify such a complex condition as swelling rock often is. In such ground special investigations and measurements should be performed and special analyses conducted to evaluate the results with respect to excavation and rock support.

## 2.4 Equivalent Dimension, *De*

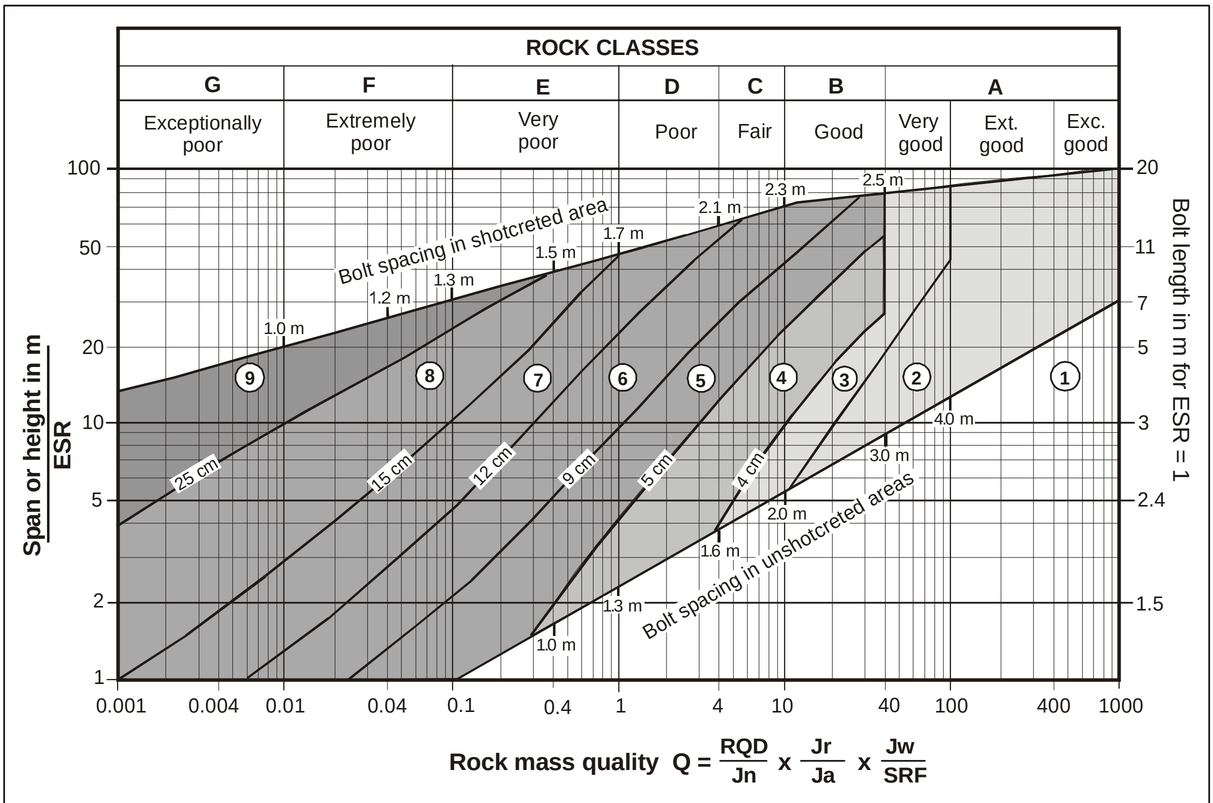
In relating the *Q* value to the stability and support requirements of underground excavations, Barton et al (1974) defined an additional parameter, which they called the *Equivalent Dimension*, *De*, of the excavation. This dimension is obtained by:

$$De = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio, } ESR} \qquad \text{Eq. (4)}$$

The value of *ESR* is related to the intended use of the excavation and to the degree of security, which is influence on the support system to be installed to maintain the stability of the excavation. Barton et al (1974) apply the values in Table 5.

Table 5. The various excavation support ratio categories (from Barton et al., 1974)

| | |
|---|---|
| Temporary mine openings. | *ESR* = 3 - 5 |
| Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations. | 1.6 |
| Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels. | 1.3 |
| Power stations, major road and railway tunnels, civil defence chambers, portal intersections. | 1.0 |
| Underground nuclear power stations, railway stations, sports and public facilities, factories. | 0.8 |




**REINFORCEMENT CATEGORIES:**

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

Figure 7. The *Q* support chart (from Grimstad and Barton, 1993)

The equivalent dimension, *De,* plotted against the value of *Q*, is used to define a number of support categories in a chart published in the original paper by Barton et al (1974). This chart has later been updated to directly give the support. Grimstad and Barton (1993) made another update to reflect the increasing use of steel fibre reinforced shotcrete in underground excavation support. Figure 7 is reproduced from this updated chart.

The *Q*-values and support in Figure 7 are related to the total amount of support (temporary and permanent) in the roof. The diagram is based on numerous tunnel support cases. Wall support can also be found by applying the wall height and the following adjustments to *Q*:

For $Q > 10$ use $Q_{wall} = 5Q$
For $0.1 < Q < 10$ use $Q_{wall} = 2.5Q$
For $Q < 0.1$ use $Q_{wall} = Q$

A road tunnel will from its use have stricter requirements to permanent safety than a water tunnel. Table 6 shows that it is small difference between the rock support in the 10m wide road tunnel (*ESR* =1.0) and in a water tunnel (*ESR* = 1.6) with the same size. Should the effect of *ESR* be that small?

Table 6. Variation in roof support in a road tunnel and a water tunnel, both with 10m span for three different *Q* values (from Palmstrom et al., 2002)

| **Type of tunnel** | ***De*** = tunnel span/*ESR* | **Roof support in different *Q* qualities** | | |
|---|---|---|---|---|
| | | for *Q* = 20 (good) | for *Q* = 2 (poor) | for *Q* = 0.2 (very poor) |
| Road tunnel | 10/1.0 = 10 | syst. bolting 2.4×2.4m | syst. bolting 1.9×1.9m<br>7cm shotcrete | syst. bolting 1.4×1.4m<br>14cm shotcrete |
| Water tunnel | 10/1.6 = 6.3 | no support | syst. bolting 1.9×1.9m<br>5cm shotcrete | syst. bolting 1.4×1.4m<br>12cm shotcrete |

However, the different countries have different requirements to safety and hence the amount of required rock support. This will designate different values of the *ESR*. Also the safety requirements for the working crew in different countries may influence on the *ESR* values to be applied in that country.

Another important issue in this connection is that the permanent rock support in certain types of underground excavations is determined independently of the ground quality. This is, for instance, the case for underground power halls in Norway. Earlier, the roof support in these was cast-in-place concrete arch; today, systematic rock bolting and fibre reinforced shotcrete are used. It is difficult to cover this and similar features in a single *ESR* number.

## 2.5 Concluding remarks on the input parameters

Only natural discontinuities shall be measured in the *RQD* core logging when the *Q*-system is applied during planning of the underground excavation (Löset, 1997). However, for support estimates during construction of drill and blast tunnels, Löset recommends that *"all breaks must be included in the estimate of RQD in tunnels, as all joints regardless of origin, influence on stability"*. This is inconsistent, as the latter includes the additional breaks in the RQD and Jn input. Thus, support estimates before and during/after excavation give different volumes (and types?) of rock support.

However, this is a shortcoming in most classification systems when observed rock mass characteristics are used to estimate the conditions for planning without including input of the excavation method of the tunnel, shaft or cavern. An excavation damage factor or similar should be applied, but none of the empirical or other tools in rock engineering makes use of this.

For all classification systems there are difficulties in observing appropriate jointing characteristics. In the same location, different people may map the joints differently, as shown in Figure 8. Along the same scanline, the 6 observers mapped 17 to 21 joints, partly in different locations. For the *Q*-system, the joint features, *Jr, Ja* and especially *Jn*, in addition to *RQD* are prone to mischaracterization as mentioned earlier. With better descriptions of some of these parameters, such errors may be reduced.

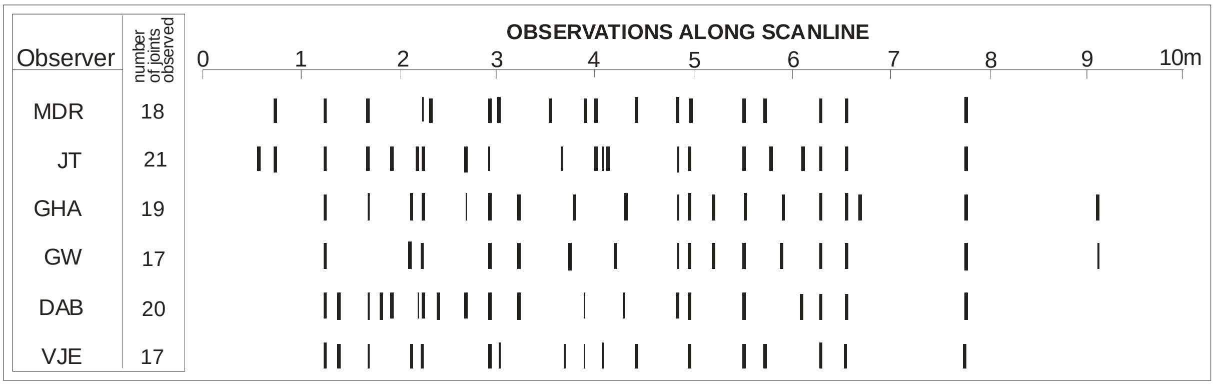



Figure 8. Position of joints recorded by different observers along the same scanline. From a test in the Kielder aqueduct tunnels (slightly modified from Ewan et al. 1983)

The following important features are not included in the *Q*-system:

- Joint orientation, which was not found to be an important, general parameter. Part of the reason for this may be that the orientations of many types of excavations can be, and normally are, adjusted to avoid the effect of unfavourably oriented joints. However, this choice is often not available. The parameters *Jn*, *Jr* and *Ja* were considered to play a more important role than orientation. If joint orientations had been included, the classification would have been less general, and its essential simplicity lost.
- Joint size. The fact is that larger joints have a markedly stronger impact on the behaviour of a rock mass than smaller. According to Piteau (1973), the size of joints is essential in evaluations, since the strength reduction on a failure surface, which contains a discontinuity, is a function of the joint size. Also ISRM (1978) and Merritt and Baecher (1981) mention this. It is often a problem to observe or measure the joint lengths, a main reason is that the whole joint plane seldom can be seen in rock exposures. This may be a main reason why this feature is not used in the *Q*-system. But practice shows that it is easy from observations to divide between cracks or small joints and large joints. Hudson and Priest (1983) recommend that a measure of joint length should only indicate the length/size interval of the joint, and the experience is that a characterization of joint length can be made in intervals of < 1m, 1 – 3m, 3 – 10m, and >10m.
- Joint persistence. Whether the joints are continuous or discontinuous may often be an important input for estimates of instability and rock support. For instance, a failure plane involving discontinuous joints (i.e. joints ending in massive rock) must partly pass through intact rock.
- Joint aperture. The opening of joints, which is very important for ground water movement and conductivity of the rock mass. This is of main importance when evaluating water inflow to underground excavations.
- Rock strength. The reason for not including rock strength in the *Q* is that it has little impact on the ground behaviour in many cases, especially for jointed rock where

instability is caused by block falls. For other types of ground, e.g. rock stress problems the compressive strength of rock material has a significant influence. But for such cases the rock properties are used to evaluate the actual behaviour (degree of bursting, degree of squeezing, etc.) for input into the *SRF*. But, when the *Q* system has been extended to cover also other fields than stability and rock support, the rock strength may play an important feature. Therefore, $Qc = Q(\sigma_c/100)$ has been introduced, see Table 1. But so far, the two authors have not found any recommendations to when *Qc* should be used instead of the "old *Q*".

Thus as shown, there are several limits in the input parameters to *Q*, especially *RQD*, *Jw* and *SRF*. The latter is a complicated factor. The impression of the authors is that *SRF* is a sort of 'correction factor' or 'fine tuning factor', rather than a factor expressing 'active stresses' aiming at arriving at a *Q* value that gives appropriate rock support.

# 3 PRACTICAL USE OF THE *Q*-SYSTEM - LIMITATIONS AND MISUSE

## 3.1 Suitability and usefulness

"*The Q support chart shows that a 10m span road tunnel (with ESR = 1.0) could remain without support when Q = 100, at least when following typical NMT philosophy. The characterization of this rock mass satisfy all the criteria of permanently unsupported excavations. It has only one joint set, which has a dilatant character, and there is no water.*" (Barton, 2002)

Such statements easily give the impression that classification systems cover larger field of ground conditions than what is the case. For instance, the quotation above is not relevant for the following conditions:

- A tunnel located at moderate depth in a dry quartzite with one joint set spaced 15 – 20cm (a flagstone) (where *Q* = (90 –100)/2 × 2/1 × 1/1 = 90 - 100). Such ground in a Norwegian tunnel would require at least roof support by systematic rock bolting.
- A tunnel located at moderate depth in a dry mica schist with foliation joints widely spaced (0.3 - 1m (with *Q* = (90 –100)/2 × 2/1 × 1/1 = 90 - 100) will at least require rock support by systematic rock bolting.
- A tunnel located at moderate depth in a dry friable, sandstone with few joints (*Q* = 100/0.5 × 1/2 × 1/1 = 100) will need support of at least a 6cm layer of shotcrete.

### *3.1.1 Caution is necessary*

The authors agree very much with a statement made by Karl Terzaghi in his latest years:
*"I am more and more amazed about the blind optimism with which the younger generation invades this field, without paying attention to the inevitable uncertainties in the data on which their theoretical reasoning is based and without making serious attempts to evaluate the resulting errors."*

The use of the *Q*-system, and also other classification systems, requires in addition to a general understanding of rock engineering, also good knowledge about the basis, the structure and the limitations for their use. It is important that the user knows the premises for the *Q*-system and that he or she has necessary understanding and experience to measure relevant values of the appropriate parameters.

In the opinion of the two authors, too little emphasis has in many papers been put on describing the originally intended field of use of the *Q* system and on discussing limitations. Though some limitations have been given for some of the many equations given in one of the later publications (Barton, 2002), one can easily get the impression that the system covers all kinds of rock masses and ground conditions. A reason is that, as soon as equations are presented in the field of rock mechanics and rock engineering, possible restrictions are quickly forgotten and the equation is easily applied uncritically.

### *3.1.2 The support predicted*

Figure 9 shows a frequent feature when using case records in empirical rock engineering methods: generally poor correlation between the ground quality and the support used.

The different excavation and rock supporting practices in various countries, as well as the requirements to the permanent support, cause that the methods and amount of rock support can vary largely from tunnel to tunnel. This is a problem when experience from different regions is used to calibrate the support recommendation in a classification system. Figure 9 shows the correlation between bolt spacing and the *Q*-value in areas without sprayed concrete. It is on the basis of such crude results that many of the support recommendations in Figure 7 are established. The users quickly forget on which averaged, inaccurate basis the rock support given in the support chart, is based. This is not a special limitation of the *Q* system, but is a common feature for most quantitative classification systems used for rock support.

Therefore, the *Q* support chart gives only indication of the support to be applied, and it should be tempered by sound and practical engineering judgement.

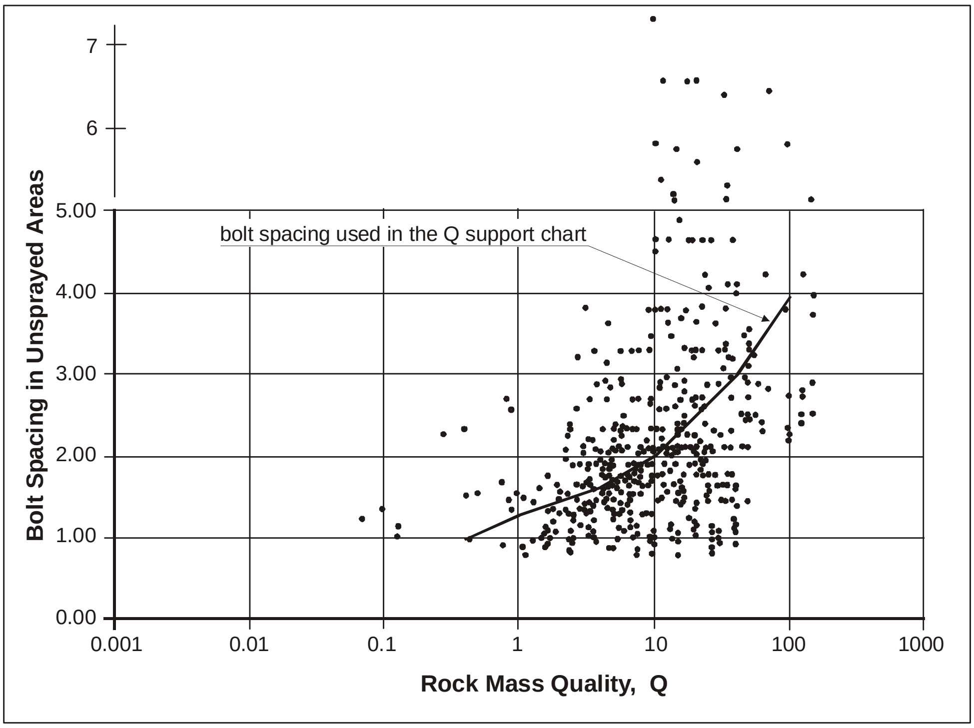



Figure 9. Bolt spacing related to *Q*-value in unsprayed areas (modified from Grimstad and Barton, 1993). The line is indicating the bolt spacing used in the *Q* support chart.

#### *3.1.3 Can the Q-system be used both in the planning and during the construction stage?*

In a comprehensive NGI-report on the practical use of the *Q*-system, Löset (1997) is of the opinion that the *Q*-values give an indication of what kind of support that is needed in a tunnel, but that the detailed design of the support, like for instance the placing of rock bolts, is not covered by the *Q*-system. However, Löset also writes that "*the Q-system may also be used at the planning stage*", thus indicating that the system as he sees it, can be used during construction.

Bieniawski (1997) is of the opposite opinion that: *"Rock mass classifications on their own should only be used for preliminary, planning purposes and not for final tunnel support."* Similarly, Hoek & Brown (1980) *"recommend classification systems for general use in the preliminary design of underground excavations".*

These statements imply that classification systems should not be used to decide support during the construction stage or for the design of the permanent support. The authors of this paper concur with these statements, but realise that, for instance, the *Q*-system may be used as an indicator and give useful documentation provided that the ground conditions are within the real limitations of the system and that the indications are subject to thorough judgement by experienced engineers.

Temporary or initial support is needed to establish safe working conditions for the tunnelling crew. It is installed at or near the tunnel working face and is meant to secure the rock until the excavation and installation works are finished. Later, additional permanent support is installed to ensure proper function of the underground excavation throughout its lifetime. Often, the types used for temporary and permanent support are the same, and sometimes all necessary (permanent) support is installed at the face, thus in this way including or replacing the temporary support. Barton et al. (1977) have shown how the *Q* system can be used for estimating the types and amount of temporary support. Referring to the mentioned different ways of performing temporary support, it is generally not possible to cover this type of rock support in the *Q* system. However, a theoretically amount of such support may be assumed if enough information is available of the stability conditions.

### 3.2 Limitations of the *Q*-system

#### *3.2.1 What is the span of the Q-system?*

In a recently published and rather comprehensive paper by Barton (2002) it is said in one of the conclusions that *"the broad, six-order of magnitude Q-value scale, and the even broader nine-order of magnitude $Q_c$-value scale, give relatively simple correlations with parameters needed for design, due to the fact that rock masses also display a huge range of strengths, stiffnesses and degrees of stability or instability. An RMR or GSI scale of only about 10-100, i.e. one order of magnitude, cannot easily correlate with phenomena as different for instance as the landmark Sugarloaf Mountain of Rio de Janeiro, where Q may approach 1000, or piping failure causing a tunnel to fill with 7000 m³ of claybearing quartzite (and much greater volumes of water). Here, Q may have approached a limit of 0.001, until $J_w$ improved due to relief of some of the extreme water pressure"*

In other words, the reader may wonder if the *Q*-system is a better system than *RMR* ("Rock Mass Rating") and *GSI* ("Geological Strength Index") because the *Q*-values, which are based on multiplication, cover a larger scale. The two authors of this paper are of another opinion.

Figure 10 shows an updated *Q* support chart based on Grimstad and Barton (1993). A critical evaluation of the usefulness of this may be as follows:

I. Good or better ground qualities in rock class A (in Figure 7) is where *RQD* > approx. 90 occur. As shown in Figure 1 large blocks are outside the limits of *RQD* to correctly characterize jointing. However, this reduced ability of *RQD* may give significant errors in the *Q* to characterize the ground where blocks larger than approx. 1 $m^3$ occur. The following two examples where *RQD* = 100, *Jr* = 4, and *Ja* = *Jw* = *SRF* = 1, but with different block sizes and numbers of joints sets, illustrate this:

   1. For $Vb_{(1)}$ = 1$m^3$ and one joint set (with spacing 0.3m) + some random joints
      $Q = RQD/Jn \times Jr/Ja \times Jw/SRF = 100/3 \times 4/1 \times 1/1 = 132$
   2. For $Vb_{(2)} = 10 \times Vb_{(1)}$ = 10$m^3$ and three joint sets (spaced 1m, 2m, and 5m) + some random joints
      $Q = 100/12 \times 4/1 \times 1/1 = 33$

   Thus, for block volume 10 times larger in example 2, the *Q* value is only 25% of example 1. This indicates that for *Q* > approx. 40, the accuracy of the *Q* is considered limited. However, regarding rock support, little or no support is generally needed in this interval, meaning that it is not important whether the *Q*-value is 40, 100 or 1000.

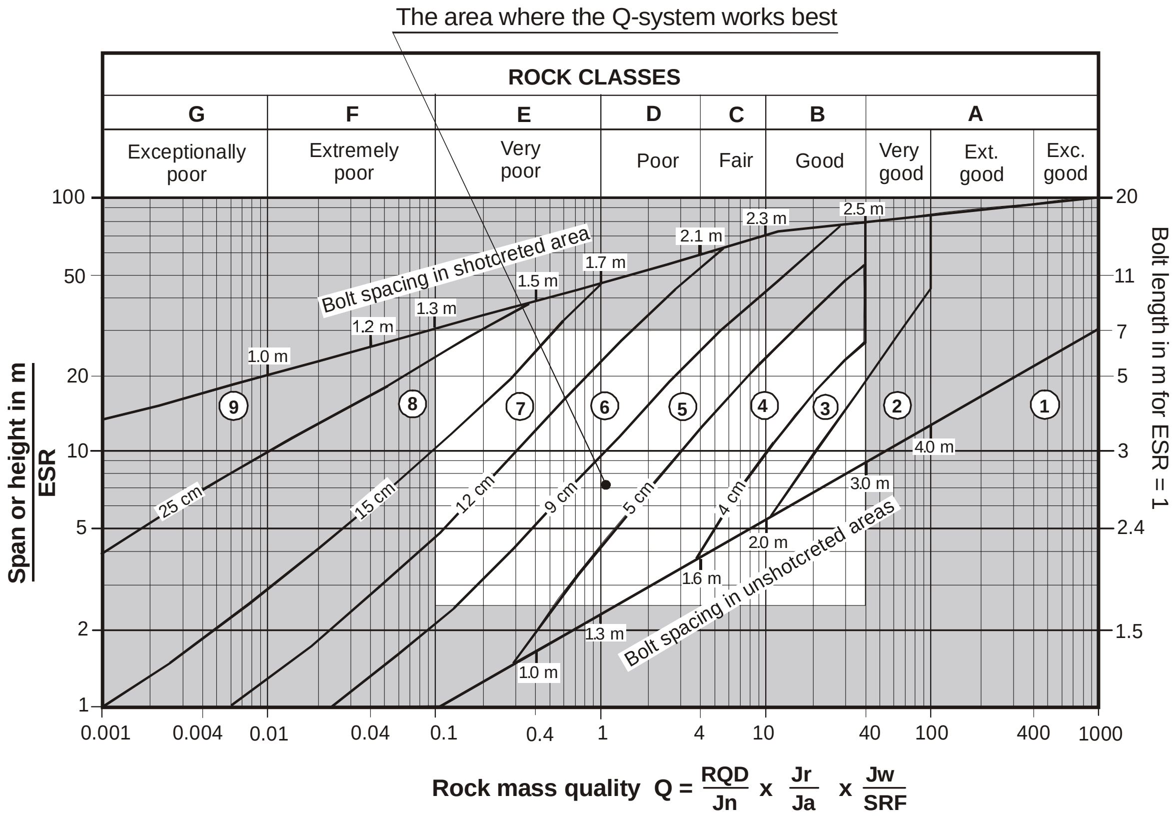
II. Rock masses belonging to classes F and G (extremely and exceptionally poor) occur mostly in weakness zones, squeezing and swelling rock, or very large water inflow. These are conditions, which are difficult to characterize, as is described in Section 2.3.2. In this interval the conditions are so difficult that in most cases it would be irresponsible to base the decision about type of support on a rock mass classification only. In such cases the local, actual conditions, which are only partly reflected in the classification system, will play an important role. As concluded by Palmstrom and Stille (2005), the *Q* system is best applied in jointed rock. The lower boundary here is in the opinion of the two authors given by the following values of the input parameters:

   *RQD* = 0, (i.e. in the Q system minimum *RQD* = 10), *Jn* = four or more joint sets = 15, *Jr* = smooth and planar = 1, *Ja* = clay filling = 12, *Jw* = no or little water (because of the clay fillings) = 1, and *SRF* = medium stress level = 1, which give $Q = 10/15 \times 1/12 \times 1/1 = 0.06$. Below here there are as mentioned, generally great uncertainties connected to the characterization of the ground features as well as lack of the *Q* system to cover such conditions.

For practical use one is therefore left with a support-diagram, which is useful for the range approx. 0.1 < *Q* < approx. 40, or "*one and a half order of magnitude*" to return to the citation above.

Concerning the equivalent dimension (span or height/ESR), the diagram in Figure 10 is also unrealistically large. In practice, the smallest dimensions of an excavation where the predicted rock support can be executed, will be 2 to 3m, while nobody should base the design of support for more than 30 to 40m span of cavern roofs or walls only on *Q*-values. In Figure 10 the shaded parts of the diagram are therefore regarded as interesting, but not reliable, being unrealistic for practical use.

We are now left with a diagram showing ground varying in quality from Very poor to Good, what might be referred to as normal hard rock conditions, and tunnels or caverns of common size (3 – 30 m span or height), which should be supported with a combination of rock bolts installed with a spacing varying from 1.3 to 3m and steel fibre reinforced shotcrete with thickness varying from 4 to 25cm. This is a recommendation most tunnel engineers with some experience easily accept for an empiric system, covering ground that mostly can be characterized as blocky, where instability in the excavation mainly is caused by block falls.




**REINFORCEMENT CATEGORIES:**

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

Figure 10. Limitations in the Q rock support diagram. Outside the unshaded area supplementary methods /evaluations /calculations should be applied (from Palmstrom et al., 2002)

### *3.2.2 Benefits and limitations using classification systems*

The use of the *Q* classification system (and also other classification systems) can be of considerable benefit during the feasibility and preliminary design stages of a project, when very little detailed information on the rock mass and its stress and hydrologic characteristics is available. At its simplest, it may be used as a check-list to ensure that all relevant information has been considered. In a more comprehensive way, the rock mass classification can be used to build up a picture of the composition and characteristics of a rock mass to provide initial estimates of support requirements for tunnels.

For this, two approaches can be taken:

– One is to only evaluate the parameters to be included in the classification used.
– The other is to accurately characterise the relevant rock mass features and then attribute parameter ratings at a later time, as shown in Figure 11.

The latter method is recommended, since it gives a full and complete description of the rock mass, which can easily be translated into either classification index. If rating values alone had been recorded during mapping, it would be almost impossible to carry out verification studies.

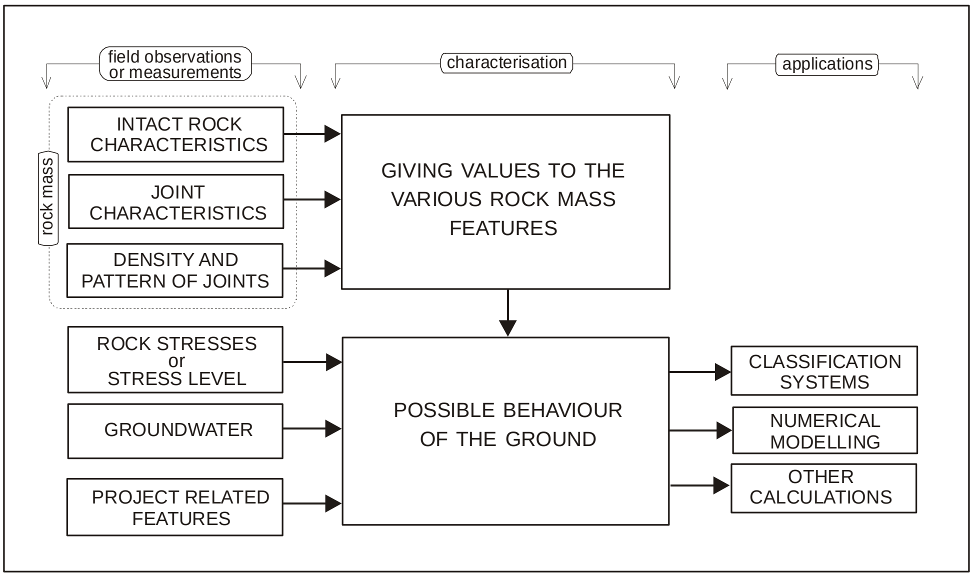



Figure 11. Rock mass characterisation and classification (from GeoEng2000 workshop in Melbourne)

It is important to understand that the use of a rock mass classification system like the *Q*-system does not (and cannot) replace some of the more elaborate design procedures. However, the use of design procedures requires access to relatively detailed information on in situ stresses, rock mass properties and planned excavation process or sequence, none of which may be available at an early stage in the project. When this information becomes available, the use of the rock mass classification schemes should be updated and used in conjunction with site specific analyses.

Important in all rock and tunnel analyses is a sound understanding of the geological formation and of the composition of the rock mass at site, the stress and ground water acting, and how the ground behaves in the tunnel.

As shown by Palmstrom and Stille (2005) the *Q*-system has its best applications in jointed rock masses where instability is caused by block falls. For most other types of ground behaviour in tunnels the *Q*-system, like most other empirical (classification) methods, has limitations, see Table 7.

It is also worth repeating that the use of at least two rock mass classification schemes is advisable during the design process (Bieniawski, 1984, 1989). For conditions outside the 'area where the Q system works best' in Figure 10, the two authors strongly recommend to also apply other tools in the rock engineering process.

Table 7. The fitness of the *Q*-system in various types of ground behaviour in tunnels (simplified from Palmstrom and Stille, 2005)

| **Types of ground behaviour** | **Suitability *)** |
|---|---|
| Stable | 2 |
| Fall of block(s) or fragment(s) | 1 - 2 |
| Cave-in | 3 |
| Running ground | 4 |
| Buckling | 3 |
| Slabbing, spalling | 2 |
| Rock burst | 3 - 4 |
| Squeezing ground | 3 |
| Ravelling from slaking or friability | 4 |
| Swelling ground | 3 |
| Flowing ground | 4 |
| Water ingress | 4 |
| **) 1 Suitable; 2 Fair; 3 Poor; 4 Not applicable* | |

## 3.3 Extended applications of the *Q*-system

Engineers are often so keen to use mathematical expressions in their work that they easily forget restrictions or limitations connected to the actual equation. For the correlation between the *RMR* and *Q* classification systems, often used when experience in one system is applied in the another, the conditions are shown in Figure 12. Because of the rather poor correlation between the two systems the equation $RMR = 9 \ln Q + 44$ or similar should be applied with great care to avoid errors. It is hoped that this is the case when RMR values have been applied by Barton (2002) and others in some of the *Q* system extensions presented.

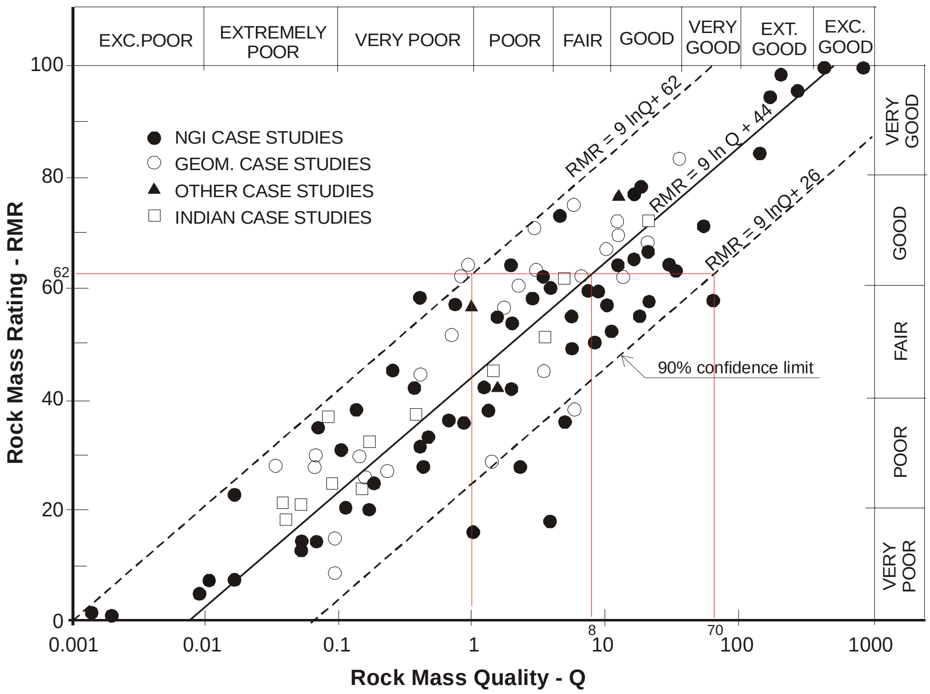



Figure 12. The correlation between *RMR* and *Q* (from Bieniawski, 1984 with data from Jethwa, 1981). Example: for *RMR* = 62 *Q* spans from 1 to 70 (very poor/poor to very good)

### *3.3.1 Q values found from seismic velocities*

Another conclusion in the already mentioned paper, Barton (2002), is the following: "*Seismic P-wave velocity $V_p$, and static modulus of deformation $E_{mass}$ can clearly be linked, due to their individual relationship with the Q-value, which has been normalised by consideration of uniaxial compression strengths different from 100 MPa. The resulting $Q_c$-value is reduced or increased in proportion to $\sigma_c$ , which removes the need for "mobilisation" of $\sigma_c$ through the strength to stress ratio found in SRF. A potential linkage of the high pressure and therefore deforming Lugeon test value with $Q_c$ has also been identified, and a theoretical basis for this has been discussed, assuming an absence of clay in the joints.*"

Reasoning and conclusions or statements of this type shows that one is stretching a classification system way beyond what it originally was developed for.

The refraction seismic measurements performed for the North Cape subsea road tunnel in Figure 5 showed seismic velocities mostly around $v_p$ = 5000 m/s. This gives according to Grimstad and Barton (1993), and Barton (2002), $Q = 10^{(Vp - 3500)/1000} = 31.6$ with spot bolting as estimated rock support for the 8.5m wide tunnel 20 - 50m below the surface. The necessary support used was cast-in-place concrete lining, which is far more than given by the *Q* system.
In this connection it is also worth mentioning that in an internal report by NGI (2001), the results from refraction seismic measurements in several Norwegian tunnels in hard rocks are compared with *Q*-values and *Q* parameters found from tunnel mapping. The following conclusions are presented: *"Jr and Jw and in part SRF seem irrelevant for connection between rock mass quality and seismic velocities."* This statement is in good agreement with our experience and opinion.

### *3.3.2 Finding Em from values of Q*

Quantitative classification systems are frequently used to estimate the deformation modulus of rock masses (*Em*). Similar to the *RMR* and the *RMi* systems, simple equations have been presented for this, which from the *Q* system can be found as

$$Em = 25 \log_{10} Q \quad (\text{for } Q > 1) \quad \text{(Grimstad and Barton, 1993)} \qquad \text{Eq. (5)}$$

or

$$Em = 10 Q_c^{1/3} = 10(Q \times \sigma_c/100)^{1/3} \quad \text{(Barton, 2002)} \qquad \text{Eq. (6)}$$

In Eq. (6) errors may have been introduced as *RMR* experience has been transformed to *Q* values. In addition, the many limitations described earlier in this paper cause that there are several restrictions when Eqs. (5) and (6) are used. The main restrictions are:

- Limited accuracy for high *Q* values (where *RQD* = 100 limits a concise characterization)
- Limited accuracy for low *Q* values in Eq. (6) (where *RQD* = 0 limits a concise characterization)
- Limits for large ground water inflows (*Jw* < 0.66)
- Limits where overstressing takes place,
- Rock strength is not considered (in Eq. (5)
- Swelling, squeezing and most weakness zones are not covered

The above restrictions mean that *Q* may only be appropriately used within the area given in Figure 10.

### 3.3.3 *Experience with the $Q_{TBM}$*

In 1999 a new method for predicting penetration rate (PR) and advance rate (AR) for TBM tunnelling was introduced (Barton, 1999). The method is based on an expanded *Q*-system and average cutter force in relations to the appropriate rock mass strength. Orientation of fabric or joint structure is accounted for, together with the compressive or point load strength of the rock. The abrasive or non-abrasive nature of the rock is incorporated via the cutter life index (*CLI*). Rock stress level is also considered. The new parameter $Q_{TBM,}$ can, according to Barton (1999), be estimated during feasibility studies, and can also be back calculated from TBM performance during tunnelling.

The components of the $Q_{TBM,}$ are as follows:

$$Q_{TBM} = (RQD_o/Jn)\times(Jr/Ja)\times(Jw/SRF)\times(SIGMA/F^{10}/20^9)\times(20/CLI)\times(q/20)\times(\sigma_\theta/5) \quad \text{Eq. (7)}$$

where: $RQD_o$ = RQD (%) measured in the tunnelling direction
*Jn, Jr, Ja, Jw*, and *SRF* ratings are unchanged from the original *Q*-system
*SIGMA* = Rock mass strength estimate (MPa) found from a complicated equation including the $Q_o$ value measured in the tunnel direction (the same as the six first parameters)
*F* = Average cutter load (ton, ~10kN ) through the same zone, normalised by 20 tons
*CLI* = Cutter Life Index (from NTH/NTNU, see Bruland, 1998)
*q* = Quartz content in percentage terms, %
$\sigma_\theta$ = Induced biaxial stress on tunnel face (MPa) in the same zone, normalised to an approximate depth of 100m

Experience with the use of the $Q_{TBM}$ is discussed by Sapigni et al. (2002) in a paper with the title ”*TBM performance estimation using rock mass classification*”. Three tunnels with a total length of 14 km in hard, metamorphic rocks in northern Italy were thoroughly mapped in accordance with the *RMR* method and the *Q*- and $Q_{TBM}$ – methods. Based on statistical analyses, the authors find a reasonable correlation between the *RMR* values and the net penetration rate, but conclude that the spread in the results are so great that the results are of limited practical use for the prediction of TBM performance.

In their conclusions the authors also say that correlation both with the *Q*-values and with the new $Q_{TBM}$ is poorer. They find it particularly difficult to explain the lack of correlation with the $Q_{TBM}$, as this is supposed to be based on rock mass-machine interactions parameters with special relevance for TBM use. Thus they say: *"In particular, $Q_{TBM}$ shows low sensitivity to penetration rate, and the correlation coefficient with recorded data is even worse than conventional Q or other basic parameters like the uniaxial compressive strength of the intact rock."* The results are shown in Figure 13.

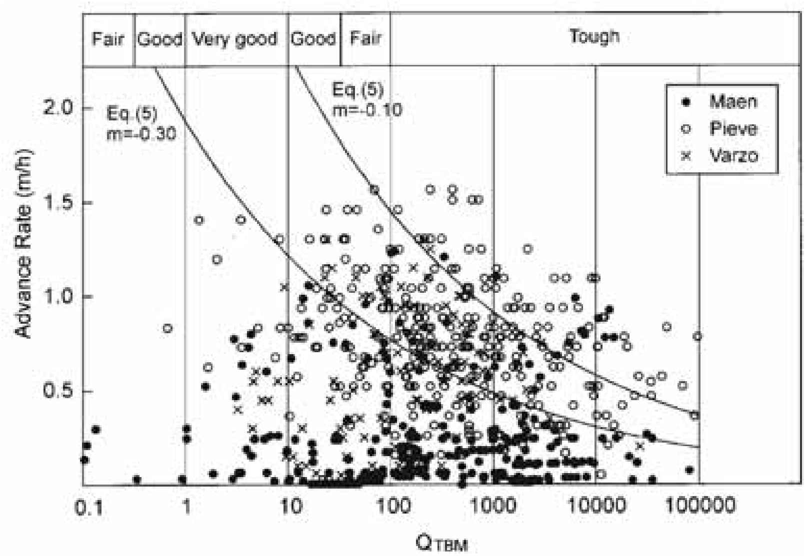



Figure 13. Advance Rate for three TBM tunnels plotted against $Q_{TBM}$ (Sapigi et al, 2002)

Sapigi et al. (2002) also say that "*obviously the reliability of the $Q_{TBM}$ model cannot be judged by an individual case, but the mismatch underlines the difficulties involved in performance prediction when so many factors (rock mass condition, machine and muck removal system characteristics, human experience) are involved.*"

Such experience clearly supports our main objection against the development, not to say misuse, of the *Q*-method that has appeared in the later years. This kind of extension goes far beyond what the method once was intended to cover.

For more information on the $Q_{TBM}$ in this connection, see Blindheim (2005).

### *3.3.4 Other examples of extending the Q system*

Attempts are being made to explore the possibilities for using the *Q*-system for evaluation of permeability and grouting of tunnels, Barton et al., 2000/2001. In this connection, the following simple connection has been presented between the Lugeon and the *Q* value:

$$L \approx 1/Q \qquad \text{Eq. (8)}$$

For this kind of use we have the following comments:

A. According to Barton et al. (2000/2001) the permeability in a rock mass generally increases with increasing degree of jointing, i.e. for decreasing *Q*. But joint aperture and possible channel formation, which both are very important in ground water movement, are not included in the input parameters to *Q*.
B. For single joints the highest permeability has the highest (*Jr/Ja*) value (and hence the highest *Q* value) as is indicated for case B in Figure 14. This is the opposite of the experience addressed in item A above.

These two simple contrary conditions are so important that the *Q*-system hardly can be regarded as suitable for evaluation of permeability, water inflow and grouting.

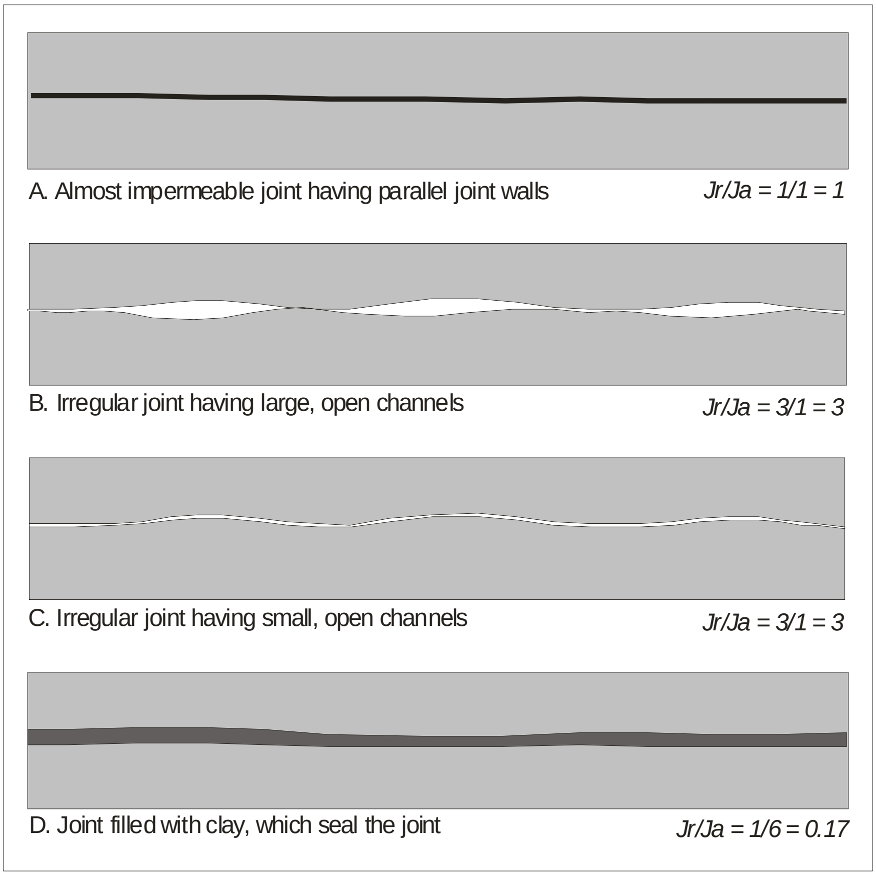



Figure 14. Four types of joints with different joint properties. A and D have very low conductivity, but different impact on the *Q* value, while B and C have the same influence on the *Q* value, but different conductivity (from Palmstrom et al, 2002)

## 3.4 Use of the *Q*-system in Norway

In Norway the *Q*-system has gained increasing use as a useful tool, particularly at the planning stage in connection with estimates of rock mass qualities and the expected volume of support. For this purpose it has proved its usefulness, as it gives the people involved the possibility to refer their experience to a common framework. Classification systems also stimulate to a more conscious use of the verbal descriptions of rock mass qualities. Used in tender documents, the *Q*-values give the tendering contractors a possibility to supplement their interpretations of the verbal descriptions of the rock masses along a tunnel alignment with some numbers.

When classification systems are used in tender documents, it is important to inform how the parameters in the calculations are obtained and interpreted. If no such information is given, it may easily end up in misunderstandings and claim cases.

Typical for Norwegian tunnelling is that contracts are based on unit prices and that the Contractor is responsible for the safety in the tunnel during excavation. Thus, the support to be applied is normally decided by the acting tunnel foreman. The types of support used are determined in cooperation with the Owner With experienced tunnel workers and foremen there has been no need to introduce classification systems for decisions about the temporary support.

As far as the authors know, no Norwegian contractor has ever made a systematic *Q*-value mapping of a tunnel. In some tunnels the Owner or the Engineer has carried out systematic tunnel mapping including $Q$-value estimations, as documentation of the ground conditions and in part final support or lining, and with the intention of building up general experience on tunnelling. It may, for instance, be of interest to compare the as-built rock mass quality with the estimated rock mass quality in the tender documents.

## 4 CONCLUDING REMARKS

Classification systems, and not least the $Q$-system, may be useful tools for estimating the need for tunnel support at the planning stage, particularly for tunnels in hard and jointed rock masses without overstressing. There are, however, a number of restrictions that should be applied if and when the system is going to be used in other rock masses and in complicated ground conditions. So far such restrictions have not been much discussed in available literature. In this paper a critical evaluation of the parameters that make up the system, is carried out. Potential users of the $Q$-system should carefully study the limitations of this system as well as other classification systems they may want to apply, before taking them into use.

Benefits using the $Q$-system to specify the need for grouting and also the effect of grouting on stability in tunnels are recently published. The authors of this paper are very doubtful to such applications of the $Q$-system.

$Q_{TBM}$ is a recently introduced tool to estimate "Penetration Rate" (PR) for TBM. Together with a performance parameter '*m*' this is supposed to give an estimate of the "Advance Rate" (AR). We regard several of the input parameters as irrelevant or even as misguiding for TBM performance, and the total effects are difficult to follow in the model. Comparisons so far are indicating very poor or no correlations to obtained results. Like Blindheim (2005) we recommend the $Q_{TBM}$ -system not be used.

When developing classification systems and other tools to evaluate or "calculate" nature, it is of crucial importance to keep in mind the innumerable variations that occur in rock masses and the uncertainties involved in observing and recording the different parameters. During planning and construction of tunnels and caverns it is thus of great importance to apply sound judgement and practical experience. A good overview of how the different parameters may have an influence is vital, as well as respect and understanding of the complexity of the task.

This paper has mostly been concentrated on the $Q$ system, but many of the comments given on the limitations apply also to other classification systems having similar input parameters as $Q$. A solution often used in works on rock engineering is to link $Q$ values to other classification systems - or opposite - applying correlation equations. This is a procedure we strongly do not recommend.

The authors want to conclude by some words of wisdom:

*"The geotechnical engineer should apply theory and experimentation but temper them by putting them into the context of the uncertainty of nature. Judgement enters through engineering geology."* Karl Terzaghi in his latest years.

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