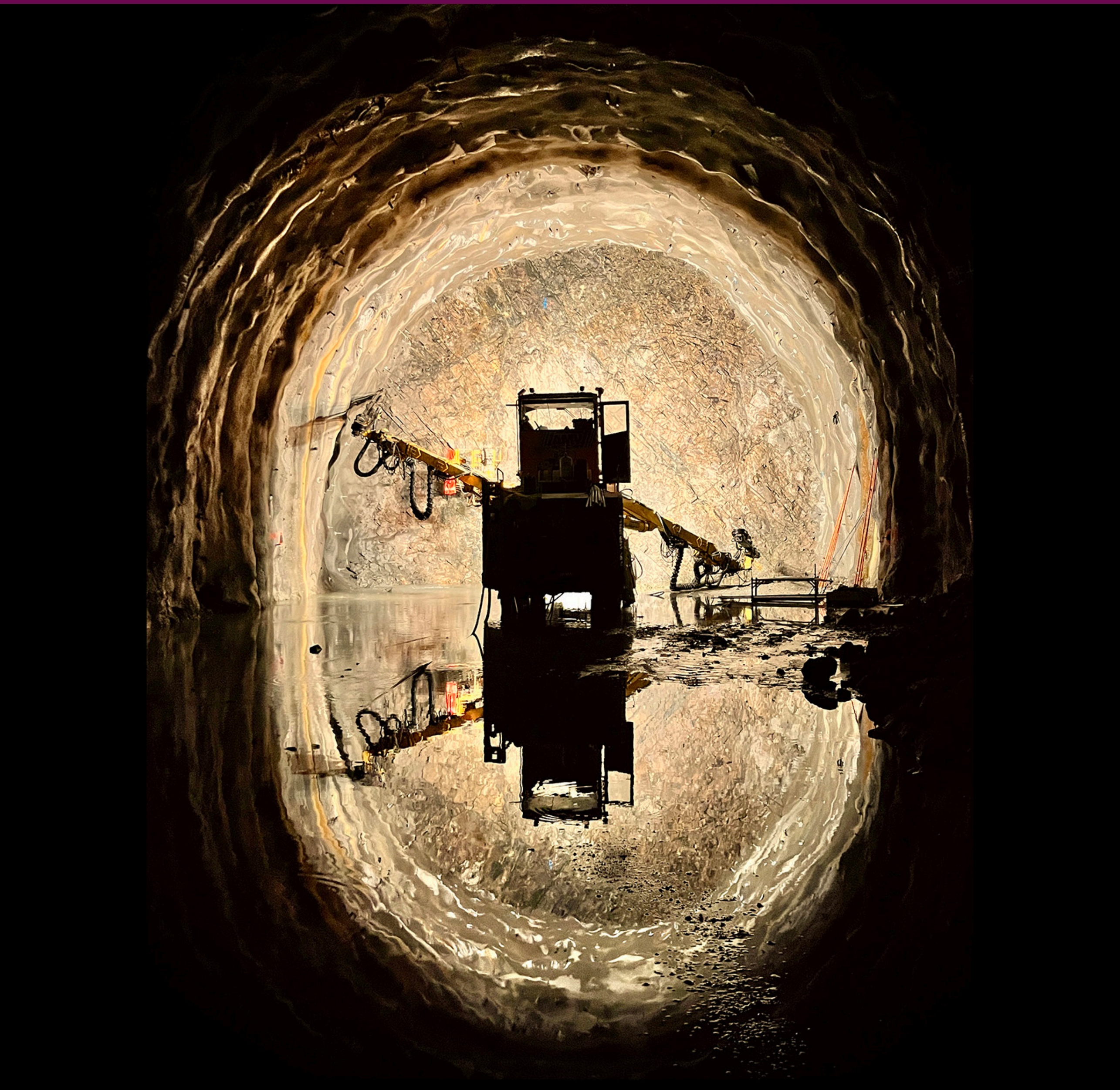
HANDBOOK

Using the Q-system

Rock mass classification and support design



Oslo, April 2025

For further information on this handbook please contact:

NGI
P.O. Box. 3930 Ullevaal Stadion
NO-0806 Oslo
Norway

www.ngi.no

Cover photograph from Mossetunnelen by Erlend Skretting

Preface

The handbook "Using the Q-System, Rock Mass Classification and Support Design" is a guide for the application of the Q-System. NGI's work on developing the Q-System for rock mass classification began in the early 1970s and was first published in 1974 (Barton, Lien and Lunde, 1974).

NGI has continuously improved and updated the system and released the first version of the handbook in 2013. Based on the increasing number of questions and feedback from users with varying backgrounds and experience, this revision of the handbook includes several clarifications with additional explanations. The new revision of the Q-handbook is primarily an update of the guidelines on how to use the Q-system. Key changes in the revised Q-handbook include:

- More clarifications and detailed explanations regarding the use and limitations of the Q-system.

- New subchapters addressing the prerequisites for using the Q-system and guidance in areas where significant variations in Q-parameters have been mapped

- Minor adjustments in support recommendations due to evolving industry practices and developments in technology and materials.

For details, see separate changelog.

Contents

1 Introduction

1.1 History

The Q-system was developed at NGI between 1971 and 1974 (Barton, Lien and Lunde, 1974). Since the introduction of the Q-system there has been considerable development within support philosophy and technology in underground excavations. Several new types of rock bolts have been introduced, and the continuous development of fibre reinforced technology has significantly changed the support procedure. The use of sprayed concrete has become widely accepted, even for good quality rock masses, due to increasing safety requirements in recent years. Reinforced ribs of sprayed concrete have largely replaced cast concrete structures.

The Q-systems support chart has been revised several times and published in conference proceedings. An extensive update in 1993 was based on 1050 registrations, primarily from Norwegian underground excavations (Grimstad and Barton, 1993). In 2002, another update was made based on more than 900 new registrations from underground excavations in Norway, Switzerland, and India. This update also included analytical research regarding the thickness, spacing, and reinforcement of reinforced ribs of sprayed concrete (RRS) as a function of the load and the rock mass quality (Grimstad et al., 2002). The recommendations for use of RRS is primarily based on experience, along with deformation measurements, load documentation, and numerical calculations.

In this revision of the Q-handbook, minor adjustments have been made to the support chart, along with some clarifications regarding the recommended use of the Q-system.

1.2 Areas of application

The Q-system is a *rock mass classification system* to assess the stability of tunnels and underground excavations. *Rock mass classification* refers to quantifying the quality of a rock mass based on defined criteria and categorizing it into specific groups.

The Q-value of a rock mass is based on six parameters, which gives a numerical value to the rock mass with a corresponding rock mass class. The Q-value is primarily used for classifying the rock mass surrounding underground excavations and tunnels but can also be applied to core logging and field mapping at the surface. The mapped Q-value can be linked to recommended permanent support through a schematic support design chart. The support chart is developed by finding the correlation between the mapped Q-value and the amount of support. This means that by calculating the Q-value, it is possible to determine the type and amount of support that has been previously used in rock masses with similar qualities. When using the Q-system for determining support, the dimensions of the underground opening/tunnel and safety requirements are also considered. Thus, the Q-system can be used as a guideline for determining necessary rock support and for documenting the quality of the rock mass.

Rock mass characterization involves an engineering approach to defining and describing the distinctive features of the evaluated rock mass. Such characterization should be used in challenging rock mass conditions, where pure rock mass classification may provide an incomplete basis for decision-making or final rock support design. For more details, see Chapters 4.7 and 5.2.

The Q-system has the following engineering geological applications:

- Mapping in tunnels and underground openings (see Chapter 5)
- Field mapping at the surface (see Chapter 6.2)
- Core logging (see Chapter 6.3)

The Q-value is most accurate when based on mapping in tunnels and underground openings. When used in connection with field mapping at the surface, core logging, and investigations in boreholes, some of the parameters may be difficult to estimate. Q-values from field mapping and boreholes as a basis for preliminary investigations for underground facilities are often associated with greater uncertainty. For more details, see Chapters 6.2 and 6.3.

1.3 Prerequisites for using the Q-system

It is assumed that users of the Q-system have basic knowledge of engineering geology and/or geology and are familiar with geological terms through professional experience or studies. The use of the Q-system for determining rock support during site follow-up requires a training period for inexperienced personnel.

Rock mass classification is based on subjective assessments, which naturally leads to some variation in the determination of Q-values from person to person. In larger projects where multiple engineering geologists are using the Q-system to determine rock support, it is recommended to arrange an early joint session where the team collectively maps the rock mass to calibrate each other's assessments. This will ensure the most consistent and agreed-upon evaluation for each Q-parameter based on the current rock mass conditions.

1.4 Limitations

When using the Q-system, it is important to be aware that the system's support recommendations are guidelines, and engineering geological assessments must always be made to determine whether the recommendations are valid for the evaluated rock mass. If one chooses to deviate from the support recommendations, this should be documented and described.

The Q-system is empirical regarding the permanent support of various rock mass types. Its parameters account for a wide range of rock conditions. However, it is important to note that most reference cases forming the basis of the support chart come from different combinations of hard and fractured rock. The Q-system may have limitations in recommending the appropriate support requirements for the following conditions:

- Rock mass conditions with weak rocks/soft rock with few or no fractures
- Extremely fractured rock mass conditions (extremely poor rock mass)
- Fractured rock mass with low confinement
- Very unfavourable geometrical conditions in fractured rock mass
- Rock mass with anisotropic properties (jointing, rock stress situation)
- Time-dependent deformations and the occurrence of swelling rock

When assessing support needs in such rock mass conditions, where the Q-system has few reference cases or limitations, other methods should be considered in addition to the Q-system. It is important to combine the use of the Q-system with deformation measurements and numerical simulations in squeezing ground or very poor rock mass ($Q<1$). More details on the use of the Q-system in very challenging rock mass conditions can be found in Chapter 4.7.

It should be noted that the Q-system's support recommendations are conservative, as they are primarily based on observations where failure has not occurred. Advances in sprayed concrete technology since much of the reference data was collected have also led to today's sprayed concrete having higher compressive strength and better energy absorption than before. Considerations for work safety and lifespan have also led to increased use of sprayed concrete in good-quality rock masses. The support diagram in the Q-system does not consider lifespan considerations regarding thickness of sprayed concrete. Therefore, in several cases, more support is applied than indicated by the Q-system's recommendations based on the mapped rock mass quality.

2 The Q-system and classification of rock masses

The Q-system is a classification system designed to classify the quality of the evaluated rock mass and recommend support for tunnels and underground openings. High Q-values indicate good stability, while low Q-values indicate poorer stability. The Q-value is calculated using six parameters according to the following equation:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

The six parameters are:

RQD = Degree of jointing (Rock Quality Designation)
J_n = Joint set number
J_r = Joint roughness number
J_a = Joint alteration number
J_w = Joint water reduction factor
SRF = Stress Reduction Factor

Each Q-parameter is determined through geological mapping using tables that provide numerical values based on a described situation. Detailed guidance for determining each Q-parameter is provided in Chapter 3.

The stability of the rock mass is influenced by several parameters, but primarily by the following three factors: *degree of fracturing (block size), friction conditions along fractures, and stress conditions*. Paired, the six Q-parameters express the three main factors which describe the stability in tunnels and underground openings:

$$\frac{RQD}{J_n} = \text{Degree of jointing (or block size)}$$

$$\frac{J_r}{J_a} = \text{Joint friction (inter-block shear strength)}$$

$$\frac{J_w}{SRF} = \text{Active stress}$$

2.1 Degree of jointing (RQD/J_n)

The degree of fracturing, or block size, is determined by the joint pattern, i.e., joint orientation and joint spacing. At a specific location within the rock mass, there will, in most cases, be a joint pattern that could be well or not so well defined. Typically, 2 to 4 fracture orientations exist systematically within the rock mass, and most of the fractures will be parallel to one of these orientations. Nearly parallel joints form joint sets, and the spacing within each set will usually have a characteristic distribution. Joint spacing can be significantly reduced along certain zones. Such zones are called fracture zones. Stability generally decreases as joint spacing decreases, and the number of joint sets increases. In weak rocks where deformation can occur independently of joints, the degree of jointing has less importance than in hard rocks.

The ratio RQD/J_n represents the relative block size in the rock masses. In addition to RQD and J_n, it is also useful to note the actual size and shape of the blocks, as well as the joint frequency.

2.2 Joint friction (J_r/J_a)

In hard rocks, deformation will occur as shear displacements along joints. The friction conditions along the joint surfaces will therefore be decisive for the stability of the rock mass. Joint friction is dependent on the waviness and the roughness of the joint surface (J_r), and the thickness and properties of any fracture filling (J_a). Very rough and undulating joint surfaces, joints without filling, or joints with only a thin, hard mineral filling will be favourable for the stability conditions. On the other hand, smooth and planar surfaces and/or a thick layer of a soft mineral will lead result in lower friction and consequently worse stability conditions. In soft/weak rock where deformation is less dependent on jonits, the joint friction factor is of less importance.

Shear strength also depends on the effective stress, which is influenced by the presence of water and water pressure. However, the value for fracture filling, J_a, is not affected by the presence of water.

2.3 Active stress (J_w/SRF)

Stresses in a rock mass usually depend on depth below the surface, tectonic conditions, and anisotropic conditions due to topography. The stability of an underground opening will generally depend on the occurring stresses in relation to the strength of the rock mass. Moderate stresses are generally favourable for the stability of underground openings, while lack of confinement can lead to unstable conditions. In rock masses intersected by zones of weak mineral fillings such as clay, or crushed rock, stresses can vary significantly within relatively small areas. Experience from tunnel projects in Norway has shown that if the maximum principal stress approaches 1/5 of the rock's compressive strength, spalling may occur (Grimstad & Barton, 1993). If the tangential stress exceeds the rock's compressive strength, squeezing may occur. The anisotropy of the rock mass is often crucial when designing rock support.

Water conditions in the rock mass can also influence the occurring stress situation. Water pressure can reduce the normal stress on joint surfaces, making it easier for blocks to slide out. Water can also affect the friction conditions along rock joints by softening and washing away the mineral infill, thereby reducing the friction on the joint surfaces. When tunnelling through rocks with high content of minerals that easily dissolve or are subject to chemical weathering upon contact with water, this must be considered in the stability assessment.

2.4 Q' (Q_{base})

To describe rock mass quality by engineering geological mapping at the surface or by core logging, without the influence of water pressure or stress conditions, Q' or Q_{base} can be used. J_w and SRF are excluded from the calculation, resulting in the following formula:

$$Q' = \frac{RQD}{J_n} \text{ x } \frac{J_r}{J_a}$$

It is emphasized that Q' cannot be used for designing rock support for an underground excavation. For this purpose, the complete Q-value must be used, where J_w and SRF are determined.

3 Calculation of the Q-value

3.1 General

The Q-value is determined through engineering geological mapping in underground openings during excavation, from surface mapping, or through core logging. Each of the six parameters is determined based on defined descriptions provided in the corresponding tables.

When determining values for each Q-parameter, it is recommended to use the table values provided in Table 3-1 to Table 3-6. Due to local variations in the rock mass, it can often be challenging to assign a single specific value for some of the Q-parameters for the mapped rock mass. In such cases, it is recommended to assign a range to the Q-parameters, e.g., RQD = 50-70. Using a range for the Q-parameters will result in a Q_{min} and Q_{max}. If Q_{min} and Q_{max} lead to different support recommendations (different support classes), an engineering geological assessment must be made to determine which Q-value best represents the support needs of the specific rock mass. It is also possible to differentiate the amount of support for different parts of the tunnel profile if it seems appropriate.

For documentation and verification of the selected parameter values, it is recommended to use the values established in the tables for each Q-parameter. Note that some Q-parameters have identical values, so it is advisable to refer to the letters associated with the selected values. Variation in rock mass quality within a mapped area can, however, be illustrated by using the maximum and minimum values for each Q-parameter. During mapping, it may also be appropriate to divide the mapping area into several sub-areas so that the Q-value within each sub-area is relatively uniform (same rock class in the support chart). This is particularly relevant for mapping tunnel rounds with large cross-sections. The sub-area with the lowest Q-value will often determine which support class should be used. In cases where one or more weakness zones influence the rock mass being evaluated, the characteristics, extent, and geometry of the weakness zone must be considered when determining Q-value/rock support. For more information on mapping sections and weakness zones, see Chapter 5.2.

3.2 Rock Quality Designation (RQD)

RQD, Rock Quality Designation, was defined by Deere in 1963 (Deere, 1963) and was intended to be used as a simple classification system for the stability of rock masses. Using the RQD-value, five rock classes are defined (A-E) as shown in Table 3-1. RQD was originally defined from drill cores as follows:

> *"RQD is the sum of the length (between natural joints) of all core pieces more than 10 cm long (or core diameter x 2) as a percentage of the total core length"*

RQD will therefore be a percentage between 0 and 100. If 0 is used in the Q-formula, it will give a Q-value of 0 and therefore all RQD-values between 0 and 10 are increased to 10 when calculating the Q-value

Table 3-1 RQD-values and volumetric jointing.

1	RQD (Rock Quality Designation)		RQD
A	Very poor	(> 27 joints per m^3)	10i) -25
B	Poor	(20-27 joints per m^3)	25-50
C	Fair	(13-19 joints per m^3)	50-75
D	Good	(8-12 joints per m^3)	75-90
E	Excellent	(0-7 joints per m^3)	90-100
Note: i) Where RQD is reported or measured as ≤ 10 (including 0) the value 10 is used to evaluate the Q-value ii) RQD-intervals of 5, i.e. 100, 95, 90, etc., are sufficiently accurate			

The number of joints per cubic meter associated with the corresponding RQD class is also shown in Table 3-1. In a tunnel or underground opening, it is usually possible to obtain a three-dimensional picture of the rock mass, allowing for an estimate of the number of joints per cubic meter. The following formula can be used to estimate the RQD value (Palmström, 2005):

$$RQD = 110 - 2{,}5J_v \text{ (for } J_v \text{ between 4 and 44)}$$

where J_v is the number of joints per m^3

RQD is determined on-site in a tunnel by examining surfaces of different orientations, such as the crown, walls, and face. A weighted average reflecting the variation in RQD is then used in the calculation of the Q-value.

There is often greater uncertainty in determining RQD values from surface outcrops. If an exposure consists of only one planar face, it may be difficult to determine the joint spacing of joints parallel or sub-parallel to this surface.

3.2.1 RQD in blasted underground excavations

According to the original definition of RQD, only natural joints should be considered. In an underground opening, however, all joints, regardless of their origin, have some impact on stability. Blast-induced cracks typically occur within a zone extending up to 2 meters from the underground opening and have therefore less significance for the overall stability compared to pervasive, natural joints. However, for the stability of individual blocks, blast-induced cracks should be considered.

RQD can also be estimated by examining the block sizes in the muck pile from a blasting round.

3.2.2 RQD in foliated rocks

In highly foliated or schistose rocks, there may be uncertainty to which joints that should be considered. A schistosity plane can represent a weakness in the rock without necessarily being a joint. On the surface, schistose rocks often split into flakes due to surface weathering, while a few meters below the surface, the rock may appear massive. In rocks with layering or foliation, the size of the blasted blocks in the muck pile can provide a good indication of the RQD value. Schistose rocks often generate significantly larger blocks than the schistosity would suggest. As a result, schistose and layered rocks (e.g., phyllite, slate, mica schist) can have high RQD values, and RQD values obtained from tunnel and underground mapping are often higher than those obtained from surface mapping.

Core samples from, for example, clay-rich rock masses can also behave in a similar way. Immediately after drilling, only a few fractures may be visible, resulting in an RQD value of 100. However, after the cores have dried for a few weeks, they may consist of thin slices, and the RQD value could drop to zero. In such cases, it is difficult to determine which RQD value should be used to calculate the Q-value, and this uncertainty must therefore be considered when deciding the rock support design.

3.2.3 RQD in soft rocks and weakness zones

Certain weak rocks may have very few or no joints, which would, by definition, give them a high RQD value. However, in such rock mass, deformation may occur independently of the joints, which can be reflected by a high SRF value. If the rocks are so weak and unconsolidated that they can be considered as soil, the RQD value will be 0 (RQD = 10 in the Q-system), even if no joints are present.

For assessing RQD in weakness zones, see Chapter 5.2.3 "Mapping of weakness zones."

3.2.4 RQD in relation to healed joints and mineral fillings

Healed joints and joints with mineral fillings can also be difficult to assess when determining the RQD value. The strength of the joint infill itself is crucial in deciding whether it should be considered a joint or not. For example, chlorite, mica, and clay typically provide weak bonding between the fracture surfaces, which can reduce the RQD value. In contrast, infills of epidote, feldspar, quartz, and calcite can bind joints or weaknesses in the rock mass together, thereby increasing the RQD value. A simple test to assess RQD in such cases is to hit the rock with a hammer and observe where the fractures occur.

3.3 Joint set number (J_n)

The size and shape of blocks in a rock mass depend on the joint geometry. A joint set is defined as a group of nearly parallel joints that occur systematically with a characteristic joint spacing. Joints that do not appear systematically or have a spacing of several meters are referred to as random joints. To obtain an overview of the joint pattern, one can measure the orientation of a certain number of joints and plot the observations on a stereonet, as shown in Figure 3-1. The different joint sets will then appear as clusters on the stereonet.

When deciding the J_n value, only the joints present at the same location and forming defined blocks should be included. In situations where the J_n value is determined from joint observations over longer sections (e.g., several rounds) in a tunnel, summing up all joint sets may result in a too high J_n value. If the joint spacing within a joint set is greater than the span or height of the tunnel or underground opening, the joints are considered random. Regardless, one should always assess the extent to which the joints affect stability when determining J_n.

Table 3-2 provides the parameter values for J_n according to the number of joint sets and random joints. Note that at tunnel intersections and portal excavations, J_n should be multiplied by 3 and 2, respectively. The extent to which an increased J_n due to tunnel intersections or portal excavations should be applied should be assessed based on the excavation's dimensions and rock mass quality. A general rule of thumb is 0.5–1 tunnel diameter in good rock.

Table 3-2 J_n-values

2	J_n = Joint set number	J_n
A	Massive, no or few joints	0,5-1,0
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12

2	J_n = Joint set number	J_n
H	Four or more joint sets, random heavily jointed “sugar cube”, etc	15
J	Crushed rock, earth like	20
Note: i) For tunnel intersections, use 3 x J_n ii) For tunnel portals, use 2 x J_n		

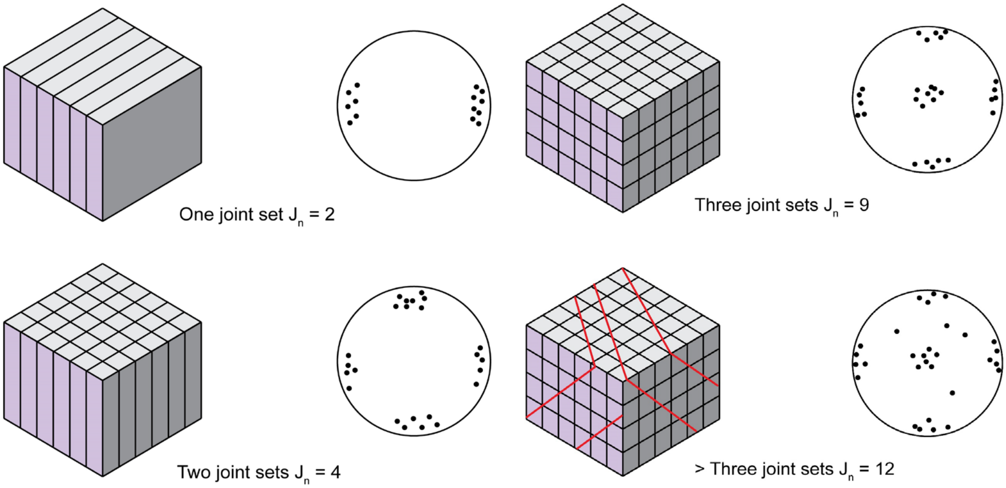


Figure 3-1 Representation of different joint patterns with corresponding stereonets.

3.3.1 J_n in relation to joint length

The length of joints is not directly included in the Q-system but is importance for the rock stability. Long joints typically have a greater impact on stability, especially if they extend through the entire cross-section of a tunnel or underground opening. Very short joints, often referred to as "cracks" can affect local stability by causing small blocks to fall out. In cases where short joints do not contribute to the formation of rock blocks, they can be considered random, even if they appear fairly systematically. If they take part in formation of blocks, they must be regarded as a joint set at the specific location where they occur.

In some cases, it is also necessary to consider the shape of the blocks formed and the potential for a block to fall out, rather than just focusing on the number of joint directions/joint sets in the rock mass. If joint sets do not intersect and form blocks, the J_n value can be reduced.

3.3.2 J_n in weakness zones

For assessing J_n in weak zones, see Chapter 5.2.3 "Mapping of weakness zones."

3.4 Joint roughness number (J_r)

The frictional conditions along a joint are influenced by two factors: surface waviness and surface roughness. In the Q-system, the joint roughness number, J_r, describes these conditions. Values for J_r are determined from Table 3-3 and/or Figure 3-3. Surface waviness and surface roughness can be assessed using the following evaluations:

Surface waviness: This refers to the character of the joint surface on a scale from centimeters to decimeters, whether it is planar, undulating, or stepped. This can be assessed by placing a 1-meter-long ruler on the joint surface or using a profile gauge to determine the amplitude and wavelength. See Figure 3-3 for examples of surface waviness on joint surfaces. Surface waviness must be considered in relation to the spacing between joint sets, which may release blocks (block size) and the likely direction of sliding. If the surfaces have irregularities with wavelengths much greater than the spacing between releasing joints, the undulating surface waviness will have low to no stabilizing effect during shear deformation, and the joints will behave as nearly planar surfaces regarding potential fall out of blocks in the rock mass.

Surface roughness: This refers to the texture/coarseness of a surface on a scale from millimeters to centimeters, whether it is rough, smooth, or slickensided. This can be assessed by running a finger along the joint surface—where the friction and the force required to move the finger along the joint surface can be felt.

All joint sets at a given location must be evaluated with respect to J_r. When calculating the Q-value, the J_r value for the joint set that is the most unfavourable concerning stability must be used, meaning the J_r for the joint set where shear deformation/sliding is most likely.

3.4.1 J_r in relation to joint infill

When determining the joint roughness number (J_r), the joint infill must also be considered. If the joints contain a thick filling of a weak mineral/clay or crushed rock material that prevents the rock surfaces from making contact during shear deformation (category "c" in Table 3-3), surface roughness is no longer relevant. In such cases, the properties of the mineral infill will be decisive for the friction, and $J_r = 1$ is used. If the infill is so thin that rock wall contact will occur before 10 cm of shear deformation (category "b" in Table 3-3), the same roughness number as for joints without infill is applied (category "a" in Table 3-3).

The thickness of the joint filling required to prevent rock contact during shear deformation depends on both surface waviness and surface roughness. For wavy, rough joints, a thicker filling is necessary compared to planar, smooth joints to avoid rock contact during shear deformation. See Figure 3-3 for illustration.

Table 3-3 J_r – values. Description refers to surface roughness and surface waviness.

3	J_r = Joint Roughness Number	J_r
a) Rock wall contact ***b) Rock-wall contact before 10 cm of shear movement***		
A	Discontinuous joints / rough, stepped	4
B	Rough or irregular, undulating / smooth, stepped	3
C	Smooth, undulating / slickensided, stepped	2
D	Slickensided, undulating	1,5
E	Rough, irregular, planar	1,5
F	Smooth, planar	1
G	Slickensided, planar	0,5
Note: i) Add 1 if the mean spacing of the relevant joint set is greater than 3 m (depends on the cross-section of the underground opening) ii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented in the estimated sliding direction		
c) No rock-wall contact when sheared (thick joint infill)		
H	Zone containing clay minerals thick enough to prevent rock-wall contact when sheared	1

3.4.2 J_r in relation to joint orientation

The waviness and roughness structure of a joint surface often has a specific orientation, meaning that a joint surface can appear planar in one direction and undulating in another. In such cases, the joint roughness number (J_r) must be determined based on the direction in which sliding is most likely to occur. This is particularly relevant for joints with pronounced lineations (slickensides), which can be smooth along the length and rough across it, or vice versa.

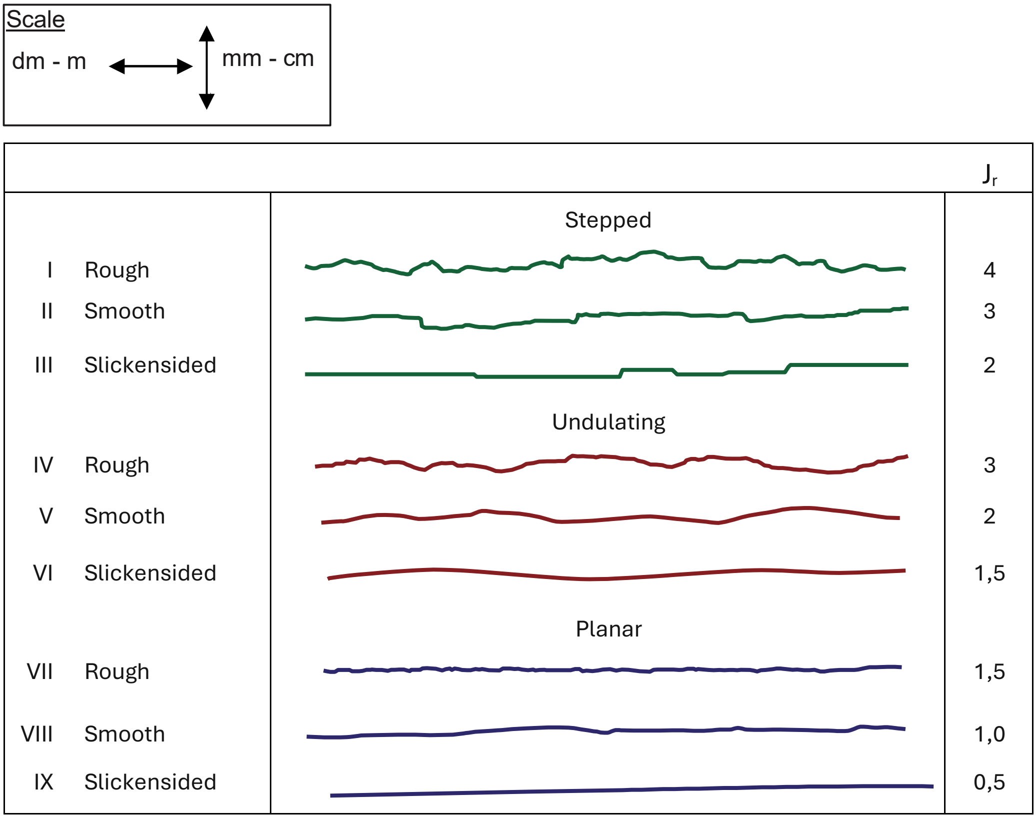


Figure 3-2 Examples of joint wall surfaces with different J_r - values. The length of each profile is in the range: 1 m (Modified from ISRM 1978).

3.4.3 J_r in rock masses without joints

When deformation in the rock mass is influenced by joints, J_r should be assigned values according to Table 3-3. For soft/weak rock without joints, J_r should be set to 1 if the material can be classified as soil ($\sigma_c \leq 0.25$ MPa according to ISRM, 1978). For very weak rocks, stronger than soil and without fractures, J_r may be irrelevant, and material deformation may depend on the strength-to-stress ratio ($J_r = 1$, category H in Table 3-3). The SRF factor is the most relevant Q-parameter to describe this situation (SRF category M-N), see Table 3-6.

In cases where weakness zones or joint fillings are so thick that contact between rock surfaces is prevented during shear deformation, J_r is given 1.

If only a few joints in the relevant joint set are exposed in the excavation at a certain point, $J_r + 1$ is used. Note that for such cases, the J_r value may differ from those listed in Table 3-3 (e.g., $J_r = 1.5 + 1 = 2.5$).

3.4.4 J_r in weakness zones

For assessing J_r in weak zones, see Chapter 5.2.2 "Mapping of weakness zones."

3.5 Joint alteration number (J_a)

In addition to the joint roughness, the joint infill is crucial for joint friction. When considering joint infill, two factors are decisive: thickness and strength. The strength depends on the mineral composition. The joint infill is categorized into the following three categories:

Category "a" – rock-wall contact
Category "b" – rock-wall contact before 10 cm shear (thin mineral fillings)
Category "c" – no rock-wall contact when sheared (thick mineral fillings)

These categories are illustrated in Figure 3-3, and detailed descriptions are provided in Table 3-4.

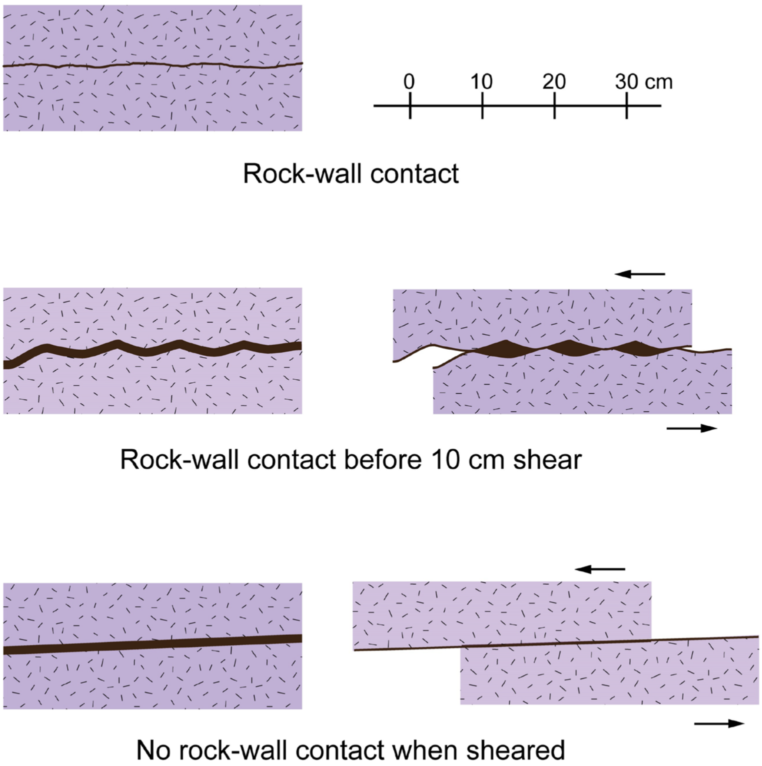


Figure 3-3 Joints with and without rock-wall contact.

Table 3-4 J_a -values, with with empirical correlation to the residual friction angle ϕ_r.

4	J_a = Joint alteration numberg	Φ_r	J_a
a) Rock-wall contact (no mineral fillings, only coatings)			
A	Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote.		0,75
B	Unaltered joint walls, surface staining only.	25-35°	1
C	Slightly altered joint walls. Non-softening mineral coatings; sandy particles, clay-free disintegrated rock, etc.	25-30°	2
D	Silty or sandy clay coatings, small clay fraction (non-softening).	20-25°	3
E	Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talc gypsum, graphite, etc., and small quantities of swelling clays	8-16°	4
b) Rock-wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, etc.	25-30°	4
G	Strongly over-consolidated, non-softening, clay mineral fillings (continuous, but < 5 mm thickness)	16-24°	6
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but < 5 mm thickness)	12-16°	8
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but < 5 mm thickness). Value of Ja depends on percent of swelling clay-size particles.	6-12°	8-12
c) No rock-wall contact when sheared (thick mineral fillings)			
K	Zones or bands of disintegrated or crushed rock. Strongly over-consolidated.	16-24°	6
L	Zones or bands of clay, disintegrated or crushed rock. Medium or low over-consolidation or softening fillings.	12-16°	8
M	Zones or bands of clay, disintegrated or crushed rock. Swelling clay. J_a depends on percent of swelling clay-size particles.	6-12°	8-12
N	Thick continuous zones or bands of clay. Strongly over-consolidated.	12-16°	10
O	Thick, continuous zones or bands of clay. Medium to low over-consolidation	12-16°	13
P	Thick, continuous zones or bands with clay. Swelling clay. J_a depends on percent of swelling clay-size particles.	6-12°	13-20

The classification of the different categories a, b, and c depends on both the roughness og the joint plane and thickness of the infill. For smooth joints, a millimetre of filling could be enough to prevent rock contact. However, for rough and undulating joints, several millimetres, and in some cases centimetres, may be required. Within each of the three categories, the J_a values are evaluated based on the characteristics of the mineral filling according to Table 3-4.

All joint sets at a given location must be evaluated. When calculating the Q-value, the J_a value for the joint set considered to be the most unfavourable for stability must be used, i.e., where shear is most likely to occur.

3.5.1 Determination of J_a based on the type of mineral type of the joint infilling

The type of mineral and its characteristics are decisive the J_a-value. Whether or not water will soften the mineral infill is also important and can be tested by placing a sample of the mineral in water to see if it dissolves. Since only small amounts of water are needed to cause swelling in some clays, a high J_a-value is usually assigned regardless of the water situation where swelling clays are present.

The J_a-value depends on the type of clay mineral in the joint infill. Swelling clays are most unfavourable for the stability. Therefore, an analysis of the clay filling may be necessary. Analyses can be carried out using relatively simple laboratory tests or more advanced X-ray diffraction techniques. When swelling clays are identified, swelling pressure tests provide valuable information. The swelling pressure measured in the laboratory should not be used directly in the design of rock support, as the rock mass's inherent load-bearing capacity will take up a significant portion of the pressure. In addition, the swelling clays are usually mixed with other minerals and rock fragments.

3.5.2 J_a in relation to friction angle

Rough, undulating, and unweathered joint surfaces with rock-wall contact (J_a-category “a”) will normally provide significant resistance to shear deformation, which is favourable for stability. When rock joints have a thin clay coating and filling (J_a-category “b”), the shear strength is significantly reduced. Renewed rock-wall contact after small shear displacement will be a very important factor to prevent block fall or collapse during excavation. If no rock-contact appears during shearing (J_a-category “c”), this will be very unfavourable for excavation stability.

The equation $\tan^{-1}(J_r/J_a)$ provides a rough estimate of the effective friction angle, φ’, that can be expected for different combinations of joint roughness and joint materials (Barton et al., 1974). Note that the effective friction angle $\varphi' = \varphi_b + i$, where φ_b is the basic friction angle (measured on a flat joint surface, often by tilt tests), and i is the dilation angle/roughness angle (depending on the surface waviness and roughness of the joint surface). The equation tan-1 (J_r/J_a) should only be used when there is rock-wall contact between joint surfaces ($J_a \leq 4$).

Table 3-4 provides an empirical relationship between J_a values and residual friction angle, φ_r. Note that the residual friction angle is always lower than or equal to the effective friction angle, and values for residual friction angle given in Table 3-4 and values for effective friction angle from $\tan^{-1}(J_r/J_a)$ cannot be directly compared.

For a more accurate determination of the friction angle, it is recommended to perform tilt tests or direct shear box tests in the laboratory according to the standard method provided by ISRM (Alejano et al., 2018, Muralha et al., 2013). More accurate calculation of the residual friction angle can be done using the equation for residual friction angle given by Barton and Choubey (1977).

3.5.3 J_a in weakness zones

For assessing J_a in weak zones, see Chapter 5.2.3 "Mapping of weakness zones."

3.6 Joint water reduction factor (J_w)

Joint water may soften or wash out the mineral infill and thereby reduce the friction on the joint planes. Water pressure may reduce the normal stress on the joint walls and cause the blocks to shear more easily.

A determination of the joint water reduction factor is based on inflow and water pressure observed in an underground opening, see Table 3-5. The lowest J_w-values ($J_w < 0.2$) represent large stability problems.

Table 3-5 J_w values

5	J_w = Joint Water Reduction Factor	J_w
A	Dry excavations or minor inflow (humid or a few drips)	1,0
B	Medium inflow, occasional outwash of joint fillings (many drips/"rain")	0,66
C	Jet inflow or high pressure in competent rock with unfilled joints	0,5
D	Large inflow or high pressure, considerable outwash of joint fillings	0,33
E	Exceptionally high inflow or water pressure decaying with time. Causes outwash of material and perhaps cave in	0,2-0,1
F	Exceptionally high inflow or water pressure continuing without noticeable decay. Causes outwash of material and perhaps cave in	0,1-0,05
Note: i) Factors C to F are rough estimates. Increase J_w if the rock is drained or if injection is performed. ii) Special problems caused by ice formation are not taken into consideration.		

3.6.1 J_w in relation to and changing water inflow

Water inflow is often observed in underground openings and caverns. However, the inflow may also originate from the invert, and may be difficult to observe or measure quantitatively. The surrounding rock mass may be drained with no visible inflow for some time after excavation. In an underground opening near the surface, inflow may vary with the seasons and amount of precipitation. Inflow may increase in periods with high precipitation and decrease in dry seasons or in seasons with freezing conditions. These conditions must be kept in mind when determining the joint water reduction factor. Sealing measures, for example grouting, will reduce inflow, and the J_w-value should then be increased according to the reduction of the inflow. In some cases, the underground opening may be dry immediately after the excavation, but inflow develops over time. Conversely, large inflow immediately after excavation may decrease after some time.

3.7 Stress Reduction Factor (SRF)

In general, SRF (Stress Reduction Factor) describes the relationship between rock stresses and the rock strength around an underground opening. The effects of stresses can usually be observed in underground excavation as spalling, slabbing, rock burst, deformation, squeezing, dilation, and block falls. Often, it may take some time before stress-related phenomena become visible.

Whereas intensive spalling and rock burst may occur immediately after excavation, slower deformations like growth of new joints or plastic deformation of weak rock masses may take several days, weeks or months after excavation to form. In such cases an SRF-value determined from mapping the underground opening immediately after excavation may be incorrect.

Before the SRF value can be determined, the category regarding the stress situation, as described in Table 3-6 must be determined. The stress situation is classified into five categories. Detailed information for each SRF category is provided in the subsequent subchapters.

a) Weakness zones that intersect the underground opening which may or may not be able to transfer stresses in the surrounding rock mass.
b) Competent rock with stability problems due to high stresses or lack of stresses.
c) Massive rock with stability problems due to high stress levels.
d) Squeezing rock with plastic deformation of incompetent rock under the influence of moderate or high rock stresses.
e) Swelling rock; chemical swelling activity depending on the presence of water.

SRF can be estimated based on the ratio between the uniaxial compressive strength of the rock (σ_c) and the maximum principal stress (σ_1), or the ratio between the maximum tangential stress (σ_θ) and σ_c in massive rock. During planning phase of an underground excavation, SRF can be estimated from the overburden and topographical characteristics or through general experience from the same geological and geographical region. More details on the relationship between stress-strength ratio and SRF in massive rock are provided in Chapter 3.7.3.

Table 3-6 SRF values

6	SRF = Stress Reduction Factor	SRF
a) Weak zones intersecting the underground opening, which may cause loosening of rock mass		
A	Multiple occurrences of weak zones within a short section containing clay or chemically disintegrated, very loose surrounding rock (any depth), or long sections with incompetent (weak) rock (any depth). For squeezing, see 6M and 6N	10
B	Multiple shear zones within a short section in competent clay-free rock with loose surrounding rock (any depth)	7,5
C	Single weak zones with or without clay or chemical disintegrated rock (depth ≤ 50m)	5
D	Loose, open joints, heavily jointed or “sugar cube”, etc. (any depth)	5
E	Single weak zones with or without clay or chemical disintegrated rock (depth > 50m)	2,5
Note: i) Reduce these values of SRF by 25-50% if the weak zones only influence but do not intersect the underground opening		
b) Competent rock with low or favourable stress conditions, mainly massive rock		*SRF*
F	Low stresses, near surface, open joints	2.5
G	Medium stresses, favourable stress condition	1
Note: ii) When the depth of the crown below the surface is less than the span; suggest SRF increase from 2.5 to 5 for such cases (see F)		
c) Competent, mainly massive rock, stress-related problems		*SRF*
H	High stress, very tight structure. Usually favourable to stability. May also be unfavourable to stability dependent on the orientation of stresses compared to jointing/weakness planes	0,5-2 2-5*
J	Moderate spalling and/or slabbing after > 1 hour in massive rock	5-50
K	Spalling or rock burst after a few minutes in massive rock	50-200
L	Heavy rock burst and immediate dynamic deformation in massive rock	200-400
Note: iii) See Chapter 3.7.3 and Grimstad & Barton (1993) for details on SRF and the stress-strength ratio.		
d) Squeezing rock: plastic deformation in incompetent rock under the influence of high pressure		*SRF*
M	Moderate squeezing rock pressure	5-10
N	Heavy squeezing rock pressure	10-20
Note: iv) Determination of squeezing rock conditions must be made according to relevant literature (i.e. Singh et al., 1992 and Bhasin and Grimstad, 1996)		
e) Swelling rock: chemical swelling activity depending on the presence of water		*SRF*
O	Moderate swelling rock pressure	5-10
P	Heavy swelling rock pressure	10-15

3.7.1 SRF and weakness zones intersecting the underground opening

Surrounding a weakness zone an anomalous stress situation may occur locally. In the Q-system, an increased SRF-value is used to lower the Q-value, ensuring that stability is maintained when determining rock support. If the weakness zone is of such a nature that stresses cannot be transferred, a stress concentration may occur on one side of the zone, while relaxation may occur on the other side. In an ordinary low stress situation, a weakness zone will usually cause stress anomalies only in the zone itself and in a limited area around it.

If there are multiple weakness zones spaced a few meters apart, a longer section of an excavation may be affected, leading to an increased SRF-value. In cases where a long section of the excavation intersects multiple weakness zones with crushed or weathered rock, it may be appropriate to classify the entire section as a "weakness zone." In such cases, categories A, B, or D in Table 3-6 a) should be used. If squeezing rock is present, use M or N in Table 3-6 c). For swelling rock conditions, use O or P in Table 3-6 d).

To detect whether the rock is destressed or not, one can strike the rock with a hammer or scaling bar. If a hollow sound is heard and small blocks easily loosen, the rock can be considered destressed / poorly confined, and an SRF-value greater than 1 may be determined. Note that a hollow sound can also occur if the strike is on a local individual block that is loose.

A visualization of weakness zones is given in Figure 3-4. In Figure 3-4 a), the tunnel is intersected by a single clay filled zone. In the vicinity of this zone there is normally an anomalous stress situation. An SRF-value of 5 has to be used for an area consisting of a weakness zone and its immediate surroundings. In Figure 3-4 b), several clay filled zones intersect the underground opening, requiring an SRF-value of 10 for this section. The extent of the area that should have an elevated SRF value when crossing weakness zone(s) depends on the quality of the rock mass outside the weakness zone, the geometry of the underground opening, and the thickness and orientation of the zone. Typically, 0.5-1.0 times the span is appropriate.

For general considerations related to the mapping of weakness zones, see Chapter 5.2.3.

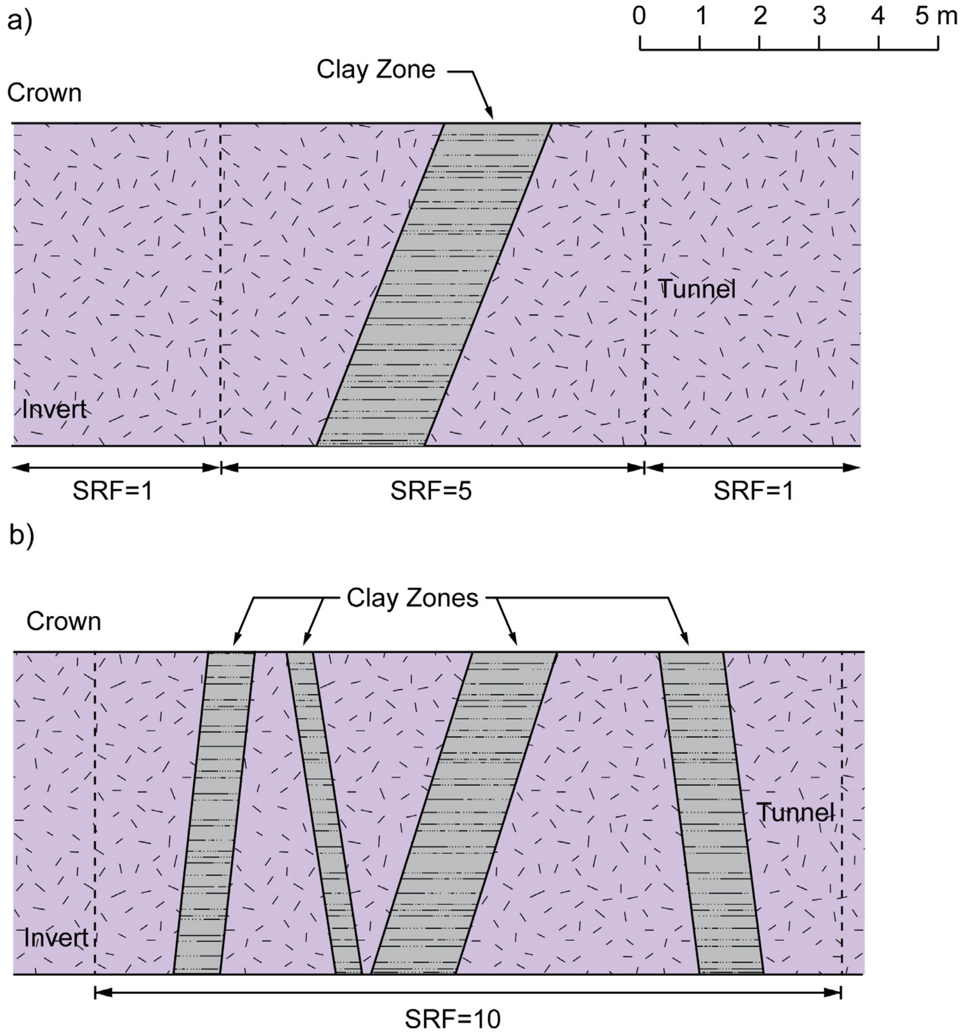


Figure 3-4 SRF-values related to single and multiple weakness zones.

3.7.2 SRF in competent rock with low or favourable stress conditions

Moderate rock stresses are generally most favourable for stability, with an SRF-value of 1. Relatively high horizontal stresses can be beneficial for the stability of the roof in an underground opening, and in some cases, an SRF-value of 0.5 can be applied.

Low stresses, which will often be the case when there is small rock overburden, can lead to reduced stability due to inadequate confinement. In such cases, the SRF value would be 2.5, or even 5.0 if the span of the underground opening is larger than the rock overburden. Poor stability due to low confinement can also occur if it is excavated near an existing underground opening.

3.7.3 SRF in competent rock, rock stress-related problems

In competent and relatively massive rock, the SRF value can be estimated when the ratio σ_c/σ_1 or σ_θ/σ_c is known. The relationship between stress-strength ratios is shown in Table 3-7. It is important not to apply this relationship uncritically. The data is primarily derived from massive rock masses with little fracturing ($RQD/J_n > 20$) under high-stress conditions (Grimstad & Barton, 1993).

Rock burst or spalling can occur under very high stress conditions. The intensity of stress-related problems and how quickly stability issues arise after blasting will be crucial in determining the SRF-value. Case J in Table 3-7 describes moderate stress problems that occur more than an hour after excavation. If problems begin around one hour after excavation, an SRF value of 20–50 should be used, depending on the intensity of the spalling. If it takes many hours or a few days before rock slabs loosen, the SRF value may be 5-10. Similar time relations apply to case K.

If problems with intense spalling occur immediately after excavation, the SRF value will be around 200. If some minutes pass before spalling occurs or spalling is less intensive, SRF will be 50–150. In the extreme cases in section L (SRF = 200–400) problems with intense rock burst start immediately after excavation, and long-term deformations may be expected despite appropriate rock support at the face. In cases with SRF > 50 it may be necessary to support the working face before starting a new round of excavation

Table 3-7 SRF-values and stress-strength ratio for category c).

7	**SRF = Spenningsfaktor (Stress Reduction Factor)**			**SRF**
c) Competent, mainly massive rock, stress-related problems		σ_c/σ_1	σ_θ/σ_c	SRF
H	High stress, very tight structure. Usually favourable to stability. May also be unfavourable to stability dependent on the orientation of stresses compared to jointing/weakness planes*	10-5	0,3-0,4	0,5-2 2-5*
J	Moderate spalling and/or slabbing after > 1 hour in massive rock	5-3	0,5-0,65	5-50
K	Spalling or rock burst after a few minutes in massive rock	3-2	0,65-1	50-200
L	Heavy rock burst and immediate dynamic deformation in massive rock	<2	>1	200-400
Note: *For strongly anisotropic virgin stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to $0.75\ \sigma_c$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c to $0.5\ \sigma_c$, where σ_c = unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_θ = maximum tangential stress (estimated from elastic theory)				

High stresses that lead to immediate spalling and rock bursting usually also result in long-term deformation of the rock mass through the development of new fractures within the rock mass until a new stability is achieved. The extent of spalling depends on the intensity and the span of the excavation. An anisotropic stress condition is particularly unfavourable when the stress levels are high and only parts of the excavation perimeter are subjected to stress-induced stability problems. This often results in an asymmetric cross-section, and the stability issues increase with rising stress levels.

In many cases the rock stress is induced by high valley sides giving high major principal stresses, high tangential stress and anisotropic stresses, as illustrated in Figure 3-5. The height of the mountainside above the excavation level compared to the rock's compressive strength can serve as a good correlation for estimating SRF.

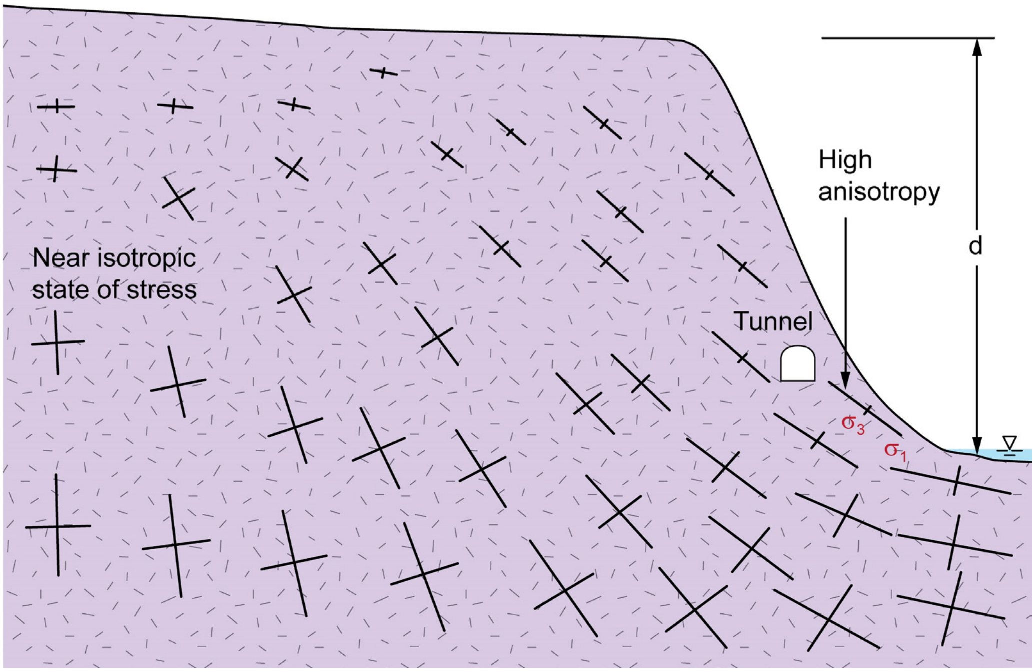


Figure 3-5 Visualization of a high valley side with high anisotropic stresses.

3.7.4 SRF in squeezing rock

In cases where high stresses are combined with fractured or soft/ductile rock mass, the compressive strength of the rock mass is more critical than the compressive strength of intact rock. In such cases, it is more likely that a squeezing effect will occur rather than spalling, as described in category d) of Table 3-6. "Squeezing rock" refers to rock masses where plastic deformation occurs under the influence of high stresses in soft or crushed rock when the stresses exceed the strength of the rock mass. Under such conditions, the time dependency of the deformations and the load on the rock support, along with the appropriate timing for the installation of the support, are very important. Temporary and permanent functions of the rock support must be analysed.

In soft rocks with few joints, stability depends on the relation between the compressive strength of the rock and the in-situ stresses. In such situations, it is recommended to supplement the Q-system with other methods such as deformation measurements and/or numerical simulation to determine the required rock support.

3.7.5 SRF in swelling rock

Swelling is a chemical process is initiated when water is added to rocks containing minerals with swelling properties. The quantity and quality of the swelling minerals will be decisive for this process and for the magnitude of the swelling pressure. It may be necessary to carry out laboratory tests to determine the potential swelling pressure as a basis for the SRF-value. Among the most common swelling minerals is anhydrite, which swells during transformation to the more commonly occurring gypsum. Another common swelling mineral is montmorillonite (the most active mineral in swelling clays), which also swells by the absorption of water. Please note that some rock masses like alum shale and certain black shales also have a swelling potential.

In many underground excavations swelling may occur a long time after excavation due to absorption of humidity from the air. In cases involving swelling, it is also important to investigate the cause of the swelling. For example, in alum shale, swelling depends on the combination of oxygen and water. In such cases, it may be more appropriate to seal the rock surface to prevent swelling, rather than designing rock support to counteract the swelling.

Note that the SRF categories O and P should only be used in cases of swelling rock, not in cases of swelling joint infill. For instances involving swelling clay minerals in joint infill, the J_a value should be used, as indicated in Table 3-4.

4 Using the Q-system to evaluate the support requirements

The Q-value and the six associated parameter values provide a classification of the rock mass. A specific Q-value indicates a certain stability condition that will require a certain extent of rock support for a given dimension of an underground opening. Based on registrations from underground excavations a relation between the Q-value and the permanent rock support has been found (see Barton et al. 1974 and Grimstad & Barton, 1993). This can be be used as guide for the design of rock support in new underground facilities.

4.1 Span width and Excavation Support Ratio (ESR)

In addition to the rock mass quality (the Q-value), two other factors are decisive for the support design in tunnels and underground openings: the safety requirements of the underground opening and its dimensions, i.e., the span or the height of the underground opening.

The need for rock support generally increases with increasing span width and increasing wall height. For the design of support using the Q-system, the span width should be used to determine the support in the crown and spring lines, while wall height should be used to determine wall support (see Chapter 4.2.2). In cases where the wall height exceeds the span width, the wall height should be used for designing the support in the crown and spring lines. When excavating a tunnel or cavern excavated in multiple drifts (sequential excavation), the final wall height or span width should be the determining factor for the design of the rock support. For underground openings with large dimensions (large span width or high walls), numerical modelling is an important supplement for the final design of the rock support.

The safety requirements will depend on the purpose of the underground opening. A road tunnel or an underground powerhouse requires a higher safety level than a water tunnel or a temporary excavation in a mine. In the Q-system, the factor ESR (Excavation Support Ratio) is used to express the safety requirements of the underground opening, see Table 4-1. A low ESR-value indicates need for a high safety level, while higher ESR values suggest that a lower safety level is acceptable. Requirements and construction traditions in each country may result in different ESR-values than those given in Table 8.

It is recommended to use ESR = 1.0 when $Q \leq 0.1$ for the types of excavation B, C, and D in Table 4-1. At such low Q-values stability problems may be severe, and the support should therefore be designed independently of the safety requirements of the underground opening.

The span width (or wall height) combined with ESR gives the "equivalent dimension," D_e, as follows:

$$\frac{Span\ width\ or\ heigth\ (m)}{ESR} = Equivalent\ dimension\ (D_e)$$

Table 4-1 ESR-values. I most case, category E (ESR = 1, bolded) is used

8	Type underground facility	ESR
A	Temporary mine openings, etc.	3-5
B	Vertical shafts*: i) circular sections ii) rectangular/square sections * Dependant of purpose. May be lower than given values.	2,5 2,0
C	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks), water supply tunnels, pilot tunnels, drifts and headings for large openings.	1,6
D	Minor road and railway tunnels, surge chambers, access tunnels, sewage tunnels, etc.	1,3
E	**Power houses, storage rooms, water treatment plants, major road and railway tunnels, civil defence chambers, portals, intersections, etc.**	**1,0**
F	Underground nuclear power stations, railways stations, sports and public facilitates, factories, etc.	0,8
G	Very important caverns and underground openings with a long lifetime, ≈ 100 years, or without access for maintenance.	0,5

4.1.1 Life span considerations in the Q-system

The Q-system does not primarily account for life span considerations. Various time-dependent effects that may degrade the performance of rock support should always be analysed separately. Some features of rock mass behaviour are time-dependent, especially those features that can lead to increased load or stress, such as swelling rock, swelling minerals in joint infill, squeezing or high rock stresses that result in spalling or rock burst. Such conditions are to some extent covered in the Q-system for the design of permanent rock support through the parameter study of J_a and SRF.

Life span considerations resulting from geochemical exposure must be carried out separately and independently of the Q-system's recommendations.

Tunnels exposed to different types of physical and chemical conditions may impose special requirements for material selection and rock support design concerning their lifespan. The Q-system does not account for the effects of such considerations on the recommended rock support. Thus, these factors must be assessed separately during the design of rock support.

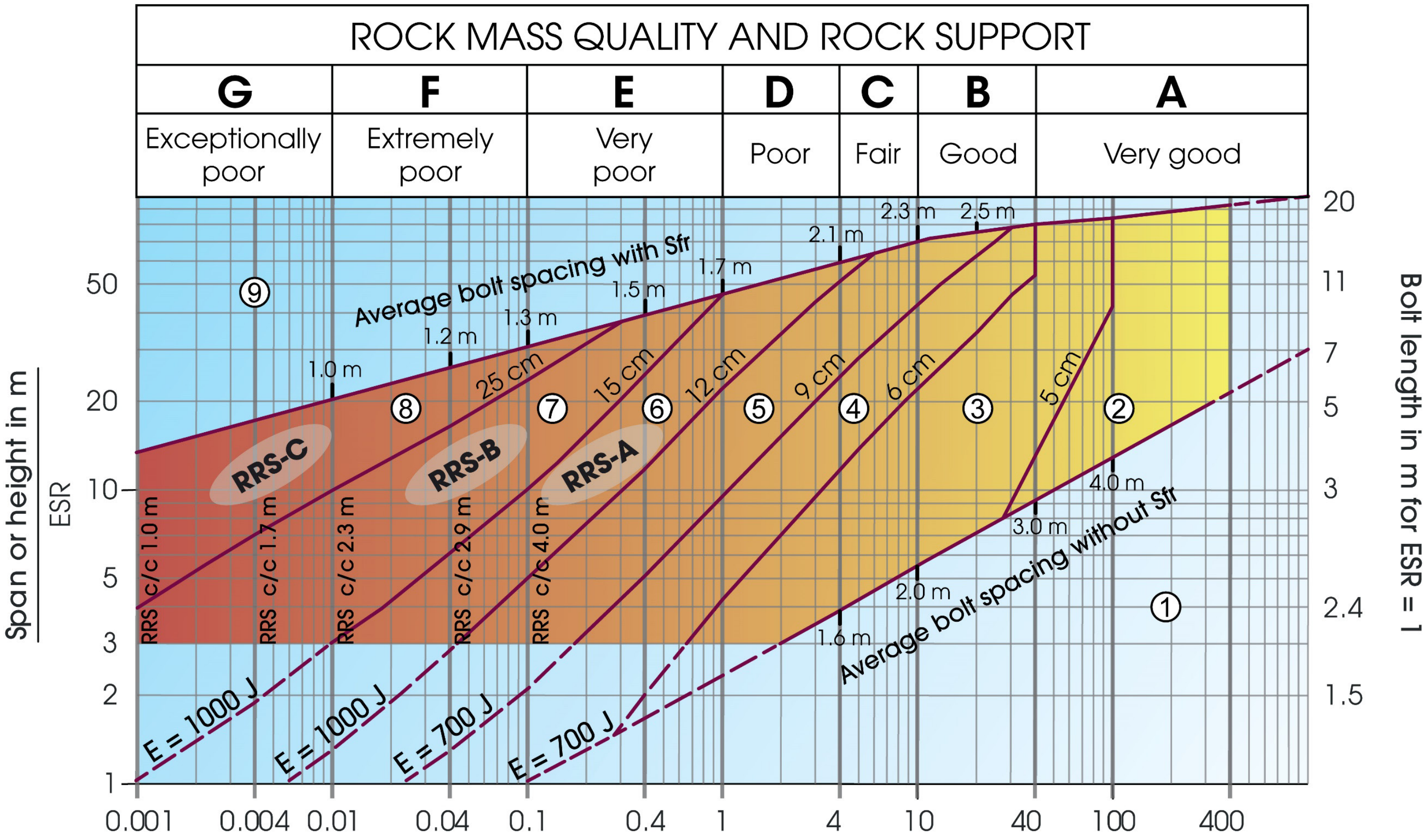
4.2 Rock support chart

The Q-value and the equivalent dimension (D_e) will be decisive for the permanent support design. In the support chart shown in Figure 4-1, the Q-values are plotted along the horizontal axis and the equivalent dimension, D_e, along the vertical axis on the left-hand side.

During the development of the support chart, all the studied combinations of Q-values and D_e in supported tunnels were plotted in a similar diagram as shown in Figure 4-1. This plotting provided the basis for the division of the support chart in terms of different support types for different rock mass conditions. The support chart is based on an average consideration of data from the analysed underground projects. Most of the studied projects implemented a conservatively high level of rock support, but in some cases, collapses occurred, either during construction or after the underground openings were put into use. The examined cases of collapses generally relate to situations where weakness zones were overlooked or not thoroughly mapped, leading to an insufficient basis for assessing the necessary rock support.

Please note that the chart is not divided into definite support classes but shown as a continuous scale both for bolt spacing and thickness of sprayed concrete. The support diagram provides recommendations on bolt spacing, bolt lengths, and the thickness of sprayed concrete. The chart also indicates the energy absorption class for fibre-reinforced sprayed concrete (Sfr) and the design of reinforced ribs of sprayed concrete (RRS).

The recommendations on rock support design suggested by the chart should be regarded as indicative. In the case of special challenges and/or challenging rock mass conditions, a separate/supplementary assessment is recommended to determine the necessary rock support. This is further described in Chapter 4.7.



Rock mass quality $Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$

Support categories

① Unsupported or spot bolting

② Spot bolting

③ Systematic bolting, fibre reinforced sprayed concrete, 5-6 cm, **B + Sfr**

④ Fibre reinforced sprayed concrete and bolting, 6-9 cm, **Sfr (E700) + B**

⑤ Fibre reinforced sprayed concrete and bolting, 9-12 cm, **Sfr (E700) + B**

⑥ Fibre reinforced sprayed concrete and bolting, 12-15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E1000) + B + RRS-A**

*⑦ Fibre reinforced sprayed concrete >15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E1000) + B + RRS-B**

*⑧ Fibre reinforced sprayed concrete >25 cm + double layer of reinforced ribs of sprayed concrete and bolting, **Sfr (E1000) + B + RRS-C**

*⑨ Special evaluation

Bolts spacing is mainly based on Ø20 mm
B = Bolting
Sfr = Fibre reinforced sprayed concrete
E = Energy absorbtion in fibre reinforced sprayed concrete
RRS = Reinforced Ribs of Sprayed concrete
ESR = Excavation Support Ratio
Areas with dashed lines have no empirical data

*For support category 7-9: The recommendations on rock support design should be regarded as indicative. Engineering geological and rock mechanical assessments should also be conducted (see Chapter 4.7).

RRS - spacing related to Q-value

RRS-A
Si30/6 Ø16 - Ø20 (span 10m)
D40/6+4 Ø16-20 (span 20m)

RRS-B
Si35/6 Ø16-20 (span 5m)
D45/6+4 Ø16-20 (span 10m)
D55/6+4 Ø20 (span 20m)

RRS-C
D40/6+4 Ø16-20 (span 5 m)
D55/6+4 Ø20 (span 10 m)
Special evaluation (span 20 m)

Si30/6 = Single layer of 6 rebars, 30 cm thickness of sprayed concrete
D = Double layer of rebars
Ø16 = Rebar diameter is 16 mm
c/c = RRS spacing, centre - centre

Figure 4-1 Permanent support recommendations based on Q-values and span/ESR

The requirement for shotcrete thickness increases with decreasing Q-value and increasing span, as shown in the support chart. In cases that fall between the lines indicating thickness of sprayed concrete, a linear approach is used to determine the necessary thickness. In situations with potentially large deformations, such as under high-stress conditions, fibre-reinforced sprayed concrete (Sfr) should be used across all support categories.

In some cases, the support chart recommends alternative support methods. For cases with high Q-values, sprayed concrete may be considered unnecessary. For such cases, the bolt spacing requirements depend on whether sprayed concrete is used or not. Due to this, the support chart is divided into two sections. The section labelled "Average bolt spacing with Sfr" refers to bolting in combination with reinforced sprayed concrete. The other section, labelled "Average bolt spacing without Sfr", indicates bolt spacing when reinforced sprayed concrete is not applied. It is important to note that the recommended bolt spacing reflects the quantity of bolts needed rather than an exact recommendation for bolt distances. The placement and orientation of each bolt should be adjusted to the joint geometry, especially in areas with wide bolt spacing. In areas where shotcrete is not used, systematic bolting is not relevant, and an engineering geological assessment is required for the placement of each individual bolt.

The length of the bolts primarily depends on the span width or wall height of the underground opening but is also influenced by the quality of the rock mass to some extent. Recommended bolt lengths are provided on the right side of the diagram (assuming ESR = 1), though a specific evaluation of the required length should always be conducted. In cases of unfavourable joint geometry, longer bolts than those recommended in the support chart may be necessary.

4.2.1 Sprayed concrete at high Q-values

According to current practice, the use of sprayed concrete is significantly higher than the reference cases on which the support chart is based. This is especially true for rock classes that previously did not require the use of sprayed concrete. Different clients have different requirements and practices regarding acceptable minimum support, even in good rock mass classes. The Q-system's recommendations in rock mass classes A and B refer to support requirements based on rock mass stability and the detailed stability of the rock surface in an underground excavation. Where the support chart in the Q-system recommends rock bolts without the use of shotcrete, surface scaling of the rock should also be considered a support method.

4.2.2 Wall support

The support chart primarily applies to the crown and the spring lines in tunnels and underground openings. The level of support required for the walls is generally lower for

Q-values greater than 1. When the Q-system is used for wall support, the wall height is used instead of the span width in the calculation of the equivalent dimension, D_e (which results in a reduction of bolt length). The actual Q-value is adjusted as shown in Table 4-2 (which results in a reduction in thickness of sprayed concrete). The value obtained after this adjustment is used directly in the support chart in Figure 4-1 to determine the necessary wall support. However, the placement and direction of each bolt should still be adapted to the joint geometry. Note that when the wall height exceeds the span width, the same Q-value is used for the support of the entire profile.

Table 4-2 Conversion from actual Q-values to adjusted Q-values for design of wall support.

9	Dimensioning of wall support	
In rock masses of good quality	Q > 10	Multiply Q-values by a factor of 5
For rock masses of poor-fair quality	1 < Q < 10	Multiply Q-values by a factor of 2.5. In cases of high rock stresses, use the actual Q-value
For rock masses of poor quality	Q < 1	Use actual Q-value
Wall height > span width	Applies for all Q-values	Use actual Q-value

4.3 Reinforced ribs of sprayed concrete (RRS)

In areas with very poor to exceptionally poor rock quality (Q < 1, support categories 6-8), reinforced ribs of sprayed concrete (RRS) are often a preferred alternative to cast concrete lining. The ribs are constructed with a combination of steel bars (usually with a diameter of 16 mm or 20 mm), sprayed concrete, and rock bolts, as shown in Figure 4-2. When using steel bars of 20 mm, the bars must be pre-bent to achieve a smooth profile. The thickness of the ribs, the spacing between them, and the number and diameter of the reinforcing bars are be adapted to the dimensions of the underground opening and the quality of the rock mass in accordance with the support chart.

The support diagram includes three RRS categories: RRS-A, RRS-B, and RRS-C. Guidelines for the use of RRS in relation to Q-values, equivalent dimensions (D_e), and spans for underground chambers are provided in the support chart in Figure 4-1 and the accompanying explanatory text.

In the description of the support diagram, the following abbreviations are used:

- “Sfr”: Fibre-reinforced sprayed concrete
- "Si": Single layer of steel bars
- "D": Double layer of steel bars
- "45": Total rib thickness of 45 cm

- "6": Six steel bars
- "c/c = 2-3": Centre-to-centre spacing of 2 to 3 meters between the ribs
- "16" or "20": Diameter of the steel bars, in mm

Note that in the support chart, the recommendations for RRS follow the support classes, meaning that the same rib dimensions are maintained diagonally across the chart. Within each area, there will be a range where the suggested spacing between the ribs will vary. An engineering geological assessment must be conducted in each case to determine the appropriate spacing between the ribs.

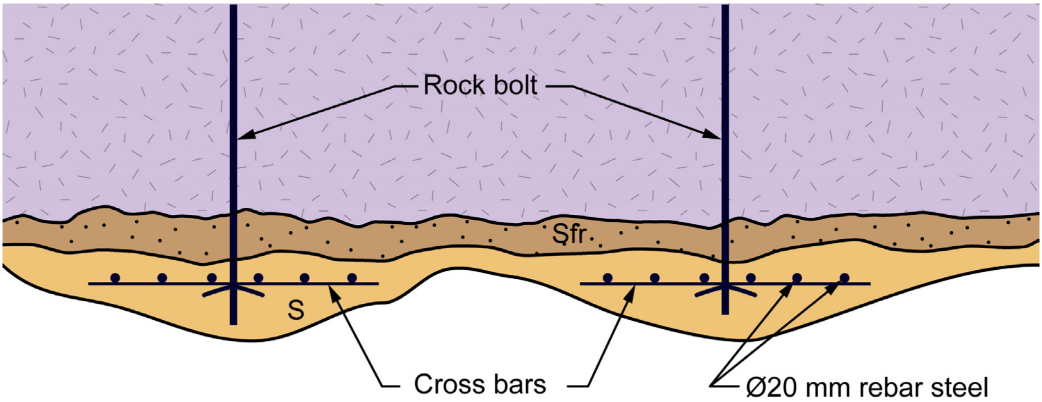


Figure 4-2 Construction principle for reinforced ribs of sprayed concrete (RRS). Note that the initial layer (smoothening layer) closest to the rock surface is fibre-reinforced sprayed concrete (Sfr), while the outermost layer of sprayed concrete is without fiber (S).

In cases where the Q-value indicates the need for RRS, a 12-15 cm thick layer of fibre-reinforced sprayed concrete is typically applied before the ribs can be installed. This layer serves as temporary support and helps to smoothen the rock surface, ensuring optimal arching effect (applicable for pre-bent reinforcement bars, Ø20 mm). The thickness of this layer is included in the total thickness of the RRS. The sprayed concrete layer applied on top of the installed reinforcement ribs should preferably be without fibre (see Figure 4-2).

As shown in the explanation of the support chart in Figure 4.1, it is recommended for support categories 6–9 to conduct further assessments for a more comprehensive rock mass characterization before determining the final rock support. This is particularly relevant for rock mass conditions that are not fully accounted for in the reference cases on which the support chart is based. See Chapter 4.7 for more details.

In support categories 7-9 additional anchoring of the RRS at the base or the need for a cast concrete invert cast concrete should be considered.

4.3.1 Comparison of RRS and lattice girders

Lattice girders are made of rolled ribbed steel bars that are prefabricated to theoretical dimensions. The geometry of a lattice girder is adjusted to fit the cross-section of the underground opening, ensuring that it aligns with the theoretical contour of the tunnel. When assembled, the lattice girder forms a continuous arch of ribbed steel in the rock support design.

The load-bearing principle of the RRS enables immediate and integrated interaction with the rock mass by ensuring that the rock support bolts are continuously anchored along the entire arch, thereby activating the rock mass as part of the structure from the moment of installation. Lattice girders can also be installed with interactive rock bolts but this requires specific adaptations during installation to ensure proper functionality.

Below are examples of when lattice girders might be considered over RRS:

- When it is known in advance that a longer section of the underground opening will be excavated through rock that requires RRS (based on low Q-values), using prefabricated lattice arches may be time saving.
- In rock caverns with large spans where the rock mass quality and the span width/height give RRS recommendation according to the support chart, lattice girders have a documented load-bearing capacity which is easier to use in analytical calculations and numerical analyses.

4.3.2 Quality control and improvement of RRS and lattice girders

The use of RRS and lattice girders for permanent rock support with long lifespan requirements imposes extra strict execution standards. Installation of rebars and subsequent application of sprayed concrete must be carried out in a way that minimizes voids ("shadows") behind the rebars.

If there is suspicion of voids in RRS or lattice girder, it is important to drill control holes with a small diameter (10-12 mm) and conduct an inspection using an appropriate video camera. Voids can be repaired by injecting a suitable long-lasting injection material.

4.4 Forepoling/spiling

In poor rock mass conditions, it may be necessary to use forepoling/spiling bolts, i.e., installing bolts longitudinally ahead of the tunnel face to avoid overbreak and maintain the tunnel profile, and/or prevent collapses (Figure 4-3). The purpose of forepoling bolts is to prevent rockfalls or collapses, thereby maintaining the tunnel contour and the support recommendations according to the Q-system are applicable. Forepoling bolts are not considered part of the permanent support structure in the Q-system.

The need for forepoling depends on the geometry of the underground opening (e.g., tunnel portals and tunnel intersections), span width, joint orientations, and the rock mass quality. A qualitative engineering geological assessment should always be conducted to determine the necessity of forepoling. Surveys giving information about the rock mass quality ahead of the tunnel face, e.g. probe drilling, can indicate the need for spiling bolts. In rock masses where there is a significant risk of "geologically induced collapse," the use of spiling bolts is recommended.

Normally, forepoling is used in combination with reduced excavation length of the blast rounds and/or excavation in multiple drifts (sequential excavation). The spacing between spiling bolts is usually around 0.3 meters (0.2 - 0.6 meters). The rear end of the bolts must be anchored in the overlying rock to prevent collapse or failure of the spiling bolts during excavation. Spiling bolts can be anchored either with steel straps in combination with radial rock bolts or anchored into an RRS.

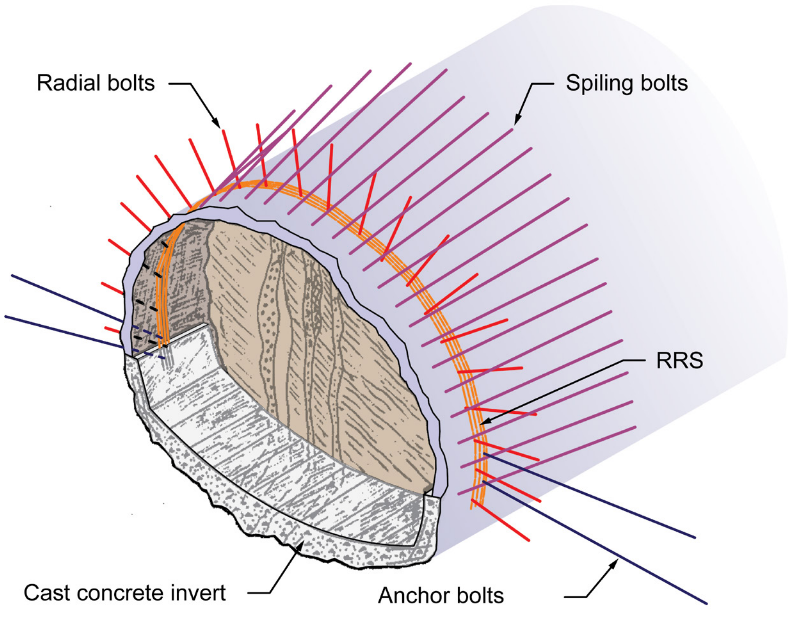


Figure 4-3 Support of poor rock masses by use of RRS and spiling bolts (after Holmøy and Aagaard, 2002)

4.5 Energy absorption of sprayed concrete

Based on the expected deformations and loads under various rock mass qualities, the energy absorption classes E700 and E1000 have been included in the support chart. These energy absorption classes correspond to those defined by EFNARC and are given in the guidelines from the Norwegian Concrete Association's Publication 7–2022, as shown in Table 4-3.

Macro synthetic fibres is an alternative to steel fibre in the sprayed concrete. Use of macro synthetic fibres give the sprayed concrete properties comparable to use of steel fibres, but the synthetic fibres are slightly more elastic. Their great advantage is that they do not corrode, which is beneficial in corrosive environments.

Table 4-3 Energy absorption classes based on the panel test as described in Norwegian Concrete Association Publication no. 7 (NB, 2022).

Energy Absorption Class	Min. energy absorption in Joule
E700	700
E1000	1000

4.6 Additional comments on stability and rock support

A Q-value provides a quantitative indication of the rock mass quality. The support chart in Figure 4-1 offers an indicative recommendation on rock support for the evaluated rock mass, derived from empirical data on rock support in rock masses of similar quality. However, the Q-value and support chart do not capture all engineering geological and rock mechanical details, and there may be specific cases where it is appropriate to deviate from the Q-system's support recommendations.

The Q-system is not suitable for determining support for individual blocks or wedges. Such rock support should be based on analytical assessments of geometric conditions and driving/stabilizing forces.

For blocky rock mass consisting of relatively large blocks, bolting is recommended to be performed before the application of sprayed concrete (see Figure 4.4e). For more fractured rock mass, it is recommended to apply sprayed concrete before bolting so that the blocks between the bolts are held in place through the interaction between sprayed concrete (Sfr) and rock bolts (see Figure 4.4f).

Other examples of unfavourable joint geometries that require special attention regarding bolting are shown in Figure 4-4 a-d). In the crown of an excavation, joints with sub-parallel strike direction to the length of the excavation but with variable dip directions may create unstable wedges (Figure 4-4 a). A combination of sub-horizontal and sub-vertical joints may require special attention because a sub-horizontal joint may intersect the rock mass just above the crown and may not be seen before failure (Figure 4-4 b). In such situations longer bolts than those recommended by the Q-system could be the solution. It is also recommended to adjust the directions of the rock bolts in such cases

In the crown, joints with a strike nearly parallel to the axis of the rock opening, but with varying dip directions, can create unstable wedges (Figure 4-4 a). A combination of near-horizontal and near-vertical fractures may require special attention because a near-horizontal fracture can cross the rock mass just above the roof and remain unseen until it causes a rockfall (Figure 4-4 b). In such situations, longer bolts than those recommended by the Q-system might be necessary. It is also recommended to adjust the bolt direction in such cases.

Inclined joints intersecting the walls in an underground opening could serve as sliding planes for unstable blocks. In such cases the stability of opposite walls may be quite different depending on the dip direction of the joints (Figure 4-4 c). If two intersecting joints form a wedge as shown in Figure 10d, a similar situation will occur.

In some specific cases with $J_r = 3$, $J_a = 1$, and $RQD/J_n < 2$ in highly fractured rock, the Q-value alone may provide a misleading basis for rock support because the small, unbonded blocks can reduce stability despite a relatively high Q-value. This can be compensated for by increasing the SRF value (as for a weakness zone) and using $J_r = 1$ (due to the lack of rock contact between the joint surfaces).

In some specific cases with $J_r = 3$, $J_a = 1$ and $RQD/J_n < 2$ in heavily jointed rock (almost sugar cube jointing), the Q-value alone may provide a misleading basis for rock support because the small, unbonded blocks may give reduced stability despite a relatively high Q-value. This can be compensated for by increasing the SRF value (as for a weakness zone) and using $J_r = 1$ (due to the lack of joint wall contact).

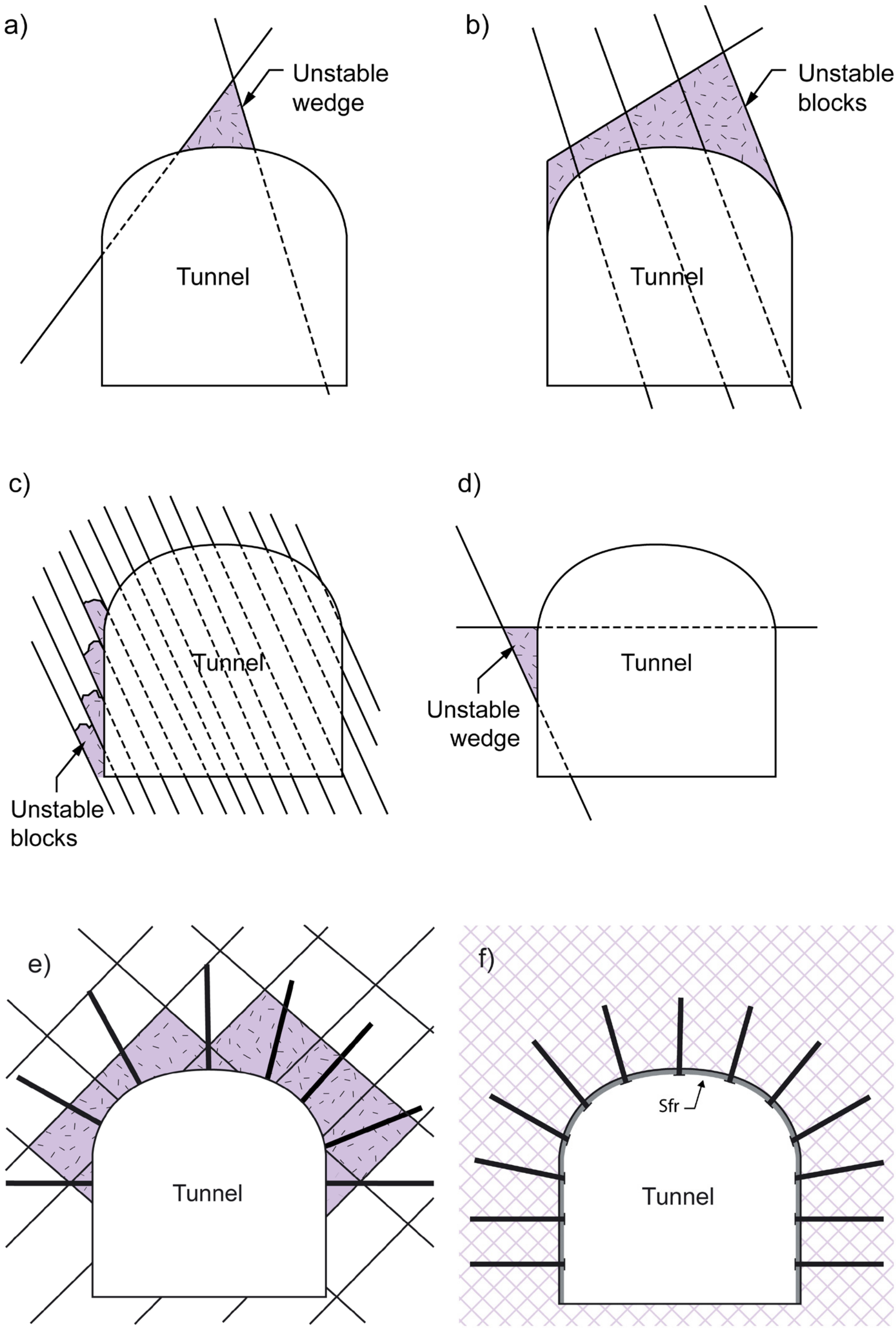


Figure 4-4 Stability problems caused by joints with unfavourable orientations. Sfr=fibre-reinforced sprayed concrete sprayed concrete.

4.7 Recommendations for challenging rock mass conditions

The Q-system's recommendations for rock support must always be considered as guidelines. A classification of the rock mass according to the Q-system may, in some cases, be supplemented with a complete rock mass characterization to provide a sufficient basis for decision-making or design when determining rock support. Therefore, it is recommended to perform engineering geological and rock mechanics assessments as a supplement to the recommendations given in the support categories (Figure 4.1). This is generally applicable to all classes of rock mass quality but will be especially important for the poorest rock mass conditions (support categories 6-9). Below are some examples where supplementary assessments to achieve a more complete rock mass characterization are recommended.

Rock masses where one or a few individual conditions are decisive for stability require engineering geological support assessments beyond the Q-system's support recommendations. A decision on rock support based on a calculated Q-value under such conditions may result in details of the properties of the rock mass being overlooked. Therefore, it is important that specially tailored solutions are based on a complete rock mass characterization. The use of such rock support solutions must be described and documented.

The majority of the reference cases on which the Q-system's support chart is based come from hard and fractured rock with various combinations of weakness zones and rock stress conditions. Thus, determining rock support using the Q-system is most applicable to these rock mass conditions. Examples of rock mass conditions that are less represented in the reference cases include:

- Rocks with low mechanical strength
- Rock stress anisotropy
- Time-dependent stability conditions and stress-induced deformations
- Especially unfavorable geometric configurations of weakness zones and rock fracturing

For such conditions, it is recommended to perform supplementary rock mass assessments, e.g. various analytical methods and/or numerical modelling, for determining an adequate rock support design. For example:

Analytical Methods

- Establish a geological model focusing on possible failure cases.
- Analytical calculation of failure cases, such as kinematic analysis and wedge calculation.

- Qualitative failure and stability analysis, with particular focus on unfavourable geometric configurations of weaknesses in the rock mass.

Numerical Modeling

- Continuous numerical analysis. Examples of software include RS2 and RS3 (Rocscience), Plaxis 2D and 3D (Plaxis BV), Flac2D and 3D (Itasca).
- Discontinuous numerical analysis. Examples of software include UDEC and 3DEC (Itasca).

A complete rock mass characterization can also be achieved through a hybrid approach, using both analytical and numerical analysis. Such approach is described by Terron-Almenara (2024).

5 Mapping in underground openings

5.1 General

The required level of rock support is generally assessed through geological mapping at the excavation face, with the Q-value serving as a good indication for the needed permanent rock support.

This handbook includes tables for each of the six Q-parameters at the back of the book, which can be used during field mapping. Nowadays, field mapping is often performed using digital mapping tools, where Q-tables are integrated into the mapping application. A description and sketch of the key geological structures also provide valuable documentation to the mapping. The Q-system's support recommendations should be considered as guidelines, and engineering geological assessments must always be made to determine if the recommendations are applicable to the evaluated rock mass. Any deviation from the support recommendations should be documented and described.

When conducting a geological mapping of a tunnel and underground opening, it is important to thoroughly visually inspect and document the observations of the rock surface around the entire tunnel before sprayed concrete is applied. In addition to visual inspection, using a scaling bar will provide important information about potential weaknesses or detached areas in the rock. Even small joint structures not visible from the face level can be observed upon closer inspection. Poor rock mass areas can have the same geological structures as the original, intact rock and may therefore not be visible from distance. To observe the rock mass up close, it is important to have access to the face and crown using suitable lifting equipment.

For most large underground projects, there is a requirement for engineering geological documentation through mapping using the Q-system.

5.2 Engineering geological mapping

Observations of rock types, rock boundaries, joint structures and geometry, weakness zones, and water from the engineering geological mapping should be included into a longitudinal engineering geological of the tunnel as a supplement to the Q-classification. The mapping should supplement photos/scans of the tunnel face, roof, and walls. Encountered rock types that represent a mechanical change should be thoroughly described. Weakness zones should be recorded and described in terms of orientation, width, and joint infilling. For the joint sets, the joint spacing and the persistence (i.e., how "long" a joint is) should be indicated in terms of the range of variation and typical value.

It is important to record the time that has passed since excavation, as values for J_w and SRF may change over time. If there are joints with clay fillings present in the rock mass, it may be necessary to take samples and conduct laboratory tests to identify the clay minerals and swelling potential for a final decision on the J_a value.

5.2.1 Use of digital mapping tools for engineering geological mapping

The use of digital tools and solutions in engineering geological mapping and rock mass classification offers several advantages that simplify both data flow and the application of the Q-system. Overall, utilizing digital solutions increases efficiency and provides a more comprehensive basis for decision-making regarding rock support. Examples of digital tools include:

- Use of a digital mapping platform (tablet and software), see Figure 5-1.
- Mapping on a digital image or 3D model from scanning/photogrammetry.
- Semi-automatic/automatic digital joint recognition to reduce geologists' subjectivity/perception when identifying rock wedges, fracture-bounded blocks, and joint surfaces with approximately similar orientation for complete characterization of joint sets.
- Semi-automatic/automatic digital fracture recognition to reduce the geologist's subjectivity/perception when identifying rock wedges, fracture-bounded blocks, and fracture surfaces with approximately similar orientation for complete characterization of fracture sets.
- Transfer of georeferenced and customized rock support information, e.g., for individual blocks and crushed zones.
- Prediction of rock type using machine learning from MWD data. MWD data can indicate changes in rock mass conditions (rock boundaries, weakness zones, slickensides, etc.), as shown in the upper figure of Figure 5-1. A visual assessment of MWD data is often sufficient to detect upcoming weakness zones or rock type changes.

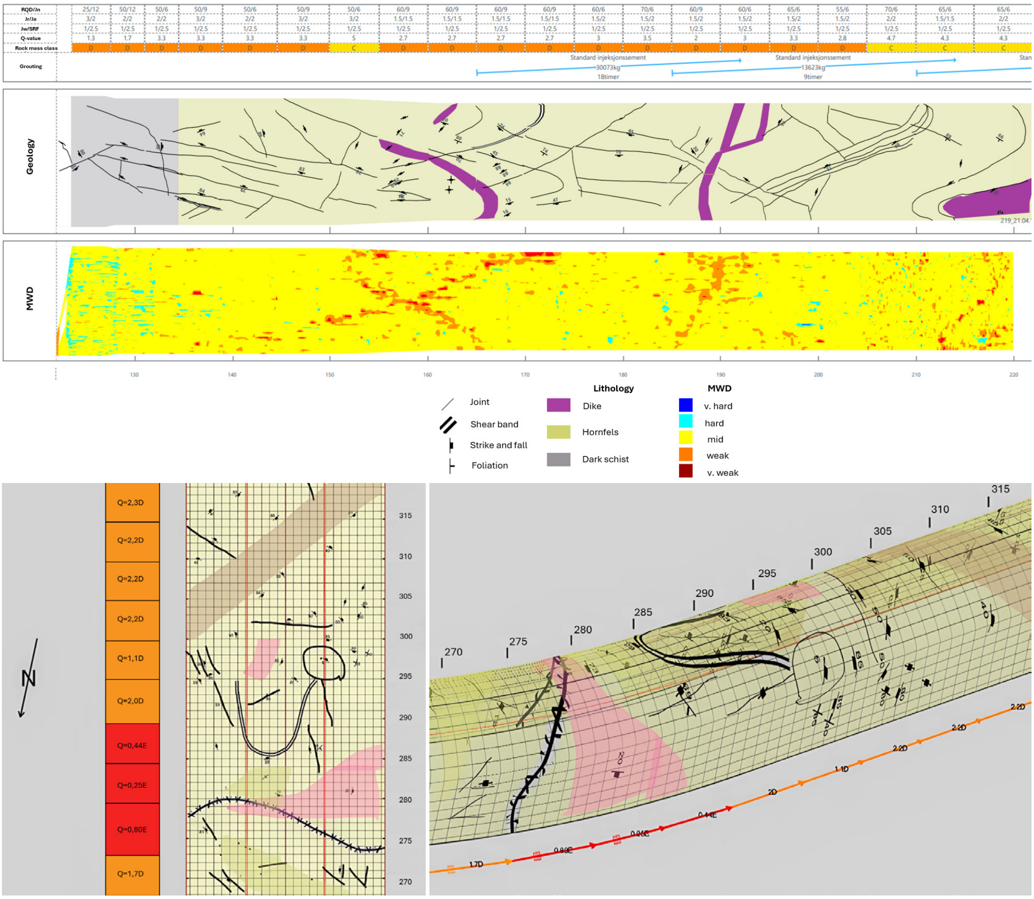


Figure 5-1 Upper figure: Digital tunnel map showing rock mass quality, geology, and MWD data. Lower figure: Geological mapping in 2D and 3D. Note that the upper and lower figures do not represent the same tunnel. Figures are from Bever Mapping.

5.2.2 Mapping of sections

When each blast round is mapped, the area can be considered as a section. However, if there are significant local variations in Q-parameters, it may be appropriate to divide the area into smaller sections (subsections). This is most relevant in tunnels and underground openings with large cross-sections. The purpose is to differentiate the rock support in the various sections (bolt spacing and length, and/or shotcrete thickness/type). RRS normally require support throughout the entire profile (wall-roof-wall) to ensure the function (arching effect) of the support.

Chapter 3.1 describes cases where it may be appropriate to assign a range of values to the Q-parameters instead of a single specific value. When dividing mapping sections due to significant local variations in the rock mass, the parameter values used to calculate the final Q-value must occur within the same subsection.

By defining minimum and maximum values for each parameter, it becomes possible to calculate a Q_{min} and Q_{max}, rather than determining a final parameter value during mapping. If Q_{min} and Q_{max} lead to different support categories, an evaluation of the impact of the different Q parameters on the stability of the underground opening must be conducted. It is crucial to weight the different Q-parameters/Q-values in the various sections based on their impact on the overall stability of the rock mass. For example:

- Weighting of joint roughness/planarity in relation to the sliding direction of potential rock blocks.
- The orientation of joints in relation to block formation and where in the profile they may cause unstable rock blocks influences the weighting of the parameters.
- The exposure of weakness zones/slip surfaces relative to the underground opening should also be assessed regarding their impact on stability.

Recording variations in Q-parameters, as well as Q_{min} and Q_{max}, provides valuable documentation of rock mass variability, in addition to supplementary text describing the variations in the rock mass.

5.2.3 Mapping of weakness zones

A weakness zone can be defined as a zone or layer in the bedrock with poorer mechanical properties than the surrounding rock mass. The width of a weakness zone varies from a decimeter to hundreds of meters in extreme cases. The most common types of weakness zones are:

- Shear zones, i.e., fault zones where the rock mass is highly fractured, folded, or crushed into small pieces and may contain clay.
- Weathered zones with altered rock, weak mineral layers with low shear strength, and/or clay.

The thickness/width of the weakness zone, its orientation relative to the tunnel, the rock mass quality within the zone, and the quality of the adjacent rock mass are factors that must be considered when mapping and deciding rock support design in weakness zones. The width of a weakness zone is measured perpendicular to the strike direction of the zone, but it is also important to consider its orientation relative to the underground opening. The more acute the angle between the zone and the axis of the excavation is, the larger the affected section of the excavation will beone.

Narrow weakness zone: A narrow weakness zone can be defined as a zone with a width ranging from a decimeter up to 2–3 meters, where the width is generally much smaller than the span of the underground opening. In a narrow weakness zone, rock support can generally be anchored in higher-quality adjacent rock mass, and the support design will typically extend about 1 meter on each side of the zone.

For narrow weakness zones, it is usually not practical to assess the support requirements based solely on the Q-value of the zone itself, as this approach may result in an unnecessarily conservative Q-value. It is therefore recommended to determine an average Q-value for both the weakness zone and the adjacent rock using the following formula (Løset, 1997):

$$logQ_m = \frac{b \times logQ_{sone} + logQ_{sr}}{b+1}$$

where:

- Q_m = Mean Q-value of weakness zone/surrounding rock mass
- Q_{sone} = The Q-value of the weakness zone
- Q_{sr} = The Q-value of the surrounding rock mass
- b = The width of the weakness zone measured along the length of the excavation

Be aware of the following when using the equation:

- Since the Q-value follows a logarithmic scale, the calculation must be performed logarithmically.
- In cases where the adjacent rock has a very high Q-value, the formula may result in an overestimated Q_m-value.
- For a relatively narrow weakness zone (i.e., b ≈ 0.5 m) parallel to the tunnel axis, the formula will give $Q_m \approx Q_{sone}$, which may result in an average Q-value (Q_m) that is too low.

Wide weakness zone: Can be defined as zones with a width greater than 2-3 m, or several consecutive narrow weakness zones. In such cases, the adjacent rock mass may be of such low quality that the rock support needs to be self-supporting. If the zone(s) represent such a large portion that they cannot be anchored in good adjacent rock, the following use of the Q-system is recommended:

- RQD and J_n for the zone should be specified. The final determination of RQD and J_n must be based on a weighted stability assessment of the entire rock mass being considered.
- J_r and J_a are determined based on the properties of the weakness zone. For thick joint filling, category c) applies for both J_r (Table 3-3) and J_a (Table 3-4).
- SRF is determined based on category a) in Table 3-7, see Chapter 3.7.1.

In cases of wide weakness zones, it is also important to identify which engineering geological and rock mechanical aspects that are decisive for the stability and rock support requirements for the underground opening. Mapping in wide weakness zones should therefore always include a rock mass characterization (see Chapter 4.7).

5.2.4 HSE during mapping

When conducting geological mapping in an underground opening, it is important to ensure that the freshly exposed rock surfaces after blasting is adequately scaled (cleaning the rock surface from loose rock after blasting). This is relevant whether the mapping is carried out from a basket/lift or from the tunnel floor. For large underground opening, e.g. large caverns, it is advisable to conduct the mapping from a basket/lift to assess the rock mass quality up close. This is particularly important for determining J_r and J_a values, which are difficult to assess from distance. Mapping from a basket/lift should only be performed if it is considered safe in terms of rock mass stability.

5.3 Mapping in tunnels excavated by TBM

Mapping rock mass quality in a TBM tunnel is more challenging than in tunnels excavated by drill & blast. In case of rock mass quality mapped as Q >1, the walls in a TBM tunnel may be quite smooth, making it difficult to identify joints and study the joint surfaces. A hammer can be useful to distinguish real joints from veins, foliation, etc., in order to estimate the RQD-value. Estimates of J_r and J_a may be inaccurate if few or no joint surfaces is exposed. By inserting a knife into the joint, the joint infilling can be evaluated, and clay infill can be detected. In poor rock masses, observation of the Q-parameters may be easier as more joint surfaces are exposed due to overbreak and fallout.

When mapping in a TBM tunnel, extra care must be taken, especially when clay-filled joints are observed. Loose wedges detached by joints with unfavourable orientations may remain in place after excavation. Unstable blocks/wedges can suddenly fail without warning. The friction angle along the joints may be difficult to observe but can be indicated by the joint properties (J_r/J_a). It is of great importance to carefully study the general geology and to observe joint orientations and joint properties.

6 The Q-system during pre-investigations

6.1 General

The Q-system can also be used during preliminary investigations for underground openings. During the planning and investigation phase of tunnel and underground projects, the Q-system can be used to make detailed descriptions of the rock mass as a basis for forecasting rock support requirements and associated costs. Aspects related to Q-values based on rock outcrops and core drilling during preliminary investigations are described in the following subsections.

6.2 Use of the Q-system during field mapping

Field mapping is often an important part of the pre-investigation for tunnels and caverns. The reliability of the results of the field mapping will depend on the available rock outcrops. Evaluation of the Q-value may be possible with a reasonable degree of accuracy if the outcrops are large and of good quality.

The rock mass near the surface will often be more jointed than unweathered rock masses at a greater depth. This may especially be the case in rocks with schistosity which often have a tendency to crumble near the surface. If there are few outcrops, often only the competent rock masses will be visible, while fractured zones may be eroded and covered by soil.

At the surface, joint infillings will often be washed away, and the J_a-value may therefore be difficult to determine. Many natural outcrops are often scoured by ice and water in Nordic countries, making it difficult to observe all the existing joints. In other countries where weathering is more common, the joints may also be hidden at the surface.

The joint infilling is often still present in road cuts or other excavated slopes. The joint surfaces are normally exposed after blasting, giving a more reliable basis for estimating J_n and J_r, in addition to J_a. The Q-value is often lower in blasted cuts and slopes compared to natural rock surfaces. In quarries, where cuts are made in different directions, the Q-value will be approximately the same as the value observed in an underground opening. Water conditions (J_w) in an underground opening are difficult to predict solely from field mapping. Water loss measurements/Lugeon tests in boreholes and/or empirical data from projects in similar rock masses are necessary for making accurate assumptions regarding water conditions.

An assumption of the SRF-value can be made based on topographical conditions and available information regarding the stress situation in the area. When estimating the SRF-value during the planning phase of an underground project, general experience from the geological region can be valuable. Information from nearby underground excavations and topographical features may be helpful. In areas with high, steep mountain sides there is often an anisotropic stress field. Geological structures, such as surface-parallel joints and exfoliation joints, are indicators of high anisotropic stresses. The occurrence of spalling/exfoliation in high rock slopes or in the rock mass surrounding an underground excavation depends on the ratio between the induced stress (determined by the slope height above the excavation) and the compressive strength of the rock. In Table 3-7, a ratio of $\sigma_c/\sigma_1 < 4\text{-}5$ (depending on the degree of anisotropy) is typically an indication that spalling may occur in an underground excavation. In hard rock, this generally occurs with rock overburden between 400 and 1100 meters in the valley slope above the excavation, depending on the compressive strength of the intact rock and the inclination of the slope (see Figure 3-5). Stress measurements can also be conducted before the excavation of underground facilities where stress-related problems are anticipated.

Given the above-mentioned aspects, it is recommended not to rely solely on the Q-value obtained from field mapping as the basis for determining rock support without conducting further evaluations.

6.3 Use of the Q-system for core logging

Pre-investigations for underground excavations often include core logging. Often, core samples are missing from sections with poor rock quality (core loss), and in such cases, it is generally assumed that the Q-value is low. Where cores are available, most Q-parameters can be determined with a relatively high degree of accuracy. However, particular attention should be given to the following:

- Only a small section of each joint surface will usually be visible, particularly for joints intersecting the borehole at an obtuse angle. Evaluation of the roughness coefficient, J_r, may therefore be unprecise. Particularly the surface waviness can be difficult to estimate (see Chapter 3.4)
- As water is used during drilling, mineral fillings like clay minerals may be washed out, making it difficult to evaluate J_a in some cases.
- The drilling direction of the borehole influences the number of joints that are intersected by the borehole. Sub-parallel joints to the borehole will be underrepresented in the cores, leading to too high RQD-values and too low J_n-values.
- Note that RQD can be calculated as 0 during core logging. In such cases, RQD should be set to 10 in the Q-system.
- Whereas RQD is often calculated for every meter, J_n must usually be estimated for sections of several metres.

Generally, a core log should only contain data obtained from the cores or measurements carried out in the borehole itself. This means that Q-values should not be included in such a log. However, by using the logging data combined with estimates of J_w and SRF, it is possible to provide a rough estimate of the Q-values for the cores, which can be used for planning forecasts of underground facilities. Water loss tests are often carried out during core drilling. The results are normally given in Lugeon (Lugeon = the loss of water in litres per minute and per metre borehole at an over-pressure of 1 MPa), and form the basis for evaluation of the J_w-value. It is also necessary to consider whether the rock mass will be grouted or not when determining the Q-value as a basis for rock support after excavation.

It is always important to evaluate how representative the cores are. Boreholes are often drilled just to investigate particular zones. It is crucial to evaluate how much of the rock mass these zones represent. If a borehole is oriented along a weakness zone, the Q-parameter values determined from core logging will be applicable to that specific zone.

7 References

Alejano, L.R., Muralha, J., Ulusay, R. et al. (2018): ISRM Suggested Method for Determining the Basic Friction Angle of Planar Rock Surfaces by Means of Tilt Tests.

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

Barton, N. & Choubey, V. (1977). The shear strength of rock joints in theory and practice. Rock Mechanics Felsmechanik Mecanique des Roches. 10. 1-54.

Bhasin, R. and Grimstad, E. (1996): The use of stress – strength relationships in the assessment of tunnel stability. Tunnelling and Underground Space Technology, 11(1): 93-98.

Deere, D.U. (1963): Technical description of rock cores for engineering purposes. Felsmechanic und Ingenieurgeologie 1: 16-22.

Grimstad, E., Kankes, K., Bhasin, R., Magnussen, A. and Kaynia, A. (2002): Rock mass quality Q used in designing reinforced ribs of sprayed concrete and energy absorption. I proceedings: International Symposium on Sprayed Concrete. Davos 2002, pp. 134-142.

Grimstad, E. and Barton, N. (1993): Updating of the q-system for NMT. I proceedings: International Symposium on Sprayed Concrete. Fagernes 1993, pp. 46-66.

Holmøy, K.H. andAagaard, B. (2002): Spiling bolts and reinforced ribs of sprayed concrete replace concrete lining. Tunnelling and Underground Space Technology 17(4): 403-413.

ISRM (1978): Suggested methods for the quantitative description of discontinuities in rock masses. International Journal of Rock Mechanics, Mining Sciences & Geomech. Abstr. 16(3): 319-368.

Løset, F. (1997): Engineering Geology - Practical use of the Q-method. NGI-rapport 592046-4.

Muralha, J., Grasselli, G., Tatone, B., Blumel, M., Chryssanthakis, P., Jiang, Y. (2014). ISRM Suggested Method for Laboratory Determination of the Shear Strength of Rock Joints: Revised Version. Rock Mechanics and Rock Engineering. 47. 291-302.

NB (2022): Sprayed Concrete for Rock Support. Publication no. 7, Norwegian Concrete Association.

Palmström, A. (2005): Measurements of and Correlations between Block Size and Rock Quality Designation (RQD). Tunnelling and Underground Space Technology, 20(4): 362-377.

Singh, B., Jethwa, J.L., Dube, A.K. and Singh, B. (1992): Correlation between observed support pressure and rock mass quality. Tunnelling and Underground Space Technology 7(1): 59-74.

Terron-Almenara, J., Skretting, E., Holter, K., Høien, A. (2024). Design of Rock Support in Hard and Layered Rock Masses Using a Hybrid Method: A Study Based on the Construction of the New Skarvberg Tunnel, Rock Mech Rock Eng 57:10491–10532.

Appendix; Tables 1 to 9

1	RQD (Rock Quality Designation)		RQD
A	Very poor	(> 27 joints per m^3)	10i) -25
B	Poor	(20-27 joints per m^3)	25-50
C	Fair	(13-19 joints per m^3)	50-75
D	Good	(8-12 joints per m^3)	75-90
E	Excellent	(0-7 joints per m^3)	90-100
Note: i) Where RQD is reported or measured as ≤ 10 (including 0) the value 10 is used to evaluate the Q-value ii) RQD-intervals of 5, i.e. 100, 95, 90, etc., are sufficiently accurate			

2	J_n = Joint set number	J_n
A	Massive, no or few joints	0,5-1,0
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four or more joint sets, random heavily jointed “sugar cube”, etc	15
J	Crushed rock, earth like	20
Note: i) For tunnel intersections, use 3 x J_n ii) For tunnel portals, use 2 x J_n		

3	J_r = Joint Roughness Number	J_r
a) Rock wall contact ***b) Rock-wall contact before 10 cm of shear movement***		
A	Discontinuous joints / rough, stepped	4
B	Rough or irregular, undulating / smooth, stepped	3
C	Smooth, undulating / slickensided, stepped	2
D	Slickensided, undulating	1,5
E	Rough, irregular, planar	1,5
F	Smooth, planar	1
G	Slickensided, planar	0,5
Note: i) Add 1 if the mean spacing of the relevant joint set is greater than 3 m (dependent on the size of the underground opening) ii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented in the estimated sliding direction		
c) No rock-wall contact when sheared (thick joint infill)		
H	Zone containing clay minerals thick enough to prevent rock-wall contact when sheared	1

4	J_a = Joint alteration numberg	Φ_r	J_a
a) Rock-wall contact (no mineral fillings, only coatings)			
A	Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote.		0,75
B	Unaltered joint walls, surface staining only.	25-35°	1
C	Slightly altered joint walls. Non-softening mineral coatings; sandy particles, clay-free disintegrated rock, etc.	25-30°	2
D	Silty or sandy clay coatings, small clay fraction (non-softening).	20-25°	3
E	Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talc gypsum, graphite, etc., and small quantities of swelling clays	8-16°	4
b) Rock-wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, etc.	25-30°	4
G	Strongly over-consolidated, non-softening, clay mineral fillings (continuous, but < 5 mm thickness)	16-24°	6
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but < 5 mm thickness)	12-16°	8
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but < 5 mm thickness). Value of Ja depends on percent of swelling clay-size particles.	6-12°	8-12
c) No rock-wall contact when sheared (thick mineral fillings)			
K	Zones or bands of disintegrated or crushed rock. Strongly over-consolidated.	16-24°	6
L	Zones or bands of clay, disintegrated or crushed rock. Medium or low over-consolidation or softening fillings.	12-16°	8
M	Zones or bands of clay, disintegrated or crushed rock. Swelling clay. J_a depends on percent of swelling clay-size particles.	6-12°	8-12
N	Thick continuous zones or bands of clay. Strongly over-consolidated.	12-16°	10
O	Thick, continuous zones or bands of clay. Medium to low over-consolidation	12-16°	13
P	Thick, continuous zones or bands with clay. Swelling clay. J_a depends on percent of swelling clay-size particles.	6-12°	13-20

5	J_W = Joint Water Reduction Factor	J_W
A	Dry excavations or minor inflow (humid or a few drips)	1,0
B	Medium inflow, occasional outwash of joint fillings (many drips/"rain")	0,66
C	Jet inflow or high pressure in competent rock with unfilled joints	0,5
D	Large inflow or high pressure, considerable outwash of joint fillings	0,33
E	Exceptionally high inflow or water pressure decaying with time. Causes outwash of material and perhaps cave in	0,2-0,1
F	Exceptionally high inflow or water pressure continuing without noticeable decay. Causes outwash of material and perhaps cave in	0,1-0,05
Note: i) Factors C to F are rough estimates. Increase J_w if the rock is drained or if injection is performed. ii) Special problems caused by ice formation are not taken into consideration.		

6	SRF = Stress Reduction Factor	SRF
a) Weak zones intersecting the underground opening, which may cause loosening of rock mass		
A	Multiple occurrences of weak zones within a short section containing clay or chemically disintegrated, very loose surrounding rock (any depth), or long sections with incompetent (weak) rock (any depth). For squeezing, see 6M and 6N	10
B	Multiple shear zones within a short section in competent clay-free rock with loose surrounding rock (any depth)	7,5
C	Single weak zones with or without clay or chemical disintegrated rock (depth ≤ 50m)	5
D	Loose, open joints, heavily jointed or “sugar cube”, etc. (any depth)	5
E	Single weak zones with or without clay or chemical disintegrated rock (depth > 50m)	2,5
Note: i) Reduce these values of SRF by 25-50% if the weak zones only influence but do not intersect the underground opening		
b) Competent rock with low or favourable stress conditions, mainly massive rock		*SRF*
F	Low stresses, near surface, open joints	2.5
G	Medium stresses, favourable stress condition	1
Note: ii) When the depth of the crown below the surface is less than the span; suggest SRF increase from 2.5 to 5 for such cases (see F)		
c) Competent, mainly massive rock, stress-related problems		*SRF*
H	High stress, very tight structure. Usually favourable to stability. May also be unfavourable to stability dependent on the orientation of stresses compared to jointing/weakness planes	0,5-2 2-5*
J	Moderate spalling and/or slabbing after > 1 hour in massive rock	5-50
K	Spalling or rock burst after a few minutes in massive rock	50-200
L	Heavy rock burst and immediate dynamic deformation in massive rock	200-400
Note: iii) See Chapter 3.7.3 and Grimstad & Barton (1993) for details on SRF and the stress-strength ratio.		
d) Squeezing rock: plastic deformation in incompetent rock under the influence of high pressure		*SRF*
M	Moderate squeezing rock pressure	5-10
N	Heavy squeezing rock pressure	10-20
Note: iv) Determination of squeezing rock conditions must be made according to relevant literature (i.e. Singh et al., 1992 and Bhasin and Grimstad, 1996)		
e) Swelling rock: chemical swelling activity depending on the presence of water		*SRF*
O	Moderate swelling rock pressure	5-10
P	Heavy swelling rock pressure	10-15

8	Type underground facility	ESR
A	Temporary mine openings, etc.	3-5
B	Vertical shafts*: i) circular sections ii) rectangular/square sections * Dependant of purpose. May be lower than given values.	2,5 2,0
C	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks), water supply tunnels, pilot tunnels, drifts and headings for large openings.	1,6
D	Minor road and railway tunnels, surge chambers, access tunnels, sewage tunnels, etc.	1,3
E	**Power houses, storage rooms, water treatment plants, major road and railway tunnels, civil defence chambers, portals, intersections, etc.**	**1,0**
F	Underground nuclear power stations, railways stations, sports and public facilitates, factories, etc.	0,8
G	Very important caverns and underground openings with a long lifetime, ≈ 100 years, or without access for maintenance.	0,5

9 Dimensioning of wall support		
In rock masses of good quality	Q > 10	Multiply Q-values by a factor of 5
For rock masses of intermediate quality	1 < Q < 10	Multiply Q-values by a factor of 2.5. In cases of high rock stresses, use the actual Q-value
For rock masses of poor quality	Q < 1	Use actual Q-value
Wall height > span width	Applies for all Q-values	Use actual Q-value

NGI

www.ngi.no