HANDBOOK

Using the Q-system

Rock mass classification and support design

NGI

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Preface

The development of the NGI Q-system for rock mass classification began in the early 1970's, and was first published in 1974. NGI has continuously improved and updated the system, and produced the Q-system handbook in 2012 as a summary of NGI's Best Practice. This current edition includes NGI developments and experience gained since the publication of the first edition.

Updates:
June 2022: Revised rock support chart from 2019, page 34.

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1 Introduction

1.1 History

The Q-system was developed at NGI between 1971 and 1974 (Barton et al. 1974). Since the introduction of the Q-system in 1974 there has been a 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 in many ways changed the support procedure. Application of sprayed concrete has gained acceptance even for good quality rock masses due to demands for a higher level of safety during the recent years. Reinforced ribs of sprayed concrete have replaced cast concrete structures to a large extent.

Since the introduction of the system in 1974, two revisions of the support chart have been carried out and published in conference proceedings. An extensive updating in 1993 was based on 1050 examples mainly from Norwegian underground excavations (Grimstad and Barton, 1993). In 2002, an updating was made based on more than 900 new examples from underground excavations in Norway, Switzerland and India. This update also included analytical research with respect to 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).

In this handbook, the guidelines for RRS in the support chart have been updated. The RRS-guidelines are simplified with regards to recent case histories in Norway.

1.2 Areas of application

The Q-value may be used for classification of the rock mass around an underground opening, as well as for field mapping. This means that the Q-value may be influenced by the blasting on the underground opening. The Q-value in an undisturbed rock mass may be different.

The Q-system is a classification system for rock masses with respect to stability of underground openings. Based on estimation of six rock mass parameters, a Q-value for a rock mass can be calculated. This value gives a description of the rock mass quality. The different Q-values are related to different types of permanent support by means of a schematic support chart. This means that by calculating the Q-value it is possible to find the type and quantity of support that has been applied previously in rock masses of the similar qualities. The Q-system can therefore be used as a guideline in rock support design decisions and for documentation of rock mass quality.

The Q-system is developed for use in underground openings. However, the system can also be used for field mapping, core logging and investigations in a borehole, but it is important to have in mind that in such cases some of the parameters may be difficult to estimate. The Q-values from field mapping and boreholes will therefore often be more uncertain than those mapped in an underground opening, and should be handled with some care.

1.3 Limitations

The majority of the case histories are derived from mainly hard, jointed rocks including weakness zones. From soft rocks with few or no joints there are only few examples, and by evaluation of support in such types of rocks, other methods should be considered to be used in addition to the Q-system for support design. It is important to combine application of the Q-system with deformation measurements and numerical simulations in squeezing rock or very weak rock.

The Q-system is empirical with regards to rock support. However, use of sprayed concrete has been extended in rock masses of good quality. The rock support recommendations are therefore conservative for these cases.

2 Rock mass stability

During underground excavation it is very important to have a close visual observation of the rock surface in the whole tunnel periphery before the rock is covered by sprayed concrete. In addition to the visual observation, hammering with a scaling rod or a hammer will give important observation of deterioration of unstable rock giving particular sounds. Also small cracks, invisible from the invert, will be observed with a closer look. Altered rock may show the same geological structures as the original fresh and unweathered rock, and may not be noticed when observed at distance. In order to have a close observation it is of outmost importance to have access to the face and crown by use of lifting equipment especially designed for this purpose.

Rock mass stability is influenced by several parameters, but the following three factors are the most important:

- Degree of jointing (block size)
- Joint frictions
- Stress

2.1 Degree of jointing

The degree of jointing, or block size, is determined by the joint pattern, i.e., joint orientation and joint spacing. At a certain location in the rock mass, there will, in most cases, be a joint pattern which could be well or not so well defined. Often joint directions exist systematically in rock masses, and most of the joints will be parallel with one of these directions. Near parallel joints form joint sets, and the joint spacing within each set will usually show characteristic distributions. The joint spacing may be reduced considerably along some zones in the surrounding rock. Such zones are called fracture zones. Stability will generally decrease when joint spacing decreases and the number of joint sets increases. In soft rocks where deformation can occur independently of joints, the degree of jointing has less importance than it has in hard rocks.

2.2 Joint friction

In hard rocks, deformations will occur as shear displacements along joints. The friction along the joints will therefore be significant for the rock mass stability. Joint friction is dependent on joint roughness, thickness and type of mineral fillings. Very rough joints, joints with no filling or joints with only a thin, hard mineral filling will be favourable for stability. On the other hand, smooth surface and/or a thick filling of a soft mineral will result in low friction and poor stability. In soft rocks where deformation is less dependent of joints, the joint friction factor is less significant.

2.3 Stress

The vertical stress in a rock mass commonly depends on the depth below the surface. However, tectonic stresses and anisotropic stresses due to topography can be more influential in some areas. Stability of the underground excavation will generally depend on the stress magnitude in relation to the rock strength. Moderate stresses are usually favourable for stability. Low stresses are often unfavourable for the stability. In rock masses intersected by zones of weak mineral fillings such as clay or crushed rock, the stress situation may vary considerably within relatively small areas. Experience from tunnel projects in Norway has shown that if the magnitude of the major principal stress approaches about 1/5 of the compressive strength of the rock, spalling may occur. When tangential stresses exceed the magnitude of the rock compressive strength, squeezing may occur. In other words; the anisotropy of the rock mass plays an important role when designing rock support.

3 The Q-system

The Q-value gives a description of the rock mass stability of an underground opening in jointed rock masses. High Q-values indicates good stability and low values means poor stability. Based on 6 parameters the Q-value is calculated using 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

The individual parameters are determined during geological mapping using tables that give numerical values to be assigned to a described situation. Paired, the six parameters express the three main factors which describe the stability in underground openings:

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

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

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

4 Calculation of the Q-value

4.1 General

Q-values can be determined in different ways, by geological mapping in underground excavations, on the surface, or alternatively by core logging. The most correct values are obtained from geological mapping underground. Each of the six parameters is determined according to a description found in tables. A complete set of these tables is inserted in the back cover of the handbook.

During the mapping it may be necessary to divide the underground excavation into several sections so that the variation of the Q-value within each section is moderate, i.e. this variation should not exceed that of a rock class in the support chart. During excavation, one blast round will often be a natural section for individual mapping. In sections of a few metres length there may be some variation, and in order to show this variation, histograms may be used during mapping.

The true Q-value at the level of excavation can only be observed in the excavation itself, and Q-values obtained by other methods will be more uncertain. The number of joint sets may be underestimated from drill cores and estimations of the parameters J_w and SRF may be cumbersome without actual observations on site. From surface mapping J_a may be particularly uncertain because joint filling may be washed out at the surface, and other joint parameters may be difficult to observe. In such cases it may be an advantage to use histograms to visualize variations in the data. By using maximum and minimum values for each parameter, the variations will be visualized, and Q-value can then be calculated by using the mean values for each parameter. In addition the maximum and minimum Q-value can also be estimated. An example from a long tunnel section is shown in Figure 1.

The Q-value varies between 0.001 and 1000. Please note that it is possible to get higher values and slightly lower values by extreme combinations of parameters. In such odd cases one can use 0.001 and 1000 respectively for determination of support.

4.2 Rock Quality Designation (RQD)

RQD 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 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 1 RQD-values and volumetric jointing.

1	RQD (Rock Quality Designation)		RQD
A	Very poor	(> 27 joints per m^3)	0-25
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			

In an underground opening or a cavern it is usually possible to get a three dimensional view of the rock mass. A three dimensional RQD may therefore be used. That means that the RQD-value is estimated from the number of joints per m^3. The following formula may be used (Palmström, 2005):

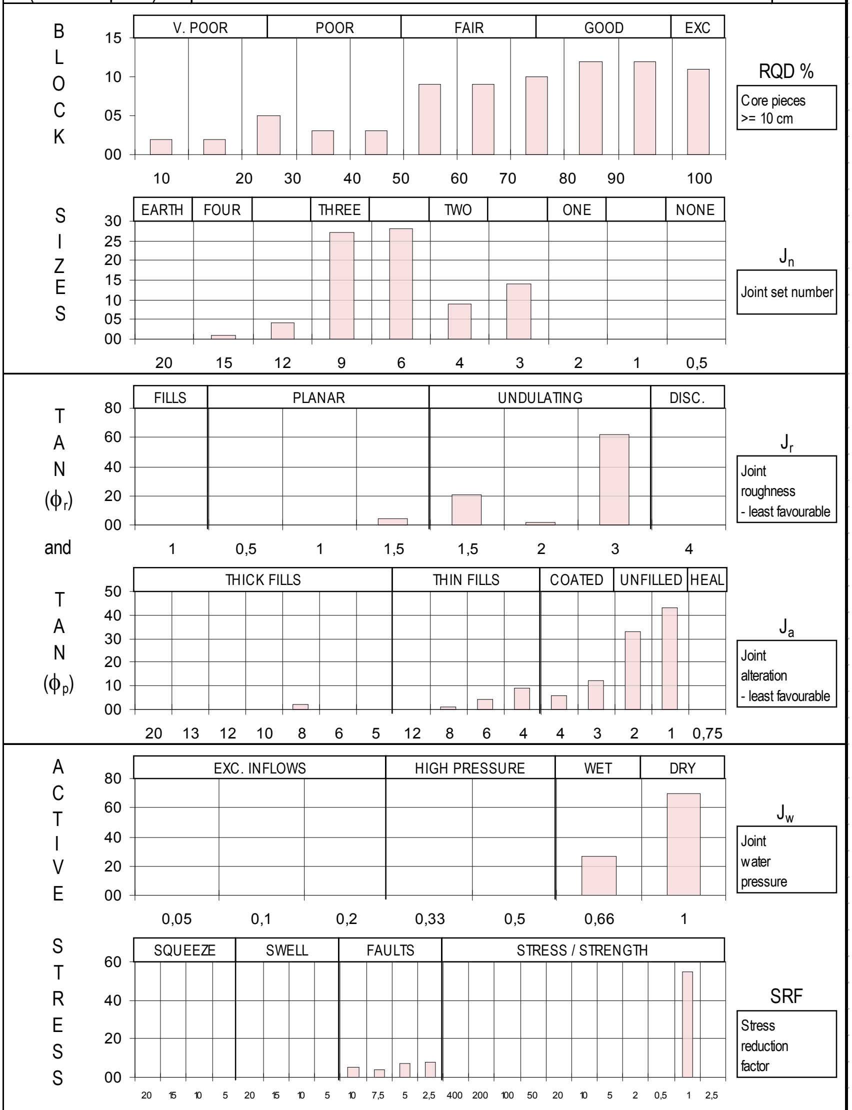
$$RQD = 110 - 2.5J_v \text{ (for } J_v \text{ between 4 and 44)}$$

where J_v is the number of joints per m^3

Based on the formula above, the number of joints per m^3 for each RQD-class is shown in Table 1. Several readings of RQD should be taken along surfaces of different orientation, if possible perpendicular to each other, and the mean value can then be used in the calculation of the Q-value. The variation in RQD-values may be shown in histograms.

In a rock exposure at the surface, it may be more difficult to obtain the correct RQD values. If an exposure constitutes of only one planar face, it may be difficult to determine the joint spacing of joints parallel or sub-parallel to this surface.

Q - VALUES:	(RQD	/	J_n)	*	(J_r	/	J_a)	*	(J_w	/	SRF)	=	**Q**
Q (typical min) =	10	/	15.0	*	1.5	/	8.0	*	0.66	/	10.0	=	**0.008**
Q (typical max) =	100	/	3.0	*	3.0	/	1.0	*	1.00	/	1.0	=	**100.0**
Q (mean value) =	71	/	6.7	*	2.6	/	2.3	*	0.91	/	2.4	=	**4.44**
Q (most frequent) =	70	/	9.0	*	3.0	/	1.0	*	1.00	/	1.0	=	**23.33**



Project name	Rev.	Report No.	Figure No.
Q - REGISTRATIONS CHART	Depth zone (m)	Drawn by	Date
	Logg	Checked	Approved

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Figure 1 An example of histogram presentation of Q-parameters from a long tunnel section.

4.2.1 RQD in blasted underground excavations

According to the original definition of RQD, only natural joints are to be considered. A close investigation of the muck pile from a relevant blast round may indicate the order of magnitude of the RQD. All types of fractures, despite their origin, may influence on the stability underground. Fractures caused by blasting will usually occur only in a zone up to 2 m from the periphery of the excavation, and may therefore be less significant for the overall stability than pervasive, natural joints. These artificial joints should not be taken into account when evaluating the RQD. However, they may be important for the stability of single blocks. Single blocks must be supported independently.

4.2.2 RQD in foliated rocks

In some cases there are uncertainties to which joints should be considered. This will often be the case in strongly foliated rocks or rocks with well developed schistosity. A schistose surface represents a weakness in the rock, and is not necessarily a natural joint. At the surface, schist often split up into thin flakes due to weathering, whereas some metres below the surface the rock may appear massive. Hence schistose and foliated rocks may have high RQD values.

Drilled cores of schist may also behave in the same way. Soon after drilling only few joints can be seen, and the RQD-value may be 100. After drying for some weeks, the cores may consist of only thin disks, and the RQD-value could be zero. In such cases it is difficult to say which RQD-value should be used in calculating the Q-value, and this uncertainty must therefore be taken into account in the support design.

In rocks with strongly developed foliation or schistosity it is often helpful to look at the rock rubble being mucked out of an excavation. The size of the blasted blocks will give a good indication of the RQD-value. In many cases apparent schistose rocks give blocks of considerable size when blasted. This means that in unweathered schists only a few of the surfaces developed from schistosity may be real joints, and rock types like phyllite, slate and mica schist may therefore in many cases have a RQD value of 100.

4.2.3 RQD in soft rocks

RQD may be difficult to define, and it is therefore important to consider RQD in relation to the other parameters such as SRF and J_n. Some soft rocks may have no or very few joints, and should therefore by definition have a high RQD-value. However, in weakly consolidated or strongly weathered non-cohesive material that can be defined as soil, the RQD-value should be stipulated to 10. In cohesive and soft material such as clay, the RQD-value should also be 10, because the material act as a weakness zone compared to the surrounding rock. When rock deformation is independent of jointing the Q-system compensates by using a high SRF value, i.e. squeezing rock.

4.2.4 RQD in relation to healed joints and mineral fillings

Healed joints and joints with mineral fillings can cause uncertainty in the calculation of RQD. The strength of the minerals in the joint filling is vital. Minerals such as chlorite, mica and clay will usually result in weak bindings between the joint walls, whereas epidote, feldspar and quartz do not necessarily mean a weakening of the rock mass. Joints with calcite filling may be more uncertain. In competent rocks, joints will form a weak surface, but in weak rocks there may be the opposite situation. A simple test is to hit the rock with a hammer and look for where the breaks occur.

4.3 Joint set number (J_n)

Shape and size of the blocks in a rock mass depend on the joint geometry. Joints within a joint set will be nearly parallel to one another and will display a characteristic joint spacing. Joints that do not occur systematically or that have a spacing of several meters are called random joints. However, the effect of spacing strongly depends on the span or height of the underground opening. If more than one joint belonging to a joint set appears in the underground opening, it has an effect on the stability and should be regarded as a joint set. The number of J_n is not the same as the number of joint sets.

Table 2 J_n – values.

2	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 portals, use 2 x J_n		

By studying the shape of the blocks in rock mass, the prominent joint directions will become apparent. Table 2 gives the parameter values for J_n according to the number of joint sets and random joints. In an underground opening the joints sets are often quite easy to identify. If the joint sets are difficult to identify directly, the orientation of a number of joints can be plotted in a stereo net, see Figure 2. The different joint directions will then occur as concentrations in the stereogram.

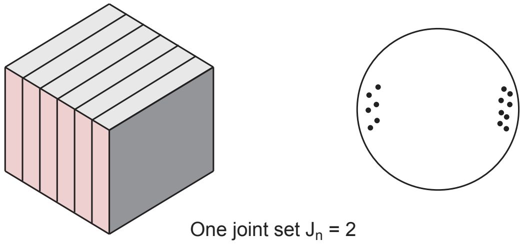
The definition of joint sets depends on the joint spacing between near parallel joints, but also on the span or height of the underground opening. If the joint spacing is generally greater than the span or height, the blocks formed by this specific joint set will usually be too large to fall out, and the joints should then be considered as random.

When calculating the J_n-value, it is very important to consider only the joints occurring at the same location and forming definite blocks. In situations where the J_n-value is determined from joint observations in a longer section of an underground excavation, summing up all the joint sets results in a J_n-value that is too high.

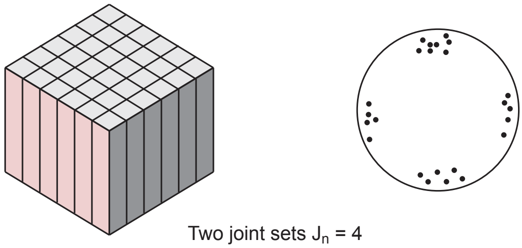
4.3.1 J_n in relation to joint length

The length of the joints does not directly affect the Q-values, but has importance for stability of the opening. Joints with considerable length, which intersect the whole cross section of an opening, will usually be more important for stability than shorter joints. Very short joints, often termed cracks, may have some importance locally, i.e. stability of small blocks and must therefore be considered. When short joints in general do not take part in the formation of blocks, they can be considered as random even if they occur rather systematically. If they take part in the formation of blocks, they must be considered as a joint set in the particular location in which they occur.

In some cases it is necessary not only to consider the number of joint directions, but also the shape of the blocks and the possibility of block fall. One example is columnar basalt which occurs among other places in Iceland, see Figure 2. The columns are usually hexagonal in shape, formed by joints in three different joint directions. The only direction fall-out can occur, is along the axes of the columns. As long as no joints occur across these axes, normally no blocks will fall. Even if there are three apparent joint directions shaping the columns, it is not correct to consider these as three joint sets, i.e. J_n should not usually be 9. A J_n-value of 4 is more reasonable since the column may be considered to be formed by two highly undulating sets of joints. Empirical data from underground openings in this type of rock often show good stability. Similar conditions can be encountered in tightly folded schists where sliding is prevented in certain directions.



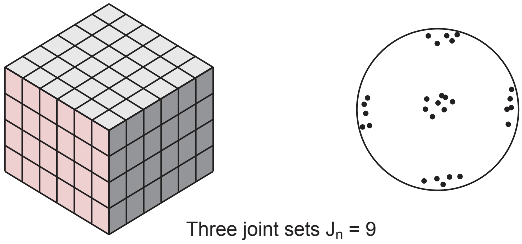
One joint set $J_n = 2$



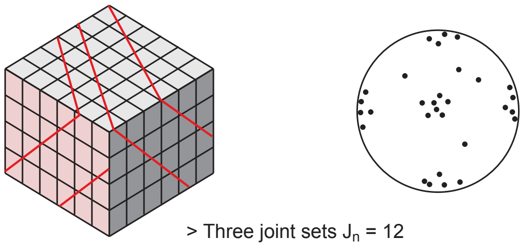
Two joint sets $J_n = 4$

Note:

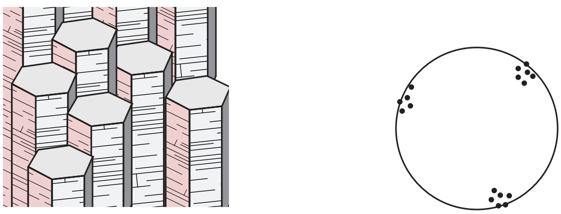
The number of joint directions is not always the same as the number of joint sets



Three joint sets $J_n = 9$



> Three joint sets $J_n = 12$



Columnar jointing with three joint directions, but $J_n = 4$

Figure 2 Different joint patterns shown as block diagrams and in stereonets.

4.4 Degree of jointing (RQD/J_n)

The fraction RQD/J_n represents the relative block size in the rock masses. In addition to RQD and J_n it is also useful to make notes of the real size and shape of the blocks, and the joint frequency.

4.5 Joint roughness number (J_r)

Joint friction depends on the nature of the joint wall surfaces, if they are undulating, planar, rough or smooth. The joint roughness number describes these conditions and is estimated from Table 3, or Figure 3. The joint description is based on roughness in two scales:

1) The terms rough, smooth and slickenside refer to small structures in a scale of centimetres or millimetres. This can be evaluated by running a finger along the joint wall; small scale roughness will then be felt.

2) Large scale roughness is measured on a dm to m scale and is measured by laying a 1 m long ruler on the joint surface to determine the large scale roughness amplitude. The terms stepped, undulating and planar are used for large scale roughness. The large scale roughness must be considered in relation to the block size and also to the probable direction of sliding.

All joint sets at a location must be evaluated with regards to J_r. When calculating the Q-value, the J_r-value for the most unfavourable joint set concerning stability of the excavation must be used, i.e., use J_r of the joint set where shear is most likely to occur.

4.5.1 J_r in relation to joint infill

When determining the joint roughness number, joint infill must also be considered. If the joints have an infill consisting of a soft mineral or crushed rock material that prevents rock wall contact during shear deformation (category “c” in Table 3), the roughness is no longer significant. The properties of the mineral infill will then be decisive for friction and $J_r = 1$ is used in these cases. If the infill is so thin that rock wall contact will occur before 10 cm of shear deformation (category “b” in Table 3) the same joint roughness number as for joints without infill is used (category “a” in Table 3).

The thickness of the joint filling necessary to prevent rock wall contact during shear deformation is dependent on the roughness. For undulating, rough joints a thicker infill will be necessary than for planar, smooth joints, please see Figure 4 for illustration.

Table 3 J_r – values.

3	Joint Roughness Number	J_r
a) *Rock-wall contact, and* ***b)*** *Rock-wall contact before 10 cm of shear movement*		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough, irregular, planar	1.5
F	Smooth, planar	1
G	Slickensided, planar	0.5
Note: i) Description refers to small scale features and intermediate scale features, in that order		
c) *No rock-wall contact when sheared*		
H	Zone containing clay minerals thick enough to prevent rock-wall contact when sheared	1
Note: ii) Add 1 if the mean spacing of the relevant joint set is greater than 3 m (dependent on the size of the underground opening) iii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented in the estimated sliding direction		

4.5.2 J_r in relation to joint planarity

In determining the joint roughness number, planarity must be related to block size. When the size of blocks is less than the wavelength of the undulations on the joint face, the undulations will be of less significance for the stability of the blocks. When determining J_r, the joint planarity should therefore be evaluated in profiles of the same order of magnitude as the size of the blocks.

4.5.3 J_r in relation to joint orientation

The roughness structure on a joint surface may often have an orientation so that a joint may appear planar in one direction and undulating in another. In such cases the joint roughness number must be derived along the direction where shearing or sliding is most likely to occur. This may especially be the case for joints with marked linear structures (slickensides) which may be smooth lengthwise and rough crosswise, or vice-versa.

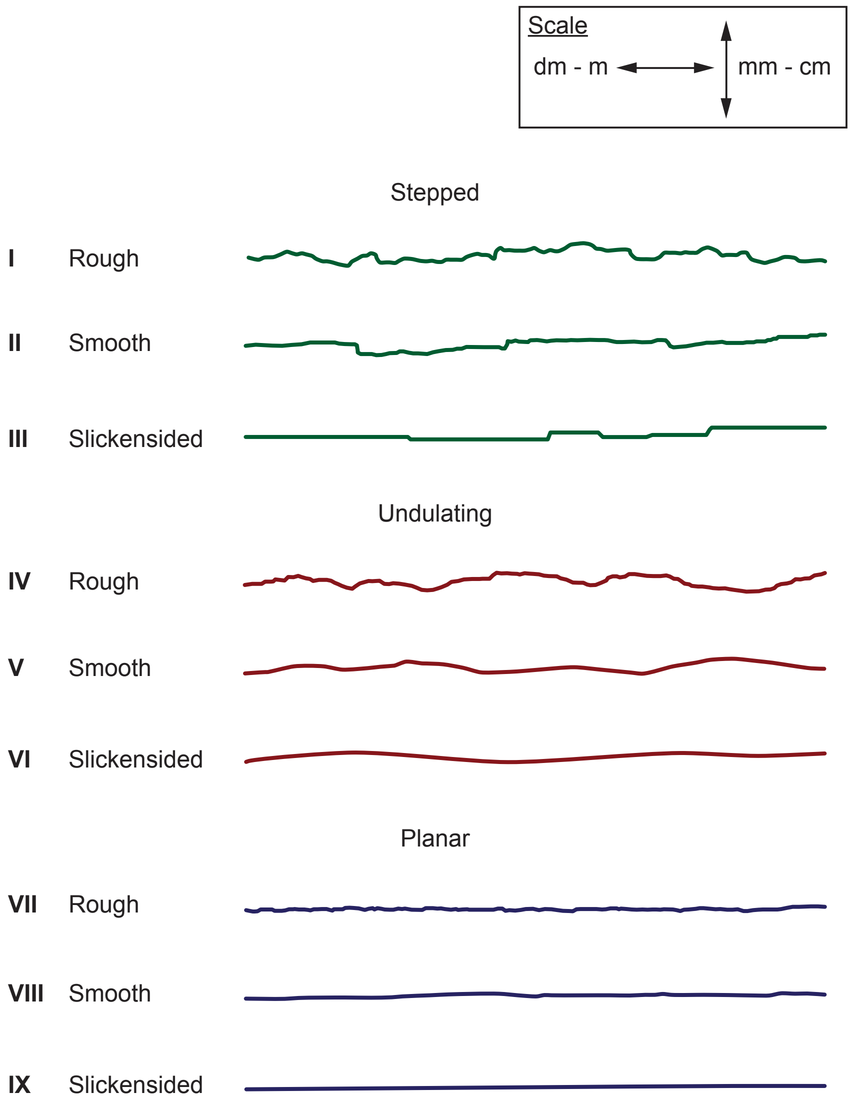


Figure 3 Examples of joint wall surfaces with different J_r - values. The length of each profile is in the range: 1 - 10 m. The vertical and horisontal scales are equal. (Modified from ISRM 1978)

4.5.4 J_r in rock masses without joints

When the deformation in the rock mass is dependent on the joints, J_r should be given values according to Table 3. Some rock masses may be almost without joints, and in hard rock J_r usually is given the value 4. For soft rocks without joints the J_r-value 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 soft rocks stronger than soil and without joints, the J_r -value may be irrelevant, and the deformation of the material may depend on the relation between strength and stress. The factor SRF is most relevant to describe this situation.

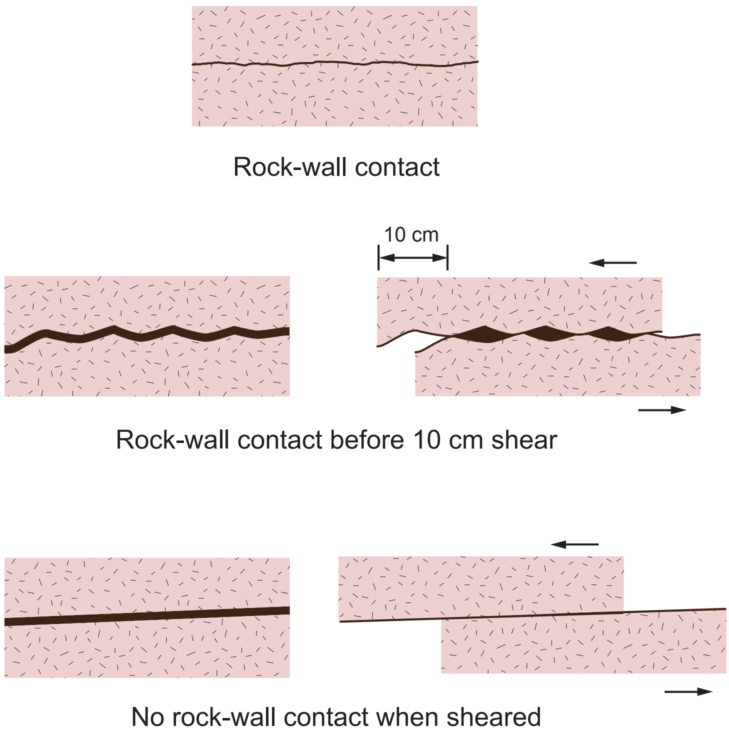


Figure 4 Joints with and without rock-wall contact.

In the cases when the weakness zones or joint fill is thick enough to prevent rock wall contact during shearing J_r is always 1. If only a couple of joints in the relevant joint set are exposed in the underground opening at a certain point, J_r +1 should be used.

Few cases are recorded from very soft rock, and the Q–values from such rock types should be handled with care and combined with numerical simulations and convergence measurements.

4.6 Joint alteration number (J_a)

In addition to the joint roughness the joint infill is significant for joint friction. When considering joint infill, two factors are important; thickness and strength. These factors depend on the mineral composition. In the determination of a joint alteration number, the joint infill is divided into three categories (“a”, “b” and “c”) based on thickness and degree of rock wall contact when sheared along the joint plane, please see Figure 4 for illustration and Table 4 for detailed description.

Table 4 J_a – values.

4	Joint Alteration Number	Φ_r approx.	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 J_a 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 roughness and thickness of the infill. For smooth joints, one millimetre of filling could be enough to prevent rock wall contact. However, for rough and undulating joints, several millimetres, or 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 fillings according to Table 4.

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

4.6.1 J_a in relation to type of mineral in a joint filling

The type of mineral and its characteristics are decisive for derivation of 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. Since only small quantities of water are necessary to cause swelling in some clays, a high J_a-value is usually given independent of the water situation where swelling clay is abundant.

The J_a-value depends on the type of clay minerals in the joint filling material. Swelling clay is unfavourable for stability. An analysis of the clay infill may therefore be necessary. Analyses can be carried out by using relatively simple laboratory tests or X-ray diffraction. When swelling clay is identified, swelling pressure tests will give valuable information. The swelling pressure measured in the laboratory should not be directly used when dimensioning the rock support since the rock mass, depending on the bearing capacity of the rock support, will take up a considerable part of the pressure. In addition the swelling clay is usually mixed with other minerals and rock fragments. Experiences from laboratory testing of extracted clay samples in Norway has indicated a swelling pressure up to five times the swelling pressure in the undisturbed weakness zone.

4.7 Joint friction (J_r/J_a)

The function tan-1(J_r/J_a) is a fair approximation of the actual friction angle that one might expect for the various combinations of wall roughness and joint infill materials (Barton et al. 1974). It is expected that rough, undulating and unaltered joints with joint wall contact (J_a-category "a"), will dilate strongly when sheared, and therefore they will be especially favourable for excavation stability. When rock joints have thin clay mineral coatings and fillings (J_a-category "b"), the shear strength is reduced significantly. Renewed rock wall contact after small shear displacement will be a very important factor for preserving the excavation from ultimate failure. If no rock wall contact appears during shearing (J_a-category "c") this will be very unfavourable for excavation stability.

The shear strength is also dependant of the effective stress, which is influenced by the presence of water and water pressure. The joint alteration number, J_a, is however not influenced by the presence of water.

4.8 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 5. The lowest J_w-values (J_w< 0.2) represent large stability problems.

Table 5 J_w – values.

5	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 crude estimates. Increase J_w if the rock is drained or grouting is carried out ii) Special problems caused by ice formation are not considered		

4.8.1 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 a 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 just after the excavation, but inflow will develop over time. In other cases large inflow just after excavation may decrease after some time.

When distinguishing between the J_w–values 1 and 0.66 the following can be applied:

- $J_w = 1$ for single drops of water dripping in a limited area of the excavation.
- $J_w = 0.66$ for a trickle or small jets of water in a concentrated area, or frequent dripping in a wide area.
- If a concentrated jet of water is coming out of a drill hole, $J_w = 0.66$.

4.9 Stress Reduction Factor (SRF)

In general, SRF describes the relation between stress and rock strength around an underground opening. The effects of stresses can usually be observed in an underground opening as spalling, slabbing, deformation, squeezing, dilatancy and block release. However, some time may pass before the stress phenomena are visible.

Both stresses in and strength of the rock mass can be measured, and SRF can then be calculated from the relation between the rock uniaxial compressive strength (σ_c) and the major principal stress (σ_1) or the relation between the maximum tangential stress σ_θ and σ_c in massive rock. During the planning phase of an underground excavation, SRF can be stipulated from the overburden and topographic features or general experiences from the same geological and geographical region.

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.

In order to determine SRF, the category must be determined first before the parameter value can be determined from the description given in Table 6.

The stress situation is classified in four categories in accordance with Table 6:

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) Squeezing rock with plastic deformation of incompetent rock under the influence of moderate or high rock stresses.

d) Swelling rock; chemical swelling activity depending on the presence of water.

Table 6 SRF-values.

	6 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 6L and 6M			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, mainly massive rock, stress problems*	σ_c / σ_1	σ_θ / σ_c	**SRF**
F	Low stress, near surface, open joints	>200	<0.01	2.5
G	Medium stress, favourable stress condition	200-10	0.01-0.3	1
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:	ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1 / \sigma_3 \leq 10$, 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) iii) 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)	*Squeezing rock: plastic deformation in incompetent rock under the influence of high pressure*		σ_θ / σ_c	**SRF**
M	Mild squeezing rock pressure		1-5	5-10
N	Heavy squeezing rock pressure		>5	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)			
d)	*Swelling rock: chemical swelling activity depending on the presence of water*			**SRF**
O	Mild swelling rock pressure			5-10
P	Heavy swelling rock pressure			10-15

4.9.1 SRF and weakness zones intersecting the underground opening (Case A to E in Table 6a)

A weakness zone is a zone that is intensively jointed or chemically altered and therefore substantially weaker than the surrounding rock. The width of a weakness zone varies from one decimetre to hundreds of metres in extreme cases. The most common types of weakness zones are:

- Shear zones, i.e. fault zones where the rock mass is intensively jointed, foliated or crushed to small pieces and may also contain clay.

- Clay zones with disintegrated and altered rock or weak mineral layers without shearing.

A narrow weakness zone may be defined as a zone of width from a decimetre and up to about 2-3 m, i.e. the width is generally much smaller than the span of the underground opening. A broad weakness zone is defined to be wider than about 2-3 m. In a narrow weakness zone the rock support in general can be anchored in the side rock of better quality. For a broad zone the rock support must be designed as an individual support without taking the quality of the side rock into account.

Surrounding a weakness zone an anomalous stress situation may occur locally, and an increased SRF-value may then be necessary to describe the stability situation. If the weakness zone is so weak that the stresses cannot be transferred through it, a stress concentration may occur on one side of the zone and de-stressing 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 several weakness zones with a spacing of a few metres, a longer section of an excavation may be affected and should be given an increased SRF-value. In cases where a long section of the excavation intersects multiple weakness zones with crushed or weathered rock, an assessment can be made whether the term “weakness zone” should be used for the whole section. In such cases, category A, B or D in Table 6a should be used. If squeezing rock conditions occur, use M or N in Table 6c. If swelling rock conditions occur, use O or P in Table 6d.

To detect if the rocks are de-stressed or not the rock mass may be struck with a hammer or a scaling rod. If a hollow sound is elicited by the blow and small blocks easily can be loosened, the rock can be considered de-stressed and a SRF-value larger than 1 may be determined. Please note that a hollow sound may be elicited also if a single local loose block is hit.

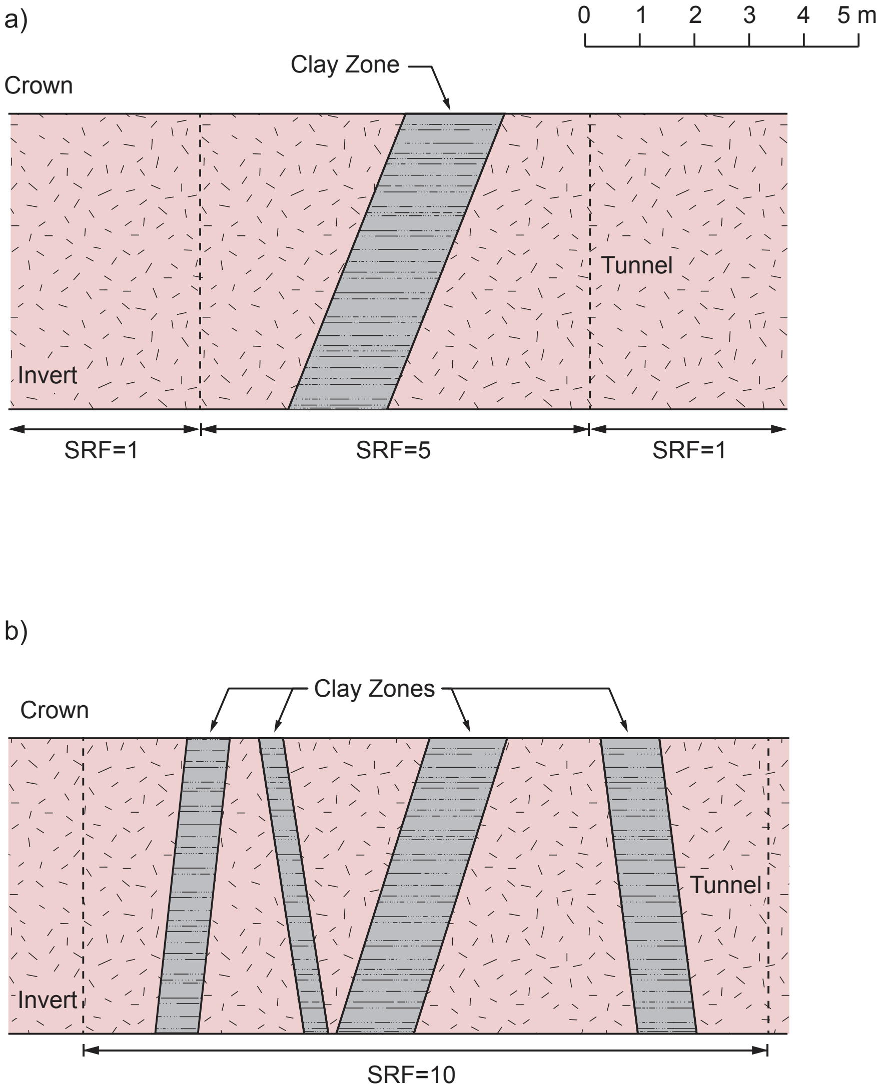


Figure 5 SRF-values related to single and multiple weakness zones.

A visualisation of weakness zones is given in Figure 5. In Figure 5a, an underground opening is intersected by a clay filled zone. In the vicinity of this zone there is normally an anomalous stress situation. A SRF-value of 5 has to be used for an area consisting of a weakness zone and its immediate surroundings. The width of the area to be given a SRF value of 5 will depend on the quality of the rock masses outside the weakness zone. In Figure 5b, several clay filled zones intersect the underground opening, and a SRF-value of 10 has to be used for this particular section, (see reference 5.6 on page 39).

4.9.2 SRF in competent rocks, rock stress problems (Case F to L in Table 6b)

The relation between the rock strength and stress is decisive for the SRF-value in category "b". Moderate stresses will generally be most favourable for the stability, and SRF will then be 1. Moderately high horizontal stresses may be favourable for the crown of caverns, and a SRF value of 0.5 may be used in some cases.

Low stresses, which will often be the case when the underground excavation has a small overburden, may result in reduced stability due to dilation. SRF will in such cases be 2.5 or even 5.0 when the span of the underground opening is larger than the rock overburden. Low stresses causing instability may also appear when excavating near other underground openings.

Spalling and rock burst may occur at very high stresses and SRF-values up to 400 may be used in some extreme situations. The intensity of the stress problems and how soon after excavation the stability problems start will be decisive for the SRF-value. The intensity of the spalling and the time between excavation and occurrence of spalling has to be considered for estimating the SRF. Case J in category "b" (Competent, mainly massive rock, stress problems) in Table 6, describes moderate stress problems more than one hour after excavation. If the problems start approximately one hour after excavation, a SRF-value of 20–50 should be used depending on the intensity of the spalling. If it takes many hours or some days before rock slabs loosen, the SRF-value may be 5–10. Similar time relations apply to case K: If problems with intense spalling and/or rock burst occur immediately after excavation the SRF-value will be approximately 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 in spite of 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.

High stresses which result in immediate spalling and rock burst, normally also result in a long term deformation of the rock mass, development of new cracks deep into the rock mass for some time until the new stability is achieved. The depth of spalling depends on the intensity of spalling and on the span of the underground opening. An anisotropic stress situation will be unfavourable if the stresses are high, and particular sectors of the circumference will often be exposed to stress induced stability problems. This often generates an asymmetric excavation periphery, and is increasing with increasing stress.

In competent and relative massive rock the SRF-value may be estimated when the ratio σ_c/σ_1 or σ_θ/σ_c is known. According to observations this is valid only for $RQD/J_n >> 10$.

In many cases the rock stress is induced by high valley sides giving high principle stresses, high tangential stress and anisotropic stresses, as illustrated in Figure 6. The height of the mountainside above the excavation level compared with the compressive strength of the rock may be a good tool for estimating the SRF.

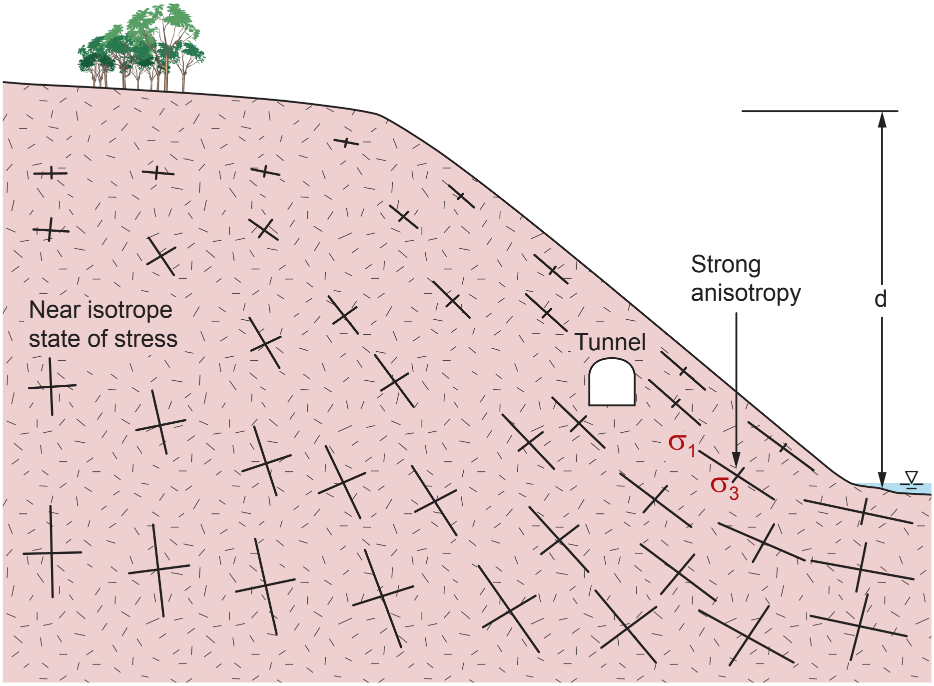


Figure 6 Visualization of a high valley side with high anisotropic stresses.

Because the stresses normally are not transferred through a jointed rock mass when released by excavation, the effect of stresses will vary. The Q-value will to a large degree be influenced by RQD and J_n.

In cases where high stresses are combined with jointed rock, the rock mass compressive strength is more important than the compressive strength of intact rock. In cases where the rock mass is heavily jointed and under high stresses, a squeezing effect is more likely to occur than spalling, and Table 6c should be used instead of 6b.

4.9.3 SRF inn squeezing rock (Case M and N in Table 6c)

"Squeezing rock" means rock masses where plastic deformation takes place under the influence of high rock stresses. This will happen in soft rocks or crushed rock when the stresses exceed the rock mass strength.

In very soft rocks with few or no joints, the stability will depend on the relation between the rock's compressive strength and the stresses, and the other Q-parameters may be difficult to decide. In such cases, the Q-system may not give any satisfactory description of the stability situation, and other methods such as deformation measurements and/or numerical modelling for support design may be used additionally.

4.9.4 SRF in swelling rock (Case O and P in the Table 6d)

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 readily 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.

4.10 Q-parameters related to pre-grouting

Investigation of the relationship between pre-grouting and improvement of rock mass properties has been carried out by Barton. A hypothetical model of potential improvements in Q-parameters due to pre-grouting results in: an increase in the effective RQD, a reduction in effective J_n, an increase in J_r, a reduction in J_a and an increase in J_w. Please see Barton (2006) for further details.

5 Using the Q-system to evaluate the support requirements

Q-value and the six appurtenant parameter values give a description of the rock mass. Based on documented case histories a relation between the Q-value and the permanent support is deducted, and can be used as a guide for the design of support in new underground projects.

5.1 Excavation Support Ratio (ESR)

In addition to the rock mass quality (the Q-value) two other factors are decisive for the support design in underground openings and caverns. These factors are the safety requirements and the dimensions, i.e., the span or height of the underground opening. Generally there will be an increasing need for support with increasing span and increasing wall height. Safety requirements will depend on the use (purpose) of the excavation. A road tunnel or an underground power house will need a higher level of safety than a water tunnel or a temporary excavation in a mine. To express safety requirements, a factor called ESR (Excavation Support Ratio) is used.

A low ESR value indicates the need for a high level of safety while higher ESR values indicate that a lower level of safety will be acceptable. Requirements and building traditions in each country may lead to other ESR-values than those given in Table 7.

It is recommended to use ESR = 1.0 when Q ≤0.1 for the types of excavation B, C and D. The reason for that is that the stability problems may be severe with such low Q-values, perhaps with risk for cave-in.

In addition to the span (or wall height) ESR gives the “Equivalent dimension” in the following way:

$$\frac{\text{Span or height in m}}{\text{ESR}} = \text{Equivalent dimension}$$

Table 7 ESR-values.

7	Type of excavation	ESR
A	Temporary mine openings, etc.	*ca.* 3-5
B	Vertical shafts*: i) circular sections ii) rectangular/square section * Dependant of purpose. May be lower than given values.	*ca.* 2.5 *ca.* 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

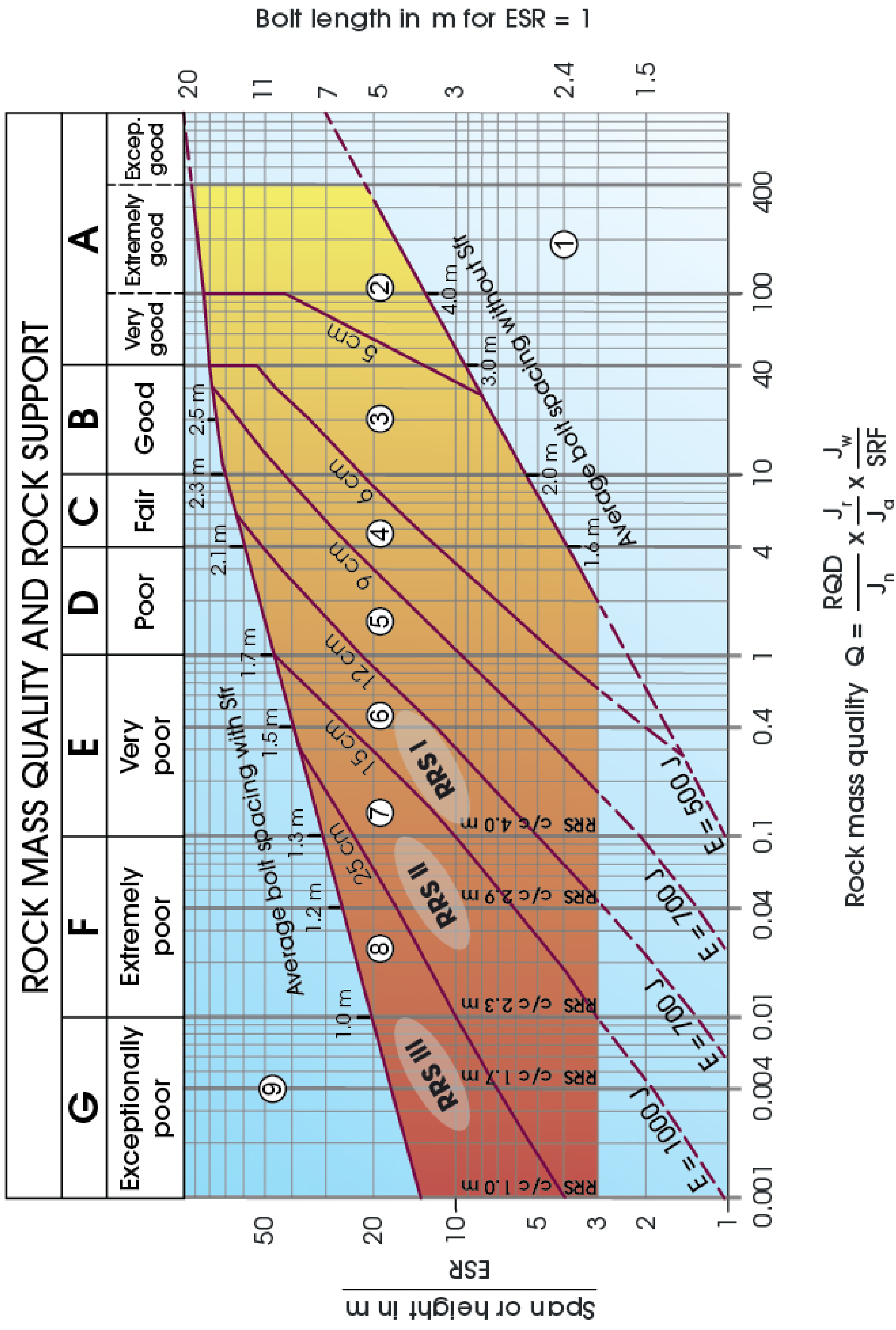
5.2 Rock support chart

The Q-value and the Equivalent dimension will be decisive for the permanent support design. In the support chart shown in Figure 7, the Q-values are plotted along the horizontal axis and the Equivalent dimension along the vertical axis on the left hand side.

The support chart gives an average of the empirical data from examined cases. In some cases the rock support represents a conservative magnitude of support, while in other cases cave in occurred during construction or years later, when the underground excavations were in service. For a given combination of Q-value and Equivalent dimension, a given type of support has been used and the support chart has been divided into areas according to type of 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. As the support chart is based on empirical data, it is able to function as a guideline for the design of permanent support in underground openings and caverns.

The support chart indicates what type of support is used in terms of the centre to centre spacing for rock bolts and the thickness of sprayed concrete. It also indicates the energy absorption of the fibre reinforced sprayed concrete, as well as the bolt length and design of reinforced ribs of sprayed concrete. Support recommendations given in the chart are general and in certain especially difficult cases, an increase in the amount or type of support may be relevant.


ROCK MASS QUALITY AND ROCK SUPPORT
Bolt length in m for ESR = 1
Span or height in m / ESR
Rock mass quality $Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$
G Exceptionally poor
F Extremely poor
E Very poor
D Poor
C Fair
B Good
A Very good
A Extremely good
A Excep. good
Average bolt spacing with Sfr
Average bolt spacing without Sfr
RRS I
RRS II
RRS III
RRS c/c 4.0 m
RRS c/c 2.9 m
RRS c/c 2.3 m
RRS c/c 1.7 m
RRS c/c 1.0 m
E = 500 J
E = 700 J
E = 1000 J
5 cm
6 cm
9 cm
12 cm
15 cm
25 cm

Support categories

① Unsupported or spot bolting
② Spot bolting, **SB**
③ Systematic bolting, fibre reinforced sprayed concrete, 5-6 cm, **B**+**Sfr**
④ Fibre reinforced sprayed concrete and bolting, 6-9 cm, **Sfr (E500)**+**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 (E700)**+**RRS I** +**B**
⑦ Fibre reinforced sprayed concrete >15 cm + reinforced ribs of sprayed concrete and bolting, **Sfr (E1000)**+**RRS II**+**B**
⑧ Cast concrete lining, **CCA** or **Sfr (E1000)**+**RRS III**+**B**
⑨ Special evaluation

Bolts spacing is mainly based on Ø20 mm

E = Energy absorbtion in fibre reinforced sprayed concrete

ESR = Excavation Support Ratio

Areas with dashed lines have no empirical data

RRS - spacing related to Q-value

I
Si30/6 Ø16 - Ø20 (span 10m)
D40/6+2 Ø16-20 (span 20m)

II
Si35/6 Ø16-20 (span 5m)
D45/6+2 Ø16-20 (span 10m)
D55/6+4 Ø20 (span 20m)

III
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 = RSS spacing, centre - centre

Figure 7 Permanent support recommendations based on Q-values and span/ESR.

The thickness of the sprayed concrete increases towards decreasing Q-value and increasing span, and lines are drawn in the support chart indicating thicknesses. For positions between these lines the thicknesses will have an intermediate value. If deformation occurs, for instance caused by high stresses, reinforced concrete should be used in all categories.

Sometimes alternative methods of support are given. At high Q-values in the support chart, sprayed concrete may or may not be used. The mean bolt spacing in such cases will be dependent upon whether or not sprayed concrete is used. Due to this, the support chart is divided into two areas. The area defined as “Bolt spacing in fibre reinforced sprayed concrete” refers to bolting in combination with sprayed concrete. The area defined as “Bolt spacing in areas without sprayed concrete” indicates bolt spacing when sprayed concrete is not used. Recommended bolt spacing is more an expression of the quantity of bolts necessary rather than an exact recommendation for the spacing. The position and direction of each bolt should be based on an evaluation of the joint geometry. This is especially important in areas where the bolt spacing is large. In areas where sprayed concrete is not used, systematic bolting is not relevant, and there should always be an evaluation for the position for each bolt.

The length of the bolts depends on the span or wall height of the underground opening and to some degree on the rock mass quality. Recommendations for bolt lengths are given on the right hand side of the diagram, but some evaluation is necessary. In unfavourable joint geometry, longer bolts than recommended in the diagram will be necessary, and there is also a general need for increasing bolt length by decreasing Q-value.

5.2.1 Sprayed concrete at high Q-values

The application of sprayed concrete has increased substantially during the years. Support categories in the chart that do not include sprayed concrete have been extended to include sprayed concrete in public underground openings due to minimum required operating time and temporary safety requirements during construction.

5.2.2 Wall support

The support chart is primarily valid for the crown and springlines of underground openings and caverns. The level of support on the walls is normally less for high and intermediate Q-values (Q>0.1). When the Q-system is used for wall support, the height of the walls must be used instead of the span. The actual Q-value is adjusted as shown in Table 8.

Table 8 Conversion from actual Q-values to adjusted Q-values for design 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	0.1 < 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 < 0.1	Use actual Q-value.

The value obtained after this conversion is then used with the chart in Figure 7 to determine appropriate wall support.

5.3 Reinforced ribs of sprayed concrete (RRS)

In sections with very poor rock mass quality (Q<1), reinforced ribs of sprayed concrete (RRS) in many cases is a preferred alternative to cast concrete. 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, see Figure 8. When using steel bars of 20 mm diameter, the bars have to be pre shaped in order to gain a smooth profile. The thickness of the ribs, the spacing between them as well as the number of ribs and diameter of the steel bars has to vary according to the dimension of the underground opening and the rock mass quality.

By monitoring and observation of RRS performance in recent case histories, there is a wide experience of RRS used in different rock mass conditions. A guideline for use of RRS in relation to Q-values and equivalent dimension of the underground opening is given in the support chart in Figure 7.

In the legend belonging to the support chart, the following notations are used:

- "Si" means single layer of steel bars
- "D" means double layers of steel bars
- "45" means a total rib thickness of 45 cm
- "6" means 6 steel bars
- "c/c = 2-3" means a centre to centre spacing of 2 to 3 metres between the ribs
- "16" or "20" gives the diameter of the steel bar in mm

In the support chart, the same dimensions for the ribs can be followed from the lower left area to the upper right area in the chart. Within each area, there will be an interval where the suggested spacing between the ribs will vary. An engineering geological evaluation has to be conducted for each case to decide the spacing between the ribs.

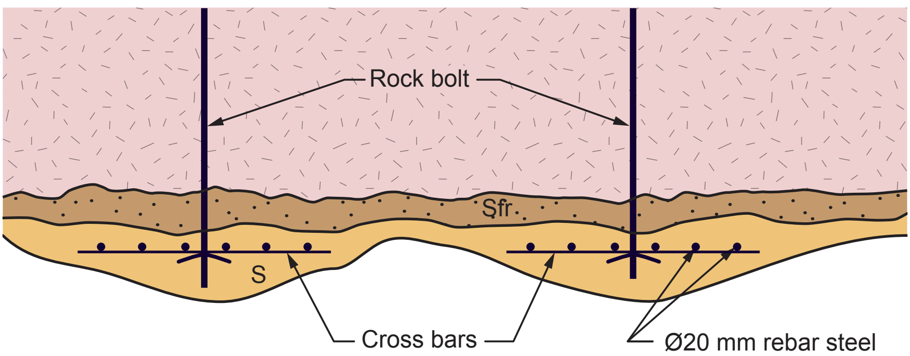


Figure 8 Principle of construction of RRS.

In cases where the Q-values indicates use of RRS, a 12-15 cm thick layer of fibre reinforced sprayed concrete normally has to be applied before installing the reinforced ribs. This layer has a function as temporary support as well as to smooth out the rock surface. The thickness of this layer is included in the total thickness of the RRS.

5.4 Forepoling

In poor rock mass qualities it may be necessary to forepole, i.e. by installation of spiling bolts in front of the working face in order to avoid overbreak or cave in (Figure 9). Forepoling is not directly included in the support chart related to the Q-system. However, the suggested rock support in rockmasses with low Q-values is based on the use of forepoling during excavation. The need for forepoling is dependent on the span of the underground opening and the rock mass quality. Normally, forepoling is used in combination with reduced excavation length of the blast rounds and/or excavation of only a part of the cross section of the excavation. Generally, it is suggested to use forepoling in rock masses with Q-values lower than 0.1–0.6, depending on the span of the underground opening. The spacing between the forepoles is normally around 0.3 m (0.2 - 0.6 m). The rear end of the forepoling has to be anchored in the overlying rock connected to radial rock bolts combined with steel straps or rebar steel and sprayed concrete in order to avoid collapse during excavation.

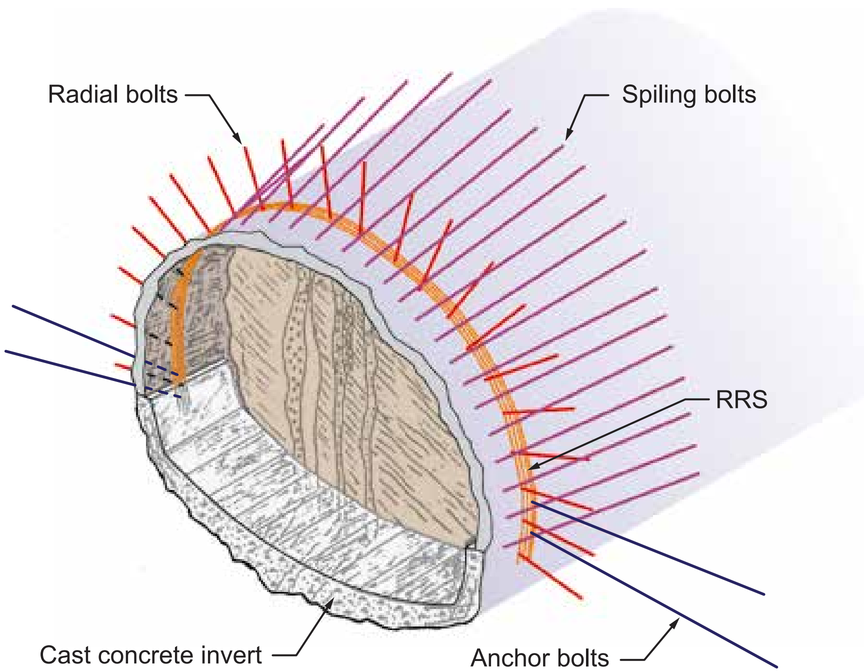


Figure 9 Support of poor rock masses by spiling bolts.

5.5 Energy absorption of sprayed concrete

Based on the expected deformations for different rock mass qualities, the energy absorption classes have been included in the support chart. These energy absorption classes in the support chart correspond with the energy absorption classes defined by EFNARC and are given in the guidelines of Norwegian Concrete Association's Publication no. 7–2011, see Table 9.

The support chart shows that the variation in rock mass quality Q and the span or height has almost an equivalent influence on the rock support and the energy absorption classes which follow the rock support or reinforcement categories.

Macro type synthetic fibre has been used instead of steel fibre at many construction sites in some countries since the beginning of 2000. These fibres give the sprayed concrete similar properties as with the steel fibres. The macro synthetic fibres are slightly more ductile than steel fibres, and to some extent elastic. Their great advantage is that they do not corrode. This may be a great advantage in corrosive environments as for example subsea tunnels, particularly if cracks are developed in the sprayed concrete.

Table 9 Energy absorption classes based on the panel test as described in Norwegian Concrete Association Publication no. 7 (NB,2011).

Energy Absorption Class	Min. energy absorption in Joule
E500	500
E700	700
E1000	1000

5.6 Support of weak narrow zones

Two factors are decisive for support in weakness zones; the width of the zone and the direction of the zone related to the excavation axis. A weakness zone can be defined as a zone requiring more support than the surrounding rock. Zones more than 2-3 m wide will usually require their own specific rock support, while narrow zones, i.e. 0.5-3 m wide, will usually be supported by anchoring the support to the surrounding rock. The quality of the surrounding rock will determine the necessary rock support.

For narrow weakness zones it is usually not convenient to design the support based only on the Q-value of the zone itself. In such cases, the support structure will usually include about 1 m on either side of the zone. In other words, for a weakness zone 1 m wide, the supported area will be about 3 m wide. The basis of which the support is decided will be the mean Q-value for the 3 m wide zone.

By using the mean values for the different Q-parameters in the supported zone, the mean Q-value can be calculated. To give an exact description of the site, it will be necessary to decide the Q-value both for the weakness zone and for the surrounding rock. Since the Q-values are related to a logarithmic scale, the calculation must be carried out logarithmically. The following formula can be used (Løset, 1997):

$$\mathrm{Log}Q_m = \frac{b \cdot \log Q_{zone} + \log Q_{sr}}{b + 1}$$

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

Please note that in cases where the surrounding rock has a very high Q-value, the formula may give too high Q_m. Whether or not the side rock can be used to anchor the support construction is decisive. A lower limit for the surrounding rock may be well inside support category 3 or better.

The width of a weakness zone is generally measured perpendicularly to the zone's strike direction, but in case of tunnel support it is necessary to consider the angle between the axis of the tunnel and the strike direction of the weakness zone. The more acute the angle is between the zone and the underground opening axis, the larger the affected section of the excavation will be. The width of the zone (measured as the length along the excavation affected by the zone) should therefore be used in the formula for Q_m. Furthermore, please notice that in case of a rather narrow zone (i.e. $b \approx 0.5$ m) parallel to the excavation axis, this equation will give $Q_m \approx Q_{zone}$, which may result in a mean Q-value (Q_m), which is too low.

5.7 Additional comments on stability and rock support

A Q-value gives description and classification of a rock mass, and by using the support chart in Figure 7, one can design the general support methods and quantities needed for a particular Q-value. The Q-value and the support chart however, do not intercept every detail or all specific cases. The stability of single blocks is more or less independent of the Q-value. The specific rock support, i.e. location of single bolts is not taken into account by the Q-system. Faulty design of the rock support may lead to failure of single blocks even if the rock support is in accordance with the Q-system. When designing rock support it is therefore necessary to consider the joint geometry specifically. If the rock bolting is carried out before application of sprayed concrete it is possible to locate each individual block.

Some examples of unfavourable joint geometries that require special attention with regards to bolting are shown in Figure 10. 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 10a). 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 10b). 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.

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 10c). 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 heavily jointed rock (almost sugar cube jointing), the Q-value alone may be give the wrong basis for rock support because the small blocks without cohesion may give reduced stability in spite of a relatively high Q-value. This may be compensated by increasing the SRF-value (as for a weakness zone) and using $J_r = 1$ (because of lack of joint wall contact).

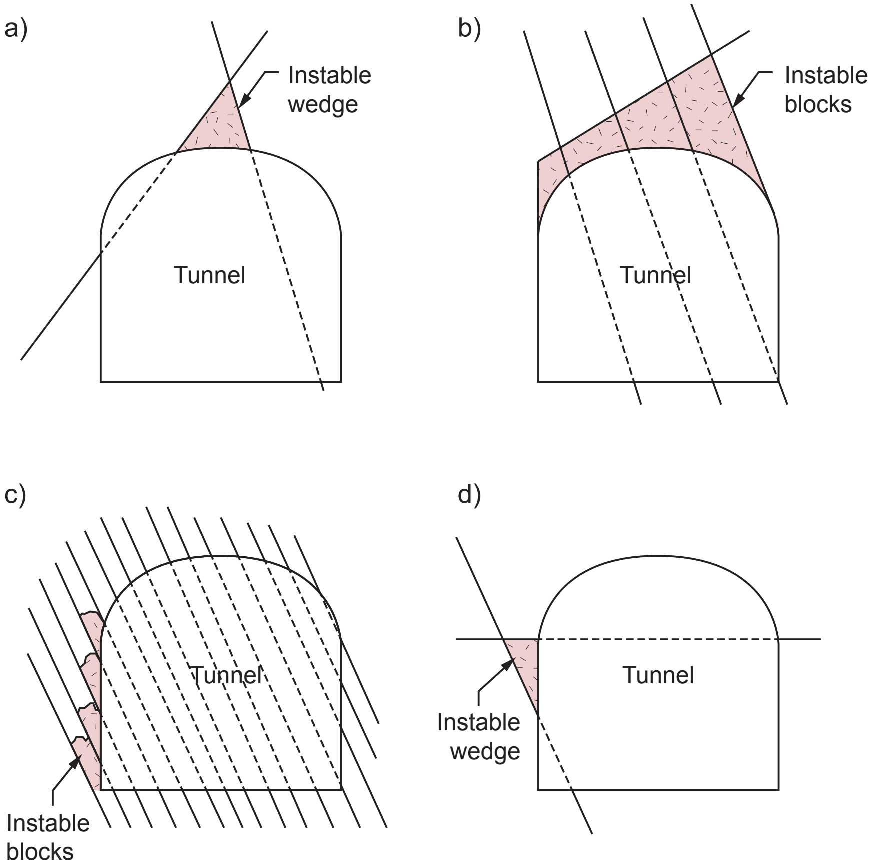


Figure 10 Stability problems caused by joints with unfavourable orientations.

6 Mapping in underground openings

6.1 General

Support design in underground opening is usually based on engineering geological mapping of the underground opening itself. When calculated, the Q-value will give a good indication of the necessary permanent rock support. However, the Q-system cannot be used to evaluate the stability of single blocks or wedges. The stability of a single block may often be more or less independent of the parameters in the Q-formula, and unstable single blocks may exist despite of a high Q-value.

The geological mapping must be carried out before sprayed concrete is applied or a cast concrete lining is installed. For many projects, engineering geologists are permanently on-site, and mapping after each excavation/blast round is then carried out. If geologists are not constantly available at the sight, it may be necessary to map longer sections of the underground opening at a time. In such cases, some sections of the underground opening may already be covered by sprayed concrete. When this is the case, the geologist may map the covered sections by studying the areas where the lining may be missing, for example near the invert. It is important in such cases to note that the description is only valid for the lower part of the wall and that the rock mass conditions in the crown may be different.

6.2 Engineering geological map

Based on the site observations a more general engineering geological map should always be made in addition to the Q-classification. During this mapping, the different rock types, structures and joint geometry should be described. Furthermore, all weakness zones should be registered and described with respect to their orientation, width and mineral content.

An example of tunnel mapping for an arbitrary tunnel section is shown in Figure 11. At the very top of the chart, there is a table for registered Q-parameters. The amount of grouting is also documented in the same table. Below the table, mapping of the geology and rock support is visualised by sketches where the walls are folded out and the tunnel crown is in the middle. The topmost sketch is used for geological description and the lower sketch is for the documentation of rock support.

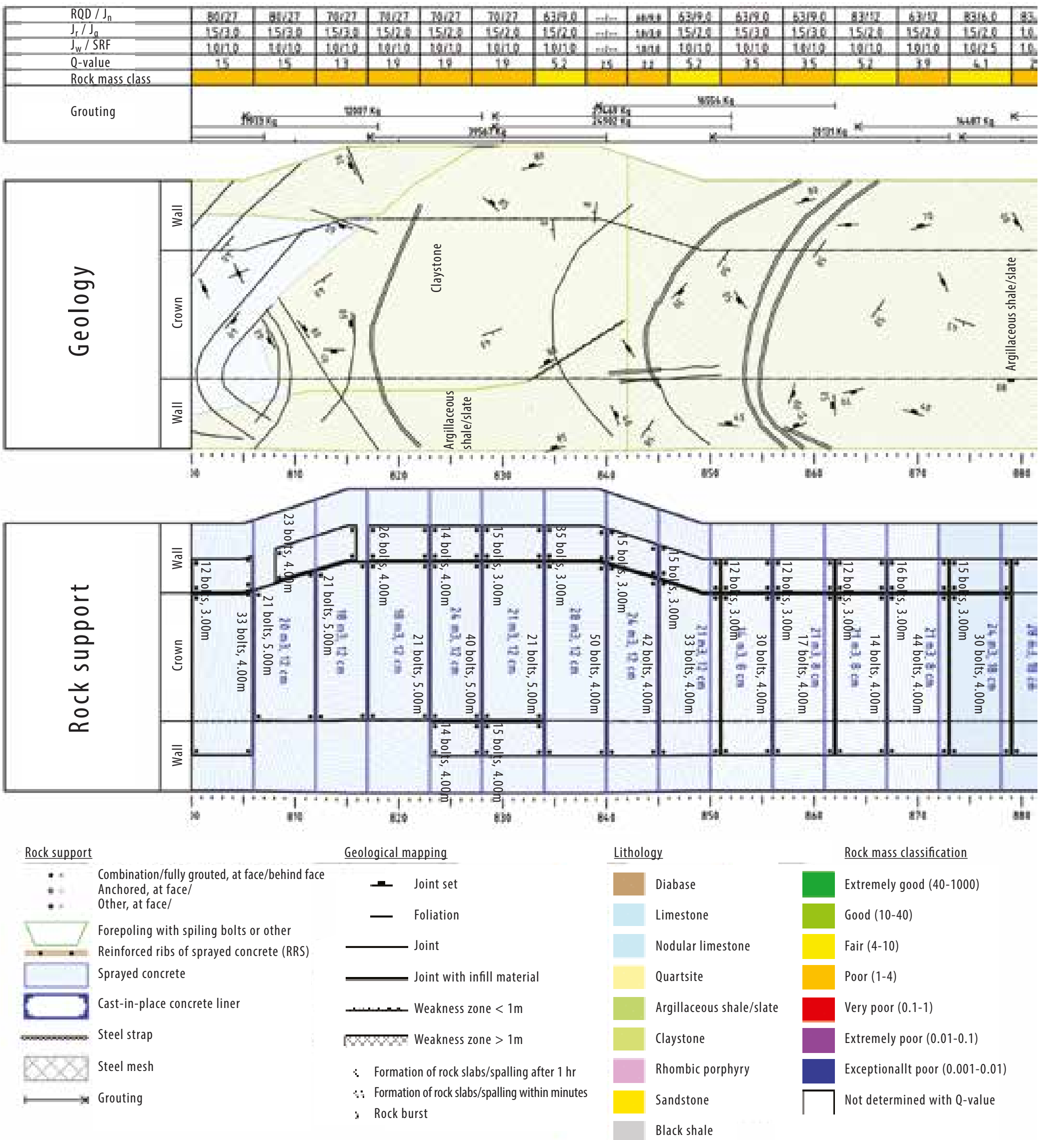


Figure 11 *Digital tunnel map with registration of the rock mass quality, geology and rock support with quantities of rock bolts and sprayed concrete separated, by using Autocad. Figure from Novapoint Tunnel.*

6.2.1 Mapping of sections

When calculating the Q-value, the underground opening must be divided into sections so that the Q-value within a section is relatively uniform. As a general rule, the variation in the Q-value for each section should not be larger than one rock class with reference to the support chart in Figure 7. There is a limit for how small a section should be when the rock mass quality varies within small areas. There is usually no point in making a separate section when the section is narrower than 2-3 metres.

When mapping each blast round, the area can usually be considered as one section. However, if there are considerable variations in the Q-value, division into smaller sections for the classification may be necessary. For longer sections of the underground opening, it may be practical to do a general geological mapping first and then divide the main section into subsections, and then determine the Q-value for each subsection.

When determining the Q-parameter values, it is usually necessary to have the parameter tables easily available. This handbook includes a laminated folder of the Q-parameters and support chart which can be used during field mapping. A description and a sketch of the main geological structures are also valuable. As mentioned earlier, the apparent values of J_w and SRF can change with time. It is therefore important to make notes for the duration of time that has gone by since the excavation of the underground opening. If there are joints with clay fillings, laboratory tests could be necessary to identify the clay minerals for estimating the J_a-value. Sampling will then be necessary.

6.3 Mapping in tunnels excavated by TBM

Mapping in a TBM-tunnel may be challenging. Compared with tunnels excavated by drill & blast, particularly for intermediate Q-values, fewer joints will be visible by TBM excavation. When the rock mass quality is intermediate to good (Q >1), the walls in a TBM tunnel will be quite smooth, and it will be difficult to identify joints and study the joint surfaces. A hammer may be helpful to distinguish real joints from veins, foliation etc. in order to estimate the RQD-value. Estimates of the J_r and J_a factors may be inaccurate if no or few joint surfaces can be studied. It is often useful to try pressing a thin blade into the joint in order to make an evaluation of the joint filling. Clay fillings may be discovered. 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 joints with clay fillings are found. Unstable wedges formed by more or less visible joints with unfavourable orientation could be left after excavation. These blocks may suddenly fail. Joint properties (J_r/J_a) give an indication of the friction angle along the joints, but may be difficult to observe. It is of great importance to carefully study the general geology and to observe joint orientations and joint properties.

7 The Q-system used during pre-investigations

7.1 General

The Q-method may also be used during pre-investigation for tunnels, caverns and for rock mechanical calculations. During planning of underground projects, it is important to make detailed descriptions of the rock mass in order to get the best possible design and reliable prognosis for rock support and costs. Attention should be given when using the Q-system during field mapping. Some of these aspects are highlighted in the following text:

It is important to have in mind that when the Q-value is calculated from data that does not come from an underground opening, the Q-value lacks some of its basic parameters like J_w and SRF and should not be applied as a basis for rock support without great caution.

7.2 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 vast and of good quality. Histograms may be used to visualise the variation in the different parameters (see Figure 1).

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. More jointed or weathered rock masses may be covered by soil.

At the surface, joint fillings 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 filling 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 estimated with a lower value in blasted cuts and slopes, compared to natural rock surfaces. In quarries with high excavated rock cuts in miscellaneous directions, the Q-value will be close to the value observed in an underground opening. The water condition in an underground opening, J_w, will be difficult to predict from field mapping only. Lugeon tests in boreholes and/or empirical data from projects in similar rock masses are necessary to obtain good predictions of the water conditions.

A prediction of the SRF-value may be made based on the topographic features and available information regarding the stress situation in the region. When an estimate of the SRF-value is made during the planning stage of an underground opening, general experience from the geological region may be valuable. Information from nearby underground excavations and topographic features may be helpful. In areas with high, steep mountain sides there is often an anisotropic stress field. Geological structures, such as fractures parallel to the surface and sickle shaped exfoliation, are indications of high, anisotropic stresses. The limit for exfoliation in high mountain sides or spalling in an underground opening is dependent on the relation between induced stress (height of slope above the underground opening) and the compressive strength of the rock. In Table 6b, the relation σ_c/σ_1 <4-5 (dependant on the anisotropy) is normally a limit for spalling in an underground opening. In hard rock this limit normally occur between 400 and 1100 m rock overburden in the valley side above the underground opening, dependent on the compressive strength of the intact rock and gradient of the mountain side (see Figure 6). Stress measurement may also be possible to carry out prior to excavation.

7.3 Use of the Q-system during core logging

Preinvestigations for underground excavations often include corelogging. Quite often, sections of the drill cores are missing and a poor rock quality must be predicted. Where cores exist, most of the Q-parameters may be evaluated with a relatively high degree of accuracy. However, special attention should be addressed to the following:

- Only a small section of each joint surface will usually be available, particularly for joints intersecting the borehole at an obtuse angle. Evaluation of the roughness coefficient, J_r, may therefore be difficult. Particularly the large and medium scale undulation may be difficult to estimate.

- 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 under-represented in the cores, and this will give too high RQD-values and too low J_n-values.

- Whereas RQD is often calculated for every meter, J_n must usually be estimated for sections of several metres.

- In massive rock it is impossible to estimate SRF from drill cores. In rock intersected by weakness zones it may be possible to give some suggestions about SRF. An estimate of SRF in massive rock could be done based on the overburden, height of a mountain side, if stress measurements are carried out in the borehole, or experiences from nearby construction sites.

In general, 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 log data combined with estimates of J_w and SRF it will be possible to get a rough impression of the Q-values of the cores, and these could be helpful during planning. 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. One also has to take into account whether the rock mass is going to be grouted or not in order to estimate 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 in order to investigate particular zones. It is then imperative to consider how much of the total rock masses these zones represent. If a borehole is orientated along a fracture zone, the parameter values for this zone will be determined.

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Table 1 RQD-values and volumetric jointing.

1	RQD (Rock Quality Designation)		RQD
A	Very poor	(> 27 joints per m^3)	0-25
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

Table 2 J_n – values.

2	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 portals, use 2 x J_n

Table 3 J_r – values.

3	Joint Roughness Number	J_r
a) *Rock-wall contact, and* ***b)*** *Rock-wall contact before 10 cm of shear movement*		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough, irregular, planar	1.5
F	Smooth, planar	1
G	Slickensided, planar	0.5
Note: i) Description refers to small scale features and intermediate scale features, in that order		
c) *No rock-wall contact when sheared*		
H	Zone containing clay minerals thick enough to prevent rock-wall contact when sheared	1
Note: ii) Add 1 if the mean spacing of the relevant joint set is greater than 3 m (dependent on the size of the underground opening) iii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented in the estimated sliding direction		

Table 4 J_a – values.

4	Joint Alteration Number	Φ_r approx.	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 J_a 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

Table 5 J_w – values.

5	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 crude estimates. Increase J_w if the rock is drained or grouting is carried out ii) Special problems caused by ice formation are not considered		

Table 6 SRF-values.

6	**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 6L and 6M			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, mainly massive rock, stress problems*	σ_c / σ_1	σ_θ / σ_c	**SRF**
F	Low stress, near surface, open joints	>200	<0.01	2.5
G	Medium stress, favourable stress condition	200-10	0.01-0.3	1
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:	ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1 / \sigma_3 \leq 10$, 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) iii) 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)	*Squeezing rock: plastic deformation in incompetent rock under the influence of high pressure*		σ_θ / σ_c	**SRF**
M	Mild squeezing rock pressure		1-5	5-10
N	Heavy squeezing rock pressure		>5	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)			
d)	*Swelling rock: chemical swelling activity depending on the presence of water*			**SRF**
O	Mild swelling rock pressure			5-10
P	Heavy swelling rock pressure			10-15

Table 7 ESR-values.

7	Type of excavation	ESR
A	Temporary mine openings, etc.	*ca.* 3-5
B	Vertical shafts*: i) circular sections ii) rectangular/square section * Dependant of purpose. May be lower than given values.	*ca.* 2.5 *ca.* 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

Table 8 Conversion from actual Q-values to adjusted Q-values for design 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	0.1 < 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 < 0.1	Use actual Q-value.