From Palmström A.: RMi – a rock mass characterization system for rock engineering purposes.
PhD thesis, Oslo University, Norway, 1995, 400 p.

APPENDIX 3

METHODS TO QUANTIFY THE PARAMETERS APPLIED IN THE RMi

> *"If no collective system and method exists, so much detail is usually recorded as to obscure the essential data needed for design."*
> Douglas A. Williamson and C. Rodney Kuhn (1988)

From Chapter 2 on data collection it has been shown that some of the uncertainties and errors in rock engineering stem from the rock mass characterizations, such as:

- The way the description is performed, or the quality of the characterization made of the various parameters in rock masses. As most of the input parameters in rock engineering and rock mechanics are found from observations, additional errors may be introduced from poorly defined descriptions and methods for data collection.
- During the measurements of joints and jointing, as the joints exposed only may be a portion of the joints considered to be representative of the entire rock mass.

This appendix has been worked out to remedy some of these tasks giving supplementary descriptions of the various methods in use to find the numerical values of the RMi parameters, either directly from measurements or observations, or from other types of measurements of rock strength properties or jointing features. Some of the methods described are time-consuming and expensive. Therefore, the methods used during collection of the field and laboratory investigations should be chosen to meet the requirements to accuracy and quality of the input data which may be determined by:

- the purpose and use of the construction;
- the stage of planning;
- the method of excavation;
- the availability to observe or measure the actual properties of the rock mass;
- the complexity of the geology; and
- the quality of the ground with respect to the type and use of the actual opening or the method of excavation.

For exchange of engineering geological data and for information to other people involved, additional documentation of the rock mass conditions is important. Therefore, a verbal description should be included in the characterization. It should describe the composition and structure of the rock mass with special emphasis on the parameters of importance for engineering properties, including how the rock masses are related to the geological setting in the area. Important during the development of the RMi system has been to:

- group the rock masses in such a way that those parameters which are of most universal concern are clearly dealt with, and at the same time
- keeping the number of such parameters to a useful minimum. Thus, the input parameters selected, which are determined numerically in this appendix, are only:
- the uniaxial compressive strength of intact rock;
- the joint characteristics given as the joint condition factor; and
- the block volume.

1 METHODS TO DETERMINE THE UNIAXIAL COMPRESSIVE STRENGTH OF ROCKS

For some engineering or rock mechanical purposes the numerical characterization of rock material alone can be used, for example boreability, aggregates for concrete, asphalt etc. Also in assessment for the use of fullface tunnel boring machines (TBM) rock properties such as compressive strength, hardness, anisotropy are often among the most important parameters. For stability evaluations and rock support engineering, however, the rock properties are mainly of importance only where they are weak or overstressed. Though rock properties in many cases are overruled by the effects of joints, it should be brought in mind that their properties highly determine how the joints have been formed, which in turn may explain their characteristics.

Although the geological classification of rocks is mainly based on formation and composition of the material, it is so well established that other methods for division of this material have not come into major use. Since, in rock mechanics and rock engineering, the rock behaviour rather than its composition is of main importance Goodman (1989) has presented the classification shown in Table A3-1.

TABLE A3-1 BEHAVIOURAL CLASSIFICATION OF ROCKS (from Goodman, 1989)

GROUPS OF ROCKS	Examples
I Crystalline texture	
A. Soluble carbonates and salts	Limestone, dolomite, marble, rock salt, gypsum
B. Mica or other planar minerals in continuous bands	Mica schist, chlorite schist, graphite schist
C. Banded silicate minerals without continuous mica sheets	Gneiss
D. Randomly oriented and distributed silicate minerals of uniform grain size	Granite, diorite, gabbro, syenite
E. Randomly oriented and distributed silicate minerals in a background of very fine grain and with vugs	Basalt, rhyolite, other volcanic rocks
F. Highly sheared rocks	Serpentinite, mylonite
II Clastic texture	
A. Stably cemented	Silica-cemented sandstones and limonite sandstones
B. With slightly soluble cement...	Calcite-cemented sandstone and conglomerate
C. With highly soluble cement	Gypsum-cemented sandstones and conglomerates
D. Incompletely or weakly cemented..................................	Friable sandstone, tuff
E. Uncemented..	Clay-bound sandstones
III Very fine-grained rocks	
A. Isotropic, hard rocks	Hornfels, some basalts
B. Anisotropic on a macro scale, but microscopically isotropic hard rocks	Cemented shales, flagstones
C. Microscopically anisotropic hard rocks.........................	Slate, phyllite
D. Soft, soil-like rocks..	Compaction shale, chalk, marl
IV Organic rocks	
A. Soft coal	Lignite and bituminous coal
B. Hard coal	
C. 'Oil shale'	
D. Bituminous shale	
E. Tar sand	

A verbal description of the rock should, in addition to the rock name and its strength, include possible anisotropy, weathering, and reduced long-term resistance to environmental influence (durability). The description of the rock could also contain information on texture, colour, lustre, small scale folds, etc. which can give information to a better understanding of the rock conditions as well as the jointing.

As rocks are composed of an aggregate of several types of minerals with different properties, arrangement and "welding", there are many factors which determine their strength properties. In addition, possible weathering and alteration can highly influence on the final strength properties of a rock. The effect of this is outlined in Section 1.6.4

Some minerals have a stronger influence on the properties of a rock than other. In rock construction the *mica and similar minerals* have an important contribution where they occur as parallel oriented continuous layers (Selmer-Olsen, 1964). Mica schists and phyllites with a high amount of mica show, therefore, strongly anisotropic properties which often influence in rock construction works as shown in Section 1.6.3.

1.1 The uniaxial compressive strength (σ_c)

In rock mechanics and engineering geology the boundary between rock and soil is defined in terms of the uniaxial compressive strength and not in terms of structure, texture or weathering. Several classifications of the compressive strength of rocks have been presented, as seen in Fig. A3-1. In this work a material with the strength ≤ 0,25 MPa is considered as soil, refer to ISRM (1978) and Table A3-1.

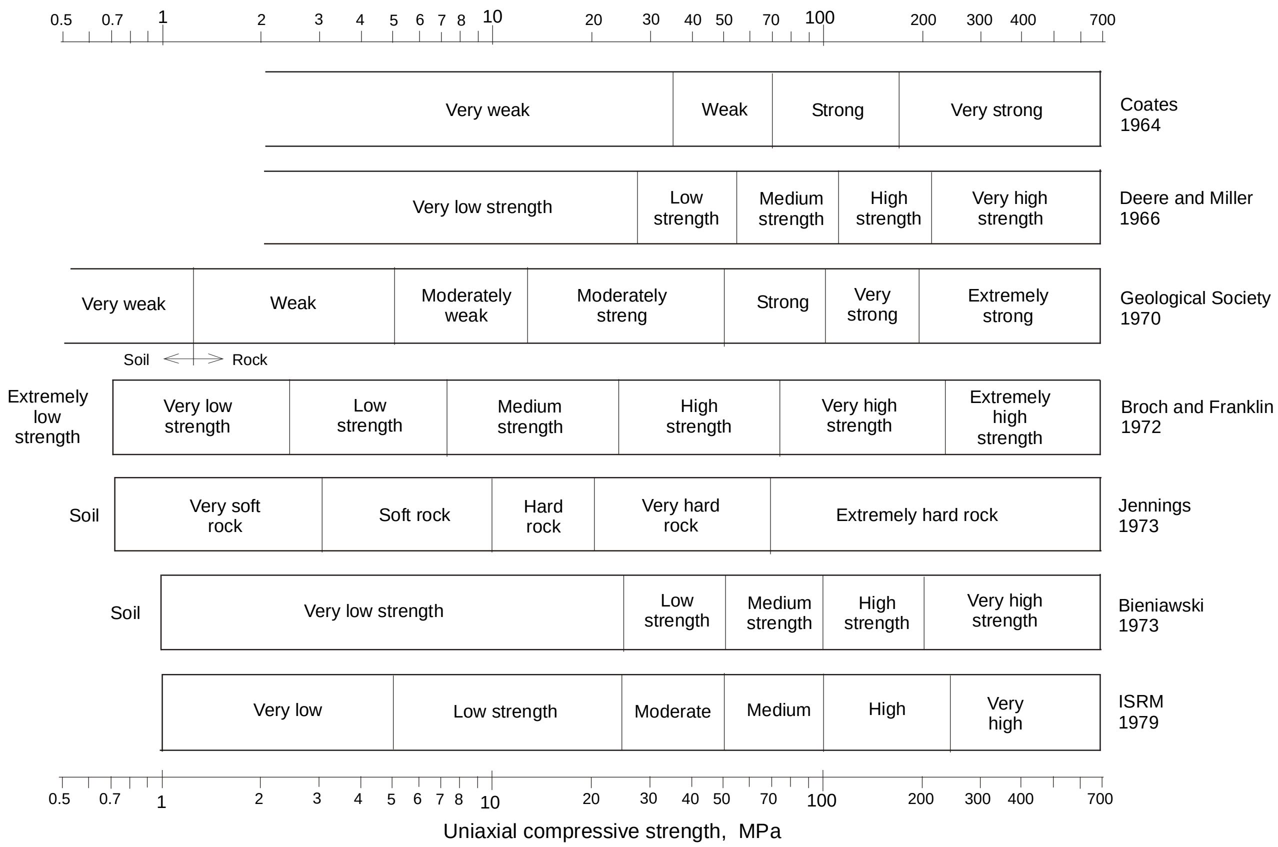


Fig. A3-1 Various strength classifications for intact rock (from Bieniawski, 1984)

The uniaxial compressive strength can be determined directly by uniaxial compressive strength tests in the laboratory, or indirectly from point-load strength test (see Section 1.4.2). The tests should be carried out according to the methods recommended by the ISRM (1972).
The classification of the uniaxial compressive strength suggested by ISRM is shown in Tables A3-1a and A3-7.

TABLE A3-1a CLASSIFICATION OF THE UNIAXIAL COMPRESSIVE STRENGTH OF ROCKS (σ_c) (from ISRM (1978)

Soil	$\sigma_c < 0.25$ MPa
Extremely low strength	$\sigma_c = 0.25 - 1$ MPa
Very low strength	$\sigma_c = 1 - 5$ MPa
Low strength	$\sigma_c = 5 - 25$ MPa
Medium strength	$\sigma_c = 25 - 50$ MPa
High strength	$\sigma_c = 50 - 100$ MPa
Very high strength	$\sigma_c = 100 - 250$ MPa
Extremely high strength	$\sigma_c > 250$ MPa

The uniaxial compressive strength of the rock constitutes the highest strength limit of the actual rock mass. ISRM (1981) has defined the uniaxial strength of a rock to samples of 50 mm diameter. A rock is, however, a fabric of minerals and grains bound or welded together. The rock therefore includes microscopic cracks and fissures. Rather large samples are required to include all the components that influence strength. When the size of the sample is so small that relatively few cracks are present, the failure is forced to involve a larger part of new crack growth than in a larger sample. Thus the strength is size dependent. This scale effect of rocks has been a subject of investigations over the last 30 to 40 years. Fig. A3-2 shows the results from various tests compiled by Hoek and Brown (1980) where the scale effect (for specimens between 10 and 200 mm) has been found as

$$\sigma_c = \sigma_{c50} \, (50/d)^{0.18} \qquad \text{eq. (A3-1)}$$

where σ_{c50} is for specimens of 50 mm diameter, and
d is the diameter (in mm) of the actual sample.

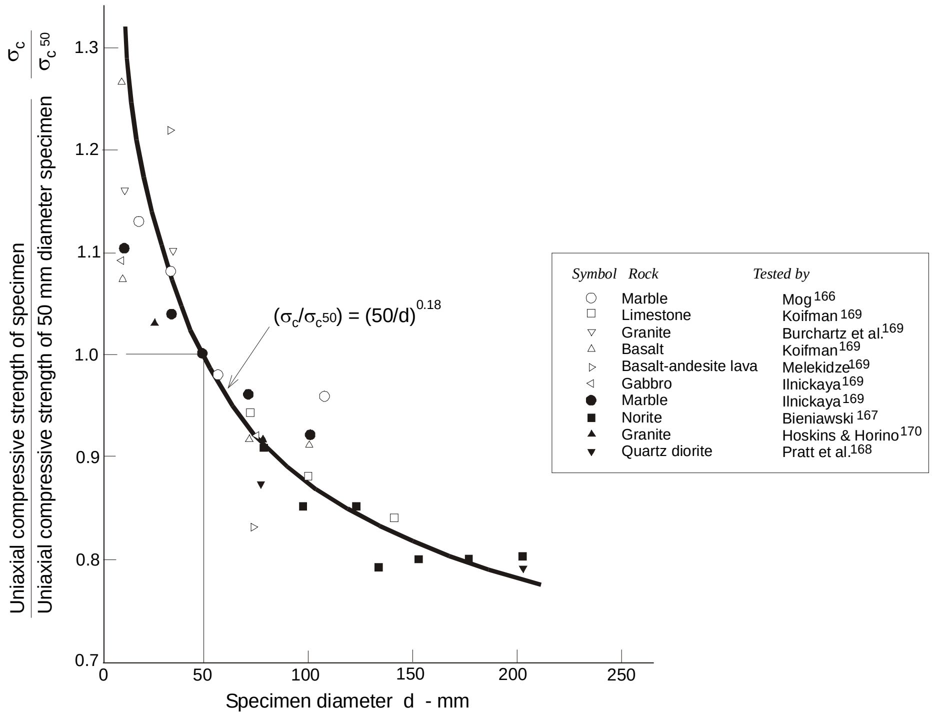


Fig. A3-2 Influence of specimen size upon the uniaxial compressive strength of intact rock. (from Hoek and Brown, 1980)

Wagner (1987) has later shown that 'samples' with diameter more than 2 metres follow the following similar equation:

$$\sigma_{cf} = \sigma_{c50}\,(50/d)^{0.2} \qquad \text{eq. (A3-1a)}$$

ISRM (1980) recommends that the uniaxial compressive strength of the rock material in an area is given as the mean strength of rock samples taken away from faults, joints and other discontinuities where the rock may be more weathered. When the rock material is markedly anisotropic in its strength, the value used in the RMi should correspond to the direction along which the smallest mean strength was found. However, in such cases it is usually of importance to record the uniaxial compressive strength also in other directions.

Many compressive strength tests are made on dry specimens. ISRM (1980) recommends that the samples should be tested at a water content pertinent to the problem to be solved. Because rocks are often much weaker for wet than for dry materials, it is important to inform about the moisture conditions. This is further discussed in Section 1.3.

The compressive strength σ_{c50} is used directly in calculation of RMi. The accuracy and quality of this measurement has been discussed by several writers, as different modes of failure occur during the tests (Lama and Vutukuri, 1978; Hoek and Brown, 1980; Farmer and Kemeny, 1992; among others). Undoubtably, compressive strength is the mostly used test on rock samples and the loading resembles often situations in the field.

The uniaxial compressive test is time-consuming and is also restricted to those relatively hard, unbroken rocks that can be machined into regular specimens. Although the strength classification is based on laboratory tests, it can be approximated by simple methods. An experienced person can make a rough five-fold classification of rock strength with a hammer or pick. Deere and Miller (1966) have shown that rock strength can be estimated with a Scmidt hammer and a specific gravity test with enough reliability to make an adequate strength characterization. According to Patching and Coates (1968) the rock strength can be quickly and cheaply estimated in the field, and more precision can be attained, if required, by laboratory tests. Also from a fully description of a rock including composition and possible anisotropy and weathering it may in many instances be possible to assess the strength. These matters are described in the following. The methods, their applications, test procedures etc. are not dealt with, merely how the results can be used.

1.2 Effect of saturation upon rock strength

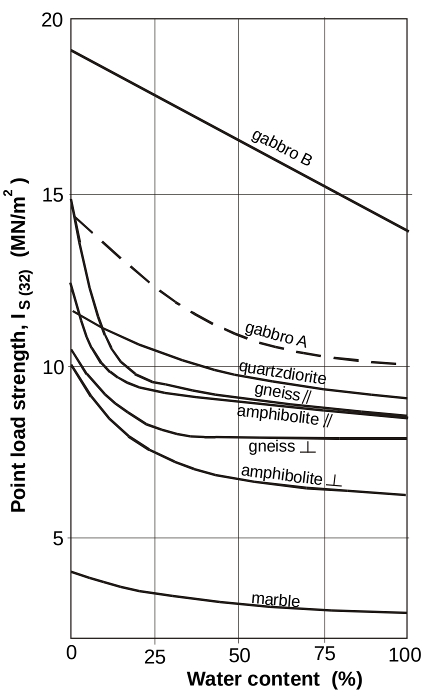
It is known that the influence of water on the strength of rocks may be considerable. From the published papers no general expressions other than the fact that moisture may reduce the strength of rocks drastically, has been established. Salustowicz (1965) has found that the decrease by moisture in sandstones and shales can be 40% and 60% of the dry strength respectively. Colback and Wiid (1965) mention a decrease from dry to saturated in the order of 50% both for quartzitic shale and for quartzitic sandstone. Table A3-2 shows compilation of some published result between compressive strength for saturated versus dry specimens. Results from Ruiz (1966) in Table A3-2 shows that wet strength may be higher than dry probably caused by heterogeneity and possible error from few tests. This effect has also been observed by Broch (1979) for some very fine-grained rocks (basalt, black shale).

TABLE A3-2 EFFECT OF SATURATION UPON STRENGTH (worked out from Lama and Vutukuri, 1978, with data from Feda, 1966; Ruiz, 1966; Boretti-Onyszkiewicz, 1966)

Number of rocks tested	Rock type	Saturated in % of dry strength	
5	basalt	45 - 116	
2	diabase	92 - 125	
1	dolomite	83	
3	limestone	56 - 118	
1	schist	48	
3	gneiss	36 - 59	
5	gneiss	88 - 112	
2	granitic gneiss	68 - 85	
4	granite	68 - 116	
1	granulite	54	
1	quartzite	80	
1	sandstone	90	
5	sandstone	67 – 87	tested normal to bedding
	"	54 – 92	tested parallel to bedding

TABLE A3-3 EFFECT OF MOISTURE CONTENT ON COMPRESSIVE STRENGTH (worked out from Lama and Vutukuri, 1978, based on data from Price, 1960, and Obert et al., 1946)

No. of rocks tested	ROCK TYPE	Relative compressive strengths for various Moisture contents expressed as % of oven-dry		
		Oven-dry	Air-dry	Saturated
5	sandstone	100	51 - 99	45 – 90
1	marble	100	99	95
1	limestone	100	96	83
1	granite	100	93	86
1	slate	100	94	80



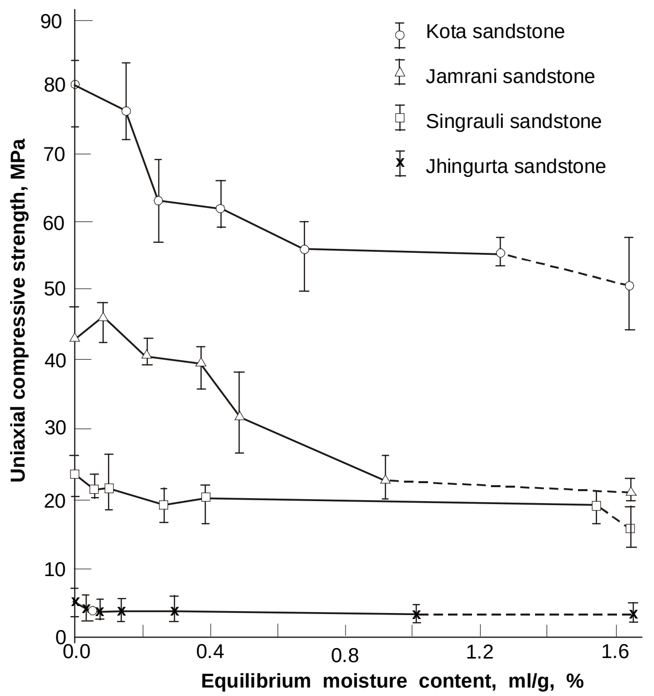


Fig. A3-3 Left: Point-load strength as a function of water content in rock cores (from Broch, 1979).

Right: Variation in uniaxial compressive strength with equilibrium moisture content for sandstones (from Seshagiri Rao et al. 1987).

The results shown in Table A3-3 differ whether the samples have been oven-dried or air-dried. This effect may strengthen the variation published in many papers on the effect of water upon strength. Broch (1979) has shown in Fig. A3-3 that the main strength reduction generally takes place for water contents less than 25%. Also Seshagiri Rao et al. (1987) have found that the main strength reduction for sandstones takes place at low moisture content (Fig. A3-3)

Broch (1979) concludes that the strength reduction from water increases with increasing amount of dark minerals (biotite, amphiboles, pyroxenes) and for increasing schistosity (anisotropy), as seen in Fig. A3-4.

From this complex picture of the effect of moisture and saturation upon rock strength Lama and Vutukuri (1978) conclude that: *"Moisture in the rocks has a very significant effect on compressive strength in many instances. Unless the values to be used for design purposes are corrected to in situ conditions, catastrophic failures can occur. Many times, saturated specimens are recommended for tests so that the values obtained are conservative."*

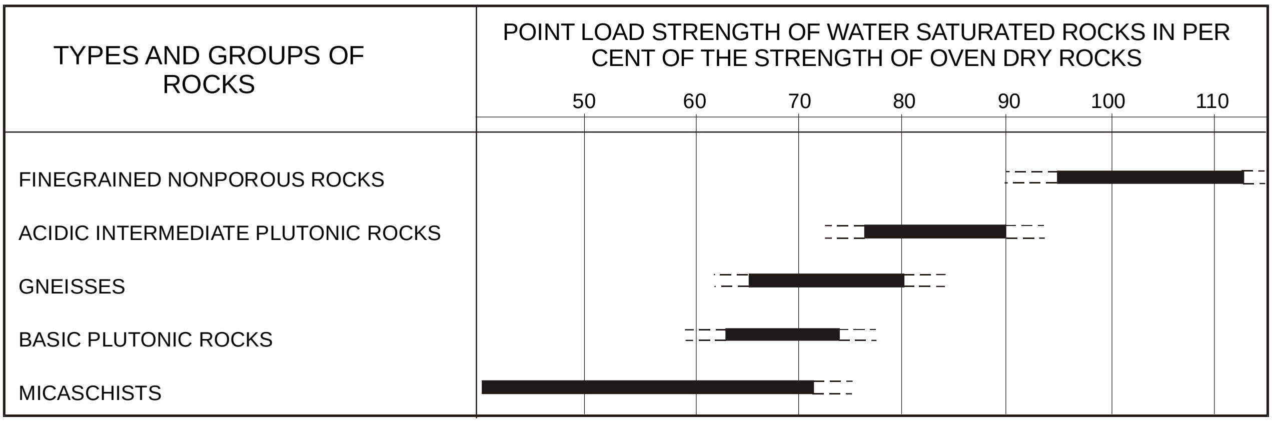


Fig. A3-4 The effect of water on point load strength of various groups of rocks (from Broch, 1979)

It is, therefore, important that the conditions at which the rocks are tested, are reported. ISRM (1972, 1981) suggests that rock samples are stored in 50% humidity for 5 - 6 days before testing.

1.3 Compressive strength determined from the point-load strength

The principle of the point load strength test is that a piece of rock is loaded between two hardened steel points. Details on the measuring procedure are described by ISRM (1985), and the method is further dealt with in several textbooks and papers (Lama and Vutukuri, 1978; Hoek and Brown, 1980, among others).

Both Franklin (1970) and Bieniawski (1984) recommend the use of point load strength index (Is) for rock strength testing. The reason is that Is can be determined in the field on specimens without preparation, using simple portable equipment. Also Broch (1983) points out the great advantage using the point load strength test as it does not require machined specimen. As long as the influence of specimen size and shape are considered in the calculation of the strength index, any piece of rock, whether the surface is smooth or rough, can in principle be tested. Although tests on irregular specimens appear to be crude, Wittke & Louis (1969) have shown that the results need be no less reproducible than those obtained in uniaxial compression.

1.3.1 The point load strength index (Is)

Greminger (1982) relates the test results, $Is_{(50)}$, to standard 50 mm thick samples; this has also been applied by ISRM (1985) in a revised edition of the 'Suggested Method for determining point load strength'. The point load strength test is a form of "indirect tensile" test, but is largely irrelevant to its primary role in rock classification and strength as a tensile characterization (ISRM, 1985). $Is_{(50)}$ is approximately 0.80 times the uniaxial tensile or Brazilian tensile strength.

Also from the point load strength test a strength anisotropy index ($Ia_{(50)}$) can be measured (Broch, 1983) from the maximum and minimum strengths obtained normal to, and parallel to the weakness planes, respectively, of the rock, i.e. bedding, foliation, cleavage, etc.

TABLE A3-4 CLASSIFICATIONS OF THE POINT LOAD STRENGTH (Is)

TERM	**Bieniawski (1984)**	**Deere (1966)**
very high strength	Is > 8 MPa	Is > 10 MPa
high	Is = 4 - 8 MPa	Is = 5 - 10 MPa
medium	Is = 2 - 4 MPa	Is = 2.5 - 5 MPa
low	Is = 1 - 2 MPa	Is = 1.25 - 2.5 MPa
very low	Is < 1 MPa	Is < 1.25 MPa

As shown in Fig. A3-3 the point load strength varies with the water content of the specimens. ISRM (1985) mentions that the variations are particularly pronounced for water saturations below 25%. At water saturation above 50% the strength is less influenced by small changes in water content, so that tests in this water content range are recommended unless tests on dry rock are specifically required.

1.3.2 The correlation between Is and σ_c

Point load strength test may often replace the uniaxial compressive strength test as it is, when properly conducted, as reliable and much quicker to measure. Hoek and Brown (1980) are of the opinion that a reasonable estimate of the uniaxial compressive strength of the rock can be obtained by means of the point load test. Other authors (Greminger, 1982; Seshagiri Rao et al., 1987) have, however, found that correlations with uniaxial compressive strength using

$$\sigma_c = k \times Is \qquad \text{eq. (A3-2)}$$

or

$$\sigma_c = k_{50} \times Is_{50} \quad \text{(related to 50 mm thick samples)} \qquad \text{eq. (A3-2a)}$$

are only approximate. They show that, though the factor k or k_{50} generally varies between 15 and 25 it may sometimes vary between 10 and 50 especially for anisotropic rocks, so that errors of up to 100% are possible in using an arbitrary ratio value to predict compressive strength from point load strength. The variation in k and k_{50} published in various papers on point load strength results is shown in Table A3-5.

Greminger (1982) found that estimation of uniaxial compressive strength of anisotropic rocks may lead to significant errors, which is documented in Fig. A3-5. Seshagiri Rao et al. (1987) found for sandstones that $k_{50} = \sigma_c / Is_{50}$ varied with the strength of the rock according to the following:

$k_{50} = 14$ for $\sigma_c = 80$ MPa, $k_{50} = 12$ for $\sigma_c = 43$ MPa

$k_{50} = 14$ for $\sigma_c = 24$ MPa, $k_{50} = 11$ for $\sigma_c = 6$ MPa

TABLE A3-5 THE VALUE OF k PRESENTED IN VARIOUS PAPERS

REFERENCE	value of k
Franklin (1970)	k = approx. 16
Broch and Franklin (1972)	k = 24
Indian standard (1978)	k = 22
Hoek and Brown (1980)	k= 14 + 0.175 D *)
Greminger (1982)	no general correlation, see Fig. A3-5
ISRM (1985)	k_{50} = 20 - 25
Brook (1985)	k_{50} = 22
Seshagiri Rao et al. (1987)	no general correlation
Hansen (1988)	no general correlation
Ghosh and Srivastava (1991)	k_{50} = 16

*) D is the diameter or thickness of the tested sample

They argued that no general relation can be established between σ_c and Is_{50} . Also Hansen (1988) is of the same opinion from numerous tests carried out at the Technical University of Norway. The trend, therefore, seems to be that the factor k or k_{50} is higher for strong rocks. Based on the results above it is suggested, where no other information is available, to use the values of k_{50} (related to standard thickness) presented in Table A3-6 in the correlation from point load strength to uniaxial compressive strength.

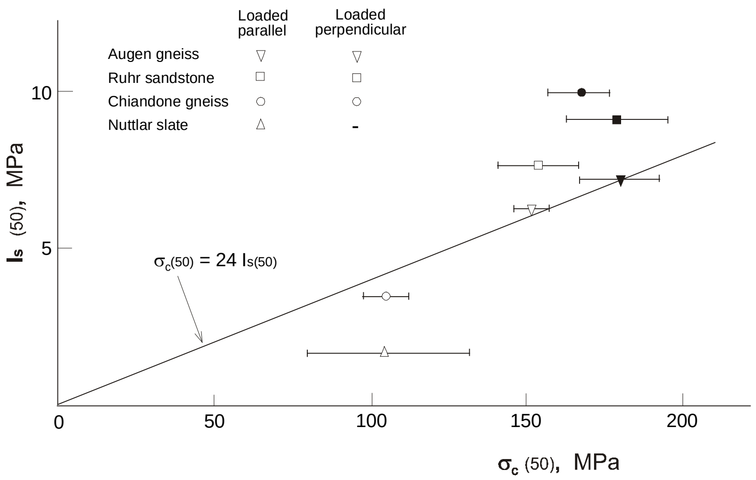


Fig. A3-5 Relationship between uniaxial compressive strength and point load strength index (from Greminger, 1982)

TABLE A3-6 SUGGESTED VALUE OF THE FACTOR k_{50} VARYING WITH THE STRENGTH OF THE ROCK

σ_c (MPa)	$I_{s\,50}$ (MPa)	k_{50}
25*) - 50	1.8 - 3.5	14
50 - 100	3.5 - 6	16
100 - 200	6 - 10	20
> 200	> 10	25

*) Bieniawski (1973) suggests that point load strength test are not carried out on rocks having compressive strength less than approximately 25 MPa.

1.4 Compressive strength estimated from Schmidt hammer rebound number

The Schmidt hammer is used for non-destructive testing of the quality of rocks and concretes. It measures the 'rebound hardness' of the tested material. The mechanism of operation is simple; a plugger, released by a spring, impacts against the tested rock surface. The rebound distance of the plunger is read directly from a numerical scale.

Measurements of rock properties with the Schmidt hammer are based on an imperfectly elastic impact of two bodies, one of which is represented by the impact of the test hammer and the other by the surface of the tested rock (Ayday and Göktan, 1992). Schmidt hammer models are designed in different levels of impact energy, and the two types L and N are commonly adapted for rock property determinations. The type L has an impact energy of 0.735 Nm, 1/3 of that of Type N.

ISRM (1978) suggests that the L hammer can be used for the testing of rocks which have uniaxial compressive strength in the range of approximately 20 - 150 MPa . ISRM (1978) has also given a complete test procedure including a chart for correlating Scmidt rebound hardness to uniaxial compressive strength.

1.5 Compressive strength assessed from simple field test

Sometimes, particularly at an early stage in the description of the rock mass, strength may be assessed without testing (ISRM, 1980). Such a first estimate of the uniaxial compressive strength (σ_c) may be made by visual and sensory description of hardness of rock or consistency of a soil (Piteau, 1970; Herget, 1982). The strength can be judged from simple hardness tests in the field with geological pick by observing the resistance to breaking under impact, as shown in Table A3-7.

TABLE A3-7 SIMPLE FIELD IDENTIFICATION COMPRESSIVE STRENGTH OF ROCK AND CLAY (from ISRM, 1978)

GRADE	TERM	FIELD IDENTIFICATION	Approx. range of σ_c (MPa)
S1	Very soft clay	Easily penetrated several inches by fist.	< 0.025
S2	Soft clay	Easily penetrated several inches by thumb.	0.025-0.05
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort.	0.05 -0.10
S4	Stiff clay	Readily intended by thumb, but penetrated only with great effort.	0.10 -0.25
S5	Very stiff clay	Readily intended by thumbnail.	0.25 -0.50
S6	Hard clay	Intended with difficulty by thumbnail.	> 0.50
R0	Extremely weak rock	Intended by thumbnail.	0.25 - 1
R1	Very weak rock	Crumbles under firm blows with point of geological hammer; can be peeled by a pocket knife.	1 - 5
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow identi-fications made by firm blow with point of geological hammer.	5 - 25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife; specimen can be fractured with single firm blow of geological hammer.	25 - 50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it.	50 - 100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it.	100 - 250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer.	> 250

The clays in grade S1 - S6 can be silty clays and combinations of silts and clays with sands, generally slow draining.

The hammer test should be made with a geologist's hammer on pieces about 10 cm thick placed on a hard surface, and tests with the hand should be made on pieces about 4 cm thick. The pieces must not have incipient fractures, and therefore several should be tested.
For anisotropic rocks tests should be carried out in different directions to the structure. The lowest representative values should be applied. It is also possible to use the foliation anisotropy factor

described in Section 1.7.3 to find the lowest value of compressive strength to be applied in the RMi.

1.6 Compressive strength estimated from rock description

Probably the most generally used single describer of rock structure has been "rock type". In this term a wide variety of geological factors is embraced, ranging from basic rock origin: igneous, sedimentary and metamorphic, to special properties such as texture and structure, mineral size and composition, anisotropy, degree of alterations, etc.

According to Franklin (1970) over 2000 names are available for the igneous rocks that comprise about 25% of the earth's crust, in contrast to the greater abundance of mudrocks (35%) for which only a handful of terms exist; yet the mudrocks show a much wider variation in mechanical behaviour and greater challenges in rock constructions. From the rock engineering point of view this vast amount of rock types can be grouped into a reduced number of about 40, regarding strength and behaviour.

Geological nomenclature emphasizes solid constituents whereas from the engineers' point of view pores, cracks and fissures are of greater mechanical significance. Petrological data can, however, make an important contribution towards the prediction of mechanical performance, provided that additional information on the anisotropy and weathering is provided.

In most instances, a simple hand-specimen name will likely be adequate to determine the rock type and name. An added term to indicate grain size may often be useful (Patching and Coates, 1968). For many rocks the name of the rock, its homogeneity and continuity can be established by visual observation in the field.

An accurate classification of rock materials used by geologists requires, however, often a detailed consideration of mineralogy and petrography for instance from thin section analysis, an investigation, which may seldom be important to engineers only in few cases.

1.6.1 <u>Main geological characteristics</u>

The minerals of most <u>igneous</u> rocks are hard and may display cleavage but are of a dense interfingering nature resulting in homogeneous materials with only slight, if any, directional differences in mechanical properties of the rock.

The minerals of <u>sedimentary</u> rocks are usually softer and are of generally weaker assemblage than the igneous rocks. In these rocks the minerals are not interlocking but are cemented together with inter-granular matrix material. Sedimentary rocks usually contain lamination or other sedimentation structures and, therefore, may exhibit significant *anisotropy* in physical properties depending upon the degree of their development. Of this group, argillaceous and sandstone rocks are usually the most strongly anisotropic.

The <u>metamorphic</u> rocks, more particularly the micaceous and chloritic schists, are probably the most outstanding with respect to *anisotropy*. The metamorphism have resulted in harder minerals in most cases; however, the preferred orientation of platy minerals due to shearing movements results in considerable directional differences in mechanical properties. Rocks with gneissic texture are generally not strongly anisotropic. Slate, due to well developed slaty cleavage, is highly anisotropic.

1.6.2 Strength assessment from rock name

The rock types often give relative indications of their inherent properties (Piteau, 1970; Patching and Coates, 1968). For many rocks, however, the correlation between the petrographic names of rocks and their mechanical properties may be poor, caused by difference in composition, grain size, porosity, cementation, anisotropy within each type. Nevertheless, a great deal of associated information about a rock can be inferred from its geological name, such as whether it may be homogeneous, layered, schistose or irregular.

TABLE A3-8 NORMAL RANGE OF COMPRESSIVE STRENGTH FOR SOME COMMON ROCK TYPES (data from Hansen, 1988 and Hoek and Brown, 1980) AND VALUES FOR THE *m* FACTOR IN HOEK-BROWN FAILURE CRITERION (from Hoek et al., 1992).

Rock name	Uniaxial compressive strength σ_c			Rating of the factor m_i [1)]	**Rock name**	Uniaxial compressive strength σ_c			Rating of the factor m_i [1)]
	low	average	high			low	average	high	
Sedimentary rocks					**Metamorphic rocks**				
Anhydrite		120'?		13.2	Amphibolite	75	125	250	31.2
Coal	16"	21"	26"		Amphibolitic gneiss	95	160	230	31 ?
Claystone	2'	5'	10'	3.4	Augen gneiss	95	160	230	30 ?
Conglomerate	70	85	100	(20)	Black shale	35	70	105	
Coral chalk	3	10	18	7.2	Garnet mica schist	75	105	130	
Dolomite	60'	100'	300'	10.1	Granite gneiss	80	120	155	30 ?
Limestone	50*	100'	180*	8.4	Granulite	80'	150	280	
Mudstone	45	95	145		Gneiss	80	130	185	29.2
Shale	36"	95"	172"		Gneiss granite	65	105	140	30 ?
Sandstone	75	120	160	18.8	Greenschist	65	75	85	
Siltstone	10'	80'	180'	9.6	Greenstone	120'	170*	280*	20 ?
Tuff	3'	25'	150'		Greywacke	100	120	145	
Igneous rocks									
Andesite	75'	140'	300'	18.9	Marble	60'	130'	230'	9.3
Anorthosite	40	125	210		Mica gneiss	55	80	100	30 ?
Basalt	100	165	355"	(17)	Mica quartzite	45	85	125	25 ?
Diabase (dolerite)	227"	280"	319"	15.2	Mica schist	20	80*	170*	15 ?
Diorite	100	140	190	27 ?	Mylonite	65	90	120	
Gabbro	190	240	285	25.8	Phyllite	21	50	80	13 ?
Granite	95	160	230	32.7	Quartz sandstone	70	120	175	
Granodiorite	75	105	135	20 ?	Quartzite	75	145	245	23.7
Monzonite	85	145	230	30 ?	Quartzitic phyllite	45	100	155	
Nepheline syenite	125	165	200		Serpentinite	65	135	200	
Norite	290"	298"	326"	21.7	Slate	120'	190'	300'	11.4
Pegmatite	39	50	62		Talc schist	45	65	90	10 ?
Rhyolite		85'?		(20)					
Syenite	75	150	230	30 ?					
Ultra basic rock	80'	160	360						

Soil materials[2)]:

Very soft clay σ_c = 0.025 MPa — Soft clay σ_c = 0.025 - 0.05 MPa — Firm clay σ_c = 0.05 - 0.1 MPa

Stiff clay σ_c = 0.1 - 0.25 MPa — Very stiff clay σ_c = 0.25 - 0.5 MPa — Hard clay σ_c = > 0.5 MPa

Silt, sand: assume σ_c = 0.0001- 0.001 MPa

* Values found by the Technical University of Norway, (NTH) Inst. for rock mechanics.
' Values given in Lama and Vutukuri, 1978.
" Values given by Bieniawski, 1984.

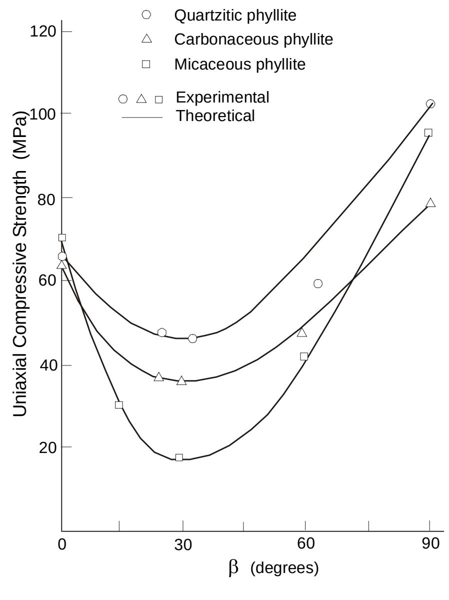
[1)] Refer to the factor, m, in the failure criterion for rock masses by Hoek et al. (1992), m_i is the parameter for intact rock, see Chapter 9, Section 1. Values in parenthesis have been estimated by Hoek et al (1992); some others with question mark have been assumed in this work.
[2)] For clays the values of the uniaxial compressive strength is based on ISRM (1978) as presented in Table A3-7.

A good description of the rock material is a prerequisite when such strength evaluations are made. Adjustments for possible anisotropy (schistosity, foliation, bedding) and weathering/alteration in rocks are further described in Section 1.7.3 and 1.7.4.

Where representative, fresh specimens of the various types occur, it is, however, often possible if not accurate values are required to make estimates of the compressive strength from the rock name as presented in Table A3-8. Additional results from compressive strength tests are given in many textbooks. Refer to Lama and Vutukuri (1978), Hoek and Brown (1980) etc. In the following some methods to estimate the uniaxial compressive strength for anisotropic and weathered or altered rocks are outlined.

1.6.3 Reduction in strength from anisotropy

Anisotropy in rock material is mainly caused by schistosity, foliation or bedding. The difference in properties is determined by the arrangement and amount of flaky and elongated minerals (mica, chlorite, amphiboles). This intrinsic rock property tends to be significant even at the scale of a laboratory test specimen.



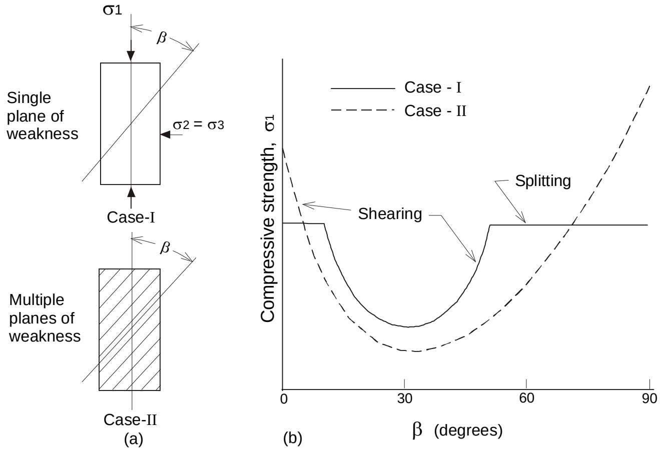


Fig. A3-6 Experimental and theoretical variation of uniaxial compressive strength with angle between schistosity plane and direction of testing (from Ramamurthy et al., 1993)

Tsidzi (1986, 1987, 1990) has presented results from tests on strength anisotropy in foliated rocks, together with measurements of their intrinsic anisotropic foliation fabric. The lowest strength value occurred when the orientation of the anisotropic fabric element (bedding, foliation) to the specimen loading axis was between 30^o and 45^o, and the highest value for orientation either 0^o or 90^o. This effect has also been shown by Ramamurthy et al., (1993) in Fig. A3-6, Hoek and Brown (1980) and many other authors. Hence, the estimation of compressive strength anisotropy of rocks in terms of values obtained only for tests parallel (0^o) and normal (90^o) to the foliation plan give only limited information on the rock strength.

From his tests carried out on slates, schists, and gneisses with compressive strength ranging between 20 and 285 MPa Tsidzi (1990) has worked out the classification which is shown in Table A3-9. From regressions Tsidzi arrived at the following expression for the uniaxial compressive strength anisotropy factor:

$$fA = 0.95 + 0.17\ Fi \qquad \text{eq. (A3-3)}$$

where Fi is the foliation index. Its ratings are indicated in Table A3-9.

TABLE A3-9 CLASSIFICATION OF FOLIATION AND ANISOTROPY OF ROCKS (from Tsidzi, 1986, 1987, 1990)

FOLIATION CLASSIFICATION - - - - - - - - - - - - - - -- ANISOTROPY CLASSIFICATION	**DESCRIPTION**	FOLIATION ANISOTROPY FACTOR **fA**
Very weakly foliated (or non-foliated) Fi < 1.5 - - - - - - - - - - - Isotropic	Platy and prismatic minerals < 10%, which may occur as discontinuous streaks or may be randomly oriented. Rock fractures are curved or folded. Usually found in high-grade regional metamorphic regions or in contact metamorphic zones. Typical rocks: **Quartzite, hornfels, granulite.**	1 - 1.2
Weakly foliated Fi = 1.5 - 3 - - - - - - - - - - - Fairly anisotropic	Platy and prismatic minerals 10 - 20%. Compositional layering is evident, but mechanically insignificant. Usually found in high-grade regional metamorphic regions. Typical rocks: **Quartzofeltspatic gneiss, mylonite, migmatite**	1.2 - 1.5
Moderately foliated Fi = 3 - 6 - - - - - - - - - - - Moderately anisotropic	Platy and prismatic minerals 20 - 40%. Thin to thick folia, occasionally discontinuous. Foliation is usually mechanically passive. Found in rocks formed by medium to high-grade regional metamorphism. Typical rocks: **Schistose gneiss, quartzose schist.**	1.5 - 2
Strongly foliated Fi = 6 - 9 - - - - - - - - - - - - Highly anisotropic	Platy and prismatic minerals 40 - 60%. Thin wavy continuous folia which may be mechanically significant. Usually formed under medium-grade regional metamorphic conditions. Typical rocks: **Mica schist, hornblende schist.**	2 - 2.5
Very strongly foliated Fi > 9 - - - - - - - - - - - - Very highly anisotropic	Platy and prismatic minerals > 60% occurring as very thin, continuous folia. Foliation is perfect and mechanically significant. Found in rocks formed by dynamic or low-grade regional metamorphism.	> 2.5

	Typical rocks: **Slate, small folded phyllite.**	

Fi = foliation index

The foliation index can be found from thin section analysis by measuring the mineral composition and the shape of the minerals as presented by Tsidzi (1986).
From the values of fA in Table A3-9 combined with the content of platy and prismatic minerals, the following equation is found:

$$fA = 1 + 2.5\ c/100 \qquad \text{eq. (A3-4)}$$

where c is the content of platy and prismatic minerals in % .

According to Tsidzi (1989), the strength anisotropy index fA is directly proportional to foliation regardless of the physical condition of rock. Thus, the minimum compressive strength of the foliated rock can roughly be assessed as

$$\sigma_{c\ min} = \sigma_{c\ max}/fA = \sigma_{c\ max}/(1 + 2.5\ c/100) \qquad \text{eq. (A3-5)}$$

In their works on anisotropic rocks Sing et al. (1989) have introduced the anisotropy ratio is defined as $R_c = \sigma_{c\ 90}/\sigma_{c\ min}$, where $\sigma_{c\ 90}$ is the uniaxial compressive strength measured at right angle to the schistosity or bedding. Their results shown in Table A3-10 indicate that the strength reduction caused by the anisotropy (R_c) is considerably higher than fA in Table A3-9.

TABLE A3-10 CLASSIFICATION OF ANISOTROPY
(from Sing et al., 1989 and Ramamurthy et al., 1993)

Anisotropy ratio R_c	**Classification**	**Rock types**
1 - 1.1	Isotropic	
1.11 - 2.0	Low anisotropy	Shales
2.01 - 4.0	Medium anisotropy	
4.01 - 6.0	High anisotropy	Slates
> 6.0	Very high anisotropy	Phyllites

A possible reason for this is that the rocks tested by Tsidzi generally exhibit stronger small scale foldings which will reduce the effect of the foliation. Also possible differences in moisture content may influence on the results. None of the authors have specified under which conditions the test were carried out.

The sonic velocity anisotropy coefficient for some rocks is presented in Table A3-11 , which shows values lower than the strength anisotropy factor of Tsidzi in Table A3-9.

TABLE A3-11 SONIC ANISOTROPY COEFFICIENTS FOR SOME ROCKS
(from Lama and Vutukuri, 1978)

ROCK	**anisotropy coefficient** $V_{\circ}/V_{Á}$.	**ROCK**	**anisotropy coefficient** $V_{\circ}/V_{Á}$.
Austin chalk	1.17	Anhydrites	1.12 - 1.16
Limestones	1.04 - 1.30	Marl	1.10
Salt	no anisotropy	Sandstones	1.0 - 1.19
Shales	1.07 - 1.40	Gneisses	1.20 - 1.27
Mica schist	1.36	Granodiorite	1.33
Serpentine	1.18		

In an earlier work performed by Bergh-Christensen (1968) correlations between compressive strength anisotropy and wave velocity anisotropy were investigated. As shown in Fig. A3-7 there is no clear correlation between sound velocity anisotropy
(V_{max}/V_{min}) and strength anisotropy ($\sigma_{c\,max}/\sigma_{c\,min}$). Most of the data fall, however, within the to lines indicated in Fig. A3-7, which can be expressed as:

$$V_{max}/V_{min} < \sigma_{c\,max}/\sigma_{c\,min} < 4 \cdot V_{max}/V_{min} - 3 \qquad \text{eq. (A3-6)}$$

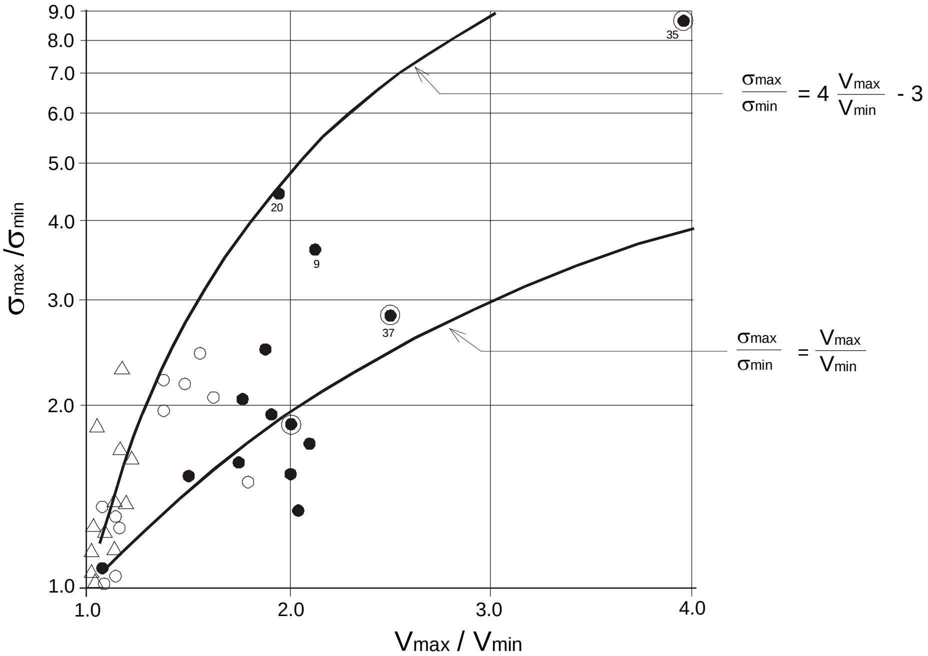


Fig. A3-7 Correlation between strength and sound anisotropy. The symbols indicate measured rock blasting indexes. (from Berg-Christensen, 1968)

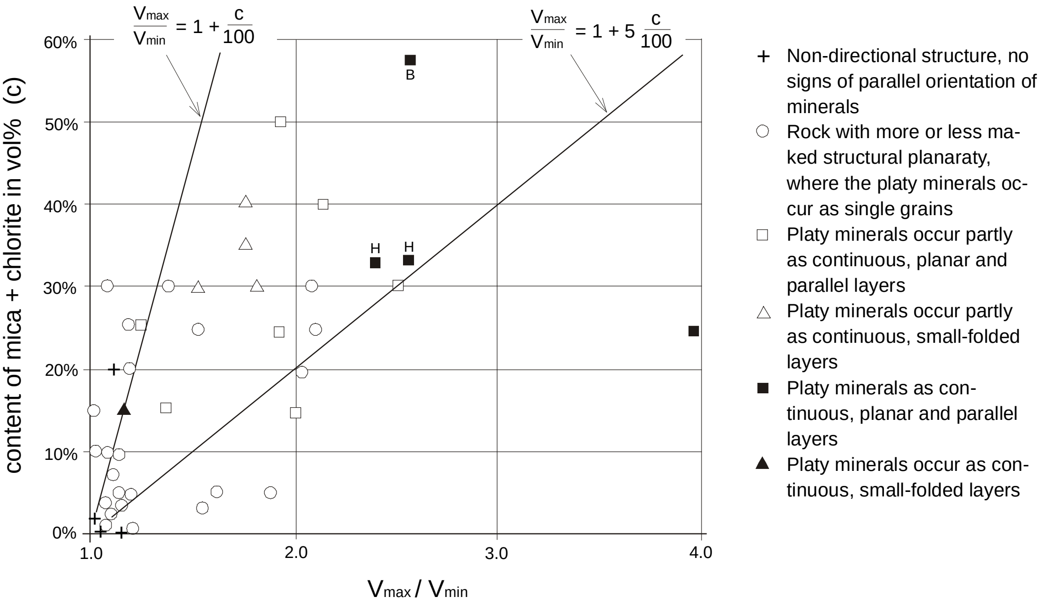


Fig. A3-8 Correlation between content of flaky minerals (mica and chlorite) and sound velocity ratio for various textures of rock (from Berg-Christensen, 1968).
(In the groups with continuous layers of flaky minerals there are rocks that do not follow the average trend. One explanation can be that the classification into the structural groups are made from simple observations (personal communication with Bergh-Christensen, 1992))

In Fig. A3-8 showing the relation between the sonic velocity anisotropy and the content of flaky minerals it is seen that most data lies within the two lines represented by the following expression:

$$1 + c/100 \; < \; V_{max} / V_{min} \; < 1 + 5c/100 \qquad \text{eq. (A3-7)}$$

where V is the seismic velocity, and c is the content of flaky minerals (mica and chlorite) given in %.

By roughly combining the lowest with the highest values in eq. (A3-6) and eq. (A3-7) it is possible to find the rock anisotropy factor, fA, from the content of flaky minerals given as:

$$1 + 4c/100 \; < \; fA < 1 + 5c/100 \qquad \text{eq. (A3-8)}$$

This equation gives somewhat higher values for fA than eq. (A3-5) found by Tsidzi; for example, for 50% content of flaky minerals fA = 3 - 3.5 with eq. (A3-8), compared to Tsidzi's fA = 2.25. Eq. (A3-8) gives, however, lower values than R_c (see Table A3-10).

From the foregoing it is apparent that it is not a well defined, simple method to estimate the effect of anisotropy upon strength. The anisotropy factor varies with the method used, in addition the definition of the anisotropy class seems to be very approximate as possible small-scale folding is not included. The method presented by Tsidzi in eq. (A3-5) seems, however to be better documented and may, therefore, be the best for rough estimates. More investigations and are required to arrive at better expressions for the rock anisotropy.

1.6.4 Reduction in strength from weathering and alteration

Weathering of rocks is a result of the destructive processes from atmospheric agents at or near the Earth's surface, while *alteration* is typically brought about by the action of hydrothermal processes. Both processes produce changes of the mineralogical composition of a rock, affecting colour, texture, composition, firmness or form; features that result in reduction of the mechanical properties of a rock. Deterioration from weathering and alteration generally affects the walls of the discontinuities more than the interior of the rock (Piteau, 1970).

TABLE A3-12 ENGINEERING CLASSIFICATION OF THE WEATHERING OF ROCKS (from Lama and Vutukuri, 1978)

CLASSIFICATION	DESCRIPTION
Unweathered	No visible signs of weathering. Rock fresh, crystals bright. Few discontinuities may show slight staining.
Slightly weathered	Penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material. Discontinuities are discoloured and discoloration can extend into rock up to a few mm from discontinuity surface.
Moderately weathered	Slight discoloration extends through the greater part of the rock mass. the rock is not friable (except in the case of poorly cemented sedimentary rocks). Discontinuities are stained and/or contain a filling comprising altered materials.
Highly weathered	Weathering extends throughout rock mass and the rock material is partly friable. Rock has no lustre. All material except quartz is discoloured. Rock can be excavated with geologist's pick.
Completely weathered	Rock is totally discoloured and decomposed and in a friable condition with only fragments of the rock texture and structure preserved. The external appearance is that of a soil.
Residual soil	Soil material with complete disintegration of texture, structure and mineralogy of the parent rock.

Characterization of the state of weathering or alteration both for the rock material and for the discontinuities is therefore an essential part of the rock parameters to be applied (ISRM, 1978). In rock engineering and construction it is seldom of interest to describe whether the process of weathering or alteration has been acting; the main topic is to characterize the result. Knowledge of the processes, which have taken place, can however be important for the understanding and interpretation of the geological conditions and of the condition of the rocks likely to be found.

In general, the degree of weathering is usually estimated from visual observations, where only the qualitative information is required. Table A3-12 shows classification of weathering/alteration similar to that presented by ISRM (1978). A more precise characterization of alteration and weathering can be found from analysis of thin sections in a microscope.

Papadopoulos and Marinos (1992) have made tests on a wide variety of petrological rock types, consisting of clayey schists, phyllites, mica schists, sandstones with secondary anisotropy due to weathering from tectonic action. Some of their results are shown in Fig. A3-9 where the grade of weathering is according to the ISRM (1978) or the similar classification by Lama and Vutukuri (1978) in Table A3-12.

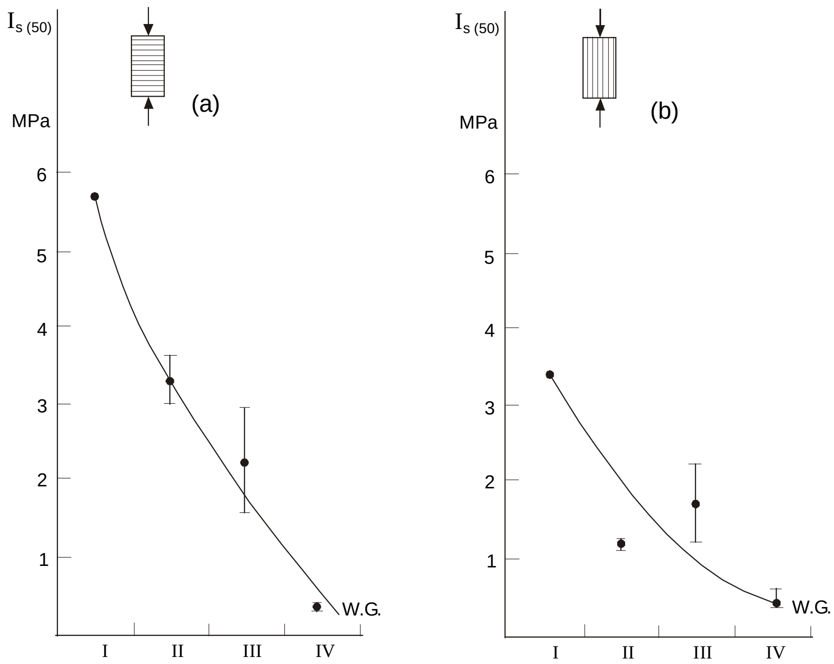


Fig. A3-9 Correlation between weathering grade I - IV and point-load strength values Is_{50} (from Papadopoulos and Marinos, 1992)

The reduction factor from weathering is found as the ratio $Is_{50fresh} / Is_{50\ weathered}$ in Fig. A3-9. Results from tests parallel and normal to anisotropy are shown in Table A3-13, in which also the rating of a rock weathering factor (fW) has been suggested. From this the point load strength of the weathered or altered rock can roughly be found as

$$Is_{50} = Is_{50\ fresh} / fW \qquad \text{eq. (A3-9)}$$

Assuming that the strength reduction for the compressive strength is similar it is approximately found from

$$\sigma_c = \sigma_{c\ fresh} / fW = k_{50} \times Is_{50\ fresh} / fW \qquad \text{eq. (A3-10)}$$

The suggested rating of the weathering/alteration factor in Table A3-13 is, however, based on very few data and it is therefore considered very rough, especially for high grades of weathering. It should, therefore, mainly be applied for rough calculations before better strength data are available.

TABLE A3-13 SUGGESTED WEATHERING/ALTERATION FACTOR (fW) OF ROCKS, fW
(worked out partly from ISRM (1978) and Papadopoulos and Marinos (1992).

GRADE and TERM	**DESCRIPTION** (from ISRM, 1978)	**REDUCTION FACTOR** (from Papadopoulos and Marinos, 1992) TEST DIRECTION		**SUGGESTED RATING OF** fW
		Normal	Parallel	
I Fresh	No visible sign of rock material weathering.	1	1	1
II Slightly	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker externally than in its fresh conditions.	1.7	1.8	1.75
III Moderately weathered	Less than half the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	2.6	2	2.5
IV Highly	More than half the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	14	10	10

1.7 Summary

In addition to compression tests in laboratory the uniaxial compressive strength of the rock may be found from several other methods which have been shortly described. These are:

- Compressive strength estimated from the point-load strength (Is), given as
  $$\sigma_c = k_{50} \times Is_{50}$$
  where values of k_{50} varying with the rock strength related to 50 mm samples, have been suggested.
- Compressive strength found from the Schmidt hammer rebound number.
- Compressive strength assessed from simple field test using a geological hammer.

Where test data of the rock is not available the uniaxial compressive strength may be estimated from the geological rock name and additional information on its structure and weathering/alteration. For anisotropic and weathered/altered rocks a rough estimate of the uniaxial compressive strength can be found from

$$\sigma_c = \sigma_{c50} /(fA \times fW) \qquad \text{eq. (A3-11)}$$

where σ_{c50} can be found for fresh rocks from published strength tables, and
fA, fW are the foliation anisotropy and weathering/alteration factors, respectively. Suggested ratings have been given for both factors, but additional investigations are required to improve these approximate values.

The lowest value ($\sigma_{c\,min}$) is applied in RMi. Therefore, it is important to check if the effect of anisotropy is included in σ_{c50}. It is also important that the rock description - on which the estimates are based - clearly delineate the composition and structure of the rock, and that the applied terms and characterizations of the rock are well defined. Further information on useful description of rocks is outlined in Section 5.

The content of moisture tends to reduce the compressive strength of most rocks.

2 METHODS TO DETERMINE THE JOINT CONDITION FACTOR (jC)

"The success of the of the field investigation will depend on the geologist's ability to recognise and describe in a quantitative manner those factors which the engineer can include in his analysis."
Douglas R. Piteau, 1970

The usually large number of joints with various conditions involved in a rock mass, cause that simplifications have to be made and that rapid and inexpensive measurements are preferred. The joint condition factor, jC, is meant to represent the main inherent variables of the friction properties of joints in a rock mass. Basically, the condition of joints are made up of the following parameters (see alt. A in Fig. A3-11):

- The roughness of the joint walls, given as the joint roughness factor (jR), similar to Jr in the Q-system. jR consists of:
  - smoothness (or unevenness) of the joint wall surface, and
  - waviness (or planarity) of the joint wall plane.
- The character of the joint wall including possible filling and its thickness, expressed in the joint alteration factor (jA).[1]
- Length and continuity of the joints, expressed as the joint size factor, jL, which is considered a scale and geometry factor to mainly include the different importance between large, pervasive joints and small, irregular joints on rock mass behaviour.

Similar, as for the Jr/Ja in the Q system, the ratio jR/jA is roughly a function of tan ϕ, the peak friction coefficient of the joint. Thus, also measurements of the joint friction angle can be used to find the ratio jR/jA as shown in Fig. A3-12. This method (alt. B) is based on measurements of the joint roughness combined with input of the actual type and size of joints. Alt. A is based wholly on observations.

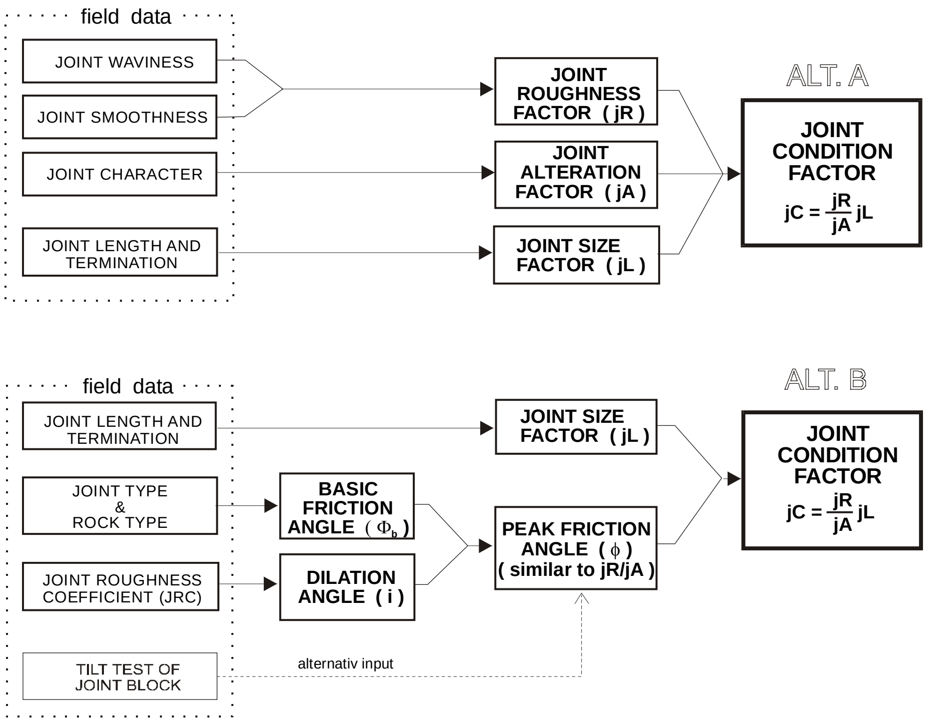


Fig. A3-11 The two main methods to find the joint condition factor, jC.

---

[1] jA is similar to Ja in the Q system, but some changes have been made to adapt it as input to RMi.

Alt. A is described in Section 2.1 and 2.2, alt. B in Section 2.3. The joint size factor, jL, which is used in both alternatives, is described in Section 2.4

2.1 Estimating the joint roughness factor (jR)

The *roughness* of joint walls is characterized by a large scale *waviness* and a small scale *smoothness* or unevenness, ISRM (1978), see Fig. A3-12. During shear displacement the waviness undulations, if locked and in contact, cause dilation since they are too large to be sheared off, while the asperities of the smoothness tend to be damaged unless the joint walls are of high strength and/or the stress levels are low. In practice, it is seldom possible to observe and measure both these features along the entire joint. Some sort of simplification has therefore to be made as outlined in this section.

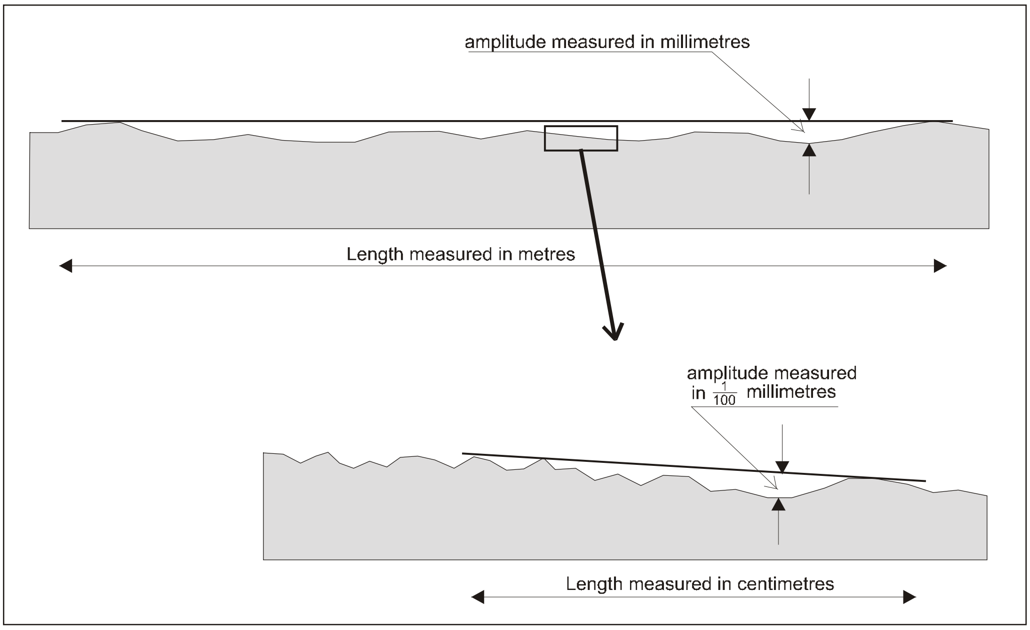


Fig. A3-12 The joint wall features can be characterized by the large scale waviness and the small scale smoothness (or unevenness).

2.1.1 Field measurements of large scale roughness

Accurate measurements of joint waviness in rock exposures is relatively time-consuming by any of the currently available procedures (Stimpson, 1982). The three most practical methods are:

1. To estimate the overall roughness (undulation) angle by taking measurements of joint orientation with a Clar Compass to which the base plates of different dimensions are attached, see Fig. A3-13 left.
2. To measure the roughness along a limited part of the joint using a feeler or contour gauge to draw a profile of the surface, Stimpson (1982). Also Barton and Choubey (1977) makes use of this method especially in connection with core logging.
3. To reconstruct a profile of the joint surface from measurements of the distance to the surface from a datum (typically a rod or rule laid or supported over the joint).

The first technique provides information on large scale roughness angles, but does not give a record of the joint profile. It is applicable primarily to large exposures of joint planes. Fecker and Rengers (1971) have from measurements using a profilograph and geological compass, shown how these results can be applied.

The <u>second</u> is a rapid method, where a few decimetres profile along the joint wall surface is obtained by a contour gauge and pattern maker. The method is well applicable for joint smoothness measurements on drill cores. By comparing the profile obtained for the joint with standard roughness profiles, for example of JRC or Jr in Fig. A3-14, the actual roughness value can be easily determined.

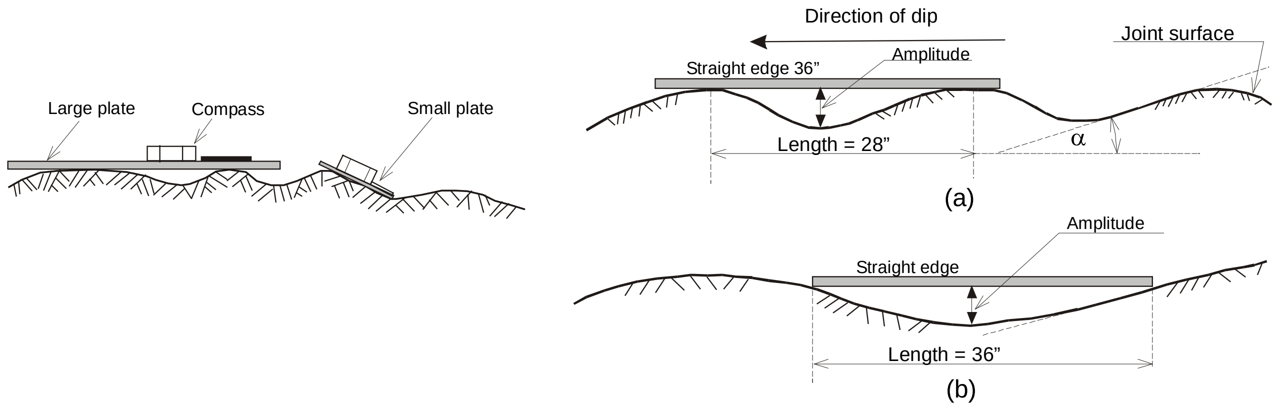


Fig. A3-13 **Left:** Measurement of different scales of joint waviness (from Goodman, 1987).
Right: Principle for the measurement of waviness by a straight edge (from Piteau, 1970).

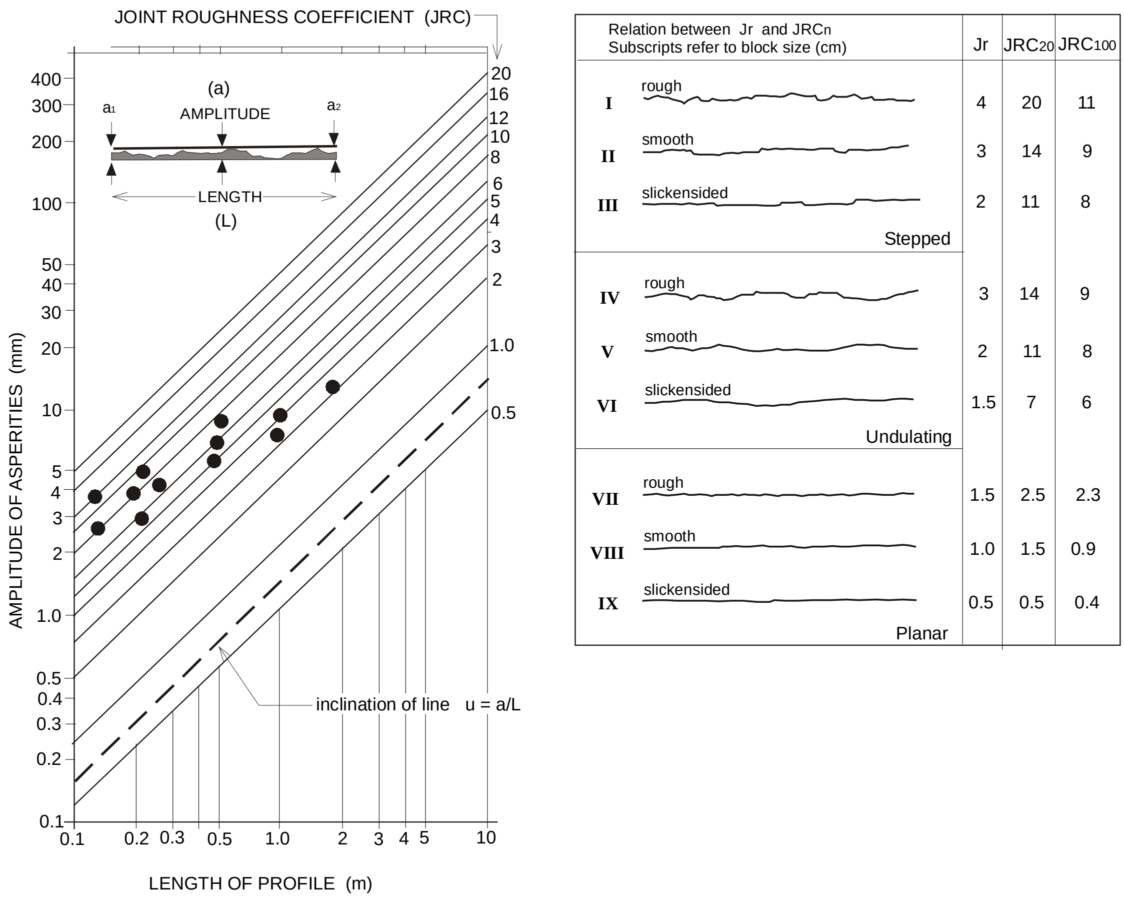


Fig. A3-14 **Left:** Diagram presented by Barton and Bandis (1992) to estimate JRC for various measuring lengths. The inclined lines exhibit almost a constant undulation as indicated.
Right: Relationships between Jr in the Q-system and the 'joint roughness coefficient' (JRC) for 20 cm and 100 cm sample length (from Barton and Bandis, 1992).

The third technique is generally laborious and, unless the data points are closely spaced along the joint, it does not provide a detailed profile of the small scale roughness. Robertson (1970) and Piteau (1970) have introduced measurement of waviness by using a standard length of 0.9 m (36 inch) straight edge placed on the exposed joint surface in a direction normal to the strike (i.e., down dip).

Amplitude (a) of the wave is a measure of the maximum offset under the straight edge as shown for two different cases in Fig. A3-13 right. This dimension is measured in millimetre. It is related to the length (L), which is recorded as either 1) the distance between adjacent points of contact on the peaks of the wave, Fig. A3-13 right (a), or 2) the standard length adopted, Fig. A3-13 right (b). Also ISRM (1978) has described a similar method to measure the waviness of the joint plane.

Kikuchi et al., (1985) has measured waviness of joint plane by selecting portions on a joint plane exposed to the ground, using a 2 m measuring scale. The height of the exposed portions at 2 cm intervals in the direction of dip has been measured to draw the roughness of the joint plane. Also Barton (1982) and Barton and Bandis (1990) make use of a ruler to determine the joint roughness coefficient JRC. Fig. A3-14 shows how different lengths of the ruler can be used to determine JRC. As indicated on this diagram the inclined lines approximately follow the expression for the undulation factor u = a/L presented later in eq. (A3-12) has been used to find the ratings of the joint waviness factor, jw, where u is applied as shown in Table A3-14.

2.1.2 The joint waviness factor (jw)

As waviness does not change with displacements along the joint surface, no shearing takes place through asperities (Piteau, 1970). The result is that waviness is considered to modify the apparent angle of dip of the joint but not the joint frictional properties. Tsidzi (1986, 1987, 1991) has observed that waviness is a more characteristic feature of the foliation surface in many metamorphic rocks than the smoothness of the surface.

Waviness of the joint appears as undulations from planarity of the joint wall. Ideally, the joint waviness should be measured as the ratio between max. amplitude over the length of the joint. As it is seldom possible to observe the whole joint plane, a simplified measurement is to find the ratio between max. amplitude and a reduced measured length along the joint plane called the undulation factor

$$u = \frac{\text{amplitude from planarity (a)}}{\text{measured length along joint (L)}} \qquad \text{eq. (A3-12)}$$

This expression has been used to characterize the waviness of joint plane. From combination of the two diagrams in Fig. A3-14 the division and ratings in Table A3-14 have been worked out. The division here of the undulation is based on the characterization presented by Milne et al. (1992) related to 1 m profile length:

Wavy joints with undulation, u > 2%
Planar to wavy joints, with u = 1 - 2%
Planar joints, defined as u < 1%.

Their measurements are also based on the 'joint roughness coefficient' (JRC) which is described in Section 2.3.

TABLE A3-14 THE JOINT WAVINESS FACTOR (jw). THE RATINGS ARE BASED ON Jr IN THE Q-SYSTEM.

TERM FOR WAVINESS	waviness factor undulation	**jw**
Interlocking (large scale)		3
Stepped		2.5
Large undulation	u > 3 %	2
Small - moderate undulation	u = 0.3 - 3 %	1.5
Planar	u < 0.3 %	1

The longest possible ruler should be applied in measurement of waviness. In many cases, however, the determination of (jw) is done from visual observations alone. Practice from ruler measurements may reduce the possible errors in such cases.

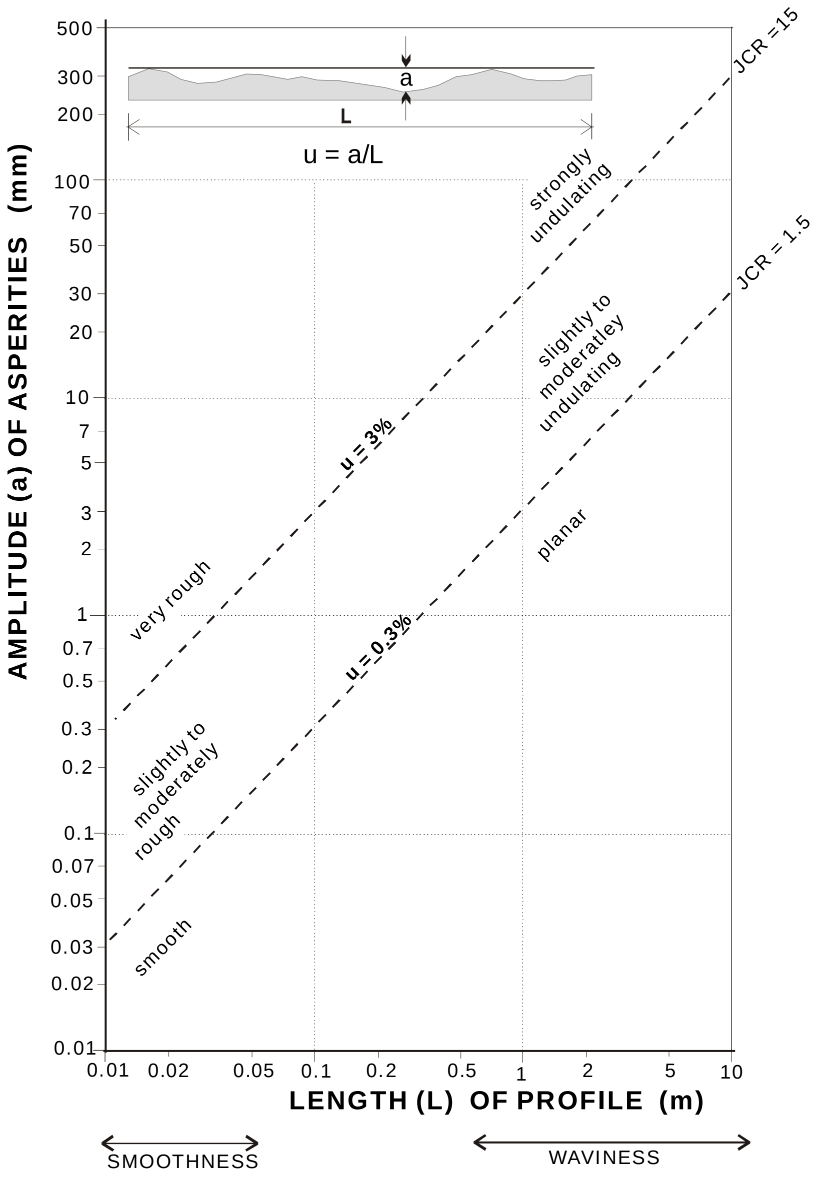


Fig. A3-15 Waviness and smoothness (large and small scale roughness) based on the JRC chart in Fig. A3-14.

2.1.3 The joint smoothness factor (js)

Small asperities or second order projections are designated *smoothness* or unevenness. If the joint surfaces are clean and closed these small asperities interlock to strongly contribute to the shear resistance especially at low stresses (Barton and Chubey, 1977; Barton, 1976, 1987, 1990b, 1993). Smoothness asperities usually have a base length of some centimetres and amplitude measured in hundreds of millimetres and are readily apparent on a core-sized exposure of a discontinuity (see Fig. A3-12 and A3-15).

As indicated in Fig. A3-12 and A3-15 the 'sample length' for smoothness is in the range of a few centimetres. There is a general problem to arrive at a numerical estimate of joint smoothness from measurements or visual observations of the joint wall surface. A possible solution is to simply touch the surface with the finger and compare it with a reference surface of known roughness, for example sand papers of various abrasivity (mesh) as indicated in Table A3-16.

The terms and ratings defined in Table A3-16 are based on Jr in the Q-system (Barton et al., 1974), while the description is also partly based on Bieniawski (1984).

TABLE A3-16 THE JOINT SMOOTHNESS FACTOR (js). THE RATINGS ARE THE SAME AS FOR Jr IN THE Q-SYSTEM. (The description is partly based on Bieniawski, 1984).

TERM FOR SMOOTHNESS	DESCRIPTION	The smoothness factor js
Very rough	Near vertical steps and ridges occur with interlocking effect on the joint surface.	3
Rough	Some ridge and side-angle steps are evident; asperities are clearly visible; discontinuity surface feels very abrasive (rougher than sandpaper grade 30)	2
Slightly rough	Asperities on the discontinuity surfaces are distinguishable and can be felt (like sandpaper grade 30 - 300).	1.5
Smooth	Surface appear smooth and feels so to the touch (smoother than sandpaper grade 300).	1
Polished	Visual evidence of polishing exists. This is often seen in coatings of chlorite and specially talc	0.75
Slickensided	Polished and striated surface that results from friction along a fault surface or other movement surface.	0.6 - 1.5 [1)]

[1)] Rating depends on the actual shear in relation to the striations.

2.1.4 The joint roughness factor (jR) found from *jw* and *js*

As described in the foregoing the smoothness and waviness can both be characterized by the ratio between the amplitude of asperities and the measuring length as shown in Fig. A3-15. A similar principle has been presented by Milne et al. (1992) who have used the joint roughness coefficient (JRC) to also assess small scale roughness (i.e. smoothness) using 10 cm profile lengths with the following division:

- smooth surfaces have JRC < 10 (undulation u < 2%)

- rough surfaces JRC > 10 (undulation u > 2%)

The joint roughness factor, jR, is the product of the smoothness and waviness factors:

$$jR = js \times jw \qquad \text{eq. (A3-13)}$$

which is shown in Table A3-17.

TABLE A3-17 JOINT ROUGHNESS FACTOR (jR) FROM SMOOTHNESS AND WAVINESS. THE STRUCTURE OF THE FACTOR AND ITS RATING ARE SIMILAR TO Jr IN THE Q-SYSTEM.

	waviness				
smoothness	planar	slightly to moderately undulating	strongly undulating	stepped	interlocking (large scale)
very rough	3	4	6	7.5	9
rough	2	3	4	5	6
slightly rough	1.5	2	3	4	4.5
smooth	1	1.5	2	2.5	3
polished	0.75	1	1.5	2	2.5
slickensided	0.6 - 1.5	1 - 2	1.5 - 3	2 - 4	2.5 - 5
For filled joints without contact between joint walls: jR = 1					

Joint roughness includes the condition of the joint wall surface both for filled and unfilled (clean) joints. For joints with filling which is thick enough to avoid contact of the two joint walls, any shear movement will be restricted to the filling, and as described later, the joint roughness will then have minor or no importance. In the cases of filled joints it is often difficult or impossible to measure the smoothness and often also the waviness. Therefore the roughness factor is defined as jR = 1 as in the Q system.

The joint roughness coefficient, jR (or Jr) can also be found from measured values of JRC using Fig. A3-14 or A3-15.

The classification systems which often make use of many a large amount of input data, the ratings for roughness are mostly found from observations. It is, therefore, not common to make detailed measurements of joint roughness profiles.

2.2 Estimating the joint alteration factor (jA)

> *"It is often more important to characterize discontinuities according to surface character as it is to note their scale parameters."*
> Tor L. Brekke and Terry R. Howard (1972)

The joint alteration factor is a collective parameter including the strength of the joint wall and the possible joint filling, whether it is a clean or a filled joint. The following features are included in this complex factor:

- The condition of the surface in *clean* joints, i. e. joints without coating or filling; with indication if alteration of the joint wall may be other than the rock.
- The type of *coating* on the joint surface.
- The type of *filling* in joints and its thickness.

2.2.1 Clean joints

Clean joints are without fillings or coatings. For these joints the compressive strength of the rock wall is a very important component of shear strength and deformability where the walls are in direct rock to rock contact (ISRM, 1978).

Close to the surface in outcrops it is imperative not to confuse clean discontinuities with "empty" discontinuities where filling material has been leached and washed away due to surface weathering. Clean joints can be:

a. *Healed or welded joints.* Joints, seams and sometimes even minor faults may be healed through precipitation from solutions of quartz, epidote or calcite. In such cases the joint plane can be regarded more appropriately as a plane of reduced strength.
b. Healed joints may, however, have broken up again, forming new surfaces. Also, it should be emphasized that quartz and calcite may well be present in a discontinuity without healing it.
c. *Fresh rock walls*. These are joint walls of unweathered or unaltered rock. They may, however, show staining (rust) on the surfaces.
d. *Altered or weathered rock walls*. Some clean joints may show alteration of the rock material on the joint surface (Piteau, 1970). The rock surface in these joints can be in the same condition as the rock elsewhere. Often, however, when *weathering* or *alteration* has taken place it is more pronounced along the joint surface than in the rock. This results in a wall strength often considerably lower than that of the fresher rock found in the interior of the rock blocks. The degree of weathering is usually estimated from visual observations. It can partly be quantified applying the ISRM classification of alteration and weathering (see Table A3-12), and should be applied in jA where it differs from the alteration of the rock material, as is shown in Table A3-21.

2.2.2 Coated joints

Coating means that the joint surfaces have a thin layer or 'paint' with some kind of mineral. The coating, which is not thicker than a few millimetres, can consist of various kinds of mineral matter, such as chlorite, calcite, epidote, clay, graphite, zeolite. Mineral coatings will affect the shear strength of joints to a marked degree if the surfaces are planar. The properties of the coating material may dominate the shear strength of the joint surface, especially weak and slippery coatings of chlorite, talc and graphite when wet.

2.2.3 Filled joints

Filling or *gouge* when used in general terms, is meant to include any material different from the rock thicker than coating which occurs between two discontinuity planes. Thickness of the filling or gouge is taken as the width of that material between sound intact rock. Table A3-18 shows the classification of joint or seam thickness presented by ISRM (1978).

Unless discontinuities are exceptionally smooth and planar, it will not be of great significance to the shear strength that a 'closed' feature is 0.1 mm wide or 1.0 mm wide, ISRM (1978). (However, indirectly as a result of hydraulic conductivity, even the finest joints may be significant in changing the normal stress and therefore also the shear strength.)

TABLE A3-18 SEPARATION OF DISCONTINUITY WALLS (from Bieniawski 1984)

Very tight	< 0.1 mm
Tight	0.1 - 0.5 mm
Moderately open	0.5 - 2.5 mm
Open	2.5 - 10 mm
Very open	10 - 25 mm

Aperture is the perpendicular distance separating the adjacent rock walls of an *open* discontinuity, in which the intervening space is air or water filled. Aperture is thereby distinguished from the width of a filled discontinuity (ISRM, 1978).)

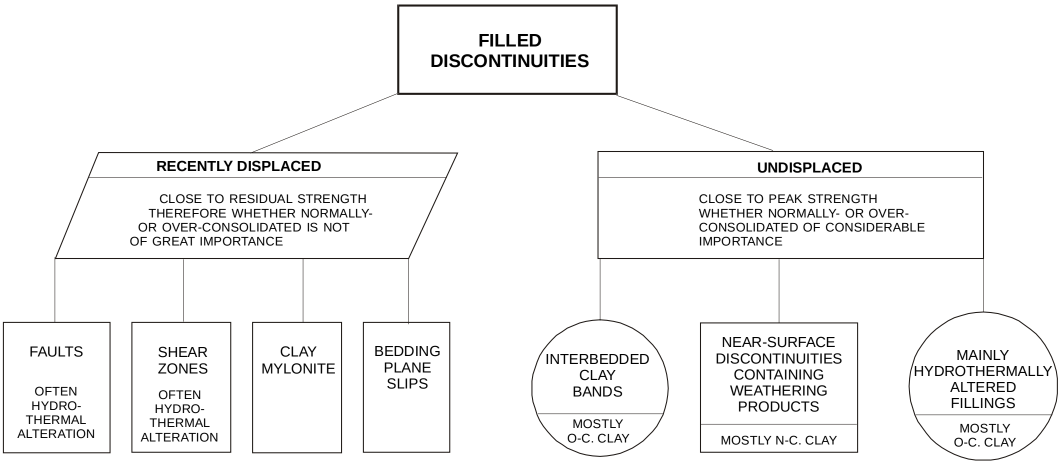


Fig. A3-16 Simplified division of filled discontinuities into displaced and undisplaced, normally- and over-consolidated categories (from Barton, 1974).

TABLE A3-19 MAIN TYPES OF COATING AND FILLING MATERIALS AND THEIR PROPERTIES, (mainly based on Brekke and Howard, 1972)

TYPE OF MINERAL FILLING	**PROPERTIES**
Chlorite, talc, graphite	Very low friction materials, in particular when wet.
Inactive clay materials	Weak, cohesion materials with low friction.
Swelling clay	Exhibits both a very low friction and swell with loss of strength because of swelling together with considerable swelling pressure when confined.
Calcite	May, particularly when being porous or flaky, dissolve during the lifetime of a construction in rock, which reduces its contribution to the strength of the rock mass.
Gypsum	May behave in the same way as calcite.
Sandy or silty materials	Cohesionless, friction materials. A special occurrence of these are the thicker fillings of *altered or crushed materials* being cohesionless (sand-like) materials which may run or flow immediately after exposure by excavation.
Epidote, quartz and	May cause healing or welding of the joint, resulting in an increased shear strength other hard materials of the joint.

TABLE A3-20 PARTICLE SIZE ACCORDING TO THE MODIFIED WENTWORTH SCALE (partly from ISRM, 1978)

TERM	**DIAMETER**	**VOLUME**
coarse sand	0.6 - 2 mm	0.13 mm3 - 4.6 mm^3
medium sand	0.2 - 0.6 mm	0.0046 mm3 - 0.13 mm^3
fine sand	0.06 - 0.2 mm	
silt, clay	< 0.06 mm	

The effect of joint filling on the strength properties of a joint is of outstanding importance. If the gouge is sufficiently thick, the filling (gouge) controls entirely the shear strength of the discontinuity. With decreasing thickness, the asperities of the rock wall tend to become more interlocked, and both the filling and the rock material contribute to the discontinuity shear strength. Thus, the main cases with respect to gouge thickness which are worth examining are (Piteau, 1970):

a. No contact between the joint walls. The sliding plane passes entirely through gouge; the shear strength is dependant only on the gouge material and no modification is considered for roughness.
b. Partly contact between the joint walls. The sliding plane passes partly through gouge and partly along the joint wall rock. The shear strength will be more complex being made up of contributions of both gouge and wall rock.
c. Gouge is present but very thin as a coating; gouge is considered only as modification of the friction angle.

In addition to the thickness of the filling the type of filling materials with different behaviour and/or properties must be distinguish. The division in Table A3-19 covers most types of gouge and coating materials. The size of the particles, fragments or blocks in the filling can be characterized according to Table A3-20 or A3-26.

2.2.4 <u>Characterization and rating of the joint alteration factor (jA)</u>

The characterization of this complex parameter and its numerical values are mainly based on the Ja in the Q-system and the features described above. The same main grouping and ratings are applied, but some changes have been done to fit jA into the RMi system. Also the layout in Table A3-21 is changed compared to the Q-system to possibly make field observations easier and quicker. The main changes to Ja in the Q-system are:

1. The *weathering/alteration* of the rock in the joint wall.
2. As the RMi system has included the rock material (with its possible alteration/weath-ering), it is only where the weathering of the clean joint wall is different from the rock, that jA influences.
3. Zones or bands of disintegrated, crushed rock or clay are not included as such weakness zones generally require special characterization as outlined in Appendix 2 and Chapter 6.

The joint alteration factor depends on the thickness, strength and basic friction angle of any material on the joint surfaces. There are often difficult to determine if sufficient material exists to reduce joint strength enough to warrant changing the jA rating from 1.0 to 4.0. Some preliminary guidelines have been published by Milne and Potvin (1992) to augment the standard joint alteration descriptions to better quantify the joint surface condition in this range:

Can be scratched with a knife	Ja = 1.0 - 1.5
Can be scratched with a fingernail and feels slippery	Ja = 2.0
Can be dented with a fingernail and feels slippery	Ja = 4.0

These guidelines which do not remove the subjectivity from this parameter, only assist in finding a small range of the jA ratings.

Major individual discontinuities (singularities) should as mentioned earlier be recorded on an individual basis.

TABLE A3-21 CHARACTERIZATION AND RATING OF JOINT CHARACTER FACTOR (jA)
(Partly based on Ja in the Q-system)

CONTACT BETWEEN THE TWO JOINT WALLS		
WALL CHARACTER	DESCRIPTION	jA
CLEAN JOINTS		
-Healed or "welded" joints	Non-softening, impermeable filling (quartz, epidote etc.)	0.75
-Fresh rock walls	No coating or filling on joint surface, except of staining (rust)	1
-Alteration of joint wall:		
1 grade more altered	The joint walls shows 1 grade stronger alteration than the rock	2
2 grades more altered	The joint walls shows 2 grades stronger alteration than the rock	4
COATING OR THIN FILLING		
-Sand, silt, calcite etc.	Coating of friction materials without clay	3
-Clay, chlorite, talc etc.	Coating of softening and cohesive minerals	4

FILLED JOINTS WITH PARTLY OR NO WALL CONTACT			
TYPE OF FILLING MATERIAL	DESCRIPTION	**Partly wall contact** thin fillings*) (< approx. 5 mm) **jA**	**No wall contact** thick filling or gouge **jA**
Sand, silt, calcite etc	Filling of friction materials without clay	4	8
Compacted clay materials	"Hard" filling of softening and cohesive materials	6	10
Soft clay materials	Medium to low over-consolidation of filling	8	12
Swelling clay materials	Filling material exhibits clear swelling properties	8 - 12	12 - 20

*) Based on division in the RMR system (Bieniawski, 1973)

2.3 Estimating the ratio jR/jA from friction angle recordings

Patton (1966) differentiates between first and second order projections on the joint wall surfaces, corresponding to large scale undulations, given as waviness, and the small scale irregularities or unevenness (smoothness). Surface roughness was determined by Patton as an angular measurement of asperities from the general dip of the joint. Effective friction angle of a rock surface is the sum of the basic friction angle (Φ_b) and the roughness angle
(i). The effect of (i) is reflected as dilation under low normal loads, whereas at very high normal loads there is very little dilation as the asperities are sheared through. At very low normal stresses the small scale roughness (smoothness) is mainly contributing to the shear strength of joints, whereas at higher stress the waviness is of main importance.

ISRM (1980) has classified the angle of friction as shown in Table A3-22.

TABLE A3-22 CLASSIFICATION OF THE ANGLE OF FRICTION FOR JOINTS (from ISRM, 1980).

Interval	**Descriptive terms**
$> 45^o$	very high
$35 - 45^o$	high
$25 - 35^o$	moderate
$15 - 25^o$	low
$< 15^o$	very low

As the peak friction angle of joints can be expressed as

$$\phi = \phi_b + i$$

the ratio jR/jA can be found from

$$jR/jA \approx \tan^{-1} \phi = \tan^{-1} (\phi_b + i) \qquad \text{eq. (A3-14)}$$

where ϕ_b is the basic friction angle of the joint, and
i is the (peak) dilation angle.

The value of ϕ_b is mostly in the range 21^o - 40^o (Goodman, 1989). For most smooth unweathered rock surfaces it varies between 25^o and 35^o, see Table A3-23. The values of ϕ_b can normally be estimated with help of the data listed here, unless the joint walls are strongly weathered, coated or the joint contain filling. Frequently, ϕ_b can be much lower when mica, talc, chlorite or other sheet silicate minerals occur on the sliding surface or when filling is present. Value as low as 6^o have been reported in saturated fillings of montmorillonite clay (see Fig. 10-63 in Lama and Vutukuri, 1978).

TABLE A3-23 BASIC FRICTION ANGLES OF VARIOUS UNWEATHERED ROCKS OBTAINED FROM FLAT AND RESIDUAL SURFACES (from Barton and Choubey, 1977)

Sedimentary rocks		basic friction angle ϕ_b	**Metamorphic rocks**		basic friction angle ϕ_b	**Igneous rocks**		basic friction angle ϕ_b
Sandstone	dry	26 - 35 (32)[*)]	Amphibolite	dry	32	Basalt	wet	31 - 36
	wet	25 - 34 (31)	Gneiss	wet	23 - 26		dry	35 - 38
Siltstone	wet	27 - 31		dry	26 - 29	Granite, - fine-grained	wet	29 - 31
	dry	31 - 33	Slate	wet	21		dry	31 - 35
Shale	wet	27		dry	25 - 30	- coarse-grained	wet	31 - 33
Conglomerate	dry	35					dry	31 - 35
Chalk	wet	30				Porphyry	wet	31
Limestone	wet	27 - 35					dry	31
	dry	31 - 37				Dolerite	wet	32
							dry	36

[*)] numbers in parenthesis are average values

Both waviness and smoothness contribute to the dilation or roughness angle, i, which can have any value between 0^o and 40^o or more at low pressures. Two different methods to estimate (i) has been roughly described in the following:

1. Using the joint roughness coefficient (JRC) as introduced by Barton (1973).
2. Combining the trace length and joint length found from photographs of natural cross sections of joints, Turk and Dearman (1985).

The JRC method has been developed by Barton (1973), Barton and Choubey (1977) and Barton and Bandis (1980, 1990) to estimate the shear strength of discontinuities. JRC ranges from 5 for smooth planar joints to 20 for rough undulating joints (Fig. A3-13). It is subjectively estimated from comparison with standard roughness profiles in Fig. A3-17. Barton and Bandis (1980) have proposed a modified way of separating the components of sliding resistance where Patton's dilation angle, i, is replaced by a bivariate measure

$$i = JRC \times \log_{10} (JCS/\sigma_n) \qquad \text{eq. (A3-15)}$$

Here JCS = the joint wall compressive strength (for fresh rocks JCS = σ_c) , and
σ_n = the normal stress across the joint. For the peak dilation angle the normal stress is very low (approximately 0.001 MPa, as in the tilt test).

This measurement of (i) in degrees includes both the small and large scale asperities. The factor $\log_{10} (JCS/\sigma_n)$ corrects JRC for asperity shearing. With low compressive strength they shear, and with high compressive strength they are overridden, refer to Barton and Bandis (1990).

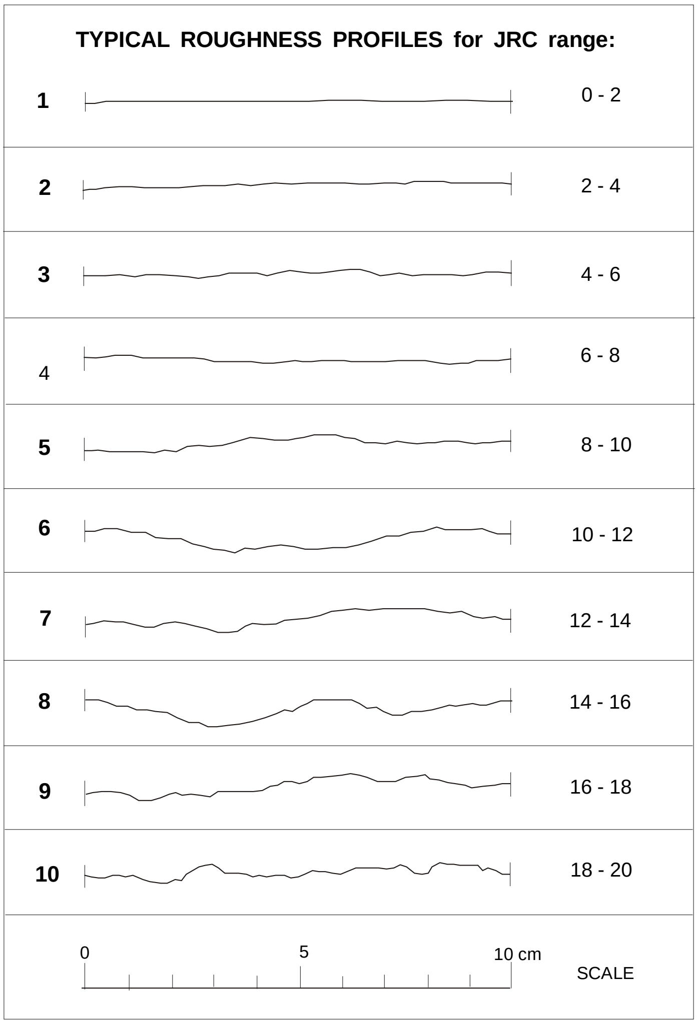


Fig. A3-17 Typical roughness profiles for various JRC ranges (from Barton and Choubey, 1977)

Barton, who recommends measuring JCS by correlation to Schmidt Hammer rebounds, had originally suggested JRC by inspection. The procedure for estimating JRC for specific sites may, according to Merritt and Baecher (1981), be prone to large measurements errors as it may be dependent on how practised the field engineer or geologist is. To circumvent this problem, Barton and Bandis (1980) recommends that tilt or push tests be used on typical block sizes in situ to obtain a more reliable measure of JRC.

The second method for estimating the dilation angle (i) is proposed by Turk and Dearman (1985). The joint trace seen on photographs of natural cross sections can be measured by some device as a 'map measure'. The roughness (dilation) angle (i) is determined by measurement of the direct distance and the trace length along the surface between the starting and finishing points for the profile measurement as

$$i = \arccos (L_1 / L_2) \qquad \text{eq. (A3-16)}$$

where L_1 = the direct length, and L_2 = the trace length on the measured joint surface.
The roughness angle (i) in eq. (A3-16 corresponds, according to Turk and Dearman, to the dilation angle measured under very low normal load (σ_n = 0.001 MPa, as used in the first method). This value can be determined on field and laboratory specimens, by simple linear measurement. The method is simple to apply and gives the peak (i) value for a particular joint surface. Also profiles found by feeler or contour gage in the second method mentioned in Section 2.2.1 may be used.

2.4 The joint size and continuity factor (jL)

The joint size and continuity factor consists of two contributions, the joint length and the termination (continuity) of the joint. The size of joints is, according to Piteau (1970), a difficult property to determine, but is essential in evaluations, since the strength reduction on a failure surface which contains a discontinuity is a function of the joint size. Also ISRM (1978) and Merritt and Baecher (1981) mention the great importance of joint size and the problem in mapping its length, both for small and large joints. A main reason for this is that the whole joint plane seldom can be seen in rock exposures, most often only the joint is seen as a trace (line), and this trace does seldom represent the largest dimension (Pollard and Aydin, 1988). In drill cores only a very small part of the joint can be studied as shown in Fig. 3-5 in Chapter 3. Hudson and Priest (1983) recommend that a measure of the joint length should only indicate the length/size interval of the joint.

TABLE A3-23 CLASSIFICATION OF JOINT PERSISTENCE (from Bieniawski, 1984)

very low persistence	< 1 m
low	1 - 3 m
medium	3 - 10 m
high	10 - 20 m
very high	> 20 m

Less frequently, in large cuttings, for example in quarries and open pits, it may be possible to record the dip length and the strike length of exposed joints and thereby estimate their persistence along a given plane through the rock mass using probability theory (Jennings, 1970; Robertson, 1970; ISRM, 1978).

The size of the joint is often proportional to the thickness or separation of the joint (Nieto, 1983; Kikuchi et. al., 1985), see Fig. A3-18. From the visible parts of the joint traces in unweathered exposures and excavated surfaces in tunnels or cuttings for example, the overall length interval of the joint set can be roughly estimated (Piteau (1970).

Discontinuous joints, i.e. joints that terminate in massive rock, will result in a rock mass with great inherent strength, since rupture must occur through intact rock before failure develops (Robertson, 1970). The ratings of joint size have been assessed from scale effect of joint length presented by Barton (1992)

$$JCS_n = JCS_0(L_n/L_0)^{-0.03JRC} \qquad \text{eq. (A3-17)}$$

where JCS_n and JCS_0 are the joint compressive strength (= σ_c) where subscripts refer to in situ joint size and lab. strengths respectively,
L_n and L_0 are the in situ and lab. size respectively
JRC refer to the lab. scale joint roughness coefficient.

Assuming an average value of JRC = 10 and that a 'normal' joint has a length in the range 1 - 10 m (average L_0 = 3 m), eq. (A3-17) can be expressed as

$$jL = JCS_n/JCS_0 \approx 1.5 \times L^{-0..3} \qquad \text{eq. (A3-18)}$$

where jL = the joint size factor, and L = the trace length of the joint, given in metre.

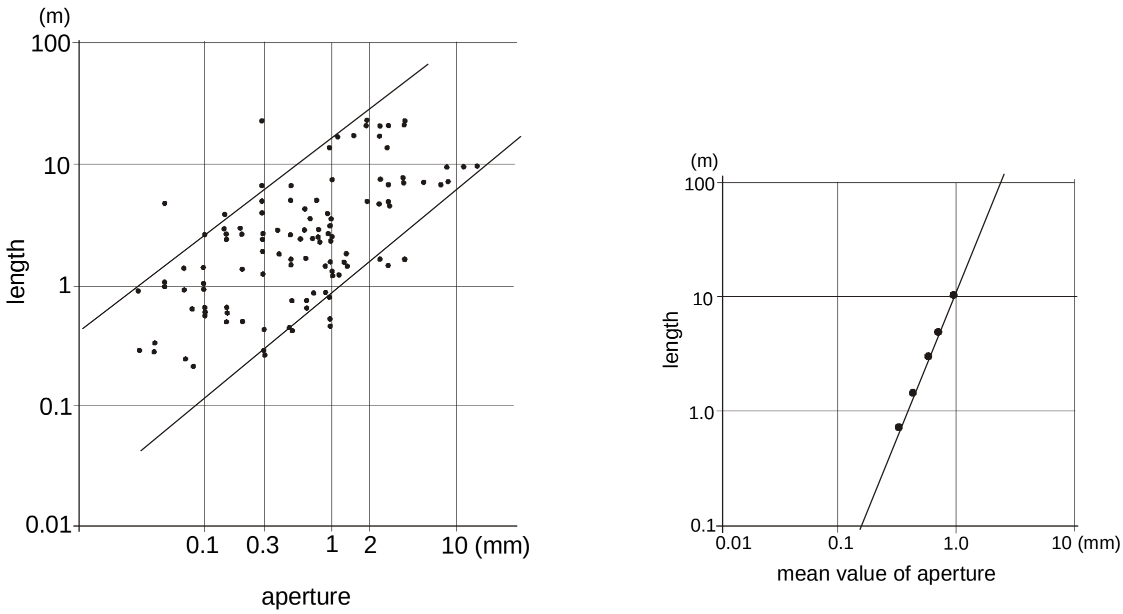


Fig. A3-18 Relation between length and aperture. Mean values are shown in the right figure (from Kikuchi et al., 1985).

Table A3-24 shows the characterization of joint size and the values of jL. From the foregoing the characterization of the joint size and continuity has been based on the length intervals of partings, short joints, medium and long joints as shown in Fig. 2-4 in Chapter 2. In a joint mapping or survey it may often be possible to differ between these types of joints. Also the classification in Table A3-23 has partly been applied.

TABLE A3-24 CHARACTERIZATION OF THE JOINT SIZE AND CONTINUITY FACTOR (jL).

LENGTH INTERVAL	**TYPE**	**Joint size factor jL*)** range	(mean)
< 1 m	bedding/foliation partings	2 - 4	(3)
0.1 - 1.0 m	short/small joint	1.5 - 3	(2)
1 - 10 m	medium joint	0.75 - 1.5	(1)
10 - 30 m	long/large joint	0.55 - 0.75	(0.7)
> 30 m	very long/large joint/seam**)	approx. 0.3 - 0.55	

*) **FOR DISCONTINUOUS JOINTS: multiply jL by 2**
**) Often a singularity, and should in these cases be characterized separately.

The influence of discontinuous joints, i.e. joints that terminate in massive rock, has been assumed as a doubling of the jL value.

2.5 Summary

Joint surface characteristics are broken into the joint roughness factor, jR, and the joint alteration factor, jA, which can be compared to Jr and Ja in the Q-system. Joint roughness is assessed at a large and small scale and joint alteration is determined from thickness and character of the filling material or from the coating on the joint surface. There is a significant degree of subjectivity in determining both these terms; some adjustments have been developed to improve their measurement.

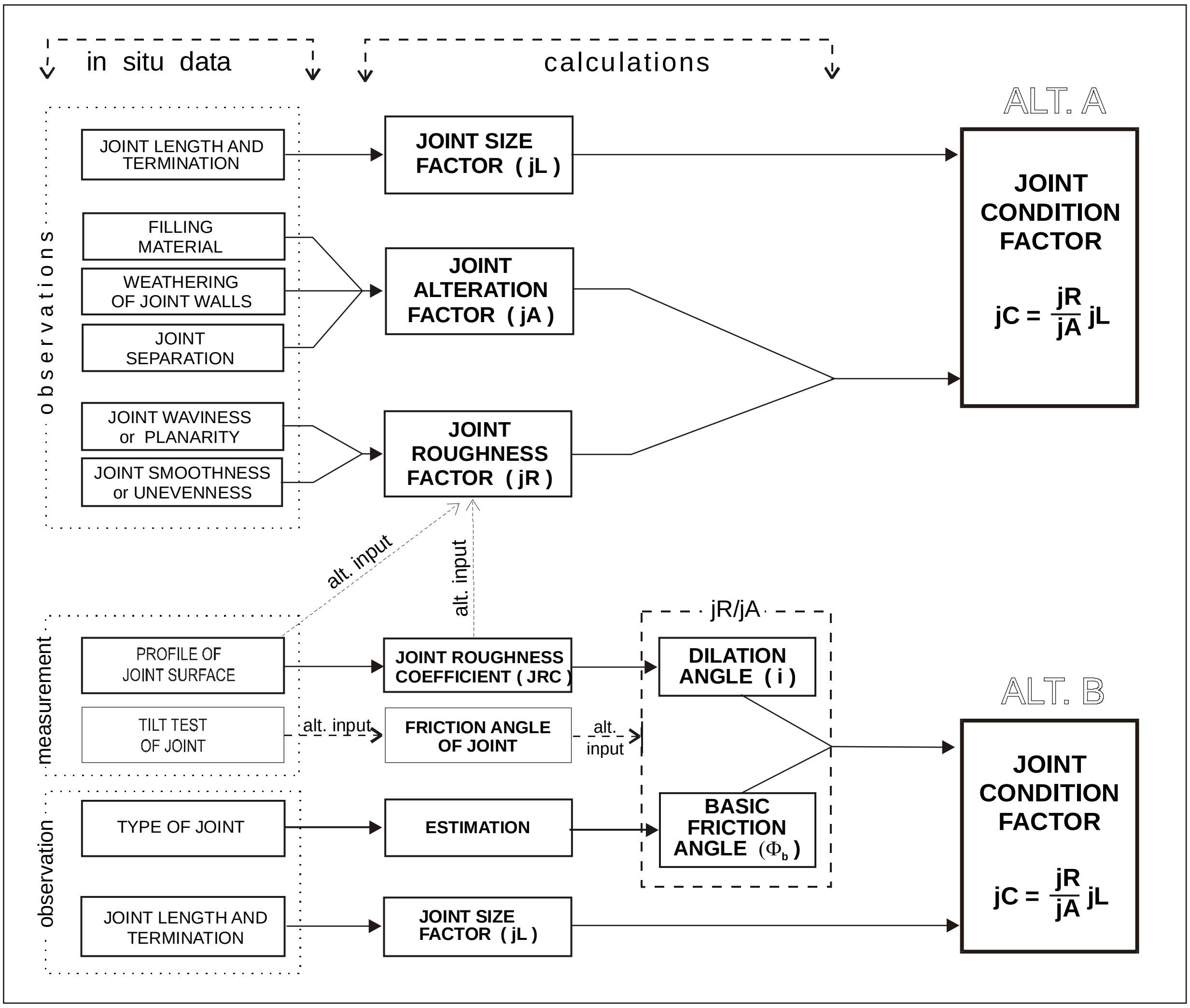


Fig. A3-19 The parameters involved and the methods applied to determine the joint condition factor (jC.)

Another method to determine the value of the joint condition factor, is to use the joint roughness coefficient, JRC, to determine the friction angle of the joint.

Fig. A3-19 shows the two main methods and the parameters and principle calculations involved in the joint condition factor jC. The factor for joint size and continuity is determined and applied in the same way in both methods.

3 METHODS TO DETERMINE BLOCK SIZE

"Since joints are among the most important causes of excessive overbreak and of trouble with water, they always deserve careful consideration."
Karl Terzaghi, 1946

The block size is a result of the detailed jointing in a rock mass mainly formed by the small and moderate joints (Selmer-Olsen, 1964). The block dimensions are determined by joint spacings and the number of joint sets. Individual or random joints and possible other planes of weakness may further influence on the size and shape of rock blocks. Impact from excavation works may also contribute to the splitting up of a rock mass into blocks.

ISRM (1978), Barton (1990) and several other authors mention that the block size is an extremely important parameter in rock mass behaviour. A wide range of scale effects in rock engineering can be explained by this feature, including compression strength, deformation modulus, shear strength, dilation, conductivity, shear stiffness, failure mode, stress-strain behaviour etc.

In addition to briefly outline how various methods can be used to measure block size and/or the quantity of joints, the following sections also show correlations between the measurements.

3.1 Types of block volume and joint density measurements

Different methods have been developed to measure the quantity or density of joints in the rock mass. The selection of the method(s) to be applied at an actual site is often a result of the availability to observe the rock and its jointing in an exposure, the requirement to the quality of the collected data, the type and cost of the investigation or survey, and the experience of the engineering geologist.

If all the blocks in a rock mass could be measured or "sieved" a block size distribution can be found, much the same used to describe particle sizes of a soil. As the joint spacings generally vary greatly, the difference in size between the smaller and the larger blocks can be large, Fig. A3-21. Therefore the characterization of block volume should be given as an interval rather than a single value.

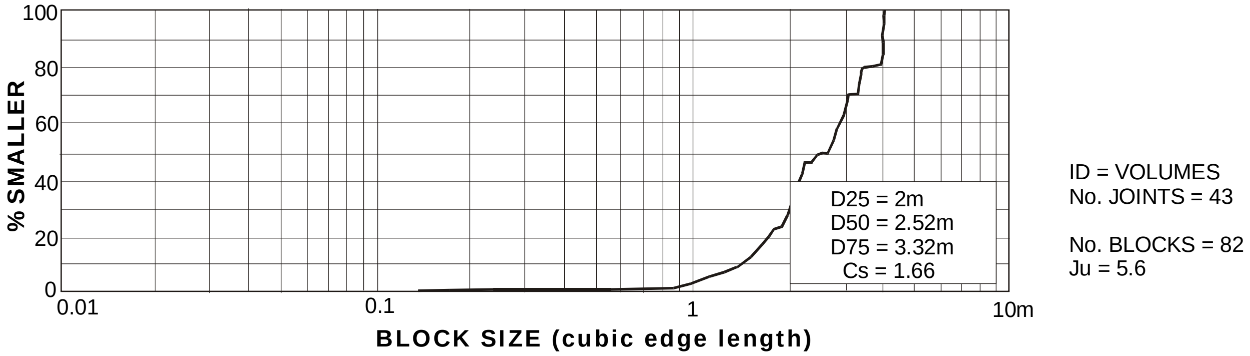


Fig. A3-21 Example of a block size distribution curve for a rock mass (from Milne et al., 1992).

Where less than 3 joint sets occur it is often expected that defined blocks will not be found. However, in most cases random joints or other weakness planes may contribute so that definite blocks occur. Also where the jointing is irregular, or many of the joints are discontinuous, it can be difficult to recognize the actual size and shape of individual blocks. Sometimes the block size and shape therefore have to be determined from reasonable simplifications where an *equivalent block volume* is assumed (see Section 3.2.3). Simplifications may also be necessary to possibly collect the data within a reasonable amount of work.

TABLE A3-26 CLASSIFICATION OF BLOCK VOLUME RELATED TO PARTICLE SIZE (VOLUME) FOR SOILS ($Vb = 0.58\ Db^3$ has been applied for the correlation between particle diameter (Db) and block volume Vb).

TERM FOR DEGREE OF JOINTING (or DENSITY OF JOINTS)	**TERM FOR BLOCK SIZE**	**VOLUME (Vb)**	**TERM FOR PARTICLE**	**APPROX. SOIL VOLUME**
			coarse sand	0.1 - 5 mm^3
			fine gravel	5 - 100 mm^3
Extremely high	Extremely small	< 10 cm^3 . .	. . medium gravel	0.1 - 5 cm^3
Very high	Very small	10 - 200 cm^3 . .	. . coarse gravel	5 - 100 cm^3
High	Small	0.2 - 10 dm^3 . .	. . cobbles	0.1 - 5 dm^3
Moderate	Moderate	10 - 200 dm^3 . .	. . boulders	5 - 100 dm^3
Low	Large	0.2 - 10 m^3 . .	. . blocks	> 0.1 m^3
Very low	Very large	10 - 200 m^3		
Extremely low (massive)	Extremely large	> 200 m^3		

TABLE A3-27 THE MAIN METHODS DESCRIBED IN THIS SECTION WHICH CAN BE USED TO MEASURE THE QUANTITY OF JOINTS AND THE BLOCK SIZE.

PARAMETER MEASURED	**SURFACE OBSERVATIONS**		**DRILL CORE or SCANLINE OBSERVATIONS**
	3-D registration	2-D registration	1-D registration
BLOCK SIZE	Block volume estimated from defined joint spacings (and angles between joint sets).		
- Block volume	Block volume estimated from Jv (see eq. (A3-27).		
	Block volume measured in the field.		Block volume of drill cores fragments [1].
- Equivalent block diameter		Estimated block diameter (Ib) according to ISRM (1978).	Indirect block diameter measure (given as RQD).
DEGREE OF JOINTING			
- Joint frequency	Registration of the volumetric joint count (Jv).	Measured number of joints intersecting an area.	Measured number of joints intersecting a line.
		*Weighted jointing density measurement.	*Weighted jointing density measurement.
- Joint spacing	Measured spacings for each joint set. (Normally used to express the block size (Vb) or the volumetric joint count (Jv)).	Measured mean joint spacings related to a plane.	Measured length of cores bits or spacings along a line (fracture intercept, (ISRM, 1978)).

* Measurement introduced in this contribution.
[1] In drill cores the block volumes refer to have the size of core diameter or less (gravel or pebbles size)

The density of joints in a rock mass is mainly found from various types of observations made on surfaces or on drill cores. The most common types are:

A. Surface observations, made as:
   - Field registration of block volume,
   - Joint spacing or frequency measurements
   - 3-D jointing density (as for the volumetric joint count, Jv)
   - 2-D jointing density (as for the number of joints in a surface)
   - 1-D jointing density (as for the number of joints along a scanline)

B. Drill core logging, recorded as:
   - Rock Quality Designation (RQD),
   - 1-D jointing density (the number or length of core pieces)

C. Geophysical measurements; the jointing density is mainly estimated from
   - Sonic velocities recorded by refraction seismic measurements.

Some of the measurements that can be made on rock outcrops, excavated surfaces and drill cores are shown in Table A3-27. The correlations between them, which is developed in this appendix, enable block volume to be determined from different sources.

As the blocks generally have varying sizes and shapes the measurements of characteristic dimensions can be very time-consuming and laborious. To remedy this, easy recognizable dimensions of the blocks and simple correlations between the different types of jointing measurements are worked out.

3.2 Block volume measurements

The block volume is intimately related to the intensity or degree of jointing. Each one of such blocks is more or less completely separated from others by various types of discontinuities. The greater the block size the smaller will be the number of joints penetrating the rock masses. Hence, there is an inverse relationship between the block volume and the number of joints.

Especially where irregular jointing occurs it is time-consuming to measure all (random) joints in a joint survey. In such cases, as well as for other jointing patterns, it is often much quicker - and also more accurate - to measure the block volume directly in the field. Where three or more regular joint sets occur, the block volume can easily be found from the joint spacings.

For each of the joint sets the spacings vary within certain ranges. The block volume in a rock mass should be characterized by a modal size together with the range i.e. typical largest and smallest block indices. (ISRM, 1978; Burton, 1965). Ideally, the range should be between about 25% and 75% of the block sizes similar to what is often practised to characterize particle distribution of soils, Fig. A3-21.

3.2.1 Block volume found from joint spacings

For blocks with less than 3 joint sets, the volume is often determined by the random joints in addition to the joints occurring in sets. For more than 3 joint sets the volume is determined by the jointing pattern and spacings as it is for 3 joint sets. Individual or random joints may further influence on the type and shape of blocks.

The volume of a block determined by 3 joint sets is given as

$$Vb = \frac{S1 \times S2 \times S3}{\sin\gamma 1 \times \sin\gamma 2 \times \sin\gamma 3} = \frac{Vb_0}{\sin\gamma 1 \times \sin\gamma 2 \times \sin\gamma 3} \qquad \text{eq. (A3-19)}$$

where $\gamma 1$, $\gamma 2$, $\gamma 3$ are the angles between the joint sets, and
S1, S2, S3 are the spacings between the individual joints in each set.
Vb_o is the volume for joints intersecting at right angles.

For a rhombohedral block with two angles between 45 - 60°, two between 135 - 150° and the last two 90°, the volume will be between $Vb = 1.3\ Vb_o$ and $Vb = 2\ Vb_o$ as is further shown in Section 4. Compared to the variations in spacing the effect from the intersection angle between joint sets is relatively small.

As earlier mentioned, no blocks will theoretically be formed where only one or two joint sets occur and random joints are very few or absent. Section 3.2.3 outlines methods to establish an equivalent block volume in such cases.

3.2.2 Block volume measured directly in situ or in drill cores

Where the individual blocks can be observed in a surface, their volume can be directly measured from relevant dimensions by selecting several representative blocks and measuring their average dimensions (ISRM, 1978). From this, the range in the block volumes can be determined.

For small blocks or fragments having volumes of dm^3 or less this method of block volume registration is often beneficial as it is easy to estimate volume compared to all the measurements of the many joints. Block volume can also be found in drill cores where the presence of many joints delineate small fragments. The laborious and time-consuming measurements of the many joints in the core in such cases is often a main reason to apply a simpler method for core logging like the RQD. By applying block volume in the rock mass characterization a significantly more accurate registration of the joint density or frequency is achieved. The quality of such measurements has been further outlined in Appendix 4 where various methods of jointing measurements have been investigated.

Also where irregular jointing occurs it may be much more convenient to directly measure the block size by eye during field inspection than to record all the joints and their locations.

3.2.3 Methods to find the equivalent block volume where joints do not delimit blocks

A minimum of three joint sets in different directions are theoretically necessary to delimit blocks in a rock mass. As mentioned in Appendix 1 there are cases with irregular jointing where blocks are formed mainly from random joints, and other cases where the blocks are delimited by one or two joint sets and additional random joints.

Sometimes, however, where the jointing is composed of one or two joint sets with no or few random joints, no defined blocks are formed. In such cases an *equivalent block volume* is applied in the calculations. Such block volume may be found from the following methods:

- The equivalent block volume may be estimated directly in the field from joint observations.
- Where only one joint set occurs the equivalent block volume may be considered similar to the area of the joint plane (i.e. $L1^2$) multiplied by the joint spacing (S1).

(Example: For foliation partings with lengths L1 = 0.5 - 2 m and spacing S1= 0.2 m the eq. block volume is between $Vb = 0.2 \times 0.5^2 = 0.05$ m^3 and $Vb = 0.2 \times 2^2 = 0.8$ m^3)

- For two joint sets the spacing for the two sets (S1 ans S2) and the length (L) of the joints can be applied: $Vb = S1 \times S2 \times L$.

- The equivalent block volume can be found from eq. (A3-27): $Vb = \beta \times Jv^{-3}$ which requires input from the block shape factor β. [2]
  β can be estimated from eq. (A4-1): $\beta = 20 + 7\ S_{max} / S_{min}$
  where S_{max} and S_{min} are the shortest and longest dimension of the block.
  Another method to estimate the value of β from the length and spacing of the joints is outlined in the following:

  Eq. (A4-1) is developed for three joint sets. Where less than three sets occur, it can be adjusted by a factor n_j representing the rating for joint sets to characterize an equivalent block shape factor:

  $$\beta = 20 + 7\ (S_{max} / S_{min})(3/n_j) = 20 + 21 \times S_{max} / S_{min} \times n_j \qquad \text{eq. (A3-20a)}$$

  or if the more accurate expression eq. (A4-2) is applied:

  $$\beta = 20 + (21/n_j)\ (S_{max} / S_{min})^{(1 + 0.1 \log (Smax / Smin))} \qquad \text{eq. (A3-20b)}$$

The ratings of n_j are given as:

3 or more joint sets	$n_j = 3$
2 joint sets + random joints	$n_j = 2.5$
2 joint sets	$n_j = 2$
1 joint set + random joints	$n_j = 1.5$
1 joint set only	$n_j = 1$

For *fissures, partings and small joints* where their length often can be found or estimated, the length and spacing of the joints correspond to the longest and shortest block dimension, hence the ratio length/spacing = L1/S1 can be applied in eq. (A3-20a):

$$\beta = 20 + 21 \times L1/(S1 \times n_j) \qquad \text{eq. (A3-20c)}$$

For long joints it is generally sufficiently accurate to use L = 4 m.

Table A3-27A shows a comparison between the values of β found from various equations. The rows for defined blocks typed in bold letters show the correct values for such blocks. With one joint set the blocks are flat, while two joint sets have been assumed to form long&flat blocks. From the table it is clear that the simplified eqs. (A3-20a) and (A3-20b) give higher values for β for ratio (= longest/smallest block dimension) smaller than 20, while for the very flat or long&flat blocks the values are lower.

Example
For 1 joint set ($n_j = 1$) spaced S1 = 0.2 m having joint length L1 = 2 m the block shape factor according to eq. (A3-20a) is $\beta = 20 + 21\ L1/(S1\times n_j) = 230$. The volumetric joint count for this set, $Jv = 1/S1 = 5$, gives the equivalent $Vb = \beta \times Jv^{-3} = 1.84$ m^3.
(For a defined block limited by 3 joints sets with spacings S1, L1, L1 the volume is
$Vb = 0.2 \times 2 \times 2 = 0.8$ m^3)

---

[2] As the volumetric joint count can be measured also where joints do not delimit defined blocks, this approach can be applied where few joints sets are found.

TABLE A3-27A COMPARISONS OF THE BLOCK SHAPE FACTOR β FOR VARIOUS TYPES OF JOINT PATTERN AND EQUATIONS

JOINT PATTERN		Values of β for ratio*) =								
		1	2	3	5	7	10	20	30	50
FLAT BLOCKS: **3 joint sets (defined blocks)** (dimensions: S1, S2 and S3 = S2)	**eq. (A3-28)**	**27**	**32**	**42**	**69**	**104**	**172**	**532**	**1092**	**2812**
1 set only ($n_j = 1$)	eq. (A3-20a)	41	62	83	125	167	230	440	650	1070
(dimensions: S1 and L1(= S3))	eq. (A3-20b)	41	63	86	138	193	284	640	1061	2061
1 set + random joints ($n_j = 1.5$)	eq. (A3-20a)	34	48	62	90	118	160	300	440	720
(dimensions: S1 and L1 (= S3))	eq. (A3-20b)	34	48.5	64	98	136	196	433	714	1380
LONG&FLAT BLOCKS: **3 joint sets (defined blocks)** (dimensions: S1, S2 and S3 = $S2^2$)	**eq. (A3-28)**	**27**	**30.4**	**36**	**50**	**65**	**90**	**185**	**295**	**554**
2 sets only ($n_j = 2$)	eq. (A3-20a)	30.5	41	51.5	72.5	93.5	125	230	335	545
(dimensions: S1 and L1 (= S3) or L2 (= S3))	eq. (A3-20b)	30.5	41	53	79	107	152	330	540	1040
2 sets + random ($n_j = 2.5$)	eq. (A3-20a)	28.4	37	45	62	79	104	188	272	440
(dimensions: S1 and L1 (= S3) or L2 (= S3))	eq. (A3-20b)	28.4	37	46.5	67	89	126	268	436	836

$$\text{eq.(A3-28)}: \beta = \frac{\left[(S2/S1) + (S2/S1)(S3/S1) + (S3/S1)\right]^3}{\left[(S2/S1)(S3/S1)\right]^2}$$ (the definition of β for 3 joint sets)

eq. (A3-20a): $\beta = 20 + 21 \cdot S_{max}/S_{min} \times n_j$

eq. (A3-21b): $\beta = 20 + (21/n_j)\,(S_{max}/S_{min})^{(1 + 0.1 \log (Smax/Smin))}$

*) ratio = longest/shortest block dimension
S1, S2, S3 = spacings for the joint sets; L1, L2, L3 = length of the joints; Nl = number of joints per m

3.3 Block diameter registrations

The *block size index* (Ib) introduced by ISRM (1987), is a measure of the block diameter. According to ISRM this measure can be compared with the particle diameter for soils. The block size index can be estimated by selecting by eye several typical block sizes and taking their average dimensions. Since the index may range from millimetres to several metres, ISRM indicates a measuring accuracy of ±10% as sufficient. *"Each domain should be characterized by a modal (Ib) together with the range i.e. typical largest and smallest block indices."* The block size index Ib seems from published papers to be seldom used and is therefore not further used in this work.

The equivalent block diameter, Db, used in the rock support charts in Chapter 6, Section 4.2 and 4.3 is another expression for the block diameter which uses the block volume and its block shape factor β given in eq. (6-9): [3]

$$Db = (27/\beta)Vb^{1/3}$$

[3] (β_0/β) has been chosen in eq. (6-6) as a simple expression to roughly find the smallest block diameter. For most cases it can be used with satisfactory accuracy for $\beta < 150$. For higher values of β a dominating joint set will normally be present for which the average joint spacing should be applied. β is described in Section 4 in this appendix.)

Where a dominating joint set occurs the joint spacing (Sa) may preferably be used as the dimension for the block diameter (Db = Sa).

3.4 Rock quality designation (RQD)

Rock quality designation is the method perhaps most commonly used for characterizing the degree of jointing in bore hole cores, although this parameter also may implicitly include other rock mass features as weathering and 'core loss' (Bieniawski, 1984).

RQD can be regarded as an indirect block size measure, as it is an expression of intact core lengths greater than a threshold value of 0.1 m along any scan line (bore hole). Increase in the number of joints in a rock mass causes decrease in RQD.

This parameter was originally restricted to characterize the amount of discontinuities in a drill core. RQD is a main input both in the Q and the RMR system; in cases where no core drilling has been carried out, the RQD value is roughly estimated from surface observations. An approximate transition between surface observations of jointing density and RQD has been presented by Palmström (1974, 1982) (see Fig. A3-22). Hudson and Priest (1983) and Sen and Eissa (1991, 1992) have later developed theoretical analytical approaches for determination of RQD from joint spacings.

From its definition 'the sum of core pieces longer than 0.1 m for a certain length, taken as a percent' and the fact that it is often assumed only from observation of rock surfaces, the use of RQD is a crude and generally inaccurate measure of the degree of jointing. This is further explained in Appendix 4.

The classification of RQD has been given by Deere (1966) as presented in Table A3-28.

TABLE A3-28 CLASSIFICATION OF ROCK QUALITY DESIGNATION (RQD) (from Deere, 1966).

TERM	RQD
very poor	< 25
poor	25-50
fair	50-75
good	75-90
excellent	90-100

3.4.1 Correlation between the RQD and the volumetric joint count (Jv)

Also from its definition - being independent of the length of individual core pieces being longer than 0.1 m - the RQD is a crude measure of the degree of jointing and hence the block size. It is, therefore as mentioned in Appendix 4, not possible to obtain good correlations between RQD and Jv or between RQD and other measurements of jointing. This is illustrated in Fig. A3-22 where the following expression is given

$$RQD = 115 - 3.3\ Jv \qquad \text{eq. (A3-21)}$$

As a consequence of this, especially when many of the core pieces have lengths around 0.1 m, the correlation above must be regarded as a very rough. It is, however, difficult to recommend any better transition from RQD via Jv to block volume than this correlation where RQD is the only jointing data available.

From the volumetric joint count the block volume can be found provided input of the block shape factor (β), see eqs. (A3-26) and (A3-27). Where β is not known it is recommended to use an assumed 'common' value of β = 40.

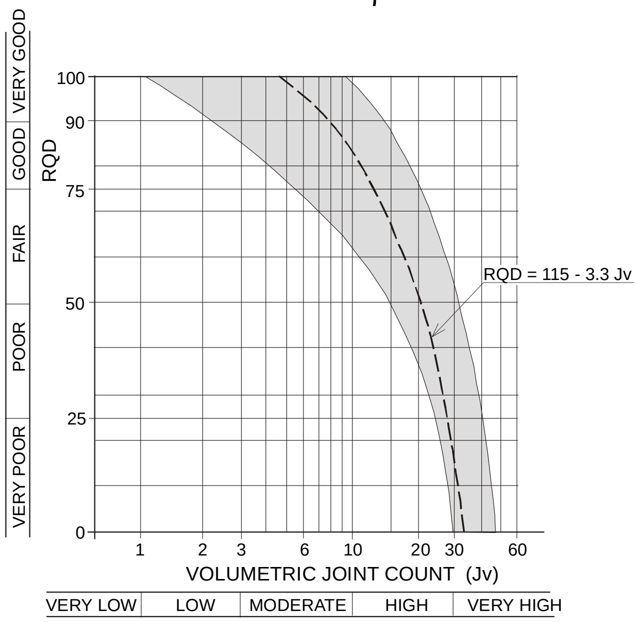


Fig. A3-22 Correlation between RQD and Jv (from Palmström, 1982).

3.5 The volumetric joint count (Jv)

The volumetric joint count, Jv, has been described by Palmström (1982, 1985, 1986) and Sen and Eissa (1991, 1992). It is a measure for the number of joints within a unit volume of rock mass, defined by

$$Jv = \Sigma\ (1/S_i) \qquad \text{eq. (A3-22)}$$

where S is the joint spacing in metres for the actual joint set.

Also random joint can be included by assuming a 'random spacing' for these. Experience indicate that this should be set to Sr = 5 m; thus, the volumetric joint count can be generally expressed as

$$Jv = \Sigma\ (1/S_i) + Nr/5 \qquad \text{eq. (A3-23)}$$

Here Nr is the number of random joints in the observation area adjusted for their length, see Section 3.6.1.

TABLE A3-29 CLASSIFICATION OF THE VOLUMETRIC JOINT COUNT (Jv) (revised after Palmström, 1982)

TERM FOR JOINTING	TERM FOR Jv	Value of Jv
massive	extremely low	< 0.3
very weakly jointed	very low	0.3 - 1
weakly jointed	low	1 - 3
moderately jointed	moderately high	3 - 10
strongly jointed	high	10 - 30
very strongly jointed	very high	30 - 100
crushed	extremely high	> 100

The volumetric joint count can easily be calculated since it is based on common observations of joint spacings or frequencies. In the cases where mostly random or irregular jointing occur the Jv can be found by counting all the joints observed in an area of known size. The accuracy of this measurement is described in Appendix 4. Table A3-29 shows the classification of Jv.

3.5.1 Block volume (Vb) estimated from the volumetric joint count (Jv)

Because both the volumetric joint count (Jv) and the size of blocks in a rock mass vary with the degree of jointing, there exist a correlation between them (Palmström 1982). Jv varies, however, with the joint spacings, while the block size also depends on the type of block as shown in Fig. A3-24. A correlation between the two parameters has therefore, to be adjusted or corrected for the block shape and the angle between the joint sets, as shown in the following.

The volumetric joint count determined from three joint sets intersecting at right angles, is expressed as

$$\mathrm{Jv} = \frac{1}{S} + \frac{1}{S} + \frac{1}{S} = \frac{S2 \times S3 + S1 \times S3 + S1 \times S1}{S2 \times S2 \times S3} = \frac{S2 \times S3 + S1 \times S3 + S1 \times S2}{\mathrm{Vb_o}} \qquad \text{eq. (A3-24)}$$

where S1, S2, S3 are the joint spacings.

Using $Vb_o = Vb \times \sin\gamma 1 \times \sin\gamma 2 \times \sin\gamma 3$ for intersections at other angles eq. (A3-24) can be expressed as

$$\mathrm{Jv} = \frac{S2 \times S3 + S1 \times S3 + S1 \times S2}{\mathrm{Vb} \times \sin\gamma 1 \times \sin\gamma 2 \times \sin\gamma 3} \qquad \text{eq. (A3-25)}$$

$$\mathrm{Jv}^3 = \frac{(\alpha 2 + \alpha 2 \times \alpha 3 + \alpha 3)^3}{(\alpha 2 \times \alpha 3)^2} \times \frac{1}{\mathrm{Vb} \times \sin\gamma 1 \times \sin\gamma 2 \times \sin\gamma 3} = \frac{\beta}{\mathrm{Vb} \times \sin\gamma 1 \times \sin\gamma 2 \times \sin\gamma 3}$$

By applying the ratio $\alpha 2 = S2/S1$, and $\alpha 3 = S3/S1$, provided $S3 > S2 > S1$, and $S1^3 = Vb_o/(\alpha 2 \times \alpha 3)$ eq. (A3-25) can be expressed as
From this the block volume is

$$\mathrm{Vb} = \beta \times \mathrm{Jv}^{-3} \times \frac{1}{\sin\gamma 1 \times \sin\gamma 2 \times \sin\gamma 3} \qquad \text{eq. (A3-26)}$$

In the cases where all angles between the block faces are 90^o, the block volume is given as

$$\mathrm{Vb_o} = \beta \times \mathrm{Jv}^{-3} \qquad \text{eq. (A3-27)}$$

The factor

$$\beta = \frac{(\alpha 2 + \alpha 2 \times \alpha 3 + \alpha 3)^3}{(\alpha 2 \times \alpha 3)^2} \qquad \text{eq. (A3-28)}$$

depends mainly on the differences between joint spacings; i.e. the block shape, and has therefore been named the *block shape factor*. A graph to determine its value from eq. (A3-28) and its use is further described in Section 4.

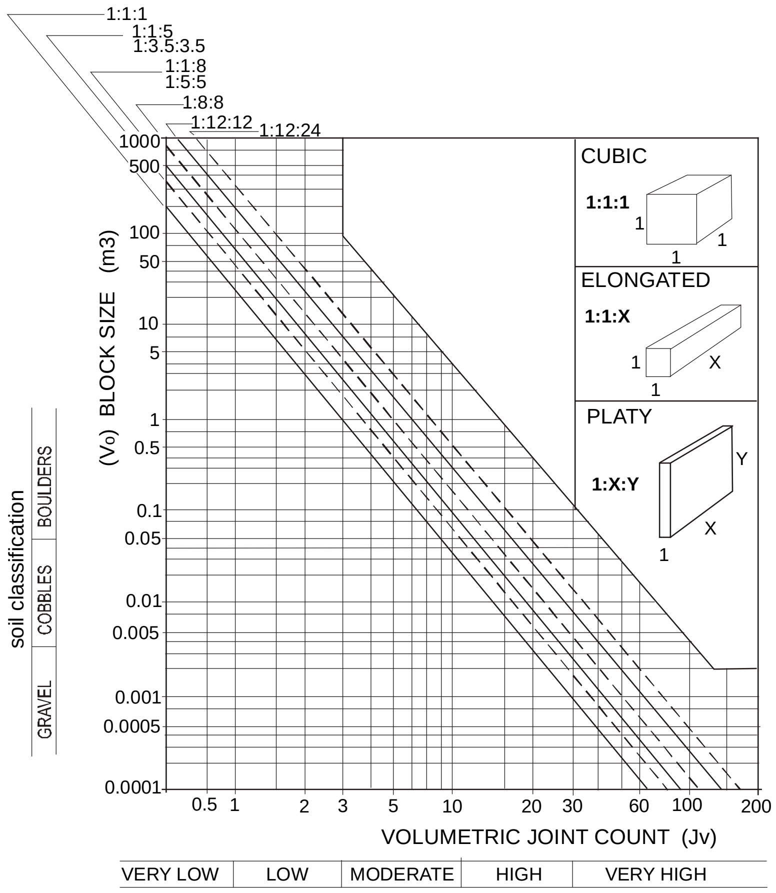


Fig. A3-24 The relation between block size and volumetric joint count, Jv for various block shapes (from Palmström, 1982).

If three joint sets occur and both Jv and β are exactly measured, the exact block volume can be calculated from eq. (A3-26) or eq. (A3-27) provided the joints intersect at right angles. Quite often, however, not all faces of a block are known; in these cases the value of β can not be exactly found. Therefore, a simplified method to determine the block shape factor given as

$$\beta = 27 + 7(\alpha 3 - 1) = 20 + 7(S3/S1) \qquad \text{eq. (A3-29)}$$

has been developed as outlined in Appendix 4, where S3/S1 = α3 is the ratio between the longest and shortest block face. By applying this in eq. (A3-26) the block volume can be estimated from

$$Vb = \frac{20 + 7(S3/S1)\, Jv^{-3}}{\sin\gamma 1 \times \sin\gamma 2 \times \sin\gamma 3} \qquad \text{eq. (A3-30)}$$

or $$Vb_o = \beta \times Jv^{-3} \approx \{20 + 7(S3/S1)\}\, Jv^{-3} \qquad \text{eq. (A3-31)}$$

These equations are valid for prismatic blocks where the joints or block faces intersect at right angles.

As the volumetric joint count (Jv) by definition takes into account in an unambiguous way all the occurring joints in a rock mass, it is often appropriate to use, Jv, in the correlation between jointing frequency registrations and block volume estimates (Palmström, 1982). Important in these is the block shape factor β which is included in all equations to estimate the block volume.

3.6 Joint frequency measurements

When the frequency is given for each joint set, it is possible to establish a correlation between joint frequencies and block volume. In other cases, when an 'average frequency' is given, it is uncertain whether the value refers to one-, two- or three-dimensional measurements, hence no accurate correlation factor can be presented.

Some of the methods described in the following to estimate the block volume from joint frequency measurements, are also described in Appendix 4.

3.6.1 <u>2-D joint frequency in an area or surface</u>

The 2-D joint frequency is the number of joints measured in an area. The length of the joints compared to the size of the area will, however, influence on the frequency and some sort of adjustments have to be made to estimate the block volume from this type of jointing measurement. Fig. A3-25a shows three different observation areas for which the joints are larger than the dimension of the area.

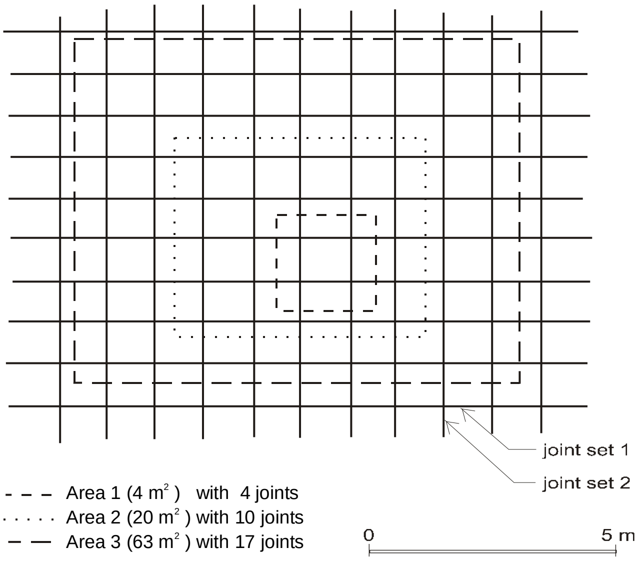
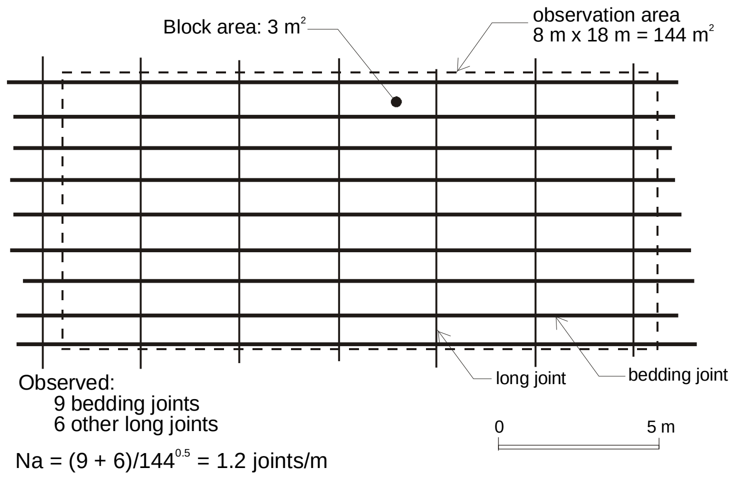


Fig. A3-25a Various sizes of the observation area and the number of joints observed. All joints are longer than the dimension of the area.

In the table below the density of joints per square metre and per metre are given. As seen the latter method gives a constant number of the joint frequency (Na), given as the density of joints per metre.

size of area	**number of joints**	**joints/m^2**	**joints/m**
A	na	Na = na/A	Na = na/$\sqrt{A}$
4 m^2	4	1	2
20 m^2	10	0.5	2.2
63 m^2	17	0.27	2.1

If the joints are smaller, a higher amount of joints may be observed within the area, as shown in the lower diagram of Fig. A3-25b.



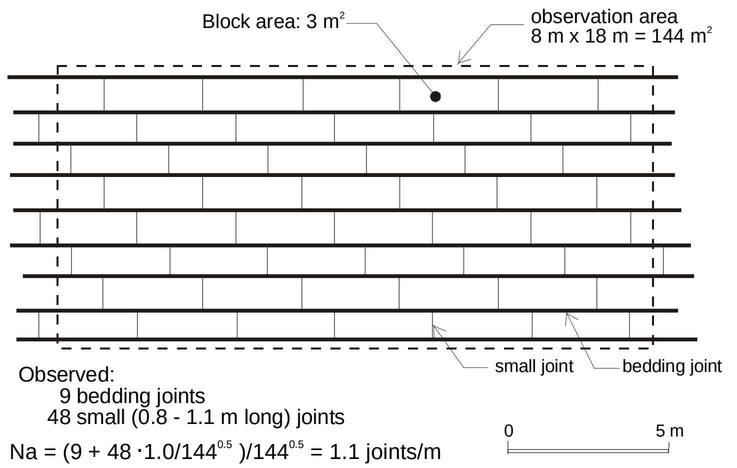


Fig. A3-25b Two 'exposures' of joints with different jointing pattern. The blocks in both have the same size. A higher amount of joints is recorded in the lower figure where the cross joints are short. Applying Na = na / $\sqrt{A}$ the same value is found for both types of jointing.

The joint frequency Na should, therefore, be adjusted for the lengths of the joints if they are shorter than the length of the observation plane, expressed as

$$Na = \Sigma(na_i \times L_i / \sqrt{A}) \qquad \text{eq. (A3-32a)}$$

where na = the number of joints with length L and
A = the area of the observation plane.

Na varies with the orientation of the observation plane with respect to the attitude of the joints. Recording of Na in several surfaces of various orientation gives a more accurate measure of the jointing. Being an average measurement, Na should be measured in selected areas with the same type of jointing. A larger area should be divided into smaller, representative areas containing similar jointing, and the variation in jointing for the whole area calculated from these registrations.

The correlation between 2-D registrations of the joint density in a rock surface and 3-D frequency values can, as shown in Appendix 4, be done using the empirical expression

$$Jv = Na \times ka \qquad \text{eq. (A3-32b)}$$

where Na = number of joints per metre measured in a surface, and
ka = correlation factor shown in Fig. A3-25c.

As seen in Fig. A3-25c, ka varies mainly between 1 and 2.5 with an average value ka = 1.5. It has its highest value where the observation plane is parallel to the main joint set.

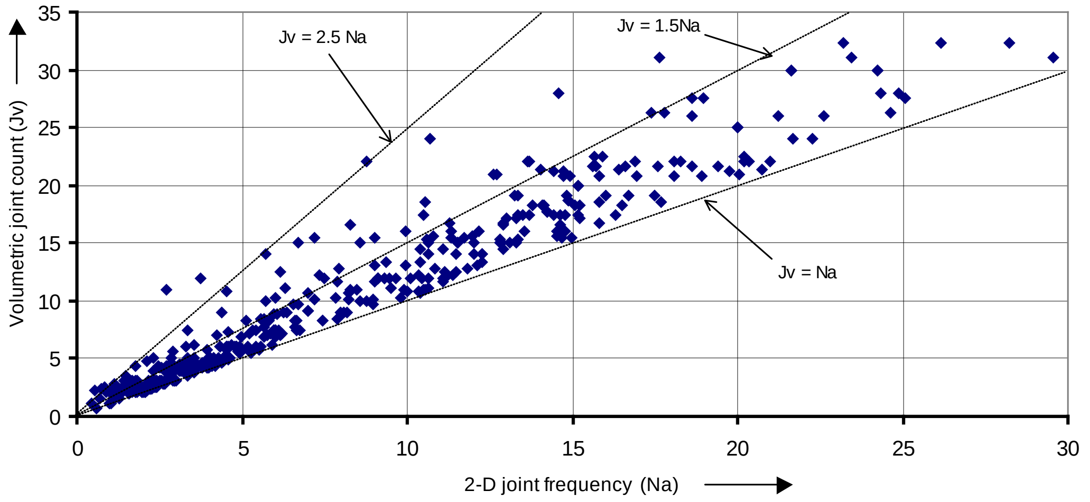


Fig. A3-25c Variation of the correlation Jv = ka × Na for various types of blocks and orientations of the observation surface.

3.6.2 1-D jointing frequency measurements along a scanline or drill core

This is a record of joint frequency in a drill core or scan line given as the number of joints intersecting a certain length (L_l).

The 1-D joint frequency is an average measure along the selected length of the core. As in other core logging methods and in surface observations it is important to select a section of the line or core length, which shows similar jointing frequency. At the start of the logging it is rational to divide the length into such sections of uniform or similar frequency.

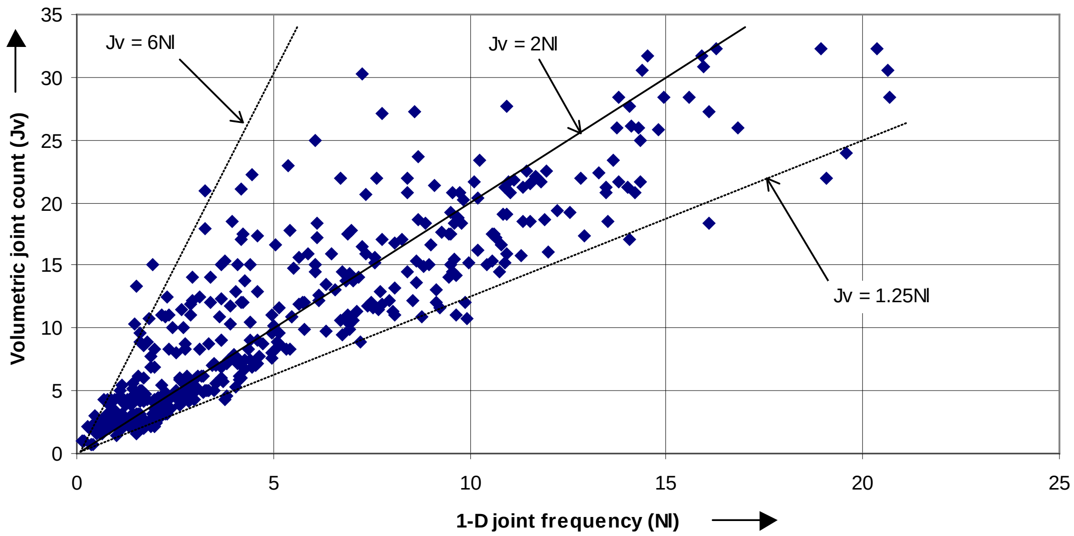


Fig. A3-26 Variation of the correlation Jv = kl × Nl for various types of blocks and orientations of the observation surface.

The 'joint frequency' can, as mentioned, be very inaccurate if it is not strictly defined what is included in the registration, and it should, therefore, be accompanied by additional information what it covers.

The correlation between 1-D joint frequency registrations in drill cores and volumetric 3-D frequency (Jv) can be done using a similar expression as eq. (A3-32)

$$Jv = Nl \times kl \qquad \text{eq. (A3-33)}$$

where Nl = 1-D joint frequency, i.e. the number of joints per metre along a core or line, and
kl = correlation factor shown in Fig. A3-26 with an average kl = 2. As expected there is a rather poor correlation between kl and Jv .

The joint spacing registrations presented in the following are similar to the joint frequency measurements.

3.7 Joint spacing registrations

The terms *joint spacing* and *average joint spacing* are often used in the description of rock masses. Joint spacing is the distance between individual joints within a joint set. Where more than one set occurs this measurement for surface observations is often the average of the spacings for these sets.

However, when the recordings are made on drill cores the spacing is often the average length of core bits.[4] Thus, the spacings or frequencies are not true recordings as joints of different sets are included in the measurement. In addition, random joints, which do not necessarily belong to any joint set, influence. As the term 'joint spacing' does not indicate what it includes, it is frequently difficult to determine whether a 'joint spacing' referred to in the literature represents the true joint spacing. Thus, there is often much confusion related to the use of joint spacing recordings.

TABLE A3-30 CLASSIFICATION OF JOINT SPACING
(from ISRM (1978)

TERM	SPACING (S)
extremely close spacing	< 20 mm
very close	20 - 60 mm
close	60 - 200 mm
moderate	0.2 - 0.6 m
wide	0.6 - 2 m
very wide	2 - 6 m
extremely wide	> 6 m

As joint spacing (S) is the inverse of joint frequency, the correlation factor between them for finding Jv is

ca = 1/ka for 2-D observations on rock surfaces (average ca = approx. 0.67), and
cl = 1/kl for 1-D observations of scanlines or drill cores (average cl = approx. 0.5).

Deere et al.(1969) have experienced that where several joint sets are present, the resulting average 'volumetric spacing' is generally 2/3 to 1/3 of the average joint spacing of any of the joint sets. For correlation purposes they consider it sufficiently accurate to use a ratio of 1/2. This is the same as

[4] Joint or fracture intercept is the appropriate term for measurement of the distance between joints along a line or bore hole.

has been found for average value of (cl) above. It must be realised, however, that this ratio may be erroneous. If, for example, there is only one dominant joint system, the ratio would be closer to unity. The ratio also depends on the orientation of the bore hole or observation surface relative to the direction of the joints.

Franklin et al. (1971) have suggested to record a direct measure of joint spacing by using the *fracture spacing index* (I_f), which refers to the average size of cored material within a recognisable geological unit. When few joints are present in the core, the I_f is the unit length divided by the number of fractures within the unit. If the core is very broken, the I_f is the average diameter of a number of separate rock fragments. The latter can be compared with direct estimates of block volume made on small fragments in drill cores which has been described in Section 3.2.2. Franklin et al. (1971) and Hudson and Priest (1983) recommend two or more inclined bore holes in different directions to obtain an accurate estimation of the fracture spacing index.

The use of *weighted joint density measurements* will generally improve the characterization of block size, also where only results from a single bore hole is available. This method may positively reduce the amount of drill holes in a site where measurement of joint density or block size is a main reason in the investigation.

3.8 Weighted joint density measurements (wJd)

R. Terzaghi (1965) points out that the accuracy of jointing measurement can be increased by replacing the number of joints measured in a surface or borehole, N_α, intersected at an angle α, by a value N_{90}. N_{90} represents the number of joints with the same orientation which would have been observed at an intersection angle of 90^o. This is expressed as

$$N_{90} = N_\alpha/\sin\alpha \qquad \text{eq. (A3-34)}$$

Terzaghi stresses the problem of correcting for small values of α, because, in these cases, the number of intersections will be significantly affected by local variations in spacing and continuity. *"Further, no correction whatsoever can be applied if α is zero. Hence N_{90} would fail to correctly indicate the abundance of horizontal and gently dipping joints in a horizontal observation surface."* The method for weighted joint density measurement presented in the following is based on measuring the angle between the joint and the observation surface or borehole. To solve the problem of small intersection angles and to simplify the observations, the angles have been divided into intervals as shown in Table A3-31. For 2-D measurements (surface observations) the weighted joint density is defined as

$$wJd = \Sigma(1/\sin\delta_i) / \sqrt{A} = \Sigma(f_i)/ \sqrt{A} \qquad \text{eq. (A3-35)}$$

and, similarly, for 1-D registrations along a scan line or drill core

$$wJd = \Sigma(1/\sin\delta_i) /L = \Sigma(f_i) / L \qquad \text{eq. (A3-36)}$$

Here δ_i is the angle between the observation plane (surface) and the individual joint.
A is the size of the area in m^2, see Fig. A3-27
L is the length of the measured section along core or line, see Fig. A3-27
f_i is the interval factor ($1/\sin\delta_i$) given in Table A3-31; its ratings have been determined in Appendix 4.

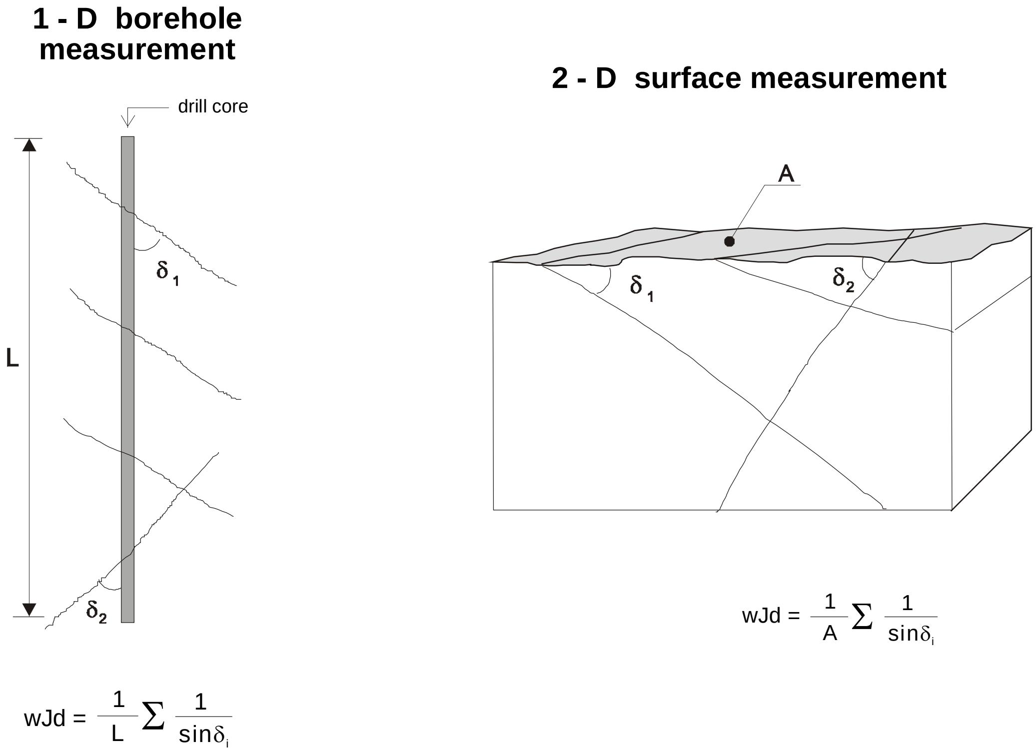


Fig. A3-27 The intersection between joints and a drill core hole (left) and a surface (right).

TABLE A3-31 SELECTED INTERVALS OF THE ANGLE δ_i AND THE CORRESPONDING FACTOR f_i (= $1/\sin\delta_i$).

angle δ_i	**factor f_i**
> 60^o	1
31 - 60^o	1.5
16 - 30^o	3.5
< 16^o	6

In practice, each joint is multiplied by the (f_i) value for its actual angle interval.
It should be possible to quickly determine the intervals in Table A3-31 for the angle δ_i by the eye after some training. The intervals chosen discards as mentioned the strong influence of the smallest angles, i.e. angles parallel or nearly parallel to the observation plane or bore hole.

The weighted joint density method reduces the inaccuracy caused by the orientation of the observation surface or bore hole with respect to the individual joints. Hence it leads to a better characterization of the density of joints and may lead to reduced amount of bore holes.

3.8.1 Correlation between wJd and Jv

The weighted joint density is approximately equal to the volumetric joint count (wJd = Jv) as is seen in Fig. A4-6 to A4-8 in Appendix 4.

3.9 Use of refraction seismic measurements to assess block volume

The use and possibilities to apply refraction seismic measurements are described in Appendix 5. Refraction seismic is probably the geophysical method most closely related to rock mass properties because the seismic velocity varies with many of the parameters in the rock mass. The results from such measurements may therefore assist in the acquisition of geo-data.

3.9.1 Influence from the intact rock and the in situ conditions

Seismic refraction measurement in the field utilizes the propagation of compression or primary (P) waves as these are the easiest to detect. Velocities of longitudinal waves vary considerably with the *type of rock* materials involved, as shown in Table A3-32. The presence of pores, cracks and flaws as in less compact and unconsolidated rocks, highly reduces the velocity, contrary to moisture, which increases the velocity.

TABLE A3-32 AVERAGE VELOCITIES (km/s) OF PROPAGATION OF LONGITUDINAL WAVES FOR SOME TYPICAL ROCKS AND SOILS (partly after Lama and Vutukuri, 1978)

Compact rocks		**Less compact rocks**		**Unconsolidated materials**	
Dunite	7	Limestone	4	Alluvium	1
Diabase	6.5	Slate and shale	4	Loam	1
Gabbro	6.5	Sandstone	3	Sand	1
Dolomite	5.5			Loess	0.5
Granite	5				

In *anisotropic* rocks the seismic velocity parallel to the layers $V_{\parallel}$ is always greater than the velocity perpendicular to the layers $V_{\perp}$. The coefficients of anisotropy (defined as the ratio of the velocity along and across the layers) for various rocks vary between 1.0 and more than 4, see Table A3-11. Increase of pressure on rock reduces the effect of anisotropy.

The seismic velocities of rocks generally increase with increasing *pressure.* A generally rapid increase in velocity at low pressures is due to a decrease in porosity from closing of cracks and defects, and an increase in the mechanical contact between the grains.

In addition to the influence from the inherent rock properties the in situ seismic velocities are mainly influenced by:

- the stresses acting;
- the block size (degree of jointing); and
- the opening (aperture) and possible filling of joints.

As for the rocks there is often a rapid velocity increase in *rock masses* with pressure increase at low pressures due to a closing of the joints. In and near the earth's surface with low stress level the joints are generally more or less open. Here, the jointing often has a strong impact on the seismic velocity. With an increase in seismic velocities by depth from increased stress level, a direct comparisons of seismic velocities in the surface and in a tunnel below can generally not be made.

Although there is generally a clear correlation between jointing and seismic velocities the latter also include the averaged effect of the factors mentioned above as further dealt with later in this section.

3.9.2 Methods to asses the degree of jointing from in situ seismic velocities

Correlations between longitudinal seismic velocity and block size may be applied before or after information on jointing from core drilling or surface mapping is available. In this way it is possible to obtain information of the actual jointing at an early stage during investigations. It should be noticed, however, that in these calculations local differences such as the composition of rock types, mineral content, etc. are averaged.
Two different methods have been shown in Appendix 5 to evaluate the degree of jointing (or block size) from seismic velocities measured in the field:

A. When no information is available on jointing at the site.

B. A minimum of two connections between jointing (jointing density and joint condition) and seismic velocities are known.

A. The connection between jointing and seismic velocity is not known.

As the distribution of joints generally is exponential (see Table A1-4 in Appendix 1) the following expression has been found to cover the 1-D joint frequency using data given by Sjögren et al. (1979) and Sjögren (1984, 1993):

$$Nl = 3(v/V_o)^{-V_o/2} \qquad \text{eq. (A3-37)}$$

Here V_o is the basic seismic velocity (km/s) for intact rock under the same stress level as in the field, and
v is the measured in situ seismic velocity (km/s).

Important for the result is the magnitude and accuracy of V_o . Where V_o is not known, it is recommended to use the velocity for intact rock under the same conditions as in the field (wet/dry, same direction relative to the stresses, possible anisotropy, etc.).

From eq. (A3-37) the volumetric joint count (Jv) and the block volume may be calculated applying eq. (A3-33) and the block volume from $Vb_o \approx \beta \times Jv^{-3}$. If the block shape factor is not known, β = approx. 40 may be applied. Joint openness and possible fillings may, however, highly disturb the accuracy of this using input of V_0 estimated from laboratory measurements or from standard tables. Therefore, the method described in the following gives more accurate results as it includes the site-dependent conditions.

B. Two or more correlations exist between jointing and seismic velocities

Sjögren et al. (1979) have presented a method to calculate the degree of jointing from longitudinal sonic velocities. The method is based on known data on jointing and seismic velocity for two different locations on a seismic profile. The degree of jointing given as joints/m is found from the following expression:

$$Nl = (V_n - v)/(V_n \times v \times ks) \qquad \text{eq. (A3-38)}$$

where V_n is the maximum or 'natural' velocity in crack- and joint-free rock under the same stress level as in the field. The velocities for some fresh rocks measured in the laboratory are shown in Table A3-33. [5]
v is the in situ seismic velocity recorded, and
ks is a constant representing the actual in situ conditions.

---

[5] The difference between V_n and V_o is shown in Fig. A5-7 in Appendix 5.

TABLE A3-33 TYPICAL SEISMIC VELOCITY VALUES OF FRESH ROCKS, FREE FROM CRACKS AND PORES (after Goodman, 1989).

Rock	V_n (km/s)	**Rock**	V_n (km/s)
Gabbro	7	Basalt	6.5 - 7
Limestone	6 - 6.5	Dolomite	6.5 - 7
Sandstone and quartzite	6	Granitic rocks	5.5 - 6

It is, however, generally seldom possible to measure V_n at the surface as the (often weathered) rocks in the surface seldom are free from joints, cracks and pores. Therefore, V_n is better found indirectly using two data sets of measured values of jointing (Nl) and the corresponding seismic velocity (v). From these data, it is possible to calculate the natural or maximum velocity

$$V_n = \frac{v_1 \times v_2 \left(Nl_2 - Nl_1\right)}{Nl_2 \times v_2 - Nl_1 \times v_1} \qquad \text{eq. (A3-39)}$$

and the factor representing in situ conditions

$$ks = \frac{1}{Nl_1}\left(\frac{1}{v_1} - \frac{1}{V_n}\right) \qquad \text{eq. (A3-40)}$$

Here Nl_1, v_1 and Nl_2, v_2 are corresponding values of joints/m and in situ seismic velocity respectively for the two pairs of measurements.

After V_n and ks are determined they are applied in eq. (A3-38) which can be used to work out a curve representing the actual connection between the measured jointing density and the sonic velocities can be established to quicker 'translate' velocities into block size (or jointing density).

From eq. (A3-38) the volumetric joint count (Jv) may roughly be calculated as described for alt. A. Thus at the stage, when the joint frequency has been measured in drill cores or from observations in rock exposures, the accuracy of the seismic velocity versus jointing can be significantly improved. This can be used to improve the interpretation of jointing as the information collected in the very limited volume of the rock mass covered by the borehole, can be extended to cover the whole seismic profile.

3.9.3 Possible errors and limitations applying seismic velocities for jointing assessments

There are several limitations when using of seismic refraction velocities in rock mass quality assessments. These mainly stem from the fact that there are several factors in a rock mass that influence the seismic velocity. It is impossible to avoid errors and uncertainties when the velocity is used to assess only one or a few of these.

Seismic velocities cannot be used to assess the joint condition (roughness and alteration of the joint surface; filling and size of the joint). The probable effect of these features should be assessed in the geological interpretation. A good knowledge of the geological conditions linked with practical experience may possibly reduce these limitations.

The increased pressure by depth reduces the possibilities of the refraction seismic measurements to effectively express variations in the degree of jointing. Therefore, refraction seismic measurements give the best results near the surface where the stress level is low.

3.10 Summary of the correlations to determine the block size

The vast variations in jointing densities and block sizes as well as in jointing patterns cause that it is very difficult to outline one or two methods for characterization which cover the whole range of variations. Sometimes the type of characterization chosen must be accompanied by a second one to fully include the conditions in the actual rock mass.

The variation range for three different methods for jointing density characterization is shown in Fig. A3-28. RQD covers only a limited area compared to the volumetric joint count (Jv) and block volume (Vb). As mentioned in Chapter 9 RQD is not a single measure for jointing density or block size, but is also including core loss and possible weathering (Bieniawski, 1984, 1988). The often poor quality of this method to characterize the degree of jointing is outlined in Appendix 4.

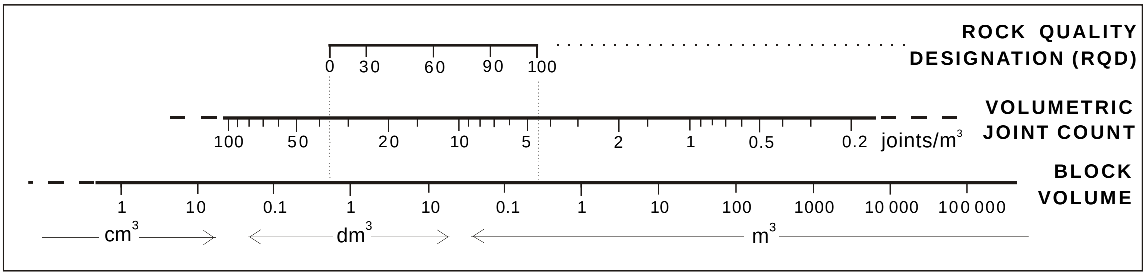
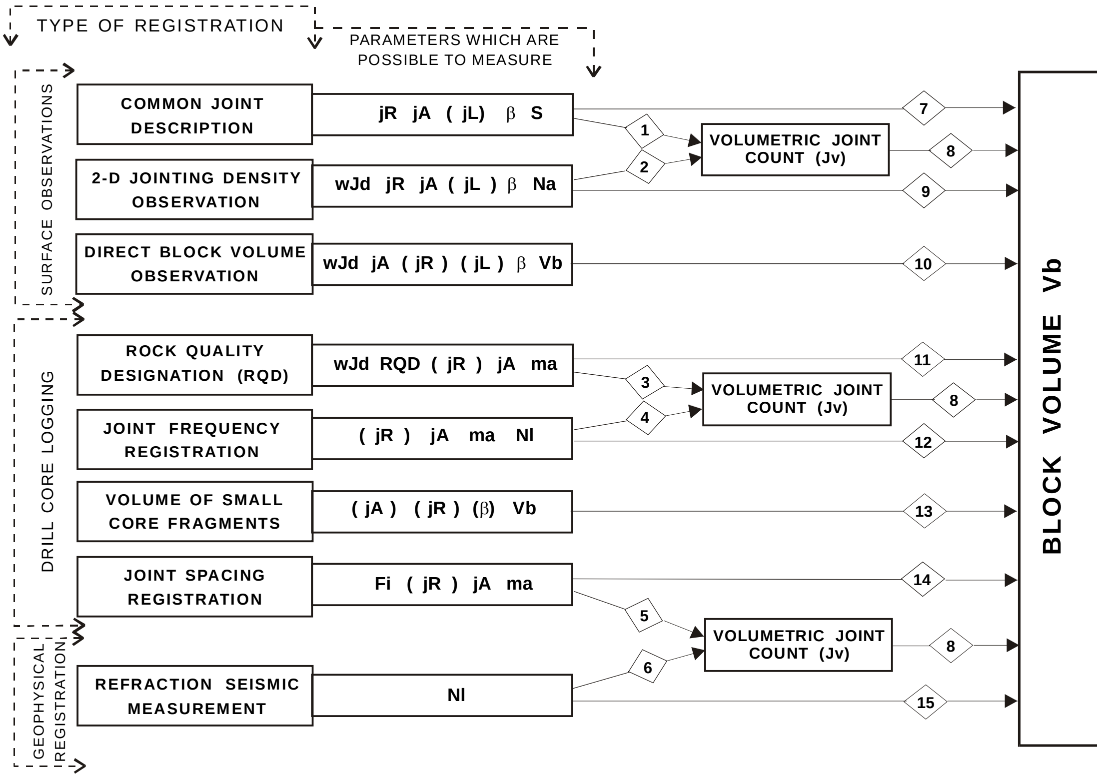


Fig. A3-28 A rough correlation between three methods for joint density measurements



ABBREVIATIONS

jA = joint alteration factor
jR = joint roughness factor
jL = joint size factor
wJd = weighted jointing density
β = block shape factor
S = joint spacing
Vb = block volume
Na = number of joints per m²
Nl = number of joints per m
Fi = fracture intercept (joint spacing)
◇ appropriate equation number
( ) cannot always be observed

Fig. A3-29 Correlations between various jointing density characterizations. The numbers refer to the equations shown in Table A3-34.

Joint spacing and joint frequency or fracture frequency should not be applied as a single measures for jointing density characterization unless it is clearly defined what they cover and how they have been applied.

The connections between the main types of jointing density, degree of jointing and block volume measurements are indicated in Fig. A3-29.

TABLE A3-34 TRANSITIONS BETWEEN VARIOUS TYPES OF JOINTING DENSITY REGISTRATIONS

	MEASUREMENTS or OBSERVATIONS MADE ON SURFACES		**REGISTRATIONS MADE ON DRILL CORES or SCANLINES**
	3-Dimensional	**2-Dimensional**	**1-Dimensional**
	From measured spacings for each joint set (S): $\mathbf{Jv = \Sigma\ (1/S_i)}$ [1] or where also random joints (Nr) occur: $\mathbf{Jv = \Sigma\ (1(S_i) + \Sigma\ (Nr/5)}$ [1]	From the average joint spacing (S_m): $\mathbf{Jv = S_m / ka}$ [2] From the average joint frequency (Na): $\mathbf{Jv = Na \times ka}$ [2] From weighted 2-D joint density measurement (wJd): $\mathbf{Jv = wJd}$ [2] From refraction seismic velocity (v): $\mathbf{Jv = 3\ kl\ (v/V_0)^{-0.5\ Vo}}$ [6]	From the average length of core pieces (fracture intercept, Fi): $\mathbf{Jv = Fi / kl}$ [5] From the average joint frequency registration (Nl): $\mathbf{Jv = Nl \times kl}$ [4] From weighted 1-D joint density measurement (wJd): $\mathbf{Jv = wJd}$ [4] From rock quality designation (RQD): $\mathbf{Jv = 35 - RQD/ 3.3}$ [3]
	From the volumetric joint count (Jv): $\mathbf{Vb_o = \beta \times Jv^{-3}}$ [8] From joint spacings (S1, S2, S3) in 3 joint sets: $\mathbf{Vb_o = S1 \times S2 \times S3}$ [7] From registration on site: **Vb = volume of block measured in situ** [10]	From weighted 2-D joint density measurement (wJd): $\mathbf{Vb_o = \beta \times wJd^{-3}}$ [9] From average joint frequency (Na): $\mathbf{Vb \approx \beta\ (Na \times ka)^{-3}}$ [9] From registration on site: **Vb = volume of block estimated from jointing pattern** [9] From refraction seismic velocity (v): $\mathbf{Vb_o \approx 0.04\beta \times kl^{-3}\ (v/V_o)^{1.5\ Vo}}$ [15]	From weighted 1-D joint density measurement (wJd): $\mathbf{Vb_o = \beta \times wJd^{-3}}$ [12] From average joint frequency (Nl) or spacing (Nl = 1/S) registration: $\mathbf{Vb \approx \beta\ (Nl \times kl)^{3}}$ [12] [14] From registration in drill cores (only pieces of core diam. or less size): **Vb = volume of small core fragments** [13] From rock quality designation (RQD): $\mathbf{Vb_o \approx \beta\ (35 - RQD/3.3)^{-3}}$ [11]

Comments:

Vb_o = block volume where joints or block faces intersect at right angles. For intersections at other angles the volume can be found from: $\mathbf{Vb = Vb_o / (\sin\gamma 1 \times \sin\gamma 2 \times \sin\gamma 3)}$
($\gamma 1$, $\gamma 2$, and $\gamma 3$ = angles between the joint sets or between the block faces)

$\mathbf{Nr = \Sigma(nr_i \cdot Lr_i) / \sqrt{A}}$ where nr = the number of joints with length Lr, and
A = the area of the observation surface

β = block shape factor; it may be estimated from $\mathbf{\beta = 20 + 7\ S3/S1}$
(S3 and S1 are longest and shortest block dimension)
β = 27 - 35 for equidimensional (compact) blocks, β = 35 - 50 for slightly long or slightly flat blocks,
β = 50 - 150 for most long and flat blocks, β = 150 - 500 for very long or very flat blocks.

Values of the correlation coefficients in the joint frequency equations:
ka = 1 - 2.5 (average ka = 1.5) kl = 1 - 7 (average kl = 2)

Approximate basic velocity (km/s) of intact rock:
V_o = 7 for gabbo, V_o = 6 - 6.5 for limestone, V_o = 5.5 - 6 for granite, V_o = 6 for basalt and dolomite,
V_o = 6 for sandstone and quartzite

4 METHODS TO CHARACTERIZE THE TYPE AND SHAPE OF ROCK BLOCKS

"Incomplete geological information on time is worth far more than complete information after decisions have been made and carried out."
Burwell and Roberts (1950)

The pattern of joints in the volumes of rocks in the earth's crust occurs as lines in a surface plane where number of joint sets, the size (lengths), the relative differences in spacings, and the angles between them present the main characteristics, refer to Appendix 1. In this work, the jointing pattern in a rock volume is expressed as the type and shape of the rock block delineated by the joint planes shown in Fig. A3-30.

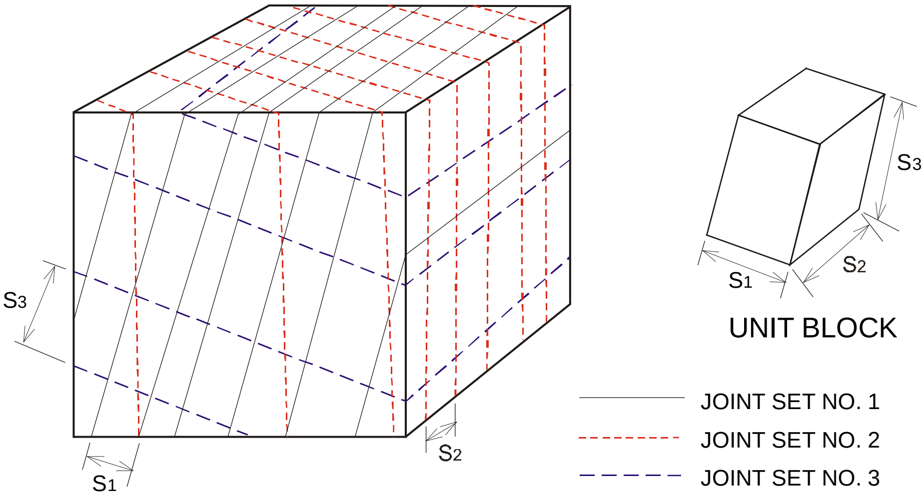


Fig. A3-30 Rock blocks delimited by joint sets (from Palmström, 1982).

As described in Appendix 1, Section 2.3 and 2.4 the block *type and shape* is determined by
- the number of joint sets;
- the difference in joint spacings; and
- the angles between the joints or joint sets;

while the block *volume* depends mainly on:
- the spacings;
- the number of joint sets; and
- the angles between joints or joint sets.

The block volume Vb can be found from the joints spacings or, as shown in Section 3.5.1, from the volumetric joint count and the block shape factor β. Methods to determine the block shape factor are described in this section.

It is mentioned in Appendix 1, Sections 2.3 and 2.4 that the types of blocks delineated by joints have been characterized in different ways and by different terms. Where relatively regular jointing exists and extensive joint surveys have been carried out, it may be possible to give adequate characterization of the jointing pattern according to the system presented by Dearman (1991), see Section 2.3 and 2.4 in Appendix 1. In most cases, however, there is not a regular jointing pattern, therefore a rougher type of block shape characterization is generally more practical, similar to that presented by Sen and Eissa (1991), who divided the types of blocks into the three main groups shown in Table A3-36 in which also the terms applied in this work are indicated.

The expression for the block shape factor given in eq. (A3-28) is in Section 3.5 is

$$\beta = \frac{(\alpha 2 + \alpha 2 \times \alpha 3 + \alpha 3)^3}{(\alpha 2 \times \alpha 3)^2}$$

where $\alpha 2$ = S2/S1 and $\alpha 3$ = S3/S1 are the ratios between the two longest sides and the shortest block face (S3 and S2 > S1, being the length of the faces). The connection between $\alpha 2$, $\alpha 3$ and β is shown in Fig. A3-31 where also the division into various block types is shown.

TABLE A3-36 TERMS USED TO CHARACTERIZE THE MAIN TYPES OF BLOCKS

Common terms used for block type	Terms used in this work
Equidimensional, cubical or blocky blocks	Compact blocks
Elongated, long, columnar or bar blocks	Long blocks
Tabular, platy or flat blocks	Flat blocks
	'Long & flat' blocks (a combination of platy and long blocks)

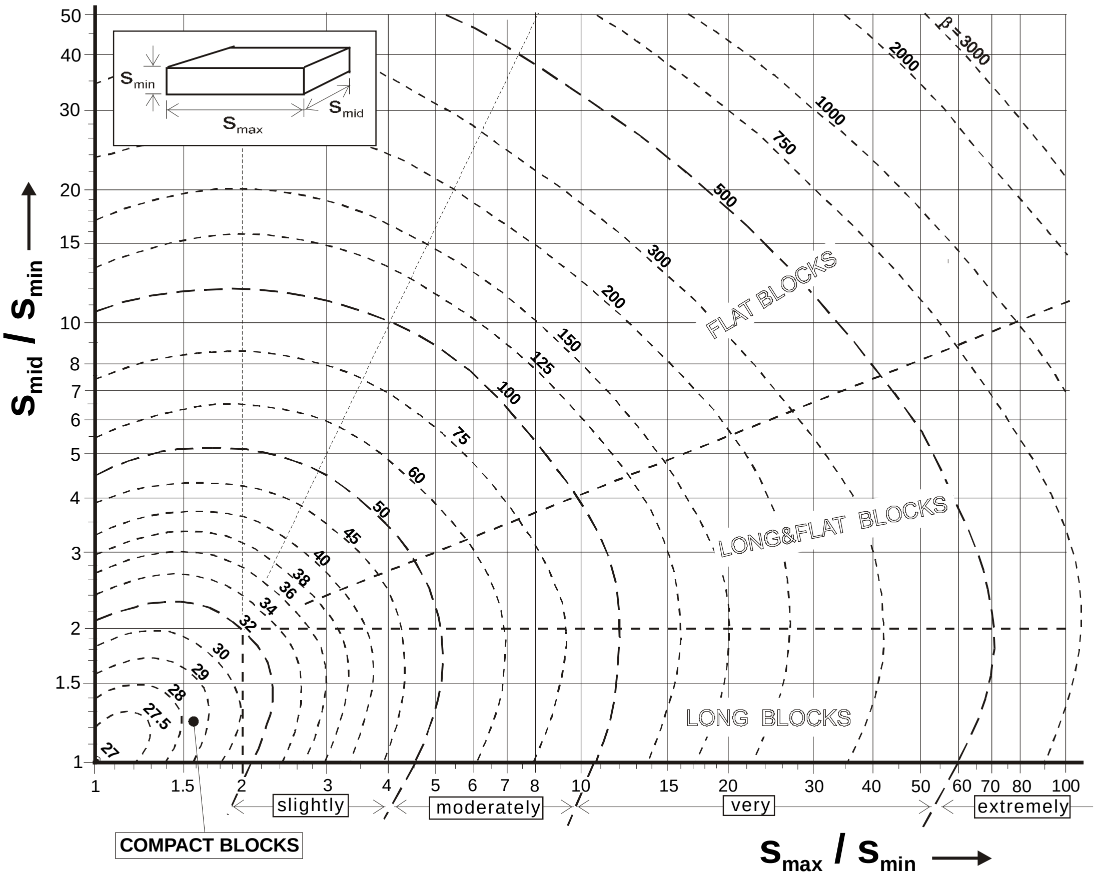


Fig. A3-31 Block types characterized by the block shape factor, β, found from the ratio between the longest and shortest side or joint spacing. The data are based on block shapes at right angles. Refer to Section 2 in Appendix 4.

TABLE A3-37 DEFINITION OF BLOCK TYPES, REFER TO FIG. A3-31

RATIO OR DIFFERENCE IN BLOCK LENGTHS	TYPE OF BLOCK
$\alpha 2 \leq 2$ and $\alpha 3 < 2$	Compact or blocky block
$\alpha 2 \leq 2$ and $\alpha 3 > 2$	Long block
$\alpha 2 > \{(\alpha 3 - 1)^{½} + 1\}$	Flat block
$2 < \alpha 2 \leq \{(\alpha 3 - 1)^{½} + 1\}$	Long & flat block

The type of block is mainly determined by the difference in dimensions between the block faces. For β = 27 - 32 the block term 'compact' is introduced; this term has been chosen to include cubical, equidimensional, blocky and other existing terms for blocks not being elongated or flat. The division chosen for block types is presented in Table A3-37.

The block shape factor β, described in Section 3.5 is used to further characterize the shape of the different block types according to Table A3-38.

TABLE A3-38 CLASSIFICATION OF THE BLOCK SHAPE FACTOR, β.

VALUE OF β	**TERM (AND BLOCK TYPE)**
32 - 50	slightly (long or flat block)
50 - 100	moderately (long or flat block)
100 - 500	very (long or flat block)
> 500	extremely (long or flat block)

The division into the block types above does not, however, include the impact of the angles between joints or joint sets. As shown in Section 3.5 they also influence on the block volume, refer to eq. (A3-26). As shown in Fig. A3-32 the angles between the block faces (or joint sets) delineate:

- right-angled or prismatic blocks;
- rhombohedral blocks; or
- obtuse-angled blocks (where more than 3 joint sets occur).

The value of β can be found using eq. (A3-28) or from Fig. A3-31 provided that the block is limited by 3 parallel pairs of planes for example 3 joint sets. This requires that all the (three) spacings or the dimensions of the (six) block faces are known. As blocks often have more than six faces, it can be difficult to find β from eq. (A3-28). Therefore, a more practical method to estimate β has been developed, as earlier mentioned in eq. (A3-29), from measurement of the longest (S3) and shortest (S1) dimension of the block

$\beta = 20 + 7\ S3/S1 = 20 + 7\ \alpha3$ This is also shown in Appendix 4.

Figs. A4-7 and A4-8 in Appendix 4 show that the shape factor of most types of blocks with β < approx. 1000 can be found from this expression within reasonable accuracy (± 25%). For very - extremely flat blocks eq. (A3-42) should be limited to β < approx. 100. See also Section 3.2.3.

TABLE A3-39 COMMON CONNECTIONS BETWEEN JOINT SETS, TYPES OF BLOCKS AND VALUES OF **β**

Number of joint sets	**Block shape**	**Type of blocks**	**Assumed common range of β**
One joint set only One set plus random	very - extremely moderately - very	flat blocks flat blocks	100 - 5000 75 - 300
Two joint sets Two sets plus random	very - extremely moderately - very	long or flat blocks long or flat blocks	75 -500 50 - 200
Three joint sets[*)] Three sets plus random[*)]	compact blocks to moderately long / flat blocks		27 - 75
Four or more sets[*)]			

[*)] Where there is a significant difference in spacing between the joint sets, very flat and very long blocks can occur also for three or more sets. In these cases the values can be β = 100 - 1000.

Input of a factor for the number of joint sets (Jn) is applied in the Q-system to partly represent block size. In Table A3-39 approximate values of β is correlated to the number of joint sets.

The influence on block volume from the angles between three joint sets can be roughly indicated as:

all angles 90^o,	volume = 100 %
two angles 90^o, one 60^o	116 %
one angle 90^o, two 60^o	130 %
all angles 60^o	150 %
all angles 45^o	280 %

Thus, the effect of joint intersection is relatively limited compared with the variations in spacings.

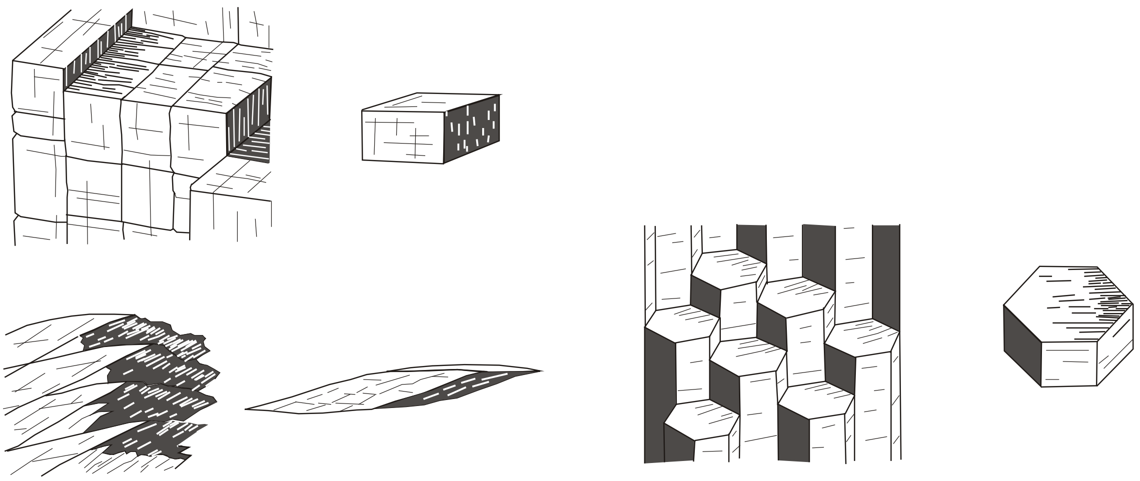


Fig. A3-32 Examples of prismatic, rhombohedral and obtuse-angled blocks (Selmer-Olsen, 1964)

5 "TRANSLATION" OF QUALITATIVE DESCRIPTIONS INTO NUMERICAL VALUES

The previous sections have shown various methods to find numerical values which can be applied to find the rock mass index, RMi. In addition to improved communication a verbal description of a rock mass is generally useful in rock engineering and construction when it contains information on the material and its possible defects. Sometimes, the description of rock masses is qualitative without numerical values. In such cases it is important that numerical values can be estimated. This section briefly outlines some possibilities and problems connected to such 'translations'.

The more detailed and complete the description is, the better quality of the numerical values may be found, especially when defined terms are used. To find the parameters in RMi the description should contain information on:

- the rock material;
- the block volume, degree of jointing, or joint density; and
- the joint characteristics (roughness, alteration, length).

Weakness zones and faults, should as earlier mentioned, be described and characterized individually as they often form separate structural regions. Also singularities, i.e. seams and filled joints may often be described and evaluated individually as separate features when their characteristics are recorded (ISRM, 1978).

Tables and figures covering most parameters in a complete rock mass description are presented in this work. They give definitions, terms and numerical values useful in the 'translations'.

5.1 Rock material characteristics

The geological name generally ought to be fairly simple, such as could be obtained by examination of a hand specimen (Burton, 1965). The strength of the material can be described according to the classification given in Section 1 using terms as 'strong', 'weak', 'hard' etc. Any anisotropic structure of the rock material such as degree of foliation or schistosity can be characterized using the classification in Table A3-9.

As weathering and alteration of rocks are likely to have great influence on their properties and behaviour the description should pay particular attention when such features occur. Terms like 'fresh', 'slightly weathered', etc. defined by to the ISRM (1978) indicate the effect of this feature as described in Section 1.6.

Rock description terms such as outlook of the intact rock, its colour, possible folding details, layering, minerals size etc., can not be directly used in the assessment of numerical values. Such additional information can, however, be of value for better understanding the geological setting and to improve communication. A complete rock description may contain the following features:

Rock material:	-> Geological name, (type of rock) * Orientation of foliation/bedding/layering -> Anisotropy, (schistosity, foliation, bedding) -> Weathering or alteration -> Strength additional information of interest: Folding, colour, appearance, mineral size and texture Porosity, density
-> used as input in RMi * included in stability and boring index	

5.2 Joint characteristics

According to Price (1966) joints may be classified and described with reference to one or more of a number of their characteristics, such as shape, size, the way they occur (bedding joints, foliation joints, etc.), or how they have been formed (cooling joints, tectonic joints, etc.) This tells the reader a lot of the nature of the joints and the jointing.

The type of joint is often indicated by the size and outlook of the joint (termed parting, joint or seam), how it occurs (foliation joint, bedding joint, sheet joint etc.) or how it is made/formed (tension joint, shear joint etc). The type of joint may often indicate several of the other characteristics, such as size, thickness, partly roughness. Therefore, describing this feature may improve the understanding of the site conditions.
The various terms for joint condition described in Section 2.1, such as joint roughness, size and alteration can easily be linked in a description, for example: smooth & planar, clay-coated foliation joint.

Sometimes, when the character or condition of the joints is not given, it can be assumed from the rock material in which the joints occur, because the development of the joint often is determined by the properties of the material, refer to Appendix 1. The main elements to be included in a description are:

Joint condition:	> Type of joint (foliation parting, bedding joint, etc.), -> Roughness (combination of smoothness and waviness) -> Joint size and continuity -> Joint alteration, included coating and filling > Aperture, thickness * Orientation
	-> used as input in RMi > partly used as input in RMi * included in stability and boring index

5.3 Block size or density of joints

The blocks formed by the detailed jointing described in terms of blocks/fragments is meant to express the density of joints and the jointing pattern. It is very important that this description is carried out in a form that can be 'quantified' into block volume and block shape because this feature cannot be estimated from other information in a description. By applying defined terms like small, moderate, large etc. for the block volume or joint density the description can be readily translated into numerical assessments.

Block type and shape, is a result mainly of the spacings or difference in spacings between the joint sets, but often also influenced by the angle between the joints or joint sets. These features are not directly used in RMi, but block shape is an important feature when the block volume is calculated from joint density measurements.

Much work is required to measure angles between block faces, especially for irregular jointing. This is the reason why this parameter is seldom used in descriptions, except where more detailed characterizations are required. Rough terms like prismatic, rhombohedral and obtuse (see Section 4) requires, however, an understanding of the distribution of joints in three-dimensions. In consequence, they cannot be found from drill cores alone.

The main elements to be included in describing the jointing density (block size) and pattern are:

Joint density and pattern:	-> Block size (volume or any measure for density of joints) > Block shape * Block diameter additional information of interest: Angles between joints or block faces
	-> used as input in RMi > partly used as input in RMi * included in stability analysis

Example: Small prismatic blocks => The volume of the blocks are in the range
$Vb = 0.2 - 10\ dm^{3.}$ and the block shape factor is probably $\beta = 30 - 40$.

5.4 Faults and weakness zones

Moderate and large weakness zones should, as suggested by Bieniawski (1984), be described individually. A complete description useful in assessment of numerical values should, in addition to location and orientation, contain such features as:

Weakness zone or fault, and singularity (seam):	-> Type of zone/seam and structure/composition > Size (thickness) > Block size / fragment size > Condition of joints > Type of gouge or filling material * Orientation * Condition of adjacent rock masses
-> used as input in RMi > partly used as input in RMi * included in stability analysis	

Description of weakness zones is also outlined in Appendix 2, Section 4.

5.5 Examples of numerical values found from qualitative descriptions

Complete descriptions may easily be comprehensive if all parameters and features of importance are included. In many cases only some of the features applied in RMi are described; in such cases rough estimates have to be made for those parameters missing. No assessments of the numerical values of a rock mass can, however, be made unless the description contains

- the block volume, jointing density, or degree of jointing, and
- the type or the strength of the rock.

The joint condition can, however, be roughly estimated based on an assumption of 'normal' characteristics, i.e. the joint condition factor varies between jC = 1 - 2.

5.5.1 <u>Example 1 (from Gjövik Olympic (underground) Stadium)</u>

Precambrian red and grey gneiss with a composition varying from granitic to quartzdioritic. The jointing is sometimes irregular, but often two to three joint sets are found, resulting in moderate degree of jointing. The main joint set occurs along the foliation of the gneiss, with mainly 2 - 5 m long joints spaced 0.2 - 0.5 m. The joints are smooth to rough and undulating with no filling, but in a few joints filling of clay, chlorite, silt/sand occurs; also calcite, epidote and quartz is found. Tectonism has resulted in an additional network of micro-joints sometimes with clay coatings.

'Translation' of the description:

- In Table A3-8 the average uniaxial compressive strength of fresh gneiss is given as σ_c = 130 MPa.
- The joint condition factor $jC = jL \times jR/jA = 3$ (mainly slightly rough and undulation joints (in Table A3-17) gives jR = 3.0; fresh joint character, i.e. jA = 1 (Table A3-20); 2 - 5 m long joints gives jL = 1 (Table A3-24)).
- Average block volume calculated from spacing S1 = 0.2 - 0.5 m of the main joint set and assumed spacing 0.5 m and 1 m for the two other sets give Vb = 0. 1 - 0.25 m^3. (Moderate degree of jointing indicates a block volume in the range Vb = 0.01 - 0.2 m^3)

By applying eq. (4-4) and the values above the jointing parameter is
- $JP = 0.2\sqrt{jC} \times Vb^{D} = 0.345\ Vb^{0.297} = 0.175 - 0.23$

  (here D = 0.297 for jC = 3) (JP can also be found from Fig. 4-4.)

Thus **RMi = σ_c × JP = 22.75 - 30**.

5.5.2 Example 2

Fresh, amphibolitic gneiss. The main joint set which occurs along the foliation, has a joint spacing of 0.3 - 1 m with 5 - 20 m long undulating, smooth - rough joint surfaces, often with staining. Frequently, small joints with strongly undulating and rough, fresh surfaces occur across foliation. They are generally spaced 0.5 -1 m. The blocks formed are long&flat blocks. They have often calcite coatings.

'Translation' into numerical values:
- Assumed compressive strength of amphibolitic gneiss is:
  $\sigma_c = \sigma_{c\ max}$ /fA = 160/1.5 = 106 MPa (σ_c and fA have been found Table A3-8 and A3-9)
- The resulting joint condition factor: jC = 2 found from:

  Main joint set: jR = 1.5 - 3 (smooth - rough & undulating, see Table A3-17)
  jA = 1 (staining only (Table A3-20))
  jL = 0.8 (5 - 20 m long, probably continuous, see Table A3-24)
  jC = 1.2 - 2.4

  Other joints: jR = 4 (rough & strongly undulating)
  jA = 3 (joint wall contact, calcite coating)
  jL = 2 (small joints)
  jC = 2.7
- Block volume Vb = 0.1 - 3 m^3 (found from spacing S1 = 0.3 - 1 for the main joint set; S2 = S3 = 0.5 - 2 m for other joints)

  JP = 0.15 - 0.4 is found from Fig. 4-4 and **RMi =** σ_c × **JP = 30 - 80**

5.5.3 Example 3

Palaeozoic mica schist with the main joint set along foliation spaced S = 0.2 - 0.5 m. The 1 - 5 m long foliation joints are smooth, strongly undulating with thin clay coatings. Random, irregular, short joints occur with 'random spacing' roughly 3 - 5 m. The jointing result in large, generally flat blocks.

'Translation':
– Rocks: mica schist, assumed compressive strength σ_{cmax} = 100 MPa (assuming that this is the maximum value). Applying an anisotropy reduction factor of fA = 2 (from Table A3-9) the compressive strength is σ_c = 50 MPa.
– The resulting joint condition factor: jC = 1 has been estimated from the following:
  - main joint set: jC = 0.75 (smooth & strongly undulating: jR = 2; wall contact, clay coating: jA = 4; 1- 5 m length: jL = 1.5)
  - random joints: jC = 5 (irregular joints: jR = 5, jA = 1; assume jL = 1)
– (The main joint set is considered to have the strongest influence on jC)
– Average block volume Vb = 1 - 10 m^3 (estimated from joint spacings).
– With JP = 0.2 - 0.5 (from Fig. 4-4), **RMi = 10 - 25**

5.5.4 <u>Example 4</u>

Medium sized, long blocks of unweathered basalt with mainly rough and slightly undulating faces.

'Translation':
- Basalt may have a compressive strength $\sigma_{c\ average}$ = 165 MPa.
- Joint condition factor, jC = 3 (rough & slightly undulating surface: jR = 3; assumed fresh joint walls: jA = 1; assumed medium joint size: jL = 1)
- Medium sized blocks: Vb = 10 - 200 dm^3

From this the jointing parameter JP = 0.09 - 0.2 and **RMi = 15 - 33**

5.5.5 <u>Example 5</u>

Strongly jointed, fresh granite.

'Translation':
- For fresh granite (from Table A3-8) average σ_c = 160 MPa.
- Strongly jointed means block volume Vb = 0.2 - 10 dm^3
- As no information is given on joint condition, an it is assumed rough, planar joints with medium length as is common in igneous rocks (see Appendix 1). The joint condition factor is then 1.5 - 2 and eq. (4-5) may be applied:

JP = 0.25 $\sqrt[3]{\mathbf{Vb}}$ = 0.015 - 0.054, and **RMi = $\boldsymbol{\sigma_c}$ × JP = 2.4 - 8.6**

5.5.6 <u>Example 6, description of a weakness zone</u>

The weakness zone (with strike/dip = $70^o/20^o$ in upstream direction) is approximately 20 m thick with tectonized, folded, moderately weathered phyllite. It splits along very smooth and planar, 1 - 5 mm thick clay-coated foliation partings (length approx. 0.2 - 2 m) spaced 1 - 5 cm. Several short, smooth joints cut across the foliation. Some few thin clay seams at an acute angle to schistosity occur occasionally in the 10 m thick central part of the zone. The volume of the loosened fragments are in general 50 cm^3. There is 3 - 5 m of transition to the surrounding rock masses consisting of fresh phyllite containing foliation joints spaced 1 - 3 m and some random joints. In the transition zone the blocks are roughly 1 - 100 dm^3. Most joints in the transition part and in the surrounding rocks are fresh having smooth - slightly rough surfaces with large undulations. The zone may be termed 'clay containing crushed zone'.

'Translation':
a. <u>The central part</u>:
– The moderately weathered phyllite has an estimated uniaxial compressive strength:
– $\sigma_c = \sigma_{c\ fresh}$ /fA/fW = 50/2/2.5 = 10 MPa (from Table A3-8, A3-9, A3-13)
- The joint condition factor: jC = 0.15 (very smooth & planar, i.e. jR = 0.75; partly wall contact + clay coatings, i.e. jA = 10; joint length 0.2 - 2 m gives jL = 2)

Using an average block volume Vb = 50 cm^3 the average jointing parameter is:

JP = 0.0003 and **RMi = 0.003**

(The presence of occasional clay seams may cause an even lower RMi than found above.)

b. The transition part:

– Here, the phyllite is fresh, therefore $\sigma_c = \sigma_{c\ fresh} / fA = 50/2 = 25$ MPa.

- The joint condition factor: jC = 2.5 (smooth - slightly rough & strongly undulating surfaces, i.e. jR = 2.5; fresh joint walls, i.e. jA = 1; joint length factor assumed 'normal', i.e. jL = 1)

– With block volume Vb = 1 - 100 dm^3 the jointing parameter is
JP = 0.04 - 0.15 and **RMi = 1 - 3.75.**

5.6 Summary

A complete rock mass description consists of information on the rock material and the jointing expressed in defined terms. From such a description numerical values of rocks strength, joint condition and block volume may be assessed.

TABLE A3-40 TERMS AND FEATURES WHICH MAY BE USEFUL IN A ROCK MASS DESCRIPTION

ROCK MATERIAL					
Rock type	**Outlook**	**Strength**	**Weathering or alteration**	**Anisotropy or structure**	**Orientation**
Geological name	(Colour) (Lustre) (Texture)	Extremely weak Very weak Weak Medium strong Strong Very strong Extremely strong	Fresh Slightly Moderately Strongly Completely	(Bedded) -Schistose -Foliated Homogeneous (Striped) (Banded) (Layered) (Folded)	(Strike/Dip)
JOINT CONDITIONS					
Smoothness	**Waviness**	**Alteration**	**Separation**	**Joint size*)**	**Joint type**
Slickensided Polished Smooth Slightly rough Rough Very rough Interlocked	Planar Slightly undulating Strongly undulating Stepped Interlocked	Welded or Healed Fresh Weathering grade Type of coating Type of filling	Closed Small separation Moderate No wall contact	Very small Small medium Large Very large	Crack Parting Joint Seam* *) Filled joint* *) *Fracture *Fissure
*) In addition continuity of the joint (i.e. continuous or discontinuous) * *) In addition information of the type of filling material					
BLOCK SIZE, JOINTING DENSITY OR DEGREE OF JOINTING					
Block volume	**Term**	**Block type**	Alternative description instead of block volume: **Joint spacing or frequency** described as		
Extremely small Very small Small Moderate Large Very large Extremely large	Slightly Moderately Very Extremely	Compact or Blocky Long Flat Long&flat	Very small Small Moderate Large Very large		

( ) not included in the definitions and numerical values given in this work
* must be defined
- additional characteristic term (small, moderate, large, etc.) should preferably be given

Tables A3-40 and A3-41 show features applied in descriptions, which may be useful in assessing numerical values of the parameters applied in RMi. Most of the terms, except some terms for rocks, have been connected to ratings in tables presented in this work.

TABLE A3-41 TERMS AND FEATURES WHICH MAY BE INCLUDED IN DESCRIPTION OF A WEAKNESS ZONE

WEAKNESS ZONES		
Type of zone *)	**Size (width)** Central part & Transitional part	**Composition** Central part & Transitional part
- Zone of weak material(s) - Tension (fault) zone - Coarse-fragmented crushed zone - Small-fragmented crushed zone - Sand-rich crushed zone - Clay-rich crushed zone - Foliation shear zone - Altered clay-rich zone - Altered leached zone - Recrystallized/cemented/welded zone	Includes the dimension (thickness) of the different parts of the zone.	Includes the same terms as used for rock material, joint condition and block size in Table A3-40
*) Additional information: - orientation of the zone - condition and composition of surrounding rock masses		