Chapter 6

THE USE OF RMi IN DESIGN OF ROCK SUPPORT IN UNDERGROUND OPENINGS

> *"The basic aim of any underground excavation design should be to utilize the rock itself as the principal structural material, creating as little disturbance as possible during the excavation process and adding as little as possible in the way of concrete or steel support. In their intact state and when subjected to compressive stresses, most hard rocks are far stronger than concrete and many are of the same order of strength as steel. Consequently, it does not make economic sense to replace a material which may be perfectly adequate with one which may be no better."*
> Evert Hoek and Edwin T. Brown (1980)

The purpose of this chapter is to show the use of the RMi in stability analysis and rock support estimates for underground excavations. The first two sections summarize some of the current knowledge on stability and failure modes in underground openings. Based on this, a system using RMi parameters in rock support design has been developed.

To clarify, definitions of a few expressions related to the behaviour of rock masses underground are presented:

Stability is here used to express the behaviour of rock masses related to their *"likelihood of being fixed in position"* (Webster's dictionary).
Stability may be felt as a relative expression. In hard rock tunnelling where often a considerable part of the tunnel can be left unsupported during the construction period as well as during operation, any instability that requires support may be regarded as being a stability problem. Tunnelling in poor rock conditions where continuous use of support or lining is required, "stability problems" are often associated only with those parts of the ground where the "standard" excavation procedure and method of support is inadequate and special measures or solutions are required. In either case, a major objective is to assess the stability behaviour correctly and select safe and economical methods for excavation and support.
Stability is a relative term also in other respects as it may be connected to the required level of safety, which may vary with the use of the construction. The level of safety may also be different in the various countries according to regulations for working conditions and safety, as well as the experience the contractor possesses.

Failure, 'the losing of strength', may, in contrast to instability ('the lack of being fixed in position') be regarded as the follower of instability. It may be simply said that failure is the result of instability. Both failure and instability are used rather inconsistently in the literature as they often overlap.

The term *ground* is frequently used in this chapter. By ground is here meant 'the in situ rock mass subjected to stress and water'.

Rock mass is as mentioned earlier 'rocks penetrated by discontinuities', i.e. the structural material which is being excavated and in which the underground opening is located.

Another expression introduced in this chapter is the *competency of the ground*. It has a similar meaning to 'competency of the rock', i.e. related to the strength of the material compared to the forces acting. While a competent rock bed is defined as "*a rock layer which, during folding, flexes without appreciable flow or internal shear*" (Dictionary of geology, 1972), competent ground is a rock mass or soil having higher strength than the stresses acting on it.

6.1 STABILITY ANALYSES AND ROCK SUPPORT DESIGN

There are no universal standard analyses for determining rock support design, because each design is specific to the circumstances (scale, depth, presence of water, etc.) at the actual site and the national regulations and experience. Support design for a tunnel in rock often involves problems that are of relatively little or no concern in most other branches of solid mechanics. The material and the underground opening forms an extremely complex structure. "*It is seldom possible, neither to acquire the accurate mechanical data of the ground and forces acting, nor to theoretically determine the exact interaction of these*" (Hoek and Brown, 1980).

Therefore, the rock engineer is generally faced with the need to arrive at a number of design decisions in which judgement and practical experience must play an important part. Prediction and/or evaluation of support requirements for tunnels is largely based on observations, experience and personal judgement of those involved in tunnel construction (Brekke and Howard, 1972). Often, the estimates are backed by theoretical approaches in support design of which three main groups have been practised in recent years, namely

- the classification systems,
- the ground-support interaction analysis (and the Fenner-Pacher curves used in NATM),
- the key block analysis.

The complex dilemma of structural analyses of tunnels is described in the guidelines for the design of tunnels of an ITA Working Group edited by Duddeck (1988), from which the following is extracted: *"The result of an analysis depends very much on the assumed model and the values of the significant parameters. The main purposes of the structural analysis are to provide the design engineer with:*

1) *A better understanding of the ground-structure interaction induced by the tunnelling process.*
2) *Knowledge of what kinds of principal risks are involved and where they are located.*
3) *A tool for interpreting the site observations and in-situ measurements.*

The available mathematical methods of analysis are much more refined than are the properties that constitute the structural model. Hence, in most cases it is more appropriate to investigate alternative possible properties of the model, or even different models, than to aim for a more refined model."

The design of excavation and support systems for rock, although based on some scientific principles, has to meet practical requirements. In order to select and combine the parameters of importance for stability in an underground opening the main features determining the stability are reviewed including various modes of failure.

6.2 INSTABILITY AND FAILURE MODES IN UNDERGROUND EXCAVATIONS

Basically, the instability of rock masses surrounding an underground opening may be divided into two main groups (Hudson, 1989):

1. One is block failure, where pre-existing blocks in the roof and side walls become free to move because the excavation is made. These are called 'structurally controlled failures' by Hoek and Brown (1980) and involve a great variety of failure modes (as loosening, ravelling, block falls etc.).
2. The other is where failures are induced from overstressing, i.e. the stresses developed in the ground exceed the local strength of the material, which may occur in two main forms, namely:
   a. Overstressing of massive or intact rock (which takes place in the mode of spalling, popping, rock burst etc.).
   b. Overstressing of particulate materials, i.e. soils and heavy jointed rocks (where squeezing and creep may take place).

Various modes of failures are connected to these groups. Terzaghi (1946) has in his classification worked out a behaviouristic description based on failure modes. Also the new Austrian tunnelling method (NATM) contains a similar description of the ground behaviour which summarizes the main types of instability in underground openings. Both descriptions are shown in Table 6-1.

Additional modes of rock mass behaviour in underground excavations described by Terzaghi (1946) are:

Spalling [1], which refers to the falling out of individual blocks, primarily as a result of damage during excavation.

Running ground, which occurs when a material invades the tunnel until a stable slope is formed at the face. Stand-up time is zero or nearly zero. Examples are clean medium to coarse sands and gravels above ground water level.

Flowing ground, which is a mixture of water and solids, which together invade the tunnel from all sides, including the bottom. It is encountered in tunnels below ground water table in materials with little or no coherence.

This has been envisaged in Fig. 6-1 where the various modes of failures in the 6 main groups of ground are indicated. The groups are defined by the continuity of rock masses as described in Section 1 in Chapter 5 and the quality of the ground, related to jointing for discontinuous rock masses and to the stress/strength ratio for continuous materials.

Most types of rock masses fall within this scheme. In addition to the continuity and the competency of the ground the time factor, the way the particle or blocks move, and the presence of water determine the development and mode of a failure.

Input from experience and knowledge of the behaviour of various types of rock masses in underground openings is important in stability analysis and rock support evaluations. Further, the understanding of how possible failure modes are related to ground conditions is a prerequisite in the estimates of rock support.

[1] This term is often used by other authors as synonymous with popping or mild rock burst.

TABLE 6-1 VARIOUS MODES OF ROCK MASS BEHAVIOUR IN UNDERGROUND OPENINGS; TERMS DEFINED BY TERZAGHI (1946) AND NATM (1993)

Terms applied by Terzaghi	**Term used in The new Austrian tunnelling method (NATM)**
Firm ground is a material which will stand unsupported in a tunnel for several days or longer. The term includes a great variety of materials: sands and sand-gravels with clay binder, stiff unfissured clays at moderate depths, and massive rocks.	**Stable** Elastic behaviour of the surrounding rock mass. Small, quickly declining deformations. No relief features after scaling. The rock masses are long-term stable.
	Loosening Elastic behaviour of the rock mass, with small deformations which quickly decline. Some few small structural relief surfaces from gravity occur in the roof.
Ravelling ground indicates a material which gradually breaks up into pieces, flakes, or fragments. The process is time-dependent and materials may be classified by the rate of disintegration as *slowly* or *rapidly* ravelling. For a material to be ravelling it must be moderately coherent and friable or discontinuous. Examples are jointed rocks, fine moist sands, sands and sand-gravel with some binder, and stiff fissured clays.	**Ravelling** Far-reaching elastic behaviour of the rock mass with small deformations that quickly decrease. Jointing causes reduced rock mass strength, as well as limited stand-up time and active span*) . This results in relief and loosening along joints and weakness planes, mainly in the roof and upper part of walls.
	Strongly ravelling Deep, non-elastic zone of rock mass around the tunnel. The deformations will be small and quickly reduced when the rock support is quickly installed. The low strength of rock mass results in possible loosening effects to considerable depth followed by gravity loads. Stand-up time and active span are small with increasing danger for quick and deep loosening from roof and working face.
Squeezing ground. *Squeezing* rock slowly advances into the tunnel without perceptible volume increase. It is merely due to a slow flow of the material towards the tunnel, at almost constant water content. The manifestations and the causes of the squeeze can be very different for different clays and decomposed rocks. Prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or of clay minerals with a low swelling capacity.	**Squeezing or swelling** The "plastic" zone of considerable size with detrimental structural defects such as joints, seams, shears results in plastic *squeezing* as well as *rock burst phenomena.* Moderate, but clear time-dependent squeezing with only slow reduction of deformations (except for rock burst). The total and rate of displacements of the opening surface is moderate. The rock support can sometimes be overloaded.
Popping or rock burst is the sudden, violent detachment of thin rock slabs from sides or roof, and is caused primarily by the overstressing of hard, brittle rock.	
Swelling ground advances into the tunnel chiefly by expansion from water adsorption. The capacity to swell seems to be limited to those rocks which contain clay minerals such as montmorillonite, and to rocks with anhydrite.	**Strongly squeezing or swelling** Development of a deep squeezing zone with severe inwards movement and slow decrease of the large deformations. Rock support can often be overloaded.

*) Active span is the span of the tunnel or the distance from the lining to the working face if this is smaller. (Lauffer, 1958)

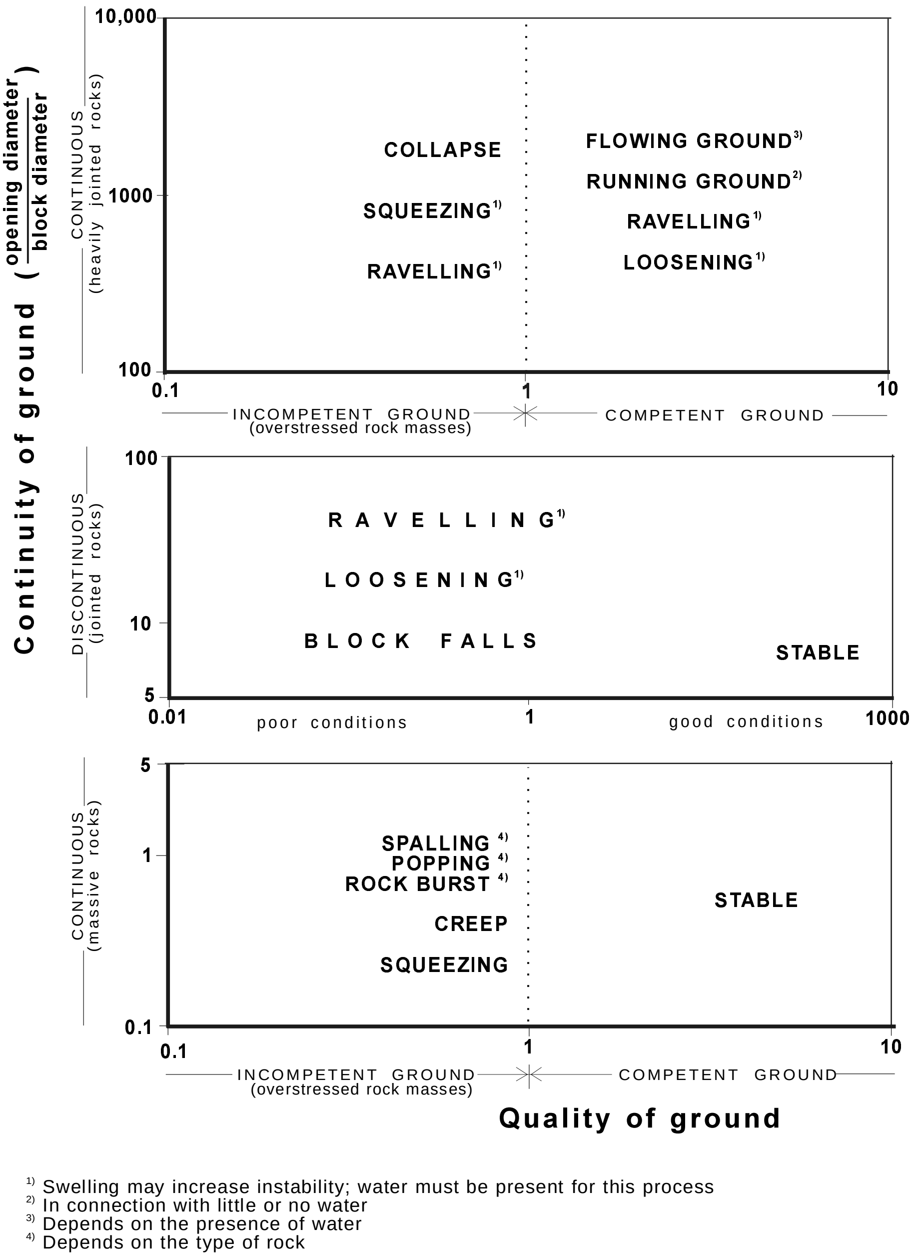

Fig. 6-1 Main types of instability in underground excavations.

6.2.1 *Special modes of instability and behaviour related to faults and weakness zones*

Faults and weakness zones often require special attention in underground works, because their structure, composition and properties may be quite different from the surrounding rock masses. Zones of significant size can have a major impact upon the stability as well as on the excavation process of an underground opening, for instance from possible flowing and running, as well as high ground water inflow. These and several other possible difficulties connected with such zones require that special investigations often are necessary to predict and avoid such events. Bieniawski (1984, 1989) therefore recommends that they are mapped and treated as regions of their own.

Many faults and weakness zones contain materials quite different from the 'host' rock from hydrothermal activity and other geologic processes. The instability of weakness zones may depend on other features than the surrounding rock which all interplay in the final failure behaviour. An important inherent property in this connection is the character of the *gouge or filling material*. Brekke and Howard (1972) has described the main types of fillings in seams and weakness zones and their possible behaviour, as shown in Table 6-2

TABLE 6-2 BEHAVIOUR OF FILLING MATERIAL AND GOUGE IN SEAMS AND WEAKNESS ZONES (revised from Brekke and Howard, 1972)

Dominant material in fault gouge/filling	**POTENTIAL BEHAVIOUR CAUSED BY THE GOUGE MATERIAL**	
	At face	**Later**
Swelling clay	Free swell, sloughing. Swelling pressure and squeeze on shield.	Swelling pressure and squeeze on support or lining, free swell with down-fall or washing if lining is inadequate.
Inactive clay	Slaking and sloughing caused by squeeze. Heavy squeeze under extreme conditions.	Squeeze on supports or lining where un-protected, slaking and sloughing due to environmental changes.
Chlorite, talc, graphite, serpentine	Ravelling.	Heavy loads may develop due to low strength, in particular when wet.
Crushed rock fragments or sand-like gouge	Ravelling or running. Flowing if surplus of water. Stand-up time may be extremely short.	Loosening loads on lining, running and ravelling if unconfined.
Porous or flaky calcite, gypsum	Favourable conditions.	May dissolve, leading to instability of rock mass.

Table 6-2 indicates that most modes of failures can take place in such zones; often two or more may act at the same time making stability evaluations of weakness zones a very difficult task. Fault gouge is normally impervious, with a major exception for sand-like gouge. Otherwise, high permeability may occur in the jointed rock masses adjacent to the fault zone. High water inflows encountered in underground openings when excavating from the weak impervious gouge in the zone, is one of the most adverse conditions associated with faults (Brekke and Howard, 1972).

6.2.2 Main types of failure development

The main modes of failures or instability in underground openings may develop in basically three different ways:

1. Loosening and falls of single blocks or fragments.
2. Collapse; i.e. the tunnel is filled with the fallen blocks which become wedged together and provide support to the remaining loose, unstable blocks.
3. Limited deformations on the surface of the opening caused by the redistribution of the stresses forming a stable arch in the surrounding rock masses.

The first and the last type - though dangerous for the tunnel workers - have generally limited consequences, while the second is the most serious mode of failure and may cause severe construction problems. It may start as a progressive failure, which develops into a collapse in particulate materials (for example highly jointed or altered rock masses).

Terzaghi (1946) has described the typical development as: *"Experience shows that the ground does not commonly react at once to the change of stress produced by the blast. The round blast creates an unsupported section of roof located between the new face and the last support of the tunnel. As soon as the natural supporting rock is removed by blasting, some blocks drop out of the roof, leaving a small gap in the half-dome. If the newly exposed roof section is left without support, some more blocks drop out after a while, thus widening the gap. Finally the entire mass of rock constituting the half-dome drops into the tunnel and a new half-dome is formed. The new half-dome also starts to disintegrate and the process continues until the tunnel section adjoining the working face is filled with rock debris."*
"The rate at which the progressive deterioration or ravelling of the half-dome takes place depends on the shape and size of the blocks between joints, on the width of the joints, on the joint filling and the active span."

These observations stress the need to closely evaluate the timing for installation of rock support and the need to follow-up the development of tunnel behaviour where low stand-up time occurs. This is further described in Section 6.4.4.

6.3 THE MAIN FEATURES INFLUENCING UNDERGROUND STABILITY

The various types of failures described in the foregoing section may be the result of numerous variables in the ground. Both the composition of the rock mass and the forces acting upon it contribute to the result. Wood (1991) and several other authors find that the *behaviour* of ground in an underground excavation depends on
- the generic or internal features of the rock mass;
- the external forces acting, (the ground water and stresses); and
- the activity of man in creating the opening and its use.

Based on published papers, especially the work by Cecil (1970) and Hoek and Brown (1980), and from own experience the following factors have been found most decisive for the stability of underground constructions in jointed rock masses:

1. The *inherent properties of the material (rock masses)* surrounding the opening. They consist mainly of:
   a. Intact rock properties.
   b. Properties of jointing and discontinuities.
   c. Structural arrangement of joints and other discontinuities.
   d. Swelling properties of rocks and minerals.
   e. Durability of the material.
2. The *external forces acting* in the ground:
   a. Magnitude and anisotropy of horizontal and vertical stresses in undisturbed rock.
   b. Ground water.
3. The *excavation features*, such as:
   a. Shape and size of the underground opening.
   b. Method(s) and timing of rock support.
   c. Method of excavation.
   d. Ratio of joint spacing/span width.
4. The *time-dependent features*, mainly consisting of:
   a. The effect of stand-up time.
   b. The long-term behaviour (caused by changes in 1. and 2.)

The influence of these features and their possible application in a method for stability and rock support design are described in the following.

6.3.1 The inherent properties of the rock mass

The geological conditions have generally greater influence on the stability than any other single factor. The exact rock mass conditions at the site will, as mentioned in Chapter 3, not be known until the excavation is made. Some of the variation in rock masses, their composition, occurrence and characteristics have been described in Appendices 1 and 2, and their numerical characterization in Chapter 5.

6.3.1.1 Properties of the intact rock and the discontinuities

The mechanical properties of the rock material can be characterized by the strength of the intact rock, while the joint characteristics and the block size are representative of many of the properties of the discontinuities. The latter are by several authors defined as the main contributors to instability underground. The parameters for rock strength, block volume and joint characteristics are all included in the RMi. They are further described in Chapter 4 and in Appendix 3.

6.3.1.2 Structural arrangement of geologic discontinuities

By the structural arrangement of discontinuities is meant the joint pattern and the geometry of blocks. Also the orientation of discontinuities with respect to the periphery of the opening and the intersection geometry of discontinuities are included. Their intersecting angle with the tunnel determines whether they influence the stability in the walls or the roof of a tunnel.

Cecil (1970) observed that multiple joint sets are most often associated with support and that single sets of joints were frequently of no concern to the stability of an opening. This may be reasonable as many joint sets generally will result in smaller blocks and hence reduced stability. Cecil also noticed that a single random joint may have a very drastic effect on an otherwise stable jointed rock mass (Fig. 6-2).

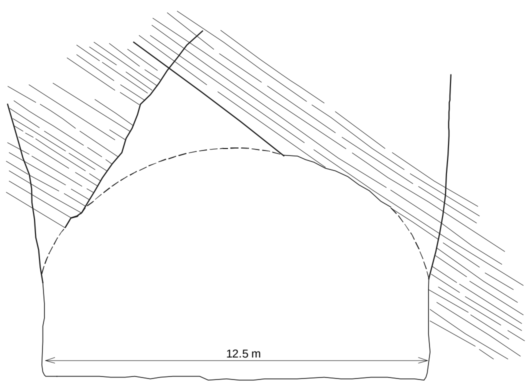

Fig. 6-2 Example of the effect of single joints on stability (from Cecil, 1970).

Deere et al. (1969) indicate from experience that two orientations of joints are particularly important:

- steeply dipping joints (45-90^o) which are parallel or subparallel to the tunnel axis,
- flat-lying (0-30^o) joints occurring in the tunnel roof.

Bieniawski (1984) has classified importance of the orientation of joints in relation to an underground opening, as shown in Table 6-3. The influence from orientation is similar also for weakness zones and singularities.

TABLE 6-3 CHARACTERIZATION OF DISCONTINUITY ORIENTATION RELATED TO AN UNDERGROUND EXCAVATION (revised from Bieniawski, 1984).

STRIKE	**Dip = 0 - 20^o**	**Dip = 20 - 45^o**	**Dip = 45 - 90^o**
Strike across tunnel axis	fair	favourable* unfavourable**	very favourable* fair**
Strike parallel to tunnel axis	fair	fair	very unfavourable

* for drive with dip, ** for drive against dip

6.3.1.3 Swelling properties of rocks and minerals

This is a special property of rocks and soils caused by the expansion of special minerals like smectite (montmorillonite, vermiculite, etc.) and anhydrite upon access to water. The swelling pressure will exert loads on the support in addition to the load from the ground stresses and gravity. Swelling may on some occasions highly influence the stability as well as the problems during the tunnel excavation (Selmer-Olsen 1964, 1988; Brekke and Selmer-Olsen, 1965; Selmer-Olsen and Palmström 1989, 1990).

In addition to several types of soils containing swelling clay minerals (bentonite, etc.) swelling materials can for example occur in:

- altered rock containing smectite;
- sedimentary rock containing anhydrite; or
- clay material in seams or filled joints either occurring singly or as parts of a fault or weakness zone.

6.3.1.4 Durability of the material

Durability is the resistance of a rock against slaking or disintegration when exposed to weathering processes. Some rocks may hydrate ("swell"), oxidize, or disintegrate or otherwise weather in response to the change in humidity and temperature consequent on excavation. An abundant group of rocks, the mudrocks, are particularly susceptible to even moderate weathering (Olivier, 1976). This will change the mechanical properties of rock and hence influence the stability.

6.3.2 The external ground features

The external forces acting on a rock mass surrounding an underground opening are related to its depth and geological location. Their magnitude and orientation may be influenced by the topography in the area, climate, and geological history.

The two main external features are mentioned below. The effect of vibrations from earth quakes or from near-by blastings, or local drainage from other near located tunnels are other features which in addition may influence the stability. Their occurrence and effect are highly connected to local features of the site and the construction to be made.

6.3.2.1 Magnitude of horizontal and vertical stresses in undisturbed ground

The in situ stress level at the location of an underground excavation may have a great impact on its stability where the stresses set up in the rock mass around the opening exceed its strength. Not only high stresses may cause instability problems, also a low stress level may increase instability in jointed rock masses because of reduced shear strength on joints from the low normal stress.

The excavation of the opening disturbs the original, virgin stresses and the stresses set up around the opening may be quite different, depending upon the ratio between horizontal and vertical stresses and the size and the shape of the opening. After the excavation has been mined, these stresses will redistribute. The ratio between the strength of the material and the stresses acting is important in this process. The stresses can be measured where a stress cell can be placed, or it can be estimated from topography, overburden and knowledge of the general stress situation in the region.

6.3.2.2 Ground water

Excessive ground water pressure or flow can occur in almost any rock mass, but it would normally only cause serious stability problems when this takes place in crushed or sand-like materials, or if associated with other forms of instability. Cecil (1970) mentions that the effect of ground water on stability is caused by reducing both the strength of rock material and the shear strength of the discontinuities. As mentioned by Selmer-Olsen (1964), Brekke and Howard (1972) and Selmer-Olsen and Palmström (1989), water can significantly reduce the strength of the filling or gouge in a fault, weakness zone or seam if swelling takes place. Swelling and the following softening also leads to reduced frictional resistance.

Although water may have little influence on stability, the presence of significant quantities of ground water can cause disruption in the excavation process. The most serious problems with ground water occur when it is encountered unexpectedly. Terzaghi (1946) mentions that quantities of 61.3 m^3/min have been experienced in granitic rocks at a depth of 265 m. Large inflows of water can also be experienced in karstic limestones. According to Brekke and Howard (1972) real hazards arise where large quantities of water in a permeable rock mass are released when an impervious fault gouge is punctured through excavation. In this instance, large quantities of gouge and rock can be washed into the tunnel.

Relatively few of the described failures in the literature are specifically related to joint water pressure, it is, however, very possible that ground water can contribute to instability in weak ground. High ground water pressures built up near the excavation have in some occasions caused instability. The impact from ground water pressure should be evaluated in cases where it has significant influence.

6.3.3 The excavation features

Excavation features are the man-made disturbances in the ground. The creation of an excavation includes a number of factors influencing on the stability in the underground opening. The most important of these are mentioned below.

6.3.3.1 The size and shape of the opening

Terzaghi (1946) found that the loads will increase linearly with size of the opening, except for swelling ground where high swelling pressures may develop regardless of the size. For excavations exposed to high rock stresses, Selmer-Olsen (1988) points out the importance of excavating a simple shape of the opening without ledges and overhang to reduce the amount of loosening and spalling. He has shown that, by shaping the tunnel with a reduced curve radius in the roof where the largest in situ tangential stress occurs, it is possible to reduce the area of over-stressing and hence the extent of instability (see Fig. 6-10).

Also, in jointed rocks without overstressing, the stability is improved where a simple shape of the opening is chosen (Selmer-Olsen, 1964, 1988). The stability diagram in the Q-system clearly shows that the amount of rock support depends significantly on the size of the opening. Further, Hoek and Brown (1980) and Hoek (1981) have shown that the shape of the opening has a significant influence on the magnitude of the stresses set up in the rocks surrounding the opening.

6.3.3.2 Method of excavation

It is commonly accepted that any method used to excavate a tunnel will cause some disturbance of the surrounding rock structure, which in turn will affect the stability. The various excavation techniques used may exert different influence on the tendency of blocks to loosen and fall out of the tunnel walls or roof. For example, mechanical tunnel excavation would tend to disturb the blocks much less than drill and blast excavation.

In most cases, it is very difficult to distinguish between the "before" and "after" conditions and whether impact from the excavation may have had an effect on the amount of rock support. Fracturing from blasting leads to reduced block size; actual loosening of rock caused by blasting is, however, often more reflected as ’overbreak’ than by additional support requirements. The influence of blasting can be substantially reduced by controlled *perimeter blasting*. Another result is a smoother surface of the opening.

6.3.3.3 Method(s) and timing of rock support

In overstressed ground where yielding and squeezing take place, the rate and size of deformations depend on the timing and strength of the confinement (method, amount and stiffness of support) placed. This is clearly shown in the ground - support interaction curves (see Chapter 8, Section 8.3).

6.3.3.4 Ratio of joint spacing and tunnel diameter

Deere et al. (1969) suggested that for a tunnel in a jointed or particulate material, a characteristic dimension, such as the tunnel diameter, may be compared with the size of the individual fragments or the joint spacing of the material. This expresses the continuity of the ground as described in Chapter 5, and is regarded as an important parameter representing the effect of block loosening, see Fig. 6-3.

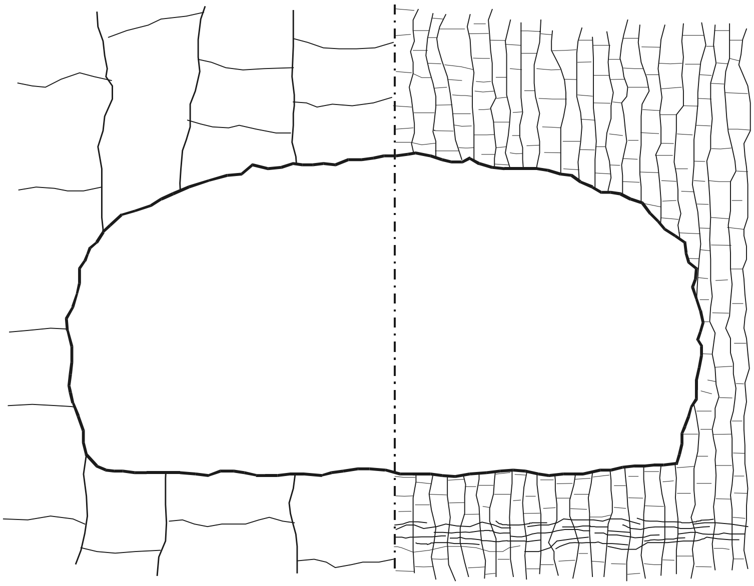

Fig. 6-3 The difference between discontinuous (left) and continuous materials (revised from Barton, 1990b). Increasing the number of blocks in the tunnel surface increases the likelihood of blocks to loosen and the volume involved in a possible failure.

6.3.4 The time-dependent features

When time-dependent behaviour of soil or rock around an underground opening is considered, there are two separate influences:

short term:

- the variation of the stress field as the face advances away from the point concerned in the tunnel (stand-up time), and

long term:

- the creep factors, i.e. creep under constant shear stress,
- the influence from the environment,
- the durability of the rock mass, and
- the effect of ground water.

6.3.4.1 Short term behaviour and the effect of stand-up time

Significant changes in tunnel stability may occur as a result of readjustment of stresses in the walls and roof of a tunnel as the face is advanced. The deformation and stress re-distribution after excavation requires time. The stability effect of this is acknowledged in the 'stand-up time' which was first systematized by Lauffer (1958). Lauffer showed that the property of the rock mass, the active span of the excavation, and the time elapsed until unstable conditions occur, are related to each other. These relations, which are key-points in stability assessments, have been applied in the new Austrian tunnelling method (NATM). Also Bieniawski (1973) has selected the principles of Lauffer in the stability diagram applied in the geomechanics (RMR) classification system.

Earlier, Terzaghi (1946) described the effect of tunnel span on what he called 'the bridge-action period' (stand-up time). *"The bridge-action period for a given material increases very rapidly with decreasing distance between supports. Thus, for instance, a very fine, moist and dense sand can bridge a span one foot wide for several hours. Yet, the same sand would almost instantaneously drop through a gap between supports with a width of five feet."*

The stand-up time diagram by Lauffer is also based on the behaviour of rock mass. A main point in this diagram is that an increase in tunnel size leads to a drastic reduction in stand-up time. The effect of time, therefore, plays an important role when stability evaluation of placing rock support are being made, especially at face when low stability (short stand-up time) conditions are encountered. This is further dealt with in Section 6.4.4.

6.3.4.2 Long-term behaviour

There are several geologic factors that may influence the long-time dependent behaviour of rock masses in an underground excavation. The influence of possible alteration of rock and gouge, or swelling, softening and weakening along discontinuities are factors that must be specially evaluated in each case. These effects are not necessarily obvious during construction, therefore, the long time stability may easily be underestimated during the construction period. Brekke and Howard (1972) have shown the effect of long-time behaviour of fillings in faults and weakness zones (see Table 6-2).

In highly stressed rocks the effect of long-time creep may change the strength of the material as described by Lama and Vutukuri (1978). Another long-term effect is the slaking of mudrocks as mentioned in Chapter 2, Section 2.1.2.

The hydraulic effect of ground water may wash out joint filling materials through piping action, and thus, in this way, influence the long-term stability of the rock masses surrounding the opening.

From this it is clear that the effect of time depends on the conditions at the specific site, and it is difficult to include this effect in a general method of stability analysis.

6.3.5 Summary of Section 6.3

It is not possible to include all the factors mentioned above in a practical system for assessing stability and rock support. Therefore, only the most important features should be selected. Based on the published material mentioned in the foregoing and the author's own experience in this field, the factors mentioned in Table 6-4 are considered the generally most important ones regarding stability and rock support.

Regarding other factors, which influence the stability in underground openings, the following comments are made:

- The effect from *swelling* of some rocks, and some gouge or filling material in seams and faults has not been included.[2] The swelling effect highly depends on local conditions and should preferably be linked to a specific design carried out for the actual site conditions.
- The *long-term* effects must be evaluated in each case from the actual site conditions. These effects may be creep effects, durability (slaking etc.), and access to and/or influence of water.
- In the author's opinion it is very difficult to work out a general method to express the *stand-up time* accurately as it is a result of many variables - among others the geometric constellations. Such variables are generally difficult to characterize by a simple number or value.

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[2] The influence from weakening and loss of friction in swelling clays is, however, included in the joint alteration factor (jA) as input to the joint condition factor (jC) in RMi.

TABLE 6-4 THE GROUND PARAMETERS OF MAIN INFLUENCE ON STABILITY OF UNDERGROUND OPENINGS

THE GROUND CONDITIONS	CHARACTERIZED BY
The inherent properties of the rock mass:	
- The intact rock strength	* The uniaxial compressive strength (included in RMi)
- The jointing properties	* The joint characteristics and the block volume (represented in the jointing parameter (JP))
- The structural arrangement of the discontinuities	(*) 1) Block shape and size (joint spacings) * 2) The intersection angle between discontinuity and tunnel surface
- The special properties of weakness zones	* 1) Width, orientation and gouge material in the zone 2) The condition of the adjacent rock masses
The external forces acting:	
- The stresses acting	* The magnitude of the tangential stresses around the opening, determined by virgin rock stresses and the shape of the opening
- The ground water	(*) Although ground water tends to reduce the effective stresses acting in the rock mass, the influence of water is generally of little importance where the tunnel tends to drain the joints. Exceptions are in weak ground and where large inflows disturb the excavation and where high ground water pressures can be built up close to the tunnel
The excavation features:	
- The shape and size of the opening	* The influence from span, wall height, and shape of the tunnel
- The excavation method	(*) The breaking up of the blocks surrounding the opening from blasting
- Ratio tunnel dimension/block size	* Determines the amount of blocks and hence the continuity of the ground surrounding the underground opening

* Applied in the system for stability and rock support (Section 6.4) (*) Partly applied

There are features linked to the specific case, which should be evaluated separately. They are the safety requirements, and the vibrations from earthquakes or from nearby blasting or other disturbances from the activity of man.

6.4 RMi APPLIED TO ASSESS ROCK SUPPORT

> *"It is essential to know whether the problem is that of maintaining stability with the pre-existing jointing pattern or whether it is the very different problem of a yielding rock mass. The stress situation is therefore one of the main parameters in stability and rock support evaluations."*
> Sir A.M. Muir Wood (1979)

Methods of applying the RMi value directly or some of the parameters in the RMi in the design of rock support are described in this section. The developments made are based on the previous sections in this chapter, published papers, in addition to the author's own practical tunnelling experience.

The behaviour of the rock mass surrounding an underground opening is the combined result of several of the parameters mentioned in the foregoing sections. The influence or importance of each of them will vary with the opening, its location, and with the composition of the rock mass at this location. In a selection of these parameters it has been found beneficial to combine parameters which have similar effects on the stability into two main groups. These are the *continuity factor* and the ground *condition factor*:

- The <u>continuity</u>, i.e. the ratio tunnel size/block size, of the ground surrounding the tunnel which determines whether the volume of rock masses involved can be considered discontinuous or not, see Fig. 6-4. This is important both as a parameter in the characterization of the ground, but also in the determination of appropriate method of analysis. As mentioned in Chapter 5, discontinuous rock masses may have a continuity factor between 5 and 100, else the ground is continuous.

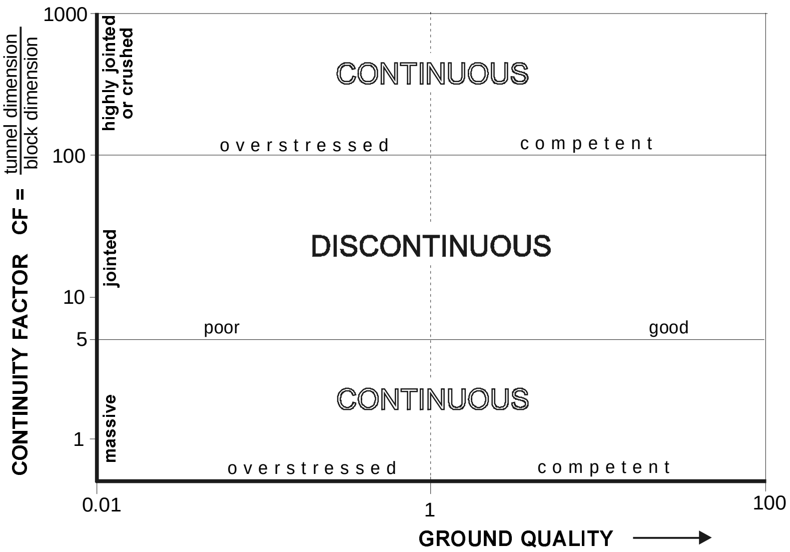


Fig. 6-4 The classification of ground into continuous and discontinuous rock masses.

- The <u>quality of the ground</u>, is composed of important properties of the rock mass and the external features of main influence on the stability of the opening. As pointed out previously, there are different parameters that determine stability in continuous and discontinuous ground. Therefore, in *continuous* ground the competency factor has been applied. It is expressed as

$$Cg = \frac{\text{strength of the rock mass}}{\text{tangential stress around the opening}} \qquad \text{eq. (6-2)}$$

In discontinuous ground, and where weakness zones are involved, a ground condition factor is introduced as further described in Sections 6.4.2 and 6.4.3.

6.4.1 Stability and rock support in continuous materials

Fig. 6-4 shows that continuous rock masses involves two categories
1. Slightly jointed (massive) rock with continuity factor (tunnel size/block size), continuity factor CF < approx. 5.
2. Highly jointed and crushed rocks, continuity factor CF > approx. 100.

Instability in continuous ground can, as mentioned in Section 6.2, be both stress-controlled and structurally influenced. The structurally released failures, which occur in the highly jointed and crushed rock masses, are described in Section 6.4.2 for discontinuous materials. According to Hoek and Brown (1980) they are generally overruled by the stresses where overstressing occurs.

The system for assessment of stability and rock support is presented in Fig. 6-5.

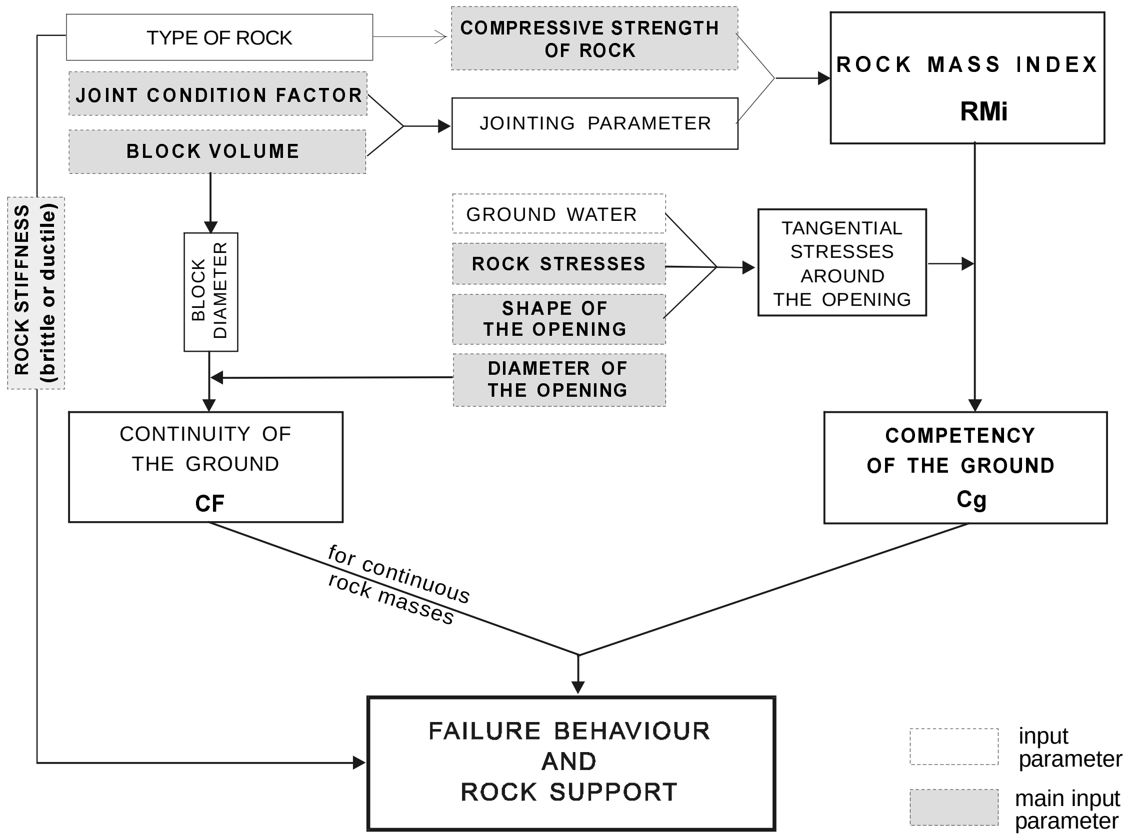


Fig. 6-5 The principle and the parameters involved in assessment of stability and rock support in continuous rock masses.

6.4.1.1 The competency of continuous ground

The excavation of a tunnel disturbs the original rock stresses and the ground water situation in the ground. After the opening is mined, the original stresses are redistributed in the remaining rock mass. This results in local increases in the stresses in the immediate vicinity of the excavation.

As the stress-controlled failures generally dominate the instability in this group of ground the competency factor (Cg) in eq. (6-2) has been selected to characterize the quality of the ground. Cg may be found by combining the induced stresses acting in the rock masses around the opening and the strength of the rock mass. As RMi is valid in continuous ground, and expresses the strength of the rock mass (as outlined in Chapter 4, Section 4.5.1), it can be used in assessing the *competency factor*

$$Cg = \frac{RMi}{\sigma_\theta} \qquad \text{eq. (6-3)}$$

where σ_θ is the tangential stress at different points around the underground opening. It can be found from the vertical rock stress (p_z), the ground water pressure (u_z), and the shape of the opening as outlined in Appendix 9.

Cg indicates whether the material around the tunnel is overstressed or not. This term has earlier been proposed by Muir Wood (1979) as the ratio of uniaxial strength of rock to overburden stress, to assess the stability of tunnels. This parameter has also been used by Nakano (1979) to recognize the squeezing potential of soft-rock tunnelling in Japan.

The greatest influence of the stresses occurs when they exceed the strength of the material, creating incompetent ground, as further outlined in Sections 6.4.1.2 - 6.4.1.4. Such incompetent ground leads to failure if confinement by rock support is not established (Fig. 6-6). If the deformations take place instantaneously (often accompanied by noise), the phenomenon is called *rock bursting*; if the deformations caused by overstressing occur more slowly, *squeezing* occurs, as further described in the following.

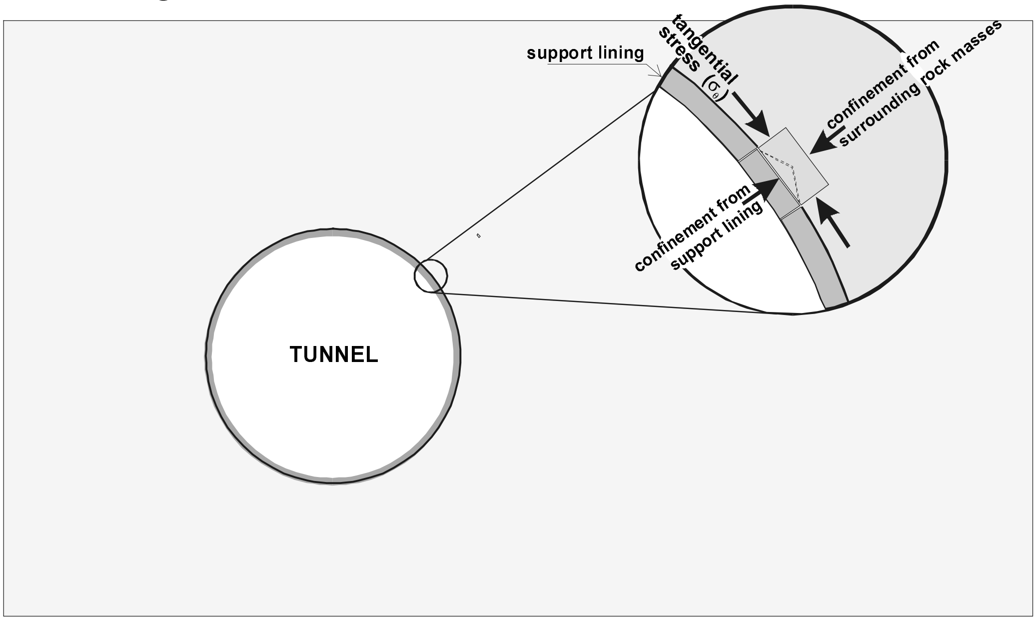
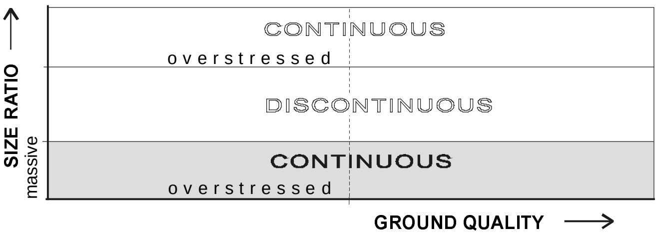


Fig. 6-6 The principle of confinement from rock support of an overstressed element in incompetent ground.

6.4.1.2 Continuous, massive ground



In massive rock with few joints the rock mass index is RMi = f $\times$ σ_c . Thus, the competency of ground is

$$Cg = RMi/\sigma_\theta = f_\sigma \times \sigma_c / \sigma_\theta \qquad \text{eq. (6-4)}$$

where f_σ is a scale factor for the compressive strength, see Section 4.2 in Chapter 4.

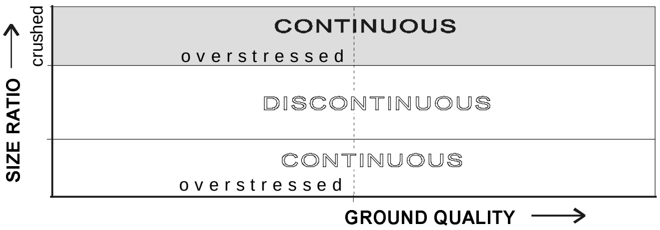
In *competent* massive ground, i.e. Cg > 1, where the new stress condition around the tunnel does not exceed the ground strength at any time, the ground moves into a new position of equilibrium. Structural reinforcement is only required to support possible loosened blocks from unfavourable combinations of the few joints present or of spalls from extension cracking.

In *incompetent* massive ground, i.e. Cg < 1, the overstressing of the rock mass will cause some form of stress induced instability:

- In *brittle* rocks they may cause breaking up into fragments or slabs 'expressed' as rock burst [3] in hard, strong rocks such as quartzites and granites. This is further described in Section 6.4.1.4.
- In the more *deformable, flexible or ductile* rocks such as soapstone, evaporites, clayey rocks (mudstones, clay schist, etc.) or weak schists, the failure by overstressing may act as squeezing; a slow inward movements of the tunnel surface, as outlined in Section 6.4.1.5.

Thus, in overstressed, massive rocks the deformation properties or the stiffness of the material mainly determine which one of the two types of stress problems that can take place.

6.4.1.3 Continuous ground in the form of particulate (highly jointed) materials



This type of ground consists of highly jointed and crushed rock masses as well as soil materials. Instability in such particulate rock masses may develop as two modes:
1) the stress dependent, and 2) the structurally induced failures.

At a relative *low to moderate state of stress* instability is dominated by *structurally controlled* gravity-induced loosening or ravelling of blocks. This disintegration may be slow or rapid. Though the blocks in heavily jointed rock masses in general are smaller than those in discontinuous rock masses (see Section 6.4.2), the properties responsible for their structurally induced instability are similar. This type of instability, which is further described in Section 6.4.2, is often experienced in weakness zones.

In *overstressed* ground instability in the form of squeezing takes place, as further described in Section 4.1.5.

Here, it should be noted that rapid failures in the form of *running ground* and *flowing ground* also may take place. They occur in earth-like rock masses and in particulate soils. Due to the very serious problems and consequences they may produce for tunnel excavation, special considerations regarding investigations, excavation and rock support are generally required. No general use of RMi seems relevant for these conditions.

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[3] Here, 'rock burst' is related to overstressing of the rock. Stress induced failures caused by lower stresses are known as 'spalling', 'popping', 'slabbing', etc.

6.4.1.4 Rock burst and spalling in brittle rocks

Stress induced failures in brittle rocks are known as *spalling,* [4] *popping* or *rock burst*, but also a variety of other names are in use, among them 'splitting' and 'slabbing'. They often take place at depths in excess of 1,000 m below surface, but can also be induced at shallow depth where high horizontal stresses are acting.

Selmer-Olsen (1964) and Muir Wood (1979) mention the importance of differences in magnitudes of the horizontal and vertical stresses. Selmer-Olsen (1964, 1988) has experienced that in the hard rocks in Scandinavia stresses might cause spalling in tunnels located inside valley sides steeper than 20^o and where the top of the valley sides are higher than 400 m above the level of the tunnel. The main reason for this is explained by the very great anisotropy between the maximum and minimum principal stresses (σ_1/σ_2 >10), as described later in Section 6.5.

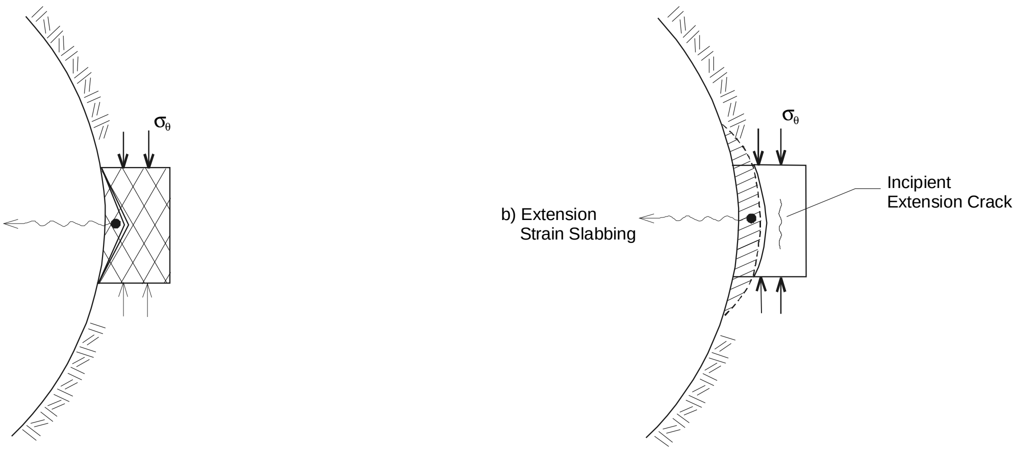

Fig. 6-7 Rock burst in the form of shear failure and 'extension-strain slabbing' in massive rock (from Deere et al., 1969).

The failure illustrated in Fig. 6-7 may consist of small rock fragments or slabs of many cubic metres. The latter may involve the movement of the whole roof, floor or both walls. These failures do not involve progressive failures, except for heavy rock burst. However, they often cause significant problems and reduced safety for the tunnel crew during excavation.

A. Instability assessments

Hoek and Brown (1980) have made studies of the stability of square tunnels in various types of massive quartzite in South Africa. In this region where $k = p_h/p_z = 0.5$, the maximum tangential stress in the walls is $\sigma_\theta \approx 1.4\ p_z$ (see Appendix 9) where the main stability problems occurred. Thus, the rock burst activity can be classified as:

σ_c/σ_θ > 7	Stable
σ_c/σ_θ = 3.5	Minor (sidewall) spalling
σ_c/σ_θ = 2	Severe spalling
σ_c/σ_θ = 1.7	Heavy support required
σ_c/σ_θ = 1.4	Possible rock burst conditions
σ_c/σ_θ < 1.4	Severe (sidewall) rock burst problems.

[4] Terzaghi (1946), Proctor (1971) and several other authors use the term 'spalling' for "drop off of spalls or slabs of rock from tunnel surface several hours or weeks after blasting".

Similarly, Russenes (1974) has shown the relations between rock burst activity, tangential stress on in the tunnel surface and the point load strength of the rock (Fig. 6-8).

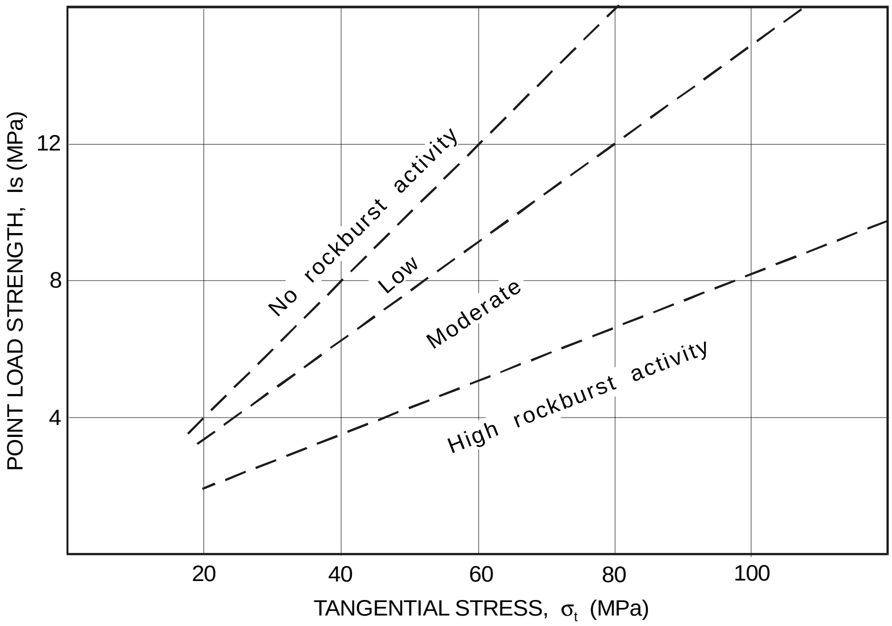

Fig. 6-8 The level of rock burst related to point load strength of the rock and the tangential stress ($\sigma_t = \sigma_\theta$) in the tunnel surface calculated from Kirsch's equations (from Nilsen, 1993, based on data from Russenes, 1974).

The following classification [5] was found for horseshoe shaped tunnels:

$\sigma_c/\sigma_\theta > 4$	No rock spalling activity
$\sigma_c/\sigma_\theta = 4 - 3$	Low rock spalling activity
$\sigma_c/\sigma_\theta = 3 - 1.5$	Moderate rock spalling activity
$\sigma_c/\sigma_\theta < 1{,}5$	High rock spalling/rock burst activity

As seen, these results fit relatively well with the results of Hoek and Brown.

Later, Grimstad and Barton (1993) made a compilation of rock stress measurements and laboratory strength tests and arrived at the following relation, which supports the findings of Hoek and Brown as well as Russenes:

$\sigma_c/\sigma_\theta > 100$	Low stress, near surface, open joints
$\sigma_c/\sigma_\theta = 3 - 100$	Medium stress, favourable stress condition
$\sigma_c/\sigma_\theta = 2 - 3$	High stress, very tight structure. Usually favourable to stability, maybe unfavourable to wall stability
$\sigma_c/\sigma_\theta = 1.5 - 2$	Moderate slabbing after > 1 hour
$\sigma_c/\sigma_\theta = 1 - 1.5$	Slabbing and rockburst after minutes in massive rock
$\sigma_c/\sigma_\theta < 1$	Heavy rockburst (strain-burst) and immediate dynamic deformations in massive rock

The value for σ_c referred to above is related to the strength of 50 mm diameter samples. In massive rock the 'sample' or block size is significantly larger - in the order several m^3. The scale effect for compressive strength determines the value of $RMi = f_\sigma \times \sigma_c$ (see eq. (4-7)). For block sizes in the range of 1 - 15 m^3 $f_\sigma = 0.45 - 0.55$. This means that $\sigma_c = RMi/f_\sigma \approx 2$ RMi; hence, the values of the ratio RMi /σ_θ as shown in Table 6-5 are approximately half of the values for σ_c/σ_θ listed above. The table has been worked out based on this.

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[5] The uniaxial compressive strength σ_c has been calculated from the point load strength (Is) using σ_c = 20 Is.

TABLE 6-5 CHARACTERIZATION OF FAILURE MODES IN BRITTLE, MASSIVE ROCK

Competency factor $Cg = f_\sigma \cdot \sigma_c / \sigma_\theta = RMi / \sigma_\theta$	**FAILURE MODES in massive brittle rocks**
> 2.5	No rock stress induced instability
2.5 - 1	High stress, slightly loosening
1 - 0.5	Light rock burst or spalling
< 0.5	Heavy rock burst

Ideally, The strength of the rock should be measured in the same direction as the tangential stress is acting. Strength anisotropy in the rock may, however, cause that the values of the competency factor in Table 6-5 may not always be representative.

High stresses in massive rock cause new cracks that form slabs parallel to the periphery. Measurements carried out by SINTEF (1990) in the 10 m wide Stetind road tunnel in Norway show that the maximum stresses occur 5 m radially outwards from the tunnel after relief joints have developed around the tunnel, see Figs. 6-9 and 6-10.

This is in accordance with the theories of stress redistribution that the stress peak moves inward in the surrounding rock mass as deformation and cracking take place.

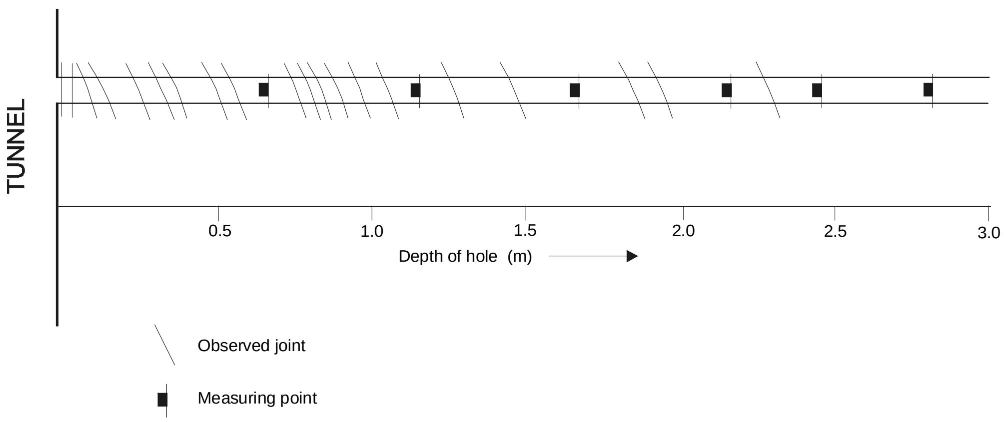


Fig. 6-9 Registration of relief joints in a core drill hole outward from the upper part of the wall. Most joints have been developed within 2.5 m from the tunnel surface (from SINTEF, 1990).

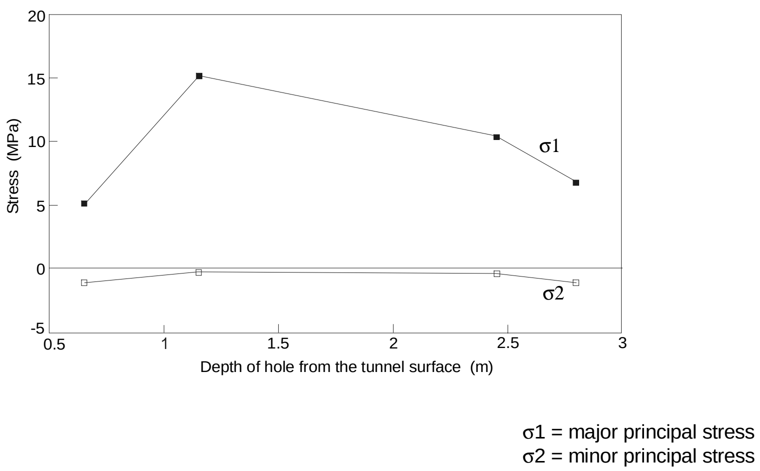


Fig. 6-10 Ground stresses measured in a drill hole from the upper part of the wall. The highest stress was measured 5 m from the tunnel surface (from SINTEF, 1990).

In Scandinavia, tunnels having spalling and rock burst problems are, in most cases, supported by shotcrete (often fibre reinforced) and rock bolts, as this has in practice been found to be most appropriate confinement. The general trends in support design are shown in Table 6-6. Earlier, wire mesh and rock bolts in addition to scaling, were used as reinforcement in this type of ground. This is only occasionally applied in Norway today.

TABLE 6-6 THE GENERAL AMOUNT OF ROCK SUPPORT IN OVERSTRESSED, BRITTLE ROCKS IN NORWAY

Stress problem	**Characteristic behaviour**	**Rock support measures**
High stresses	May cause loosening of a few fragments	Some scaling and occasional spot bolting
Light rock burst	Spalling and falls of thin rock fragments	Scaling plus rock bolting
Heavy rock burst	Loosening and falls, often as violent detachment of fragments and platy blocks	Scaling + rock bolt spaced 0.5 - 2 m, plus 50 -100 mm thick shotcrete, often fibre reinforced

Only two cases have been studied during this work as described in Appendix 7. Their characteristics are shown in Table 6-7. Fig. 6-11 summarizes the various modes of rock burst and appropriate rock support.

TABLE 6-7 ROCK MASS AND GROUND CHARACTERISTICS, AND APPLIED ROCK SUPPORT IN TUNNEL ROOF IN CONTINUOUS GROUND (from descriptions in Appendix 7). THE VALUES HAVE BEEN PLOTTED IN FIG. 6-11

Project	**Ground characteristics** Continuity factor **CF**	 Rock mass index **RMi**	 Competency factor **RMi/σ_θ**	**Applied rock support** B( ) = rock bolt (spacing) F( ) = fibrecrete (thickness)
Stetind road tunnel -chainage 15750	4.2	41.5	0.2	B(1.5 m) F(50 - 80 mm)
Haukrei headrace t. -chainage 200	3.0	54.8	3.4	no support

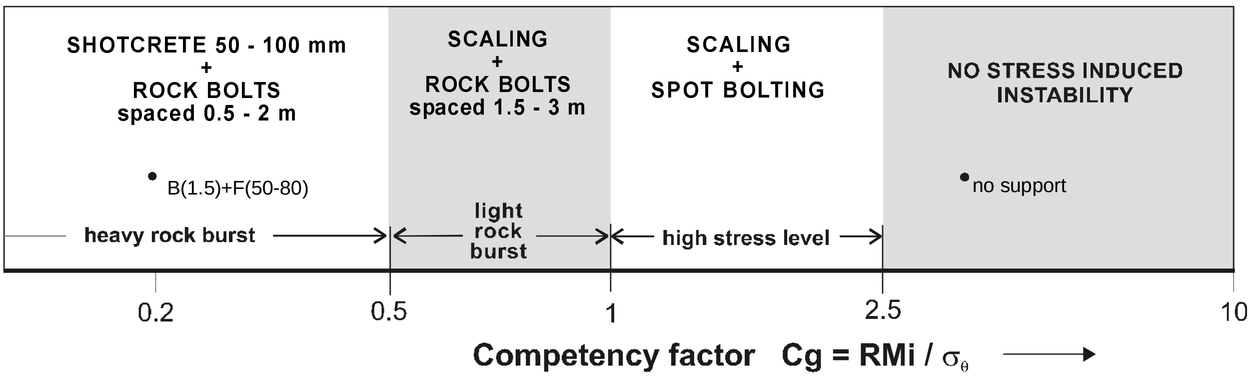


Fig. 6-11 Relationship between the competency factor, failure modes and rock support in continuous, massive and brittle materials.

B. Possible measures to reduce rock stress problems

There is usually some rock breakage from excavation in drill and blast tunnels which contributes to form a zone of relaxation around the skin of the opening (Goodman, 1989). Thus, the cracks from the blasting result in that the stresses redistribute away from the opening. This may explain the experience gained in Scandinavia that rock burst is less developed in blasted tunnels than in TBM tunnels. Increased development of joints and cracks from additional blasting in the periphery of the tunnel is, therefore, sometimes used in Scandinavia to reduce rock burst problems. This experience indicates that rock with joints or fissures is less subject to rock burst than massive rock under the same stress level, as is further described in the next section.

The importance of the shape and size of an excavation upon the magnitude of the stresses and on the stability has been shown by several authors. Through an example Hoek and Brown (1980) show how the amount of rock support can be greatly reduced by optimizing the shape and layout of a cavern. Selmer-Olsen (1964, 1988) mentions that in highly anisotropic stress regimes with rock burst, a method of reducing the extent of rock support is to reduce the radius in the roof where the largest in situ tangential stress occur. In this way it is possible to reduce the overstressed area where highest amount of support is required, see Fig. 6-12.

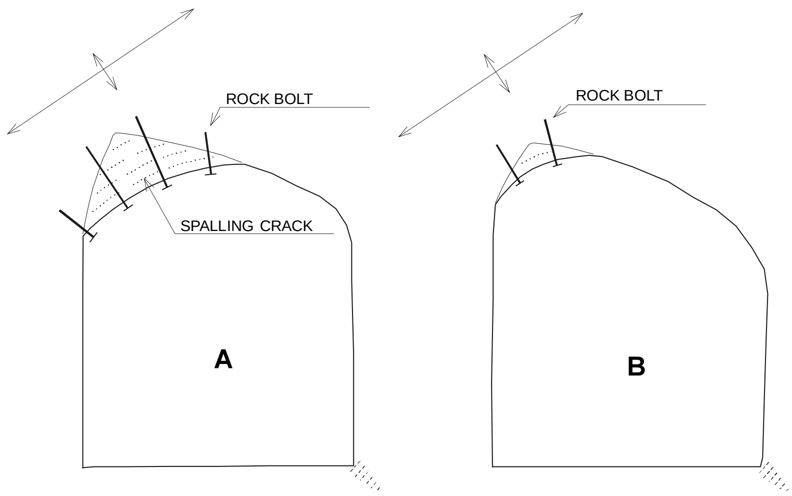
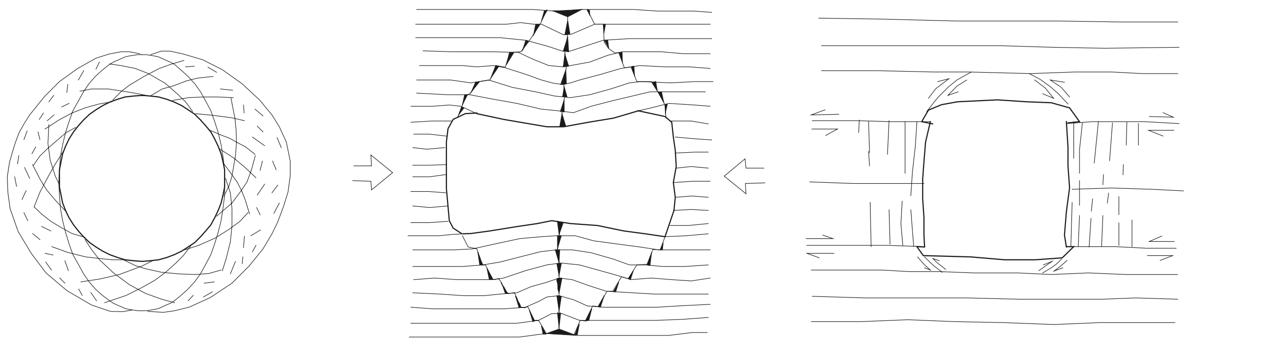


Fig. 6-12 If high anisotropic stresses occur, the extent of spalling (or rock burst) may be reduced by favourably shaping the tunnel. 'A' shows the situation in a tunnel with symmetric shape, and 'B' the situation in the tunnel with an asymmetric shape with reduced radius (from Selmer-Olsen, 1988).

6.4.1.5 Squeezing ground

Squeezing can occur both in massive (weak and deformable) rock and in highly jointed rock masses as a result of overstressing. It is characterized by yielding under the redistributed state of stress after excavation. The squeezing can be very large; according to Bhawani Singh et al. (1992) deformations as much as 17% of the tunnel diameter have been measured in India. The squeezing can occur not only in the roof and walls, but also in the floor of the tunnel.

Squeezing is related to time dependent shearing, i.e. shear creep. A general opinion is that squeezing is associated with volumetric expansion (dilation),as the radial inward displacement of the tunnel surface develops. Einstein (1993) writes, however, that squeezing does not necessarily involve volume increase, and that it often may be associated with swelling.



a complete shear failure,

b buckling failure,

c tensile splitting shearing and sliding

Fig. 6-13 Main types of failure modes in squeezing ground (from Aydan et al., 1993).

Aydan et al. (1993) have pointed out three possible developments of squeezing failures in the ground surrounding an underground opening (Fig. 6-13):

a) Complete shear failure.
   This involves the complete process of shearing of the medium in comparison with the rock-bursting, in which the initiation by shearing process is followed by splitting and sudden detachment of the surrounding rock as shown in Fig. 6-13 (a). It is observed in continuous ductile rock masses or in masses with widely spaced discontinuities.
b) Buckling failure.
   This type of failure is generally observed in metamorphic rocks (i.e. phyllite, mica-schist) or thinly bedded ductile sedimentary rocks (i.e. mudstone, shale, siltstone, sandstone, evaporitic rocks)
c) Shearing and sliding failure.
   This is observed in relatively thickly bedded sedimentary rocks and involves sliding along bedding planes and shearing of intact rock.

The above division has been worked out from 21 tunnels in Japan in which squeezing occurred. The most common rock types are mudstones, tuffs, shales, and serpentinites. Most rocks have compressive strength $\sigma_c < 20$ MPa. From their observations Aydan et al. (1993) have pointed out five main states of straining in the rock masses surrounding the tunnel (Fig. 6-16):

1. Elastic state — Rock behaves almost linearly and no cracking is visible.
2. Hardening state — Microcracking starts to occur and the orientation of microcracks generally coincide with the maximum loading direction.
3. Yielding state — After exceeding the peak of the stress - strain curve, micro-cracks tend to coalesce to initiate macro-cracks.
4. Weakening state — Initiated macro-cracks grow and align in the most critical orientations.
5. Flowing state — Macro-cracks along the most critical orientations completely coalesce and constitute sliding planes or bands, and fractured material flow along these planes.

Other examples of squeezing behaviour are shown in Figs. 6-14 and 6-15.

Fig. 6-17 shows the experience gained from the practical studies made by Aydan et al. (1993). They have used the compressive strength of the rock as the parameter for the materials (which have strengths $\sigma_c < 20$ MPa). No description of the rocks is presented in their paper; it is in the following assumed that the rocks contain few joints, as the presence of joints is not mentioned.

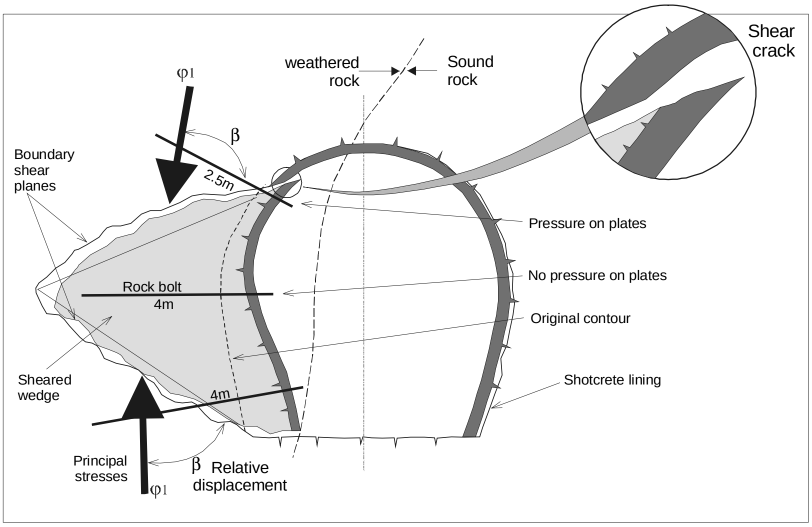


Fig. 6-14 Principle of shear failure in overstressed ground based on ideas from Rabcewicz (revised from Hagenhofer, 1991)

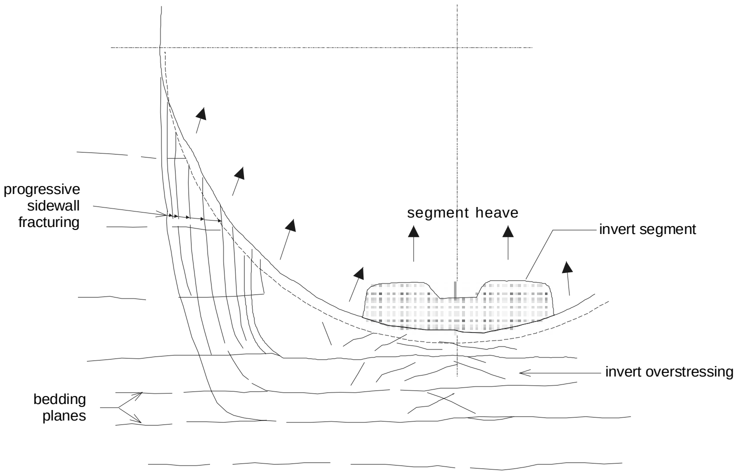


Fig. 6-15 Example of overstressing mechanism in the lower sidewall and in invert of a tunnel in Cyprus (from Sharp et al., 1993)

Applying straight lines instead of the slightly curved ones in Fig. 6-17, the division given in Table 6-8 has been found. In this evaluation the following assumptions have been made:

- $k = p_h / p_z = 1$ and $p_z = \gamma \times z = 0.02$ z (in MPa). (Aydan et al. measured $\gamma = 18 - 23 MN/m^3$)
- Circular tunnels for which the ratio $\sigma_\theta / p_z \approx 2.0$ in roof (Hoek and Brown, 1980). The tangential stresses can be found from the method presented by Hoek and Brown (1980) as outlined in Appendix 9.
- The expressions above are combined into $\sigma_c / z = (2 \times 0.02)\sigma_c / \sigma_\theta$. It is probable that scale effect of compressive strength has been inluded in Fig. 6-17; therefore σ_c has been replaced by RMi, and the values for the ratio RMi $/\sigma_\theta$ in Table 6-8 have been found. This table is based on a limited amount of results and should, therefore, be updated when more data from practical experience in squeezing ground - especially in highly jointed ground - can be made available.

Bhawani Singh et al. (1992) developed another empirical criterion, based on the Q-system, which constitutes another possibility for evaluating the competency of rock masses. Incompetency resulting in squeezing may occur if the height above the excavation is

$$z > 350\ Q^{1/3} \qquad \text{eq. (6-5)}$$

where Q is the rock mass quality in the Q-system.

This expression has several limitations as it is restricted to deformable (ductile) rock masses. Neither the influence of tectonic or residual stresses, which in many parts of the world results in considerable horizontal stresses leading to stability problems, is included.

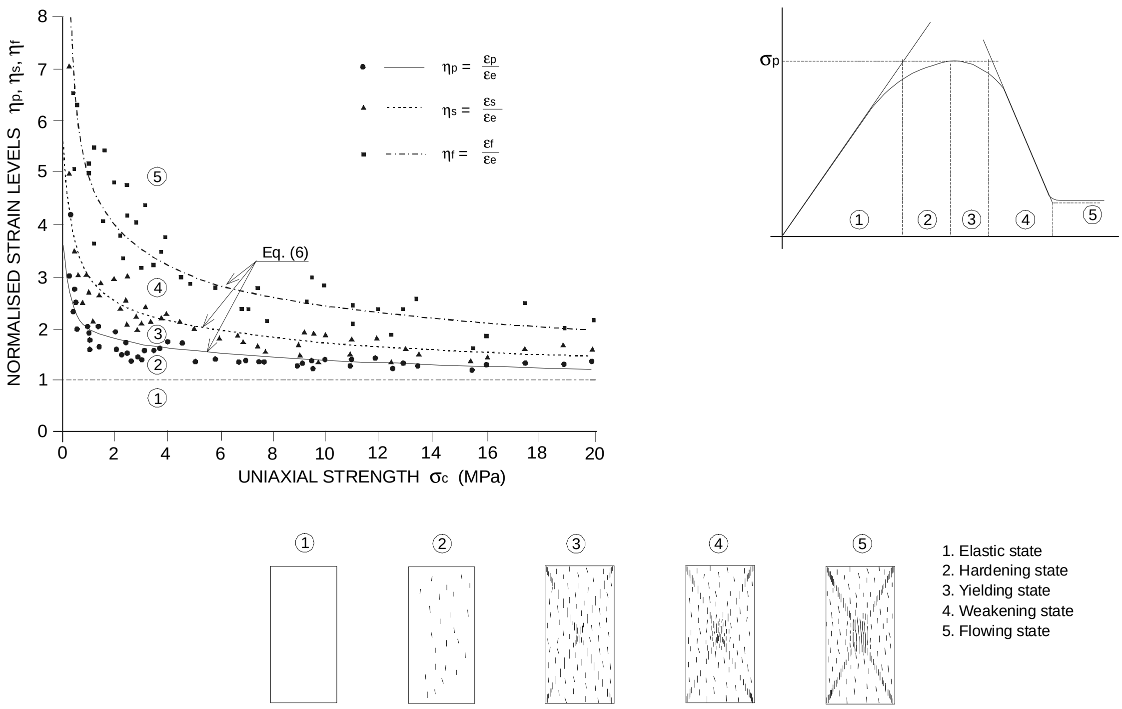
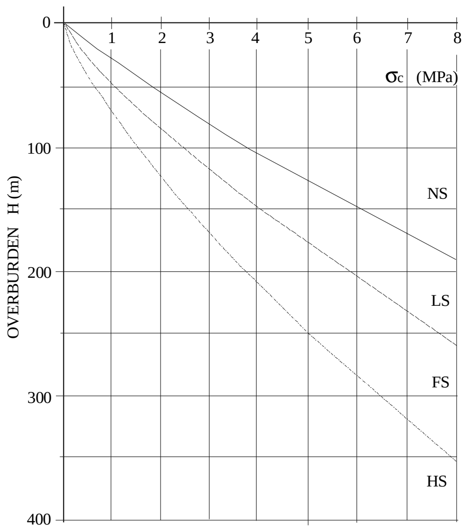


Fig. 6-16 Idealized (left) stress-strain curves with corresponding development of squeezing and plots (right) of normalized strain levels (from Aydan et al., 1993).



For straight lines between:
NS (no squeeze and light squeeze) $\sigma_c/H = 1/25$
LS and FS (fair squeeze) $\sigma_c/H = 1/35$
FS and HS (high squeeze) $\sigma_c/H = 1/50$

Fig. 6-17 A chart for estimating the possibility for squeezing (after Aydan et al., 1993)

According to Seeber et al. (1978) the rock support in squeezing ground may be as shown in Table 6-9. These data have been used in Fig. 6-18 where the rock support has been related to the competency factor.

TABLE 6-8 CLASSIFICATION OF SQUEEZING (based on Aydan et al., 1993)

Squeezing class competency range	**The tunnel behaviour according to Aydan et al. (1993)**
No squeezing $RMi/\sigma_\theta > 1$	The rock behaves elastically and the tunnel will be stable as the face effect ceases.
Light squeezing $RMi/\sigma_\theta = 0.7 - 1$	The rock exhibits a strain-hardening behaviour. As a result, the tunnel will be stable and the displacement cease.
Fair squeezing $RMi/\sigma_\theta = 0.5 - 0.7$	The rock exhibits a strain-softening behaviour, and the displacement will be larger. However, it will cease away from the face effect.
Heavy squeezing $RMi/\sigma_\theta = 0.5 - 0.35^{*)}$	The rock exhibits a strain-softening behaviour at much higher rate. Subsequently, displacement will be larger and will not tend to cease away from effect.
Very heavy squeezing $RMi/\sigma_\theta < 0.35^{*)}$	The rock flows resulting in very large displacements; the medium will collapse if not supported appropriately, and it will then be necessary to re-excavate the opening and install heavy support.

*) This value has been roughly estimated

TABLE 6-9 CONVERGENCE AND ROCK SUPPORT IN SQUEEZING GROUND (based on Seeber et al., 1978)

NATM ÖNORM B 2203 (1983)	**English term**	**Approx. convergence and rock support according to Seeber et al. (1978) for tunnel with diameter 12 m** Without support	With support installed		
		Convergence	Convergence	Support pressure	Possible rock support
Stark gebräch oder druckhaft	Squeezing or swelling	min. 2 · 5 cm = 10 cm - - - - - max. 2 ·30 cm = 60 cm	2 · 3 cm = 6 cm - - - - - 2 · 5 cm = 10 cm	0.2 MPa - - - - - 0.7 MPa	bolts[1] spaced 1.5 m - - - - -- bolts[1] spaced 1.5 m shotcrete 10 cm
Stark druckhaft	Heavy squeezing or swelling	min. 2 · 40 cm = 80 cm - - - - - max. > 2 m	2 · 10 cm = 20 cm - - - - - 2 · 20 cm = 40 cm	0.8 MPa - - - - - 1.5 MPa	bolts[1] spaced 1 m shotcrete 10 cm - - - - - - bolts[2] spaced 1 m shotcrete 20 cm

[1] bolt length 3 m   [2] bolt length 6 m

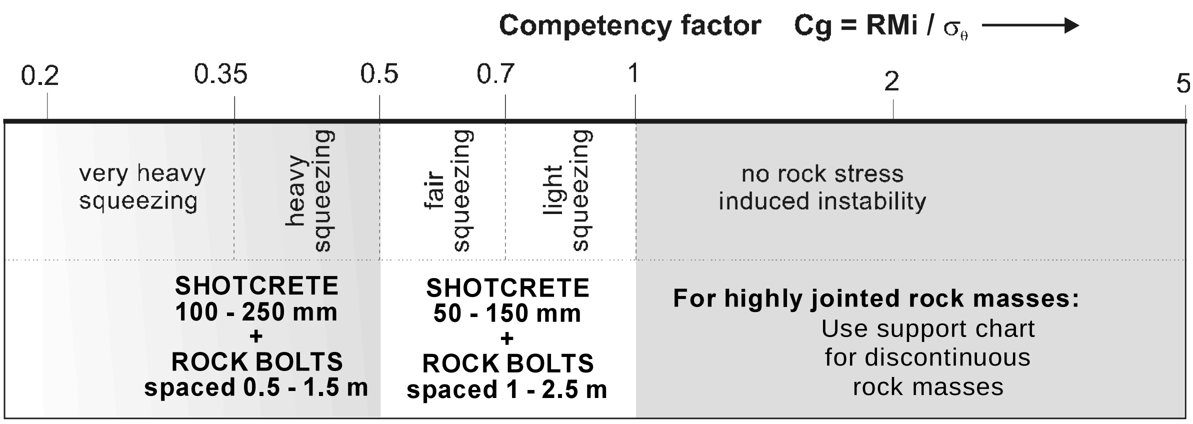


Fig. 6-18 Relationship between the competency factor, failure modes and support (12 m diameter tunnel) in highly jointed rock masses and in massive, 'ductile' rocks.

A. The use of analytical methods to determine rock support in squeezing ground

As it is considered theoretically that a plastic zone is formed, elastic-plastic solutions similar to the ground response interaction analysis may be applicable in calculating the behaviour. There is, however, a limit at which the problems of rock behaviour and support may be considered in plane strain in two dimensions (Muir Wood, 1979). The advance of a tunnel develops a complicated three-dimensional stress pattern in the vicinity of the face. Even for the simple case of a circular tunnel in ground considered as isotropic and elastic with a hydrostatic stress distribution only simplified analysis can be used. The designer has the difficult task of determining realistic values of the strength parameters ϕ and c of the ground (Deere et al., 1969). By applying the RMi, the values of m may be easier and better characterized. The actual analyses may involve the ground response curves as applied in the NATM support system (Seeber et al., 1978), or the Hoek-Brown criterion, refer to Chapter 8.

Also, for the rock stresses applied in the analysis there are uncertainties connected to their measured magnitudes and directions. It may be difficult to carry out reliable rock stress measurements in deep drill holes from the ground surface to the actual location before construction. Therefore, rough estimates of the stress level as described in Section 6.5 have often been applied, based on the weight of the overburden.

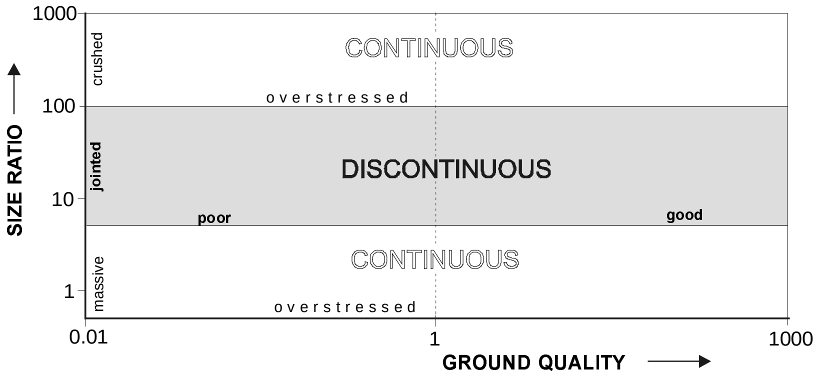
The stand-up time is a main feature during excavation in incompetent, continuous ground. The close timing of the excavation and the rock support carried out as initial support plays an important part in weak ground tunnelling as manifested in the NATM concept.

Another important feature in tunnelling is the influence on the rock load from the arching effect of the ground surrounding a tunnel. Terzaghi (1946) introduced the term *arch action* for this capacity of the rock located above the roof of a tunnel to transfer the major part of the total weight of the overburden onto the rock located on both sides of the tunnel. By allowing the material to yield and crush to some extent in such incompetent ground while the inward redistribution of stresses takes place, its potential strength can be mobilized. The high ground stresses close to the tunnel dissipate as the rock masses dilate or bulk (increases in volume). In this way only a reduced support is needed to contain the cracked rock surrounding the tunnel. Terzaghi (1946) mentions that because of this *arch action* in completely crushed but chemically intact rock and even in some sands, the rock load on the roof support does not exceed a small fraction of the weight of the ground located above the roof. The utilization of this effect is one of the main principles in the NATM.

There has not been time to work further on squeezing ground to develop a support chart from data on tunnel, stresses, and rock support for the squeezing ground. From case examples including characterization of the rock masses combined with analytical and modelling works, a similar chart as for discontinuous (jointed) rock may prove to be appropriate for this type of ground.

6.4.2 Stability and rock support in discontinuous (jointed) materials

"Paradoxically, the excavation of an underground opening in a highly stressed environment is likely to be less hazardous when the rock is jointed than when it is intact."
Nick Barton (1990)



The failures in this group of jointed rocks occur when wedges or blocks, limited by joints, fall or slide from the roof or sidewalls. They develop as local sliding, rotating, and loosening of blocks and may occur in excavations in jointed rocks at most depths. The properties of the intact rock are of relatively little importance as these failures, in general, do not involve development of fracture(s) through the rock (Hoek, 1981). The strength of the rock influences, however, often the wall strength of the joint and may in this way contribute to the stability.

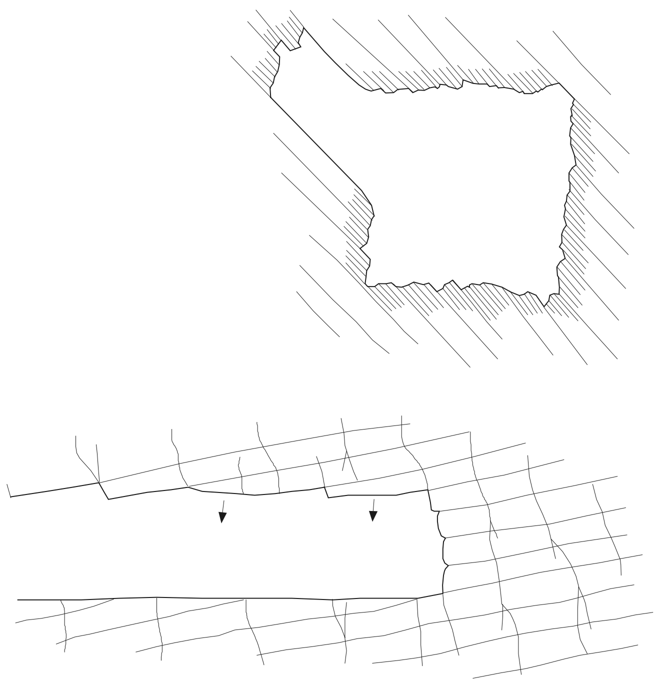


Fig. 6-19 The influence from discontinuities on block loosening and overbreak (from Stini, 1950). Upper figure: Overbreak caused by smooth foliation partings in a quartzphyllite. Lower figure: Layering joints and long cross joints cause instability in the roof.

The stability in jointed rock masses may be divided between instability of an individual block and cases in which failure involve two or more blocks. Two examples of overbreak from failure caused by joints are shown in Fig. 6-19.

The key block method may be used as analysis in this group as it applies knowledge of orientation and condition of significant, joints and weakness planes in the rock mass; refer to Goodman (1989). The principles in this analysis and the methods for data collection have been also been described by Hoek and Brown (1980).

As the condition, orientation, frequency and location of the joints in the rock mass relative to the tunnel are the main controlling factors, the stability can generally not be predicted by equations derived from theoretical considerations (Deere et al., 1969). A common solution is to apply charts or tables in which the amount and types of support are found from combination of several rock mass and excavation parameters. This principle has been applied among others in the Q and the RMR systems.

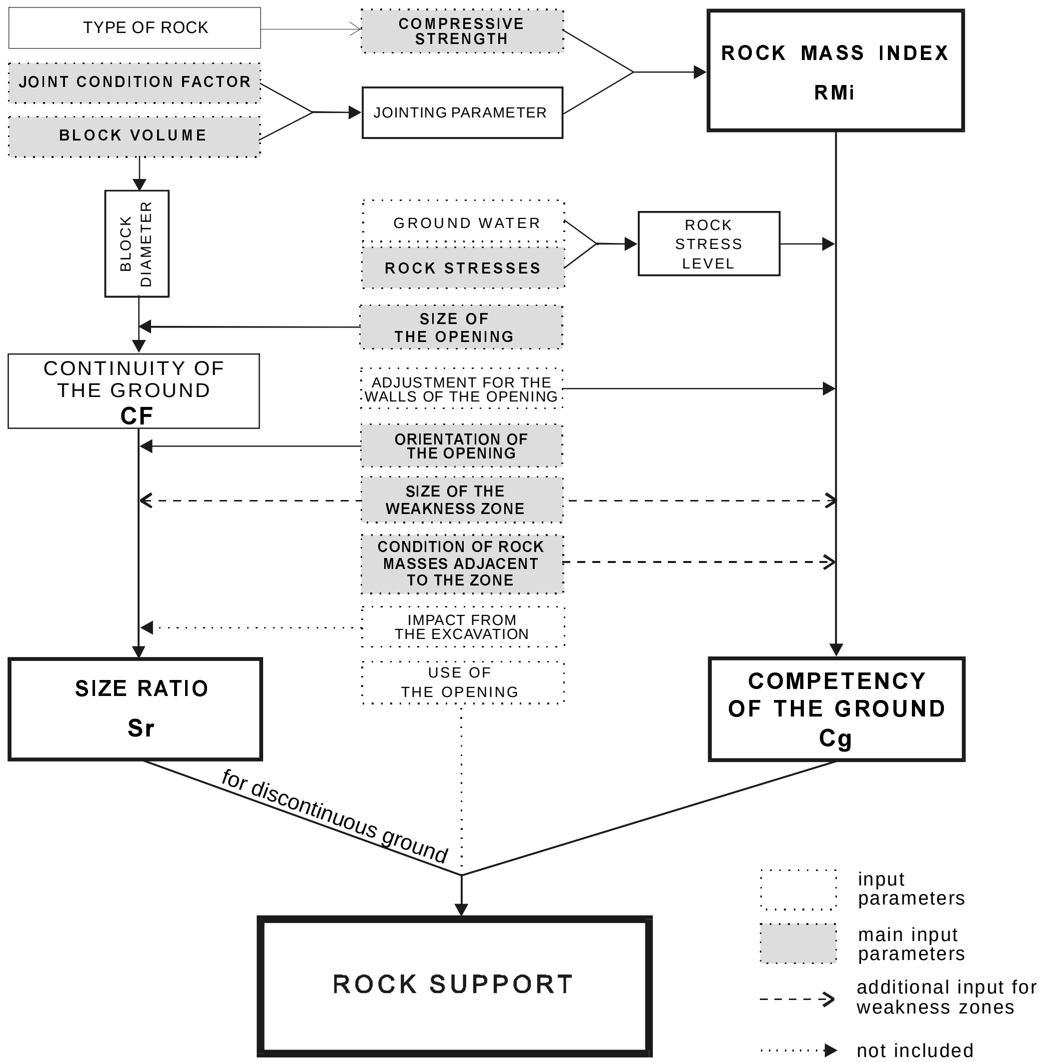


Fig. 6-20 The parameters involved in stability and rock support assessment in discontinuous ground.

As shown in Fig. 6-20 the following main parameters influencing stability have been selected to characterize discontinuous rock masses for design of rock support:

- a factor for the ground conditions, and
- the size features, which is related to the dimensions and orientation of the opening relative to the blocks and the joints.

6.4.2.1 The ground condition factor (Gc)

The ground conditions consist of the general *inherent* rock mass features of main influence on stability and the *external* stresses acting. The main features are:

- The block volume, Vb,	)
(representing the quantity of joints)	)
- The condition of joints, jC	) expressed in the rock
(smoothness, waviness, size, etc.)	) mass index (RMi)
- The strength of the joint surface	)
(compressive strength of rock)	)
- The stress level in the ground	

The combination of these parameters is indicated in Fig. 6-20.

The rock mass index (RMi) has been selected to represent the inherent rock mass properties as it contains all their main internal factors.

A. Effect of stress level in the ground

In addition to the inherent properties of the material the stability is influenced by the stresses acting across the joints in the rock mass surrounding the tunnel. A relatively high stress level will contribute to a 'tight structure' with increased shear strength along joints and, hence, increased stability. This has often been observed in deep tunnels. For the same reason a low stress level is unfavourable to stability. This effect is frequently seen in portals and tunnels near the surface where the low stress level often is 'responsible' for loosening and falls of blocks.

The Q-system uses the impact of stresses in jointed rock in its 'stress reduction factor' (SRF) as shown in Table 6-10. From this it is seen that there is a factor of 5 between the most and the least favourable SRF for jointed rock masses.

In a jointed rock mass containing variable amount of joints with different orientations it is not possible in a simple way to calculate and incorporate the stresses acting across the joints. Therefore, a general stress level factor (SL) similar to that in the Q-system has been chosen.

TABLE 6-10 CLASSIFICATION OF STRESS LEVEL (FOR ANY SHAPE OF OPENING) AND SRF VALUES (from Barton et al., 1974)

Low stress, near surface	$\sigma_c/\sigma_1 > 200$	SRF = 2.5
Medium stress	$\sigma_c/\sigma_1 = 200 - 10$	SRF = 1
High stress level, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	$\sigma_c/\sigma_1 = 10 - 5$	SRF = 0.5 - 2

The stress level referred to here is the *total* stresses. The influence of pore pressure or joint water pressure is generally difficult to incorporate in the stress level. Often, the joints around the tunnel will drain the ground water in the volumes nearest to the tunnel, hence the influence from ground water pressure on the effective stresses is limited. The total stresses have, therefore, been selected. The ratings of SL have roughly been chosen as $SL \approx 1/SRF$. In some cases, however, where unfavourable orientation of joints combined by high ground water pressure will tend to reduce the stability by extra loading on key blocks, the stress level factor should be reduced. The reduction of SL given in Table 6-11 for these cases are roughly assumed.

TABLE 6-11 THE RATINGS OF THE STRESS LEVEL FACTOR (SL)

Term	**Maximum stress** σ_1	**Approximate overburden** (for $k \approx 1$)	**Stress level factor (SL)**$^{*)}$	
				average
Very low stress level (in portals etc.)	< 0.25 MPa	< 10 m	0 - 0.25	0.1
Low stresses level	0.25 - 1 MPa	10 - 35 m	0.25 - 0.75	0.5
Moderate stress level	1 - 10 MPa	35 - 350 m	0.75 - 1.25	1.0
High stress level	> 10 MPa	> 350 m	1.25$^{**)}$ - 2.0	1.5$^{**)}$

$^{*)}$ In cases where ground water pressure is of importance for stability, it is suggested to:
- divide SL by 2.5 for moderate influence
- divide SL by 5 for significant influence

$^{**)}$ For stability of high walls a high stress level may be unfavourable. Possible rating SL = 0.5 - 0.75

There is an obvious greater stability of a vertical wall compared to a horizontal roof. Milne et al. (1992) have introduced a gravity adjustment factor to compensate for this where the wall is given a factor of 5 and horizontal backs 1. Similarly, Barton et al. (1975) has applied a wall/roof factor as an adjustment of the Q-value. This factor depends, however, on the quality of the ground. Its value is 5 for good quality (Q > 10); 2.5 for medium (Q = 0.1 - 10); and 1.0 for poor quality ground (Q < 0.1).

Based on Milne and Potvin (1992) the ground condition factor (Gc) is adjusted by a gravity adjustment factor

$$C = 5 - 4 \cos\beta \qquad \text{eq. (6-6)}$$

where β = angle (dip) of the surface from horizontal. (C = 1 for horizontal surfaces, C = 5 for vertical walls.)

Based on the considerations above the ground condition factor is thus expressed as

$$Gc = SL \times RMi \times C \qquad \text{eq. (6-7)}$$

B. Possible instability induced from high ground stresses.

The experience is, as mentioned earlier in this section, that rock bursting is less developed in jointed rock than in massive rock under the same stress level. At depths where the stresses developed around the excavation may exceed the strength of the rock, both stress induced and structurally controlled failures may occur simultaneously. According to Hoek (1981) one of these two forms, tends to dominate at a particular site where they both occur.

Terzaghi (1946) describes this type of stress controlled failures in jointed rock as *"If the rock masses around the tunnel is in a state of intense elastic deformation, the connections or interlocks between blocks such as A and B in Fig. 6-21 and their neighbours, may suddenly snap, whereupon the block is violently thrown into the tunnel. If such an incident occurs, it is necessary to provide the tunnel with the support prescribed for popping."*

Little information has, however, been found in the literature on this effect. Barton (1990) has experienced that, if jointing is present in highly stressed rock, extensional strain and shear strain can be accommodated more readily and are partially dissipated. The result is that stress problems under high stress levels are less in jointed than in massive rock. This has also been clearly shown in tunnels where destress blasting is carried out in the tunnel periphery with the purpose to develop additional cracking and in this way reducing the amount of rock bursting.

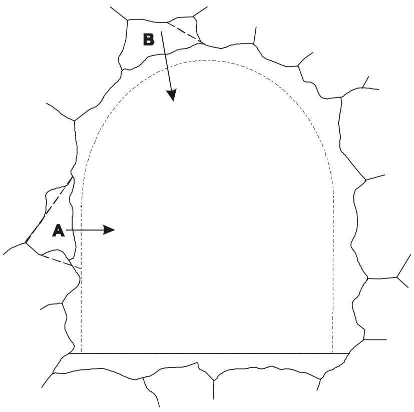


Fig. 6-21 Possible instability in jointed rock masses exposed to high rock stress level (from Terzaghi, 1946).

Under high stress level in moderately to slightly jointed rock masses cracks may develop in the blocks and cause reduced stability from the loosening of fragments. This phenomenon has been observed in the Thingbæk chalk mine in Denmark described in Appendix 7.

6.4.2.2 The size ratio

The size ratio is meant to represent the geometrical conditions at the actual location. It includes the dimension of the blocks and the underground opening and is expressed as:

$$Sr = (Dt/Db) \times (Co/Nj) \qquad \text{eq. (6-8)}$$

where Dt is the diameter (span or wall height) of the tunnel.

Db is the block diameter represented by the smallest dimension of the block, which often turns out to be the spacing of the main joint set. Often the *equivalent block diameter* is applied where joints do not delimit separate blocks (for instance where less than 3 joint sets occur). In these cases Db may be found from the following expression which involves the block volume (Vb) and the block shape factor (β) as shown in Appendix 3, Section 4: [6]

$$Db = (\beta_o/\beta)\ Vb^{-} = (27/\beta)\ \sqrt[3]{Vb} \qquad \text{eq. (6-8)}$$

N_j is a factor representing the number of joint sets as an adjustment to Db in eq. (24) where more or less than three joint sets are present. As described by Barton et al. (1974) the degree of freedom determined by the *number of joint sets* significantly contributes to stability. The value of N_j is found from the expression

$$Nj = 3/n_j \qquad \text{eq. (6-9)}$$

where nj = the number of joint sets (n_j = 1 for one set; nj = 1.5 for two sets plus random joints; nj = 2 for two sets, etc.)

---

[6] (β_0/β) has been chosen in eq. (6-8) as a simple expression to find the smallest block diameter. It is most appropriate for β < 150. For higher values of β a dominating joint set will normally be present for which the average joint spacing (S1) should be applied.

Co is an orientation factor representing the influence from the *orientation* of the joints on the block diameter encountered in the underground opening. The ratings of Co in Table 6-12 are based on Bieniawski (1984) and Milne et al. (1992). The strike and dip are measured relative to the tunnel axis. As the jointing is three-dimensional, the effect of joint orientation is often a matter of judgement, often the orientation of the main joint set is has the main influence and is applied to determine Co.

TABLE 6-12 THE ORIENTATION FACTOR FOR JOINTS AND ZONES. THE DIVISION IS BASED ON TABLE 6-3.

IN WALL		IN ROOF	TERM	**Rating of orientation factor Co**
for strike > 30^o	for strike <30^o	all strike values		
dip < 20^o	dip < 20^o	dip > 45^o	favourable	1
dip = 20 - 45^o	dip = 20 - 45^o	dip = 20 - 45^o	fair	1.5
dip > 45^o	-	dip < 20^o	unfavourable	2
-	dip > 45^o	-	very unfavourable	3

6.4.2.3 Rock support chart for discontinuous materials

The rock support chart shown in Fig. 6-23 is developed from case examples in Table 6-13 and from experience gained in numerous tunnels excavated in hard rock, mainly in Norway. To simplify and limit the size of the support diagram Vb = 10^{-6} m^3 (= 1 cm^3) has been chosen as the minimum block (or fragment) size. This means that where smaller particles than medium gravel occur, Vb = 1 cm^3 or block diameter Db = 0.01 m is used.

Roughly, for *'common' hard rock mass conditions*, i.e. SL = 1, 3 joint sets (nj = 3) and RMi = 40 $\sqrt[3]{Vb}$ (for σ_c = 160 MPa and jC = 1.75), the following simplified expressions can be applied:

The ground condition factor:

$$Gc = 40C\sqrt[3]{Vb} \qquad \text{eq. (6-10)}$$

(C = 1 for horizontal roofs and C = 5 for vertical walls)

The size ratio (for β= 40 and Co =1.5 (fair joint orientation))

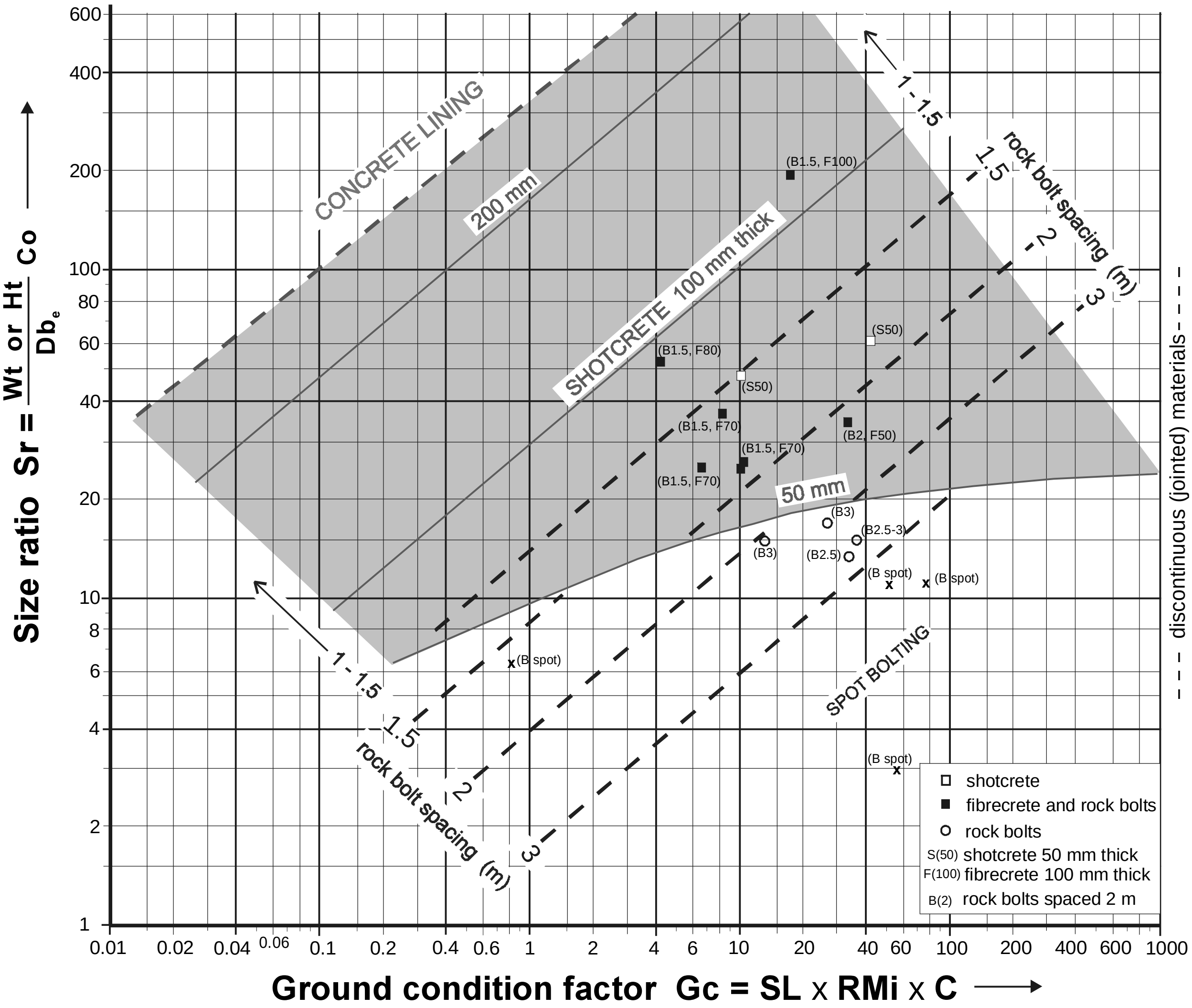
$$Sr = \frac{Wt}{\sqrt[3]{Vb}} \quad \text{or} \quad Sr = \frac{Ht}{\sqrt[3]{Vb}} \qquad \text{eq. (6-11)}$$

The various excavation techniques used may disturb and to some degree change the rock mass conditions. This may increase the tendency of blocks to loosen and fall out of the tunnel walls or roof. Especially, excavation by blasting tends to develop new cracks around the opening. This will cause that the size of the original blocks will be reduced, which will cause an increase of the size ratio (Sr) and a reduction of the ground condition factor (Gc). Knowing or estimating the change in block size from excavation it is, therefore, easy to calculate the adjusted values for (Sr) and (Gc) and thus include the impact from excavation in the assessments of rock support.

TABLE 6-13 SUMMARY OF GROUND CHARACTERISTICS AND INSTALLED ROOF SUPPORT IN DISCONTINUOUS ROCK MASSES FROM DESCRIPTIONS IN APPENDIX 7. THE VALUES FOR Gc AND Sr HAVE BEEN PLOTTED IN FIG. 6-23.

Project and location		**Ground characteristics**			**Applied rock support**
		Jointing parameter & Rock mass index **JP & RMi**	Ground condition factor **Gc**	Size ratio **Sr**	B( ) = rock bolt (spacing) F( ) = fibrecrete (thickness) S( ) = shotcrete (thickness)
Gjövik Olympic mountain hall		0.21 & 17.2	17.2	189	B(2 m) length 5 m B(5 m) length 12 m F(100 mm)
Granfoss road tunnel,	chainage 400	0.21 & 8.4	6.7	23.8	B(1.5 m) F(70 mm)
	chainage 1875	0.13 & 8.1	8.1	37.8	B(1.5 m) F(70 mm)
	chainage 1320	0.11 & 4.3	4.3	52.4	B(1.5 m) F(80 mm)
	chainage 1420	0.19 & 11.6	11.6	26.7	B(1.5 m) F(70 mm)
	chainage 1700	0.19 & 11.5	11.5	25.2	B(1.5 m) F(70 mm)
Haukrei headrace tunnel,	chainage 200	0.46 & 54.8	54.8	3.0	no support
Horga headrace tunnel,	chainage 470	0,13 & 13.5	13.5	15	B(3 m)
	chainage 1485	0.27 & 27	27	16.8	B(3 m)
Tromsö road tunnel,		0.4 & 39.5	33	13.5	B(2.5 m)
	roundabout	0.4 & 39.5	33	27	B(2 m) F(50 mm)
Nappstraumen road tunnel		0.35 & 42.4	36.4	15	B(2.5 - 3 m)
Njunis, access tunnel,	chainage 6250	0.24 & 48.5	72.7	12	spot bolting
Sumbiar road tunnel,	chainage 650	0.24 & 49	49	12.5	spot bolting
	chainage 1315	0.21 & 41.9	42	61	S(50 mm)
	chainage 2100	0.05 & 10.7	10.7	48.4	S(50 mm)
Thingbæk chalk mine		0.84 & 0.84	0.8	6.7	B(spot)

The support chart in Fig. 6-23 covers the rock support of walls as well as roof in the underground opening. Examples of calculating the RMi value and the input factors Gc and Sr to the support chart are shown in Appendix 7.



C is a factor for other surfaces of the excavation than horizontal roof: C = 5 - 4 cosβ (β = angle of the surface from horizontal)

Fig. 6-23 Rock support chart for discontinuous (jointed) rock masses.

6.4.3 Stability and rock support of faults and weakness zones

Weakness zones consist of rock masses with properties significantly poorer than those of the surrounding rock masses. Included in the term weakness zones are faults, zones or bands of weak rocks in strong rocks, etc. as described in Appendix 2. Weakness zones occur both geometrically and structurally as special features in the ground. The following features in the zones are of main importance for stability:

1. The *geometry and dimensions* of the zone.
   The instability and problems in weakness zones will generally increase with the *width* of the zone. However, this feature should always be assessed in relation to the attitude of the zone and to the frequency, orientation, and character of adjacent joint sets, the existence of adjacent seams or faults (if any), and the quality of the adjacent rock mass. Brekke and Howard (1972) note that several severe slides in tunnels have occurred where each individual seam or fault has been of small width, but where the interplay between them has led to failure.

The orientation of the zone relative to the tunnel can have a considerable influence on the stability of the opening. As for joints, the problems in general increase as the strike becomes more parallel to the opening and when the zone is low-dipping. This comes also from the fact that for such orientations the zone affects the tunnel over a longer distance.

2. The *reduced stresses* in the zone compared to the overall ground stresses.
   An important effect in weakness zones is the fact that the stresses in and near the zone will be other than normal. Selmer-Olsen (1988) has experienced that faults and weakness zones may cause large local variations in the rock stresses. Although the overall stresses in an area may indicate that a weakness zone should be overstressed and behave as incompetent (squeezing) ground when encountered in an excavation, this will seldom be the case. The reason is the greater deformability in the zone and transfer of stresses onto the adjacent rock masses. Failures in weakness zones will, therefore, seldom be squeezing, but gravity induced. Very wide zones, however, are expected to have stresses and behaviour equal to those of the surrounding ground.

   Also for this case, the quality of the rock masses surrounding the weakness zone may contribute to the stability of the zone.

3. The *arching effect* from the surrounding rock masses
   Terzaghi (1946) explained that the rock load on the roof support, even in sand and in completely crushed rock, is only a small fraction of the weight of rock located above the tunnel because of the *arch action* or silo effect. Where the width of the zone is smaller than the tunnel diameter, additional arch action from the stronger, adjacent rock masses leads to reduced the load exerted on the rock support compared to that of a rock mass volume with the same composition.

4. Possible occurrence of *swelling, sloughing, or permeable* materials in the zone.
   These features are further discussed in Section 6.4.3.3.

The composition of weakness zones and faults can be characterized by RMi and/or by its parameters. Many weakness zones occur as continuous materials when compared to the the tunnel size or zone, and may be considered as such in the calculations. Based on the comments above a similar system as has been presented for discontinuous (jointed) rock masses in Section 6.4.2, has been found to cover most types of zones. It applies a ground condition factor and a size ratio adjusted for features of the zone as shown in Fig. 6-24.

As mentioned in the introduction to this section, the interplay between the properties of the zone and the properties of the adjacent rock masses plays an important role, especially for small and medium sized zones. The inherent features of both can be characterized by their respective RMi values. The RMi_z for the zone is adjusted for the size (Tz) of the zone and for the quality of adjacent rock masses expressed by their jointing parameter (RMi_a).

6.4.3.1 The ground condition factor for zones

Löset (1990) has developed an expression to characterize weakness zones for application in the Q-system. The expression includes the size of the zone and combines the quality of the zone with the quality of the rock masses on both sides of the zone. The 'combined' quality is

$$\log Q_m = (Tz \times \log Q_z + \log Q_a)/(Tz + 1) \qquad \text{eq. (6-14)}$$

where Tz = the width of the zone in metres, Q_z = the quality of the zone, and Q_a = the quality of the adjacent rock masses.

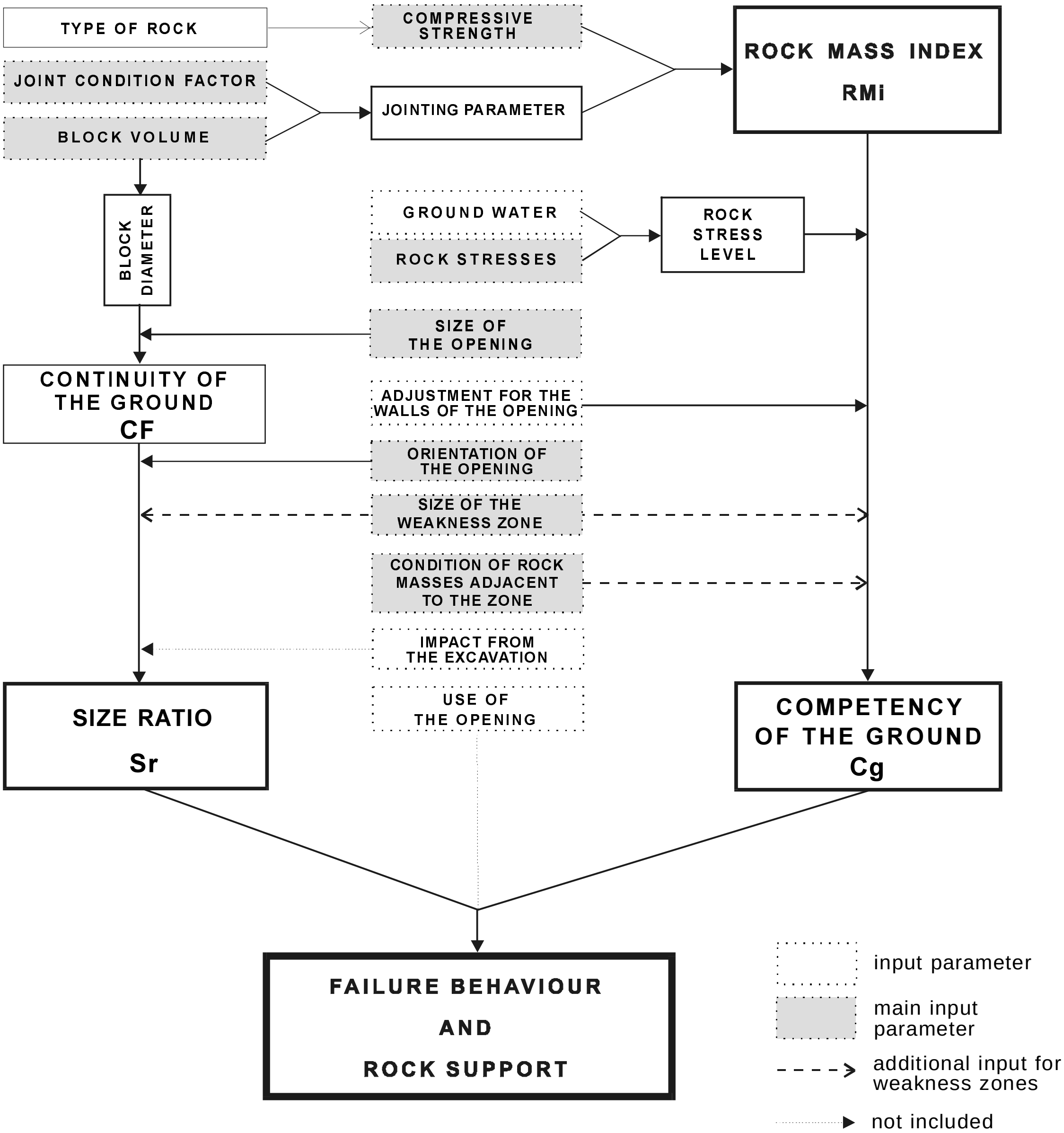


Fig. 6-24 The parameters and their combination for assessing the stability and rock support of weakness zones.

The same principles can be applied for rock masses characterized by the RMi

$$\log RMi_m = \frac{Tz \times \log RMi_z + \log RMi_a}{Tz + 1} \qquad \text{eq. (6-15)}$$

or

$$RMi_m = 10^{\frac{Tz \times \log RMi_z + \log RMi_a}{Tz+1}} \qquad \text{eq. (6-16)}$$

For Tz = 0 (no weakness zone) eq. (6-16) is $RMi_m = RMi_a$

As an alternative to the complicated eq. (6-16) a simplified expression has been developed

$$RMi_m = (10Tz^2 \times RMi_z + RMi_a)/(10Tz^2 + 1) \qquad \text{eq. (6-17)}$$

The correlation between the two expressions for RMi_m is shown in Fig. 6-25.

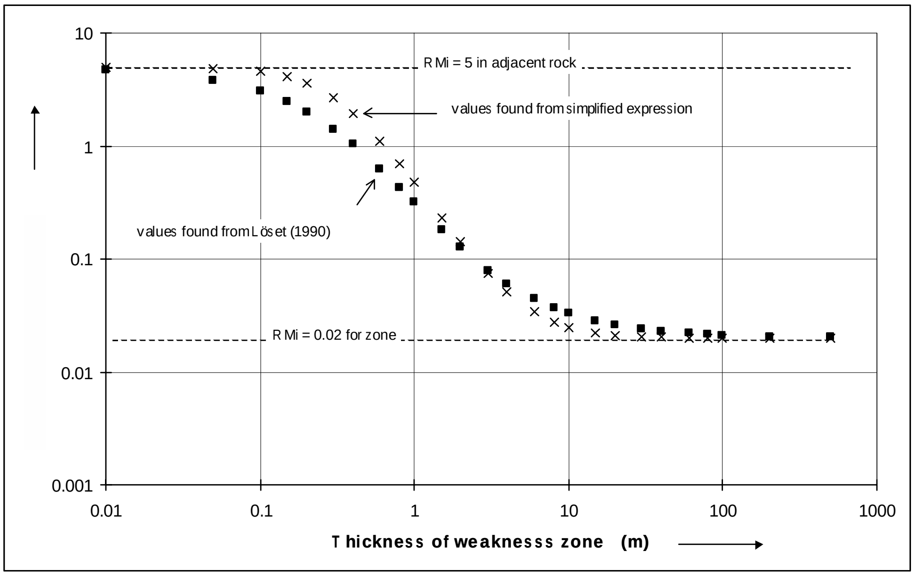


Fig. 6-25 The variation of RMi_m with thickness of weakness zone (Tz) using the expression based on Löset (1990) (eq. (6-11)) and the simplified expression (eq. (6-13)). Input values: $RMi_z = 0.02$ for the zone, and RMi_a = 5 for the adjacent rock masses.

For larger zones the effect of arching is limited; the ground condition for such zones should therefore be that of the zone (= RMi_z). From eqs. (6-14) to (6-16) and Fig. 6-25 is found that for 20 m thick zones $RMi_m \approx RMi_a$. The stress reduction in zones may probably take place also in larger zones than this.

An expression for the ground condition factor has been chosen for weakness zones similar to that for discontinuous (jointed) rock masses.

$$Gc_z = SL \times RMi_m \times C \qquad \text{eq. (6-18)}$$

It may be discussed if the stress level factor (SL) has significant influence on stability in weakness zones since they, as mentioned in the beginning of this section, often exhibit reduced stresses compared to those in the adjacent rock masses. However, stresses influence on the shear strength along joints and hence the stability, especially in clay-free crushed zones. Another argument for including SL is the benefit of simplicity to apply similar expressions for Gc and Gc_z .

6.4.3.2 The size ratio for zones

It is earlier mentioned that weakness zones show increased arching effect compared to the overall rock mass when they have thickness less than approximately the diameter (span) of the tunnel. For such zones the size ratio (Dt/Db)(Co/Nj) is adjusted for the zone ratio Tz/Dt to form the size ratio for zones [7]

[7] This ratio is applied provided $Tz/Db_{zone} < Dt/Db_{adjacent}$

$$Sr_z = \frac{\text{diameter of tunnel}}{\text{block size (in zone)}} \times \frac{\text{size (width) of zone}}{\text{diameter of tunnel}} \times (\text{orientation of zone}) = \frac{Tz}{Db_{zone}} \times \frac{Co}{Nj} \qquad \text{eq. (6-19)}$$

here, Tz = thickness of zones smaller than the diameter (span or height) of the tunnel;
Co = factor for the orientation of the zone as shown in Table 6-12
Dt = the diameter (span or wall height) of the tunnel.
Nj = the rating for the number of joint sets in the zone.

For zones thicker than the tunnel diameter the size ratio described for discontinuous (jointed) rock masses (eq. (6-9)) should be applied ($Sr_z = Sr = Dt/Db \times Co/Nj$).

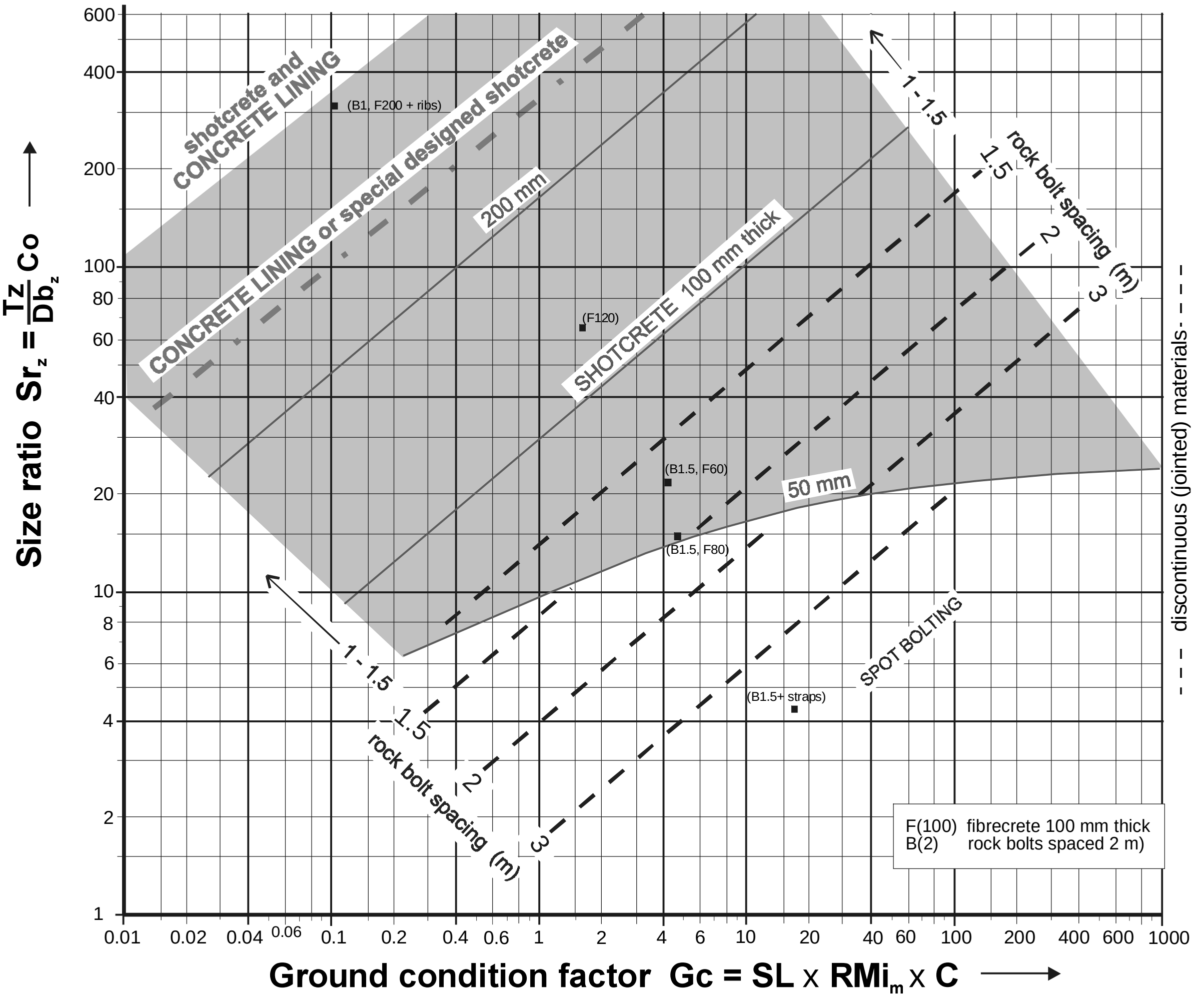
Similarly, as for jointed rock masses, a minimum block size Vb = 1 cm^3 or block diameter Db = 0.01 m has been chosen. The support chart for weakness zones is shown in Fig. 6-26. For the few data collected for this type of ground (Table 6-14) there is relatively good agreement between the ground characteristics and the applied rock support.

TABLE 6-14 SUMMARY OF GROUND CHARACTERISTICS AND APPLIED ROOF SUPPORT IN WEAKNESS ZONES (from descriptions in Appendix 7).

Project and location	**Ground characteristics**			**Applied rock support**
	Jointing parameter & Rock mass index **JP & RMi**	Ground condition factor **Gc**	Size ratio **Sr**	B( ) = rock bolt (spacing) F( ) = fibrecrete (thickness)
Haukrei headrace tunnel, chainage 110	0.04 & 3.7	4.7 (3.7)	14.4 (21.5)	B(1.5 m) F(80 mm)
Vinstra headrace tunnel	0.01 & 0.12	0.1 (0.09)	311 (311)	B(1 m) F(200 mm) + ribs
Horga headrace tunnel, chainage 810	0.008 & 0.75	1.6 (0.75)	67.8 (67.8)	F(120 mm)
Njunis acces tunnel, chainage 6300	0.026 & 5.3	4.2 (2.7)	22.8 (34.2)	B(1.5 m) F(60 mm)
Sumbiar road tunnel, chainage 600	0.05 & 10.6	18.1 (10.6)	4.2 (42)	B(1.5 m) straps + wire mesh
The numbers in brackets are corresponding values applying expressions for discontinuous (jointed) ground				

6.4.3.3 Problems related to special features in weakness zones

Faults and weakness zones have been further described in Appendix 2 and in Section 6.2.1 of this chapter. Many faults and weakness zones contain materials quite different from the surrounding rock. Various geologic processes may have caused alteration of the materials in the zone into clays, often with swelling properties. The special properties of swelling clays having very low friction and loss of strength in addition to heavy loads on support structures from swelling, can strongly influence and often overshadow other properties of the zone. Zones showing moderate and low swelling properties may behave similarly to moderate and low squeezing ground. In such cases the rock support may also be similar. The long-time effect of swelling, dissolving, and outwash may easily be underestimated during the construction period, and the permanent rock support recommendations taken to ensure long time stability may, therefore, prove inadequate.



$RMi_m = (10Tz^2 \times RMi_z + RMi_a) / (10Tz^2 + 1)$

C is a factor for other surfaces of the excavation than horizontal roof: C = 5 - 4 cosβ
(β = angle of the surface from horizontal)

Sr = Dt x Co / Db is applied where $Sr_z > Sr$ and Tz > Dt (i.e. Wt or Ht)

Fig. 6-26 Rock support chart for moderate and small weakness zones, excluding zones with high swelling properties or high permeability. The diagram is similar to the chart in Fig. 6-23.

High water inflow in underground openings encountered when excavating from the weak central part of impervious gouge is, as mentioned earlier, one of the most adverse conditions associated with faults. Such ground conditions can seldom be characterized and assessed by a general system.

With the many varieties in structure and composition of faults and weakness zones (see Appendix 2) it will in many cases be relevant to carry out observations, tests and calculations adapted to each individual zone, and treating each of them as a special case in the engineering design process.

Brekke and Howard (1972) wrote that it is not uncommon that the construction problems associated with faults or seams have been described as "unexpected" while the fact has been that one knew of their existence, but their behaviour was incorrectly assessed prior to or during construction. Inadequate characterization may be a possible explanation for this.

6.4.4 Comments to the RMi method for assessing rock support

The main features and parameters influencing stability of underground openings were discussed in Section 6.3. Based on this the parameters and features of importance for stability have been selected. The two main groups of ground applied in the methods are:

- *continuous ground*, which can be either massive rock or heavily jointed (particulate) rock masses; and
- *discontinuous ground*, composed of jointed rock masses.

6.4.4.1 On the input parameters applied

The behaviour of the materials in the two groups is completely different. Therefore, the two approaches to assess the rock support are different. Common for both groups is that the composition and inherent properties of the structural material (i.e. rock mass) can be characterized by the rock mass index, RMi. The influence from stresses is, however, different. For continuous ground the tangential stresses (σ_θ) set up in the ground surrounding the opening are applied, while for discontinuous ground a stress level factor has been selected. The magnitude of the stresses in the ground can be estimated by applying the method outlined in Appendix 8.

The influence/impact from ground water has not yet been outlined. In *continuous* ground it can be included in the effective stresses applied to calculate the tangential stresses set up in the rock masses surrounding the underground opening. In *discontinuous* ground the direct effect of water is often minor, hence this feature has not been selected. It is, however, possible to adjust the stress level factor where water pressure has a marked influence on stability.

Various methods to collect and determine the values of the parameters applied in the RMi have been described in Appendix 3. Among these, the block volume (Vb) is the most important, as it is also included in the continuity factor. Great care should be taken when this parameter is determined.

As described in Chapter 5 there is, however, often a problem in characterizing the variations in rock mass composition. The block size varies within wide limits and the calculations must often be based on a variation range. Where less than three joint sets occur, it is a general problem in measuring the block volume. Methods of assessing the equivalent volume have been shown in Chapter 5 based on the tools presented in Appendix 3. As shown in the latter the (equivalent) block volume can be found from the volumetric joint count and a block shape factor which is defined from the ratio between joint spacings. Common values for β are given in Table A3-27a. The block shape can also be estimated from the longest and shortest dimension of the block using eq. (A3-42)

$$\beta = 27+7(a3/a1 - 1)$$

The compressive strength (σ_c) of the rock can, for support assessments of discontinuous (jointed) rock masses, often be found with sufficient accuracy from simple field tests (Schmidt hammer, simple hammer test) or it is in many cases sufficient to estimate σ_c from the name of the rock using for example Table A3-8.

6.4.4.2 The support charts

The support charts in Figs. 6-11, 6-23 and 6-26 cover most types of rock masses except squeezing. They are mainly based on Scandinavian practice where shotcrete (wet method, often fibre reinforced) plays an important part. The charts have been worked out from the author's own experience in addition to the 24 cases presented in Appendix 7 from visits to Norwegian and Danish tunnels. The compressive strength of the rocks varies from 2 MPa to 200 MPa and the jointing intensity from crushed to massive. In squeezing ground work remains to develop more adequate support charts. Also for this group of ground the application of RMi in the stability and support calculations seems very promising.

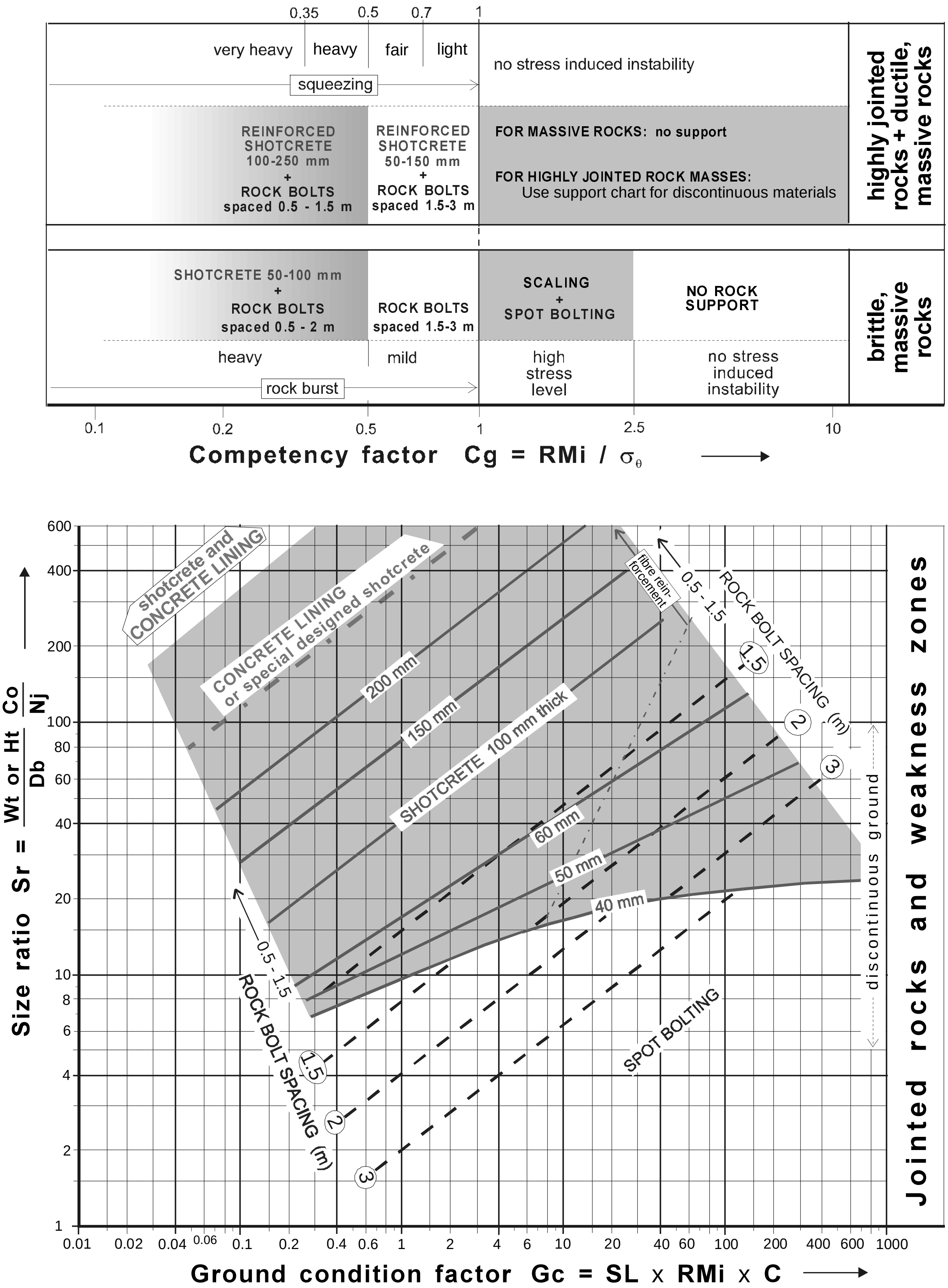
All support charts presented in the foregoing have been combined in Fig. 6-27, which covers most types of rock masses for estimating the types and amount of rock support. It is based on the condition that loosening and falls, which may involve blocks or large fragments, should be avoided. This also includes appropriate timing of rock support and excavation as is discussed later in this section.

In this connection it should be pointed out that, as the loosening or failures in jointed rock is mainly geometrically related, i.e. determined by the size of the blocks and the orientation and location of each joint, it is impossible to develop a precise support chart. Generally, support charts can only give the average amount of rock support. They can, therefore, be considered as an expression for the 'statistical average' of appropriate rock support. Further, a support chart can only give the amount and methods for support based on the support regulations and experience in the region. In other regions where other methods and applications have been developed, other support charts can be worked out based on the current practice.

The required level stability and rock support is determined from the utility of the underground opening. The Q-system applies the ESR (excavation support ratio) as an adjustment of the span to include this feature. From the current practice in underground excavations it is, however, difficult to include the different requirements for stability and rock support in a multiplication factor. For example, the roof in underground power houses will probably never be left unsupported even for a Q-value higher than 100; the same practice which is applied in traffic tunnels in Central Europe, seems to be gradually more common also in Norwegian road tunnels. Also, in large underground storage caverns in rock the roof is generally shotcreted before benching, because falls of even small fragments may be harmful in high caverns. Caused by this, a chart is preferably worked out for each main category of excavation.

6.4.4.3 What is new in the RMi support method compared to existing methods?

The support method developed differs from the existing support classification systems. While these directly combine all the selected parameters to arrive at a quality or rating for the ground conditions, the RMi method applies an index to characterize the construction material, i.e. the inherent rock mass and its properties. This index is applied as input in the ground conditions. The splitting up in the RMi support method into continuous and discontinuous rock masses and the introduction of the size ratio (tunnel size/block size) are new features in this method compared to existing methods.



For weakness zones:

$Sr_z = Tz \times Co_z / Db_z$ for zones where $Sr_z < Sr$ and $Tz < Wt$ or $Tz < Ht$ else $Sr_z = Sr$

$Gc_z = SL \times RMi_m \times C$ where $RMi_m = (10\,Tz^2 \times RMi_z + RMi_a) / (10\,Tz^2 + 1)$

Fig. 6-27 Combination of the support chart in Figs. 6-11, 6-18, 6-23 and 6-26.

The application of the RMi in rock support involves a systemized collection and application of input data. RMi makes also use of a clearer definition of the types of ground covered. It probably includes a wider range of ground than the two existing main support classification systems; the RMR and the Q-system. A comparison between the parameters selected in the different methods is given in Chapter 9.

Using the RMi in assessment of rock support may seem complicated at first glance. Possible beginner problems using the support chart should be relatively quickly overcome. Descriptions and collection of input data require, however, involvement of experienced engineering geologists, as is the case for most rock engineering projects.

The structure of RMi and its use in rock support engineering allows for accurate calculations where high quality data are available. It is, however, also possible in this method to apply simplified expressions for the ground conditions and size ratio when only rough sup-port estimates are required. Thus, for *'common' hard rock mass conditions,* i.e. SL = 1 and RMi = 40 $\sqrt[3]{Vb}$ (for σ_c = 150 MPa and jC = 1.75), the simplified expressions in eqs. (6-10) to (6-13) can be applied. As they only require input from the block volume, alternatively the spacing of the dominating joint set, the support estimates can quickly be carried out.

6.4.4.4 On the timing of rock support installation

It is common to distinguish between the primary or initial rock support installed to ensure stability <u>during</u> construction, and the <u>final</u> or permanent rock support usually added to ensure stability for the lifetime of the structure. The time expired between the two stages can, however, vary considerably as the latter often is performed after completion of the excavation works. In the following some comments are given for installation of the initial support.

A. In low stability ground

The great influence of time in tunnel construction was first clearly formulated by Lauffer (1958). Since then several papers have been published to further develop the stand-up time concept of Lauffer. The best known are the works by Bieniawski (1974, 1984, 1989) where the stand-up time is related to rock mass quality in the RMR system.

Also Fairhurst (1988) writes that time is a variable of potentially major significance in tunnel excavation. Delay of the installation of a support system can result in increased instability that can lead to collapse of the excavation. In rocks with very short stand-up time at the face it is always a problem to design a support system because of:

- the variability and uncertainty of the structural properties (strength and deformability) of the rock mass being encountered; and
- the uncertainties regarding stresses and loads.

There is neither any time nor accessibility to carry out necessary observations to make the calculations that are needed to analyse the stability problem. The decisions have often to be made quickly based on experience and available equipment.

In squeezing ground it is essential that a confining rock support be applied in proper time. This can increase the stability of the rock behind the excavation surface very effectively so that the rock remains in position to create an arching effect of the tunnel system (Müller, 1982). In NATM, which originally was developed for squeezing ground, timing of the rock support installation is one of the main features. Müller (1982) is of the opinion that *timing* is a factor in tunnelling, that can *" hardly be computed or even assessed by a rock mechanics specialist if he is not provided with deep geological knowledge or if he does not intimately collaborate with an engineering geologist."* Experience of the people involved may be the most important contribution in such situations.

According to Müller (1982) there are two main possibilities to solve the problem of unstable conditions shortly after blasting:

- One is to reduce the rounds - a measure which is very effective.
- The other is to divide the excavation face from fullface to heading and benching, or by dividing even the heading section in two or three parts.

Also the method(s) of support and the skill of the contractor determine how quickly the support can be installed after excavation.

B. In hard rock regimes

From studies of several tunnels during construction in Scandinavian hard rocks Cecil (1971) has worked out the following time-stability-support classification:

Type 1. Stable at blasting:
- no anticipated falls, no support;
- minor rock falls or overbreak at blasting, support not considered necessary for prevention of loosening;
- support in anticipation of loosening;
- unsupported, gradual deterioration and subsequent support.

Type 2. Falls at blasting:
- support in anticipation of progressive loosening.
- no support immediately after blasting, progressive loosening, support applied to prevent further loosening.
- support shortly after blasting to prevent or stop progressive loosening.

Type 3. Support installation shortly after blasting:
- failure of support, thereafter, additional support.

From these types it is the experience that:

Type 1 requires mainly limited amount of immediate support.
Type 2 requires immediate support to be strengthened by the permanent support.
Type 3 occurs in ground with low stand-up time where quick execution of support and reduced length of round often are necessary. Later, this support must be strengthened by permanent support.

The installation of support is often decided by the mining crew and is to a great extent determined from similar behaviouristic observations as shown here. The experience is further that initial rock support is often installed without any documentation of the ground condition, either because of the limited access to the face and the short time available, or because it is selected by the tunnel crew. Where the surface in the tunnel is covered by shotcrete concrete lining or other materials before a full description of the rock mass conditions has been made, it is not possible to determine the appropriate permanent rock support.

The support charts are valid only where the rock support is applied at the right time.