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CHARACTERIZING ROCK BURST AND SQUEEZING BY THE ROCK MASS INDEX

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ABSTRACT

Both rock burst and squeezing occur as instability caused by overstressing of continuous rock masses. Therefore, the Rock Mass index (RMi) characterizing the strength of rock masses can be applied directly in stability analyses. The *competency factor* expressed as the ratio between rock mass strength and the tangential stress (σ_θ) around the opening (Cg = RMi/σ_θ) is applied to indicate whether the ground is overstressed or not. σ_θ may be found using the method described by Hoek and Brown (1980).

Failures known as *spalling, popping* or *rock burst* are caused by overstressing of brittle, massive rocks often at depths in excess of 1,000 m below surface. These failures can also be induced at shallower depth where high horizontal stresses or strongly anisotropic stresses are acting. Based on published data a characterization has been worked out to determine the mode of instability and appropriate support.

Squeezing can occur both in massive (weak and deformable) rocks and in highly jointed rock masses as a result of overstressing. It is characterized by yielding under the redistributed state of stress during and after excavation. The squeezing can be very large; deformations as much as 17% of the tunnel diameter have been reported in India. Based on published data a numerical characterization has been developed for 'ductile', massive rocks, and possibly also for highly jointed rock masses.

1. INTRODUCTION

Rock burst and squeezing are two main modes of underground instability caused by overstressing of the ground. Both modes are generally related to continuous ground. The volume required for a 'sample' of a rock mass to be considered continuous is a matter of judgement. It depends on the size and range of blocks making up the 'sample' volume. This matter has been discussed by several authors:

- John (1969) suggests that a sample of about 10 times the average (linear) size of the single units of discontinuum may be considered a uniform continuum. It is clear that this will depend to a great extent on the uniformity of the unit sizes in the material or the uniformity of the spacings of the discontinuities. For a unit of 1 m^3 the size of such sample would be 10^3 m^3 and contain 1000 blocks.
- Another approximate assumption is based on the experience from large sample testing at the University of Karlsruhe, Germany, where a volume containing at least 5·5·5 = 125 blocks is considered continuous (Mutschler, 1993).
- Deere et al. (1969) have tied the 'sample' size to the tunnel size.

In this paper, the division into continuous and discontinuous materials is based on Deere et al. (1969) who used the ratio tunnel size/block size to characterize the continuity of the ground. They found that a 'sample' should be considered discontinuous *"when the ratio of fracture spacing to a tunnel diameter is between the approximate limits of 1/5 and 1/100. For a range outside these limits, the rock may be considered continuous, though possibly anisotropic."* Thus continuous rock masses involves two categories:

1. Slightly jointed (massive) rock with continuity factor
   CF = tunnel size/block size < approx. 5.
2. Highly jointed and crushed rocks, CF > approx. 100.

2. THE COMPETENCY FACTOR

Whether overstressing of the ground takes place is determined by the ratio between the stresses set up in the ground surrounding the opening and the strength of the ground (i.e. rock masses). As the rock mass index (RMi) is valid in continuous ground, and expresses the (relative) strength of the rock mass (Palmström, 1995), it can be used in assessing the *competency factor* given as $Cg = RMi/\sigma_\theta$ eq. (1)

σ_θ is the tangential stress around the underground opening. It can be found from the vertical rock stress, the ground water pressure, and the shape of the opening as outlined by Hoek and Brown (1980). The term competency factor 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 tunnels in Japan.

The rock mass index is given as $RMi = \sigma_c \cdot JP$ where JP, the jointing parameter, is a measure for the intensity of jointing (given as block size) and the joint characteristics (Palmström, 1995). In massive rock where the jointing parameters JP = 1, the rock mass index is $RMi = f_\sigma \cdot \sigma_c$ and $Cg = RMi/\sigma_\theta = f_\sigma \cdot \sigma_c / \sigma_\theta$. eq. (2)

f_σ is the scale effect for the compressive strength given as $f_\sigma = (50/d)^{0.2}$ (d is the block diameter measured in mm).

In highly jointed and crushed rock masses $Cg = \sigma_c \cdot JP/\sigma_\theta = RMi//\sigma_\theta$ eq. (3)

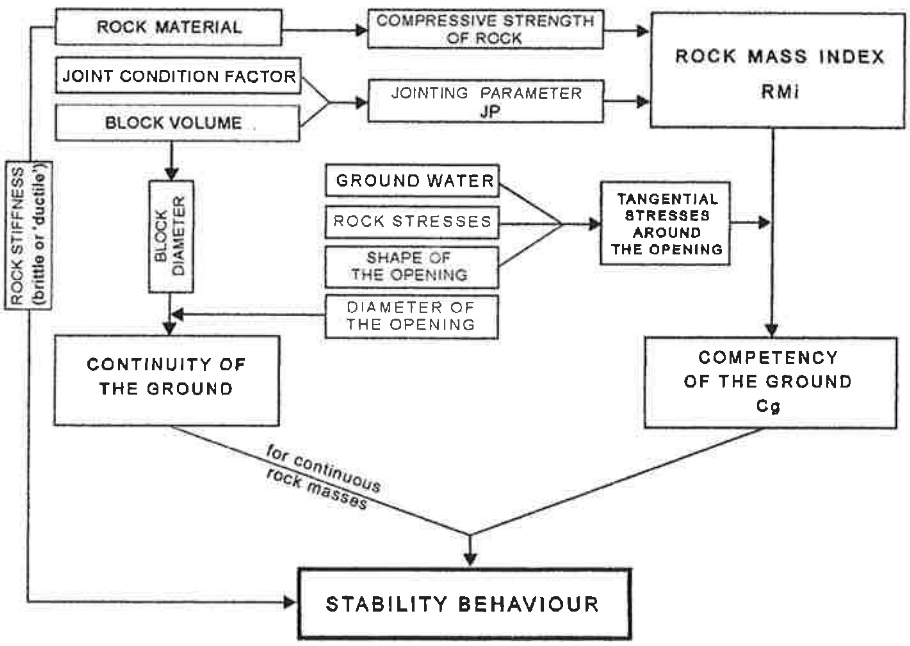

Fig. 1 The principle and the parameters involved in assessment of stability and rock support in continuous rock masses.

Overstressed (incompetent) ground leads to failure if not confinement by rock support is established. If the deformations take place instantaneously (often in connection with sound), the phenomenon is called *rock bursting*; if the deformations caused by overstressing occur more slowly, *squeezing* occurs.

- Rock burst occurs as breaking up into fragments or slab in hard, strong *brittle* rocks such as quartzites and granites.
- Squeezing acts as slow inward movements of the tunnel surface in *deformable, flexible or ductile* rocks such as soapstone, evaporites, clayey rocks (mudstones, clay schist, etc.) or weak schists, as well as in the crushed or highly jointed rocks.

Thus, in overstressed, massive rocks the deformation properties or the stiffness of the rock material mainly determines which of the two types of stress problems that will take place.

3. ROCK BURST AND SPALLING IN BRITTLE ROCKS

Rock burst is also known as *spalling*[1)] or *popping*, but also a variety of other names are in use, among them 'splitting' and 'slabbing'. They take often 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 significant impact from great differences between horizontal and vertical stresses. Selmer-Olsen (1964, 1988) has experienced that in the hard rocks in Scandinavia such anisotropic stresses might cause spalling or rock burst in tunnels located inside valley sides steeper than 20° and with the top of the valley reaching higher than 400 m above the level of the tunnel. The main reason for this is explained by very great anisotropy between the maximum and minimum principal stresses.

Rock burst failures can 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. They cause, however, often significant problems and reduced safety for the tunnel crew during excavation.

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

$\sigma_c/\sigma_\theta > 7$ stable
$\sigma_c/\sigma_\theta = 3.5$ minor (sidewall) spalling
$\sigma_c/\sigma_\theta = 2$ severe spalling
$\sigma_c/\sigma_\theta = 1.7$ heavy support required
$\sigma_c/\sigma_\theta < 1$ severe (sidewall) rock burst problems.

Similarly, Russenes (1974) has shown the relations between rock burst activity, tangential stresses in tunnel surface and the point load strength of the rock (Fig. 2). Assuming that the transition coefficient between point load strength and compressive strength is $k = \sigma_c/Is = 20$, the following classification is found from Fig. 2:

$\sigma_c/\sigma_\theta > 4$ 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, this fits relatively well with the results of Hoek and Brown shown above.

The Q-system indicates by its stress reduction factor (SRF) possible stress problems in massive rock from the ratio between uniaxial compressive strength of rock (σ_c) and the main

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

principal stress (σ_1). By combining this with the tangential stresses in the roof of horseshoe-shaped openings for $k = p_z / p_h = 1 - 2$ the following division is found:

Division used in the Q-system		corresponds for k = 1 to:	and for k = 2 to:
$\sigma_c/\sigma_1 = 2.5 - 5$	mild rock spalling	$\sigma_c/\sigma_\theta = 1.25 - 2.5$	$\sigma_c/\sigma_\theta = 1 - 2$
$\sigma_c/\sigma_1 < 2.5$	heavy rock spalling	$\sigma_c/\sigma_\theta < 1.25$	$\sigma_c/\sigma_\theta < 1$

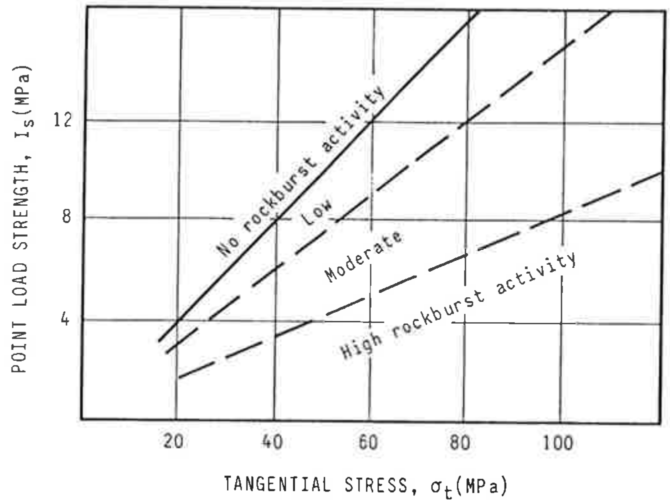

Fig. 2 The level of rock burst related to point load strength of the rock and the tangential stress ($\sigma_t = \sigma_\theta$) in the tunnel surface calculated from Kirsch's equations (from Nilsen and Thidemann, 1993, based on data from Russenes, 1974).

None of the methods above includes the effect from stress anisotropy, and the two latter neither the influence from the shape of the opening. They, therefore, do not give sufficient information to develop a competency factor for massive and intact rocks. Table 1 has been estimated based on the results from Hoek and Brown (1980) and Russenes (1974).

The value for σ_c referred to above is related to the strength of 50 mm thick samples. In massive rock the 'sample' or block size is significantly larger - in the order of some m^3. The scale effect of compressive strength causes that the compressive strength of the rock mass in such cases is $RMi = f_\sigma \cdot \sigma_c$. For block size 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 1 are approximately half of the values given for σ_c/σ_θ above.

TABLE 1 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

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 1 may not always be representative.

Rock burst and spalling involve development of new cracks parallel to the periphery. Measurements carried out by SINTEF (1990) in the 10 m wide Stetind road tunnel in Norway exposed to high rock burst show that the maximum stresses occur 5 m outwards from the

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tunnel after relief joints have developed around the tunnel. This is well in accordance with the theories of stress redistribution that the stress peak moves inward the surrounding rock masses as deformations and cracking take place.

In Scandinavia, tunnels with spalling and rock burst problems are in most cases supported by shotcrete (often fibre reinforced) and rock bolts, as this has practically been found to be most appropriate as confinement. The general trends in support design is shown in Table 2. Earlier, wire mesh and rock bolts in addition to scaling, were used as reinforcement in this type of ground. The latter solution is only occasionally applied today in Norway. Fig. 3 summarizes the various stress dependent features in brittle rock and the support used.

TABLE 2 COMMON ROCK SUPPORT METHODS APPLIED IN NORWEGIAN TUNNELS SUBJECTED TO ROCK BURST PHENOMENA

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 bolts spaced 1.5 - 3 m
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

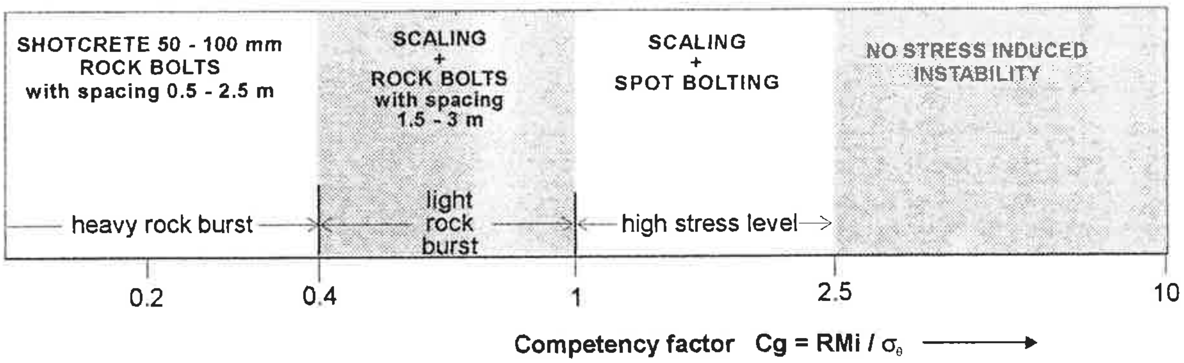


Fig. 3 Relationship between the competency factor, failure modes and rock support in continuous ground of massive, brittle rocks.

3.1 Possible Measures to Reduce Rock Burst Tunnelling 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 damage by cracks from the blasting causes the stresses to redistribute quicker off from the opening. This may be the reason why the experience in Scandinavia is 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. Also this experience indicates that rock with joints or fissures is less subjected to rock burst than massive rock under the same stress level.

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 highly reduced by optimizing the shape an layout of a cavern. Selmer-Olsen (1964, 1988) mentions that in high anisotropic stress regimes with rock burst, the extent of rock support can be reduced by reducing the radius in the roof where the largest in situ tangential stress occur. In this way it is possible to limit the overstressed area where highest amount of support is required, see Fig. 4.

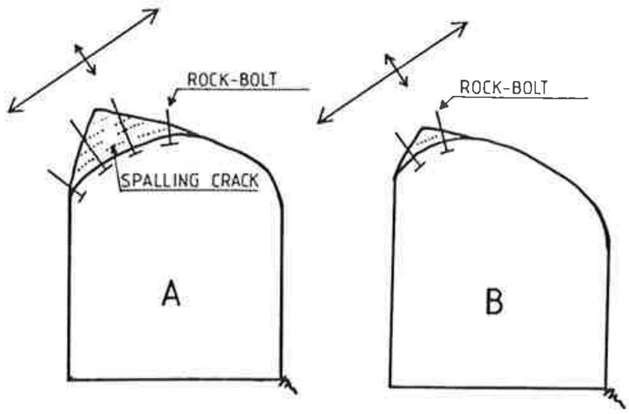
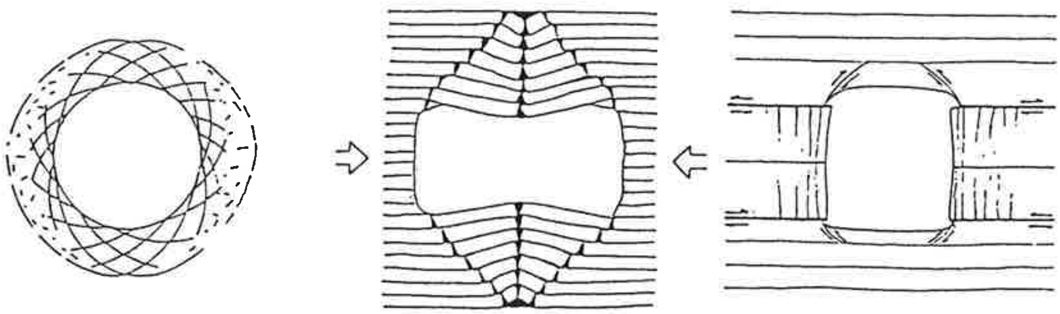


Fig. 4 If high anisotropic stresses occur, the extent of spalling (or rock burst) may be reduced by favourably shaping of the tunnel. 'A' shows the situation in a tunnel with symmetric shape, and 'B' the situation in the tunnel with an asymmetric shape with reduced radius (from Selmer-Olsen 1988)

4. SQUEEZING GROUND

The squeezing can be very large; according to Bhawani Singh et al. (1992) deformations as large 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, see Fig. 6. 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. Examples of squeezing behaviour are shown in Fig. 5 and 6.



a Complete shear failure, **b** buckling failure, **c** tensile splitting shearing and sliding

Fig. 5 Main types of failure modes in squeezing ground (from Aydan et al., 1993).

Fig. 7 shows the experience gained from practical studies made by Aydan et al. (1993) from studies of 21 tunnels in Japan with squeezing located in mudstones, tuffs, shales, serpentinites and other 'ductile' rocks with compressive strength $\sigma_c < 20$ 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. Applying straight lines instead of the slightly curved ones in Fig. 7 the division given in Table 3 has been found. In this evaluation the following assumptions have been made:

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- $k = \sigma_h / \sigma_v = 1$ and $\sigma_v = \gamma \cdot z = 0.021z$ MPa. (Aydan et al. measured γ= 18 - 23 MN/m^3)
- Horse-shoe shaped tunnels for which the ratio $\sigma_\theta / \sigma_v \approx 2.0$ in roof (Hoek and Brown, 1980).
- The expressions above are combined into $\sigma_c / z = (2 \cdot 0.021)\sigma_c / \sigma_\theta$. It is probable that scale effects have been inluded in Fig. 7; therefore σ_c has been replaced by RMi, and the values for the ratio RMi $/ \sigma_\theta$ in Table 3 have been found.

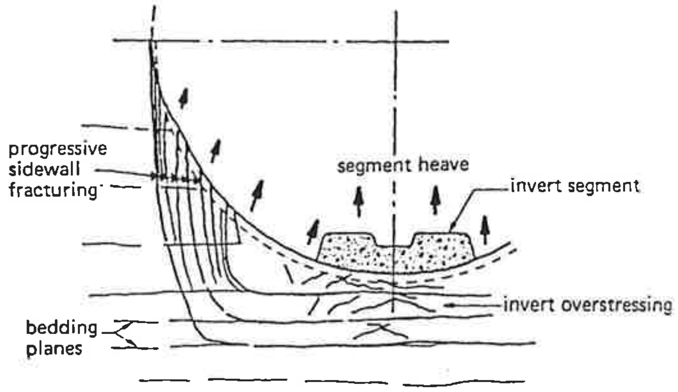
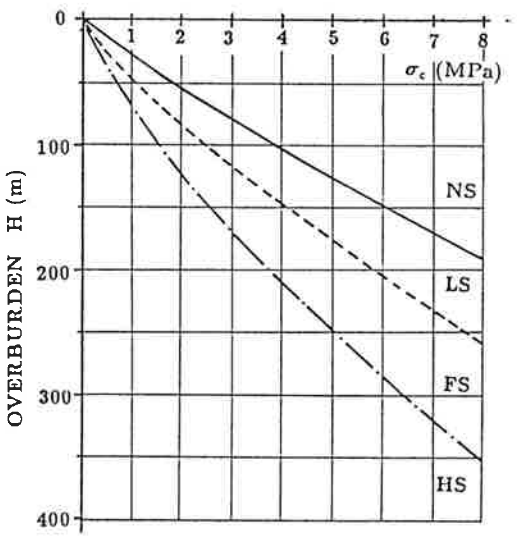


Fig. 6 Example of overstressing mechanism in the lower sidewall and in invert of a tunnel in Cyprus (from Sharp et al., 1993)



For straight lines:

NS = no squeeze	$\sigma_c/H > 1/25$
LS = light squeeze	$1/25 > \sigma_c/H > 1/35$
FS = fair squeeze	$1/35 > \sigma_c/H > 1/50$
HS = high squeeze	$\sigma_c/H < 1/50$

Fig. 7 A chart for estimating the possibility for squeezing (from Aydan et al., 1993)

Bhawani Singh et al. (1992) developed another empirical criterion, based on the Q-system, which constitutes another possibility to evaluate the competency of rock masses: Squeezing may occur if the height above the excavation is $z > 350\ Q^{1/3}$.
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.

The classification of squeezing in Table 3 is based on a limited amount of results from massive rocks and, therefore, should be updated when more data from practical experience in squeezing ground - especially in highly jointed ground - can be made available.

TABLE 3 CHARACTERIZATION OF GROUND AND SQUEEZING ACTIVTY (based on Aydan et al., 1993)

Squeezing class competency range	**The tunnel behaviour according to Aydan et al. (1993)**
No squeezing RMi /σ_θ > 1	The rock behaves elastically and the tunnel will be stable as the face effect ceases.
Light squeezing RMi /σ_θ = 0.7 - 1	The rock exhibits a strain-hardening behaviour. As a result, the tunnel will be stable and the displacement will converge as the face effect ceases.
Fair squeezing RMi /σ_θ = 0.5 - 0.7	The rock exhibits a strain-softening behaviour, and the displacement will be larger. However, it will converge as the face effect ceases.
Heavy squeezing RMi /σ_θ = 0.35 - 0.5	The rock exhibits a strain-softening behaviour at much higher rate. Subsequently, displacement will be larger and will not tend to converge as the face effect ceases.
Very heavy squeezing RMi /σ_θ < 0.35[*)]	The rock flows which will result in the collapse of the medium and the displacement will be very large and it will be necessary to re-excavate the tunnel and install heavy supports.

[*)] This value has been roughly estimated

4.1 The Use of Analytical Methods to Determine Rock Support in Squeezing Ground

As it is considered that a plastic zone is theoretically formed, elastic-plastic solutions similar to the ground response interaction analysis may be applicable to calculate the behaviour. There is, however, a limit to which the problems of rock behaviour and support may be considered in a plain 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 RMi the values of s and m in the Hoek-Brown failure criterion for rock masses, as well as c and ϕ may be easier and better characterized. The actual analyses may involve the use of ground response curves as applied in the NATM support system (Seeber et al., 1978), the Hoek-Brown or other models.

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 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.

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