STABILITY EVALUATION AND DESIGN OF TUNNEL OPENINGS IN BRITTLE-MASSIVE ROCK MASSES

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ABSTRACT
In this study, the simple and reliable method for stability assessment of tunnels in massive brittle rocks is presented taking into account the scale effect of rock mass and variation of **k**= $\frac{\sigma_h}{\sigma_v}$. The design procedure for steel fibre reinforced shotcrete lining exposed to rock burst is also provided in an analytical format.

1. INTRODUCTION

Tunnel and underground mine openings in massive brittle rocks give rise to a problem of unstable mode of the rock mass surrounding to openings, especially in deep openings where high and anisotropic stresses are likely to occur. In essence, the nature of unstable mode of massive brittle rocks around high stressed openings is observed in the form of sudden failures with explosive violence. This behavioural mode of tunnel and underground openings is named as "rockbursting" in rock mechanics literature. Rockbursting failures are seen as breaking up into fragments, blocks or slab, depending on type and magnitude of rock burst. Apart from damaging to the stability of the openings, a most vital feature of rock burst is bringing about, fatalities and serious injuries in tunnel and mining sector. In brief the effects of rock burst should be taken into account very carefully when dealing with selection and dimensioning of appropriate support systems to be utilized in overstressed tunnel and underground mine openings.

From a design point of view this paper presents the principle and the factors involved in evaluation of stability of overstressed tunnel openings in massive brittle rocks. A simple and reliable method to assess the stability of tunnel openings is also provided. In addition, a general principle for design of steel fibre reinforced shotcrete placed using the wet mix method is explained in an analytical format.

2. STABILITY ASSESSMENT OF TUNNELS IN MASSIVE BRITTLE ROCKS

2.1 General Principles

The term "stability index" for the assessment of underground mine openings/tunnels has been utilized by several researchers such as Zaslavskiy, 1972 ; Sikora and Kidybínski; Nakano, 1979 ; Aldorf and Exner,1986; Aydan et.al, 1993; Arıoğlu, 1995; and Palmstrøm, 1996. In massive brittle rock, the stability index "**S**" can be defined as :

$$\mathbf{S} = \frac{\sigma_m}{\sigma_t} \qquad (1)$$

In which,
σ_m = the uniaxial compressive strength of a rock mass
σ_t = the tangential stress

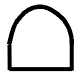
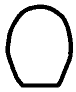
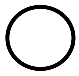
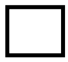
$\sigma_{t,r}$ = the tangential stress in roof
$\sigma_{t,w}$ = the tangential stress in wall

The tangential stresses acting in rock masses around a tunnel can be approximately computed by the following expressions (Hoek and Brown, 1980)

$$\sigma_{t,r} = (\mathbf{A.k - 1}).\ \sigma_v \qquad (2)$$

$$\sigma_{t,w} = (\mathbf{B.k - 1}).\ \sigma_v \qquad (3)$$

where
A, B= roof and wall factors for various tunnel shapes. For example (Table 1),

Table 1 Roof and wall factors depending on tunnel shapes			
			
A= 3.2	**A**= 3.1	**A**= 3.0	**A**= 1.9
B= 2.3	**B**= 2.7	**B**= 3.0	**B**= 1.9

k = the ratio of average horizontal "σ_h" to a vertical stress "σ_v" . To estimate **k** values a simplified expression put forward by Sheorey,1994 can be utilized :

$$\mathbf{k = 0.25 + 7\ E_h \left(0.001 + \frac{1}{H}\right)} \qquad (4)$$

where, **H** is the depth (m) and $\mathbf{E_h}$ (GPa) is the average rock mass elastic modulus. For example, in the Scandinavian Precambrian and Palaeozoic and in the Canadian crystalline rocks, the horizontal stresses are remarkably highly than the vertical stress down to a few hundred meters (Palmstrøm, 1996). Based upon a preparatory assessment of graphs given by Arjang, 1998 the following approach can be used for calculation of $\mathbf{k} = \frac{\sigma_h}{\sigma_v}$ values in

- Crystalline rock types such as most hardrock mines-road tunnels

  $$\mathbf{k = 5.13\ H^{-0.16}}\ ,\ \mathbf{H}\ (\mathrm{m}) \qquad (5)$$

  Equation 4 ($\mathbf{E_h}$= 100 GPa)
- Low stiffness sedimentary and satisfied rocks :
  Equation 4 ($\mathbf{E_h}$ < 100 GPa , preferably 25-75 GPa)

In evaluation of the stability of underground openings continuity of the ground plays an important role in characterizing the ground. A continuity factor "**CF**" is defined as a ratio

$$\mathbf{CF} = \frac{\mathbf{D_t}}{\mathbf{D_b}} \qquad (6)$$

Continous rock masses exist as
- Massive rocks (slightly jointed) with a continuity factor **CF** ≤ ~ **5** ; or
- Highly jointed and crushed rocks, where the factor under consideration **CF** > ~ **100**

In other words, discontinuous rock masses possess **CF** factors between the values above mentioned (Palmstrøm, 1996). Taking into account continuity factor corresponding to massive rocks the uniaxial compressive strength of rock mass "σ_m" can be estimated (Arıoğlu and Girgin, 1998).

$$\sigma_m = f_s . \sigma_{lab.} = \left(\frac{d_{lab.}}{D_b}\right)^{0.2} . \sigma_{lab.} = \left(\frac{d_{lab.}}{\frac{D_t}{CF}}\right)^{0.2} . \sigma_{lab.} \quad (7)$$

$$A_t = \frac{\pi}{4} D_e^2 = 0.785\, D_e^2 \quad , \quad D_e = 1.128 \sqrt{A_t}$$

CF= 5

$\mathbf{d_{lab.}= 0.05\ m}$

For $\mathbf{D_t}$ = 6.0-16.0 m, the mean value of $\mathbf{D_t^{-0.2}}$ can be taken as **0.65**.

$$\sigma_m = 0.758\, D_t^{-0.2} \sigma_{lab.} \approx 0.50\, \sigma_{lab} \quad (8)$$

In which,
σ_m= the uniaxial compressive strength of rock mass
$\mathbf{f_s}$= a factor defining the scale effect for the uniaxial compressive strength
$\sigma_{lab.}$= the uniaxial compressive strength of a laboratory rock sample with a diameter of $\mathbf{d_{lab.}}$= 0.05 m, intact rock strength, MPa
$\mathbf{D_b}$= the block diameter measured , m
$\mathbf{D_t}$ = the diameter of tunnel , m
$\mathbf{D_e}$ = the equivalent diameter of tunnel , m
$\mathbf{A_t}$= the area of tunnel, m^2
The Geological Strength Index (GSI), proposed by Hoek,1994 provides a tool for predicting the reduction in rock mass strength for different geological conditions. In this analysis, it is interesting to make use of Hoek's approach for hard, strong, brittle rocks such as Gneiss, Granite, Granodiorite, Diorite, Gabbro. It is reasonable to assume GSI= 80 for massive brittle rocks as a mean value. Using relationship between ratio of cohesive strength "$\mathbf{C_m}$" -in situ strength -to uniaxial compressive strength of intact rock "$\sigma_{lab.}$" and GSI given by Hoek and Brown,1998 for GSI= 80 the value of $\mathbf{C_m}/\sigma_{lab.}$ can be determined to be 0.10. In addition, the friction angle "ϕ'" for different combinations of GSI and $\mathbf{m_i}$ (rock constant for intact rock) was provided in the same source. Using the values of GSI and $\mathbf{m_i}$ the value of ϕ' is read as 45^0-47^0 (For the above-mentioned rocks $\mathbf{m_i}$ can be assumed to be in the order of 33 and 27).
Making use of the Mohr-Coulomb criterion , the uniaxial compressive strength of the rock mass and the slope of the line relating major principal stress and minor principal stress can be estimated from

$$\sigma_m = 2\sqrt{K}\, C_m \quad (9)$$

$$K = \frac{1+\sin\phi'}{1-\sin\phi'} = \frac{1+\sin 45}{1-\sin 45} = 5.828 \quad (10)$$

For massive brittle rocks in question the uniaxial compressive strength of the rock mass may be as follows

$$\frac{C_m}{\sigma_{lab.}} = 0.10$$

$$\sigma_m = 2\sqrt{5.828}\,(0.10\, \sigma_{lab}) \cong 0.483\, \sigma_{lab.}$$

As can be seen, the results of two different approaches seem to be in good agreement. In passing, the scale factor "$\mathbf{f_s}$" was reported to be 0.45-0.55 in Palmstrøm, 1995, which agrees with the values obtained from the approaches given herein.
Having estimated the uniaxial compressive strength of the rock mass, the stability index "**S**" (Eq-1) can be rewritten in the following form

$$\mathbf{S} = \frac{\mathbf{0.5}\,\sigma_{\mathbf{lab.}}}{\mathbf{(A\,k-1)}\,\sigma_{\mathbf{v}}} = \mathbf{18.5}\,\frac{\sigma_{\mathbf{lab.}}}{\mathbf{(A\,k-1)\,H}} \tag{11}$$

$$\sigma_{\mathbf{v}} \cong \mathbf{0.027\,H} \quad , \text{MPa} \;\; , \mathbf{H} \;\; (\text{m})$$

Equation 11 for arch shaped tunnel (**A**=3.2, **B**=2.3) is plotted in Fig.1 which illustrates the stability index "**S**" variation against the depth of tunnel "**H**" in a function of the uniaxial compressive strength of intact rock- laboratory strength- "$\sigma_{\mathbf{lab.}}$" This chart gives directly the stability index under following conditions :

- High and variable anisotropic stresses owing to tectonic stresses are not taken into account. The ratio horizontal/vertical stress is reasonably well predicted by Equation 5 based on actual stress measurements.
- The strength of the rock sample should be determined in the same direction as the tangential stress is occuring . If strength anisotropy in the rock is well defined this property can be included in the analysis taking a proper reduction factor into consideration.
- Characterization of failure mode in massive brittle rock reported by Palmstrøm, 1995 and 1996 is adopted :

Stability index “S”	Failure modes
> 2.5	No rock stress induced in stability
2.5 - 1	High stress-slightly loosening
1 - 0.5	Light rock burst or spalling
< 0.5	Heavy rock burst

These values are also indicated in Fig.1. Examining Fig.1, the main results can be summarized as follows :

- For a given uniaxial compressive strength of rock sample "$\sigma_{\mathbf{lab.}}$" , as the depth of tunnel increases the stability index decreases. For example, for $\sigma_{\mathbf{lab.}}$= **125** MPa if the depth of tunnel is approximately 450 m the tunnel under consideration will be subjected to "light rock burst" or spalling in roof. If the depth of tunnel is about 1000 m the tunnel is likely to be exposed to "heavy rock burst".
- In case of constant depth of tunnel as the uniaxial compressive strength of rock sample increases - the rock becomes brittle- the stability index increases. For example, if **H** is taken as 400 m for $\sigma_{\mathbf{lab.}}$= 50 MPa the stability index can be estimated to be 0.5 i.e "heavy spalling" will occur in the tunnel in question. If the uniaxial compressive strength is 250 MPa the stability index for the same tunnel can be predicted as about **2.25**. This means that the tunnel will be overstressed. From a failure mode point of view, slight loosening may be expected.

2.2 Stability Problems of Road Tunnels in Norway

In this section, by using engineering values belonging to the Heggura abd Kobbskaret road tunnels prediction capacity of Fig.1 was illustrated. The values under consideration were obtained from Myrvang and Davik, 1997. As can be seen from Fig.1 the Heggure road tunnel ($\sigma_{\mathbf{lab.}}$= 120-195 MPa, **H**=600-700 m) is in zones of high stress-slightly loosening and high rock burst/spalling. As for the Kobbskaret road tunnel ($\sigma_{\mathbf{lab.}}$= 36-143 MPa, **H**=25-600 m), this tunnel is subjected to high rock stresses and heavy rock burst. According to actual behaviour of the

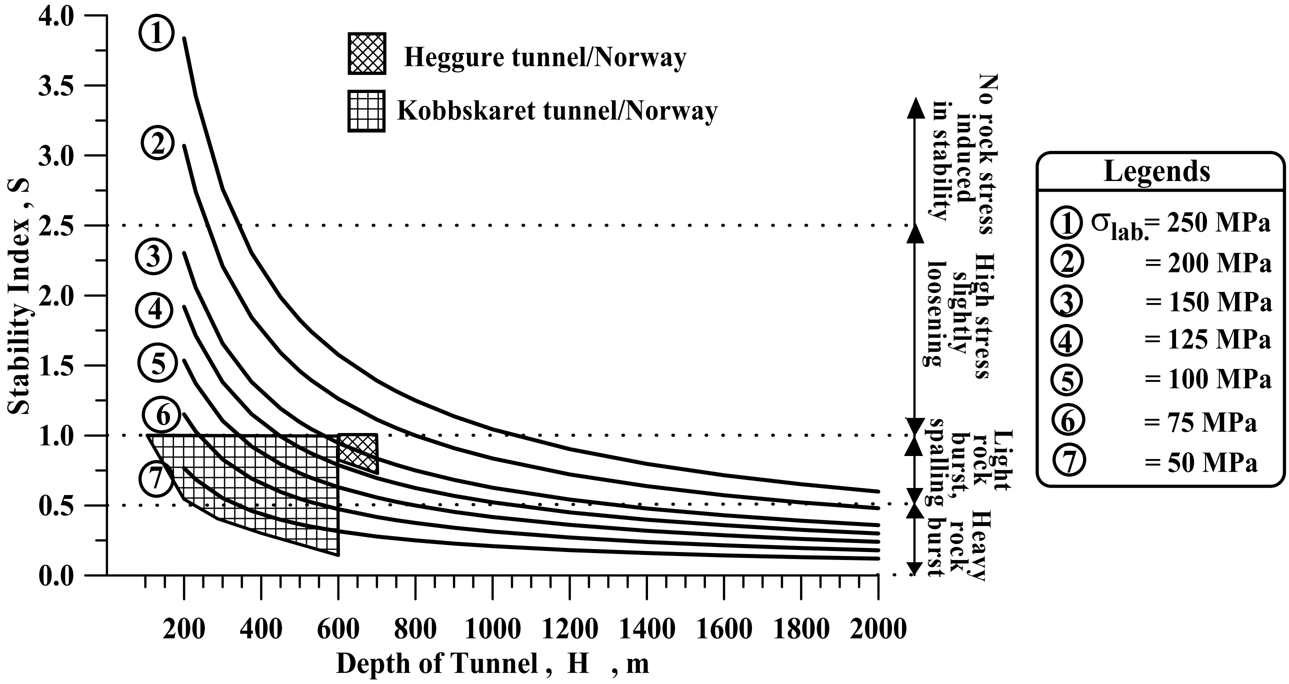


Fig.1 Relationship between depth of tunnel and stability index for different uniaxial compressive strength of rock sample.

tunnels reported in the above mentioned reference, heavy spalling owing to high rock stresses was experienced in both tunnels. In brief, the findings of Fig.1 can be said to be in good agreement with true behaviours of tunnels in review.

3. DESIGN OF STEEL FIBRE REINFORCED SHOTCRETE (SFRS) TUNNEL LININGS SUBJECTED TO ROCK BURST

3.1 General

Steel fibres are broadly employed for shotcrete owing to holding several advantages (Moyson, 1994):

• Ductility : the addition of steel fibres provides the shotcrete an important degree of ductility. This ductility enhances flexural loading capacity of steel fibre reinforced shotcrete linings. Especially for tunnels subjected to rock burst, the structural performances of SFRS linings are said to be very effective due to the fact that they provide a remarkably additional deformation capacity after exceeding their strength depending upon type and geometry of steel fibres (Arıoğlu and Girgin, 1998).

• Practical : the steel fibres are easy to add to the shotcrete mixtures. In comparison to conventional mesh system a good deal of time saving and labour reduction can be realized in a tunnel project.

• Economy : the shorter execution time and savings in labour expenditure make SFRS a competitive and economical support element.

3.2 Design Procedure and Numerical Example

The design procedure of steel fibre reinforced shotcrete makes (SFRS) for tunnel linings subjected to rock burst is outlined in Table-2.

In design the critical parameter is estimation of permanent radial support pressure required to stabilize the roof of an opening exposed to rock burst. Estimation of support pressure can be made from well known formula in rock engineering literature (Barton,Lien and Lunde, 1974).

According to this source,for 0-2 joint sets (J_n<6) the support pressure "P_s" can be computed from

$$P_s = \frac{2\,Q^{-0.333}\,J_n^{0.5}}{30\,J_r} \quad , P_s \ (\text{MPa}) \tag{12}$$

the Tunneling Quality Index "**Q**" (Barton,Lien and Lunde, 1974) is defined by :

$$Q = \frac{RQD}{J_n}\,\frac{J_r}{J_a}\,\frac{J_w}{SRF} \tag{13}$$

where :

RQD = Rock Quality of Designation

$\mathbf{J_n}$ = Joint Set Number. In massive brittle rocks $\mathbf{J_n}$ can be assumed to vary in value from 0.5 to 6. In this analysis, the mean value of $\mathbf{J_n}$ is taken as **3**.

$\mathbf{J_r}$ = Joint Roughness Number describing both large and small scale surface texture for discontinuities. Assuming rough condition $\mathbf{J_r}$ can be adopted as **1.5**.

$\mathbf{J_a}$ = Joint Alteration Number defining the surface alteration and frictional resistance of the critical joint set. In the case of massive brittle rocks as the joint alteration is negligible the value of $\mathbf{J_a}$ can be taken equal to **1**.

$\mathbf{J_w}$ = Joint Water Reduction Number. $\mathbf{J_w}$ varies from 1.0 for dry openings to 0.05 for openings with excessive water pressure and large inflow.In this analysis $\mathbf{J_w}$ is assumed to be equal to **1**.

SRF : Stress Reduction Factor. According to Grimstad and Barton,1993 , in the case of massive brittle rocks **SRF**=1 for favourable stress condition ; **S**= 0.5-2 for high stress ; **S**= 5-50 for moderate spalling /slabbing ; **S**= 50-200 for slabbing and rock burst and **S**=200-400 for heavy rock burst.

For assumed values given in Table-2, Fig.2 displays relationship between the stress reduction factor and support pressure in function of various the rock quality designation. From Fig.2 it is important to notice that as **SRF** factor increases the expected radial pressure on tunnel support system enhances markedly. For example, for normal condition corresponding to **SRF**=1 the support pressure is about 3 t/m^2 (0.03 MPa). If the tunnel is subjected to heavy rock burst (**SRF**= 200-400) then the rock pressure acting on the support system may be in order of 16 t/m^2 (0.16 MPa). In brief, in the case of rock burst in massive brittle rocks the support pressure should be increased when dealing with the design of support systems.

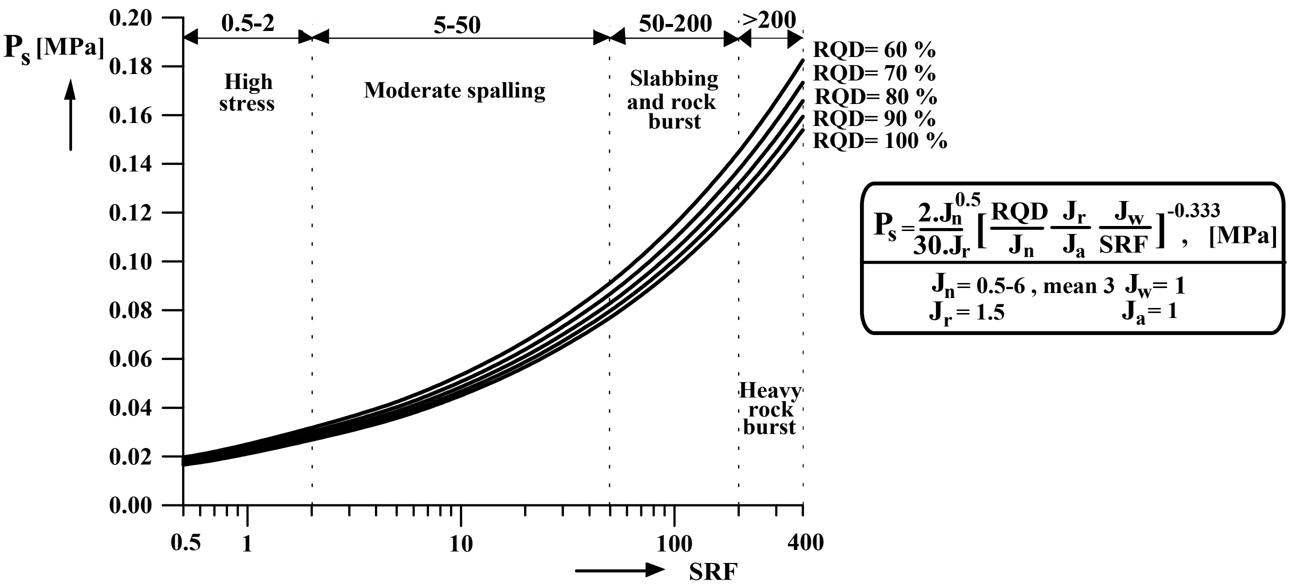


Fig.2 Relationship between the stress reduction factor (**SRF**) and support pressure $\mathbf{P_s}$ in function of the Rock Quality of Designation (**RQD**)

Table 2 Design of Steel Fibre Reinforced Shotcrete (SFRS) Lining Subjected to Rock Burst

DESIGN OF SFRS LININGS SUBJECTED TO ROCK BURST

Data :

- Depth of tunnel (**H**)
- Diameter of tunnel ($\mathbf{D_t}$)
- Uniaxial compressive strength of laboratory rock sample ($\sigma_{lab.}$)
- Characteristic compressive strength of shotcrete at 28 days ($\mathbf{f_c}$)

↓

- Scale effect $\mathbf{f_s}$ for uniaxial compressive strength of massive rocks taking **CF=5** ,

$$f_s = \left(\frac{d_{lab.}}{D_t / CF} \right)^{0.2}$$ is calculated

- Tangential stresses acting on roof and wall of tunnel are estimated ;
  for roof $\sigma_{t,r} = (A.k - 1)\,\sigma_v$ and for wall $\sigma_{t,w} = (B.k - 1)\,\sigma_v$
- Stability index $S = \dfrac{f_s \cdot \sigma_{lab.}}{\sigma_t}$ is determined (Fig-1)

↓

- Stress Reduction Factor **SRF** can be accessed by taking into account previous field data, experience and value of stability index **S** , $S \rightarrow SRF$
- Tunneling Quality Index **Q** is defined by $Q = \dfrac{RQD}{J_n} \dfrac{J_r}{J_a} \dfrac{J_w}{SRF}$
- Support pressure $P_s = \dfrac{2\,Q^{-0.333}\,J_n^{0.5}}{30\,J_r}$ (MPa) is estimated. In massive brittle rocks, for

$(RQD)_m = 90\ \%$, $(J_n)_m=3$, $J_r= 1.5$, $J_a=1$, $J_w=1$, $P_s= 0.0217\,(SRF)^{0.333}$ (MPa) (Fig-2)

↓

- Span length ($\mathbf{l_b}$) and spacing distance of rock bolts (**a**) are found depending on **Q**
- Maximum bending moment is calculated → $M_{max.} = P_s . a . l_b^2 / 8$

↓

- First crack flexural strength of plain shotcrete can be predicted from $f_o = 0.4\, f_c^{0.66}$
- Toughness quotient ($\mathbf{R_e}$) is described depending on fibre type and in situ dosage.
- Average post cracking (equivalent) flexural strength is determined $f_e = \dfrac{R_e}{100} f_o$

↓

- Thickness of SFRS layer **t** is computed from $\dfrac{M_{max.}}{a\,t^2/6} \leq f_e$

which corresponds to maximum bending moment of simply supported beam uniformly loaded. In other words, the steel fibre reinforced shotcrete layer acts as a bearing element supported by bolts and arches (Fig.3).

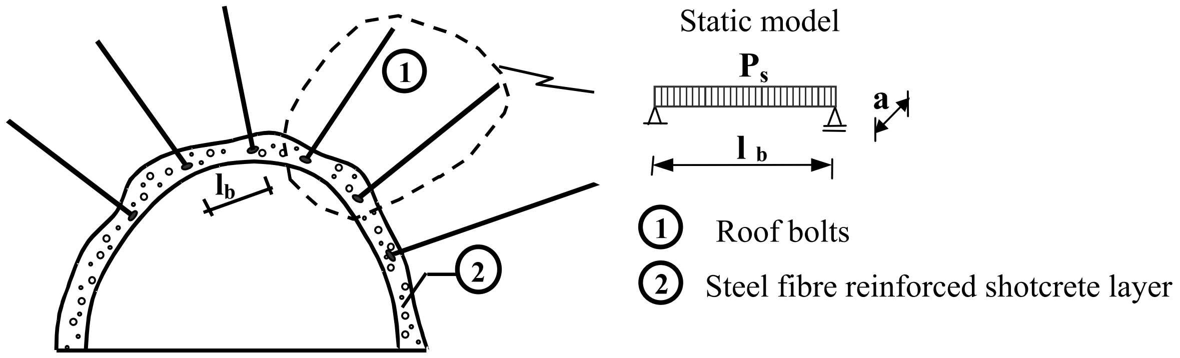


Fig.3 Cross section of tunnel with steel fibres reinforced shotcrete layer

The magnitude of maximum flexural stress "$\sigma_{max.}$" is computed with the following formula :

$$\sigma_{max.} = \frac{M_{max.}}{W} = \frac{0.125\ P_s .a.l_b^2}{a\ t^2/6} = 0.75 \frac{P_s .l_b^2}{t^2} \leq f_e \quad (15)$$

The average post cracking strength or equivalent flexural strength "f_e" can be expressed as

$$f_e = \frac{R_e}{100}.f_o = 4x10^{-3}.R_e.f_c^{0.666} \quad , \text{ MPa} \quad (16)$$

Putting Eq.15 into Eq.14 SFRS thickness can be obtained from the following expression

$$t \geq 13.7\ l_b \left(\frac{P_s}{R_e.f_c^{0.666}} \right)^{0.5} \quad , \quad P_s \text{ (MPa)} \quad (17)$$

where

W = section modulus of the crack free section , mm^3

l_b = span length, mm (Fig.3)

a = width of the shotcrete layer-spacing distance of bolts , mm (Fig.3)

t = thickness of shotcrete layer , mm

f_e = equivalent flexural strength , MPa

f_o = first crack flexural strength of plain shotcrete. Its value can be predicted from the following empirical formula

$f_o = 0.4 f_c^{0.666}$ MPa (N/mm^2) . In which f_c is 28 days compressive strength measured on cubes, N/mm^2 (Vandewalle, 1997)

R_e= toughness quotient depending upon type and in situ dosage of steel fibres. Toughness quotient values can be directly obtained from identity charts established by N.V.Bekaert S.A . R_e can be also predicted by means of the following expression (Arıoğlu and Girgin, 1998).

$$R_e = A_0 . m_f^2 + B_0 .m_f + C_0 \quad (18)$$

where

A_0, B_0 and C_0 = constant values belonging to (Eq.18) (Table.3)

m_f = Fibre dosage-in situ , kg/m^3

r= Correlation coefficient

Table 3 Constants of regression relationship for Dramix ® steel fibres

Fibre type	A_0	B_0	C_0	r
RC 65/30	-0.0262	3.0548	-10.5	0.996
RC 65/35	-0.0167	2.0881	14.571	0.999
ZP 305	-0.0271	3.15	-17	0.998

3.3 Numerical Example

A cross section of the tunnel is shown in Fig.3. The material properties of shotcrete and geomechanical conditions are described as follows :

- Compressive strength of shotcrete at 28 days $\mathbf{f_c} = \mathbf{40}$ MPa- cube specimen
- Depth of tunnel **H**= 850 m
- Type of rock : Gneiss
- Uniaxial compressive strength of rock sample $\sigma_{lab.}$**=130** MPa
- Rock Quality of Designation **RQD** = 80 %

Access the stability of this tunnel and dimension steel fibre reinforced shotcrete layer (SFRS) to be utilized in this tunnel project, adapted procedure outlined in Table 2 can be utilized.

- Evaluation of stability

The ratio horizontal/vertical stress is

$$\mathbf{k = 5.13\ H^{-0.16} = 5.13\ (850)^{-0.16} = 1.74} \quad \mathbf{, H}\ (\mathrm{m})$$

The uniaxial compressive strength of rock mass is

$$\sigma_m \cong \mathbf{0.5}\ \sigma_{lab.} \mathbf{= 0.5x130 = 65}\ \mathrm{MPa}$$

The stability index **S** is determined to be

$$\mathbf{S = 18.5 \frac{\sigma_{lab.}}{(A\,k - 1)\,H} = 18.5 \frac{130}{(3.1x1.74 - 1)x850} = 0.64} \quad \mathbf{, A=3.1} \quad (\text{Table-1})\ (\text{Fig.1})$$

According to Palmstrøm, 1995 and 1996, failure mode under given conditions may be light rock burst or spalling.

- Estimated support pressure $\mathbf{P_s}$

  In the case of spalling, mean value for **SRF** can be assumed to be **125** (Fig.2).

$$\mathbf{P_s = 0.0217\ (SRF)^{0.333} = 0.0217\ (125)^{0.333} = 0.108}\ \mathrm{MPa}$$

$$\mathbf{P_s = 10.8}\ \mathrm{t/m^2}$$

- Dimensioning of thickness of SFRS
  - First crack flexural strength of plain shotcrete

$$\mathbf{f_o = 0.4\ f_c^{0.666} = 0.4x(40)^{0.666} = 4.66}\ \mathrm{MPa}$$

  - Toughness quotient of the steel fibre reinforced shotcrete for fibre Dramix® RC 65/35 and fibre dosage-in situ- 40 kg/m^3 from the Dramix Identity Chart the value in question can be obtained as $\mathbf{R_e}$**= 72**

  - Average post cracking flexural strength of SFRS

$$\mathbf{f_e = \frac{R_e}{100} f_o = \frac{72}{100} x4.66 = 3.35}\ \mathrm{MPa}$$

  - Spacing distance of roof bolts

The **Q** value is computed as follows :

$$\mathbf{Q = \frac{RQD}{J_n} \frac{J_r}{J_a} \frac{J_w}{SRF} = \frac{80}{3} x \frac{1.5}{1} x \frac{1}{125} = 0.32}$$

According to Chouquet and Charette, 1988 $\mathbf{l_b}$ x $\mathbf{l_b}$ pattern for span length of roof bolts can be predicted from

$$\mathbf{l_b = \frac{1}{(-0.227 \ln Q + 0.839)^{0.5}} = \frac{1}{(-0.227 \ln 0.32 + 0.839)^{0.5}} \cong 1.0\ m}$$

- Thickness of SFRS layer from Eq.17 can be calculated as mm

$$\mathbf{t \geq 13.7\, l_b \left(\frac{P_s}{R_e \,.\, f_c^{0.666}} \right)^{0.5} = t \geq 13.7\ x\ 1000 \left(\frac{0.108}{72\ x\ 40^{0.666}} \right)^{0.5} = 155}\ \text{mm}$$

The following design values can be established :

• Support pressure $\mathbf{P_s}$**= 0.108** MPa
• First crack flexural strength of plain/shotcrete $\mathbf{f_o}$**= 4.66** MPa
• Average post cracking flexural strength of SFRS $\mathbf{f_e}$**= 3.35** MPa
• Span length of bolts $\mathbf{l_b}$**= 1000** mm
• Thickness of SFRS **t= 155** mm

4. CONCLUSIONS

• A simple and reliable method for preliminary stability assessment of tunnel to be subjected to rock burst is suggested by taking into account variation of the ratio $\mathbf{K} = \frac{\sigma_h}{\sigma_v}$ and the scale effect of rock mass (Fig.1).

• The design procedure for steel fibres reinforced shotcrete lining is also described in an analytical format (Table-1). Since the choice of **SRF** is very important in the case of rock burst in this procedure the modified stress reduction factor **SRF** in the Q-system has been taken into consideration to estimate appropriate rock pressure on the tunnel support systems (Fig.2) .

• Recent trials and careful observations carried out in Norwegian road tunnels disclose that steel fibre reinforced shotcrete (SFRS) linings with roof bolts display a good structural performance in rock burst conditions. Nevertheless, the thickness of SFRS should be dimensioned by paying attention to the parameters (support pressure, geometry of roof bolting, compressive strength of plain shotcrete, average post-crack flexural strength, fibre type and dosage etc.) involved in the design (Section 3.2).

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