State-of-the-Art of Tunnelling Through Squeezing Ground Conditions

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ABSTRACT

On excavation of rock mass, the equilibrium of in situ stresses is disturbed around the excavation and redistribution of stresses takes place. If the induced stresses exceed the strength of the rock mass around periphery of the underground opening, failure of rock mass takes place developing a broken zone around it. The radius of the broken zone depends upon value of in situ stresses and the rock mass quality whereas its shape varies with the shape of the tunnel and in situ stress anisotropy. The failed rock mass around the tunnel periphery starts advancing in the tunnel. The excessive tunnel closure is required to be arrested by installing supports in time. In very poor rock masses under the influence of high in situ stresses, this closure is very high and leads to squeezing ground conditions. The paper summarizes the state-of-the-art with regard to prediction of squeezing ground condition, tunnel closure and support pressure for future direction of research.

Keywords: Squeezing ground; Tunneling; Anisotropy; Support pressure; Deformation; Empirical approach

1. INTRODUCTION

The trend of utilizing underground space is increasing day by day in the form of traffic, rail & road tunnels in hilly regions, hydro tunnels /caverns, underground repositories for burial of high level nuclear waste (HLNW), ammunition storage for defence purposes, storage of petroleum products and underground research laboratories. A silent tunnelling revolution is now going on in India. Most of the underground excavations are carried out in Himalayan region in India. The geology of this region is extremely fragile and exhibits very complex rock mass behaviour. Some of the regions are highly tectonically active leading to high horizontal in situ stresses which affects the underground excavation work in the form of squeezing in weak rock masses and rock bursts in competent & strong rock masses even at a shallow depth. Squeezing may be defined as follows (Barla,1995).

"Squeezing of rock is the time-dependent large deformation, which occurs around a tunnel and other underground openings, and is essentially associated with creep caused by (stress) exceeding shear strength (limiting shear stress). Deformation may terminate during construction or continue over a long time period".

The squeezing conditions are common in the Lower Himalaya in India, the Alps and other young mountains of the world where the rock masses are weak, highly jointed, faulted, folded and tectonically disturbed and the overburden is high.

Although many studies have been carried out in India and abroad to tackle the squeezing ground conditions, however the existing knowledge still needs refinement for reliable predictions of support pressures and closures for such rock masses. Barla (2001) reviewed the existing approaches with regard to the design of tunnels under squeezing ground conditions and concluded that even today, with significant steps forward in Geotechnical Engineering, the fundamental mechanisms of squeezing are not fully understood. Some of the facts which lead to inadequate understanding in analyzing squeezing ground conditions are that (i) the effect of size of opening is not well understood, (ii) rock mass strength under the prevailing stress conditions at the periphery of the opening is still a difficult problem due to presence of discontinuities, (iii) effect of in situ stresses and the closure behaviour of openings is complex especially due to anisotropic nature of rock masses and (iv) assumption of simple shape (circular).

A study is proposed to be carried out for refinement of the existing knowledge in order to predict reliable support pressures and closures in underground excavations for squeezing ground conditions.

2. STATUS OF TUNNEL DESIGN IN SQUEEZING GROUND

The design approaches on tunnelling in squeezing ground has been classified into six broad categories; (i) observational, (ii) empirical, (iii) semi-empirical, (iv) theoretical, (v) numerical, and (vi) physical modelling as following.

2.1 Observational Approaches

Observational approaches provide a qualitative solution to a problem. It is derived from the experience gained by observation while working. Some of these approaches pertaining to the squeezing ground conditions have been considered.

NATM (New Austrian Tunnelling Method), a technique for supporting a tunnel developed by Rabcewicz (1964) is the best example of this approach. This technique is based on the observation of the performance of the installed supports and modification of the same at every stage, if required. The philosophy of this technique is “support as you go”. Further, Muller (1978) listed five important principles of this techniques: (i) mobilization of the strength of the surrounding rock mass, (ii) prevention of rock mass from loosening and excessive deformation, (iii) instrumentation to assess the influence of time on behaviour of rock mass and support system, (iv) permanent support and lining must be thin walled to minimize bending moment, and (v) statically, the tunnel is considered as a thick-walled tube, consisting of rock and the support and/or lining.

Selmer-Olsen and Broch (1977) described an old rule of thumb in Norway: if the valley side height above the tunnel is 500 m or more with slope of 25° or steeper, there is a possibility of stress induced instability. This rule of thumb was developed on the basis of repeated experiences that in tunnels running parallel to fjords (a long narrow inlet of the sea between steep cliffs; common in Norway) with steep hill sides, rock-burst problems occurred in the tunnel-wall and in the part of the roof that was closest to the fjord.

Ward (1978) felt that tunnelling through squeezing ground is an art and observed that a support installed close to the face attracts higher load.

Dube (1979) carried out field instrumentation in Giri tunnel of lower Himalayas subjected to squeezing conditions and developed a graphical method to assess the radius of broken zone which was observed to be 2-10 times the radius of the tunnel. It was also concluded that the in situ stresses are critical parameters that affect the geometry of the broken zone, support pressure and displacement at the periphery of the openings.

Jethwa (1981) observed support pressures and closures by instrumentation in Chhibro-Khodri tunnel of Himalayan region under squeezing conditions and discovered the existence of compact zone adjacent to the tunnel periphery within the broken zone in supported tunnel. In the compact zone, the volume of failed rock mass reduced with time because of support reaction. It was concluded that the ultimate support pressures would be 2 to 3 times the short-term support pressures in squeezing ground conditions.

Whittaker et al. (1983) carried out instrumentation in three mine roadways in Britain and concluded that yield zone in competent rock masses developed after a relatively shorter period of time (3 days) and tunnel advances (9m), whereas complete development of yield zone in the weaker rock masses was found to be time-dependent.

Malan and Basson (1998) studied the rock mass behaviour around the underground openings of a deep South African gold mine and concluded that the possibility of squeezing behaviour becomes more pronounced with increase in depth of the opening and decrease in quality of rock mass.

Kontogianni et al. (2006) analyzed the closure data obtained from two Greek and two French tunnels and concluded that time dependent deformation is ignored many times but it is observed to contribute more than 50% of the total deformation that is contributed by time-dependent or creep effect and face advance effect.

2.2 Empirical Approaches

Empirical approaches are based purely on experience and comparison of the effects of parameters in the field. Various research workers have proposed empirical approaches for the assessment of potential squeezing phenomenon, which are as following.

Wood (1972) initially proposed the concept of Competence Factor to assess the stress induced instability in tunnel. The factor is defined as the ratio of the unconfined compressive strength of the rock mass (σ_{cm}) to overburden stress. When this factor is less than 2, the ground will undergo squeezing. This parameter has been used by many authors in many cases to recognize the squeezing potential of tunnels. However, σ_{cm} needs to be estimated by empirical correlations using either uniaxial compressive strength of intact rock (σ_{ci}) or rock mass quality.

Saari (1982) suggested the use of the tangential strain of tunnels as a parameter to assess the degree of squeezing of the rock, and a threshold value of 1% was also suggested for the recognition of squeezing (in Shrestha, 2005).

Singh et al. (1992) developed an approach to predict squeezing behaviour of rock mass on the basis of rock mass quality *Q* (Barton et al., 1974) and overburden depth *H* (m). The approach was developed after analyzing 41 tunnel sections data (17 from case histories given in Barton et al., 1974 and 24 were obtained from tunnels of Himalayan region). This approach has given a demarcation line (Eq. 1) to differentiate squeezing condition from non-squeezing condition (Fig.1).

$$H = 350\ Q^{1/3}\ m \qquad (1)$$

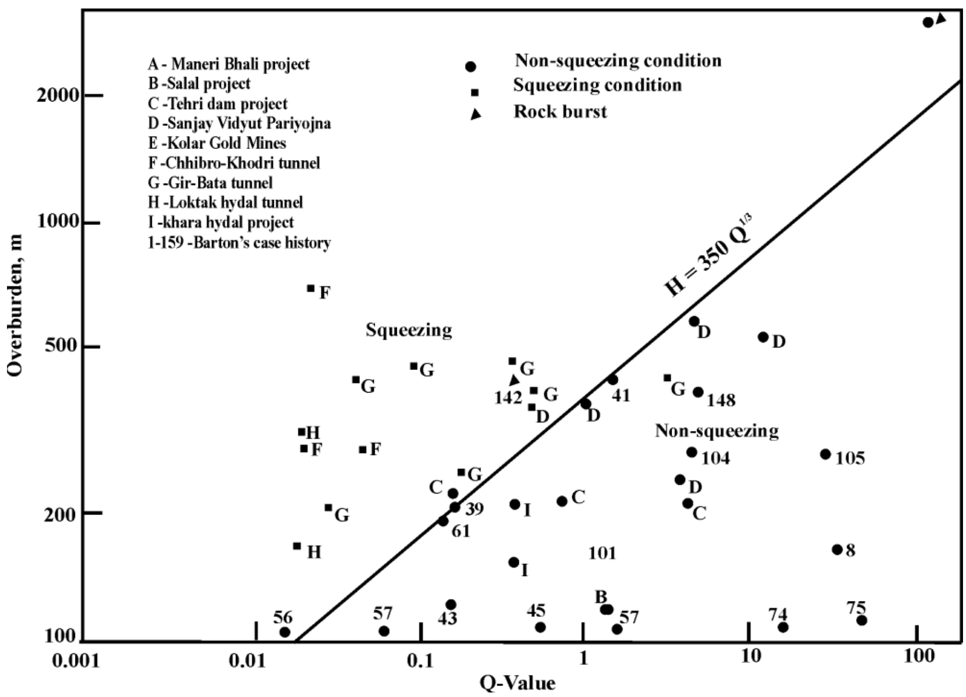

Fig. 1 - Prediction of squeezing ground condition (Singh et al., 1992)

The data points lying above the demarcation line represent squeezing conditions, whereas those below this line represent non-squeezing conditions. This can be summarized as follows:

For squeezing condition, $H > 350\ Q^{1/3}\ m$ (2)

For non squeezing condition, $H < 350\ Q^{1/3}\ m$ (3)

In addition to the above, following approaches were developed by Singh et al. (1992) for prediction of support pressure using Barton's Q-value.

$$p = \frac{0.2}{J_r} Q_i^{-1/3} f . f' . f'' \text{MPa} \quad (4)$$

where

Q_i = $5Q$ for short-term support pressure,
= Q for ultimate support pressure,
f = $(1 + H - 320) / 800$ for overburden > 320m,
$\geq$ 1
f' = correction factor for tunnel closure (from Fig.2),
f'' = correction factor for time after excavation,
= $\log (9.5\ t^{0.25})$, where t is time (in months) after excavation, and
Q = actual post construction rock mass quality.

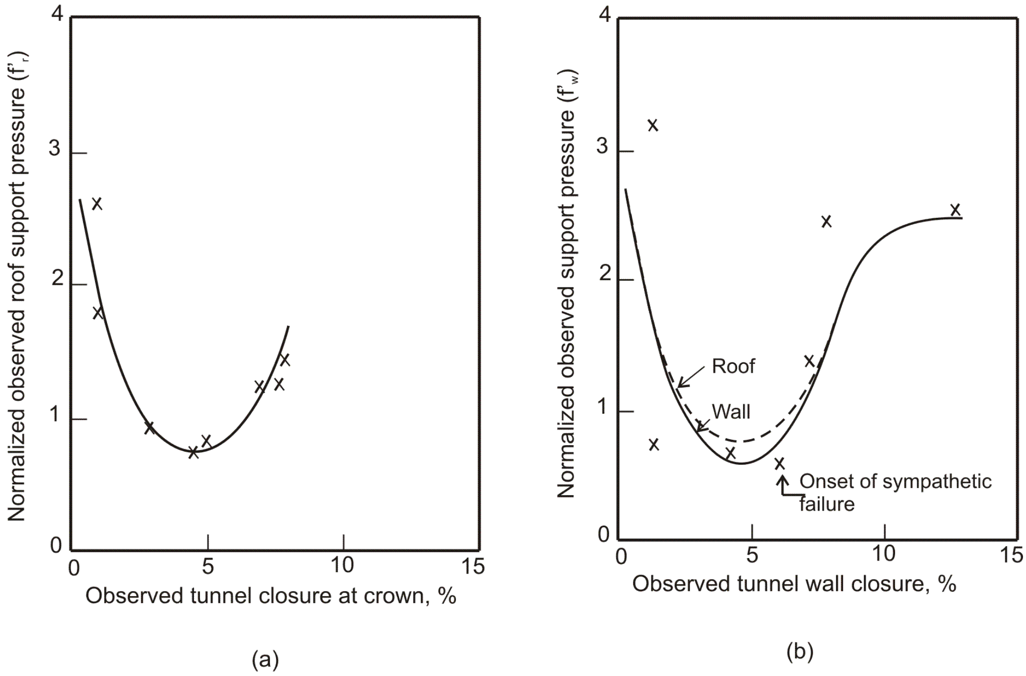

Fig. 2 - Correction factor for (a) roof closure and (b) wall closure under squeezing ground condition (Singh et al., 1992)

The above approach is a general one and not specifically for squeezing or non-squeezing conditions. It does not include size and shape of tunnel which also play very important role as these influence the degree of anisotropy, especially in poor rock mass.

Mehrotra (1992) conducted laboratory tests for mechanical properties of intact rock and analysed the plate load data obtained from various tunnels in lower Himalayas (LH) and concluded that the saturation decreases the value of deformation modulus significantly i.e. by 90% in poor and 75% in fair rock masses. In addition to this, the range of modulus values in LH region (0.75-2.4GPa for poor and 2.4-7.5GPa for fair rock mass) was also suggested.

Verman (1993) determined the ground and support reaction curves from the data of instrumented tunnels of Himalayan region and developed correlation using RMR for estimation of deformation modulus of rock mass. Correlations were also proposed for estimation of short-term support pressures in tunnels (Eq.5).

$$P_{if} = \frac{1-[(1-e)-(b_f/a)^2 e-2(b/a)u_b+(u_b/a)^2]^{1/2}-(u_{ao}/a)}{(S.A/A_s.E_s)+(0.86\,a^{1.05}/t_b.E_{bf})} \tag{5}$$

where

$$u_b = \frac{(1+\nu)}{RF.E_{min}}\,[p_o sin\phi_p + c_p cos\phi_p]$$

P_{if} = short-term support pressure,
E_{bf} = modulus of elasticity of backfill at support pressure of P_{if},
E_s = modulus of elasticity of steel,
A = cross sectional area of tunnel,
S = spacing of steel ribs from centre to centre,
A_s = cross sectional area of steel rib,
u_{ao} = initial radial tunnel closure before installation of support,
u_b = radial displacement of elastic-plastic boundary,
e = coefficient of volumetric expansion of for failed rock mass,
a = radius of tunnel,
b = radius of elastic broken zone,
b_f = radius of fractured broken zone,
t_b = thickness of backfill,
ν = Poisson's ratio of rock mass,
p_o = hydrostatic in situ stress,
RF = reduction factor,
Φ_p = peak angle of internal friction of rock mass in elastic zone, and
E_{min} = smaller of two moduli of deformation of rock mass in horizontal and vertical directions.

Goel (1994) developed an empirical approach based on the rock mass number *N*, defined as Q with *SRF* = 1. *N* was used to avoid the problems and uncertainties in obtaining the correct rating of parameter *SRF* in *Q* method. Considering the overburden depth *H*, the tunnel span or diameter *B*, a log-log plot between *N* and $HB^{0.1}$ was made using rock mass number *N* from 99 tunnel sections. Out of 99 tunnel section data, 39 data were taken from Barton's case histories and 60 from Indian projects. Out of those 60 data, 38 data were from 5 projects in Himalayan region. All the 27 squeezing tunnel sections were observed in those 5 projects in Himalayan region. Other 72 data sets were from non-squeezing sections. Line AB in the plot (Fig. 3) distinguishes the squeezing and non-squeezing cases. The data points lying above the line represent squeezing conditions, whereas points lying below the line represent non-squeezing conditions. Empirical approaches were also developed to estimate support pressure and closure in tunnels under squeezing conditions.

As shown in the Figure 3, a line AB separates the squeezing and non-squeezing cases. The equation of this line is as following:

$$H = (275\, N^{0.33})\, B^{-0.1} \quad (6)$$

where

H = height of overburden above tunnel or tunnel depth, m,
N = rock mass number (rock mass quality Q with SRF=1) and
B = width of tunnel, m.

The data points lying above the line represent squeezing conditions, whereas points below the line represent non-squeezing conditions.

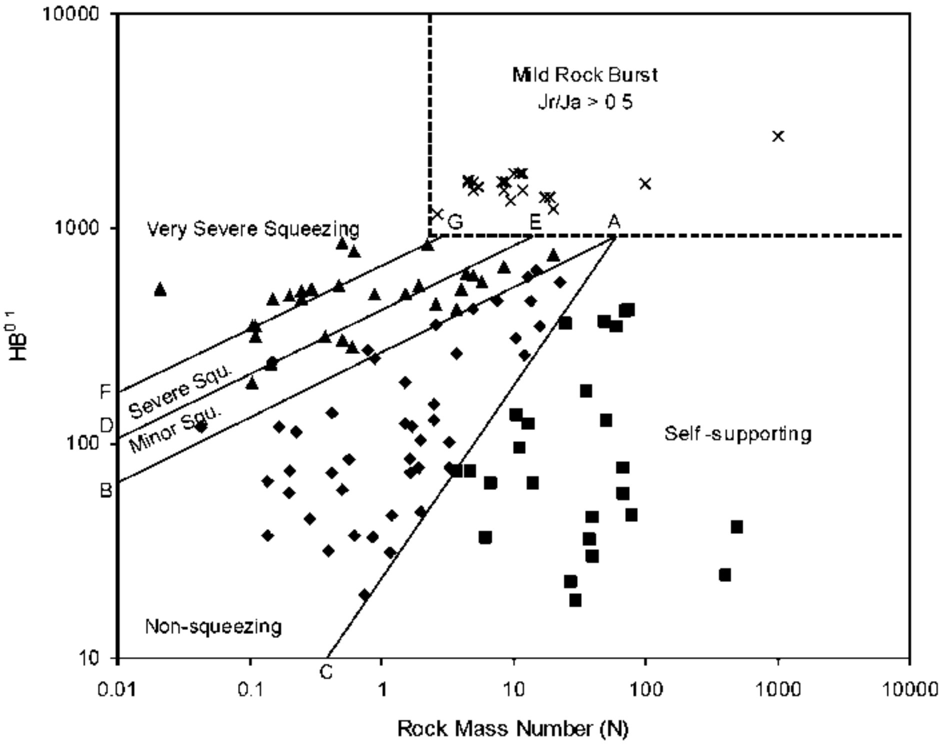

Fig. 3 - Prediction of squeezing ground condition (Singh and Goel, 2011)

Goel (1994) also concluded from the study that the effect of tunnel size and height of overburden is less in non-squeezing conditions, but, it is significant in squeezing conditions. In addition, he suggested the following empirical correlations for prediction of ultimate support pressure and radial closure for squeezing ground condition:

$$p = \left(\frac{f}{30}\right) 10^{\frac{H^{0.6} a^{0.1}}{50 N^{0.33}}} \tag{7}$$

$$u_a = \frac{1}{10.5} \frac{a^{1.12} H^{0.81}}{N^{0.27} K^{0.62}} \tag{8}$$

where

p = ultimate support pressure in squeezing ground conditions, MPa,
f = correction factor for closure (from Fig.4),
H = depth of tunnel, m
a = radius of tunnel, m
N = rock mass number,
u_a = radial tunnel closure (cm), and
K = effective support stiffness (MPa).

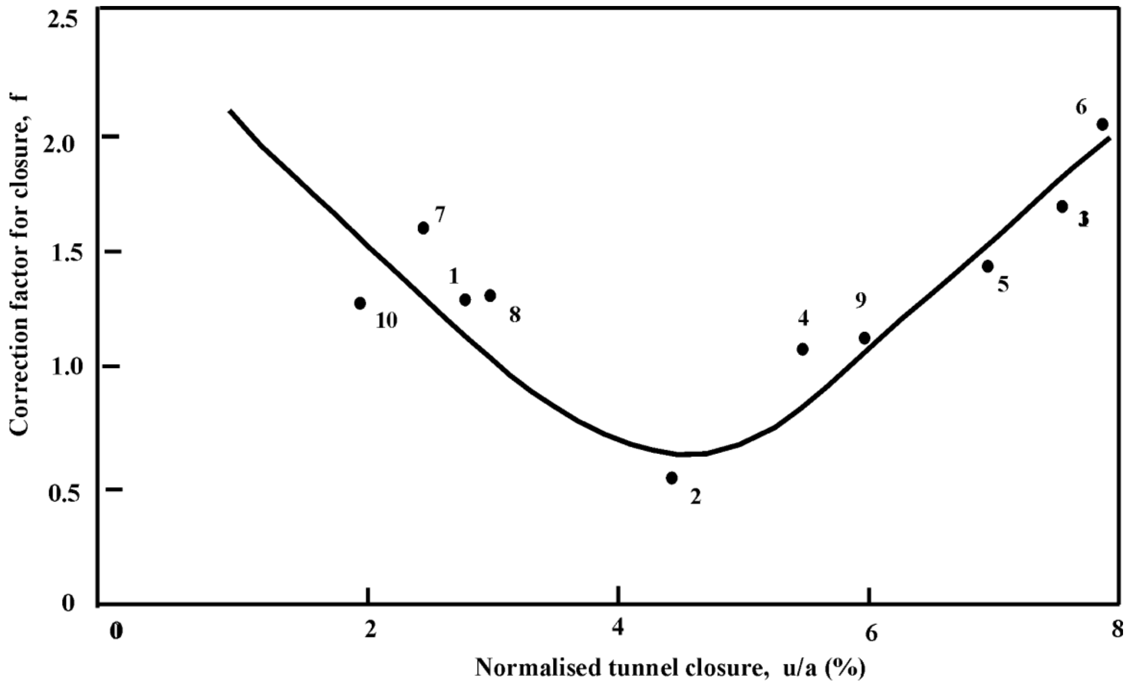

Fig. 4 - Correction factor for tunnel closure (Goel, 1994)

Bhasin and Grimstad (1996) proposed following equation to estimate the support pressure.

$$p = \frac{0.4}{J_r} B . Q^{-1/3} f . f' . f'' \text{MPa} , \quad \text{for } Q < 4 \tag{9}$$

where, B is span or diameter of the tunnel in meteres and rest symbols are same as used in Eq. 4.

Chern et al. (1998) showed that, for tunnels constructed in Taiwan, problems with tunnel stability occurred when the 'strain' exceeded about 1% (in Shrestha, 2005).

Kumar (2002) studied various existing rock mass classification systems and concluded following points: (i) Q was not reliable for squeezing ground conditions, (ii) Support pressure prediction by Unal (1983) were unsafe for squeezing conditions, (iii) RSR overestimated support requirement in non-squeezing conditions, (v) RMR was unsafe for both non-squeezing and squeezing conditions, (vi) RMi highly over-estimated the rock pressure. It was also suggested that in over stressed conditions, if $J_r/J_a \geq 0.5$, rock burst was observed otherwise squeezing occurred.

Shrestha (2005) evaluated the required supports using various empirical approaches for Khimti-I and Melamchi tunnels in Nepal. It was concluded that there was good agreement on support pressures and closures in non-squeezing conditions but not in squeezing conditions. In the analysis of squeezing behaviour of the Khimti tunnel, valley-side effect of the topography has been observed. The valley side slope was 22°. This effect was not considered in any of the available squeezing prediction criteria. It was recommended for further study to correlate the valley side slope and maximum topographical height with the stress increase in the tunnel. Moreover, on observation of the strong effect of rock mass strength on squeezing behaviour, it was suggested to include rock mass strength as a parameter in approaches for prediction of squeezing behaviour of ground.

Viladkar et al. (2008) (Part-I & II) suggested an approach for determination of ground reaction curve for non-squeezing and squeezing ground conditions on the basis of analyzing field instrumentation data of nine different tunnelling projects in India and observed the dependency of deformation modulus of poor rock on support pressure and reduction in support pressure by intermediate in situ stress along the tunnel length. An approach was also suggested to determine stiffness of backfill between rib support and rock.

2.3 Semi-Empirical Approaches

Following semi-empirical approaches have been proposed for estimation of the tunnel closures and support pressures.

Detourney and Fairhurst (1987) proposed a semi-empirical elasto-plastic model for a long cylindrical tunnel like cavity to obtain an explicit solution for stresses and deformations under non-hydrostatic stress field. A significant prediction, based on the model, is that the direction of maximum convergence becomes perpendicular to the direction of the maximum in situ compressive stress if the rock failed is large enough. This provides the possible explanation of large deformation in tunnels driven through squeezing ground condition having high vertical stress.

Aydan et al. (1993) developed correlations amongst strains (elastic, plastic, squeezing, and rupture) and uniaxial compressive strength. This approach is based on analogy between the axial stress-strain response of rocks in laboratory tests and tangential stress-strain response of rocks surrounding the tunnels.

On the basis of experience gained with tunnels in Japan, Aydan et al., (1993) proposed the following correlations between uniaxial compressive strength of the intact rock (σ_{ci}) in MPa and strain levels:

$$\eta_p = \frac{\varepsilon_p}{\varepsilon_e} = 2\,\sigma_{ci}^{-0.17}\,,\ \eta_s = \frac{\varepsilon_s}{\varepsilon_e} = 2\,\sigma_{ci}^{-0.25},\ \eta_f = \frac{\varepsilon_f}{\varepsilon_e} = 2\,\sigma_{ci}^{-0.32} \tag{10}$$

where η_p, η_s and η_f are normalized strain levels and other strain levels are defined in Fig. 5. Values of the strain at different conditions are calculated using the following relations:

$$\varepsilon_e = \frac{1+\nu}{E}\,(p_o - p_i) \tag{11}$$

$$\varepsilon_p = \frac{1+\nu}{E}\,(p_o - p_i)\,\frac{R_{pp}^{f+1}}{a} \tag{12}$$

$$\varepsilon_{sf} = \frac{1+\nu}{E}\,(p_o - p_i)\,\eta_{sf}\,\frac{R_{pb}^{f^*+1}}{a} \tag{13}$$

$$\frac{\varepsilon_p}{\varepsilon_e} = f(q, \beta, \alpha, f) \tag{14}$$

$$\frac{\varepsilon_{sf}}{\varepsilon_e} = f(\eta_{sf}, q, \beta, \alpha, f, q^*, \alpha^*, f^*) \tag{15}$$

where p_o = overburden pressure (hydrostatic condition is assumed), p_i = support pressure, R_{pp} = radius of perfect plastic region (the region after residual plastic region till elasto-plastic boundary), R_{pb} = radius of residual plastic region (upto some distance from tunnel boundary, a = radius of opening, $\eta_{sf} = (\eta_s+\eta_f)/2$, $\varepsilon_{sf} = (\varepsilon_s+\varepsilon_f)/2$, f =ratio of radial to axial strain with ν for perfect plastic part, $\beta = p_i/p_o$, $\alpha = \sigma_{ci}/p_o$, $q^* = (1+\sin\phi^*)/(1-\sin\phi^*)$, * = relates the respective values for plastic condition or failed rock mass.

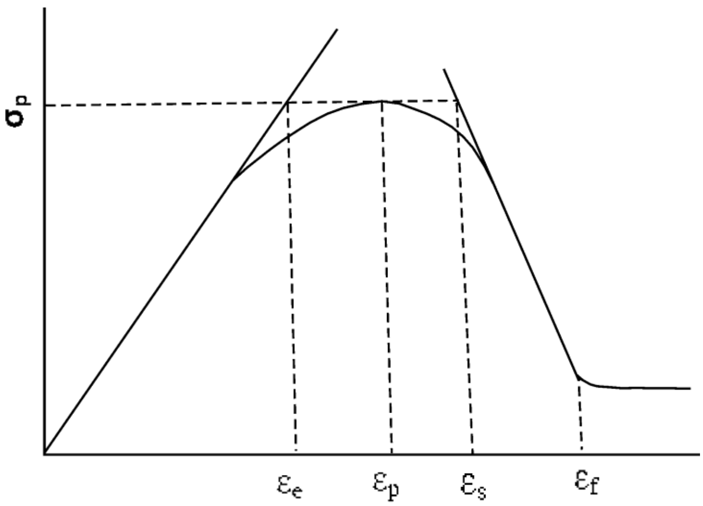

Fig. 5 - Idealized stress-strain curves (Aydan et al., 1993)

Equations 13 and 14 are used to estimate the strain ratio and then degree of squeezing is found by comparing them with the values calculated from Eq. 9. If squeezing is predicted, then support (p_i) will be provided. In addition to σ_{ci}, this method requires laboratory tests to find out Poisson's ratio for perfect plastic and residual plastic conditions and friction angle for intact and failed rock masses. The fundamental concept of the Aydan et al. (1993) approach is based on the analogy between the axial stress-strain response of rocks in laboratory tests and tangential stress-strain response of rocks surrounding tunnels. It considers $\sigma_1 = \sigma_\theta$ and $\sigma_3 = \sigma_r = \sigma_{pi}$. Figure 6 shows the boundary rock conditions in squeezing tunnels.

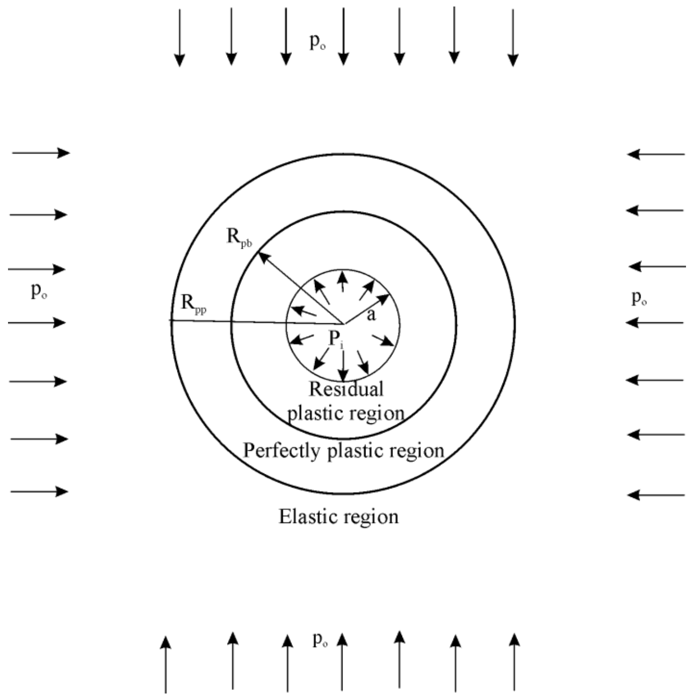

Fig. 6 - Boundary rock conditions in squeezing tunnel (Aydan et al., 1993)

Kovari (1998) developed an approach for circular openings and assuming isotropic, homogenous and elasto-plastic material behaviour. An approach was developed for displacement at the boundary of the excavated opening, for a given displacement at the boundary of the plastic zone.

A few semi-analytical approaches have been proposed for estimation of the deformation caused by squeezing and estimation of support pressure required in the squeezing tunnel. These are discussed and compared in the following sections.

Following equation was suggested by Kovari (1998) for the displacement u_a at the boundary of the excavated opening, for a given displacement u_ρ at the boundary of the plastic zone (broken zone).

$$u_a = u_\rho \left(\frac{\rho}{a}\right)^k \tag{16}$$

where ρ and a are radii of plastic zone and the excavated opening respectively. Volume change is taken into account using the parameter k. Its value varies between 1 and (1+sinϕ)/ (1-sinϕ). Value of 'k' is evaluated in reference to ρ/a ratio. Following equations were given to calculate ρ and stresses:

$$\frac{\rho}{a} = \left[(1 - sin\phi)\frac{p'_\alpha}{p'_a}\right]^{\frac{1-sin\phi}{2sin\phi}} \quad (17)$$

$$p'_\alpha = p_\alpha + c\ cot\phi \quad (18)$$

$$p'_a = p_a + c\ cot\phi \quad (19)$$

Above equation shows that $u_a = f(p_a, p_\alpha, c, \phi, a, k)$

where

p_α = vertical in situ stress,
p_a = stress on the lining,
c = cohesion, and
ϕ = angle of internal function.

Hoek and Marinos (2000) suggested that a plot of tunnel strain (ε) against the ratio of uniaxial compressive strength to hydrostatic in situ stress could be used effectively to assess tunnelling problems under squeezing conditions (Eq. 20). Hoek and Brown's criterion for estimating the strength and deformation characteristics of rock masses assume that rock mass behaves isotropically. Highly fractured rock mass also behaves isotropically therefore, this criterion can also be applied to weak heterogeneous rock masses too (Eq. 21).

$$\varepsilon = \left[0.002 - 0.0025\ \frac{p_i}{p_o}\right]\left(\frac{\sigma_{cm}}{p_o}\right)^{\left(24\frac{p_i}{p_o} - 2\right)} \quad (20)$$

$$\sigma_{cm} = (0.0034\ m_i^{0.8})\ \sigma_{ci}\left\{1.029 + 0.025\ e^{(-0.1m_i)}\right\}^{GSI} \quad (21)$$

where

ε = closure strain,
p_i = internal support pressure (MPa),
p_o = overburden pressure (γH),
σ_{cm} = uniaxial compressive strength (MPa),
m_i = a constant depending on the frictional characteristics of rock material,
GSI = geological strength index.

For unsupported condition, value of support p_i is zero. The value of p_i is increased till an acceptable value of strain to obtain appropriate value of support from Equation 19.

This analysis is a simple closed-form solution which assumes circular shape with hydrostatic stress field condition and proper contact of support throughout the periphery. These assumed conditions are seldom met in the field especially in the tunnels being excavated by drill & blast method. So, the predictions made by the approach may not be reliable.

All the above mentioned three semi-empirical approaches consider a circular opening in homogeneous rock material with a hydrostatic stress state to estimate squeezing deformation. Hoek and Marinos's and Aydan et al.'s approaches also consider the same condition for the estimation of supports, whereas Kovari's approach can also accommodate anisotropic stress conditions. The approaches consider instantaneous squeezing deformation.

Singh et al. (2007) tested jointed specimens in the laboratory and came out with following approach for prediction of squeezing ground condition:

$\frac{\varepsilon_\theta^a}{\varepsilon_\theta^e} > 1$ where $\varepsilon_\theta^a (= u_a/a)$ is the peak tangential strain at the periphery of the tunnel and ε_θ^e is the elastic strain. However, the critical strain was a function of rock mass properties and not a level of 1% as suggested by Sakurai (1997).

2.4 Theoretical Approaches

Panet (1975) assumed that the rock mass remains elastic initially but suffers lot of strength failure and undergoes volume increase on account of failure when an opening is made. He concluded that a tunnel may remain stable if the residual strength of the rock mass in close proximity of the tunnel periphery is not destroyed fully, say by using rock bolts.

Fairhurst (1976) emphasized that it would be more rational to design underground structures in squeezing ground conditions on the basis of stability concepts of tunnel mechanics. Thus, a support system may be allowed to underground plastic deformations so long as the post failure capacity of the support system is greater than the pressure acting on it.

Lee and Lo (1976) suggested that ground squeezes due to long-term recovery of strain energy.

Kaiser (1980, 81) recognized the necessity of considering the effect of the loading history on the rock mass response in stress analysis around underground openings. He emphasized to use different elastic constants for the elastic and broken zones and suggested that the modulus reduction associated with progressive failure of the rock mass can alone account for observed tunnel closures.

Panet and Guenot (1982) studied the effect of face advance on tunnel closure and suggested that 90% of the tunnel closure occurred when the face is 1.8 to 3.7 times the tunnel radius away from the location where the closure has to be estimated. This is applicable when the radius of broken zone is 1 to 2 times the radius of an opening. He further added the time dependent closure can be evaluated by monitoring the tunnel closure at a distance of more than 1 to 2 times the diameter from the tunnel face.

Lu (1986) used modified Mohr-Coulomb yield criterion to point out that consideration of the strain hardening behaviour results in a higher tangential stress at the tunnel periphery, and a smaller radius of broken zone.

Sulem et al. (1987) suggested a convergence law and differentiated between the effect of face advance and the time dependent behaviour of the rock mass on tunnel convergence. The authors used two case histories of Frejus tunnel between France and Italy and the Las-Planas tunnel in the south of France for analysis of convergence.

Corbetta et al. (1991) developed a method to find the effect of distance of support from the tunnel face on tunnel convergence to use convergence-confinement for elastic-perfectly plastic ground. The support pressure and convergence were evaluated considering plasticity of the ground.

Daemen (1975) developed a closed form solution and a numerical method for estimation of support pressure in circular tunnels under squeezing ground conditions. The developed closed form solution is comprised of following parameters: peak angle of internal friction of intact rock applicable to the elastic zone, residual angle of internal friction of failing rock mass applicable to the broken zone, peak cohesion of intact rock applicable to the elastic zone, and residual cohesion of failing rock mass applicable to the broken zone. The closed form expression is based on the assumption that the in situ stresses are hydrostatic and gravity acts towards the centre of the tunnel so that the problem becomes axisymmetric.

It was commented that the concept of constant volume increase throughout the broken zone (as assumed by Labasse, 1949) was an over simplification. Instead, he suggested that these displacements were due to elastic relaxation of the broken zone which has a lower modulus as compared to that of the rock mass in elastic zone. Further, it was suggested that the volumetric expansion ratio (k) ranges between 0.01 and 0.05 which are one order of magnitude lower than those proposed by Labasse (1949).

In another study, Daemen (1975) used a strain-softening dialatant continuum model to incorporate the effect of face advance on support pressure and concluded that the stiffer support mobilizes higher support pressure. Further, supports installed close to the face attract higher pressure.

Dube (1979) modified the closed form solution proposed by Daemen (1975) to obtain short-term vertical and horizontal support pressures in non hydrostatic primitive stress field.

Jethwa (1981) modified the closed form solution given by Daemen (1975) by incorporating the three dimensional effect considering the influence of face advance and shear stresses across the tunnel axis for obtaining the short-term tunnel support pressure.

Further, on the basis of field observations, it was suggested that k-value varies between 0.003 and 0.01 for commonly occurring soft rock masses.

Fritz (1984) assumed that the plastic zone behaviour is governed mainly by the properties of plastic St. Venant element (as modified by Fritz, 1982) and conducted elasto-plastic analysis. The initial deformation was characterized by the residual strength. Modified Mohr-Coulomb criterion, characterizing both the peak and the residual strengths, was used to represent the rock mass behaviour.

Sharma (1985) developed an approach to estimate the tunnel closure for good rock masses with high overburden considering five parameters viz., yield strength of rock mass, support pressure, cover pressure, joint frequency (number of joints per metre), modulus of elasticity of intact rock and average joint stiffness.

Stille et al. (1989) and Indraratna and Kaiser (1990) proposed closed form elasto-plastic solutions for underground openings supported with rock bolts. Using modified Mohr-Coulomb failure criterion and non-associated flow rule, the ground reaction curves for the rock mass after installation of the grouted rock bolts were obtained.

2.5 Numerical Approaches

Gioda and Cividini (1996) carried out numerical modelling using finite element method studied time dependent behaviour of rock mass in squeezing and swelling conditions. He found that rock salt also exhibits the time dependent behaviour.

Shalabi (2005) investigated movement and pressure on lining of still-water tunnel (Utah, USA). Axisymmetric finite element analysis was conducted using power law and hyperbolic creep models for modelling of squeezing ground to show the differences between the results obtained from each model. The conclusion of the study was that lining pressure and deformation can be predicted using power law creep model, if the delay time before lining-erection is considered.

Shrestha (2005) carried out numerical modelling for Khimti-1 and Melamchi hydro tunnels of Nepal and recommended numerical modelling to supplement analytical calculations for recognizing critical stress situation and deformation magnitude for big and non circular tunnels.

Sitharam et al. (2005) developed a FISH program to incorporate joint factor which is the integration of the properties of joints to care of the effects of joint frequency, orientation and strength of joints to be used for modelling jointed rocks by FLAC-3D using Duncan and Chang 91970) hyperbolic model. The settlement observations reported from the field studies carried out in the Nathpa Jhakri power house cavern in India were compared with the predicted observations from the 3-D numerical analysis and found the model suitable for analysis for both single and multiple joints (in non-squeezing conditions).

Bhasin et al. (2006) conducted numerical modelling using 2-D plastic finite element program and concluded that support pressure increases significantly with tunnel size in an elastic-plastic rock mass. The study showed that maximum axial force on shotcrete lining doubles when diameter of tunnel is increased from 5m to 20m. However, the effect of tunnel size on support pressure is very small in case of elastic rock. Further, he also modified the empirical approach of Barton et al. (1974) by introducing diameter of tunnel as a new parameter.

Lian-chong et al. (2008) analysed the closure and failure behaviour of tunnels using Rock Failure Process analysis (RFPA2D) for numerical modelling and concluded that initiation of creep failure is governed by the ratio of the far field stresses (k). He further suggests that the creep failure initiates always in the direction of the minimum far field stress component since in that direction the octahedral shear stress reaches the highest value. In the case for $k \neq 1$, the rock is more unstable as compared to the case for $k=1$, where k is the ratio of horizontal to vertical in situ stress.

2.6 Physical Modelling

Analysis of closure behaviour of tunnels essentially involves analysis of strength and deformational behaviour of jointed rock masses under a given stress environment. A good understanding of jointed rocks under uniaxial, biaxial, triaxial and polyaxial conditions before and after failure is therefore very important.

Physical modelling has been used as one of the most effective way to study the engineering problem of jointed rocks and rock masses. Some notable studies carried out in the past have been as following.

Patton (1966) conducted laboratory tests to study mechanisms of different modes of failure and their effect on shear strength along rough joints. The first hypothesis for strength of rough joints was developed on the basis study of more than 300 stable and failed rock slopes. The given model is valid for shearing along a regularly indented rock surface in which at failure, the teeth have the same geometry and the degree of interlocking as at the beginning of shearing. These assumptions are hypothetical and are not satisfied in reality.

Goldstein et al. (1966) suggested that uniaxial compressive strength, UCS (σ_{cj}) of rock mass depends on the relative size and shape of separate blocks for load acting normally to the joints.

Brown (1970 a & b) and Brown and Trollope (1970) tested specimens of jointed block mass under unconfined and confined states. The specimens were formed out of cubical elemental blocks (2.5cm side), parallelopiped (height: 2.03cm, length: 3.18cm) and hexagonal (1.59cm side) shapes. Various combination of failure modes, splitting, shearing and sliding were observed during failure of rock specimens.

Walker (1971) and Lama (1974) observed an asymptotic variation in strength of rock mass and found that asymptotic value reached for 5 to 6 joints in the case of horizontal joints. The reduction in strength was observed to be 50% and 30% respectively. Walker (1971) reported that asymptotic value of strength reached only for 2-3 joints in the case of vertical joints. Further, Lama (1974) showed by study that σ_{cj} and deformation modulus of rock mass (E_j) reached to their minimum value, if the blocky mass contained at least 150 elements.

Ladanyi and Archambalut (1972) simulated behaviour of rock mass with two sets of orthogonal joint sets by conducting biaxial tests on large sized specimens of blocky mass. The elemental square sized blocks were cut from commercial concrete bricks. The modes of failure were found to be dependent on orientation of principal discontinuities and value of confining pressure.

Einstein and Hirschfield (1973) conducted tests on jointed block mass to study the effect of joint orientation, joint spacing and number of joint sets on the strength response of jointed mass. For higher values of confining pressure, the shearing was observed along several roughly parallel surfaces along with increase in plastic flow. The transition between sliding and fracturing was found to coincide with brittle ductile-transition.

Yaji (1984) studied the effect of roughness and inclination of joints on the response of jonted cylindrical specimens of plaster of Paris, sandstone and granite. Following conclusions twere drawn; (i) mode of failure changes with β of the joint plane, (ii) UCS of jointed rocks was minimum when β was between 30^o to 45^o, (iii) at higher confining pressure, the mode of failure changes from splitting and slabbing to shearing along a shear plane, ignoring the presence of joints, and (iv) cohesion of jointed rocks follows the behaviour of σ_{cj}.

Arora (1987) conducted UCS tests on jointed specimens of plaster of paris, Jamrani sandstone and Agra sandstone with different orientation of joints and values of number of joint per metre (J_n) and reported that reduction in strength for different rock was of similar order for same number of joints. Modulus was also observed to behave in the same way. Anisotropy in the strength behaviour due to a single joint was also observed in the tested specimens. The minimum strength for the rock types tested was found to be 30^o, 40^o and 30^o respectively.

Roy (1993) reported strength anisotropy for cylindrical specimens having one close joint or single joint filled with two types of gauge materials. The minimum strength was obtained at $\beta=34^o$.

Ramamurthy and Arora (1994) conducted about 250 uniaxial compressive strength tests and 1300 triaxial tests on jointed and intact specimens made of plaster of Paris, Jamrani sandstone and Agra sandstone prepared in the laboratory. Based on this extensive experimentation, a joint factor (J_f) has been evolved to account for the number of joints per meter length (J_n), inclination parameter for the sliding joint (n) and the shear strength along this joint (r).The joint factor takes into account anisotropy of rock mass strength realistically.

Vutukuri et al. (1995) conducted study on smooth (sandstone) and rough (coal) joints and observed that minimum strength occurred between $\beta = 30^o$ to 45^o for smooth joints, whereas for rough joints the minimum strength occurred at 30°. Joint factor takes into account anisotropy of rock mass strength realistically.

Singh (1997) conducted laboratory testing of block models made of sand-lime brick having 6 elemental blocks in each direction (total about 260 elemental blocks) to overcome the scale effect. Following conclusions were drawn; (i) specimens failed due to shearing, splitting or combination of both for horizontal or vertical continuous joints, (ii) for upto dip of 30°, the mode of failure depends on interlocking introduced by stepping and for low/no stepping, mode shifts towards shearing and splitting, (iv) for dip of 50-60°, there is no effect of stepping and specimen fails in sliding only.

Tiwari and Rao (2004), on the basis of experimental studies, concluded that intermediate principal stress significantly enhances the strength of rock mass. Thus there is a need for considering the effect of intermediate principal stress along the tunnel axis in the elasto-plastic-brittle-failure analysis around openings under non-hydrostatic in situ stress conditions.

3. DISCUSSIONS

Observational approaches enhance the experience and provide a direction for research but cannot give an optimal solution. Semi-empirical approaches still contain some assumed parameters and therefore do not take real inputs leading to inappropriate solutions.

Mathematical models (closed form solutions) have many restrictions and assumed parameters like hydrostatic in situ stresses, circular opening etc. Hence, these are unable in providing good solutions to tunnels of different shapes in the field.

Numerical modelling results are case specific and cannot be generalized for all the cases. Furthermore, there is no simple method available for modelling the rock masses as these contain natural discontinuities of varying size, strength and orientation. In practice, it is almost impossible to explore all of the joint systems or to investigate all their mechanical properties and implementing them explicitly in a theoretical model. Further, getting undisturbed samples from the field for experimental study is very difficult.

Since long, many research workers have been trying to characterize the rock mass and giving solution to the rock pressure problems in tunnels and other underground openings. The research workers of the above studies agreed upon a common point that none of the existing rock mass classification approaches has been able to reliably predict the support pressure in tunnels under squeezing ground conditions. Barton’s Q-system predicts the support pressures reliably for tunnels under non-squeezing ground conditions but not for squeezing ground conditions. On the other hand, analytical approaches need values of strength parameters and in situ stresses as input, which are very difficult to assess and are time consuming also. In situ stresses are very important parameters but these are either assumed as hydrostatic or only vertical stress is considered. Further, size of tunnel matters a lot with regard to support pressure and closure in case of squeezing rock conditions (Goel et al., 1996; Bhasin et al., 2006) whereas according to Barton et al. (1974) and Jethwa (1981), there is no effect of tunnel size on support pressure.

An empirical approach based on RMR proposed by Unal (1983) predicts entirely unsafe support pressure for tunnel in squeezing conditions (Goel and Jethwa, 1991). Moreover, similar approach based on Bartons' Q-value suggested by Bhasin and Grimstad (1996) gives higher value of support pressure for tunnels greater than 5m of diameter (Goel et al., 1995 and Singh et al., 1992). On the basis of review of the above literatures, it can be concluded that the existing empirical and analytical approaches/correlations for prediction of support pressure and closure/deformation in tunnels involve number of assumptions. In addition to this, these correlations involve numerous parameters, determination of which is sometimes a difficult task. For instance, empirical correlation developed by Verman (1993) for prediction of short-term support pressure involves 18 parameters.

Closure behaviour of tunnels in squeezing ground is essentially a problem involving strength and deformational behaviour of jointed rock masses under a given stress environment.

A reliable empirical approach having less number of parameters which can be easily assessed in the field can become a handy tool to solve the support pressure and closure problems.

4. CONCLUSIONS

In view of the above discussed gaps in the previous studies, there is a need to develop a user friendly approach to predict support pressure and closure in tunnels under squeezing ground conditions. Involvement of easily assessable geo-mechanical parameters would make the approach user friendly. Development of semi-empirical approaches involving easily assessable geo-mechanical parameters for prediction of support pressure and closure in underground openings under squeezing condition are urgently required specially for tunnelling in Himalayan region which is highly tectonically active and squeezing problem has been frequently faced by various geologist and constructional engineers engaged in tunnelling in the region.

The classification approaches (*Q*, *RMR* and *N*) are purely empirical. On the other hand, concept of joint factor (J_f) developed by Ramamurthy and co-workers (Arora, 1987; Ramamurthy, 1993 & 2004; Ramamurthy and Arora, 1994) has been derived through extensive experimental studies in the laboratory. Moreover, this concept involves few parameters (only three) which can be easily assessed in the field and hence may be used for development of new approach to assess ground condition, support pressure and tunnel closure as it accounts for anisotropy of rock mass strength. In addition to this, shape of opening also plays an important role especially in elasto-plastic ground conditions. Therefore, shape of the underground opening may also be included for making the correlations more generalized.

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