Rock Mass – Tunnel Support Interaction Analysis: Part I - Ground Response Curves

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ABSTRACT

The application of rock mass – tunnel support interaction analysis in designing the tunnel support system is well known. The interaction analysis includes the prediction of both ground response and the support reaction curves. In this Part-I of the paper, an approach has been proposed for quick and reliable determination of the ground response curve, both for self supporting / non-squeezing ground condition and the squeezing ground condition. The dominating influence of the intermediate principal stress has been accounted for in the analysis. The approach has been proposed on the basis of field studies conducted in nine different tunnelling projects in India and the analysis of field instrumentation data. Using this field data, correlations have also been proposed for predicting the tunnelling conditions, modulus of deformation of rock mass, influence of depth on modulus of deformation and the apparent strength enhancement.

Keywords: Rock mass; Tunnel support interaction; Ground response curve.

1. INTRODUCTION

Morrison and Coates (1955) were the first to assume the reduced strength of rock mass in plastic zone around the tunnel and used an elastic-brittle plastic model for prediction of displacements and stresses around its periphery. Later, several authors considered an elastic-strain softening behavior of rock mass using a tri-linear stress-strain law (Diest, 1967; Daemen and Fairhurst, 1971; Hendron and Aiyer, 1972; Egger, 1974; Panet, 1976; Korbin, 1976; Brown et al., 1983 and Sharma, 1985). Fritz (1984) assumed that rock mass behavior in plastic zone is primarily governed by the properties of the plastic St. Venant element.

Brown et al. (1989) proposed solutions for stresses and strains around an axi-symmetric excavation in an infinite media considering the power law and exponential variation of elastic modulus with minor principal stress. Histake et al. (1989) considered peak and residual strength criteria and non-linear stress-strain relations which change with confining pressure. Carter and Booker (1990) studied the influence of the rate of excavation on stress distribution around circular tunnels and concluded that rapid excavation may result in a significant change in short-term stress distribution.

Early solutions proposed by Morrison and Coates (1955), Hobbs (1966), Bray (1967), and Diest (1967) did not include any treatment of plastic volumetric strains, although some of them allowed for a strength reduction in the plastic zone. Labasse (1949), however, evaluated an average plastic dilation in the rock mass. The concept was later used by others including Lombardi (1970), Daemen and Fairhurst (1971), Ladanyi (1974) and Jethwa (1981). Influence of parameters like face advance and shear stress on support pressure were studied by Jethwa (1981) who modified Daeman's (1975) equation for short-term support pressure to include these effects.

Convergence confinement method of tunnel design, which is based on rock mass – tunnel support interaction concept was discussed in detail by Gesta et al. (1980), Duddeck (1980) and Lombardi (1970). Based on a comparison of analytical results with field measurements, Eisenstein and Branco (1991) concluded that while the method is applicable to deep tunnels, it is not suitable for shallow tunnels due to non-axi-symmetric mode of deformation and development of plasticity in the latter. Corbetta et al. (1991) included the effect of distance from the tunnel face at the time of support installation in convergence confinement method and applied it to an elastic – perfectly plastic ground.

2. STATEMENT OF PROBLEM

Prediction of ground response curves in elastic and squeezing grounds, using the approaches discussed earlier, depends upon a number of input parameters some of which are difficult to estimate, thus affecting the reliability of analytical results. Limited studies are available as regards cross-checking of theoretical results with field observations. Mohr-Coulomb theory is not valid for anisotropic and jointed rock masses and in-situ stress along the tunnel axis (intermediate principal stress) may reduce support pressures drastically. There is, therefore, a need to develop a simple, yet reliable approach for the prediction of ground response curves directly from the data of instrumented tunnels.

3. FIELD INSTRUMENTATION AND MONITORING OF TUNNELS

Monitoring of the rock mass behavior by field instrumentation is the backbone of the observational method of tunnel support design based on “Design As You Go” philosophy of NATM. The tunnel instrumentation formed an important part of a comprehensive field study carried out at nine tunnelling project sites in India with a view to develop an approach for determination of ground response and support reaction curves. The field study comprised of:

(i) instrumentation to measure support pressures and tunnel deformations,
(ii) estimation of Barton’s (1974) rock mass quality, Q and Bieniawski’s (1979) rock mass rating, RMR at instrumented and other tunnel sections,
(iii) collection of other relevant data such as type of rock mass, modulus of deformation, uni-axial compressive strength etc. wherever available,
(iv) collection of geometric details of tunnels such as direction of tunnel axis, size and shape of tunnel, depth/height of overburden, and
(v) collection of details related to tunnel support systems such as dates of tunnel excavation and support installation for each instrumented section, section of steel ribs used, type and thickness of backfill etc.

This field study was carried out at 63 different tunnel sections of nine tunnelling project sites in India. Out of these, details of about 30 sections of 9 tunnelling projects are presented in Tables 1 and 2 for tunnels excavated in non-squeezing and squeezing ground conditions respectively along with details of various rock types encountered, rock mass quality, Q, depth of overburden, H and the tunnel size.

Table 1 - Tunnel sections in India in non-squeezing ground condition

S. No.	Project	Section chainage (m)	Rock type	Q	Height of overburden (m)	Excavation diameter (m)
1	Tehri Hydro Project, lower Himalaya, Uttar Pradesh	a) 828, HRT-3	Phyllites Grade (Gr)-III	0.36	295	9.5
		b) 829, HRT-3	Phyllites Gr-III	0.36	295	9.5
		c) 683, LBDT-1	Phyllites Gr-I	14	225	13.0
		d) 614, RBDT-1	Phyllites Gr-II with bands of Gr-I	3.2	240	13.0
		e) 615, RBDT-1	Phyllites Gr-II with bands of Gr-I	3.2	240	13.0
2	Maneri Uttrakashi Tunnel, lower Himalaya,Uttarakhand	a) 789.5, u/s Heena	Metabasics with a 1.5 m thick shear zone	0.66 – 0.10	367	5.8
		b) 1060, u/s Heena	Foliated Metabasica	3.4 – 6.8	234	5.8
		c) 738.5, d/s Maneri	Moderately Foliated Quartzites	3 – 6	250	5.8

		d) 1310, u/s Uttarkashi	Foliated Metabasics	3.4 – 6.8	467	5.8
3	Maneri stage-II, lower Himalaya Uttarakhand	a) 1568.5, u/s Dharasu	Greywackes	2.75	100	7.0
		b) 1680.75, u/s Dharasu	Greywackes	1.02	175	7.0
4	Tandsi Mine, M.P.	a) 0.60	Talchirs	11.8	16	5.4
		b) 0.80	Talchirs	32	18	5.4
		c) 0.22	Talchirs	1.1	67	5.4
		d) 0.265	Talchirs	9.07	88	5.4
5	Bagur-Navile Tunnel, Hemavathy Irrigation Project, Karnataka	a) 6380	Schistose Gneiss	0.08	45	6.0
		b) 10678	Schistose Gneiss	13.53	50	6.0
		c) 8695	Schistose Gneiss	13.53	49	6.0
6	Lower Periyar Project, Kerala	a) 2361	Granite-biotite gneiss	4.4	120	6.8
		b) 6218	Granite- biotite gneiss	5.5	197	6.8

Notation: Q – Barton's rock mass quality; HRT – Head race tunnel; LBDT – Left bank diversion tunnel; RBDT – Right bank diversion tunnel; u/s – Upstream; d/s – Downstream

Table 2 - Tunnel sections in India in squeezing ground condition

S. No.	Project	Section chainage (m)	Rock type	Q	Height of overburden (m)	Excavation diameter (m)
1	Maneri stage-II, lower Himalaya, Uttarakhand	a) 50.5 Dhanarigad drift (u/s)	Metabasics	0.88	710	2.5
		b) 51 Dhanarigad drift (u/s)	Metabasics	0.88	710	2.5
		c) 777.2 Dhanarigad drift (u/s)	Crished Quartzites	0.18	705	7.0
2	Giri Hydro Project, lower Himalaya, H.P.	-	Completely crushed phyllites	0.062 – 0.32	240	4.8
		-	Very blocky & seamy slates	0.32 – 0.82	380	4.8
3	Chhibro-Khodri Tunnel, Yamuna Hydro Project, lower Himalaya, Uttarakhand	a) 2575	Red shales	0.025 – 0.1	280	3.0
		b) 2621	Black clays & seamy slates	0.016 – 0.03	280	3.0
		c) 1199	Crushed red shales	0.012 – 0.05	680	9.0
4	Loktak Hydro Project, Manipur	-	Shales	0.011 – 0.044	300	4.8

The main purpose of field work was to study the response of ground upon excavation and support behavior at various tunnel sections, with a view to develop empirical/semi-empirical approach for predicting ground response and support reaction curves. The scheme of instrumentation, therefore, consisted of monitoring these two parameters using : i)mechanical load cells with dial gauge and vibrating wire type electrical load cells having a battery operated portable electrical read-out unit and a digital display, for measuring hoop load in steel ribs, ii)contact pressure cells to measure the contact (support) pressure which were installed before the backfill was placed on the outer flange of steel ribs, iii)tape extensometers / distomat to measure the tunnel closure, and iv)single point and multiple point borehole extensometers (SPBX/MPBX) installed at some of the tunnel sections to monitor deep seated radial displacements in rock mass around the tunnel periphery at different depths.

4. IMPORTANCE OF UNRECORDED DATA

It is often not possible to commence the measurement of tunnel closure and support pressures with time immediately after excavation as some time is always involved in mucking-out operation, installation of supports and instrumentation. The unrecorded tunnel closure data lost during this period can be very significant as it influences the observed support reaction curve and therefore the equilibrium conditions of the tunnel. This unrecorded data could be obtained by: i)plotting the recorded radial tunnel closure with time on log-log scale, ii)extending the initial straight line portion of the curve to the ordinate axis to get the logarithmic value of extrapolated closure corresponding to date of excavation, and iii)conversion of the extrapolated logarithmic value of closure to natural scale so as to obtain the actual value of tunnel closure, extrapolated to the date of excavation which is then added uniformly to the values of radial closure in order to account for the missing data. This is shown in Fig. 1. It is often seen that dates of excavation, support installation and first observation of closure are totally different. The correct coordinates of the point of intersection, C in Fig. 1, are XDOE which is the final closure extrapolated to the date of excavation and XDOSI which is the support pressure after extrapolation to date of support installation. This is because whereas the ground response curve (GRC) starts at point, A immediately after excavation, the support reaction curve (SRC) comes into picture only after the supports are installed (Point B). It would therefore be incorrect to extrapolate both support pressure and tunnel closure to the date of excavation (Point D) to obtain the observed support reaction curve. Similarly their extrapolation to the date of support installation would also be incorrect. Also, it would be incorrect to plot SRC on basis of only the recorded data without due consideration to unrecorded observations.

5. PREDICTION OF GROUND CONDITION

Before performing rock mass – tunnel support interaction analysis, it is important to know the ground condition a priory, since ground behavior and the approach for prediction of ground response curve differs according to the ground condition. Here, an approach has been proposed to predict three ground conditions; namely, i) self supporting condition, ii) non-squeezing ground condition and iii) squeezing ground condition. The ground response curves for all these three conditions are qualitatively depicted in Fig. 2.

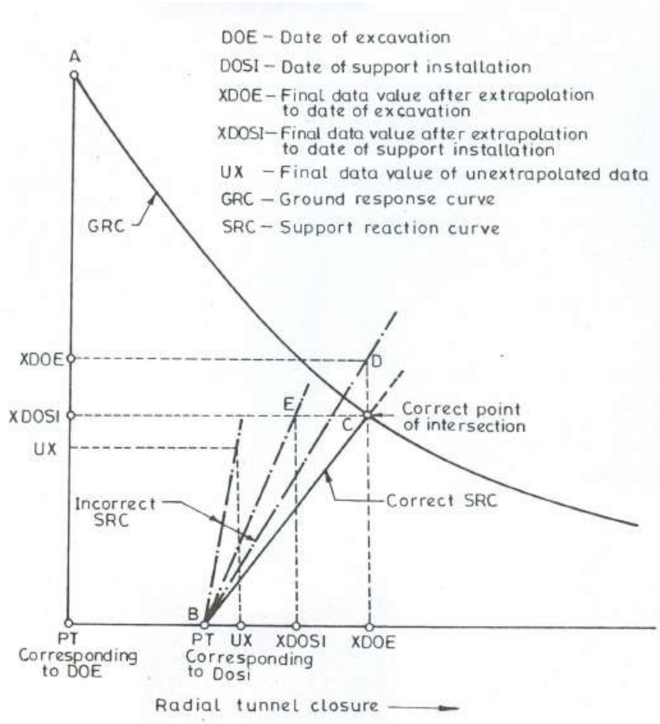

Fig. 1 - Influence of unrecorded data on support reaction curve and point of intersection.

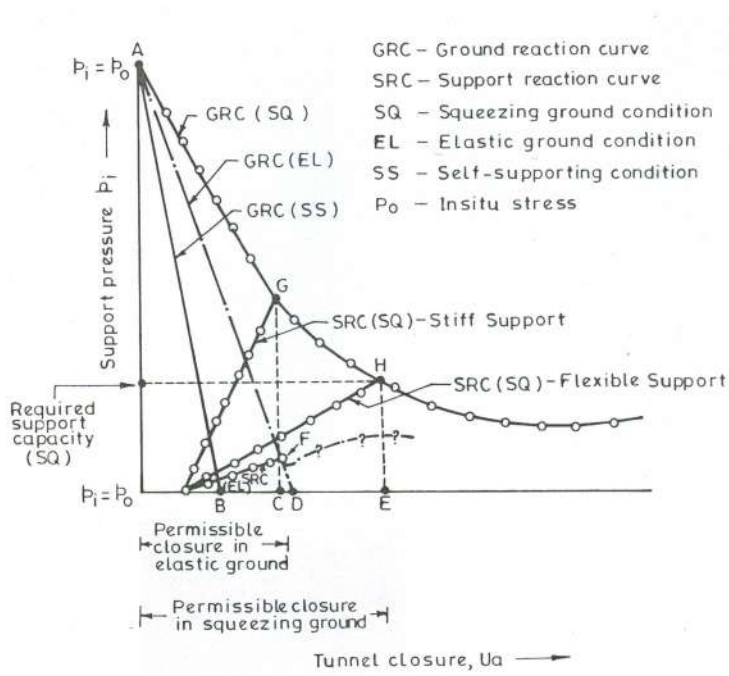

Fig. 2 - Ground response and support reaction curves for three tunnelling conditions

Data obtained during various field studies conducted at several Indian tunnel project sites, both in non-squeezing and squeezing ground conditions and also the data from some case histories reported by Barton et al. (1974) were analyzed. This analysis is presented in Fig. 3 in the form of a log-log plot of Barton's rock mass quality, Q versus $[H (B - B_s)^{0.1}]$, where H is the height of overburden (m), B, the tunnel width or span (m) and B_s, the self supporting span (m) given by Barton et al. (1974) as –

$$B_s = 2 (ESR) . Q^{0.4} \quad (1a)$$

where ESR is the excavation support ratio, which for power tunnels, minor tunnels, rail and road tunnels is usually 1.6. The Eq. 1a, can therefore be written as –

$$B_s = 3.2.Q^{0.4} \quad (1b)$$

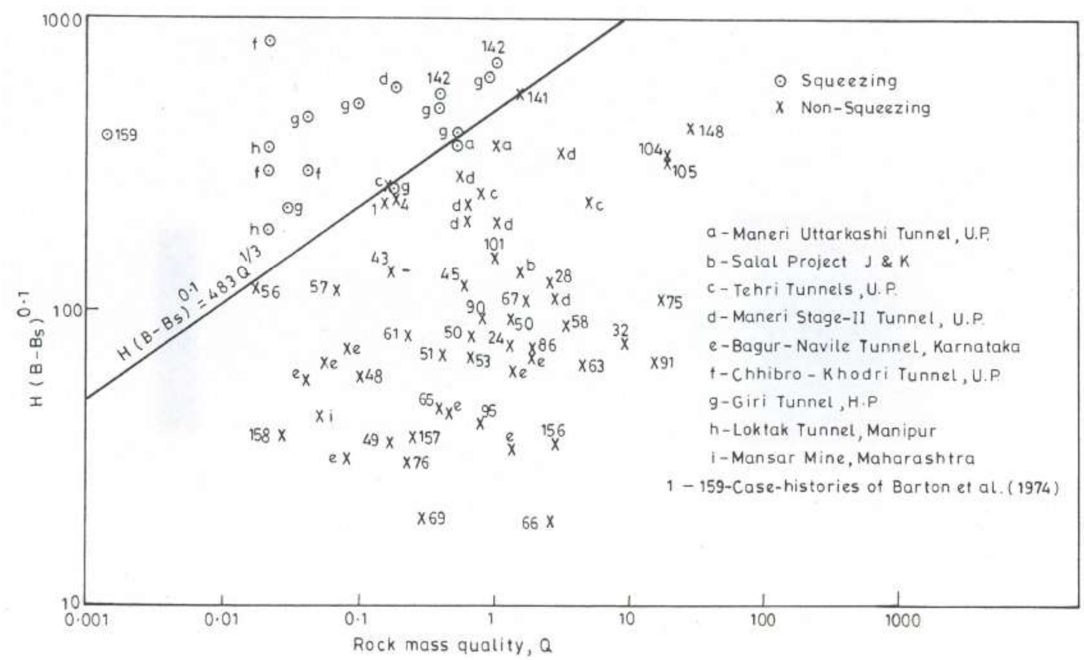

Fig. 3 - Correction for prediction of ground condition

In Fig. 3, points pertaining to squeezing cases may be clearly separated from those belonging to the non-squeezing (elastic) cases by an inclined line, defined by -

$$H (B - B_s)^{0.1} = 483 Q^{1/3} \quad (2)$$

Therefore, for squeezing to occur, left hand side of Eq. 2 should be greater than the right hand side. Occurrence of a particular tunnelling condition may, therefore, be predicted by the following empirical correlations:

$$(B - B_s) < 0 \quad \text{(for self-supporting condition)} \quad (3a)$$

$$H (B - B_s)^{0.1} < 483 Q^{1/3} \quad \text{(for non-squeezing condition)} \quad (3b)$$

$$H\,(B - B_s)^{0.1} > 483\,Q^{1/3} \qquad \text{(for squeezing condition)} \tag{3c}$$

$$J_r/J_a < ½ \tag{3d}$$

In brittle, massive rocks, rock burst may occur instead of squeezing as predicted from Eq. 3c where $J_r/J_a > 0.5$. It may be mentioned here that theoretically (according to the proposed strength criterion, Eq. 13c), squeezing condition around a tunnel opening may be encountered if,

$$\sigma_\theta > q_{cmass} + A.p_o/2 \tag{4a}$$

where σ_θ is the tangential stress and q_{cmass} is the uni-axial compressive strength of the rock mass and p_o is the in-situ stress along tunnel axis. Equation (4a) may be written as follows for a circular tunnel under hydrostatic stress field:

$$2P > q_{cmass} + A.p_o/2 \tag{4b}$$

where P is the magnitude of in-situ stress and A is a rock mass constant.

Use of the theoretical criterion for prediction of squeezing ground condition, given by Eq. (4b), poses practical difficulties as the measurement of in-situ stress and in-situ compressive strength of rock mass is both expensive and time consuming especially in developing countries. This problem can be overcome by using the empirical criteria (Eqs. 3a, b, c) for prediction of squeezing.

Results of the above analysis are presented in the form of a design chart (Fig. 4) plotted on log-log scale. It is clear from this chart that once the values of depth of overburden, H and rock mass quality, Q are known, the designer can pick up the critical value of B below which squeezing is not likely to occur. For doing so, the first step is to pick up the critical value of $(B-B_s)$ for given values of H and Q from the upper part of the design chart, which is based on Eq. 2. The value of B_s for this Q value is then selected from the lower part of the chart, which represents Eq. 1b, and this value of B_s is added to the critical value of $(B-B_s)$ to arrive at the critical value of B. It may be noted that:

(a) For performing rock mass – tunnel support interaction analysis, it is necessary to know the ground condition as the approach for prediction of ground response curve is different for non-squeezing and squeezing ground cases. The ground condition may be predicted approximately using Eqs. 3a, b and c. Thus, classifying a rock mass as a squeezing rock is not correct. Any rock mass may turn into squeezing rock condition at higher overburden.

(b) In case a tunnel is likely to experience squeezing ground condition, the tunnel alignment may possibly be changed to obtain a better rock mass quality, Q or reduced overburden or both so as to avoid squeezing and thereby eliminate/reduce the support problems. Similarly non-squeezing ground condition may possibly be changed to the self-supporting condition by obtaining a better rock mass quality, Q as a result of the changed tunnel alignment.

(c) Alternatively, two or three smaller tunnels may be chosen instead of a larger tunnel in order to avoid squeezing ground conditions thereby reducing the support problems and the construction time. This was done in Chhibro-Khodri tunnel in the state of Uttarakhand (India) when it became extremely difficult to drive a 9m diameter tunnel through squeezing ground condition.

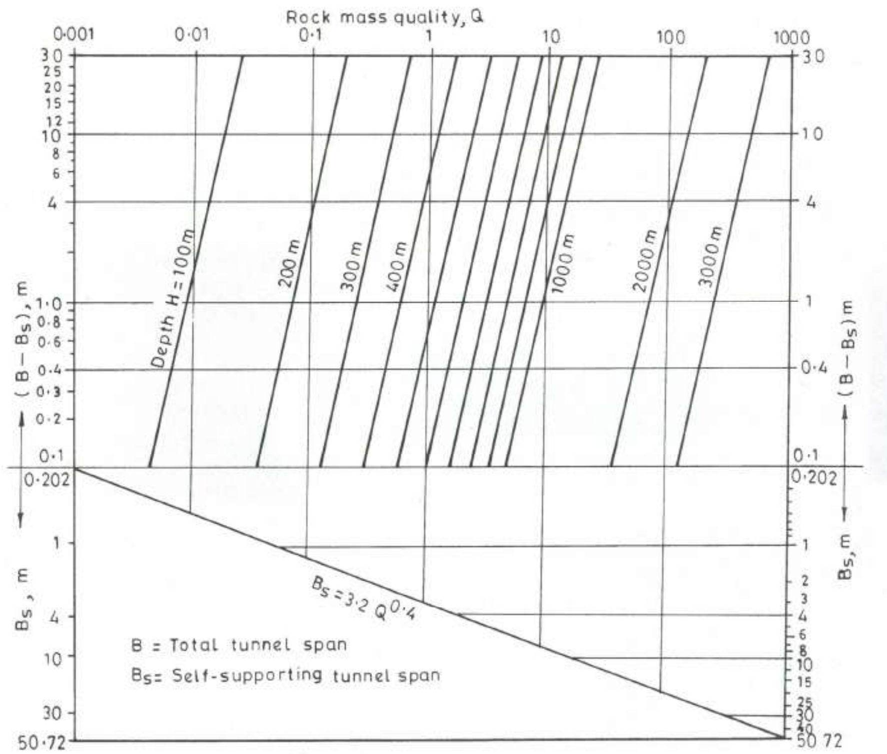

Fig. 4 - Design chart for selecting tunnel size for given tunnel depth and rock mass quality to achieve favorable ground condition

6. PREDICTION OF GROUND RESPONSE CURVE

6.1 Self Supporting/Non-Squeezing Ground Condition

For a circular tunnel driven through homogeneous, isotropic and linearly elastic rock mass experiencing hydrostatic stress condition, ground response curve for elastic ground condition (representing both self-supporting and non-squeezing conditions) may be obtained from theory of elasticity using the following equation –

$$u_a/a = (1+\nu)\,(p_o - p_i)\,/E_d \qquad (5)$$

where, u_a = the radial tunnel closure, a = the radius of tunnel opening, ν = the Poisson's ratio of rock mass, E_d = the modulus of deformation of rock mass, p_o = the in-situ hydrostatic stress and p_i = the required short-term support pressure. The ground response curve may be obtained by plotting p_i/p_o versus u_a/a. It may be seen from Eq. 5

that the ground response curve is predicted to be a straight line relationship for the non-squeezing (elastic) ground condition.

6.1.1 Empirical correlation for modulus of deformation of rock mass

The modulus of deformation in Eq. 5 is normally obtained from expensive and time consuming uni-axial jacking tests which often give a large scatter in results. Therefore, the following simple empirical correlation has been obtained to determine the modulus of deformation of nearly dry rock masses, E_d :

$$E_d = f \cdot 10^{(RMR - 20)/38} \quad \text{GPa} \tag{5a}$$

where, RMR represents Bieniawski's rock mass rating, and f, the correction factor for the effect of depth. The above correlation is based on back analysis of values of modulus of deformation obtained from the data of support pressures and tunnel closure which were observed at several tunnel sections in non-squeezing ground condition (RMR values ranging from 31 to 68). The back analysis was performed by using Eq. 5 for which the observed values of u_a and p_i and assumed value of ν equal to 0.25 were used for different tunnel sections. Assuming a hydrostatic stress field, p_o was considered equal to γH and its values for different tunnel sections were accordingly obtained. The back-analyzed values of modulus of deformation have been plotted versus RMR in Fig. 5 which shows the best-fit curve represented by Eq. 5a and has a correlation coefficient of 91% . Mehrotra (1992) also obtained nearly the same correlation with f =1 from uni-axial jacking tests on dry rock masses. Thus, one may use Eq. 5a with confidence in poor rock conditions also. Empirical correlations have also been proposed earlier by Bieniawski (1978) and Serafim and Pereira (1983) between modulus of deformation, E_d of the rock mass and RMR (Eqs. 5b and 5c). It is interesting to note the similarity between the proposed E_d versus RMR curves (Fig. 5) and Eq. 5c (Serafim and Pereira, 1983).

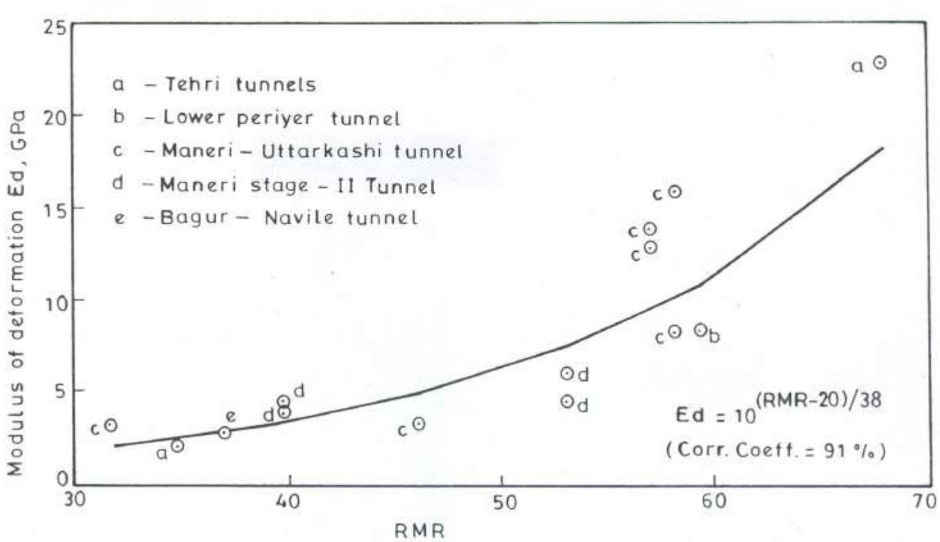

Fig. 5 - Correlation between RMR and modulus of deformation of rock mass

$$E_d = (2\,RMR - 100) \quad \text{GPa for } RMR > 50 \qquad (5b)$$

$$E_d = 10^{(RMR - 10)/40} \quad \text{GPa} \qquad (5c)$$

6.1.2 Effect of depth on modulus of deformation of rock mass

The back analysis of the values of modulus of deformation also highlight its dependence on the height of overburden, H for which correction factor, f, was introduced in Eq. 5a. The correction factor,

$$f = (E_d / 10^{(RMR - 20)/38}) \qquad (5d)$$

has been plotted against the height of overburden, H in Fig. 6 from which the following correlation could be obtained:

$$f = 0.3\,H^{\alpha} \qquad (5e)$$

where α = 0.16 to 0.3 and H > 50 m. Eq. 5a may, therefore, be written as:

$$E_d = 0.3\,H^{\alpha} \cdot 10^{(RMR - 20)/38} \quad \text{GPa} \qquad (5f)$$

where, H is in meters. According to Singh's (1997) analysis of the same case histories, a better correlation for poor rocks is,

$$E_d = H^{0.2} \cdot Q^{0.36} \quad \text{GPa} \quad \text{for } Q < 10 \qquad (5g)$$

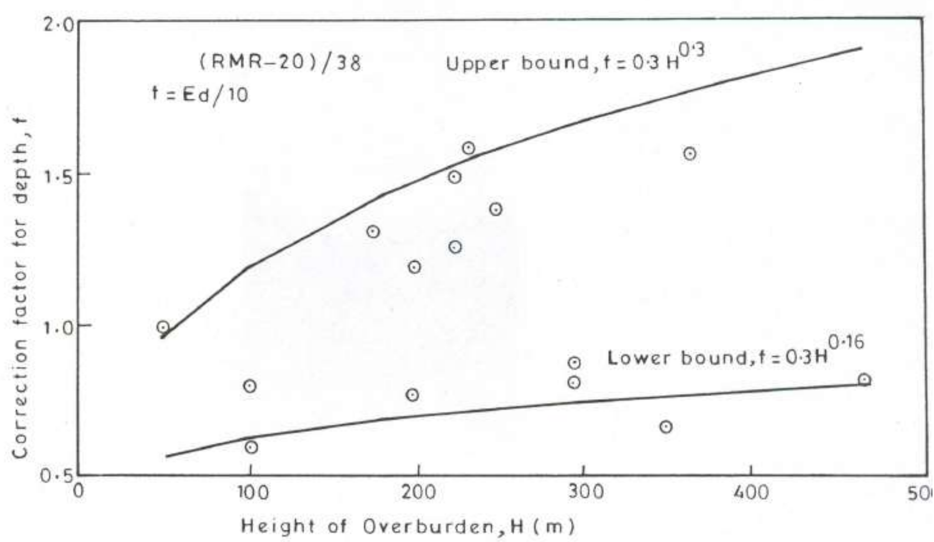

Fig. 6 - Correction factor for effect of depth on modulus of deformation of rock mass

Thus, poor rock masses exhibit pressure dependent modulus of deformation. The case histories which have been considered to arrive at Eq. 5f pertain to poor to good rock mass quality (RMR = 31 to 68). It is quite likely that for rock masses with RMR value greater than 68 i.e. good to very good rock mass, the value of α is lower than 0.16 and for rock masses with a lesser RMR, i.e. less than 31 (i.e., very poor to poor rock mass), the value of α is greater than 0.3. This argument originates from growing evidence from laboratory experiments (Kulhawy, 1975; Santarelli and Brown, 1987; Brown et al., 1989; Duncan Fama and Brown, 1989) which suggests that – a) the modulus of elasticity increases with confining pressure and has a relationship similar to Eqs. 5f and 5b. This pressure dependency of modulus of elasticity, reflected in the value of α, is more pronounced in weaker rock materials and is almost absent in strong and brittle rock materials. Rock mass rating is some times unreliable in poor rock masses. However, rock mass quality is more reliable in poor rock masses and hence Eq. 5g may be used for estimating E_d.

6.2 Squeezing Ground Condition

Several authors have presented elasto-plastic analysis of tunnels (using either elastic-perfectly plastic, elastic-brittle plastic or elastic-strain softening stress-strain models) to obtain solutions for stresses and displacements. In the present study, a semi-empirical approach based on Daemen's (1975) analysis has been proposed.

6.2.1 Equations for support pressure

Daemen (1975) proposed the following equation for short-term support pressure in circular tunnels having radius, r = a under squeezing ground condition, namely,

$$p_i = [\, p_b + c_r \cdot \cot \phi_r \,].\, M_\phi - c_r.\, \cot \phi_r \pm \gamma.\, (b-a).\, M_r \quad (6a)$$

$$p_b = 0.5\, (\sigma_{re} + \sigma_{\theta e})\, .\, (1 - \sin\phi_p) - c_p.\cos \phi_p \quad (6b)$$

in which σ_{re} and $\sigma_{\theta e}$ are the radial and tangential stresses on the elastic side of elasto-plastic interface (r = b), c_p and ϕ_p , the peak and c_r and ϕ_r, the residual strength parameters in elastic and broken plastic zones respectively, p_b , the radial pressure at r = b and γ, the unit weight of rock mass. Equation 6b ignores the effect of intermediate principal stress along the tunnel axis.

$$M_\phi = (a/b)^\alpha \quad (6c)$$

where

$$\alpha = 2.\, [\text{Sin}\ \phi_r/(1-\sin \phi_r)] \quad (6d)$$

and

$$M_r = [a/(b-a)]\, .\, [(1-\sin \phi_r)/(1-3.\sin \phi_r)][(a/b)^{\alpha-1}-1] \quad (6e)$$

Daemen (1975) substituted $2p_o$ for the term ($\sigma_{re} + \sigma_{\theta}e$) for the case of hydrostatic in-situ stress field, where p_o is the in-situ hydrostatic stress. The positive and negative signs pertain to support pressures in the roof and floor portions respectively.

6.2.2 Equations for tunnel closure

Daemen (1975) assumed the rock mass to dilate at failure and allowed for following three variations in the volumetric expansion:

(i) Constant volume expansion throughout the broken zone,
(ii) Volume change due to elastic relaxation of the broken zone with the axial stress calculated from an elastic plane strain assumption and
(iii) Volume change due to elastic relaxation of the broken zone with the axial stress calculated from a plastic plane strain assumption.

Daemen (1975) suggested Labasse's solution (1949) for the first condition and derived expressions for the other two conditions. The final expressions for radial tunnel closure (or tunnel wall displacement), u_a, for the above three conditions are respectively given by Eqs. 7a, 7b and 7c.

For condition (i) above,

$$u_a = a - [a^2 (1+e) - b^2 e - 2b.u_b + u_b^2.]^{1/2} \tag{7a}$$

For condition (ii) above and for $\sin \phi_r = 1/3$,

$$u_a = (b/a).\, u_b + \frac{(1+\nu)(1-2\nu)}{a.\,E_d} [p_o (b^2 - a^2) - p_i \{(b^3 - a^3)/a\} - 3c_r \cos \phi_r b^2 \{(b/a) - 1\} + \gamma \{(b^3 - a^3)/3 - b^3 \log (b/a)\}] \tag{7b}$$

Similarly, expressions have been given by Daemen (1975) for the conditions when $\sin \phi_r = 0$, $\sin \phi_r \neq 0$ and $\sin \phi_r \neq 1/3$.

For condition (iii) above and for $\sin \phi_r = 1/3$,

$$u_a = (b/a)u_b + \frac{p_o(1-2\nu)(b^2 - a^2)}{a.E_d} - \frac{3.c_r.\cos\phi_r (b-a)}{a^2.E_d} . \left[(1-2\nu)\, b^2 + \frac{\nu(b^2+ab+a^2)}{9}\right] - \frac{1-2\nu+\frac{\nu}{9}}{a.E_d} [p_i \{(b^3-a^3)/a\} \mp \gamma \{(b^3-a^3)/3 - b^3 \log (b/a)\}] \tag{7c}$$

Similarly, expressions have been given by Daemen (1975) for the conditions when $\sin \phi_r = 0$ $\sin \phi_r \neq 0$, $\sin \phi_r \neq 1/3$. In the above equations,

e = coefficient of volumetric expansion for failed rock mass which is defined as the ratio of increase in volume of failed rock mass to its original volume,
u_b = radial displacement of elastic-plastic boundary (r = b),
= $\{(1+\nu)/E_d\}\, p_o\, [p_o\, .\, \sin \phi_p + C_p.\cos \phi_p]$ (7d)
E_d = modulus of deformation of rock mass,
p_b = radial stress at the elastic-plastic boundary (r = b) and
ν = Poisson's ratio of rock mass.

Average values of 'e' are given in Table 3 on the basis of back analysis of tunnel closure in squeezing ground conditions.

Table 3 - Observed values of coefficient of volumetric expansion (e) of Rock Mass after failure (Jethwa, 1981)

Type of Rock Mass	Coefficient of Volumetric Expansion, e
Phyllites	0.03
Clay stones/silt stones	0.01
Black clays	0.01
Crushed sandstone	0.004
Crushed shaled	0.005
Crushed metabasics	0.006

6.2.3 Determination of input parameters for Daemen's equations

The ground reaction curve may be obtained from Daemen's (1975) approach by calculating the values of support pressure, p_i, and radial tunnel closure, u_a, for different values of b/a ratio, using the equations given above (Eqs. 7a, b, c, d) These equations, however, contain several input parameters, some of which are difficult to estimate. In particular, the modulus of deformation, the peak and residual values of both cohesion and angle of internal friction of rock mass are required to be determined from expensive and time-consuming field tests. While a correlation has already been proposed for determination of modulus of deformation of the rock mass (Eq. 5c), a semi-empirical relationship will now the proposed for prediction of the rock mass cohesion.

(a) Proposed semi-empirical correlation for peak cohesion of rock mass

Daemen (1975) used following constitutive equation for unbroken rock mass at the periphery of the broken zone:

$$\sigma_{\theta e}\,(1 - \sin\phi_p) = \sigma_{re}\,(1 + \sin\phi_p) + 2c_p \cos\phi_p \tag{8}$$

Therefore, $$\sigma_{re} = \frac{2p_o - q_{c\,mass}}{1 + K} \tag{8a}$$

where, q_{cmass} = uniaxial compressive strength of rock mass in elastic zone

$$= [2c_p \cos\phi_p / (1 - \sin\phi_p)] \tag{8b}$$

and $$K = \left[\frac{1+\sin\phi_p}{1-\sin\phi_p}\right] \tag{8c}$$

For squeezing to begin (or the rock mass to fail), σ_{re} should be greater than zero. Therefore, at the instant when squeezing starts, σ_{re} = 0 or from Eq. 8,

$$P = 0.5\ q_{cmass} \tag{9a}$$

From the empirical correlations (Eqs. 3b and 3c) for prediction of squeezing ground condition, it may be inferred that at the instant when squeezing begins,

$$H = \frac{483.Q^{1/3}}{(B - B_s)^{0.1}} \tag{9b}$$

Multiplying both sides of Eq. 9b with γ and substituting γH with p for hydrostatic stress field,

$$H = \frac{P.483.\gamma.Q^{1/3}}{(B - B_s)^{0.1}} = \frac{q_{cmass}}{2} = \frac{c_p.\cos\phi_p}{(1 - \sin\phi_p)} \tag{9c}$$

The mobilised cohesion, c_{pm} will therefore be

$$c_{pm} = \frac{483.\gamma.Q^{1/3}}{(B - B_s)^{0.1}} * \frac{(1 - \sin\phi_p)}{\cos\phi_p} \quad (t/m^2) \tag{10}$$

where, γ is the unit weight of rock mass in t/m^3. Obviously, b = radius of the broken zone and ϕ_p may be obtained from block shear test or from RMR as suggested by Bieniawski’s (1979) classification.

It may be recalled that the uniaxial compressive strength of rock core decreases with $d^{0.18}$ where d is the diameter of core (Hoek and Brown, 1980). In field, d may be taken as average spacing of joints. Similar size effect is also observed in Eq. 10.

(b) Mobilised strength of rock mass around underground openings

Figure 7 shows a comparison between mobilised cohesion, c_{pm} and the cohesion c_p, determined from Bieniawski’s RMR, and from Mehrotra (1992), for a tunnel of 9m diameter. The c_p values given by Bieniawski (1979) are based on the field test data from rock slopes compiled by Hoek and Bray (1977). Mehrotra (1992) obtained the shear strength parameters from block shear tests conducted on dry rock mass blocks in the lower Himalayan region. The c_{pm} / c_p ratio is plotted against RMR in Fig. 8.

It is clear from Fig. 8 that there is definitely a need to account for a strength enhancement factor (= c_{pm} / c_p), which increases with increasing RMR. This strength mobilisation around the underground openings, known as an apparent strength enhancement, has been recorded by several investigators (Hobbs, 1966; Hoskins,1969; Daemen and Fairhurst ,1971; Santarelli and Brown 1987; Guenot, 1989 and Fuenkajorn and Daemen, 1992) during laboratory tests on thick-walled hollow cylinders. Daemen and Fairhurst (1971), for instance, found no indication of fracturing around the borehole when the external hydrostatic pressure applied to thick-walled hollow cylinders of Indiana limestone and concrete reached levels at which linear elastic analysis gave tangential stress at the borehole wall of at least four times the measured uni-axial compressive strength of the material. Final collapse occurred at even higher pressure. Guenot (1987) presented a survey of results of such laboratory tests on hollow cylinders conducted by ten researchers on seven rock types. The ratio between the maximum (elastically) calculated compressive stress at which the failure occurs and the uni-axial compressive strength is typically about two. More recently, Fuenkajorn and Daemen (1992) obtained following empirical equation from biaxial borehole stability tests on cylindrical tuff samples:

$$\sigma_{\theta f} = 312.2 \exp(2.05\, \sigma_{H2}/\sigma_{H1}) \quad \text{MPa} \tag{11}$$

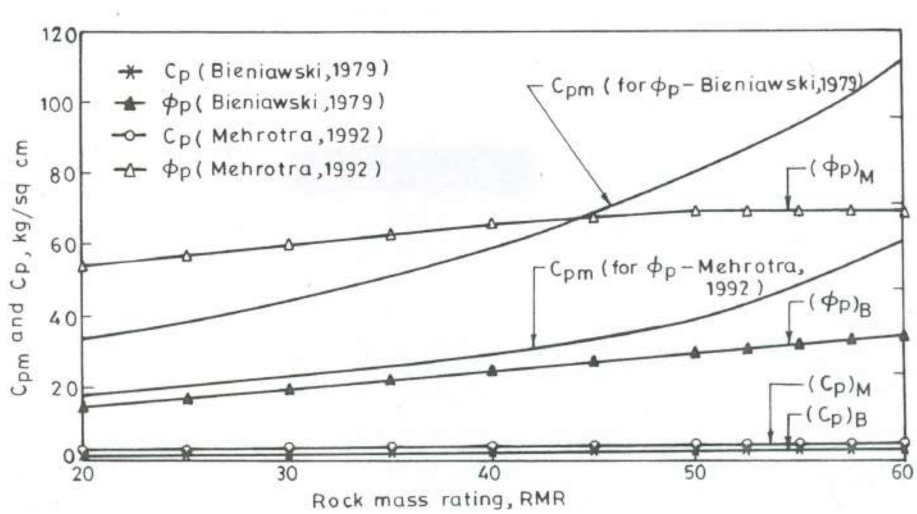

Fig. 7 - Mobilized cohesion (Eq. 10) with respect to RMR in a tunnel of 9 m Span

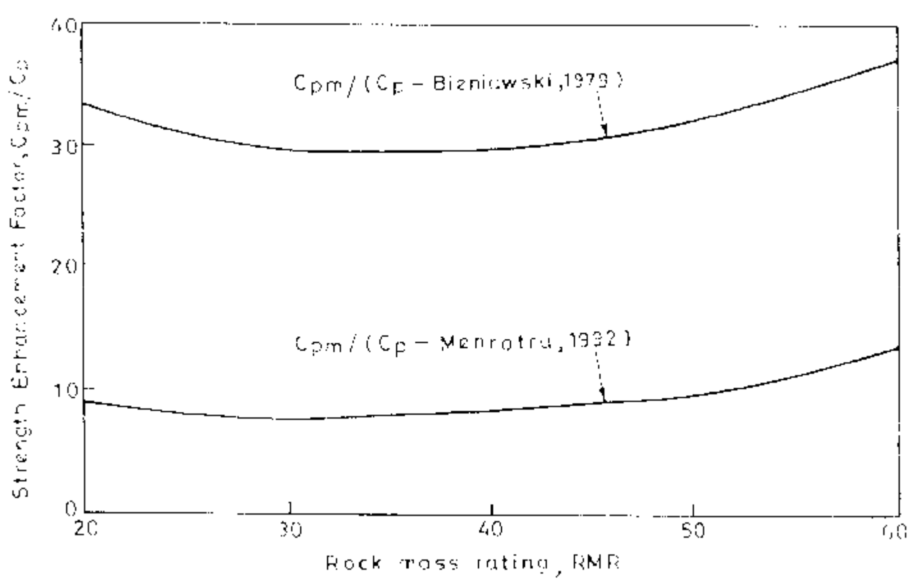

Fig. 8 - Recommended strength enhancement factor for cohesion parameter from block shear test

where σ_{H2}/σ_{H1} is the ratio of minimum and maximum applied boundary stresses and $\sigma_{\theta f}$, the tangential compressive stress at borehole wall immediately before the fracture occurs. Fuenkajorn and Daemen (1992) however, acknowledged that Eq. 11 might overestimate the rock mass strength around large borehole due to the size effect and that the incorporation of this effect was not possible due to the lack of test data on large boreholes. Equation 11 further suggests that strength enhancement will be less in case of a general biaxial in-situ stress condition.

The reason for apparent strength enhancement is that the shear strength behavior of jointed rock mass is highly anisotropic (Hoek and Brown, 1980). RMR classification gives lower limit of the strength parameter as obtained from failure of rock slopes. However, all round squeezing would not take place unless the tangential stress (2P) exceeds the maximum limit of the uni-axial compressive strength of rock mass. Hence, the mobilized cohesion (c_{pm}) may represent the upper limit of cohesion of anisotropic rock mass. Moreover, uni-axial compressive strength statistically varies from one element of the same rock mass to another element depending upon the distribution of fractures. All round squeezing will not take place until tangential stress exceeds the upper statistical limit of the uni-axial compressive strength of rock mass. It may also be noted that the rock mass quality, Q is obtained from the visual inspection of the excavated face of tunnel which is likely to be poorer than the rock mass quality in the elastic zone. Further, joints in tunnels are of smaller length and tightly closed unlike those on slopes.

Hudson (1993) suggested that the apparent strength enhancement indicated in Fig. 8 could be due to the difference in the condition of shearing in rock slopes (where full dilatancy is operative) and that for underground openings (no dilatancy).

Another reason for apparent strength enhancement around underground openings appears to be the fact that failure stresses are calculated assuming the classical (constant modulus) linear elasticity, whereas the deformation modulus has been found to increase with increasing confining pressure (Eq. 5c). There is a growing evidence to suggest that linear elasticity approach can give misleading prediction of the onset and extent of fracture, particularly in softer rocks (Guenot, 1987; Kaiser et al. 1985; Maury 1987; Santarelli, 1987). Santarelli and Brown (1987) derived closed-from solution for stresses and strains around an axi-symmetric well bore assuming a confining pressure dependent modulus of elasticity and concluded that tangential stresses at or near the well bore wall could be much lower than those predicted from theory of elasticity and that maximum tangential stress which occurred some distance from the well bore wall are given by:

(i) Normalised tangential stress at elastic-plastic interface,

$$\sigma_{\theta e}/p_o = K_1(\sigma_{re} / p_o) - K_2 (\sigma_{re} / p_o)\, \alpha \quad (12a)$$

(ii) Normalised radial stress at elastic-plastic interface,

$$\sigma_{re}/p_o = [(p_i / p_o)^{(1-\alpha)} -1\} \{b/a\}^{(1-\alpha) K_2} +1]^{(1/1-\alpha)} \quad (12b)$$

where, $K_1 = [\nu(1-\alpha)-1] / [(1-\nu)(1-\alpha)]$ (12c)

$$K_2 = [(2\nu -1)(1- \alpha)-1] / [(1-\nu)(1-\alpha)] \tag{12d}$$

and α = constant of rock mass in Eq. 5e and 5f.

Figure 9 shows the variation of tangential stress with radial stress for different values of α from which it is clear that stress concentration factor is much below the value of 2.0, which is obtained when modulus of elasticity ($\alpha = 0$) is considered to be constant and that the maximum tangential stress occurs some distance away from the periphery.

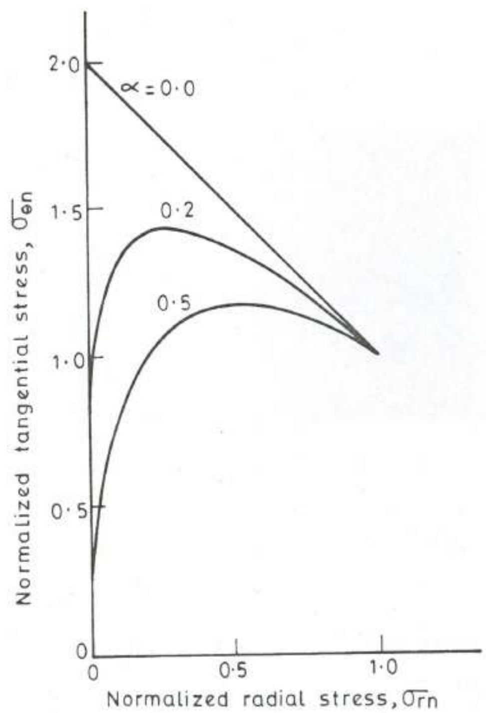

Fig. 9 - Variation of normalised tangential stress with the normalised radial stress for different values of A (Santarelli and Brown, 1987)

6.2.4 New theory of peak strength of rock mass

Singh et al. (1998) have proposed a new criterion for peak strength of anistropic jointed rock masses, according to which,

$$\sigma_\theta - \sigma_r = q_{cmass} + \frac{(P_o - \sigma_r)}{2} A \tag{13a}$$

$$q_{cmass} = q_c [E_d / E_r]^{0.7} \tag{13b}$$

where, q_{cmass} represents the average uni-axial compressive strength of rock mass; q_c, the average uni-axial compressive strength of rock material; E_d, the modulus of deformation of rock mass; E_r, the modulus of elasticity of rock material and p_o is the in-situ effective stress along the tunnel axis. Thus, the condition for squeezing is:

$$\sigma_\theta > q_{cmass} + (A.p_o/2) \tag{13c}$$

Squeezing may therefore occur where A is small. Indeed squeezing has taken place where Jr/Ja < 1/2 which is a vital condition. The above strength criterion also explains enormous strength enhancement. Daemen's equation (Eq. 6) may be rederived easily using new strength criterion and only differs in the expression of P_b as follows:

$$P_b = \frac{\sigma_{re} + \sigma_{\theta e} - q_{cmass} - 0.5Ap_o}{2 + 0.5A} \tag{13d}$$

6.2.5 Suggestions for plotting ground response curve

The ground response curve for squeezing ground condition can now be plotted using Eq. 7 along with Eqs. 7a, 7b and 7c for tunnel closure and Eqs. 6a, 6b and 6c for support pressure. In equations 6a,b,c, the value of (σ_{re} + $\sigma_{\theta e}$) should be picked up from Fig. 9 according to the actual value of α and the radial pressure at the inner boundary of the elastic zone. Further, peak cohesion parameter, c_p in Eqs. 6a, 6b and 6c should be substituted by c_{pm} (recommended) from Fig. 8 so as to account for the strength enhancement factor in the elastic zone.

The peak angle of internal friction, ϕ_p may be taken from RMR classification system (Bieniawski, 1979). The residual cohesion, c_r may be taken as about 0.1 MPa. However, the residual angle of internal friction, ϕ_r may be taken as equal to $(\phi_p - 10^o) \geq 14^o$ in the broken zone for (b/a) < 5.

6.2.6 Rapid sympathetic failure in large broken zones

Figure 10 shows the empirical ground response curve between observed support pressure (normalised with respect to short term support pressure of Barton et al. 1974) and observed tunnel closure (normalised with the tunnel diameter). It may be noted that there is an onset of a rapid sympathetic failure of rock mass within the broken zone when tunnel closure exceeds 6 % of the tunnel diameter. This observed phenomenon may be simulated by considering $c_r = 0$ after deviatoric strain exceeds a critical limit (10 % for weak rocks). The result would be a ground response curve as shown in Fig. 11 which is similar to the empirical curve in shape. It is therefore suggested that tunnel closure should be controlled to within 4 % of the tunnel width as otherwise the support pressure may jump drastically.

7. CONCLUSIONS

The work presented in this paper draws its strength from field studies carried out at 63 different sections of tunnels at various project sites in the Lower Himalaya and the peninsular India. These field studies involved instrumentation and monitoring of data related to tunnel closure, deep seated deformations in rock mass, contact pressures between rock mass and steel sets and the loads in steel ribs, apart from other data related to geometry and rock mass classification. This data has been analyzed with the aim of proposing a practical approach for prediction of ground response curve for self supporting, non-squeezing and squeezing ground conditions.

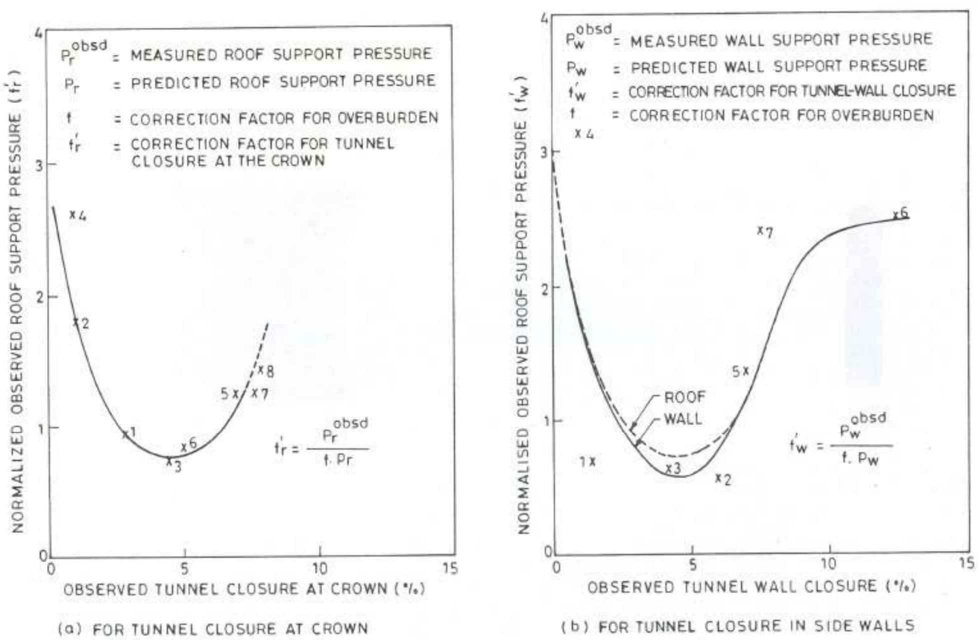

Fig. 10 - Empirical ground response curves

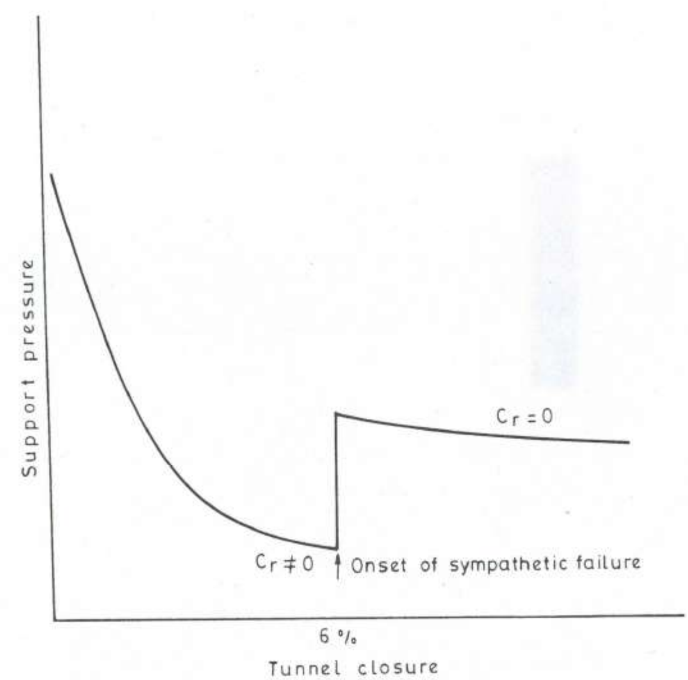

Fig. 11 - Effect of sympathetic failure of rock mass on ground response curve

It may be concluded that –

(i) Condition for squeezing depends upon in-situ stress magnitude along the tunnel axis as well as the depth of overburden (Eqs. 3 a-d).
(ii) Modulus of deformation of poor rock masses has been found to be pressure dependent (Eqs. 5a-f).
(iii) Support pressure in the squeezing ground is reduced drastically by in-situ stress along the tunnel axis because all rock wedges are pre-stressed by σ_2 along the tunnel axis.
(iv) Strength enhancement in jointed rock mass has been found to be significant (Eq. 10).
(v) Peak strength criterion for the anisotropic rock masses (Singh et al., 1998) is given by Eqs. 13 a and 13b which includes the effect of σ_2.
(vi) Rapid sympathetic failure of rock mass within the broken zone occurs when tunnel closure exceeds 6 % of the tunnel diameter. So tunnel closure should be controlled within 4 % of tunnel size.

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