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SYNOPSIS: Data regarding the performance of three tunnelling projects in the Himalayan region has been collected. It is seen that rock loads or deformations calculated on the basis of Barton, Bieniawski or RMR approach do not match the field data. A mathematical model has been developed incorporating modifications in the approach of Brown et.al of rock-support interaction, using elastic-strain softening-plastic ground characteristics. A non-linear relationship between radial and tangential strains around the tunnel has been considered and the method of calculation of stresses and deformations altered to incorporate exact integration of the governing differential equation for a thin cylindrical annulus replacing finite difference approximation. It is seen that a closer match and a more rational explanation of the observed data from the tunnelling project is provided by the mathematical model.

INTRODUCTION

There are two main approaches for estimation of rock loads and deformations of underground openings. The first approach is analytical and mostly utilises continuum mechanics principles making certain simplifying assumptions regarding ground behaviour, yield criterion and volume changes, etc. The second approach is empirical in nature and is based on statistical analyses of observed behaviour of underground openings in different geologic environments. Both the methods have their own relative advantages and are applicable with varying degrees of reliability for several different categories of rock masses ranging from massive to heavily jointed. However, none of the approaches seem to work for tunnels through squeezing grounds.

In an attempt to predict rock loads and deformations of tunnels through squeezing grounds, the approach of Brown et al(1983) of rock support interaction has been modified using elastic-strain softening-plastic ground characteristics. A non-linear relationship between radial and tangential strains around the tunnel has been considered and the method of calculation of stresses and deformations has been altered to incorporate exact integration of the governing differential equation for a thin cylindrical annulus replacing finite difference approximation.

The model developed has been tried for the calculation of rock loads and deformations of three tunnelling projects in the Himalayan region.

THE MATHEMATICAL MODEL

Literature survey reveals that there are three main considerations which distinguish one mathematical model from the other. The first consideration concerns the yield criterion for the strength of the rock mass. Most of the earlier researchers have used Coulomb's or Mohr-Coulomb's criterion. Some others have used the second order parabolic criterion of Fairhurst. In the more recent past, the non-linear criterion of Hoek and Brown has gained popularity, and the same has been used in the present work. The second consideration is regarding the stress-strain behaviour of the ground. Whereas closed form solutions are available for elasto-plastic and elastic-brittle-plastic materials a step by step procedure can be used for elastic-strain softening-plastic material. The third consideration that matters is concerning the change in volume of rock as it yields. Here again, different researchers have used different assumptions starting from no volume change to the ones following flow rules.

Use of Non-linear Relationship between Strains

It is seen from the results of experiments described above that the relationship between axial and radial strains of rock samples after failure is non-linear. The rate of increase of radial strain is large initially as the material yields but it decreases as the axial strain further increases.

A similar relationship between the radial and tangential strains of yielding rock around the circular tunnel can be anticipated. If the initial slope of $\epsilon_\theta - \epsilon_r$ curve be h_1 and if h_2 be the amount by which the secant slope of curve reduces as the major principal strain increases from an initial value $\epsilon_{\theta e}$ to infinity, the secant slope h of the curve corresponding to any intermediate value of ϵ_θ is given by

$$h = h_1 - h_2 \left(1 - \frac{\epsilon_{\theta e}}{\epsilon_\theta}\right) \qquad (1)$$

where h is ratio of radial strain ϵ_r to axial strain ϵ_θ at any stage, such that $\epsilon_r = -h\epsilon_\theta$, h_1 is the initial tangent of $\epsilon_\theta - \epsilon_r$ curve, and $(h_1 - h_2)$ is the corresponding secant slope as

ϵ_θ goes to infinity.

$\epsilon_{\theta e}$ is the major principal elastic strain at which the rock yields and goes from elastic to a plastic behaviour. The assumed relationship between the two strains is shown in the Fig.1.

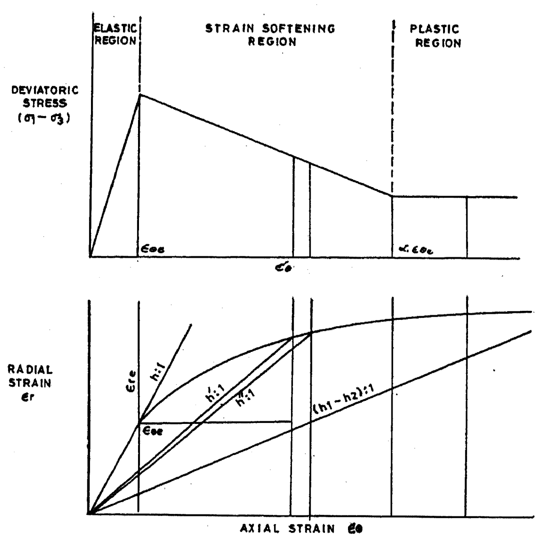

Fig.1 Assumed Material Behaviour Model

Use of Integration

The entire rock mass surrounding the tunnel is divided into a number of thin concentric annular rings. The radius, stresses and strains at one surface of any ring are known. For the calculation of corresponding radii, stresses and strains at the other surface of the ring, Brown et.al(1983) have used a finite difference method. It is, however, possible to replace the finite difference approximation by actual integration of the differential equation for the thin annulus and thus use an approach which is part integration and part finite-difference.

From the conditions of axial symmetry, we know that

$$\epsilon_\theta = -\frac{u}{r} \qquad (2)$$

$$\epsilon_r = -\frac{du}{dr} \qquad (3)$$

where ϵ_θ and ϵ_r are the tangential and radial strains, u is the radial displacement at a radius r.

As the ϵ_θ-ϵ_r relationship is non-linear, an average value of h is assumed for the thin annulus between radius r_1 and r_2, such that

$$\epsilon_r = -h\,\epsilon_\theta \qquad (4)$$

If the radial and tangential strains at the elastic-plastic boundary ($r = r_e$) are given by ϵ_{re} and $\epsilon_{\theta e}$ respectively

$$(\epsilon_{r_1} - \epsilon_{re}) = -h_1(\epsilon_{\theta_1} - \epsilon_{\theta e}) \qquad (5)$$

and $$(\epsilon_{r_2} - \epsilon_{re}) = -h_2(\epsilon_{\theta_2} - \epsilon_{\theta e}) \qquad (6)$$

where ϵ_{r_1}, ϵ_{r_2} and h_1, h_2 are the radial strains and the value of parameter h at the two surfaces of the annulus respectively. Combining these equations we have

$$\frac{du}{dr} = -h\frac{u}{r} - (h-1)\,\epsilon_{\theta e} \qquad (7)$$

where h is the average value of h_1 and h_2. Integrating the equation yields

$$u = -\frac{(h-1)}{(h+1)}\,r\,\epsilon_\theta + C_1\,r^{-h} \qquad (8)$$

The constant C_1 can be evaluated by using the condition $u = -r_e\,\epsilon_{\theta e}$ when $r = r_e$.

$$\epsilon_\theta = -\frac{u}{r} = \left\{(h-1) + 2\left(\frac{r_e}{r}\right)^{h+1}\right\}\frac{\epsilon_{\theta e}}{(h+1)} \qquad (9)$$

Applying this equation to the annulus with radii r_1 and r_2 and on re-arranging

$$r_2 = \frac{r_1}{\left\{\frac{(h+1)\,\epsilon_{\theta_2}}{2\,\epsilon_{\theta_1}} - \frac{(h-1)}{2}\right\}^{\frac{1}{(h+1)}}} \qquad (10)$$

This equation can be used to get the radius of the second surface r_2.

COMPLETE STEPWISE PROCEDURE

To summarise, the complete procedure for calculation of radial convergence is enumerated in the following steps.

1. The following data are required

   - E = Modulus of deformation of rock mass
   - μ = Poisson's ratio.
   - p_o = In-situ hydrostatic stress
   - σ_c = Uniaxial compressive strength of rock material
   - m,s = Constants describing the peak strength of rock mass.
   - m_r,s_r = Constants describing the residual strength of failed rock mass.
   - h_1,h_2 = Constants relating radial strain to the tangential strain.
   - r_i = Radius of the tunnel.

2. Three different zones are assumed to exist around the tunnel(Fig.2).

   a) an elastic zone remote from the tunnel

   b) an intermediate plastic zone in which the stresses and strains are associated with the strain softening portion of Fig.2.

   c) an inner plastic zone in which the

stresses are limited by the residual strength of the rock mass.

3. The yield criterion of Hoek and Brown (1980) has been used, which gives the peak strength as

$$\sigma_1 = \sigma_3 + (m\sigma_c\sigma_3 + s\sigma_c^2)^{1/2} \qquad (11)$$

where σ_1 = major principal stress

σ_3 = minor principal stress

σ_c = uniaxial compressive strength of the rock material

m,s = constants which depend upon the naturee of rock mass and the extent to which it is broken before it is subjected to stresses σ_1 and σ_3.

4. It is assumed that in the broken zone the parameters m and s will be reduced to m_r and s_r, and the residual strength of the rock mass will be given by

$$\sigma_1 = \sigma_3 + (m_r\sigma_c\sigma_3 + s_r\sigma_c^2)^{1/2} \qquad (12)$$

5. The rock mass is assumed to be linearly elastic with Young's modulus E and Poisson's ratio μ, until the initial strength for the appropriate value of σ_3 is reached. Thereafter the strength gradually drops with increasing strain as shown in Fig. 3. Strength reduction from the peak and continued deformation at residual strength are accompanied by plastic dilation. Some elastic volume increase may also occur when the stresses are reduced. These aspects are covered in the appropriateness of constants α, h_1 and h_2.

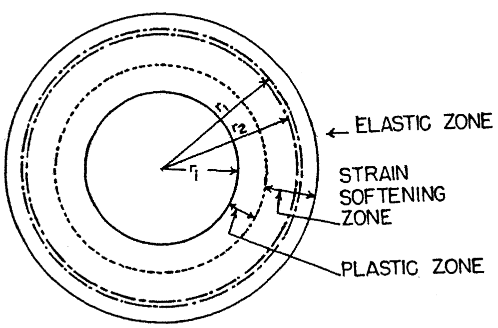

Fig.2 Assumed Zones Around the Tunnel

6. To start with, the initial value of the radius of the plastic zone i.e. the boundary of elastic-plastic region is calculated using the closed-form solution. The radial and tangential strains and the stresses at the boundary are also calculated using the closed-form solution for elastic brittle plastic case. The relevant equations are reproduced,

i) radius of broken zone

$$r_e = r_i \exp\left\{N - \frac{2}{m_r\sigma_c}(m_r\sigma_c p_i + s_r\sigma_c^2)^{1/2}\right\} \qquad (13)$$

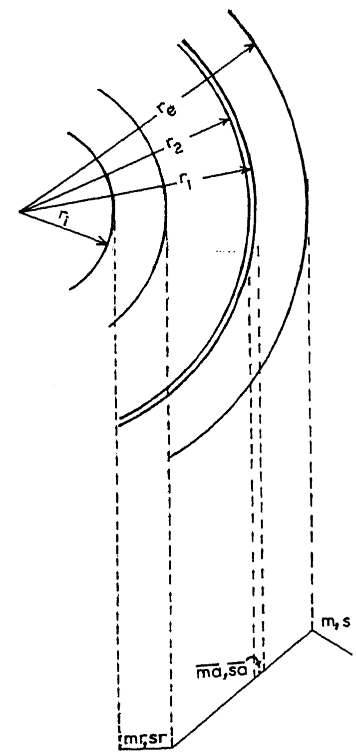

Fig.3 Interpolation for the Values of m and s in the Strain-Softening Region.

where $$N = \frac{2}{m_r\sigma_c}(m_r\sigma_c p_o + s_r\sigma_c^2 - M m_r\sigma_c^2)^{1/2} \qquad (14)$$

ii) Strains

$$\epsilon_{\theta e} = \frac{M\sigma_c}{2G}, \qquad \epsilon_{re} = -\frac{M\sigma_c}{2G} \qquad (15)$$

where $\epsilon_{\theta e}$ = major principal strain corresponding to yielding stress

ϵ_{re} = minor principal strain corresponding to yielding stress and

$$G = \frac{E}{2(1+\mu)} \qquad (16)$$

iii) Stresses

The radial stress at the boundary of the broken zone is given by

$$\sigma_{re} = p_o - M\sigma_c \qquad (17)$$

7. The entire plastic zone including the strain-softening portion is divided into a number of thin concentric annular rings. The tangential strain is increased by an amount $d\epsilon_\theta$ as we move from one ring to the next adjoining one. The new values of the tangential and radial strains are calculated with the help of material behaviour model as shown in Fig. 3. If

the values corresponding to two-surface of annular ring are subscripted by 1 and 2 respectively, we know ϵ_{r_1}, ϵ_{θ_1}, ϵ_{r_2}, ϵ_{θ_2} and r_1. The corresponding radius r_2 can be calculated from the equation

$$r_2 = \frac{r_1}{\left\{ \frac{(h+1)\,\epsilon_{\theta_2}}{2\,\epsilon_{\theta_1}} - \frac{(h-1)}{2} \right\}^{\frac{1}{(h+1)}}} \qquad (18)$$

where $h = \frac{(h' + h'')}{2}$

$$h' = h_1 - h_2 \left\{ 1 - \frac{\epsilon_{\theta e}}{\epsilon_{\theta_1}} \right\}$$

$$h'' = h_1 - h_2 \left\{ 1 - \frac{\epsilon_{\theta e}}{\epsilon_{\theta_2}} \right\}$$

8. Once the radius of the annular ring is known, the radial stresses may be calculated from the equation

$$\sigma_{r_2} = b - \sqrt{b^2 - a} \qquad (19)$$

where $a = \sigma_{r_1}{}^2 - 4k\left\{ \frac{1}{2}\,\bar{m}_a\,\sigma_c\,\sigma_{r_1} + \bar{s}_a\,\sigma_c{}^2 \right\}$

$b = \sigma_{r_1} + k\,\bar{m}_a\,\sigma_c$

and $k = \left\{ \frac{r_1 - r_2}{r_1 + r_2} \right\}^2$

9. The above mentioned procedure is continued till we reach the strain exceeding which implies that we are in the plastic region. The value of constants m and s in the strain softening region $\overline{ma}$, $\overline{sa}$ which are calculated by linear interpolation, are taken as m_r and s_r in the plastic region. The process of calculation is further continued till we reach a value of radius r_2 for which σ_r equals the internal pressure p_i.

10. This value of r_2 is made equal to the given radius of tunnel r_i and all the other values of r_2 and r_1 are reduced in the the same proportion.

11. The axial strain at the tunnel boundary when multiplied with the radius r_i gives the convergence u_i.

It is seen that by making these changes, the iterative procedure converges faster and the results for a particular case of elastic-brittle-plastic rock are closer to those obtained from closed-form solutions (Sharma 1985, Sharma and Ramamurthy, 1986, Ramamurthy 1986).

FIELD STUDIES

The response of the rock mass during and after tunnelling was observed on three tunnelling projects in the Himalayan region. The salient features of the projects along with response data are briefly described below :

lectric Project

of the Giri hydro-electric l Pradesh, India) has an instal- 2 units of 30 MW each. Besides age and an intake regulator, the water conductor system of the project comprises a concrete-lined tunnel of 7.4 km. length, (3.66 m. diameter finished) through the ridge separating the valleys of the Giri and the Bata. In addition, it has access adits of about 0.8km. length, bringing the total tunnelling work in the project to over 8 kms.

Geology of the Tunnel Areas

The tunnel passes through an area which is geologically very complex. The most important feature from engineering geology view point is the occurrence of three major thrusts lying in close proximity of one another. Rocks of different ages have come together as a result of large scale movements resulting in folding, faulting and cutting up by numerous joints and occasional crushing. The alignment crosses two major regional thrusts viz. Krol and Nahan.

From the inlet heading, the tunnel initially traverses rock formations of Bailani series of carboniferous age for a length of about 1500m. The bailanis are dark grey black quartzitic slates containing angular to round pebbles and boulders firmly embedded in a clay-silt matrix. The boulders vary in size and show great variation in the density of their distribution and very often the matrix is without any boulders. The rock formations show extensive jointing and shearing at places and tunnel alignment generally follows the strike.

In between, along the tunnel alignment, the strata changes to claystones and siltstones which are highly jointed and deteriorate on saturation. The material in the vicinity of the faults is highly saturated, soft and plastic. Towards the outlet of the tunnel, the strata generally comprises sandstones with occasional thin bands of claystone and siltstones. The rock is jointed but generally competent, except when moisture is present and claystone bands are present.

Rock Behaviour during Construction

The initial support system was part circular section of rolled steel joints 150 mm x 79 mm x 19 kgs., placed at a spacing of 1 metre centre to centre, and back-filled with lean concrete laid over reinforced concrete laggings. The initial collapse in the inlet tunnel indicated the possibility of the section being inadequate. The support system in these reaches was then modified to 150 mm x 150 mm x 37.5 kg. steel section at the spacing of one metre centre to centre. After some time it was noticed that even these ribs began exhibiting signs of distress by opening joints and twisting. These supports were strengthened by additional steel. As the movements became more pronounced, the spacing was reduced to ½ metres centre to centre. The following observations were made at the site.

i) The supporting system comprising heavy steel sections of 150 mm x 150 mm x 37.5kg. @ ½ m. c/c showed evidence of twisting and buckling even where the space between the rock and the ribs had been completely back-filled with lean concrete. The phenomenon was observed a few weeks after the erection of the support system.

ii) Wherever the rock mass was exposed the rock had intruded into the space between the supports. This intrusion has been considerable, even as much as 15-20 centimetres. The intrusion was observed all around the diameter and the flowing rock was good even after two years and had not detached from the parent rock mass.

iii) The full circular steel rings, though considered better in resisting the rock pressure, showed evidence of much worse twisting and buckling as compared to the deformation noticed in the adjacent D-type ribs. In both the cases, the ribs were of the same size and placed at the same spacing. In the case of D-type ribs, some deformation was absorbed in the inward movement of the sides, which were free to move at the bottom, causing relaxation in loads.

iv) In an experimental reach, the ribs were completely embedded in pneumatically placed concrete of 10 MPa compressive strength. The distortion of ribs was much worse in this zone. The concrete showed evidence of cracks at numerous places and had bulged out by as much as 15-20 centimetres, without showing any signs of detachment from the parent rock mass with which it had developed a bond.

v) The phenomenon of the opening up of the ribs was observed at a number of locations. In some cases, the bottom segment had absolutely detached from the rib but the arch was still in position. In case, the loads were due to loosening, the ribs should have failed or collapsed due to sustained load over it. It shows that the load manifested only when the deformation was resisted. Once the rock created room for deformation, by the failure of the joints and the twisting of the ribs, the loads were 'shed' to the surrounding strata and hence even the ribs without significant bottom support were standing.

In the back-ground of the above, it was decided to use compressible back-fill in the space available between the rock face and the rib. This gave some relief to the support system, but it was only a matter of time, when the closure caught up with the empty space, and thereafter, even the back-filled sections were equally affected.

Time Closure Observations

The horizontal closure of the ribs was observed daily with respect to the day of the installation. The average closure was found in the range of 25-30 centimetres, the maximum being of the order of 50 centimetres. The tunnel showed appreciable closures in all reaches, irrespective of the type of the support and the type of backfill.

The rate of closure was very high in the first 20-30 days of the installation of the ribs. Typically, the closure was 26 centimetres at the end of 20 days in the case of the concrete back-filled ribs, which rose to 28 centimetres at the end of 75 days.

In the case of reaches with loose back-fill, the closures took place more gradually, in a more uniform manner, though the ultimate magnitudes were of the same order.

In some reaches, the closures continued even after 300 days, though at a much reduced rate. Figs.4 shows typical closure observations with respect to time, for reaches with loose packing.

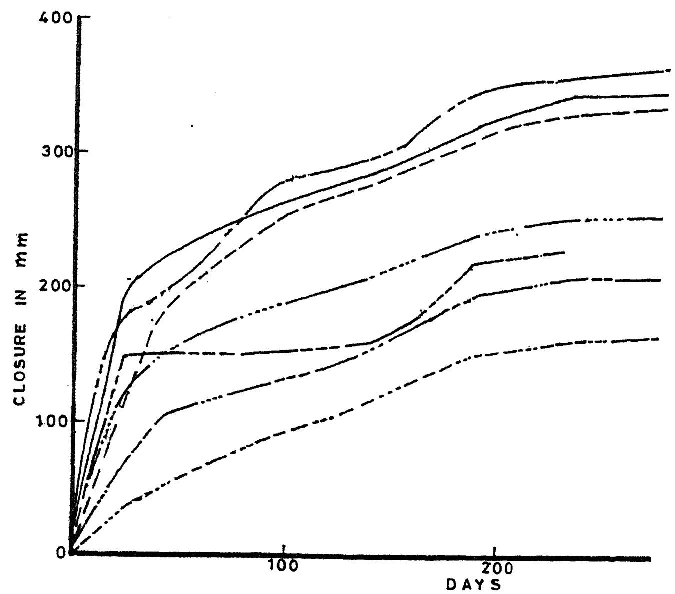

Fig.4 Giri Project - Closure vs. Time for a Zone with Loose Packing

Analysis of Data

Using the Rock Mass Classification Systems and the correlations that go with them, the following magnitudes of rock loads are calculated:

i) Terzaghi's Theory, rock load = 2.31 to 4.41 kg./cm².

ii) Bieniawski's RMR System, RMR = 34, rock load = 0.71 kg./cm²

iii) Barton's System, Q = 0.6875, rock load = 4.5 kg./cm².

iv) Using the mathematical model described above, the rock load corresponding to a radial deformation of 28.56 cms; = 1.9 kg./cm².

v) Actually observed load = 2.0 - 2.4 kg./cm².

YAMUNA HYDRO-ELECTRIC PROJECT

General Features

Yamuna Hydro-electric Project located in the foot-hills of the Himalayas in Uttar Pradesh is being constructed in four stages. The Stage II, Part II of this scheme envisages utilisation of 64m. drop available between tail race of Chhibro underground powerhouse and Khodri Powerhouse. A 7.5m. dia (finis and 5.6 km. long tunnel has been construct for carrying water from Chhibro to Khodri Powerhouse. The excavation of the tunnel has been done through i) a shaft at Chhibro which goes down to about 40m. below the river bed, ii) an approach adit at Khodri and iii) through an inclined adit at Kalawar which was initially excavated for the purpose of investigation of intra thrust zone material characteristics.

Geology of the Head-race Tunnel

According to earlier geological investigations, the tunnel was to pass through bands of sand-stones, siltstones and claystones on the down-stream side, and quartzites and slates on the upstream side. In between, these two forma-tions, an intra-thrust zone of about 300 metre width, bounded by Krol and Nahan thrusts, comprising sheared red shales and 'sabathu' clays was to be met with along the tunnel alignment.

As the work of excavation of the tunnel progr-essed, the red shales of intra-thrust zones were encountered again and again due to tear faulting which had uplifted the 'Sabathu' shales along the tunnel alignment.

Ground Behaviour in the Intra-thrust Zone

The Kalawar inspection gallery was excavated for the investigation of intra-thrust zone material. It was later used as an adit for excavation. The gallery starts in Nahan sand-stones, where the steel supports of 152mm x 152mm sections with 175 mm x 90 mm channel laggings were provided with wooden struts for back packing. The Nahan thrust was exposed in the gallery at chainage 164 to chainage 240. Twisting and inward displacement of ribs was reported from this region after a period of about about a month and a half. The supports were strengthened initially by providing struts, and later by changing the shape of supports to a circular one and adding heavier sections to increase their moment of inertia. In certain reaches 152mm x 152 mm H sections with steel plates of 20mm thickness welded on its flanges have been placed at a spacing of 400 mm. centre to centre.

Field Measurements in Red Shales

The Central Mining Research Station, Dhanbad, carried out measurements of load on steel supports, closure of tunnel surface and borehole extension in a 3.0m diameter section of the head race tunnel in red shale at chain-age 15m. towards Chhibro, from the junction of Kalawar adit and head race tunnel.

The load cells recorded an equivalent pressure of 2.3 - 4 kg./cm². A maximum closure of 94mm. along vertical axis and 45mm along horizontal axis at this section was recorded. It was observed that the time-load and time-closure curves were somewhat identical in nature. This prompted them to plot the load-closure curve and give a linear relationship between load and closure.

$$y = 0.2x + 1.75 \qquad (20)$$

where y is the load in tonnes and x is the closure in mms.

The borehole extensometers showed an extension of about 30mm. The depth of anchors was only 3 metres in this study which is just equal to the tunnel diameter, hence a limitation. Fig.5 shows time-closure relationship.

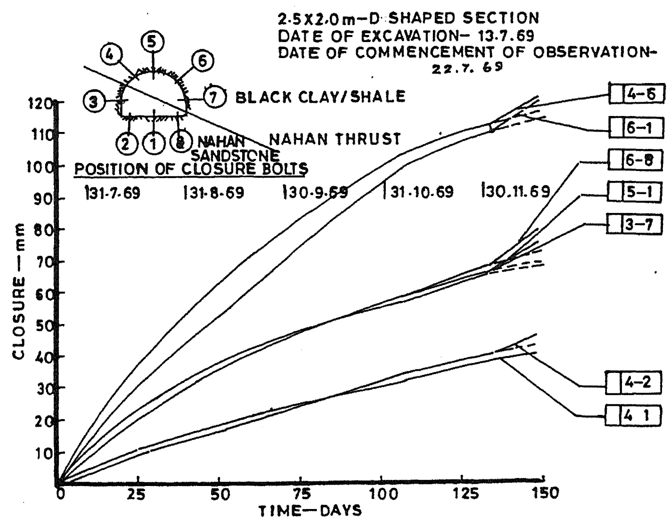

Fig.5 Yamuna Project : Time-Closure Relation-ship

Rock Mass Properties

TIWAG radial jacking test was conducted in red shales in the 3.0m diameter tunnel. Pressures of 3.0, 5.0 and 7.0 kg/cm² were applied in the increasing order. Somewhat erratic deformations on the sides were observed. The average modu-lus of the deformation of the rock mass on the basis of the test was worked out as 0.1×10^5 kg/cm².

Flat jack tests were conducted in the cross cut portion of the gallery in black clay portion of the formations. These clays are similar to shales in their behaviour and properties. The modulus of deformation obtained from this test was 0.061×10^5 kg./cm².

Other properties of red shales are as follows (Jethwa 1981).

i)	Unit weight	2.73 g/cc
ii)	Unconfined compressive strength	21.0 kg/cm²
iii)	Shear strength	c = 1.0 kg/cm²; Ø = 8.40°

Rock Loads and Closures

i) Terzaghi's Rock Load Theory ; rock load = 3.3 to 7.4 kg/cm²

ii) Rock Mass Rating = 34; rock load = 1.34 kg/cm²

iii) Barton Rock Mass Quality ; Q = 0.033 ; Rock load = 3.12 kg/cm².

iv) On the basis of mathematical model the rock load corresponding to 97.5mm = 3.73 kg/cm².

v) Actually measured load = 4.0 kg/cm².

MANERI BHALI PROJECT

General Features

Maneri Bhali Hydro-electric Scheme Stage I, harnesses the power potential of water flowing down the river Bhagirathi between Maneri and Uttar Kashi. The project works included the construction of a 8.63 km long, 4.75m diameter circular concrete lined tunnel, besides a concrete dam, an intake cum sedimentation chamber and a powerhouse. The tunnel passes through complex geological formations under rock covers of 30m to 900m.

Regional Geology

Rocks encompassed by the project area are Quartzites, Quartzites interbedded with thin slate bands, chloritic schists and phyllites, metabasics and basic intrusives belonging to the the Garhwal group of rocks. Towards the north and north-east of the project area, the rocks of the Garhwal group have a thrusted contact (Main Central thrust) with the Central crystallines. Similarly towards the south and south west of the area, the Srinagar thrust(North Almora thrust) separates the two rocks from the Chandpur group.

The metabasics are bright green or greyish green in colour and are interbedded with phyllitic and sericitic quartzite which varies in thickness from 10 cms to 25 m. The bedding in the rocks is demarcated by lithological variations (argillceous and arenaceous components). The metabasics are in a highly folded condition. A major assymmetrical anticline is exposed at the top. The resident geologist at the site had prepared three-dimensional logs of the geology of the tunnel as actually revealed. Taking the orientation of joints from these logs and using stereographic projection techniques, poles have been plotted to see if the weaknesses have any preferred orientation. The strike of most of the weakness planes is nearly the same, but the dips are different. This is what would be expected in a folded stratum of this nature.

Properties of Rock Material and Rock Mass

A number of plate jacking and flat jack tests have been conducted in the drift. The average modulus of the rock mass (secant value) is of the order of 0.5×10^5 kg/cm². The laboratory tests on rock samples have given a modulus of elasticity of 3.6×10^5 kg/cm² and a poisson's ratio of 0.17. The unconfined compressive strength of the rock material was 316 kg/cm² and triaxial shear tests on rock material have shown c = 75 kg/cm² and ϕ = 43.5°.

Ground Behaviour on Tunnelling

The tunnel is constructed by the conventional drilling and blasting technique. Steel supports of 150mm x 150 mm H Section have been provided at a spacing of 0.3m, 0.5m and 1.0m depending upon the judgement of the engineer at the site. Precast concrete laggings have been provided in between and the remaining space has been filled back with lean concrete. The system started showing distress in course of time when the steel supports started twisting and buckling. As the foundation provided for the steel girders was nominal, not much resistance was offered to the converging movement. At some places a horizontal strut was welded to increase its stiffness. These girders got sheared off in course of time.

Closures and rock loads have been measured at a number of locations; for the present purpose, we would consider a reach in metabasics which is located between chainages 1000-1200 from Heena adit. The observed closures are plotted and shown in Fig. 6. It is seen that even after a closure of about 380 mm, the system has still not stabilised. A rock pressure of the order of 4 kg/cm² has been recorded.

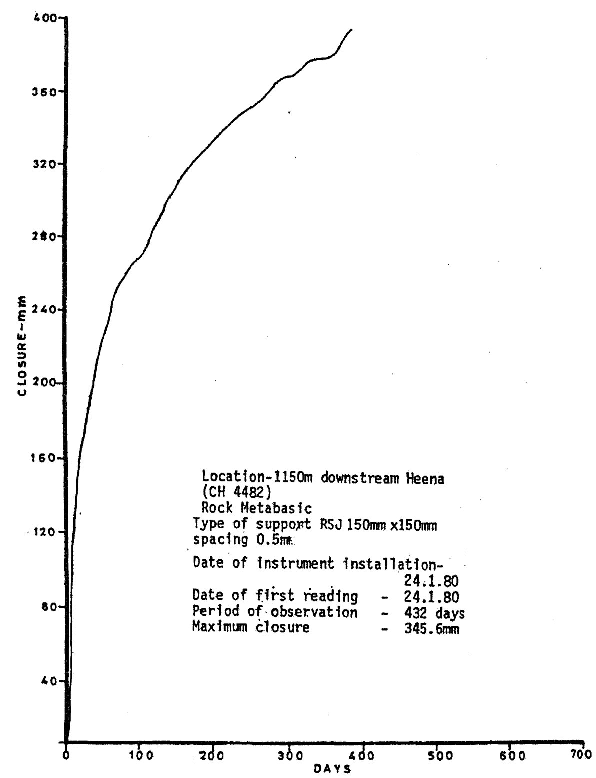

Fig.6 Maneri Bhali Project - Closure Observations at Chainage 1150

Rock Loads & Closures

i) Terzaghi's theory,
rock load = 2.64 kg/cm²

ii) Rock Mass Rating = 39, rock load=0.8kg/cm²

iii) Barton's Rock Mass Quality,Q = 0.5,
rock load = 1.26 kg/cm².

iv) Using the mathematical model, the rock load corresponding to convergence of 380 mm = 7.9 kg/cm².

v) Actually measured rock load 4.0 kg/cm^2
(with convergence continuing)

CONCLUSIONS

Comparison of data regarding performance of three tunnels through squeezing grounds, shows that convergence-confinement method offers a better choice in the estimation of radial closures and rock loads. With appropriate modifications the existing methods of evaluation of ground convergence characteristics can be made more effective.

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