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Squeezing Problems in Indian Tunnels

J. L. Jethwa
Central Mining Research Station, Dhanbad, India

A. K. Dube
Central Mining Research Station, Dhanbad, India

B. Singh
Central Mining Research Station, Dhanbad, India

Bhawani Singh
Central Mining Research Station, Dhanbad, India

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J.L. Jethwa

Assistant Director, Central Mining Research Station, Dhanbad, India

A.K. Dube

Assistant Director, Central Mining Research Station, Dhanbad, India

B. Singh

Director, Central Mining Research Statin, Dhanbad, India

Bhwani Singh

Professor, Department of Civil Engineering, University of Roorkee, India

SYNOPSIS Case histories of three Indian tunnels indicate that squeezing conditions are created due to plastic flow of rock masses under the influence of high cover pressures. These examples emphasize that a tunnel experiencing squeezing conditions must be allowed to deform to optimize support costs and avoid delays. Allowance for desirable tunnel deformations must, therefore, be made while planning the size of excavation.

Field data has shown that a flexible support system of compressible backfill and steel ribs may be used as an alternative to shotcrete support which is unpractical in Indian tunnels excavated largely by conventional methods. Instrumentation indicates that large broken zones are associated with late stabilization and that the coefficient of volumetric expansion of failed rock masses is significantly lower than believed so far. Comparison of measured rock pressures with those estimated from available methods shows that the elasto-plastic theory may provide reliable predictions provided that the strength parameters of rock masses are known precisely.

INTRODUCTION

A majority of hydropower projects in India are located in the Himalayan region where the rock masses are young and disturbed due to tectonic activities. Construction experience during the last decade in this region has shown that commissioning of these projects is often delayed due to unforeseen problems of tunnelling when the weak rock masses fail under the influence of high cover pressure. These failing rock masses squeeze into the tunnel openings and undergo large deformations. The customary support system, consisting of steel ribs with lean concrete backfill, attempts to curb these displacements and attracts high rock pressures. These supports have buckled in many cases to such a large extent that the tunnels required rectification to provide space for the concrete lining. The inevitable results are cost escalation besides delays.

CASE HISTORIES

Case histories of Chhibro-Khodri, Giri-Bata and Loktak hydroelectric tunnels are described in brief to highlight the supporting problems under squeezing rock conditions.

Chhibro-Khodri Tunnel

Geological section of the 5.6 Km long and 7.0 - 7.5m diameter Chhibro-Khodri tunnel is shown in Fig. 1. The case history of this tunnel has been discussed in detail elsewhere (Jethwa et al, 1980). The main structural features of the area are two main boundary faults, locally called as the Nahan and the Krol thrusts. These thrusts have been repeated thrice across this tunnel due to the presence of a series of tear faults. The intra-thrust zones consist of soft and plastic black clays and crushed, brecciated and sheared red shales with pockets of siltstones.

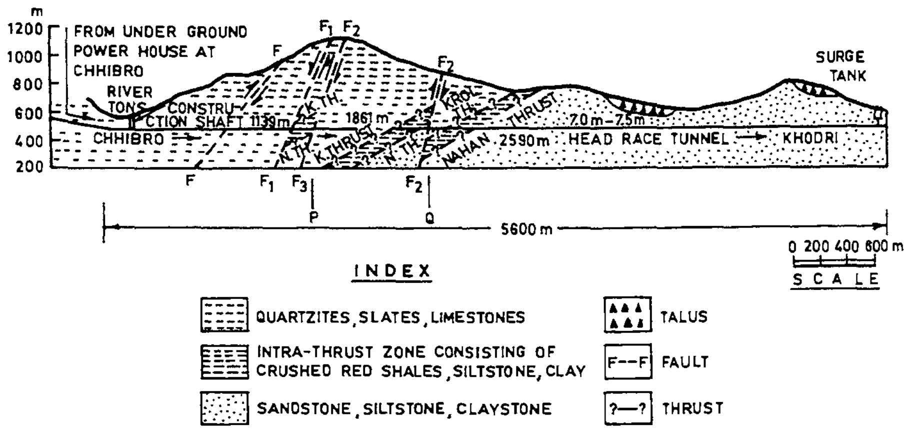

Fig. 1 - Geological section of Chhibro-Khodri tunnel

Tunnelling was started from the two ends at Chhibro (inlet) and Khodri (outlet). Two additional faces were opened within the first intrathrust zone (towards Khodri) through an incline called Kalawar Inspection gallery. A 3.0m diameter pilot tunnel was driven on both sides at the invert level of the main tunnel which was proposed to have 9.0m excavated diameter in this zone to accommodate 75-90 cm thick concrete lining.

Supporting problems at Khodri end

Steel ribs for the Kalawar Inspection gallery passing through the intra-thrust zone under a cover of 280m were first designed for Terzaghi's rock load factor of 1.1 $(B+H_t)$ where B is the width and H_t is the height of the opening ($B=2.0m$, $H_t = 2.5m$) corresponding to squeezing rock at 'moderate depths'. To arrest excessive rib deformation, the rock load factor was gradually increased to 3.5 $(B + H_t)$ corresponding to squeezing rock at 'great depths'. Rock pressures of 3 and 12 kg/cm^2 were recorded in the red shales and the black clays respectively in the 3.0m diameter pilot tunnel. The 3.0m section was subsequently widened to 9.0m diameter by heading and bench method and the ribs of 6 and 20 kg/cm^2 capacities were satisfactorily employed in the red shales and the black clays respectively.

Supporting problems from Chhibro end

Red shales were again encountered at 1139m from the Chhibro end at a depth of 600m. Steel ribs of 6 kg/cm^2 capacity, successfully employed at the Khodri end in the similar rock mass, deformed excessively. Excavation was done by heading and bench method. The rib capacity was gradually increased to 12 kg/cm^2 and the excavation of the bench was stopped. Thus, the semi-circular ribs, which did not have any invert support for long periods, were free to deform under the influence of side pressure. Instrumentation indicated that though the rib capacity was increased to 17 kg/cm^2, they failed at 12 Kg/cm^2 due to the absence of the invert support.

Trifurcation of the tunnel

The project was already delayed by over six years due to a very slow rate of tunnelling (5-6m per month) through the two intra-thrust zones At this rate it would have taken additional five and a half years to excavate the remaining 800m between points P and Q (Fig.1) from the two ends The project authorities, therefore, considered it wiser in 1975 to replace the larger tunnel by three smaller tunnels each of 5m excavated diameter. However, since tunnelling through this zone was completed by the end of 1982, this decision of trifurcating the tunnel at extra costs has become questionable.

Giri-Bata Tunnel

The geological section of the 7.1km long Giri-Bata tunnel with a finished diameter of 3.6m is shown in Fig. 2 . Dube (1979) has discussed the case history of this tunnel in detail. The tunn was excavated by conventional drill and blast method and supported by steel ribs. It passed through conglomerates, phyllites, slates, shales quartzites, sandstones, clay stones/siltstones and sandstones from the inlet to the outlet end.

Supporting problems

The phyllites of the infra-Krols proved to be the most difficult tunnelling media. The rib capacity was gradually increased from 1.25 to 5 kg/cm^2 to contain the squeezing pressures with out success. The deformed ribs had to be replaced in a length of about 500m to obtain required clear opening.

Instrumentation of the tunnel indicated that tunnel deformations varied between 2 to 20 percent of the tunnel size within one year of excavation.

Loktak Tunnel

The geological section of the 6.5km long Loktak tunnel with a 3.81m diameter horse-shoe section has been shown in Fig. 3. The tunnel crosses a

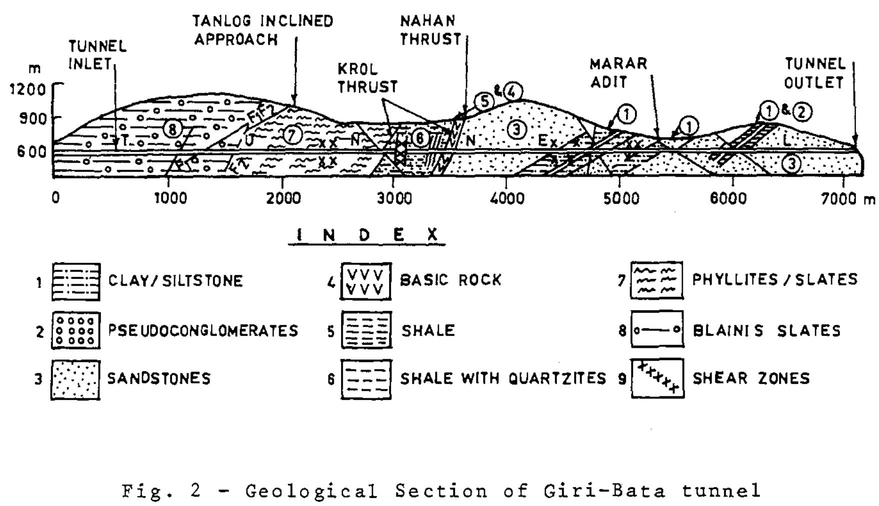

Fig. 2 - Geological Section of Giri-Bata tunnel

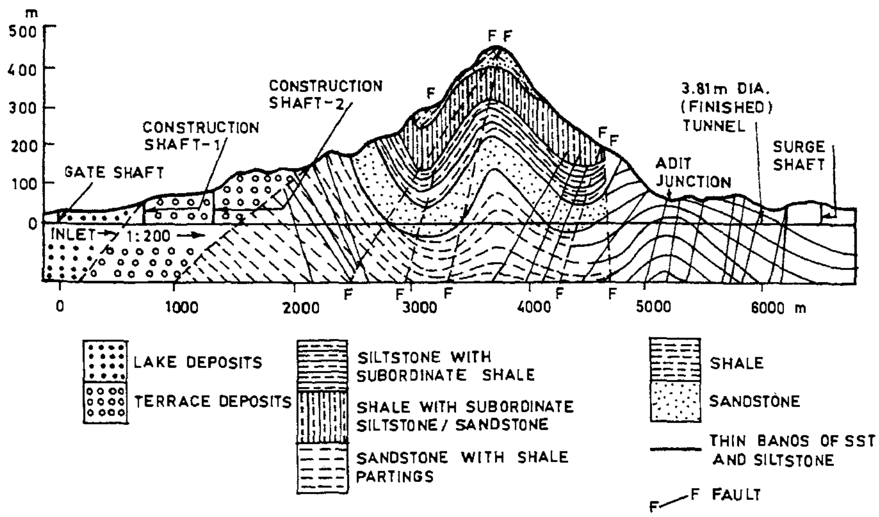

Fig. 3 - Geological section of Loktak tunnel

N-S trending syncline. Thick layers of sandstones and siltstones occur at the trough of this syncline whereas splintary shales with thin bands of sandstones and siltstones occupy the flanks. The axial portion of the syncline has been refolded into several N-W trending cross folds. Both limbs of the fold have been affected by the presence of a number of faults. A detailed account of the tunnelling problems has been given by Golser et al (1980).

Supporting problems

Tunnelling by conventional method was started in 1970. The shales started squeezing under a cover of less than 200m. The rib deformation increased with the increase in the cover. The deformations at 400m depth were so high that reexcavation became necessary. Jethwa et al (1979) recommended to increase the excavated diameter of the tunnel by 10-15cm to provide for the deformations. The New Austrian Tunnelling Method was adopted for the first time in Indiato quicken the rate of tunnelling which was as low as 6-10m per month in this queezing zone. The support adopted initially consisted of 3m long rock bolts at 1m spacing with 15cm mesh-reinforced shotcrete. The deformations with this support system were 20cm per day soon after excavation. At places, the bottom heaved by 1.8m and the shotcrete sheared in the lower wall portion. The shotcrete support was subsequently supplemented successfully by 150x 150mm steel ribs at 1m spacing.

It may be seen from these case histories that the supporting problems were due to occurrence of the squeezing conditions and the attempts to contain tunnel deformations by use of stronger supports.

APPLICATION OF ELASTO-PLASTIC THEORY

Intolerable rib deformations under squeezing conditions in these tunnels are the result of the plastic flow of soft rock masses under the influence of cover pressure. The support philosophy under such conditions is best explained by the ground-reaction-curve concept derived from the elasto-plastic theory. This concept advocates the use of flexible supports which allow the failing rock mass around a tunnel to deform under controlled conditions. The rock mass itself is made to act as support and the use of artificial support is brought down to the minimum in the process.

Theoretical ground reaction curves for the red shales of the Chhibro-Khodri tunnel, the phyllites of the Giri-Bata tunnel and the shales of the Loktak tunnel are shown in Fig. 4. The strength parameters (c and Ø) were estimated by the geomechanical classification system of Bieniawski (1973). It can be seen in Fig. 4 that attempts to curb tunnel deformations by the use of rigid supports are bound to attract very high

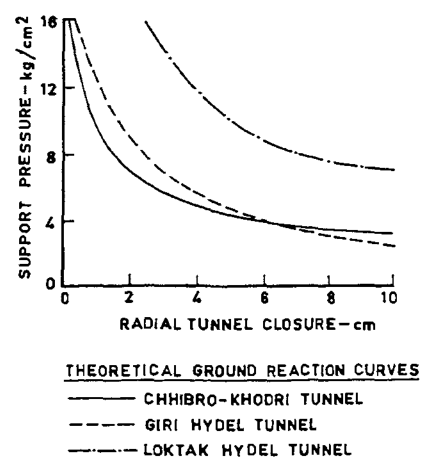

Fig. 4 - Theoretical ground reaction curves

rock loads. The required support capacity decreases with tunnel deformations. Thus, rectification operations could be avoided if an allowance for desirable tunnel deformations are made while planning the excavated size of a tunnel likely to experience squeezing conditions. In case the squeezing is not foreseen, the excavated size must be increased at the first signs of squeezing.

APPLICABILITY OF NATM IN INDIA

The New Austrian Tunnelling Method (NATM) has proved to be ideal in the squeezing problems in many countries. However, the excavated periphery of a tunnel should be even for its economic application. The periphery of a majority of the Indian tunnels remain uneven as these are excavated by the conventional drill and blast method. As such, successful adoption of the shotcrete support of the NATM in Indian tunnels is questionable. In order to develop an alternative to the shotcrete support, trials were made with a flexible support system consisting of steel ribs and a 'compressible backfill' in the Chhibro-Khodri tunnel. This backfill consisted of loosely packed tunnel muck around the steel ribs. The shuttering over the ribs was provided with 75mm thick pre-cast reinforced cement concrete sleepers. Test-sections with such a compressible backfill and the rigid backfill of lean concrete were established both in the red shales and the black clays. Rock pressure on the steel ribs at these test-sections was measured. These measurements, plotted in Fig. 5, indicate that the compressible backfill reduces the rock pressure to a considerable extent. This result agrees with the observations of Lane (1957) in the Garrison dam tunnels. Encouraged by these results, the compressible backfill was adopted successfully in a length of 4km in the Giri-Bata tunnel.

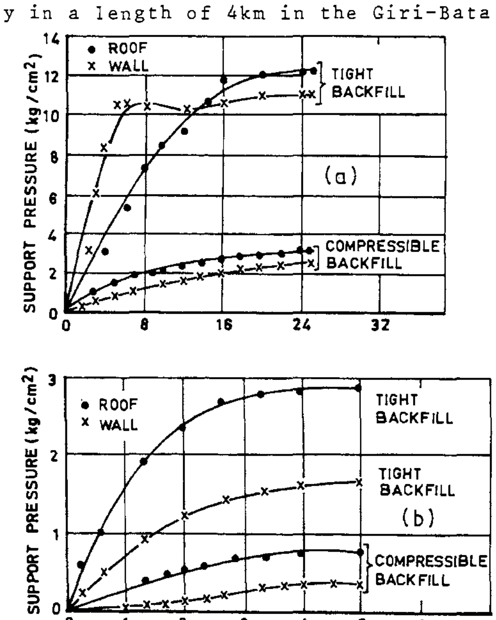

Fig. 5 - Indluence of compressible backfill on support pressure in
(a) soft and plastic black clays
(b) Crushed and brecciated red shales

EVALUATION OF METHODS FOR PREDICTING ROCK PRESSURE

Starting with Terzaghi (1946), many classification systems have been developed in various parts of the world to predict the rock pressure for tunnel-support design. The few important methods are the Protodyakonov's (1963) system, the RQD based approach of Deere et al (1969), the RSR method of Wickham et al (1972, 1974) and the Q-system of Barton et al (1974, 1975). However, these methods have not been evaluated by field measurements for their suitability under squeezing rock conditions. Jethwa et al (1981,1982) employed these systems and the elasto-plastic theory to predict rock pressures for various reaches of the three Indian tunnels. The geological details for estimating the Rock Mass Quality Q, the Rock Mass Rating RMR and the Rock Structure Rating RSR were collected at sites. The estimated values of Q and RMR for the Indian case histories have been superposed on those given by Bieniawki (1979) in Fig. 6. It may be seen that the Indian case histories also agree with Bieniawski's correlation. This agreement gave encouragement to the authors in picking up various parameters of the classification systems.

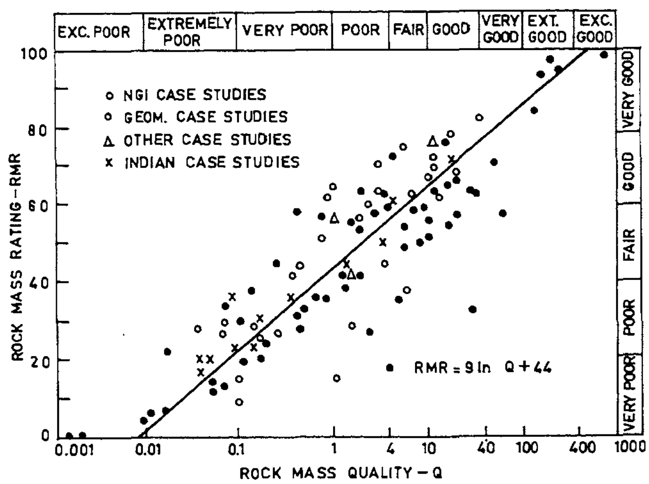

Fig. 6 - Correlation between RMR and Q (after Bieniawski, 1979; Indian case histories added by Jethwa, 1981).

The predicted rock pressures have been compared with the observed values in Fig. 7. It may be seen that Terzaghi's method is unsafe for small tunnels and conservative for a large tunnel, probably because the rock pressure was assumed to vary directly with the tunnel size by Terzaghi. Protodyakonov's approach is seen to be consistently unsafe. The Q-system, surprisingly, predicted the rock pressures within a range of 2-4 kg/cm^2 against the measured pressure being as high as 12 kg/cm2, indicating that the Q-system is not reliable for squeezing conditions. Barton (1980) agrees with this observation.

The geomechanics classification system of Bieniwski (1973) was used to obtain a range of c and values of rock masses for applying the elasto-lastic theory. It can be seen in Fig. 7 that lthough the average values of the predicted ock pressures agree reasonably well with the easured values, the predicted ranges of press-res are too wide because the strength parame-ers of rock masses were not precise. It is, hus, necessary to have precise value of rock lass strength for successful application of the lasto-plastic theory.

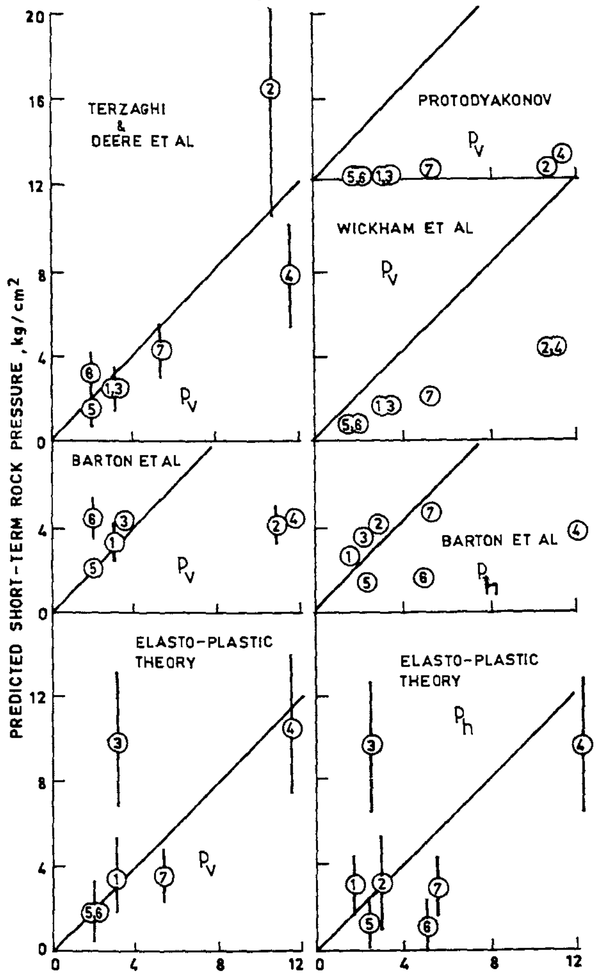

Fig. 7 - Comparison of predicted and observed rock pressures
CHHIBRO-KHODRI TUNNEL
Red Shales - (1) 3m dia (2) 9m dia
Black clays (3) 3m dia (4) 9m dia
GIRI-BATA TUNNEL, 4.2m dia
(5) Slates (6) Phyllites
LOKTAK TUNNEL
(7) Shales, 4.8m dia

DATA OF INSTRUMENTATION

The three Indian tunnels were instrumented to observe rock loads on steel ribs by load cells and contact pressure cells, tunnel deformations by tape extensometers and deep seated displacements around the tunnel periphery by borehole extensometers. The displacement data was plotted to obtain the size of the plastic or broken zone as explained by Dube et al (1982). These plots were also used to compute the coefficient of volumetric expansion k of the failed rock masses.

Stabilization of broken zones

It was observed that the broken zone expanded with face advance. The non-dimensional face advance z/a, where z is the face advance and a is tunnel radius, has been plotted against the radius b of the fully developed broken zones in Fig. 8. It is pertinent to note that the face advances for the stabilization of the broken zones varied widely between 2 and 40 times the tunnel radius. However, the limiting face advance is seen to be 4 times the radius of broken zone b. Hence, attempts to minimize support capacity by allowing larger broken zones are likely to delay the stability of the tunnel opening. As such, the placement of concrete lining must also be delayed.

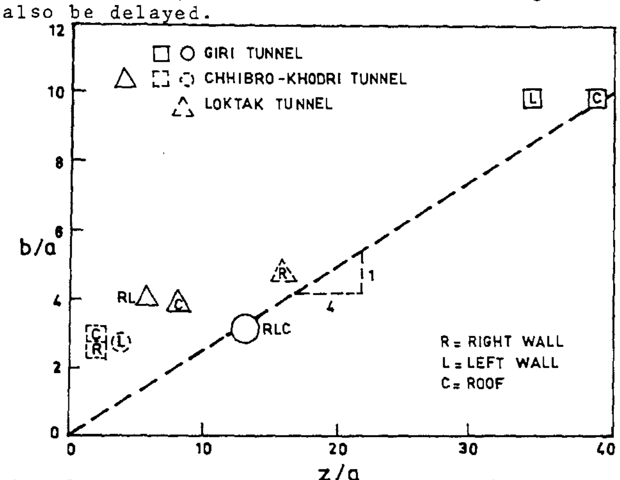

Fig. 8 - Influence of broken zone radius b on time of stability of tunnel

Coefficient of volumetric expansion

The values of coefficient of volumetric expansion k (defined as the increase in volume of the failed rock mass to its original volume) of some of the commonly occurring soft rock masses prone to squeezing are given in Table I. These k values are two orders of magnitude lower when compared to the k values of 1 - 1.5 suggested by Labasse (1949). Daemen's (1975) range of 0.1 - 0.15 is also relatively higher than the observed values. Thus, the desirable tunnel deformations are likely to be smaller than those estimated after Labasse (1949) and Daemen (1975).

Table I - Measured k values in Indian tunnels

Rock Mass	k
highly jointed phyllites	0.003
Soft sandstones	0.004
Crushed and sheared shales	0.005
Soft plastic clays	0.01

CONCLUSIONS

Case histories of the three Indian tunnels demonstrate that the squeezing of soft rock masses was responsible for intolerable rib deformations, tunnel rectifications and consequent delays. Allowance for desirable tunnel deformations must therefore be made while planning the excavated size of a tunnel if squeezing conditions are foreseen.

The use of flexible backfill with steel rib supports, as an alternative to the shotcrete support of the NATM, has shown promise for tunnels constructed by the conventional method under squeezing rock conditions.

The elasto-plastic theory has shown potential to provide reliable estimates of rock pressures under squeezing conditions provided that precise values of strength parameters (c and Ø) of rock masses are known.

Instrumentation has shown that the stability of a tunnel under squeezing condition is delayed if larger broken zones are mobilized to optimise support requirements. This observation helps in deciding the time of providing the concrete lining.

The data also indicates that the values of coefficient of volumetric expansion vary within a range of 0.003 to 0.01 which is significantly lower than believed so far.

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