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Case History of Maneri-Uttarkashi Power Tunnel

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SYNOPSIS : A 8.56 km long circular tunnel of 4.75 m diameter has been constructed under Maneri hydel project on the river Bhagirathi. The tunnel passes alternatively through quartzitic and metabasic rock formations of the young Himalayan terrain. Tunnel excavation was started from four faces, one at the upstream end at maneri, two from an intermediate adit at Heena, and one at the downstream end near Uttarkashi where a 80 MW surface powerhouse is located.

In excavating the tunnel from different faces, the problems of tunnel face collapse with or without heavy ingress of water, cavity formations and large tunnel closures leading to buckling of steel ribs on account of squeezing ground conditions were encountered. In the paper the approach of combating these problems has been dealt in detail.

The predicted values of support pressure obtained from Terzaghi, Barton et al and CMRS approaches have been compared with the observed values of support pressure. The CMRS approach shows promise for better results in both squeezing and the elastic ground conditions.

INTRODUCTION

A large number of tunnels have been located to tap the vast power potential of rivers flowing from the Himalayas. The Himalayan mountain ranges contain rock formations of different ages and represent a mixed lithology and tectonic activity. This bounty of nature is therefore associated with several tunnelling problems e.g. sudden inrush of water, roof collapses, occurrence of methane gas, squeezing and swelling ground conditions etc.

A 8.56 km long circular tunnel of 4.75 m finished diameter has been constructed through such a geological set-up under Maneri-Bhali Hydel Scheme Stage-I on the river Bhagirathi, a tributary of the mighty Ganga to utilise a head of 184 m for generating 80 MW of power. The tunnel passes alternatively through quartzitic and metabasic (chlorite-schist) rock formations of the young Garhwal-Himalayan terrain. The tunnel excavation was started from four faces, one at the upstream end at Maneri, two from an intermediate adit at Heena, and one at the powerhouse end near Uttarkashi, a place of pilgrimage.

REGIONAL GEOLOGY, TUNNELLING PROBLEMS AND REMEDIAL MEASURES

Regional Geology

The regional geology of the area has been described by Jain et al (1976). The rocks exposed in the area are quartzites, quartzites interbedded with thin bands of slate, chlorite schists, phyllites, metabasics and basic intrusives belonging to the Garhwal group. Towards the north and north-east of the project area, the formations of the Garhwal group have a thrusted contact (main central thrust). Similarly the Srinagar thrust (north Almora thrust) seperates these formations from the Chandpur group towards the south and south-west.

The general strikes of the rock masses in different locations are variable. However, some generalisations are made below :

Maneri area : Strike N15^{o} to 20^{o} - S15^{o} to 20^{o}E; dip 25^{o} to 35^{o} towards S70^{o} to 75^{o} W directions

Heena area : Strike N40^{o}E - S40^{o}W; dip 30^{o} to 45^{o} in S 50^{o} W direction

Tiloth end : Strike N20^{o}W - S20^{o}E to N20^{o}E - S20^{o}W ; dip 35^{o} to 45^{o} in NE to SE direction

The surface geological mapping of the area between Heena and Tiloth has been done by Jain et al (1976). The individual rock units as observed in the area from north to south are described below :

Quartzites	
Metabasics	Garhwal Group
Quartzites with minor slate bands	

These lithounits have been intensely folded and faulted due to high tectonic disturbance. The tectonic activity in the area has developed close joints, brecciation and shearing even in quartzites, which are considered competent.

The longitudinal geological cross-section along the original straight alignment and along the alternative alignment No.2 between Heena and

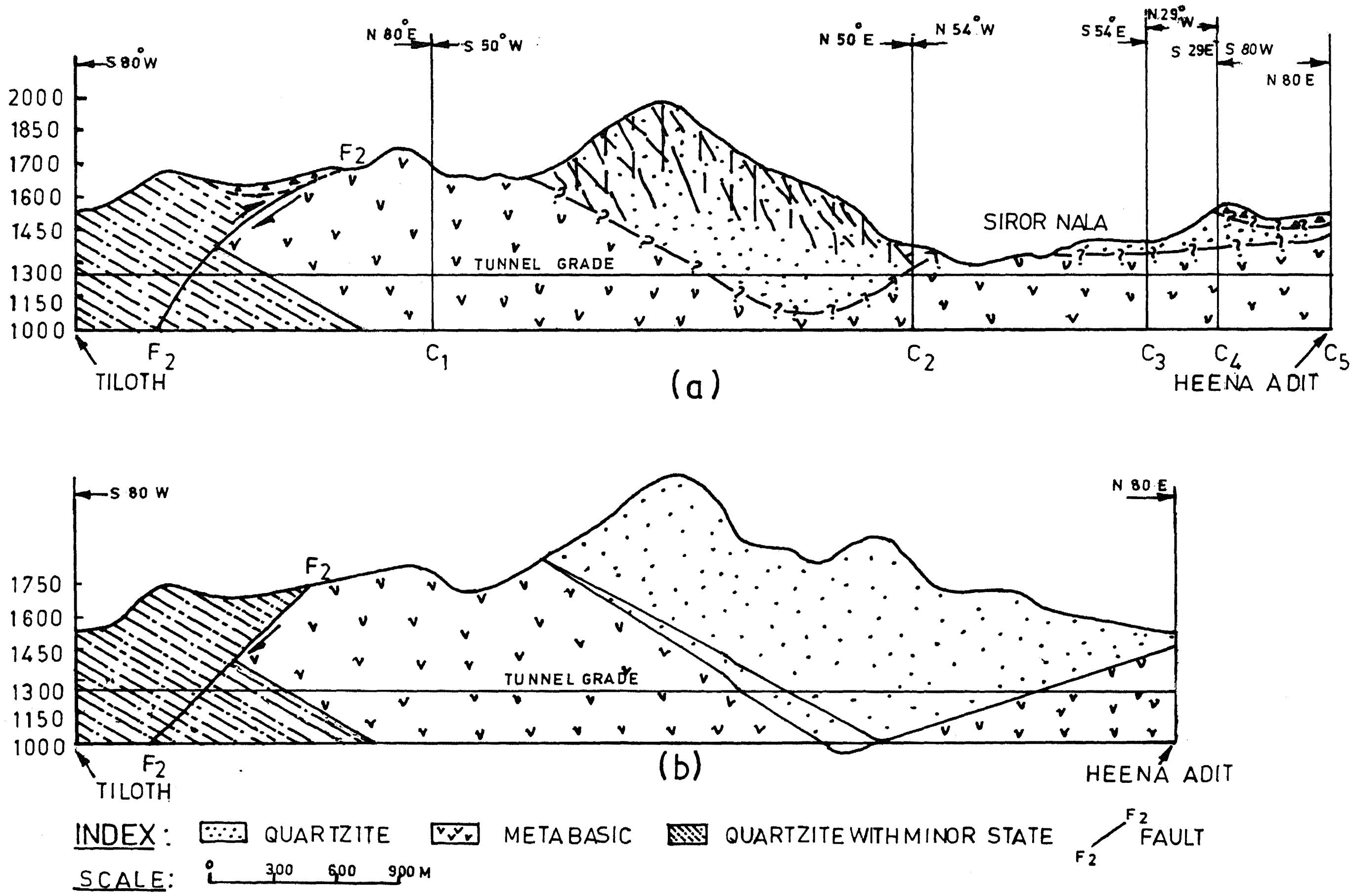


Fig.1 Geological Sections Between Heena and Tiloth along (a) Alternative Alignment No.2 and (b) The Straight Tunnel Alignment (after Jain et al, 1976)

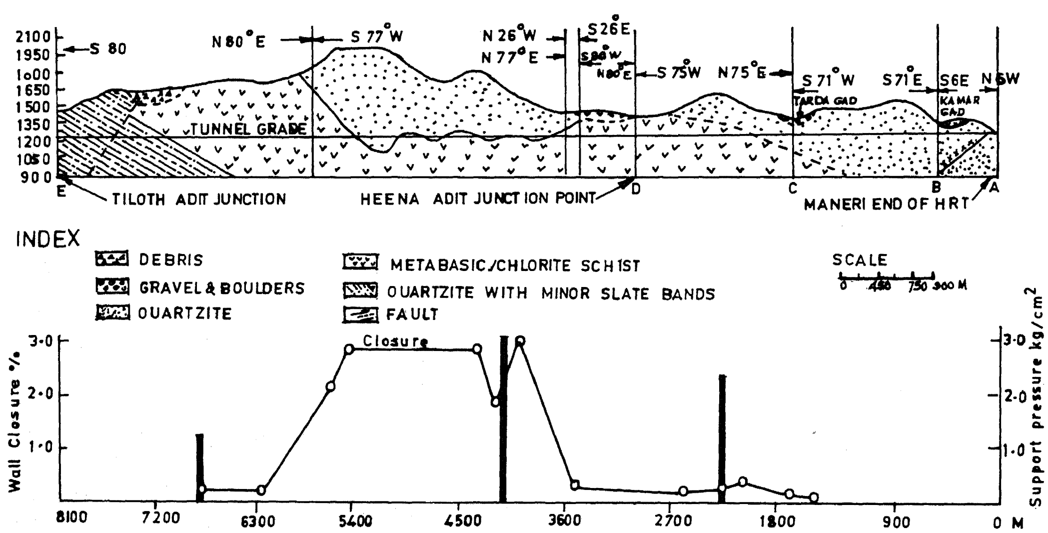


Fig.2 Longitudinal Geological Cross-Section Along the Power Tunnel (after G.S.I.)

Tiloth has been shown in Fig.1, while the longitudinal section along the final alignment has been shown in Fig.2.

Tunnelling Problems and Remedial Measures

In excavating the tunnel from different faces, a number of problems were encountered. The major problems were tunnel face collapses with or without heavy ingress of water, cavity formations and large tunnel closure leading to buckling of steel ribs on account of squeezing ground conditions.

Water inrush, ch.3549 m

During tunnel driving on the downstream side of the intermediate adit at Heena, the tunnel face collapsed suddenly at ch 3549 m on October 19th, 1974 with a fall of about 300 cum of loose rock with heavy inrush of water at a rate of 6 cusecs. This was followed by sliding of 200 cum of muck exactly two months later on 18th December. The ingress of water, which started at a rate of 6 cusecs in October 1974, stabilised at a rate of 1.28 cusecs in February 1975.

The tunnel passes through basic-metabasic-chlorite-schist type of rocks upto ch 3530 m. Beyond this place the tunnel passed through jointed and blocky quartzites folded in a synclinal form Fig.2. The quartzites occurring in the core of the syncline were bounded by relatively impervious metabasic chlorite-schist formations on both the sides. The quartzites occurring in the syncline were, thus, heavily charged with water, the water head being 80 m. Two cross shear zones of about 40 cm width were intersecting close to the crown of the tunnel (Fig.3). The presence of these two shear zones, their intersections at the tunnel grade and the presence of a water reservoir inside the hill triggered the collapse of the tunnel face and the sliding of a huge quantity of muck with heavy inrush of water. The washings of the drill holes indicated that the material in the shear zones was not groutable with ordinary portland cement.

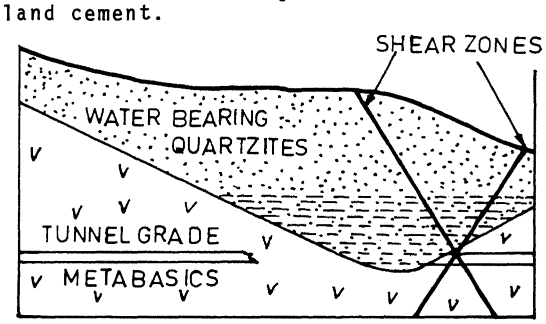


Fig.3 Detailed Sketch Showing the Causes of Water Inrush

It was decided to divert the tunnel alignment slightly to the right side at a distance of about 50 m behind the collapsed face, i.e. ch 3495 m, with an objective to keep the tunnel alignment through the metabasic chlorite schists parallel to the contact plane and then to investigate by deep drilling a suitable place to enter into the quartzite zone and follow the original alignment. It was ascertained that after driving parallel to the contact plane for a distance of about 75 m the quartzites would be suitable for taking grout. It was considered that the tunnel could be driven by umbrella grouting in the crushed quartzite charged with water. The tunnel was, therefore, turned towards the original alignment and the tunnel excavation was continued cautiously with advance probe holes ahead of the tunnel face (Fig 4). When the face reached 5 m from the contact plane, umbrella grouting was done through 16 numbers of 20 m deep and 75 mm diameter drill holes along the periphery of the tunnel face. These holes were inclined at 10^{0} upwards to the tunnel axis. Perforated pipes of 50 mm dia were inserted in the drill holes and the 20 m deep zones around the tunnel was grouted with cement in stages using packers, at a pressure of 35 kg/cm^2 (Fig 5). The probe holes made after grouting indicated that the grouted zone had become a solid mass and there was no in-rush of water through the holes. The tunnel was then driven without difficulty leaving a bulkhead of 5 m in front. While grouting the first group of 16 holes, it was found that the holes in the bottom half

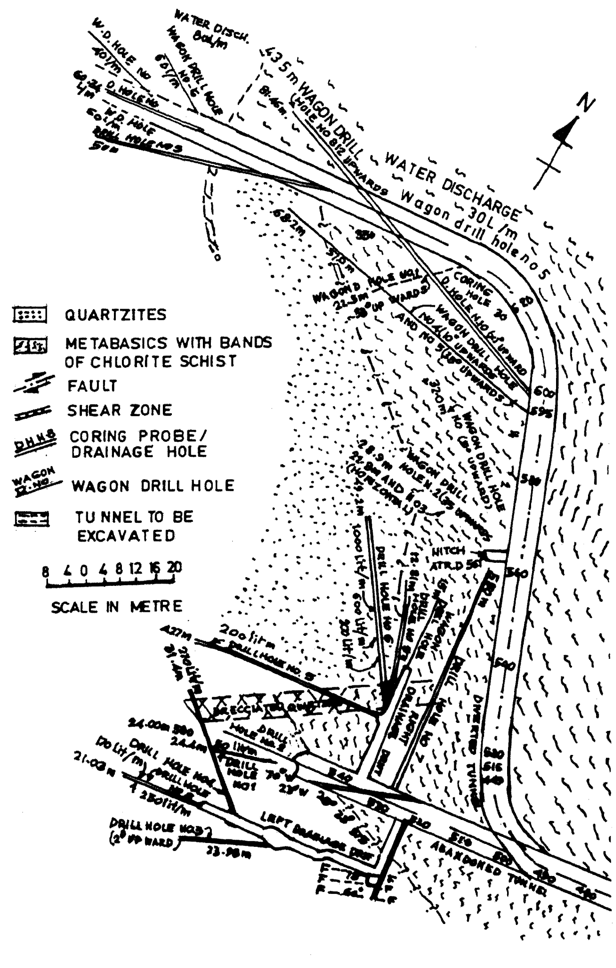
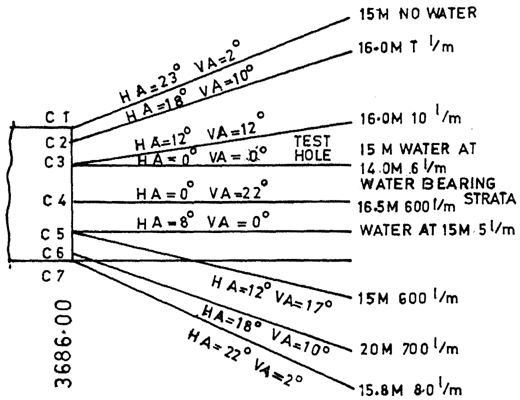
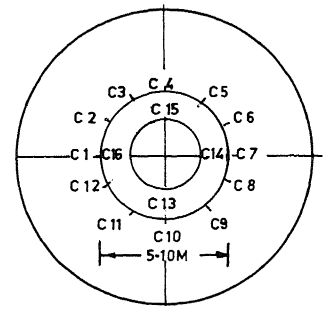


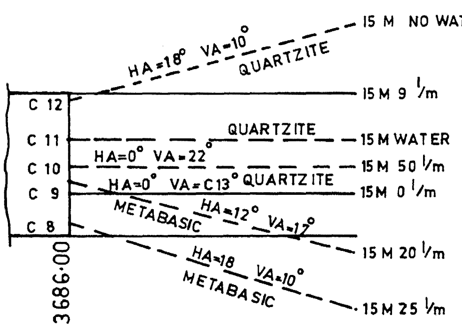
Fig.4 Plan at EL 1266 m Showing the Details of Exploration Work



GROUT HOLES ABOVE SPRINGING LEVEL



POSITION OF WASON DRILL HOLES



GROUT HOLES BELOW SPRINGING LEVEL

DETAILS OF GROUTING	
HOLE NO	CEMENT IN BAGS
C 1	14
C 2	1
C 3	5
C 4	36
C 5	100
C 6	26
C 7	37
C 8	16
C 9	20
C 10	8
C 11	6
C 12	2
C 13	10
C 14	11
C 15	8
C 16	9
TEST HOLE	3

Fig.5 Pattern of Grout Holes

section did not take any grout. Holes only in the top half section of the tunnel were, therefore, drilled and grouted in the subsequent phases of the umbrella grouting operations.

Alternate alignments

The tunnelling operations were stopped beyond ch. 3530 m downstreams of Heena because of sudden in-rush of water and loose rock fall. Despite the continuous efforts for six months, the excavation work at this face remains standstill. It was decided to change the tunnel alignment by diverting the tunnel towards the right side from ch 3530 m.

Three alternate alignments (as shown in Fig.6) for diverting the tunnel towards the right side were proposed with the objective of maximising the rate of tunnelling through a safe and better tunnelling media, i.e. through the metabasic formations. The length of the tunnel in the process increased by 0.47 to 1.85 km but the tunnel length through the highly water charged and fractured quartzites could be reduced by about 280 to 680 m in the case of alternatives I & III and completely avoided in alternative II.

The total tunnel length and the length through water charged quartzites in different alternate proposal are given in Table-I.

The added advantage in alternatives I & II was that two additional working faces could be obtained.

Cavity formation, ch.5038-5055 m

A number of small and big cavities were formed during the excavation of the tunnel. A major cavity was formed during excavation between ch. 5038 - 5050 m. The tunnel grade at ch 5050 crossed a shear zone of crushed quartzites heavily surcharged with water. The total volume of the cavity was estimated as 813 cum. Mucking had to be stopped because of continuous inflow of the muck from the top of the muck pile.

The face was sealed after fore-poling with rolled steel joists. Drainage holes were provided on both sides of the tunnel to drain the seepage water. The cavity above the forepoles was then filled with concrete and grouting was done to control the flow of water and to consolidate the muck. The quantities of concrete and grouting were about 67 cum and 3295 cement bags (each of 50 kg.) respectively. This attempt did not prove effective and, therefore, a side drift was excavated on the left side of the tunnel for draining the seepage water to

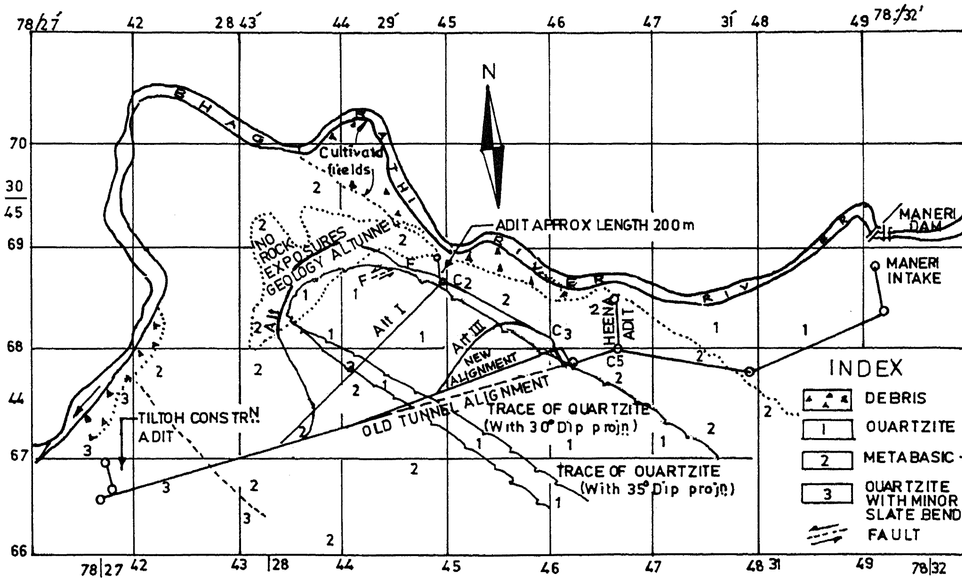


Fig.6 Plan at EL 1250 m Between Maneri and Tiloth Showing Alternative Tunnel Alignments

Table I

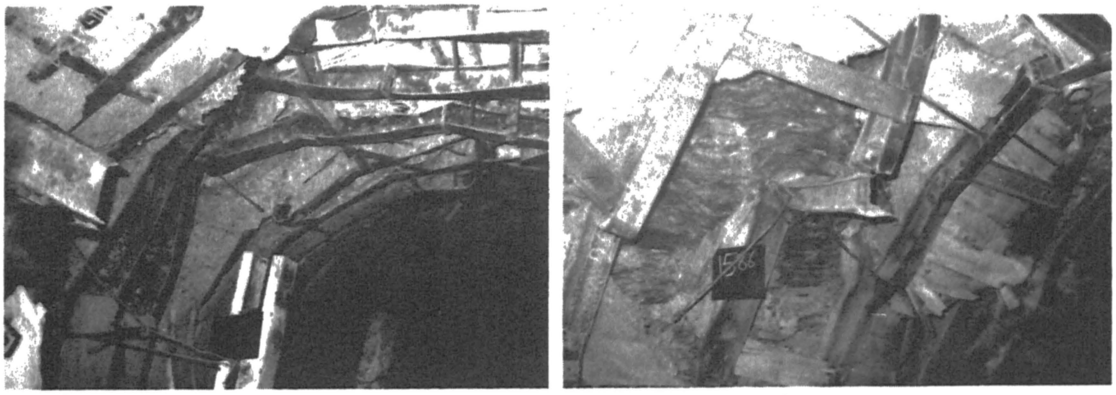
Sl. No.	Proposed layout	Total Tunnel length between Heena and Tiloth, m	Tunnel length through water charged Quartizites, m	Increase in tunnel length, m
1.	Original	5065	1200	-
2.	Alternative I	5940	920	875
3.	Alternative II	7170	-	2105
4.	Alternative III	5535	920	470

the possible extent. This drift met the centre line of the tunnel at ch 4036 m beyond the shear zone where good metabasic rock mass was encountered. The bulkhead at the face was then opened and excavation started in the heading. But, this attempt did not succeed due to inflow of crushed material with water. The face was, therefore, sealed again and some drainage pipes were provided through this bulk head. The muckpile was then grouted. Tunnel construction in this reach was now accomplished by the multidrift method from the two opposite ends.

Squeezing problems, ch.5250-5550 m

Tunnelling activities at depths varying from 700 to 900 m between ch 5550 m and 5250 m through partially wet and thinly foliated metabasics were beset with high squeezing ground conditions. The tunnel was supported by ISMB 150 mm x 150 mm ribs spaced at 810 mm to 965 mm. Blocking concrete was done upto the outer flange of these ribs. No problem was faced during the excavation or the supporting of this tunnel section. However, after a period of 5-6 months, it was observed that the blocking concrete started cracking and the ribs started deforming due to squeezing pressure as shown in Figs.7(a) and 7(b).

For solving the problem of cracking of blocking concrete and deformation of steel ribs in the squeezing ground the ribs were strengthened with laggings of ISMB 150 mm x 75 mm and blocking concrete was done upto the inner flanges of the ribs. The treatment proved helpful in controlling further deformations of the ribs. However, at the time of providing the final concrete lining in this reach in November-December 1982, nearly three years later, it was found that most of the ribs in this reach had twisted and deformed to such an extent that their removal was considered necessary to obtain the required finished bore. The invert heave was as much as 80 cms. The twisted ribs and the blocking concrete were removed and the rock mass was trimmed and resupported by 150 mm x 150 mm ribs spaced 750 mm apart for obtaining the required finished diameter. This rectification job was completed in the most of the affected length but while removing the twisted ribs between ch 5509 m and 5517 m, there was a heavy rock fall from the roof. A cavity of 430 cum was formed. This cavity was tackled by putting forepoles of 40 mm dia bars, 13 to 15 m long at a spacing of 300 mm centre to centre and grouting the muck above the forepoles at a pressure of 5 kg/cm^2, thus forming a concrete ring as roof support. Thereafter the tunnel in this portion was reexcavated and supported by ISMB 150 mm x 150 mm ribs spaced 60 cm centre to centre.



(a) (b)

Fig.7 (a) and (b) Buckling of Steel Ribs and Damage to the Blocking Concrete

PREDICTION OF TUNNEL BEHAVIOUR

Rock mass classification systems proposed by Terzaghi (1946) and Deere (1969), are based upon qualitative assessment of rock masses. These are at best applicable to hard and fractured rock masses at shallow depths only.

In recent years, efforts have been made to include various other geomechanical factors and to describe the rock masses in quantitative terms. Consequently, a few quantitative classifications have been developed. Notable amongst these classification systems are those of Bieniawski (1973) and Barton et al (1974). Some modifications in the Barton's classification have been made by incorporating the effects of overburden and tunnel closure, the two factors important in squeezing conditions. A quantitative classification is a useful tool in assessing the quality of rockmass in quantitative terms and hence it is not only possible to compare the probable engineering behaviour, but also to predict the support pressure. However, the reliability of the predictions will depend on the reliability of the geological information. The approaches of Terzaghi, Deere et al, Barton et al and CMRS (Jethwa, 1986) have been used to predict the support pressures and the predictions have been compared with the measured values in Table II.

TABLE II : PREDICTED AND MEASURED SUPPORT PRESSURE (after Jethwa et al, 1981)

Rock Description	Predicted Support Pressure, kg/cm²				Observed Support Pressure Kg/cm²
	Tenzaghis	Barton et al		CMRS approach	
	p_v	p_v	p_h	p_v	p_v
1. Moderately fractured quartzites	0.3 to 0.7	1.3 to 1.5	0.8 to 1.0		
a=2.4 m, h=225 m, γ=2.5gm/cc QD=75%, Q=3.6, U_a/a=0.06%	(0.5)	(1.4)	(0.9)	1.23	-
2. Foliated metabasics	0.3 to 0.7	1.0 to 1.4	0.8 to 1.0		
a=2.4 m, h=550 m, γ=2.5 gm/cc RQD=82%, Q=3.4-6.8, U_a/a=0.05%	(0.5)	(1.2)	(0.9)	1.56	1.22
Thinly laminated foliated and wet metabasics(squeezing)	5.02 to 10.8	1.34 to 1.69	0.99 to 1.24		
a=2.4 m, h=800 m, γ=2.5 gm/cc RQD=60%, Q=1.64-3.28, U_a/a=17%	(7.91)	(1.51)	(1.11)	3.46	4.36*
3. Sheared metabasics	0.8 to 2.6	1.6 to 3.0	1.0 to 2.4		
a=2.4 m, h=340 m, γ=2.5 gm/cc RQD=60%, Q=0.3-3.3, U_a/a=0.4%	(1.7)	(2.3)	(1.7)	2.37	2.36

a = radius of tunnel opening; h = thickness of cover, γ = unit weight of rock mass, RQD = Rock Quality Designation, Q = Barton's rock mass Quality, U_a = observed tunnel wall displacement, p_v = roof support pressure, p_h = wall support pressure, * = estimated from support capacity; average values of the predicted pressures are shown in brackets.

OBSERVED TUNNEL BEHAVIOUR

A tunnel instrumentation programme was adopted to evaluate the predicted support pressures and to modify the support as per actual requirements during construciton of the tunnel. The instrumentation programme consisted of measuring hoop load on the steel arches by hydraulic -load cells and "tunnel closure", defined as reduction in the size of the tunnel opening, by closure meter to an accuracy of ± 0.1 mm.

The instruments installed were designed and developed at the Central Mining Research Station, Dhanbad (India). The closure bolts and load cells were installed at a few locations as shown in Fig.2.

The data analysis shows that the maximum closure was of the order of 430 mm (8.9 percent) in 600 days at the synclinorium contact of metabasics and quartzites at ch. 4180 m (Fig.8). At other locations, the closure varies from 10 mm to 20 mm (0.2 to 0.4 percent) except at ch. 5510 cm, where the total closure observed in 100 days was about 105 mm (2.18 percent) at a depth of 750 m (Fig 9). It is clear from Fig.9 that the wall closure increased in stages. This is due to the buckling of the steel ribs. Support pressure estimated from the load-cell data varies from 1.22 kg/cm^2 at ch.6760 to 3.05 kg/cm^2 at ch.4140 m (please see Fig.2).

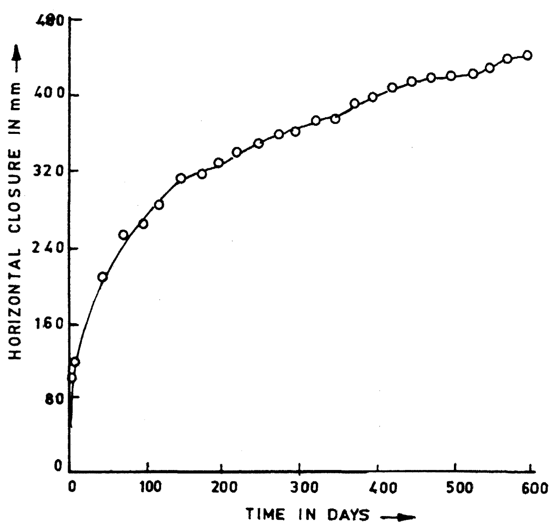


Fig.8 Closure-Time Relation at ch.4180 m

The support pressure of 3.05 kg/cm^2 could not be predicted because of the complex geological conditions and the presence of an inferior rock mass were not known at the design stage.

The comparison of predicted and observed support pressure (Table II) shows that the predicted values by Barton et al and CMRS approach are more or less equal to the observed values for non-squeezing ground conditons. Unfortunately, no instruments were installed at ch. 5500 m, where the squeezing problems were encountered. However, the back calculation from the

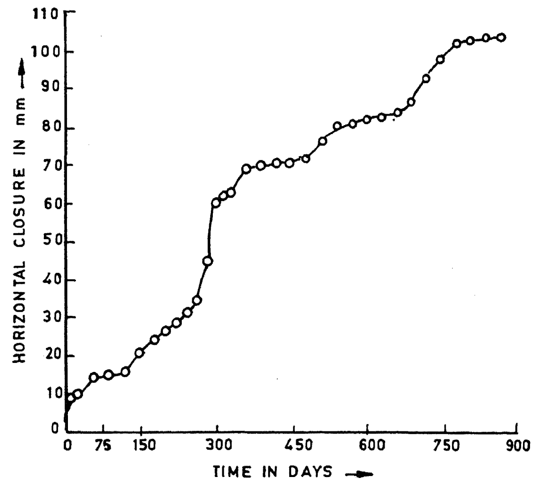


Fig.9 Closure-Time Relation at ch.5510 m

supports shows that the predicted values of support pressure from CMRS approach is nearer to the design pressure for uniform loading (Table II) while Terzaghi's approach is conservative. Buckling of ribs may have occurred due to a possible non-uniform pressure distribution around the ribs.

CONCLUSIONS

From the case-history of the power tunnel of the Maneri Hydel Project, Stage I and its instrumentation, the following conclusions are drawn :

i) Inadequate surface and sub surface investigations have been responsible for wrong planning of the tunnel alignment.

ii) The classification method of Barton et al leads to reliable predictions for non-squeezing conditions only. On the other hand, the CMRS approach leads to more reliable predictions for both squeezing and non-squeezing ground conditions. However, the data is limited to draw generalised conclusions.

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