Geotechnical Assessment of Diversion Tunnels, Lower Subansiri Hydro- Electric Project Assam & Arunachal Pradesh

A.K.Choudhury* and Biswajit Das**

Abstract

Five tunnels each of 9.5m diameter and 600m average length have been constructed through the left abutment of proposed 132m high concrete dam across Subansiri river for the purpose of diverting water during construction.

In view of the close proximity of tunnels to each other, road headers have been used for excavation, instead of conventional drilling and blasting. Shotcreting and rock bolting have been executed as tunnel support. Tunneling has been done from inlet side towards exit end.

Medium to coarse grained Middle Siwalik Sandstone, dipping steeply towards downstream, constitutes the tunneling media, traversed by four sets of joints. The intersection of these joints has caused roof failure in the form of wedge at the kink portions of each tunnel.

The overall tunneling condition was "Fair to Good" having RMR values 50 to 70 and Q values 5.5 to 37.5. The present paper presents geo-technical parameters of these tunnels and associated construction problems, including inlet and outlet portals.

Introduction

A 132 m high concrete dam is under construction across Subansiri river near the gorge mouth on the Assam - Arunachal Pradesh border to generate 2000 MW hydro power. This is the largest hydro power project in India taken up by NHPC. BGS-SGS-SOMA has been assigned construction of main gravity dam and diversion tunnels, while HRT, Surge Shaft, Pressure Shaft and Power House Complex have been awarded to L&T. Five number of 9.5 m finished diameter Horse Shoe tunnels have been constructed through the left abutment of the proposed dam to divert water during dam construction. A plan showing the layout of the project is presented in Fig. 1.

Salient Features

These tunnels are spaced at 22.70 m apart and have been excavated from inlet side only. The length of the various tunnels is as follows:

Tunnel No.	Total Length (m)	Total Aggregate Length (m)
DT 1	494.36	2953.38 (2.953 km)
DT2	533.99	
DT3	594.33	
DT4	639.90	
DT5	690.80	

Methodology of Tunnel Excavation

In view of the close proximity of the tunnels to each other (22.70 m rock ledge in between) conventional drilling and blasting has not been adopted for tunnel excavation. It was apprehended that the rock ledge between the tunnels may develop cracks and fail during construction and operation.

Road header has therefore been used for this purpose. The main constraints and hindrance in this method of tunneling was very low progress with creation of huge quantity of dust during cutting operation.

* Director (Retd.) Geological Survey of India
** Executive Director Soma Enterprise Limited, New Delhi

Table 1. Lamoblide locations with respect to tunnel

Date	Chainage (E.L / m)	Bank	Type of Slide	Probable cause of failure
(1) 19.03.05	RD 255 – 275 (E.L 182-186)	Right	Rock-cum Debris	Cutting of slope in weathered rock
(2) 23.02.06 to 16.04 06	R.D 150 to 190	Left	Rock Slide	Valley dipping joint having average orientation 300^0 / 50^0 – 60^0 responsible for sliding and rock fall. The joint plane is clay filled
(3) 28.05.06	R.D 255 to 260 (E.L 139 – 144) R.D 226 to 238 (E.L 120 – 122.50	Left	Rock Slide	
			Rock Fall	
	R.D 226 (E.L 144)			
(4) 01.06.06	R.D 245 – 270 (E.L 178m-crest)	Left	Rock-cum debris slide	Weathered rock having four sets of joints. Bedding plane (140^0 / 75^0) create toppling. 280^0 – 320^0 / 50^0 – 60^0 valley ward dipping joint responsible for sliding, sub parallel to nala. Clay coating on joint surfaces.
(5) 06.01.06	R.D 260-270 (E.L 124– 128)	Right	Rock slide	Slide along valley dipping joint 170^0 / 40^0.
(6) 14.01.06	DT 4 out let (E.L 126-130)	Right	Rock slide	Slide along valley dipping joint 170^0 / 40^0.

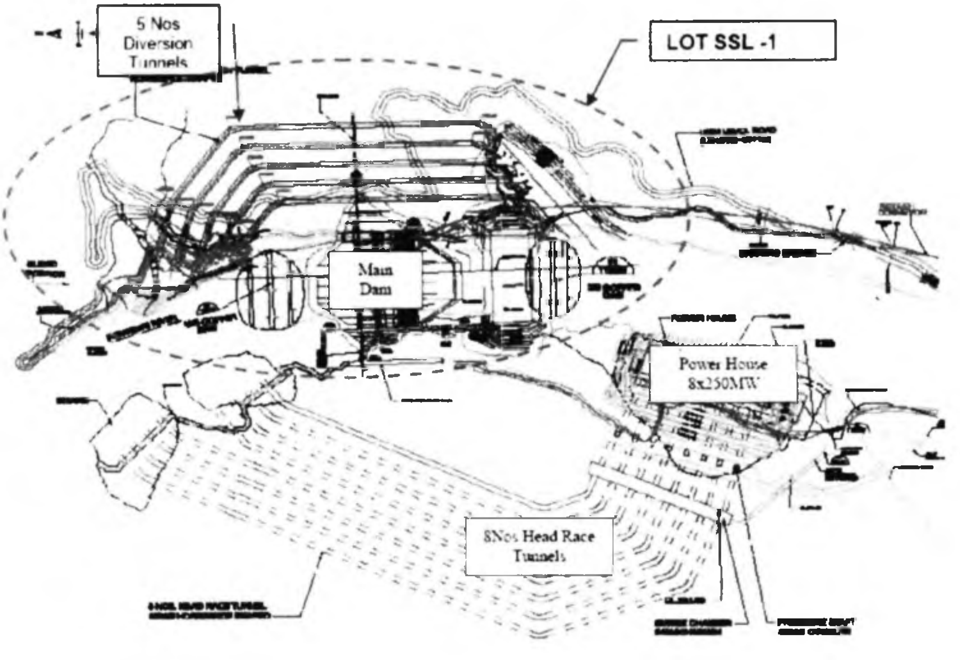
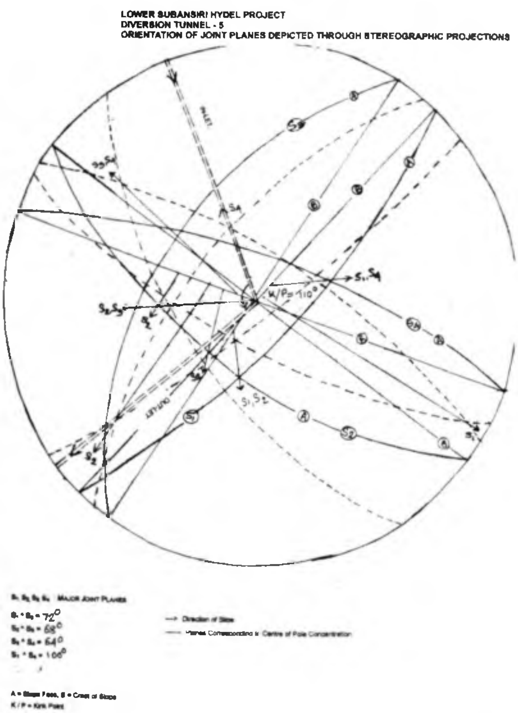


Fig. 1: Project General Layout Plant

Fig. 2: Road Hender PAURAT-242 after commissioning

Fig. 3: Road Header PAURAT-242 in operation inside tunnel



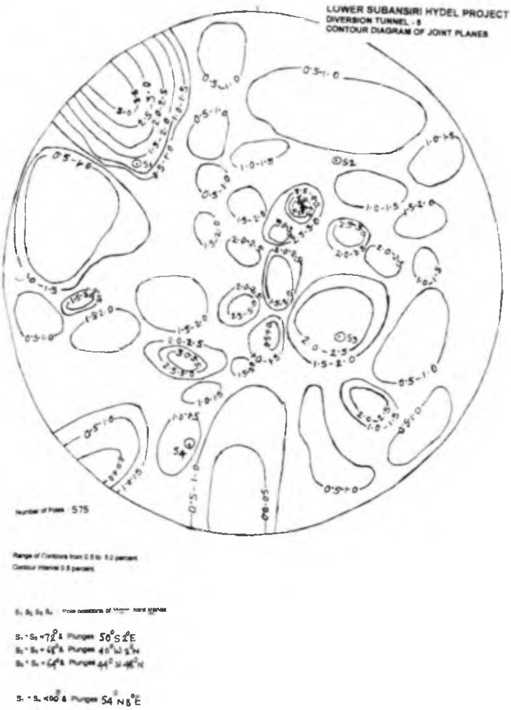


Fig. 5

As against 51 cum per hour predicted average output for rock strength of 12 to 20 MP_a the cutter head produced only 5 to 8 cum per hour. Rock cutting machines could only grind and scrape the rock instead of breaking, resulting in creation of dust (Das, 2007). Deployment of heavy duty Road Headers (PURAT 242) could improve the production to an average 15 to 20 cu.m per hour, but

Photograph showing Shear Zone on left bank of Deonala

Fig. 7

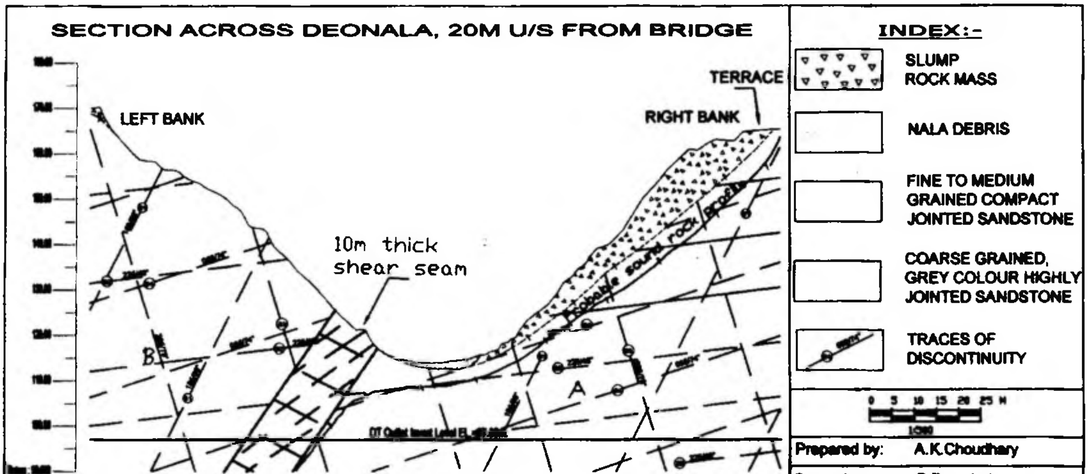


Figure 8

could not achieve the rated output of 120 to 140 cum/hr. (Picture of PRAT 242 after commissioing and operation inside the tunne, Fig. 2 & 3) The support system in the tunnels consisted of 5 cm thick initial shotcreting followed by rock bolting (2 to 3 m length) and final shotcreting (10 cm thick) with wire mesh.

Geology along Diversion Tunnels

Geological logging (Face logging) of individual tunnels has been carried out. A comprehensive plan has been prepared from the Face logs, 3 D logs and presented in. There is no gross lithological variation except in the grain size, from coarse to medium grained. Occasional splintery shale bands have been encountered. List of slides along is given in table 1.

Petrographic examination of the rock sample has given the following result:

Quartz – 60 to 70 %

Mica – 3 to 15 %

Feldspar – 2 to 3 %

Carb. Matrix – 15 to 25 %

Strained Quartz – 15 to 20 %

The most important geological structure observed in these tunnels is various joint sets as described below.

Joints	Strike	Dip
S 1 (Bedding)	N 40^0 E – S 40^0 W	75^0 to 80^0 SE
S 2	N 55^0 W – S 55^0 E	45^0 SW
S 3	N 55^0 E – S 55^0 W	42^0 NW
S 4	N 45^0 W – S 45^0 E	60^0 NE

Two prominent shear planes have traversed the tunnels towards inlet side. The strike of these shear planes is N 48 ^{0} E – S 48^0 W dipping at 65 ^{0} to 80^0 towards inlet side, oriented parallel to bedding plane. Another shear plane dipping 55^0 to 65^0 towards inlet has cut across the tunnels in the kink portion. Inter bedded thin shears are also common. Although the rock appears to be massive because of widely spaced joints, it is soft, poorly cemented and friable.

Joint Patterns and its influence on tunneling

The various joint patterns observed in tunnel 1 to 5 have been plotted stereographically. It can be broadly concluded through the projections that the tunnel alignment from inlet to kink point is mostly influenced by S 2, S 3, and S 4 joint sets (Fig. 4 & 5).

Wedges are formed in the kink area of the tunnels due to intersection of four joint sets. This kink area is the most critical portion of the tunnel in respect of joint orientation. The portion of tunnel between kink point and outlet is relatively free from joint influence. S1 and S2 joints are widely spaced in this stretch.

Strength Parameters

The representative value of Unconfined Compressive Strength in dry state ranges between 12 and 20 MP_a (Av. 13 MP_a). The same gets reduced to 1.21 to 7.21 MP_a (Av. 4.46 MP_a) in wet condition. Reduction of strength is around 65 % (Source DPR).

Rock Mass Description

Rock mass classification of the excavated portion of individual tunnels has been carried out. The results are summarized below:

RMR ratings indicate that the tunneling media

Tunnel No.	Range of Q Values	Range of RMR Values	Hindrance Class
DT 1	3.41-37	37.5 - 77	III A, B & IV
DT2	1.60-37.5	18.75 - 77	II, III A, B & IV
DT3	2.66-37.5	31.00 - 77	II, III A, B &IV
DT4	5.10-37.5	23.00 - 77	III A, B & IV
DT5	2.08-37.5	35.00 - 77	III A, B & IV

belongs to "FAIR" rock mass class. The rock mass quality as per Q values is "FAIR TO GOOD".

Over break in Tunnels

Apart from crumbling of rock with dust formation due to variation in feed pressure of the Cutter Head, rock fall has occurred wherever intersection of joints have formed wedges. The dimension of this rock block is about one cum on an average. The overbreak is more at the kink area. "Popping" phenomenon has also been reported between kink and outlet.

Cumulative effect of high percentage of deleterious material, poor U.C.S under dry and wet condition and "Fair" rock quality as per "RMR" values have all contributed to the over break in the tunnels. Stress release around tunnel periphery resulted in Popping phenomenon. All these geological features are responsible for over break (Choudhury, 2007).

Diversion Tunnel Inlet

The diversion tunnels inlet is located in Hathinala depression having a vertical rock height of 28 m above the tunnel invert E.L 102 m. A 3 m wide bench has been provided above the tunnel portal & the entire hill face shotcreted.

After opening of tunnel portals of DT 3, 4 & 5, and excavation progress of 10m or so, cracks appeared on the vertical face in a criss-cross pattern. Excavation work stopped subsequently and nine meter long rock bolts inserted at 3m interval in a staggered pattern on the cut face.

Two sets of prominent joints, i.e. S2 and a near horizontal to 10^0 dipping joint set towards the valley might have caused the movement resulting in development of cracks.

Instrumentation

Multiple Bore Hole Extensometers (MPBX) were installed along the slope cut at E.L 132.65m (17 m above tunnel crown) at six locations to monitor the movement. The extension rods were of 5m, 10m, 15m and 20m length in each locations. At four locations 5m long extension rods showed movement and rest less or no movement. A possible geological explanation may be that five meter section behind the cut slope is releasing stress along S2 and horizontal joints. Rock behind 5m is stable. No slope movement was recorded after installation of 9m bolts.

Diversion Tunnel Outlet

The outlet portion of Diversion tunnel is located in Deonala, a major valley depression. The nala flows from NE to SW to meet the Subansiri river. The diversion tunnel outlet portal area is located on the right bank of this nala. The High Level road crosses this nala in its upper reach. A 30m wide collection channel has been constructed on the nala bed to lead the water to Subansiri river.

Pre-Construction Stage

Originally the nala bed was filled up with boulders, silt and mud. A steel girder bridge across this nala used to serve as only communication by road to inlet portal before completion of high level road.

The whole area was in a disturbed condition. Rock exposures on the left bank extended uphill up to high level road. A huge slided rock mass was resting on the right bank hill slope, apparently dislodged from top. Scar face behind the slide was visible with top at E.L 150.0 m. Undisturbed terrace rested on the top of the mass.

A major shear zone traversed the entire area along the nala course. Highly sheared and crushed rock mass with black gougy material was exposed in a 10 m wide zone on the left bank of Deonala, 20-30m up stream of girder bridge. The trend of this shear zone was parallel to nala course having 50 ^{0} dip component towards left bank, i.e south-easterly.

The right bank hill slope was equally disturbed. Unconsolidated terrace material rested on the highly jointed rocky slope.

Expert's site visit (Chaturvedi, 2005) pointed out that the major problems in this portion will be:

i) Stability of overhead right bank slope over the exit portals,

ii) Stability of cut slope for the collection channel and design of the channel structure based on the sheared material in the nala bed.

iii) Head ward erosion from riverside along the nala.

The first author examined the Deonala area and classified the rock into two litho units A & B (Chaudhury, 2005). Unit 'A' comprises fine to medium grained hard, brown colored, jointed, blocky sandstone and unit – 'B' coarse grained, grey coloured, soft, micaceous sandstone, often containing carbonaceous material.

The boundary between the two units more or less follows Deonala in the lower reach. The entire right bank is occupied by unit 'A' and left bank by unit 'B'. The boundary between the two units is marked by shear zone. (Geological section and photograph Fig. 6).

Construction Stage

(a) ***Slope Character:*** The area around Deonala was a critical area having different litho units on either banks, a shear zone along nala bed and weathered rock block (Slided mass) resting on the right bank slope.

Right bank: Right bank slope was dressed to remove all dislodged slumped masses before excavation in steps from High Level Road to tunnel invert E.L 97.0 m.

A tentative geological section across Deonala 20 m up stream of steel girder bridge (Choudhury, 2005) depicts set up prior to slope dressing.

Slope dressing consisted of cutting of slope at 1:1 in overburden and 1:4 in all kinds of rock with intermediate berms as per design drawing issued by NHPC in October, 2005. Shotcreting and 9m deep systematic rock bolting were carried out to stabilize the slope right from H.L.R level.

Left bank

The rock on the left bank comprises soft to medium grained grey color slightly to moderately weathered sandstone. Rock mass is dissected by four joint sets of which S_3 ($290^0 - 325^0 / 40^0 - 65^0$) is valley dipping. Some of the joints have slickensided surface and clay filled. Soft nature of rock, disposition of valley ward dipping S_3 joint set and presence of bedding shears imparted instability in the cut slope.

(b) ***Slope Failure:*** Reports on slope failure with photographs, submitted from time to time by Geotechnical section of SOMA, gives the following sequence of failures:

Deonala bed

The collection channel of Diversion Tunnel spreads across Deonala bed excavated down to R.L 97.00 m. The bed consists of medium to fine grained sandstone. Most prominent joint sets are (i) N 50^0 E – S 50^0 W, dip 65^0 – 85^0 towards SE, (ii) N 36 ^{0} W – S 30^0 E strike, dip 50^0 towards SW, (iii) N 70^0 E – S 70^0 W dip 65^0 – 85^0 towards SW.

Joints are very closely spaced close to the outlet portals of DT 4 & DT 5. A prominent near vertical shear plane trends parallel to nala course towards left bank. The thickness of shear plane associated with crushed rock varied from 5 mm to 15 mm.

The bed has been provided with 3 m thick concrete raft. A concrete wall protects hill slope from direct impeachment of water discharge. 11m deep Tendon anchors have been provided.

Geotechnical Discussions and Conclusions

Excavation of five tunnels parallel to each other at 22 m apart through soft medium grained sandstone presented a great challenge at Lower Subansiri Project. The sandstone has poor Unconfined Compressive Strength and medium RMR and Q values. Four sets of joints traversing the rock induced rock fall along the tunnel length. NATM method of tunnel support was successful in stabilization of tunnel cavity and prevented major tunnel failure.

Opening of tunnel portals at inlet and outlet ends involved deep open excavation through weathered material. Slope failures were common. Benching, Shotcreting and rock bolts helped in stabilizing the slope.

The most critical part of the tunnel construction was deployment of appropriate machinery for excavation, since conventional drilling and blasting was not allowed. In spite of deployment of large capacity Road Header (PURAT 242), the output was much below the rated capacity. The Cutter Head grinded the rock instead of breaking, resulting in high energy input with less production and high wear and tear. Generation of dust was a major problem. Water spray was found ineffective as it transformed muck into slush. Ultimately providing dust depression by Foam at Cutter Head proved effective in controlling the dust formation.

In conclusion, it can be stated that the timely completion of diversion tunnels was possible by proper interpretation of geotechnical features and deployment of appropriate machineries in close interaction of various disciplines. It is expected that the experience gained in this project will be utilized in future projects in Himalayan terrain having identical geotechnical set up.

References

Chaturvedi, S.N (2005): A geotechnical note on Subansiri Lower Hydro-electric Project. (Unpublished)

Choudhury, A.K (2005): A geo technical report on Subansiri Lower Hydro-electric project (Unpublished)

Choudhury, A.K (2007) : Geo technical causes of over break in Diversion tunnels (Unpublished)

Das, Biswajit (2007) : Challenge in tunneling in Subansiri Lower H.E.Project (2000 MW) in India. Hydro 2007