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Tunnelling at Ranjit Sagar Dam Project - A case study

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Abstract

Large diameter tunnels have been excavated through young sedimentary rocks of Lower Siwaliks at Ranjit Sagar Dam Project, a multipurpose scheme across the river Ravi in the outer Himalayas. This paper discusses in brief the tunnel support system and the major problems faced during tunnelling and reservoir impoundment. Causes of distress and remedial measures have also been outlined in brief.

Introduction

Ranjit Sagar Dam Project, earlier known as Thein Dam, is a multipurpose river valley project located on the river Ravi at the border of Punjab and Jammu and Kashmir states, about 30 km from Pathankot. The main components of the project include a 148 m high, 617 m long earth core-cum-gravel shell dam, a 133 m wide, 547.5 m long, left flank chute spillway with a roller bucket, and a 600 MW (150 x 4) capacity surface power house. The reservoir area is 87 sq km with 3280 MCM gross storage and 2344 MCM live storage capacity. The underground works comprise diversion cum power tunnels, penstocks, drainage tunnels and foundation gallery at the base of the dam. The project envisages additional irrigation in 3.48 lakh hectares area, power generation to the tune of 1509 KWH per year and flood moderation in the command area. Power generation has already begun at the project.

Geotectonic setting

The project is located in Himalayan foothill zone in a sedimentary pack, intensely mobilized during terminal phase of Himalayan orogeny. All the major project components lie on the southern limb of WNW-ESE trending regional anticline, known as "Mastgarh Anticline", the axis of which is located about 0.50 km upstream of the dam. Other major structures around the dam include MBF, the Basoli Thrust and the Satilita Thrust, besides some transverse faults. The region constitutes a domain of seismicity occurring at the margin of Kishtwar and Kangra seismotectonic units (Pande, 1999).

Alternate sequence of sandstone and claystone/siltstone of Lower Siwalik Formation (Upper Miocene) occur at the project site. The general strike of lithounits is N50°-65° W - S50°-65° E with 60°-70° dips due southwest, i.e. in downstream direction. At the tunnel inlet portal area, the rock mass is poor to very poor in nature due to presence of local faults, shears and gouge/plastic seams. In rest of the project area, rock mass is fair to good quality though thin shear zones and plastic/gouge seams are present, mostly along the bedding planes. Sandstone is light grey, medium grained and is not very firmly lithified, though calcareous cementation is also recorded at places making it hard and

Geological Survey of India, Kol Dam Project, District Bilaspur, H.P.

compact. The claystone is purplish red in colour, loosens when comes in contact with water and exhibits desiccation cracks when exposed to the air. Two sets of joints, viz. bedding joint and NE-SW trending joints with subvertical dips, are the most conspicuous throughout the project area.

Tunnels

Salient features

Four circular shaped diversion tunnels of 12 m finished diameter, having cumulative length of 3448 m are located on the left abutment. These are spaced at 50 m c/c for peak discharge of 14286 cumecs. The four tunnels have maximum discharge capacity of 9800 cumecs with velocity of 20 m/sec. Two of the diversion tunnels P1 and P2 are being used as power tunnels. The other two tunnels T1 and T2 would be used as irrigation outlets.

Tunnel excavation

The four tunnels have been excavated through a total of 41 alternating sandstone and claystone/ siltstone bands which are about 40° askew to the tunnel alignment (Srivastava *et al.* 1981-83). The average percentage of sandstone and claystone/ siltstone encountered in the tunnels is about 40:60. The tunnelling has been through fair to good rock mass in most of the reaches except for inlet portal reach where rock is poor to very poor with Q values in sandstone and claystone/siltstone of 0.58 and 0.055, respectively (Pande, 1999). The corresponding roof support pressure worked out to 1.7 and 3.5 kg/cm^2, respectively. According to C.S.I.R. classification of Bieniawski (1974), the RMR of sandstone varies from 52 to 75 for poor and good rock, while that of claystone/siltstone is 41. The support pressures as calculated by Pande (1999) are given in Table-2.

The tunnel support system and rock load

Table-1: Geomechanical properties of the rocks

S. No.	Property	Sandstone	Claystone / Siltstone
1.	Specific gravity	2.6	2.7
2.	Compressive strength oven dry / wet	200 to 360 kg/cm^2 34 to 63 kg/cm^2	202 to 411 kg/cm^2 45 to 47 kg/cm^2
3.	Poisson Ratio	0.25-0.33	0.33-0.35
4.	Modulus of Elasticity	2 x 10^5 kg/cm^2 0.28 x 10^5 kg/cm^2	1.03 x 10^5 kg/cm^2 0.31 x 10^5 kg/cm^2
5.	Shear strength ϕ	33°	25°
6.	Ultimate bearing capacity	41 to 56 kg/cm^2	38 kg/cm^2
7.	Pull out strength	64 to 73 tonnes	35-57 tonnes
8.	RQD	70-100	10-80

Table-2: Calculated support pressures

Sl. No.	Relationship	Vertical Rock Pressure		
		Sandstone	Claystone	Under worst condition
1.	Protodaykonov	1.39	2.50	5.28
2.	Engineers	4.89	5.28	9.08
3.	Bieniawaski	1.01	1.57	3.14
4.	Barton	0.75	1.50	3.50
5.	Terzaghi	0.87	2.45	9.35
6.	Deere	1.50	1.40	9.35

Table-3: Terzaghi's classification for tunnel supports

Normal dry reaches	0.35 (B+Ht)
Normal saturated reaches	0.5(B+Ht)
Poor rock reaches	0.7 (B+Ht)
Very poor rock reaches	1.1 (B+Ht)

(Where B = excavated width and
Ht = excavated depth)

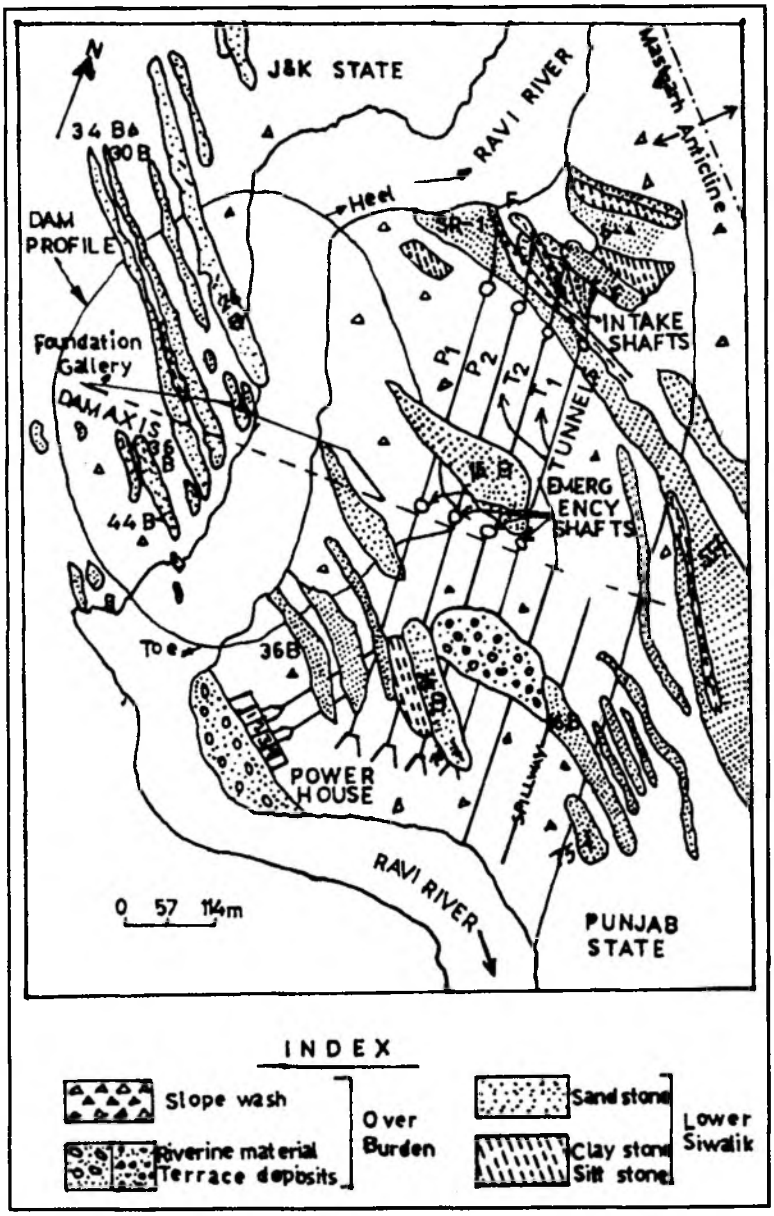

Fig. 1 Geological plan of Ranjit Sagar Dam Project.

design at Ranjit Sagar Dam's diversion tunnel is based on Terzaghi's classification (1946), which is very conservative and factor of safety is fairly high. For the design of tunnel supports, Terzaghi's classification (as given in Table-3) was adopted on consideration that i) the excavated diameter was very large, ii) the rock mass was young and incompetent, particularly the claystone bands and iii) conventional method of drilling and blasting which caused further loosening. Tunnel excavation commenced in 1981 using heading and benching method through conventional drilling and blasting. The drilling of 37 mm blast holes in a set pattern was done by jack hammers and rock drills with compressed air at 80-100 p.s.i. Explosives comprised special gelatine 60% in cartridges and half-second delay detonators were used. An average consumption of explosives was 0.7 kg per m^3 of rock. The tunnels were supported with steel ribs (RHS 250 mm x 125 mm) spaced from 0.45 m to 0.90 m depending on rock conditions. In emergency gate reach, the ribs were closely spaced as the excavated diameter was large (19-20 m). A short reach in P1 tunnel was supported with shotcrete and rock bolts in sandstone band 14B and claystone/siltstone band 13B on an experimental basis. This was discontinued as rock fall occurred in the claystone band due to failure of shotcrete (Andotra & Pande, 1984-86). The ribs were backfilled with lean concrete of 1000 p.s.i. strength by pneumatic placers. The tunnels up to RD 26 m from outlet portal are horseshoe shaped and beyond RD 26 m these are circular in shape. The excavated diameter of tunnel in normal reaches was 13.5-14.5 m while in special reaches (intake shaft and emergency gate reaches) it was even 19-20 m. The benching was carried out in two steps from inlet end.

The tunnel lining is designed for internal water pressure condition as well as for tunnel empty condition to take part of saturated rock mass load. For normal reaches rock load for tunnel lining is taken as 50% of 0.5 (B + Ht) and for intake and emergency gate shaft reaches it is 1.1 (B + Ht), where B is the excavated width and Ht is excavated depth. Based on this, minimum thickness of reinforced lining (first stage) during diversion stage in normal reaches and special reaches is 70 cm and 110 cm, respectively. After diversion stage, 2nd stage reinforced concrete lining in special reaches, viz. emergency gate shaft reach and its downstream is of 1.5 m-3.0 m thickness. The 2nd stage lining is placed between the steel liner and the first stage concrete by pneumatic placers in most of the reaches, except in emergency gate reaches where chute system was used in power tunnels P1 and P2.

Old failures in the inlet reach

The diversion tunnels suffered major set back in 1988 when due to sudden flash floods in Ravi, 24000 cumec discharge passed through tunnels, saturating and silting them up. It started with the collapse of the crown of tunnel P2, which in Jan 1989 led to the collapse of 49.5 m reach of its inlet portal. On 30th Jan. 1989, T1 also failed in parts up to 46.4 m length from inlet portal and a cavity extended up to El. 456 m benches.

As discussed earlier, the diversion tunnels inlet portal reach comprises highly incompetent rock mass. The reasons of the collapses can be attributed to (Andotra, 1988-89) :

- Inadequate rock cover
- In the heading and benching method of excavation, removal of benches before providing support to the excavated tunnels.

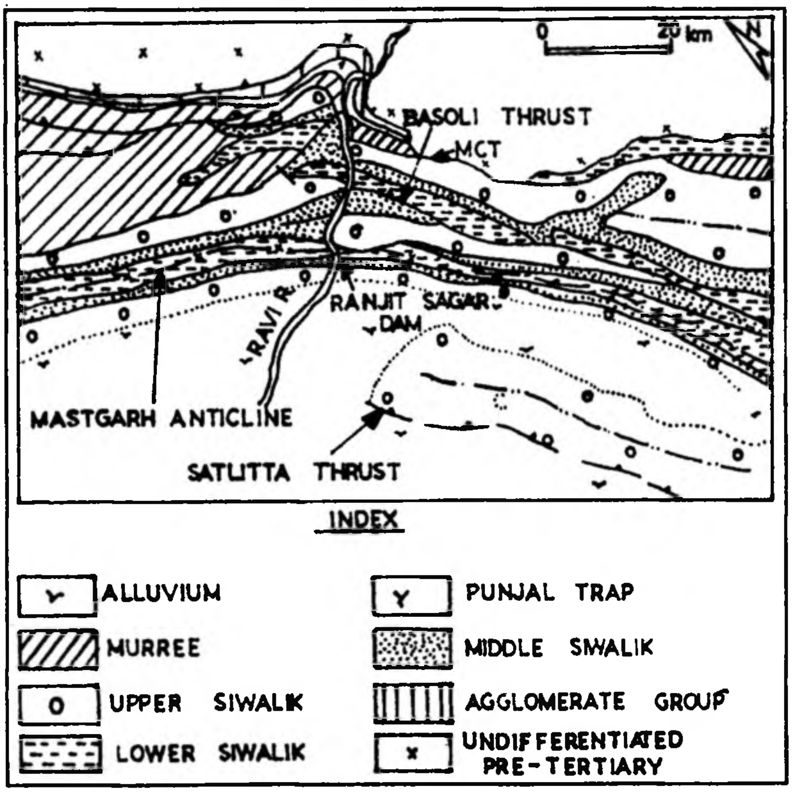

Fig. 2 Regional geological map of the area around Ranjit Sagar Dam Project (after ONGC).

- Loss of shear strength following deterioration of rock mass dissected by faults and shears due to ingress of rain/ flood water.
- Inducement of ground acceleration due to heavy blasting in the vicinity. As remedial measures, heavily reinforced false portals were made with provisions of portal benches as per the recommendations of the R.S.D.B.C.

Snags in tunnels during reservoir filling

The reservoir filling started on 15/02/99 with closure of tunnel T-2 inlet. A heavy seepage through concrete in the cut and cover section, located in random fill of tunnel T-2, occurred just downstream of the false inlet portal. This made plugging of T2 difficult. The snag was removed by way of grouting and patching through plates. The reservoir filling was restarted in March 1999. Emergency gate shaft of P1 and P2 were kept in closed position so that water entering through intake shaft at El. 473 m did not pressurise the butterfly valves of the power tunnels. In the first week of August 1999, bulging of upstream side P1 E.G. shaft's bonnet chamber plate was observed. The bulge was 145 mm maximum at 11.3 m from the base of the bonnet. This rendered P1 emergency gate unoperational. As methodology of repairs was being evolved, bulging of steel liners in rectangular (9 m x 5 m) section of P2 E.G. reach, was also noticed. The bulging was in about 10 m long section, with maximum bulge of 245 mm in the bottom and 118 mm and 130 mm in the left and right sideliner plate, respectively. Seepage of water was recorded at places in the liner plate along cracked joint. When the bottom bulged liner plate was punctured, water gushed through at a pressure of 1.5 kg / cm^2.

In November 1999, the bulging of bottom liner plate was noticed just downstream of the emergency gate of the P1 tunnel in a length of about 16 m rectangular and transition reach, with maximum bulge being 490 mm. The sideliner plates were intact.

Reasons of distress

With snags in emergency gate shaft reach of both the power tunnels, project authorities and some of the Board of Consultants' members apprehended some geotechnical problem. P1 E.G. reach is located on sandstone band 16B and claystone/siltstone band 17B (Fig. 3). The proportion of sandstone and claystone/siltstone in distress reach is 3:1. In P2 E.G. reach, claystone/ siltstone band 15B and sandstone band 16B are present. The proportion of sandstone and claystone/ siltstone in distress reach is 9:11. The disposition of bands is about 40% askew to the tunnel alignment. There is no adverse geological feature present in the P1 and P2 emergency gate reach or around it. As already discussed, the tunnel support system is based on Terzaghi's classification, which is very conservative. Apart from steel rib supports back filled with lean concrete, heavily reinforced (40 mm dia) concrete lining has been provided with thickness of concrete varying from 3 m± to more than 5 m in the E.G. reach. With such heavy support system, the possibility of rock failure can easily be ruled out. Moreover, the tunnel behaved normally during diversion stage since 1992 onwards. The problem seems to have resulted due to direct water pressure acting behind steel liners. As evident during the repairs, the water seeped through the gaps and honeycomb in second stage concrete from the pressured upstream (of emergency gate) to the downstream of emergency gate.

Dr. Evert Hoek, renowned rock mechanics consultant form Canada is also of the view that the problem occurred due to direct water pressure acting behind the steel liners.

The reason for distress in steel liners in

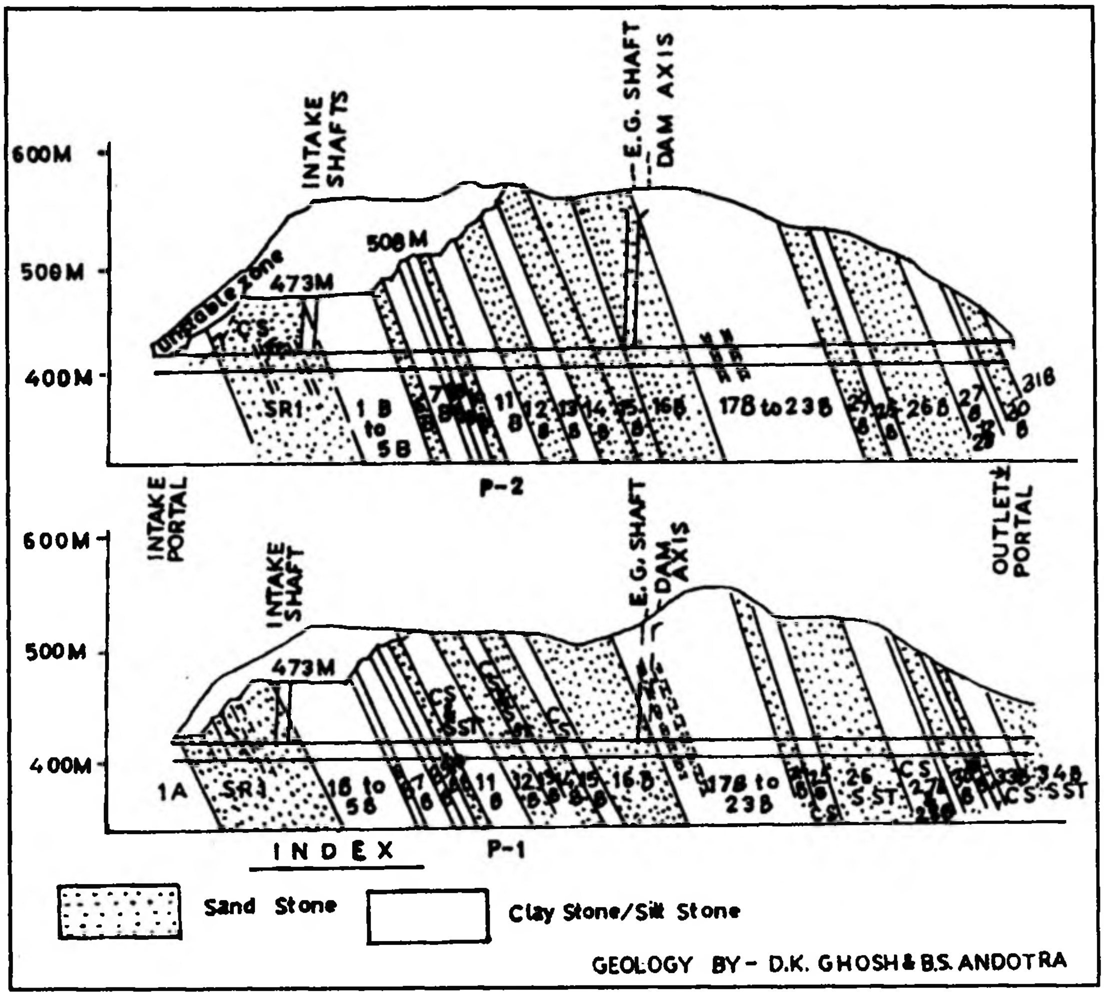

Fig. 3 Geological section along Power Tunnels P-1 and P-2.

P1 and P2 power tunnels, as investigated during repairs, is interpreted to be due to external water pressure acting immediately behind steel liner, which in turn may be due to following reasons (Kumar, l998-99):

- The emergency gates of P1 and P2 were in closed portion due to which half of the tunnel was pressurised while that downstream of gate was not.
- Hollow spaces behind steel liner and concrete in the emergency reach.
- The second stage concrete in the emergency reach was not properly placed as evident from the gaps and honeycomb features reported during repair stage. This provided paths of seepage of water from upstream of emergency gate to the downstream liner.
- Improper bond between first and second stage concrete.
- Improper grouting after placement of concrete.
- Drainage system behind steel liner did not function properly.
- The rectangular section is more vulnerable against external water pressure as compared to circular section.

One of the paths of leakage upstream of emergency gate through second stage concrete to downstream of gate was established during repairs, when grouting in second stage concrete was being carried out in both P1 and P2 tunnels.

In case of P1 bonnet chamber, distress in steel liner was due to accidental filling of water in the P1 emergency gate shaft through unplugged holes in the P1 gate body and seals which pressurised P1 tunnel up to the prevalent reservoir level, *viz*. El. 478 m, as the butterfly valves were in closed position. The water in the shaft entered between liner plate and concrete through some crack/ opening and exerted pressure on the liner, resulting in the bulging.

Repair/Remedial measures

Broadly, following measures were undertaken in P1 and P2 emergency gate shaft reach :

i) The bulged liner in the rectangular portion downstream of the emergency gate, along with loose concrete, was removed. The concrete was chipped and epoxy concreting in 5 m lifts was provided on the sides. On the bottom portion, M-30 concrete with one mat of reinforcement of 16/20 mm @ 2000 mm in both directions, was placed 15 mm thick epoxy plaster with epoxy paint was provided on the sides.
ii) Extensive grouting by conventional cement grout and epoxy resin has been carried out in the second stage concrete and in the contacts between second and first stage concrete. This has been done in the rectangular and transition reach in the upstream and in the distressed portions downstream of gates.
iii) Staggered drainage holes of 200 mm dia at 1.5 m c/c spacing, have been provided in the treated portion whereas, 10 mm dia holes at 1m c/c spacing in steel liner have been provided in the transition zone downstream of the gate.
iv) Epoxy cut-off has been provided at contact between steel liners and concreted in upstream and downstream of rectangular reach.

Bonnet Chamber of P1

The distressed 20 mm thick steel plate and third stage concrete have been replaced. The new 25 mm thick steel plate has been further strengthened by providing additional anchorage with epoxy grouted anchors, loop anchors and epoxy grouting of second stage concrete, and the contact between third stage concrete and liner plate. The repair have been completed successfully much ahead of schedule by project engineers and the commissioned tunnels are properly functioning.

Conclusions

Bulging of liner plates in emergency gate reach has also been reported from P1 and P2 power tunnels of Pong Dam. The designers may examine the possibility of dispensing steel liners in the rectangular emergency gate reach of power tunnels in lieu of conventional concrete section with provision of epoxy mortar for preventing corrosion and smooth flow, and proper drainage holes to release water pressure in tunnel during empty condition.

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