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1.0 Introduction

Tehri Hydro Development Corporation Ltd. has successfully commissioned the 4X250 MW Hydro Power Plant on river Bhagirathi/Bhilangana in Tehri District of Uttarakhand. The Tehri Project mainly consists of major civil structures viz. Earth & Rock fill Dam 260.50 M high, Chute & Shaft Spillways in the left and right banks of the river, Water Intake, 4 Nos. Head Race Tunnels, Maintenance Gate Shafts, Penstock & Butterfly Valve Chambers, Vertical & Horizontal Penstocks, Machine Hall & Transformer Hall Caverns, Upper & Lower Expansion Chambers and Tail Race Tunnel. An Interface Facility area has also been developed for power take off at 400 KV.

Fig. 1: General layout plan

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At present all major civil structures including Dam, Spillway & Power House have been successfully completed.

Four vertical maintenance gate shafts (11 m dia each and 120 m high) have been provided for the maintenance of the HRT's. The geology in this area is highly adverse and influenced by numerous major and minor intersecting shears. Owing to this the unique construction methodology and innovative techniques have been adopted for the slope stabilization housing intake structures and construction of maintenance gate shafts.

The present paper deals with an overview of special features adopted during construction of water intakes along with Maintenance Gate shafts of HRT's for Tehri Hydro Power Plant. The paper being presented shall focus on major difficulties and uncertainties encountered by the Project while constructing Maintenance Gate Shafts, platform at EL-840.0 for housing gantry structure and operation of maintenance gates and hill slope protection in the U/S of Maintenance Gate Shafts where rock column is only 40.0 m from the shafts.

2.0 Geology of hill slope at water intake area

It is emphasized that stabilization of 107 m long shafts could not have been possible unless the hill slopes in U/S were fully protected and there is no movement due to presence of D-3 shear. Also the stabilization of platform at EL-840.0 was necessarily required and any subsidence in the platform required to be stopped to avoid any damage to the shafts on account of avoidable subsidence.

The HPP and PSP Maintenance Gate Shafts (MGS) Nos. 3 and 4 have been repeatedly accompanied by complications caused by particular features of the geological structure of the rock mass. The structure of rock mass in question are brought close to each other in this area in space and have resulted in formation of negative consequences encompassing rock displaced in the process of construction within the contact zone. This zone is usually called D-3 zone, though, the causes of complications are far from being limited only by the presence of a large diagonal dislocation D-3 and its unfavorable orientation. The most significant features of the geological structure of this area and the character of these complications are indicated as under.

The slopes of the left bank in the area of water intakes towards gate shafts at EL 710.00-840.00 m are formed of phyllite and clay phyllite layers 5-30 cm wide with several beddings of quartzitic phyllite upto 10 m in thickness. The rocks are schistic and crumpled into highly compressed folds dissected by longitudinal and diagonal faults and fractures with favourable general orientation regarding slope stability. Near the daylight surface the rocks are additionally weakened by the processes of de-stressing and weathering.

Thus the rock mass conditions between MGS-3 and MGS-4 were very poor, influenced by numerous major/minor diagonal (D) and longitudinal (L) shears intersecting at different levels. The intersection of these L and D shears has resulted into formation of multiple structural wedges at different levels and all these wedges are confined within the major wedge defined by a major D-shear (D-3) and L-shear (L-11) of the block tectonic model of Tehri Dam site. (Fig.
3.0 Nature of the problem

In 1986 while excavating a pilot shaft in MGS-3 a pocket of weathered loose material was struck at EL-826.0 which led to formation of a cavity in the pilot shaft roof and resulted in spreading of the failure zone to the daylight surface and formation of a large hole. All attempts to clear up the pilot shaft resulted only on further increase of the zone of deformations of the weakened rocks near the daylight surface. The consequences of these complications were eliminated only in 1996 when THDC started re-construction of MGS-3. But as a result of this process a zone of technogenical material with lower physical and mechanical properties than the parameters of the original heavily jointed de-stressed and weathered rock was formed in the area of MGS-3 at EL 800.0-840.Om.

By the time of construction commencement the upper portion of the weakened zone under consideration within EL 790.0-840.0 m was presented by a fragment of the lower portion of the ancient rock landslide.

In March’ 1998 following several persistent and intensive rains in the Himalayas a part of the
ancient rock landslide got activated at the stretch between the HRT-3 intake and at EL-840.0 m. Upper end of landslide with tension joints cut away part of the site between the head structures of MGS-2 and MGS-3. A zone of deconsolidation started to develop at the site at EL-840.0 m near the upper edge of the slide, which has lead to progressing degradation of the physical and mechanical properties of the rock.

4.0 Measures adopted for slope stabilization and protection

The situation had been stabilized only after construction of deep cast-in-place concrete piles extending to the roof of relatively intact rock and integrated by massive pile caps. A system of concrete beams was provided between MGS-2 and MGS-3 at EL 835.0-840.0 m. These remedial measures stabilized the upper portion of the slide mass, which had positive effect on the overall stability of the slope. During the monsoon period of 1999-2001 no movements have been observed along the main slip surface and only surface erosion of the slide was detected.

In order to protect the hill slopes in the U/S of MGS Shafts, six pile-shafts at the berm at EL818.0 m and one pile shaft at EL-805.0 m were constructed in the years 2000-2002 which considerably increased the stability of the slide top portion in this area.

**Fig. 3: Longitudinal section of HRT-3**

In May 2002 during the berm clearing at EL-790.0 the upper layers of slopes were undercut resulting in its partial collapse at EL 790.0-818.0 m. This complication was provoked by
attempts to activate civil works on slope protection at EL 790.0-818.0 m. The rock mass which got into motion in the hole is represented by the slip zone of the ancient rock landslide. The rocks composing this zone have extremely poor strength properties during wet conditions.

The platform at EL-840.0 m had a tendency to creep down slowly during rainy season due to presence of poor geological strata in the entire slopes from EL-706.0 to EL-840.0 m. The presence of rock mass and its properties have been indicated as above. The platform has been successfully protected by construction of pile shafts as deep as 50.0 m which have been driven into the sound rock at various elevations at EL-790.0 m, 5 nos. pile shafts, EL-805.0, 6 nos. pile shafts, EL-818.0, 6 nos. pile shafts, EL-835.0, 7 nos. pile shafts, EL-840.0, 6 nos. pile shafts.

With the construction of above pile shafts followed by construction of pile caps and the concrete cladding over the pile cap and extensive grouting operations at EL-840.0, the platform at EL-840.0 has started behaving in the perfect manner with no further movement observed during the last 2 years. Intensive consolidation grouting operations were further carried out by deep bore holes ranging from 25 m to 100 m deep at a pressure of 10-15 kg per sq.cm. Fig. 3 & 4 shows the geological section and stabilization details along HRT 3 & 4.

5.0 Construction of maintenance gate shafts

The excavation of Maintenance Gate Shaft No. 3 & 4 was taken up by THDC in presence of heavily weathered and deformed rock mass in the year 1997. The excavation was started by making a pilot shaft of 3.0 m dia from EL-840.0. The shaft was driven to a length of 107.0 m with full face excavation. In March 2000, during excavation of the lower bench of the transition portion of HRT-4 the temporary ribs support was undercut which resulted in the
collapse of MGS-4 in the zone of the influence of fault D-3 zone.

From March 2000 to June 2000, failure of shaft wall surfaces was spreading upwards to above EL-730.0 m accompanied by deformation and failure of the rock support ribs. Only after backfilling with sand and gravel into MGS-4 upto EL-770.0 m it became possible to stop the process of deterioration of the tunnel roof and shaft walls.

Nevertheless the zone of deformations of the MGS-4 walls gradually merged with the zone of loose material confined to zone D-3 at the stretch between MGS-3 and MGS-4. By the end of June 2000 the zone of wall failure had reached EL-763.0 m. The steel ribs were squeezed out into the shaft at EL 755.0-763.0 m in the zone of intersection of fault D-3 with the southwestern wall. Above EL-763.0 m, the steel ribs have not sustained deformation as subsequent failure developed not upward the shaft, but at an angle along fault D-3. With further development of this process, pieces of weak rock fell down from zone D-3 into the shaft from a big height which created additional dynamic loads on the rock support. By the end of June 2000, sand filling in the shaft had been brought to EL-765.0 m which slowed down further deterioration of the shaft walls. But during this time a large cavity got formed in zone D-3 because of large rock falls into the shaft. The roof of this cavity was caving in continuously while the cavity was going upward along the zone of diagonal fault. On 03.07.2000 it reached EL-840.0 m and a crater of about 1000 cum in volume and 7-8 m deep developed at the site. In plan, part of this crater extends beyond the limits of the site at EL-840.0 m. The Russian Consultants and THDC Engineers came to the unanimous decision about backfill grouting of zone D-3 at the stretch between MGS-3 & MGS-4. This work was started early in July 2000. More than 1000 cum of cement-sand mortar was injected into this zone.

Reconstruction of the shaft including its transition with HRT-4 became the most challenging and stupendous work for THDC. Since all 4 HRTs were already constructed, concrete lined, it was all the more important to reconstruct the shaft at the same location. Reconstruction of the shaft was taken up in Sept 2000 by backfilling the shaft with sand from EL-730.0 to EL-771.50 m. With the filling of the shaft, facilities for material handling were developed by making a bench at EL-771.50 m. All the ribs already installed in the shaft had buckled/twisted and were replaced by new ribs at a spacing of 1.0 m. The walls were concreted and rock bolted properly to ensure that no further movement of the rock was prevailing at the time.

The concrete lining was simultaneously taken up above EL-771.50 to EL-840.0 by making a key at EL-771.50 m(1.0X 1.5 m in two parts).

The excavation of the shaft was carried out in phases i.e. first upto EL-760.0 and then upto EL-757.0 , then upto EL-750.0 m and finally upto EL-746.50 m. The shaft was supported by construction of multi-drift 2X2 m size ensuring that the drifts are excavated at least 2.0 m deep into the sound rock. The drifts were filled with reinforced concreting. A total of 9 nos. drifts were constructed at EL-746.50 m. The construction of drifts had virtually stopped any movement in the shaft due to presence of D-3 shear and stabilized the shaft.
The other half of the shaft was backfilled with reinforced concrete to a depth of 1.5 m, thus protecting the entire shaft. Further excavation of the shaft was carried out by making a pilot shaft from EL-746.50 to EL-733.0 m. The widening of the shaft was carried out by removing the loose soil and erection of steel ribs followed by backfill concrete. The entire shaft was earlier shotcreted properly before erection of steel ribs. The widening of the shaft upto EL733.0 i.e. top of the transition was completed in May 2004. The reinforced concrete lining of the shaft was taken up from EL-736.0 to EL-771.0 m. The second stage concreting was taken up from EL-746.0 to EL-840.0 m and the same was completed in May 2005. The second stage concrete lining from EL-734.0 to EL-746.0 was taken up after completion of transition with HRT-4.

6.0 Construction of HRT-4 and MGS-4 transition

The construction of transition portion of MGS-4/HRT-4 which got collapsed in the year March 2000 was reconstructed under extreme trying and difficult circumstances. The entire collapsed mass including the deformed ribs of MGS-4 from EL-770.0 to E-736.0 had accumulated in the transition zone from EL-736.0 to EL-720.0 in a total length of 45.0 m. The geological conditions in this area has been defined elaborately as above. The entire area was having highly weathered rock mass and deformed rock strata full of clay and loose soil with highly fractured mass.
Fig. 5: L-section of HRT- 4 at transition

The reconstruction of the transition zone was taken up from Feb’2004 in D/S side and from Sept’2004 in U/S side. As already stated above, the already erected steel ribs were buckled and deformed in the entire reach of 45.0 m. this meant removal of the deformed ribs, treatment of rock mass and installation of new ribs in the entire reach of transition followed by reinforced concrete.

The stabilization measures of the transition zone was taken up from D/S side as a first priority. To start with treatment of the circular portion of HRT-4 invert in a length of 4.30 m was taken up. The erection of steel ribs of size 400X140 mm in the overt portion was taken up followed by backfill concrete and backfill grouting 76 mm dia core/bore hole drilling 4 to 5 kg/cm2 pressure. This followed stabilization measures in the side walls of the circular portion of the tunnel which included 6.0 m deep 32 mm dia rock bolts in the entire length of 4.30 m followed by reinforced concrete of grade M-25. The twisted and buckled ribs in the crown portion of this reach were removed followed by removal of the loose rock mass. Instillation of new steel ribs of size 400X140 mm followed by reinforced concrete lining.

Stabilization measures in the next reach of 5.50 m portion was started. To begin with treatment of the crown portion was taken up by making small drifts followed by rock bolting and erection of steel supports of size 400X140 mm. The entire crown portion was stabilized with the erection of new steel ribs followed by reinforced concrete. The next step was stabilization of the side walls with removal of loose rock mass, installation of rock bolts, shotcreting and erection of new steel ribs followed by reinforced concrete.
The stabilization measures up to the centerline of the gate shafts was taken up in 3 steps. The first phase was removal of loose rock mass from the crown followed by erection of steel ribs in the entire crown portion of the transition. This was followed by stabilization measures of the side walls followed by reinforced concrete.

The stabilization measures from U/S side was taken up from Sept'2004. The stabilization measures consisted of removal of loose rock mass, treatment of the rock mass by installation of rock bolts followed by installation of steel ribs of size 400X 140 mm. The crown portion of the transition was started with multi-drift method which included removal of loose soil in the phased manner from small pockets of size 2X2 m. The installation of the steel ribs was done in the crown portion of the constructed drifts in 3 phases. The erection of the steel rib in the crown portion was carried out up to the centerline of the gate shafts in the entire reach of the transition portion. This activity was followed by stabilization of the side walls with removal of loose rock mass, installation of rock bolts, shotcreting and erection of steel ribs.

After stabilization measures in the crown portion of the transition was completed both from U/S and D/S side and also the stabilization measures of the side walls was completed, concrete lining in the entire transition zone both in the U/S and D/S was taken up simultaneously. Concrete lining in the invert portion was carried out in the U/S and D/S side simultaneously.

The concreting of the complete shaft along with the transition with HRT-4 was completed successfully in Oct’2005. After completion of the structure intensive grouting operations were taken up. The sequence of grouting operations with intensive drilling operations to stabilize and strengthen the shaft and platform EL-840.0 is given herein under:

**Transition Of MGS-4 with HRT-4**

76 mm dia holes up to rock mass were drilled in the crown at highest point and backfill grouting was resorted to fill any cavity between rock and concrete up to a pressure of 4 to 5 kg/cm. About 8640 nos. of cement bags were injected to fill the hollow space in the crown.

During second stage lining of transition, steel perforated pipes of dia 80 mm up to a length of 2 to 2.5 m were left up to rock to facilitate drilling and grouting at later stage. These pipes were placed in a pre-decided manner in a pattern in which consolidation grouting was proposed in a fan like manner.

From the center of MGS-4, 20 m in upstream side and 16 m in down stream side pipes were laid in pattern (i.e. 3.0 m c/c in staggered manner in entire overt & invert portions) all around the transition tunnel. Total 274 nos. of holes in which perforated pipes were provided for taking up grouting.

First 76 mm dia holes were drilled inside in the pipe up to 1 m depth in rock. Grouting was taken up to 20 kg/cm² pressure. Grouting was started first with consistency of 10:1 (W:C) and later it was packed with a consistency of 0.6:1 (W:C).

After few days when grout consolidation in the hole and around the rock mass and behind the concrete took place, again the hole was re-drilled 76 mm dia up to a depth of 6.0 m in the rock. Grouting procedure as enumerated above was repeated with a pressure of 20 atm.

For completing the above said procedure 6600 bags of cement were injected in the 36m reach of transition zone itself. Further for checking the efficacy of above grouting work, test bore holes
were drilled at random in this zone. 14 nos. of test bore holes were designated for evaluating the effect of grouting.

For test bore hole again 76 mm dia bore holes were drilled through the lining concrete upto a depth of 6 m inside the rock. Water was injected through the bore hole at a pressure of 20 atm. For application of water test the loss of water shall be less than 2 litres per minute. Then the area shall be declared as properly grouted.

**Out of 14 test bore holes not even a single bore hole failed in water test. The entire transition was properly grouted.**

**Maintenance Gate Shafts**

After concrete lining the shaft was also grouted with the same procedure and standard holes were drilled by making platform inside shaft at EL-734, EL-743, EL-746 and EL-770 m. Holes were drilled in fan link manner around the shaft. About 16 nos. holes were drilled at every elevation around by diamond drilling method of dia 76 mm. Pressure applied for grout was upto 15 Kg/cm2. Total cement injected was 1057 bags.

After completing above said grouting from inside again 38 nos. vertical holes of dia 152 mm with ODEX method was drilled from platform at EL-840. These holes were drilled in two stages of 35.0 m each, drilled & re-drilled and grouted with high pressure of 15-17 kg/cm2. After a depth of 70 m again these holes were re-drilled & drilled and grouted after every 8 & 6 m height of shaft. This procedure was repeated in alternate holes till we reached the transition zone below. These holes were drilled in stages of 8/6 M and upto a depth of 124 m by way of grouting, re-drilling and re-grouting.

**Sketches of these grouting scheme are attached herewith for ready reference.**

From MGS-4 above 24950 nos. of bags were injected to fill cavities around shaft from outside. By way of resorting to this intensive grouting scheme the entire MGS-4 shaft along with platform from where the D-shear zone passes has been made fully stabilized. For taking care of shear zone by following method :-

Vertical holes of 76 mm dia upto a depth of 60 m were drilled from EL-840 between MGS-3 & 4 and grouting was resorted to. Several such holes were drilled and grouted with a pressure upto 5 kg/cm2 and consistency (10:1 to 1:0.8) (W:C) and total 4800 bags of cement were injected.

Shear zone affected the MGS-3 upto depth of 50/60 m from EL-840 plated that area was also treated by same method as done in MGS-4 i.e. 152 mm ODEX method 10 m stage drilled & re-drilling grouting with 38 nos. holes, total 51620 bags is injected to fill this cavity. The pressure was kept upto 15 kg/cm2.

The EL-840 platform was also grouted after tension crack appear in Mar’ 1998, 776 bags through gravity consistency (2:0.1:1 to 1:0.1:0.6) W:S:C pressure upto 4 kg/cm2 & 13130 bags cement during cavity formation MGS-3 in July 2000, by grouting of consistency (2:0.1:1 to 1:0.1:0.6) W:S pressure upto 5 Kg/cm2.
Total grouting in transition portion = 15,240 bags  
Maintenance Gate Shaft = 4,715 bags  
Grouting at EL-840 platform = 95,976 bags  
Curtain Grouting U/S of MGS-4 = 20,872 bags  

Total 1,36,803 bags  

Thus, it can be seen that to stabilize Platform at EL-840.0 & MGS-4 an elaborate arrangement and technique was adopted which has resulted in stabilizing the shaft and platform at EL-840.0 m.

References

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