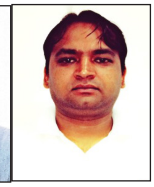
Geological and Geotechnical Challenges Faced during Excavation of the Underground Powerhouse and Head Race Tunnel for Mangdechhu Project, Bhutan, India

A.K. Mishra

R.K. Chaudhary

P. Punetha



I. Ahmed

A.K. Mishra, R.K. Chaudhary, P. Punetha, I. Ahmed

Introduction

The under construction 720 MW Mangdechhu Hydro-Electric Project (MHEP), comprises of 112 m high concrete gravity dam, two underground desilting chambers, a 13.5209 km long Head Race Tunnel (HRT), a 13.5 m diameter and 152 m deep open to sky surge shaft, two pressure shafts of stepped configuration comprising four horizontal and three vertical limbs and finally bifurcating into two penstock each, an underground power house complex, and a 1.333 km long tail race tunnel. During the excavation of the underground power house complex and HRT various geological and geotechnical challenges were faced which were successfully tackled by adopting advanced excavation methodology, support system and monitoring rock mass behaviour by systematic geotechnical instrumentation program.

During the excavation of the central gullet of the underground power house caverns a previously un-envisaged shear zone of 1.5 to 2 m thickness with associated fracture zone of 2 to 6 m thickness was encountered. The shear/fracture zone aligned along the long axis of the underground caverns and cutting across both machine hall and transformer hall caverns appeared detrimental for both the caverns. Apart from encountering a shear zone, loose falls occurred between RD 0 to 30 m of machine hall cavern resulting in formation of 10.5 m high cavity and while widening of central gullet of transformer hall sharp rise in deformation and load were observed at RD 13 m. Total displacement and load observed during widening were 18.1 mm and 290.7 KN respectively. By adopting advanced excavation methodology and support system, comprising of in-situ treatment of the weak feature, stability analysis of the caverns by systematically evaluating the rock mass behavior, by three dimensional numerical modeling to optimizing excavation methodology and sequence, along with revision of support system and progressively validating it and monitoring rock mass behavior by a systematic geotechnical instrumentation program excavation of caverns was successfully completed within the schedule.

Major challenges faced while excavating HRT includes poor rock mass strata, tunnel squeezing and mild rock burst in high cover zone, heavy ingress of water, presence of shear zones and structural control on tunnelling. These challenges were successfully tackled by implementing advance tunnelling methodology comprising of probing strata ahead of tunnel prior to excavation, ground improvement by pre-excavation grouting, fore poling and pipe roofing, controlled blasting, installation of adequate and timely support system in form of shotcrete, and rock bolts as primary support along with installation of steel ribs and lattice girders as secondary support, and drainage of water away from the face by providing drainage holes prior to face advancement in water charged strata, with simultaneous monitoring rock mass behavior by aid of geotechnical instrumentation.

Engineering design of the powerhouse complex and salient features of HRT

The powerhouse complex comprises of 155 m long, 23 m wide and 41 m high machine hall and a 135.5 m long, 18 m wide and 23 m high transformer hall caverns, separated by 40 m thick rock pillar between them. The caverns are aligned in N80°E direction which is almost parallel to the principal horizontal stress direction of N70°E. The access to the caverns is provided by a 679 m long, 8 m diameter main access tunnel and a 363 m long, 6.5 m diameter ventilation tunnel. The machine hall cavern comprises of 30m long service bay with floor level at elevation (El.) 1028.4 m, a 25 m long control block comprising of five floors and a 100 m long machine hall area for housing four Pelton turbines of 180 MW each. A 1.45 m wide crane beam is provided at El.1041 m for erecting two Electric Overhead Travelling (EOT) crane of 225 tons capacity each. The transformer hall cavern comprises of transformer floor to house thirteen transformers and a GIS floor at El. 1040 m for housing gas insulated switch gear (Figure 1). The power generated by turbines will be transferred to transformer hall through four bus ducts from there it will be evacuated via 347.5 m long cable tunnel opening into pothead yard located along the highway on the left bank of the river.

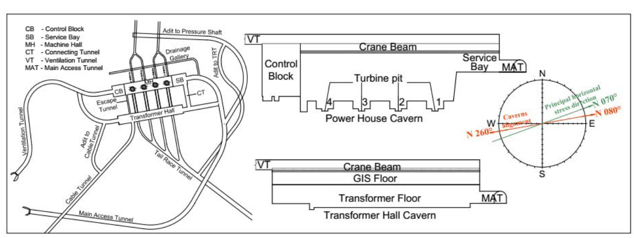


Figure 1. Layout plan and sections of underground power house complex.

The 6.5 m diameter, modified horse shoe shaped, and 13.5209 km long HRT is located along the left bank of the Mangdechhu River. The invert level at the intake is at El. 1718 m while that at the terminal end it is at El. 1650 m. With a total drop of 68 m the gradient along the HRT is 1:198.86 m which allows a design discharge of 118 cumec to flow at a velocity of 3.71 m/sec. To facilitate excavation of HRT, five construction adits were provisioned to provide ten faces for excavation.

Geological set up of the powerhouse and HRT area

The project area falls in Central Crystallines of Higher Himalaya (Ganser, 1964; Ganser, 1983) and stratigraphically is represented by two distinct suits of rocks namely the Thimpu Gneissic Complex and the Chekha Formation; intruded by granitic batholith. The Thimphu Gneissic Complex comprises of granite gneiss, augen gneiss with subordinate bands of garnetiferous mica schist and phyllite with lensoidal intrusions of granite and mafic rocks. The Chekha Formation mainly comprises of micaceous quartzite and mica schist with pegmatitic and amphibolitic intrusions (Nautiyal et al., 1964; Guha, 1979; Jangpangi, 1974; Bhargava, 1995).

The powerhouse caverns are located in Chekha Formation and are housed below a spur in between two streams with maximum super incumbent cover of 180 m. The bed rock comprises of quartzite with bands of mica schist and intrusive pegmatite/granite. The foliation dips towards south and south west in some of the outcrops near the highway, whereas in the stream cuttings on both the sides of the spur westerly dipping foliations were observed, indicating broad open fold plunging towards the river. The rock mass encountered during the excavation was represented by grey colored, slight to moderately jointed, strong quartzite with intercalated schist bands of 1 to 5 cm thickness and intruded by pegmatite and amphibolites. Both the caverns were traversed by a 1.5 to 2 m thick shear zone with attitude of N30°E – S30°W dipping 10 to 24°due N60°W. The mean attitude of foliation observed along the caverns was N 28° W – S 28° E dipping 33° due S 62° W. Apart from foliation the rock mass was traversed by three other prominent sets of discontinuities along with some randomly oriented joints. The attitude of discontinuities along with the attitude of shear zone is shown in Figure 2

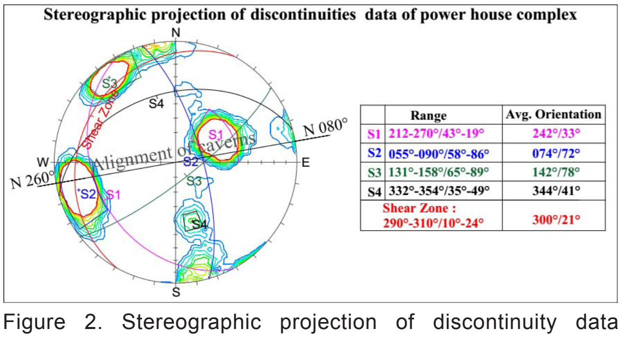


Figure 2. Stereographic projection of discontinuity data recorded along powerhouse caverns.

Along the HRT alignment, the rocks of Thimphu Gneissic Complex were encountered up to RD 5394 m of HRT including adit-1 and 2. Tertiary leuco-granitic batholith which on surface covers central part of HRT between RD 4070 to 7000 m was encountered only between RD 5394 to 5620 m and was represented by granitic gneiss of contact zone area. The Chekha Formation dominates the downstream side of the HRT from RD 5620 m onwards including adit-3, 4 and 5. The plan and section showing different litho zones as encountered along HRT are shown in Figure 3.

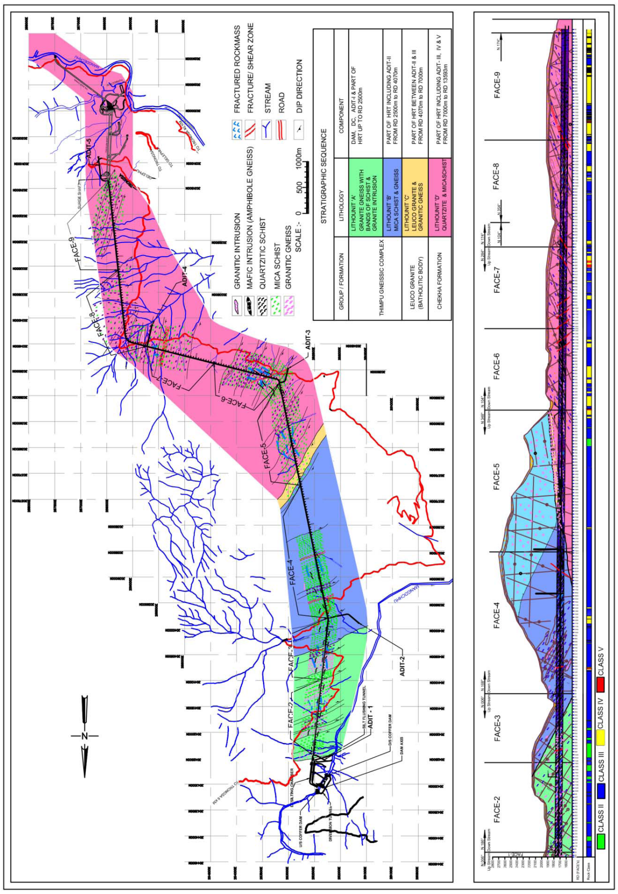


Figure 3. Plan and section showing encountered litho-zones

Geological and geotechnical challenges faced during excavation of powerhouse and HRT

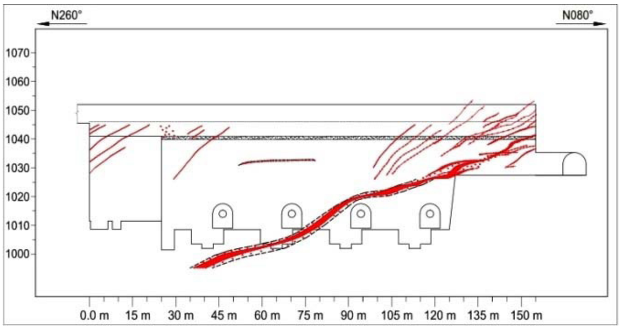
The major geological and geotechnical challenges faced during excavation of underground powerhouse complex and HRT are discussed in Part A and B respectively.

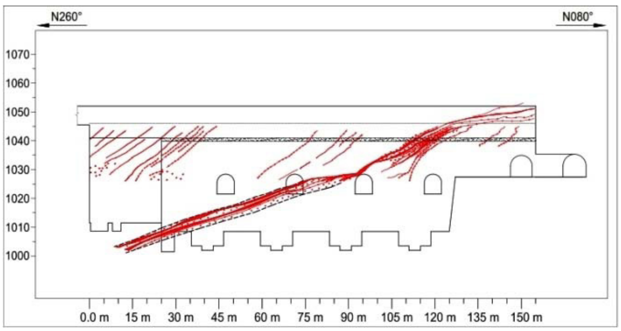
Part A: Geological and Geotechnical Challenges Faced during Excavation of Powerhouse

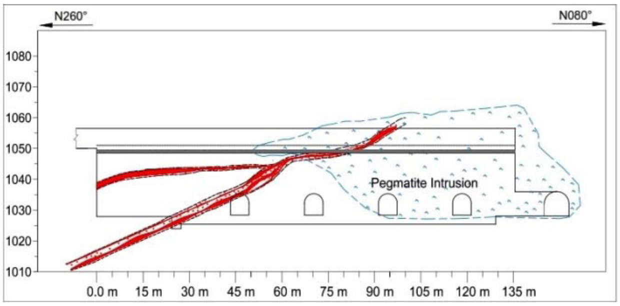
Encountering of 1.5 to 2 m thick shear zone with associated fracture zone of 2 to 6 m thickness along the central gullets of both the caverns, formation of cavity between RD 0 to 30 m in machine hall cavern and sharp rise in deformation and stress observed at RD 13 m of transformer hall cavern, were the major challenges faced during the excavation of the caverns. The challenges, their implications and tackling methodology are discussed below:

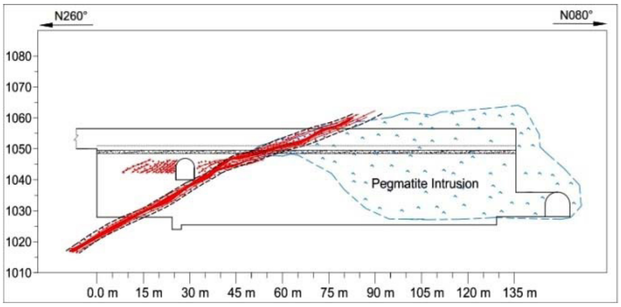
Shear zone cutting along and across the caverns

During the excavation of the central gullets, a shear zone of 1.5 to 2 m thickness with associated fracture zone of 2 to 6 m thickness was encountered between RD 125 to 155 m of machine hall and between RD 70 to 90 m of transformer hall caverns. The attitude of the shear zone was N30°E–S30°W dipping 10 to 24° due N60°W. The disposition of the shear was cutting along the walls of the caverns and also the rock pillar between them (Figure 4).









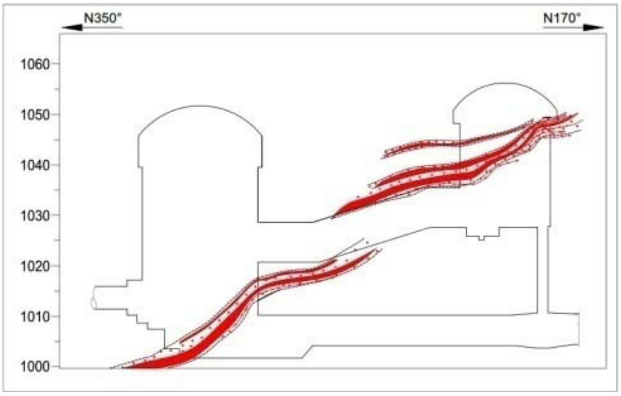


Figure 4. Geological section showing disposition of shear zone along (a) upstream wall (b) downstream wall of power house cavern (c) upstream wall (d) downstream wall of transformer hall cavern (e) across the caverns.

The presence of shear/fractured zone cutting along and across both the caverns was expected to have the following implications during excavation:

1. Instability of the crown above the service bay between RD 105 to 155 m in machine hall cavern and between RD 70 to 110 m in transformer hall cavern.
2. Differential settlements in the crane beams.
3. Instability of both upstream and downstream walls of the caverns
4. Instability of the rock column between the two caverns corresponding to the area above and between the bus ducts.
5. Instability of the bus ducts opening and rock pillar between them.

Loose falls in machine hall cavern

Two loose falls occurred between RD 0 to 30 m after widening of the central gullet resulting in formation of 10.5 m high cavity. First loose fall occurred between RD 0 to 12 m, extending 6m towards upstream side and 10 m towards downstream side. The height of the cavity formed due to loose fall was about 5 m. Second loose fall occurred between RD 2 to 30 m. This loose fall extended the height of previous cavity to 10.5 m (Figure 5).

Figure 5. Photograph showing (a) cavity formed by loose falls (b) Rock mass condition of the cavity zone.

Assessment of the geological conditions along cavity area revealed presence of two shear seams and fractured rock horizons about 2 m above the periphery of the crown. Further low cover zone between the crown of the cavern and overlying drift with distressed intervening rock mass with a possibility of yield zone of both the excavations overlapping each other and inadequate support system, allowed the radial loosening to encroach deeper into the crown. Encountering of shear zone and formation of cavity induced fear psychosis among the workers who refused to continue working in the caverns. An additional access to the cavern was provided by excavating a glory hole escape way above the main access tunnel, to eliminate their fear psychosis of getting trapped into the cavern in an event of any other loose fall.

Rising deformation and stress in transformer hall cavern

During widening of the central gullet of transformer hall cavern, sharp rise in deformation and stress were recorded by instruments installed at RD 13 m. Total increase in displacement and load observed during widening was 18.1 mm and 290.7 KN respectively. The rise in load almost reached the 60% of the ultimate load bearing capacity of the installed support system. In view of continuous and sharp rise in deformation and stress, chances of roof collapse were high. To avoid any loose fall and formation of cavity widening of the cavern was cautiously undertaken by controlled and low intensity blasting. To arrest deformations and stresses steel rib support was also provided. After erection of the steel rib support comprising of ISMB 350 steel ribs at a spacing 0.5 m, the rise in deformation and load/stress ceased (Figure 6 a & b), indicating the stability of the crown after providing rigid support (Mishra et al., 2015a).

Treatment of the caverns

Presence of previously un-envisaged shear/fracture zone of 1.5 to 8 m thickness appeared detrimental for the stability of the caverns; as such, reassessment of stability of both the caverns, while also revalidating the rock support and excavation methodology, was undertaken. The consultant suggested shifting of the caverns by 40 m laterally toward valley side whereby the crown of the caverns could be relatively free from the effect of the shear zone. However, this would have resulted in the shear/fractured zone intersecting the turbine foundation and its continuity along the walls and rock column between the caverns would still have not been avoided, further this would have necessitated re-alignment of pressure shafts, which were excavated for a considerable length, along with realignment of main access tunnel, involving considerable time and financial losses (Mishra et. al., 2015b) Therefore, project authority opted for in-situ treatment of the shear/fracture zone.

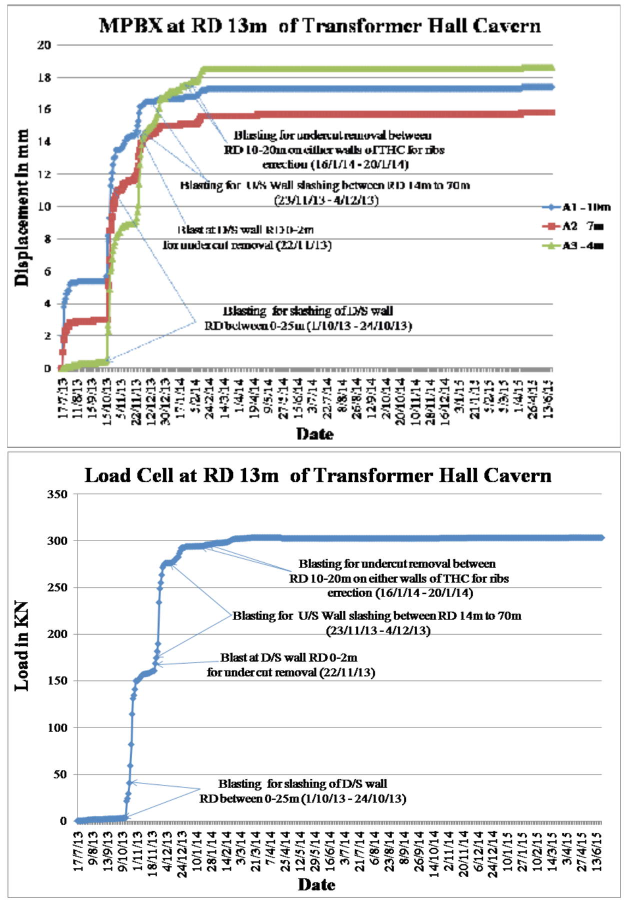


Figure 6. (a) Displacement v/s time graph of MPBX (b) Load v/s time graph of Load Cell at RD 13 m of transformer hall cavern.

To ensure long term stability of the crown reaches affected by shear/fractured zones, formation of cavity and observed rising deformation trends, steel ribs support comprising of ISMB 350 steel ribs, spaced 0.5 m c/c between RD 0 to 35 m and RD 100 to 155 m in the machine hall cavern and between RD 0 to 20 m and RD 60 to 120 m in the transformer hall cavern were provided, followed by extensive contact and consolidation grouting to the depth of 6 m into the rock mass. Longer rock bolts of 12 m length at spacing of 3 m were also provided along the crown of both the caverns in addition to 7.5 and 9 m long rock bolts as proposed in construction drawings. While the crown reaches affected by shear zone were strengthened by providing steel rib support, the shear zone along the wall of the caverns was treated progressively during course of excavation by scooping out the sheared material to a depth of 3 m, followed by cleaning and filling of the void with M30 grade concrete. The concrete cladding was stitched to the bed rock by 18 m long rock bolts. Consolidation grouting to a depth of 20 m with ultrafine cement with grout consistency ranging from 5:1 to 0.8:1 (water cement ratio by weight) and pressure of up to 14 kg/cm^2 was also carried out. To assess the behavior of the rock mass during further benching down of the caverns and to validate/optimize support system progressively, three dimensional numerical modeling studies using 3-DEC software, which is a three-dimensional numerical program based on the distinct element method for discontinuum modeling (Anon, 2013) of the caverns were undertaken. Based on the results of the numerical modeling analysis; excavation methodology, sequence and support system were optimized. Revalidation of the installed support system was also undertaken after each benching sequence.

Excavation methodology and support system

In view of the encountered shear zone and high displacements anticipated along the walls, the excavation sequence as suggested in contract document appeared unsuitable as such revised excavation methodology and sequence was adopted. In the revised excavation methodology after widening of the crown to full width benching down of both the caverns was taken up by restricting the bench height to 3 m instead of 6 m as planned earlier, with concurrent installation of the revised support system and revalidating it before further benching down by 3-DEC analysis. The sequence of excavation of bus ducts and TRT manifolds was evaluated by 3-DEC modeling. It was found that excavation of bus ducts before excavation of TRT manifolds results in lesser deformations and better stability (NIRM, 2015; Mishra et al., 2017a; Mishra et al., 2017b), accordingly excavation of bus ducts was taken up before excavation of TRT manifolds. The sequence of excavation adopted for excavation of both the caverns and appurtenant structures is schematically shown in Figure 7.

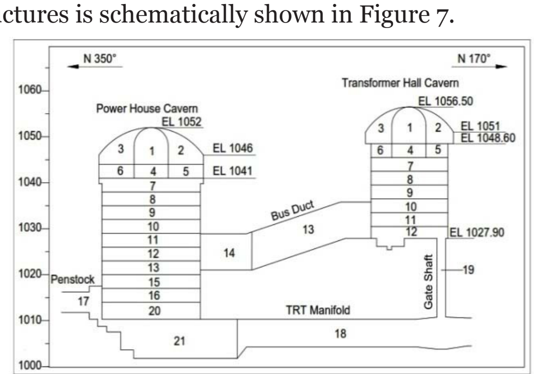


Figure 7. Schematic section showing sequence adopted for excavation of caverns and appurtenant structures.

The support system for crown of both the caverns as per design drawings comprised of 200 mm thick shotcrete and 7.5 m and 9 m long rock bolts, while along the walls shotcrete of 150 mm thickness and rock bolts of 9 m length were proposed. Subsequent to encountering of shear zone the efficacy of suggested support system to maintain the stability of the caverns was evaluated by three dimensional numerical modeling, and it was found that the proposed support system in view of changed geological conditions would not be adequate to maintain the stability of the caverns. Based on the results of the analysis the support system was modified to ensure the safety of the structure. The revised support system installed along the crowns of the caverns comprised of 250 mm thick steel fibre reinforced shotcrete, followed by installation of 12 m long, 36 mm dia. rock bolts in addition to already installed 7.5 and 9 m long rock bolts. To ensure stability of the crown affected by shear zone secondary support in form of passive support, comprising of ISMB 350 steel ribs, and spaced 50 cm centre to centre were installed between RD 100 to 155 m in machine hall and RD 60 to 120 m in transformer hall caverns. Steel ribs between RD 0 to 35 m were provided in machine hall cavern to take the load of the back-fill concrete. Steel ribs were also provided between RD 0 to 20 m of transformer hall cavern to arrest increasing deformations and inward movement of stress as evident by rising trend of load cell and MPBX data (Mishra et. al., 2017a).

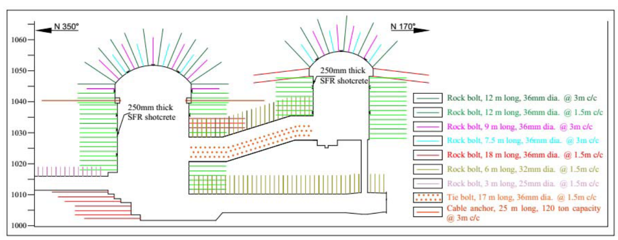
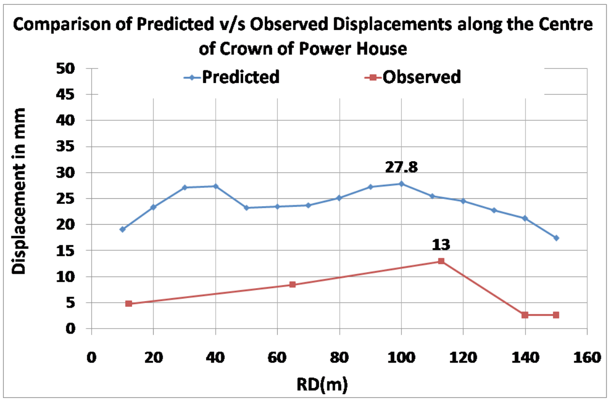
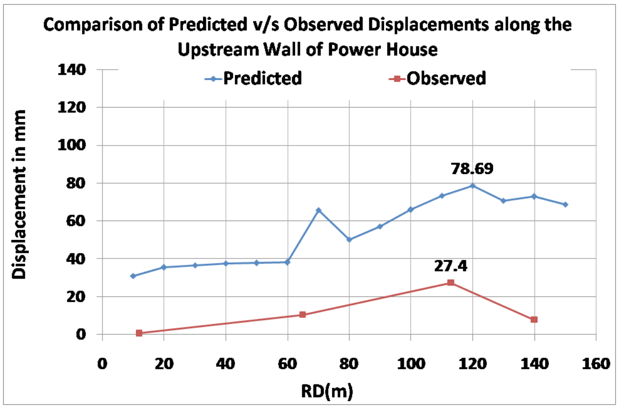


Figure 8. Schematic section showing installed support system along caverns and appurtenant structures.

The support system installed along the wall of the caverns comprised of 250 mm thick shotcrete, followed by 12 m long, 36 mm diameter rock bolts at a spacing of 1.5 m centre to centre in staggered pattern. The cladding along the shear zone area was stitched to the bed rock by 18 m long rock bolts at a spacing of 1.5 m. The rock mass below El. 1009 m along upstream wall of machine hall cavern was also supported by 18 m long rock bolts. The gable end walls of both the caverns were supported by shotcrete with wire mesh followed by 7.5 m long, 36 mm dia. rock bolts at a spacing of 1.5 m in staggered pattern. The crane beams of machine hall cavern were tied to the bed rock by 25 m long, 120-ton capacity cable anchors spaced 3 m apart. The rock pillar between the bus ducts was stitched by 17 m long 36 mm dia. tie bolts spaced 1.5 m apart in staggered pattern. The installed support along both the caverns and appurtenant structures is schematically shown in Figure 8. To further improve the rock mass strength consolidation grouting by aid of 6 m deep holes into the rock mass was carried out, along the caverns except for the shear zone area, where the depth of grouting was increased to 20 m and ultra fine cement was used instead of ordinary cement for effective penetration of the grout in fine shear zone material.





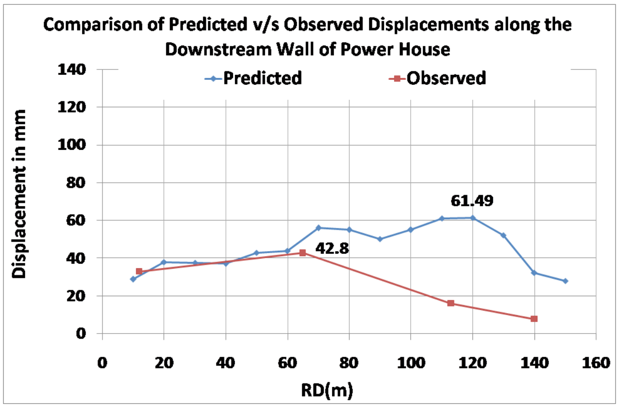
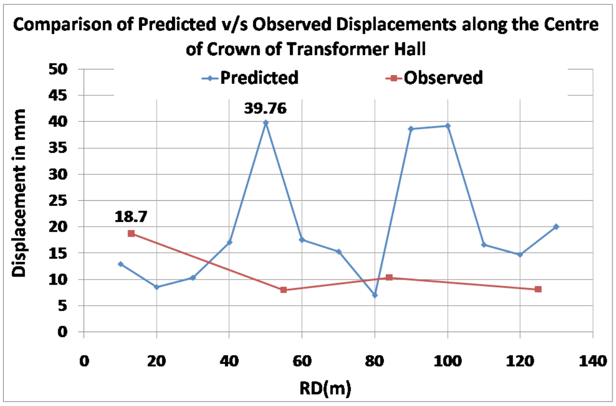


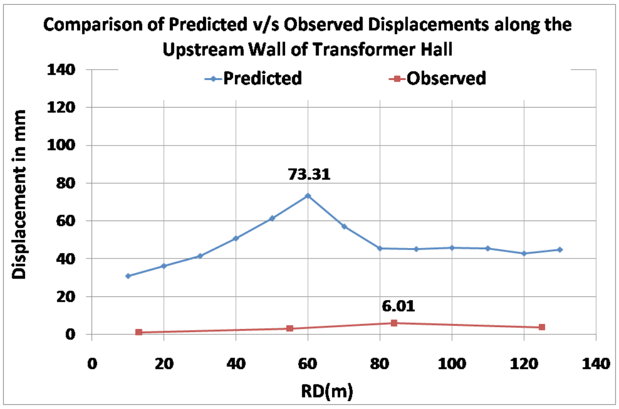
Figure 9. Comparison of anticipated and observed deformations along (a) Crown (b) Upstream wall (c) Downstream wall of machine hall cavern.

Monitoring of the caverns

The behavior of the rock mass during the course of excavation was carefully monitored by aid of a systematic geotechnical instrumentation program. The parameters monitored include change in stress patterns by aid of load cells, displacement of rock mass at different depth inside the excavated periphery of the caverns by single and multi point borehole extensometers, while surface deformations were monitored by tape convergence point and survey target points. A total of 261 instruments (43 MPBX, 28 SPBX, 58 load cells, 8 survey targets and 124 TCP) were installed in the entire power house complex. The readings were recorded and analyzed on daily basis. The observed displacements were regularly compared with the anticipated deformations as obtained from numerical analysis. The deformations recorded at site during excavation were much less than those anticipated from 3-DEC numerical modeling analysis. The maximum deformation anticipated along the crown in machine hall cavern was 27.8 mm, while actual observed maximum deformations along crown was 13 mm only. Similarly, maximum anticipated deformations along upstream and downstream walls were 78.69 and 61.49 mm respectively against which only 27.4 and 42.8 mm of deformation was recorded along upstream and downstream walls, respectively (Figure 9a, b & c).

Along the crown in transformer hall maximum deformations of 39.76 was anticipated of which only 18.7 mm of deformation was observed during excavation. Along upstream and downstream walls of transformer hall cavern maximum deformation recorded was 6.01 and 5.7 mm respectively against anticipated deformation of 73.31 and 60.72 mm respectively (Figure 10 a, b & c).





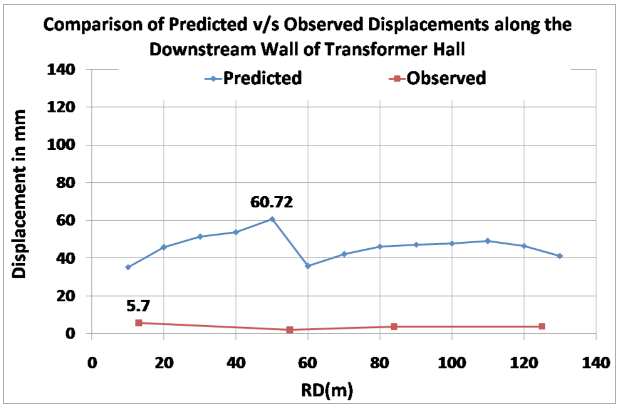


Figure 10. Comparison of anticipated and observed deformations along (a) Crown (b) Upstream wall (c) Downstream wall of transformer hall cavern.

Part B: Geological and Geotechnical Challenges Faced during HRT Excavation

The major challenges faced during HRT excavation include, structural control on tunnelling, tunnel squeezing and rock burst in high cover zones where super incumbent cover varied from 750 to 1000 m, heavy ingress of water of up to 120 liters/minute, encountering of shear zones and poor rock mass conditions, where excavation was carried out by continuous rib support along with strengthening crown by aid of pipe roofing, fore polling and grouting which were time consuming and resulted in slow progress, as such delaying the overall construction schedule. To overcome these challenges and to maintain schedule and ensuring stability of the structure, continuous improvement and changes in excavation methodology and support system were implemented. The geological and geotechnical challenges faced, and methodology devised to tackle them is discussed below.

Structural control on tunnel excavation

Tunnelling and tunnel stability to a great extent is governed by the relation of tunnel alignment with the attitude of discontinuities. The structural control on tunnel determines the quantum of over breaks, stability and excavation rate. Along the HRT, structural control on tunnelling was observed along Face-6, 7, 8, 9 and 10, which were excavated through gently dipping strata of Chekha Formation intersected by four major set of discontinuities. The mean attitude of the discontinuities recorded along these faces and their relation with respect to tunnel alignment/drive direction is tabulated in Table 1.

Gently dipping and thinly foliated strata along Face-6, 7, 8, 9 and 10 induced slabbing along the crown. Along Face-6 and 7, tunnel alignment was oblique at high angle to the strike of foliation as such slabbing was less pronounced while along Face-8, 9 and 10, which were aligned almost parallel to the strike of foliation, slabbing was more pronounced and governed the tunnel profile. Steeply dipping S2 joint set which also strikes parallel to tunnel alignment also contributed adversely in slabbing and also resulted in failure along the walls of the tunnel.

Intersection of discontinuities along Face-6 occasionally gave rise to wedge failure along the left side of the crown. However, gentle foliation dips restricted, the height of unstable wedges to 1 to 1.5 m. Along Face-7, intersections of steeply dipping S2, S3 and S4 joint set resulted in formation of wedges of considerable height of 3 to 5 m towards the right side of crown. Although intersection of steeply dipping joint set gave rise to formation of unstable wedges of considerable height but gently dipping foliation joint, which cuts across the wedges generally, restricted the wedge formation to its theoretical apex height, as such the height of the wedges formed was generally less than that expected

Face (drive direction)	Discontinuity set	S1	S2a	S2b	S3	S4
Face-6 ($N1040^0$)	Strike	030^0-210^0	167^0-347^0		070^0-250^0	150-330^0
	Dip direction/amount	$300^0/20^0$	$077^0/60^0$		$160^0/67^0$	$240^0/26^0$
	Relation/angle with drive direction	74^0	63^0		34^0	46^0
Face-7 ($N284^0$)	Strike	000^0-180^0	173^0-353^0		090^0-270^0	065^0-245^0
	Dip direction/amount	$270^0/27^0$	$083^0/64^0$		$180^0/66^0$	$335^0/65^0$
	Relation/angle with drive direction	76^0	69^0		14^0	39^0
Face-8 ($N174^0$)	Strike	160^0-340^0	175^0-355^0	005^0-185^0	084^0-264^0	070^0-250^0
	Dip direction/amount	$250^0/27^0$	$085^0/65^0$	$275^0/67^0$	$174^0/67^0$	340^065^0
	Relation/angle with drive direction	14^0	1^0	11^0	90^0	76^0
Face-9 ($N354^0$)	Strike	010^0-190^0	001^0-181^0	002^0-182^0	089^0-269^0	077^0-257^0
	Dip direction/amount	$280^0/24^0$	$091^0/78^0$	$272^0/78^0$	$179^0/80^0$	$347^0/78^0$
	Relation/angle with drive direction	16^0	7^0	8^0	85^0	83^0
Face-10 ($N174^0$)	Strike	028^0-208^0	170^0-350^0	005^0-185^0	087^0-267^0	081^0-261^0
	Dip direction/amount	$298^0/25^0$	$080^0/61^0$	$275^0/81^0$	$177^0/78^0$	$351^0/84^0$
	Relation/angle with drive direction	34^0	4^0	11^0	87^0	87^0

Table 1: Attitude of major discontinuities along different faces and their relation with tunnel alignment

with a flat end instead of a conical apex. Along Face-8 and 9 intersections of steeply dipping joints gave rise to formation of cubical wedges with a flat apex formed by the dip surface of foliation, which cuts across the cubical wedges. Along Face-8, the wedges were formed along the right side of the crown while along Face-9, they were restricted to the left side of the crown.

Tunnel squeezing/closure and rock burst in high cover zones

Tunnel squeezing/closure and mild rock bursts were observed between RD 4035 to 5950 m along a total tunnel length of 1915 m, covering part of Face-4 and 5 under high cover zone where super incumbent cover varied from 750 to 1000 m. As per DPR (2008), strong and massive granite, with good tunnelling conditions was anticipated along this high cover stretch. However, during excavation, granite was not encountered, and tunnel negotiated through mica schist, quartzitic schist and quartz biotite gneiss. Fragile/incompetent biotite schist was encountered along the stretch being excavated through Face-4, while comparatively competent quartzitic schist and gneiss were encountered along Face-5. High vertical stresses along Face-4 induced tunnel squeezing/closure and buckling of the steel ribs (Figure 11).

Figure 11. Photograph of buckled ribs in high

Figure 12. Post excavation dilation resulting in slabbing

Along Face-5 in high cover zone mild rock bursts and post excavation slabbing due to dilation of rock mass under high stress with crown failure up to 1.5 m depth were observed (Figure 12). The most affected stretch of HRT where tunnel closure was observed lied between RD 1196 to 1428 m of Face-4, where convergence of more than 100 mm was generally observed, with a maximum convergence of 562 mm recorded at RD 1358 m. The tunnel closure observed along this stretch is shown in Figure 13.

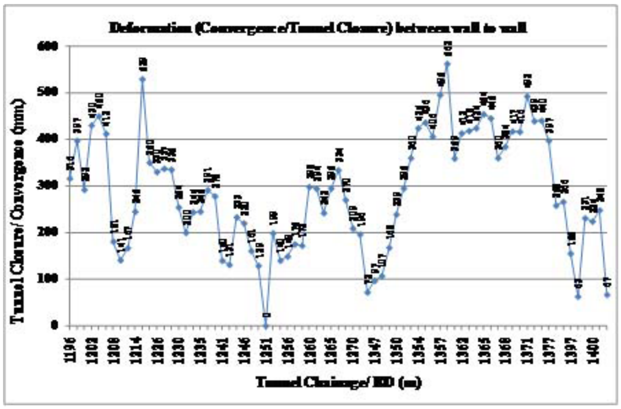


Figure 13. Graph showing convergence (wall to wall) along Face-4 (RD 1196 to 1428 m)

To tackle the squeezing, excavation was carried out by heading and benching method. Designed support system comprising of 100 mm Steel Fibre Reinforced Shotcrete (SFRS), 4 m long 25 mm dia. rock bolts were replaced by 150 mm SFRS, 6 m long, 32 mm dia. rock anchors with wider base plates (200 x 200 mm) for better load distribution. 9 to 12 m long, 76 mm dia. stress relief holes were provided for release of stresses. In class-IV (Poor) rock mass, the invert of the HRT was excavated in arc form instead of straight line, as in, D shaped. Invert struts, where squeezing continued during benching, were also provided. To maintain the tunnel profile, excavation was carried out by precision (line) drilling with low intensity blasting. Along Face-5, where mild rock bursts were observed/anticipated, excavation was carried out by controlled blasting with line drilling of dummy holes along the tunnel periphery to minimize the effect of blast induced vibrations on the rock mass along the tunnel periphery. SFRS was replaced by plain shotcrete along with welded steel wire mesh. 4 m long, 25 mm dia. rock bolts were replaced by 6 m long, 32 mm dia. rock anchors. Stress relief holes were also provided.

Ingress of ground water

Interception of ground water is one of the major challenges faced during the excavation of the tunnel. Ingress of ground water during tunnelling may result in face and roof collapse hampering the progress (Sharma and Tiwari, 2015). High pore pressure behind the tunnel periphery may also adversely affect the support system and cause its failure (Maurya et al., 2010). Along the

HRT, ground water with ingress of more than 100 liters/ minute was intercepted along Face-8, 7 and 2, with some intermittent stretches along Face-3, 5, 7 and 9 (Figure 14 and 15). Total stretch of water charged strata was 705 m.

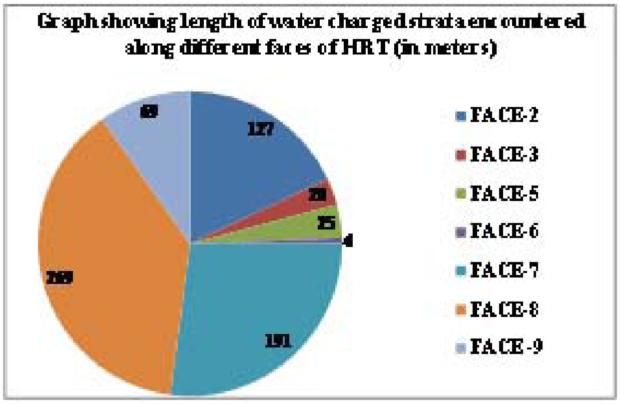


Figure 14. Graph showing length of water charged strata along various faces

Figure 15. Photograph showing ingress of water from drainage holes drilled prior to face advancement

The methodology adopted to negotiate through these water-charged reaches comprised of diverting water ahead of face by drilling 10 to 15 m long drainage holes of 76 mm diameter, inclined at an angle of 30 to 450 and radiating 15 to 200 outwards from the face (Figure 16), followed by normal excavation methodology as per rock class. Timely support as per rock class was provided to ensure tunnel stability. Additional drainage holes were also drilled after providing primary and secondary support to ensure free flow of water from the supported reach to avoid accumulation of water and building up of pore water pressure behind the supported tunnel span.

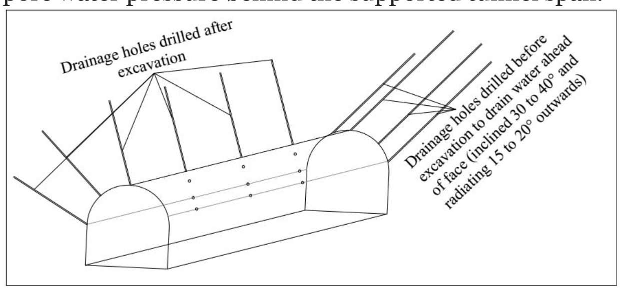


Figure 16. Schematic Section showing configuration of drainage holes.

Shear zones

Shear zones characterized by highly deformed, sheared/pulverized, water charged poor rock mass conditions, is the biggest nightmare of tunnel engineer. Serious tunnelling problems have been experienced when the rock mass is affected by shear zone. The most common problems associated with shear zones are; loose fall, muck flow, chimney formation, squeezing and heaving of the ground, face and crown collapse (Panthi, 2007; Maurya et al., 2010; Sharma and Tiwari, 2015). These problems are attributed to low stand up time of poor rock mass of shear zone where RMR value is less than 20 (Bieniawski, 1989). Along the entire length of the HRT, only two major shear zones were encountered. A foliation parallel shear zone of 7 to 8 m thickness was encountered at RD 287 m of Face-7 and a 4 m thick at RD 1424 m of Face-9. The encountered shear zones resulted in formation of 7 to 8 m high cavity between RD 287 to 293 m of Face-7 and RD 1424 to 1426 m along Face-9 with continuous loose fall. The continued loose fall was restricted by spraying many layers of shotcrete at a regular interval till falling of loose/sheared material stopped, followed by cautiously erecting steel ribs and back filling the cavity. After treatment of the cavity the extent of shear zones was probed by aid of drilling 15 m long probe holes. Based on the probe hole data, thickness of shear zone along Face-7 was estimated to be 8 m and its extent along the tunnel was 63 m i.e. up to RD 350 m (Figure 17), while the thickness of shear zone along Face-9 was deciphered to be 4 m and extending for a length of 43 m along the face i.e. up to RD 1467 m (Figure 18).

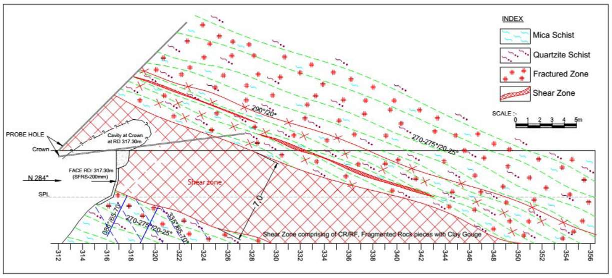


Figure 17. L-Section along HRT Face-7 showing disposition of shear zone between RD 287 to 350 m

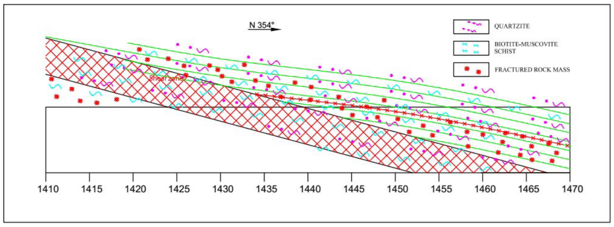


Figure 18. L-Section along HRT Face-9 showing disposition of shear zone between RD 1424 to 1467 m

Once the extent of the shear zone was estimated, excavation was cautiously carried out by heading and benching method. Prior to excavation, strata ahead of the face, especially, crown was strengthened by aid of pipe roofing, followed by grouting with micro-fine cement with a grout consistency varying from 3:1 to 1:1 (water: cement by weight), under pressure between 2 to 12 kg/cm². The excavation in the shear zone area was carried out by heading and benching method by mechanical means and comprised of first excavating 1 m span of tunnel up to spring level, followed by erection of crown portion of the rib and back filling the void between rib crown and rock surface. This was continued till the heading was completed for 5 to 6 m length. After heading for 5 to 6 m length and installation of support system along the crown, benching for this stretch was undertaken in stages of 1 to 2 m, followed by erection of column sections. The same methodology of excavating and supporting crown to 5 to 6 m length by heading in stages of 1 to 2 meter and then benching down to invert level in stages of 2 m was continued till the entire shear zone strata was negotiated.

Poor rock mass strata

Adverse geological conditions in the form of poor rock mass strata were encountered along the entire Face-10, major part of Face-9, 7 and 6, along with some intermittent stretches along Face-2, 3, 4, 5 and 8. Length of poor rock mass condition along various faces is graphically shown in Figure 19. Total length of poor rock mass conditions encountered along the HRT was 3387 m corresponding to 25% of total tunnel length. The poor rock mass along Face-6, 7, 9 and 10 comprised of slight to moderately weathered, thinly foliated biotite schist and quartzite with bands of schist of low strength and intense jointing and were characterized by presence of interfolial shear seams of 1 to 10 cm thickness. This low dipping stratum, especially along Face-9 and 10 where the strike of foliation was almost parallel to tunnel drive direction resulted in continued slabbing and over breaks.

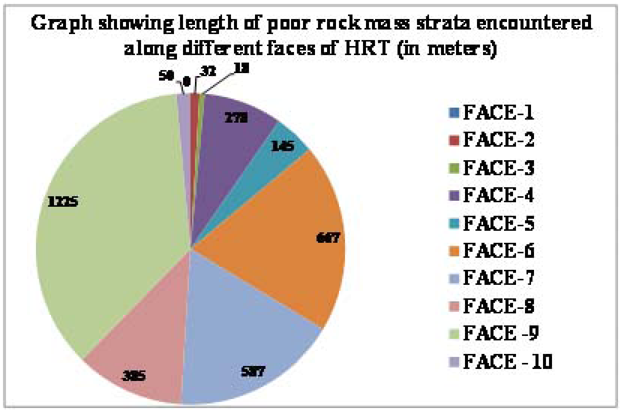


Figure 19. Graph showing length of poor rock mass strata along various faces

The excavation along poor rock mass stretches was carried out by providing steel ribs/lattice girders support. Lattice girders were provided along faces where tunnel profile was not structurally controlled and over breaks were negligible especially in Face-4 and 5 (Figure 20). Crown strengthening by pre-grouting, fore polling and pipe roofing to create an umbrella of consolidated rock mass around tunnel profile (Figure 21) was undertaken for increasing stand up time, for installation of primary and secondary support. The excavation methodology was continuously evolved so as to arrive at an optimum blasting pattern comprising of optimum drilling pattern, charging pattern and delay sequence to achieve desired pull with minimum over breaks.

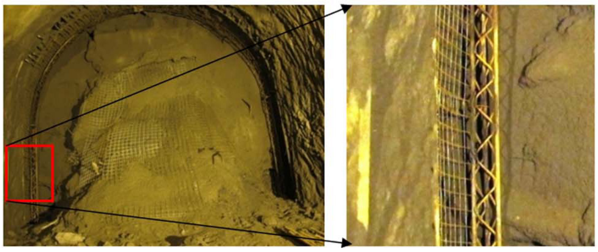


Figure 20. Photograph showing tunnel supported by lattice girders in poor rock mass strata.

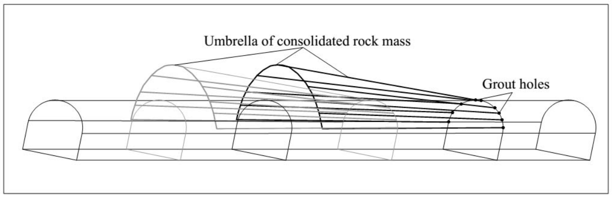


Figure 21. Schematic section showing formation of umbrella of consolidated rock mass around tunnel periphery by grouting.

Apart from improving the rock mass strength by conventional method of grouting, fore polling and pipe roofing, emphasis was given on reducing the cycle time. Initially monthly progress of 20 to 25 m was achieved in poor rock mass conditions with continuous steel ribs support. However, after evolving an optimum drilling and blasting pattern, reduction in cycle time, and introduction of progress linked incentives a progress in access of 45 m per month with continuous steel ribs support in poor rock mass conditions was achieved.

Conclusion

Underground excavation in Himalayas is a tough challenge attributed to complexity of the geology brought by the complex orogenic processes. The underground power house caverns of the Mangdechhu hydroelectric project, were housed within quartzite of high strength and stability. However, encountering of a shear zone of 1.5 to 2 m thickness with 2 to 6 m thick associated fracture zone, formation of 10.5 m high cavity and sudden increase in deformations and stresses, were the major geological and geotechnical challenges faced during the excavation of caverns. These challenges were successfully tackled by adopting advanced excavation technique, where the behavior of the rock mass in response to excavation was evaluated by three-dimensional numerical modelling analysis. Based on the analysis, the excavation methodology and support system were optimized, while also revalidating the installed support system before benching down to next level. In-situ treatment of the shear zone and rock mass as a whole was undertaken to consolidate and strengthen it to ensure better stability.

The complex geological conditions, along HRT necessitated continuous modification and optimization of excavation methodology and support system, which was achieved by implementing advanced tunnelling methodology comprising of ground improvement by grouting, crown strengthening by fore polling and pipe roofing, controlled blasting, installation of timely and adequate support system, monitoring of rock mass behavior by geotechnical instruments and probing strata ahead of face to infer the rock mass conditions for proper planning of excavation methodology, support requirement and resource planning in advance to ensure timely commissioning of the project, while also ensuring the stability of the structure.

- -

A. K. Mishra *graduated in Civil Engineering in 1977, thereafter completed M. Tech from IIT, Delhi, India in 1989. He has contributed extensively in the planning, design & engineering, contracts, execution and monitoring of hydroelectric projects in India and Bhutan. He has taken over as Managing Director, Mangdechhu Hydroelectric Project Authority (MHPA), Bhutan, on 27 August 2010. He got superannuated from NHPC Ltd on 31st December 2015 as Executive Director, thereafter, reappointed as Managing Director, MHPA for a period of another 3 years. He is also working at the risk mitigation model for hydroelectric power projects for his research work. MHPA has been conferred the prestigious award on 3rd January 2018 at New Delhi by CBIP, Government of India under the category, "Best project developer in hydro sector".*
Corresponding E-mail: md.mhpa@gmail.com

R. K. Chaudhary *graduated in Civil Engineering in 1988, there after completed Advanced Diploma in Financial Management from IGNOU, New Delhi, India. He worked as part time Lecturer in Civil Engineering Department BIT Sindri, before joining NHPC on 1st May 1989. In NHPC he was associated with investigation, planning, design, cost estimation, construction and operation and maintenance of various hydroelectric projects in India. He joined MHPA as Director (Technical) in 2015 and looks after all the civil, hydro-mechanical and geotechnical aspects of the project.*
E-mail: rajkumar0610.rkc@gmail.com

P. Punetha *post graduated in Geology (Hons.) from Centre of Advance Study in Himalayan Geology, Punjab University, Chandigarh, India in 1980. He has served NHPC Ltd. for 19 years. He left NHPC Ltd. in the year 2006 and joined Energy Infracted Pvt. Ltd. as Vice President, Geo-tech Division where he was associated with geo-technical aspects of planning, investigation, design and construction of mega hydropower projects in India, Bhutan and Uganda. He joined MHPA in 2012 as Chief (Geology) and is responsible for geo-technical construction aspects of various civil structures.*
E-mail: ppunetha@yahoo.com

I. Ahmed *M.Sc. in Geology from HNB Garhwal University Srinagar, Uttaranchal, India in 2004 thereafter obtained degree of Doctor of Philosophy in the year 2011. He joined Energy Infratech Pvt. Ltd. in the year 2010 where he was involved with the investigation, construction and preparation of DPR for various Hydroelectric projects in India, Bhutan, and Africa. He joined MHPA in 2013 as Geologist and looks after geological and geo-technical aspects of the project. He has also authored numerous papers covering various geological and geotechnical aspects of hydro-electric projects in Himalayan terrain.*
E-mail: iqrar_a@hotmail.com

References

Anon, "3DEC Version 5 Manual", Itasca Consulting Group, Inc. USA, 2013
Bhargava, O.N., "The Bhutan Himalaya: a geological account", Spec. Publ. Geol. Surv. India., Vol.39, 1995.
Bieniawski, Z.T., "Engineering rock mass classification", John Willy and Sons Inc., New York, 1989.
Detailed Project Report, "Mangdechhu Hydroelectric Project (4x180 MW) Trongsa, Bhutan. Site investigation and geology", Volume IV, 2008.
Ganser, A., "The Geology of the Himalayas", Wiley Interscience, New York, 1964.
Ganser, A., "The Geology of the Bhutan Himalayas" Birkhavser, Basel, 1983.
Guha Sarkar, T.K., "Geology of the Gaylephug-Trongsa-Byakar area, Central Bhutan", Publ. Geol. Surv. India Misc. Pub., Vol. 41(1), 1979.
Jangpangi, B.S., "Stratigraphy and tectonics of parts of Eastern Bhutan", Himalayan Geology, Vol. 4, 1974.
Mishra, A.K., Baidya, G., Punetha, P., Ahmed, I., Krishna, L., "Geotechnical instrumentation and monitoring of excavation of underground power house caverns of Mangdechhu Hydro-Electric Project", Special Publication, Journal of Engineering Geology, 2015a.
Mishra, A.K., Baidya, G., Punetha, P., Krishna, L., Ahmed, I., "Numerical modeling and re-evaluation of support system while excavating underground cavern in weak rock", Special Publication, Journal of Engineering Geology, 2015b.
Mishra, A.K., Punetha, P., Ahmed, I., "Engineering Design and Construction of Underground Power House Caverns of Mangdechhu Hydro-Electric Project (720 MW), Bhutan", Proc. World Tunnelling Conference (WTC 2017), Norway, 2017a.
Mishra, A.K., Chaudhary, R.K., Punetha, P., Ahmed, I., "Numerical Modeling a Tool to Evaluate Stability

and Support System of Large Underground Caverns: A Case Study of Underground Power House Caverns of Mangdechhu Hydro-Electric Project", Proc. INDOROCK 2017:7th Indian Rock Conference, New Delhi, 2017b.

Maurya, V.K., Yadav, P.K., Angra, V.K., "Challenges and strategies for tunnelling in the Himalaya region", Proceedings: Indian Geotechnical conference, GEOtrendz, Mumbai, 2010.

Nautiyal, S.P., Jangpangi, B.S., Singh, P., Guha Sarkar, T.K., Bhate, V.D., Raghavan, M.R., Sahai, T.N., "A preliminary note on the geology of Bhutan Himalaya", Proc. 22nd Int. Geol. Cong., New Delhi, Vol.11, 1964.

National Institute of Rock Mechanics (NIRM), "Report on 3D stress analysis of underground powerhouse complex at Mangdechhu hydroelectric project, Bhutan (Report no. NM-13-03C/05)", 2015.

Panthi, K.K., "Underground spaces for infrastructure development and engineering geological challenges in tunnelling in the Himalaya", Hydro Nepal: Journal of Water, Energy and Environment, Vol. 1(1), 2007.

Sharma, H.R., Tiwari, A.N., "Challenges of tunnelling in adverse geology", Proceedings: EUROCK and 64th Geomechanics, Colloquium, Schubert (ed.), 2015.