EXCAVATION AND CHALLENGES OF 720MW MANGDECHHU HYDROELECTRIC PROJECT UNDERGROUND POWERHOUSE

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Abstract

Powerhouse is one of the major components of hydropower project where the hydro turbine and turbo-generator are housed. The Powerhouse can be located over ground or underground depending on the suitability of terrain and geo-technical parameters. The potential energy of water stored in the dam is converted from mechanical to electrical energy in the powerhouse.

The major civil components in hydropower are expected to service for more than 60 years without major problem. Powerhouse is the largest underground opening in the hydropower project. The excavation of underground space for powerhouse is carried out in the confined space and access. The structural stability, besides geology depends largely on the size of the opening and the accuracy of excavation to the lines and levels. Achieving smooth excavated roof profile is essential for stability. This paper will describe the processes and the challenges in the excavation of underground powerhouse of 720 MW Mangdechhu Hydroelectric Project.

1. Introduction

Mangdechhu Hydroelectric Project is one of the intergovernmental projects being constructed in Trongsa Dzongkhag with installed capacity of 720 MW. The estimated cost is Nu. 36.572 Billion at March 2008 price level, including IDC & financing charges but excluding transmission line cost. The project is fully financed by the Government of India as 30% grant and 70% loan at 10% interest to be paid in thirty semiannual equal installments. The project utilizes net head of 692m and expected to generate 2923.7 MU of electricity annually with load factor of 47.3% (DPR March 2008). The project is set to be commissioned by first quarter of 2018.

The Powerhouse cavern has size of 155m x 23m x 50m (L x W x H), from the bottom of the turbine pit and is located in strong quartzite and medium to soft mica schist. It is separated from Transformer Hall by vertical rock cover of 40m. The cavern is located at average depth of 100m from the natural slope. The long axis of the cavern is aligned more or less perpendicular to foliations strike for better stability of the caverns.

The stability of underground powerhouse excavation besides geology is function of:

(i) vertical and lateral cover,
(ii) size of excavation and
(iii) orientation with respect to direction of major principal stress and major discontinuities.

The Powerhouse should be located at appropriate competent lateral and vertical cover. Lesser cover tends to exhibits structural failure due to loss of strength of rock mass from weathering action and higher cover will result in stress related rock failure. There are three principal stresses in the rock mass. The long axis of Powerhouse should be aligned parallel to the direction of major principal stress so that the longitudinal wall of the cavern is subjected to minimum horizontal stress and reduce convergence during excavation. On the other hand the long axis of the powerhouse should be aligned normal to the major geological discontinuities to minimize the possibilities of rock failure at particular shorter section which otherwise could extend throughout the length of the cavern.

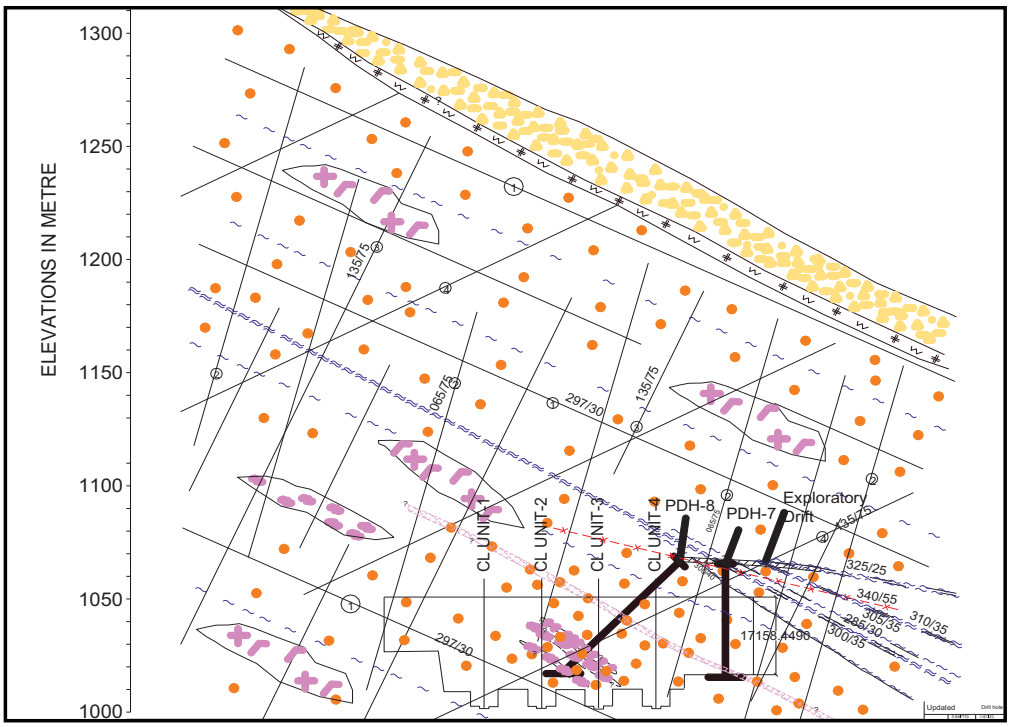

Fig-1 Geological L-section of powerhouse (DPR March 2008)

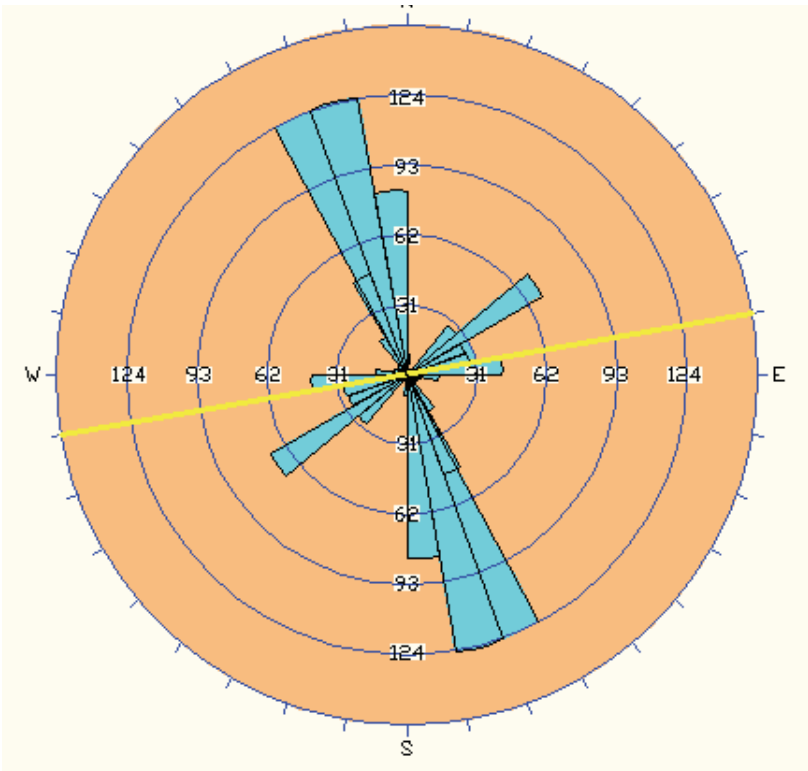

Fig-2 Alignment of Powerhouse (yellow line indicates alignment of powerhouse), L.K.K. Singh, MHPA.

The sitting of powerhouse can be fairly determined with detailed study during investigation stage. The surface mapping of major discontinuities at initial phase of investigation which later to be corroborated by core drilling and drifting would provide clear information to enable to arrive decision on locating powerhouse. The in-situ stress measurement including direction of major principal stress is essential for deciding alignment. The stress measurement at present are carried out commonly employing following two methods:

(i) Hydraulic fracturing, and

(ii) Triaxial stress measurement by drill hole overcoring.

Understanding mechanism of failure of rock mass and correct interpretation of geo-technical data are the key to technically informed decision for sitting of underground powerhouse. This is where the experience would provide vital inputs, and help to prevent expensive rock supports and construction variations at later stage. It is advisable to conduct detailed studies at investigation stage and the cost

incur would easily be offset by the saving from construction variations, and the time and cost overrun.

2. Excavation Methodology

The excavation of underground powerhouse is the most challenging part in the hydropower project for simple reason of larger size comparing to other structures. This challenge will exponentially worsen in the poor rock mass increasing the cost substantially. Proper technical approach which can take care of all factors affecting the stability needs to be adopted to excavate powerhouse with concurrent rock supports and instrumentation.

Underground Powerhouse is accessible through means of different service tunnels which are used as accesses during the construction. For the first one-third, Powerhouse excavation was led through Ventilation Tunnel (VT) leading to the crown of the Powerhouse, from EL 1052m to EL 1028m. The VT was excavated first with portal elevation of EL 1078m which met the Powerhouse at crown with invert EL of 1048m. Main Access Tunnel, Tailrace Tunnel, Pressure Shaft and other tunnels leading to powerhouse were also excavated simultaneously. When the rock is competent, all tunnel preceding excavation of powerhouse are excavated up to the lines of powerhouse. When the rock is poor the excavation of these tunnels shall be terminated 10-20m short of reaching powerhouse and continued after excavation of powerhouse has reached the intersection with the particular tunnel.

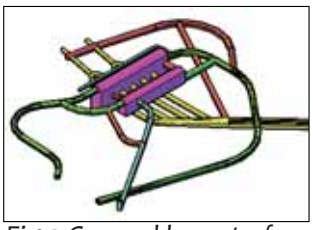

Fig-3 General layout of powerhouse complex

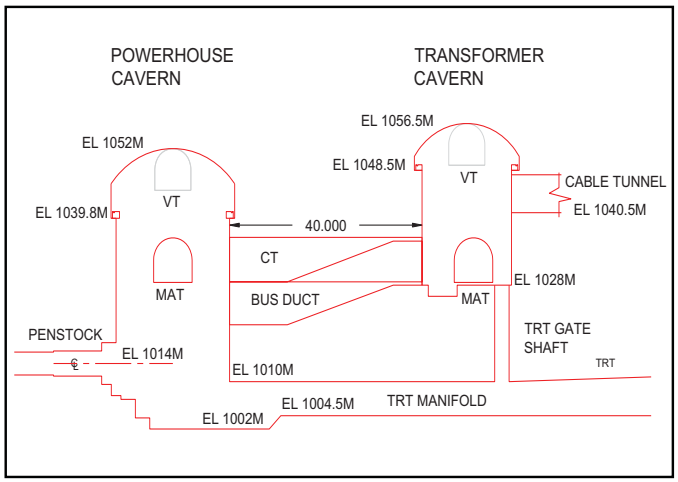

Fig-4 Powerhouse complex cross section

After VT was excavated upto powerhouse junction, excavation of central gullet from the centre of the powerhouse was carried out with the same size of VT by extending its excavation in a similar manner to full length of Power house. While excavating gullet rock support of 36mm diameter and 7m long resin grouted rock bolts were provided in the crown. Temporary supports were provided at other points wherever necessary. Following the excavation of central gullet, side slashing / widening of both sides were carried out in alternate steps of regular length of 10-20m of upstream and downstream walls. This is to enhance stability of the cavern providing self supporting mechanism on other half while excavating one half and minimize otherwise costly temporary rock supports. Complete rock supports were ensured in the crown before benching had commence, which otherwise later would become inaccessible and cumbersome.

Benching began after full length widening up to EL 1048m was completed from EL 1048m up to the level of entrance of MAT to Powerhouse at EL 1028m with ramp on one side and excavating all around. The slope of the ramp was maintained at 10% to be easily negotiable by heavy duty machinery. The removal of excavated muck until EL 1028m was done through VT.

The excavation of ramp from EL 1048m to EL1028m and further benching providing similar ramp from EL 1028m to EL 1010m was done through Main Access Tunnel (MAT). The ramp from EL 1028m to EL 1010m and excavation up to EL 1009m was carried out from the Penstocks at EL 1010m. The excavation below EL 1009m consisted of drainage gallery, turbine pits and pits for dry cooling system, septic tank and flood water pits which were carried out in individual pockets keeping the rock covers between each of the individual pits. This excavation below EL 1009m upto bottom most level of EL 1000m was done through Tailrace tunnel via Tailrace manifolds and penstock using most feasible option of the two.

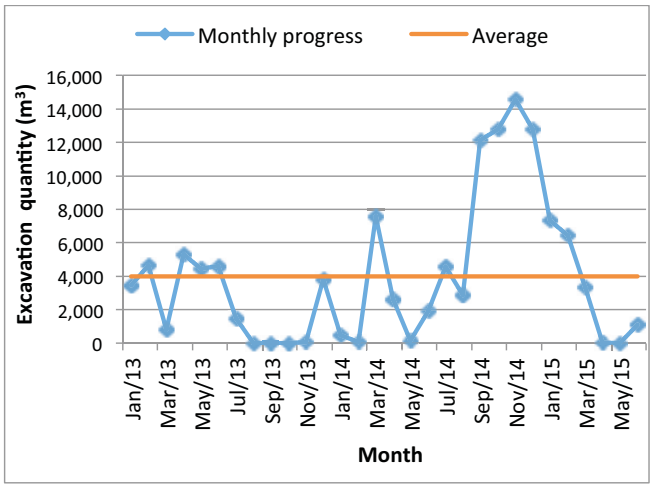

Fig-5 Powerhouse monthly excavation progress

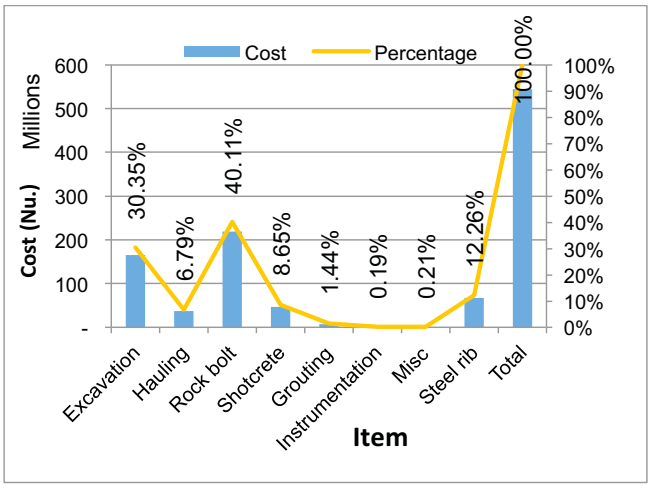

Fig-6 Distribution of powerhouse excavation cost

Maximum excavation progress of 14550m3 per month was achieved and the average progress obtained per month excluding the major incidences was 6105m3. The down progress during March 2013, June-November 2013, April-July 2014 and April-June 2015 was on account of the incidents of encountering shear zone at gullet, cavity formation, casting of crane beam and shotcrete detachment respectively, which had affected the progress due to temporary suspension of work. Once benching has commenced, minimum set of two Drill Jumbos, 1 Shotcreting Machine, 2 Excavators and sufficient fleet of Dumpers depending upon distance to mucking yard were required.

Achieving good excavated final profile was essential for any underground rock excavation to avoid stress concentration. There were frequent necessities to conduct control blasting to avoid excessive damages to surrounding rock and adjacent structures. These were achieved by optimizing controlled perimeter blasting and in special cases by line drilling. Line drilling is expensive operation and should be used only in the specific circumstances such as need to provide special configuration where the controlled perimeter blasting will damage the rock beyond required lines and level. Line drilling is a single row of uncharged holes drilled along minimum excavation line at close spacing of two to four times the drill hole diameter. The idea is to form a pre-planned weakness surface to which the primary blasting operation can break. It is important to note that line drilling is considered successful and acceptable only if 50% of the individual drill hole traces for each round are visible in the final required excavation line after removal of shattered and loose rocks. Line drilling was employed for cutting rock ledge for EOT crane beam in the powerhouse at EL 1039.8m. The EOT crane beam was partly supported on rock ledge and partly on RCC columns which was raised after completion of excavation.

3. Rock Supports

All rock supports were installed concurrent with the progress of excavation. Originally the rock support in the powerhouse consisted of (i) resin grouted 36mm Ø rock bolt of length 7.5m and 9m installed at 3m c/c, and steel fibre reinforced shotcre (M35) of thickness 200mm in the crown and (ii) resin ground rock bolt of same diameter of length 9/12m (upstream/downstream wall) and 150mm thick SFRS. Due to poor geology than generally expected, the rock supports had to be revised. Based on numerical studies, additional rock bolts of 12m long was installed and SFRS thickness increased to 250mm in the crown. In the walls, the length of rock bolts increased to 12m in the upstream and 15m in the downstream, and the SFRS thickness to 200mm. Systematic consolidation was incorporated to strengthen the rock mass. In the shear zone and poorer rock mass 18m long rock bolts was installed, and SFRS was replaced by wire-mesh / chain-link mesh reinforced shotcrete.

During later part of widening and beginning of benching, loose fall occurred from the crown between chainage 0 to 30m owing to which additional 12m long rock bolt at 3m c/c were installed in the crown. Steel rib arch roof using ISMB350 with backfill concrete was installed at the cavity area between RD 0-35m and in the highly fractured zone between RD 100-155m for long term safety of the cavern. Steel arch roof was design for roof support pressure of 10.1 ton/m2 and load from backfill concrete of 75 kN/m. Roof support pressure (Pr) was calculated as per IS 13365 (Part 2) as given below (Design memo):

$P_r = \frac{2Q_r^{-\frac{1}{3}}}{J_r} * f$, and $f = 1 + (H - 320)/800$, where;

(i) Pr is ultimate roof support pressure in kg/cm2.

(ii) Jr is Joint roughness number.

(iii) Qr is Rock mass quality for roof.

(iv) f is correction factor for overburden.

(v) H is overburden above crown.

The sequence of rock bolt and shotcrete was depended upon the nature of excavated rock mass. In competent rock, bolts were installed first followed by shotcrete while reverse sequence was done in the poor rock. It was ensured that all rock supports were installed within the standup time of excavated section. All rock bolts were pre-tensioned to 60% of the bolt capacity after grout / resin had gained sufficient strength.

Systematic consolidation grouting was done by drilling 45mm Ø hole to the depth of 6m at 3m c/c spacing. In the fractured medium, pre-grouting with high pressure upto 40 kg/cm2 was done prior to excavation. Based on the groutability test / grain size analysis of the medium, ultra fine cement with Blaine value of 1000 m2/kg having maximum particle size of 15 micro (100% passing) was used. The grout mix varied from 5:1 to 0.8:1 (water:cement) and 2 to 3% of bentonite by weight of cement was also added to improve consistency of the grout. The grout injection was initially carried with lean mix at low pressure of 2 kg/cm2. The pressure was gradually raised when intake fell below 5 litre/min and in the shear zone maximum pressure upto 30 kg/cm2 was employed. Water permeability tests were conducted prior and after grouting operation to assess the efficacy of the grout.

For relieving the pore pressure due to seepage, pressure relieve hole of 4m long of 45mm Ø at spacing of 6m c/c was provided at all the faces of Powerhouse inside which 36mm Ø slotted PVC pipes were inserted.

4. Instrumentation and Monitoring

Instrumentation is essential for monitoring the behavior of rock mass. Regular reading and interpretation of instrumentation should provide good early warning to the engineers enabling to take timely corrective measures preventing otherwise possible costly failures. Following instrumentation was provided at regular interval both along longitudinal and cross section progressive with excavation:

(i) Tape convergence point / Target point
(ii) Single Point Borehole Extensometer (SPBX)
(iii) Multi Point Borehole Extensometer (MPBX)
(iv) Load cell
(v) Piezometer

Tape convergence point will measure degree of convergence taking place at that particular point. Borehole Extensometer will measure the displacement. SPBX has, as name suggests one point and MPBX has multiple points. MPBX is more advantageous and provides the additional information of movement of rock mass occurring between particular points. In MBPX total displacement will be accumulated in longer point. The movements within all shorter points will be recorded in the longer point, while when there is movement from within longer point, shorter points will move together with rock mass and therefore the readings from longer points shall not be recorded in the shorter points. Load acting on the

rock support is measured by Load cell and pore pressure by piezometer. Regular reading of the instrumentation is extremely essential.

Reading of all instrumentation was taken once daily every morning at 8:30 AM. Convergence until February 2015 was negligible. The maximum displacement of 29mm and 27mm was observed at EL 1039.8m and EL 1029m respectively on the downstream wall at RD 65m. All the load readings were within the capacity of the support system. Maximum load observed was 228.6 kN in the crown at RD 113m followed by 148.7 kN at RD 65m EL 1045.5m. Some of the noticeable displacements and loads observed at each of the locations as on February 2015 are given in the table below.

Elevation	RD 12m	RD 65m	RD 113m	RD 150 m / 140m
EL 1052m (10m)	18.5mm	6.5mm	13mm	1.9mm
EL 1039.8m (20m)	12.4mm (D/S)	29.1mm (D/S)	7.6mm (U/S)	4.2mm (U/S)
EL 1029m (20m)	8.2mm (D/S)	27.1mm (D/S)	6.2mm (D/S)	0.12mm (D/S)

Table-1: MBPX readings (total displacement) in Powerhouse (EL 1052m is in the crown. D/S and U/S indicates downstream and upstream wall)

Elevation	RD 12m	RD 65m	RD 113m	RD 150m / 140m
EL 1052m	9.4	22.83	228.6	2.20
EL 1045.4m U/S	-1.4	18.6	9.8	-3.3
EL 1045.5m D/S	52.5	148.7	33.5	-2.3
EL 1039.8m U/S	-0.2	1.4	3.4	13.6
EL 1039.8m D/S	1.2	4.8	37.7	4.3
EL 1029m U/S	-0.5	8.6	3.9	-
EL 1029m D/S	24	12.1	72.7	-3.3

Table-2 Load cell readings in the Powerhouse (in kN)

Displacements and loads observed were consistent with the geology encountered during excavation.

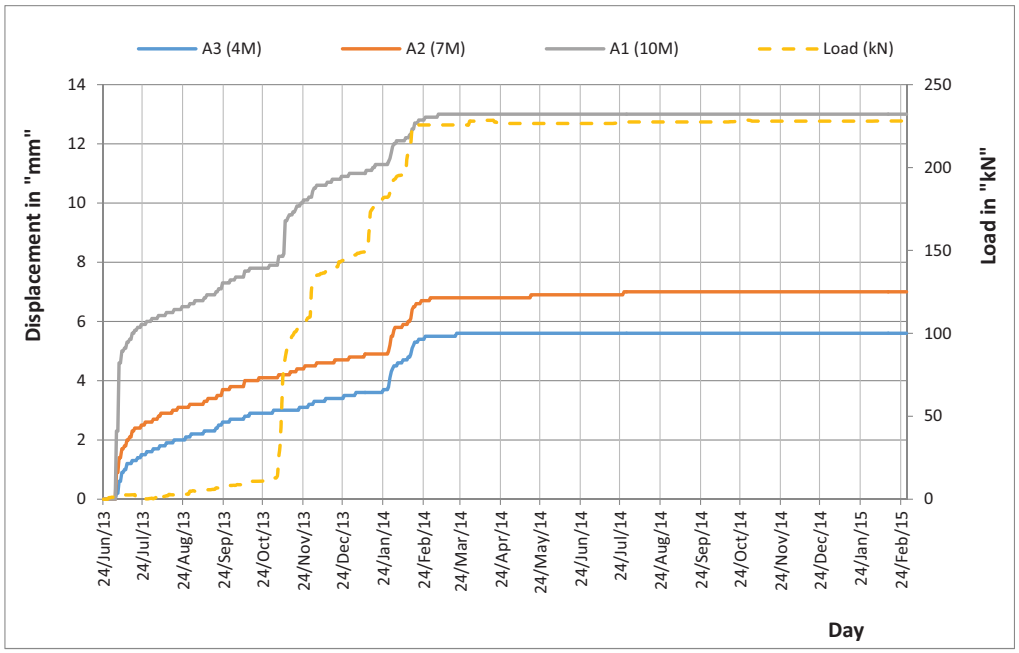

Fig-7 MPBX and Load cell reading at EL 1052m (crown) and RD 113m

5. Numerical Modeling

To corroborate the instrumentation data, and check the adequacy of the rock supports and treatment of cavity and shear zones, 3D numerical modeling using 3DEC was done incorporating the actual geological and geo-technical parameters encountered during the excavation. The study was again repeated after completion of excavation incorporating as-built support system to check the performance of rock support and stability of the powerhouse.

The result indicated that the displacement observed by instruments is marginally higher than predicted by the model. The maximum displacement of 21.9mm was observed in the crown at RD 120m. The model indicated the extent of movement in the crown and on the walls to be 3-3.5m and 2-2.5m respectively. The installation of steel ribs in the crown had helped to re-distribute the displacement reducing the maximum to 21.9mm in the crown. In absence of steel rib, the maximum displacement in the crown was observed to be 29mm and 45.5mm in the upstream wall at RD 20m. The factor of safety (FOS) in the parent rock mass was substantially higher (more than 3) which was gradually reducing towards the excavated zone.

In the fractured mass and shear zone, the FOS is found to be lower than 1 which indicates lower strength. FOS was significantly improved when the fractured mass or shear materials was removed and the space was backfilled by rich concrete. The model with steel ribs as installed gave higher FOS than the one without steel ribs. Only few bolts failures in the downstream wall of the cavern were observed.

The results showed that the rock supports provided in the powerhouse was adequate and the treatment of cavity and fractured mass was effective.

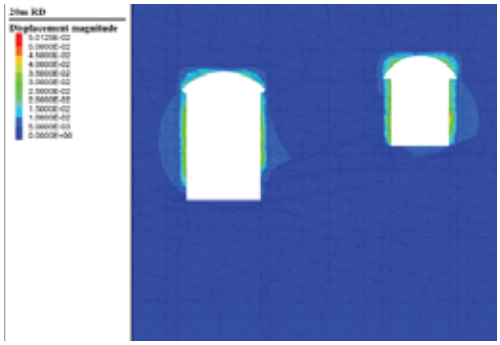

Fig-8 Displacement contour at RD 20m (Sripad, NIRM)

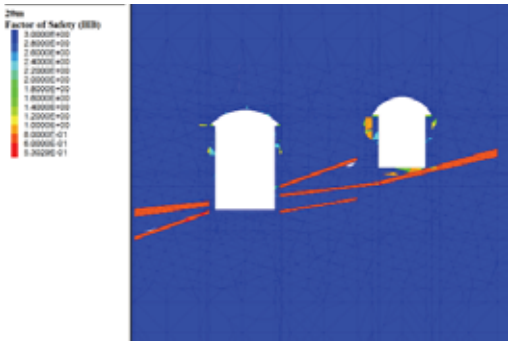

Fig-9 Factor of safety contour at RD 20m (Sripad, NIRM)

6. Challenges

Encountering of shear zones, poor geology resulting in intermittent loose fall and formation of cavities gave lots of hindrances for smooth excavation of powerhouse. These situations famously termed "Geological surprises" while to certain extent can be limited through detailed investigation at planning and design stage, cannot be avoided completely for one simple reason of complexity of geology, especially in a Himalayan geomorphology. The major challenges encountered that led to significant increase in the cost to the project are numerated below sequentially.

6.1 Shear zone and poor geology in the crown

During the excavation of central gullet shear zone with an attitude 290° – 310°/15°-20° with varying thickness with maximum of 1.5m has been encountered beyond RD 120m. This was more prominent along downstream wall and passing over crown beyond RD 135m. The rock mass have been further aggravated by the presence of highly fractured pegmatite zone with tendency to collapse in presence of water. Excavation was stopped for some time and detailed geotechnical studies were carried out for reviewing the rock support system. The rock supports were revised following the studies. This incidence delayed the excavation for about 2 months.

6.2 Loose fall and cavity formation

On 10th July 2013 during rock bolts installation, the working crew reported cracking sound between RD 0 – 15m, and at about 11 PM loose fall occurred from the crown between RD 0 – 12m, about 6 – 10m wide on either side of the centre-line and 3-6m in height. Again on 20th July 2013 at around 4PM, second loose fall took place at same position forming huge cavity of 10.34m at RD 16m extending previous cavity upto RD 35m. On closer inspection of the cavity, low dipping multiple shear seams were observed 2-3m above the original excavated line which had intersected ventilation tunnel at RD 40-50m away from powerhouse junction. The excavation in the Powerhouse cavern had to be suspended in view of need for expert view and design of corrective measures. Additional 36mm dia. 12m long rock bolts was provided in the cavity and surrounding areas as immediate strengthening measures. Leaving cavity open carries future risk of collapse. In order to ensure stable crown and prevent any future failure for safety of servicemen and equipment, the project authority approved to provide combination of steel arch roof using 350mm ISMB and backfill concrete from RD 0-35m in the cavity reach and RD 100-155m in the geologically poor rock mass. The formation of cavity and providing remedial measures delayed the projects by more than 6 months costing additional expenditure of about Nu. 200 million. The important point to note is that these additional expenditures shall be insignificant should the failure occur at later stage when the excavation had progressed deeper or during O&M phase.

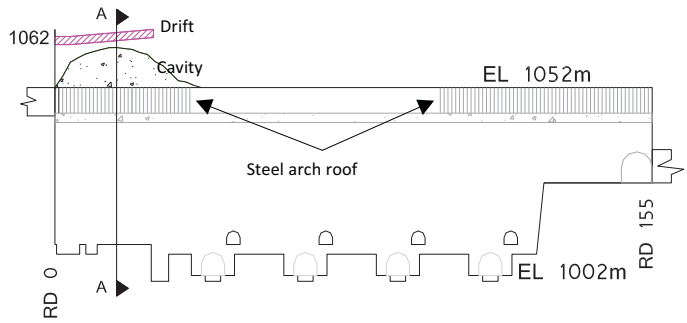

Fig-10 L-section of cavity and steel arch roof

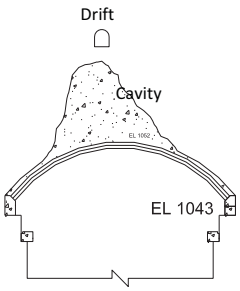

Fig-11 Section-AA

6.3 Detachment of shotcrete from the crown

Huge block of shotcrete measuring 5.2m long by 1.7m wide (maximum) fell from the crown at RD 66.7m which lies directly above Unit 3, on 18 March 2015 fatally injuring a worker who was fixing bulkheads for steel rib at Penstock 3 opening into powerhouse. The excavation in the powerhouse had reached EL 1016m in the Control Block and EL 1021m in the machine when this incident happened. Physical verification of dislodged and surrounding surfaces was carried out to check distress

in the rock mass. Comparison of pre and post event instrumentation data was also done which did not indicate any major variations in the readings. The study concluded that already loosened shotcrete block at the interface of rock had dislodge and the excavated rock mass was not distressed. The failed surface was then treated with following:

- Overhead traveler was mounted on the EOT Crane beam to reach the crown.
- All loose materials / rocks were scaled out and the surface was washed with water.
- 200mm thick layer of shotcrete was applied with 2 layers of welded wire-mesh, overlapping the peripheral old shotcrete by 1.5m.
- Finally, 3000mm long rock bolt of 25mm diameter was installed at spacing of 2m c/c.

6.4 Treatment of shear zones on the wall

Low dipping shear zone encountered at RD 120m during gullet excavation and widening was intersecting walls all its height. Many localized shear zones were also encountered both in the upstream and downstream walls. These shear zones had to be treated and strengthened for stability and safety of the structure. Treatment was carried out in the following order.

- Shear materials were scooped out to depth of 1.5-2 times the width of shear zone.
- Space was backfilled by reinforced concrete of grade stronger than surrounding rock mass up to the excavated line.
- Rock bolt of 18m long and 36mm diameter spaced at 2.5m c/c was installed through the backfill concrete, and finally
- Shotcrete of 250mm thick layer with welded wire-mesh was applied.

Conclusions

Poor geology and cavity formation led to substantial variation in rock supports, and time and cost overrun, costing an additional amount of approximately Nu. 200 million more for excavation alone. The rock failures from crown during advanced stage of benching is not only life threatening, but technically difficult to rectify and expensive. Detailed and conclusive investigation for selection of site will minimize costly construction variations. Due care should be taken especially while excavating roof to obtain smooth profile essential for achieving stronger self supporting mechanism. The shotcrete in the roof / crown for large cavern in the

Himalayan geology with jointed rock mass should be provided with weld wire-mesh instead of steel fibre reinforcement to ensure its structural integrity that could be compromised by poor fibres' uniform dispersion. The curing of shotcrete, which is often ignored needs to be ensured especially in the caverns where shotcrete serves as permanent support.

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Author's Profile

Karma Gayley graduated in Civil Engineering from PSG College of Technology, Bharathiar University, India in March 2001 and obtained a Master Degree in Hydropower Engineering from Norwegian University of Science and Technology (NTNU), Trondheim (Norway) in June 2011. He has worked in the construction of 1020 MW Tala, 124 MW Dagachhu and 720 MW Mangdechhu Hydroelectric projects before becoming the Director of Department of Hydropower Services in Construction Development Corporation Limited.