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DESIGN AND CONSTRUCTION ASPECTS OF DESILTING CHAMBER OF TEESTA-III

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ABSTRACT: The Teesta River is the western most tributary of the Brahmaputra River and is one of the main Himalayan River originating from the glaciers of State of Sikkim in the North Eastern part of India. This river carries large quantity of sediment load during snow melting season and monsoon season. The average annual sediment load including bed load has been estimated to be about 1.2M cum. The Teesta-III Hydroelectric project (1200MW) has vertical pelton turbines operating under a rated head of 780m and therefore restricting the rate of damage to under water parts by way of minimizing the erosion on account of sediment size and its concentration in the water passing through its turbine is very essential. The design and construction of two desilting chambers have been quite innovative. Pre-grouting and consolidation grouting of cavern crown was adopted as important construction practice. Also the monitoring of blast vibrationduring excavationhelped in ensuring that the integrity of surrounding rock mass is maintained. The excavation was completed without any geotechnical problems undue over-break and large cavity formation. The project construction stands completed in August,16 and commissioning of generating units are in progress. This paper summarizes the salient features of design and constructional aspects of desilting chamber of this Himalayan project as a case study for similar projects.

I. INTRODUCTION

The Teesta-III hydroelectric project is run of the river scheme located in the Himalayan state of Sikkim. The project comprise 60m high concrete face Rockfill dam, two nos desilting chamber, a 13.8Kmlong headrace tunnel 7.5m diameter, a 13m diameter and 154m deep surge shaft, along with two nos 4.0 dia steel lined pressure shaft feeding six units of vertical pelton turbine 200MW each housed in an underground power house. The 1Km long tail race tunnel takes back the discharge from the turbines to the river. The project envisages utilizingabout 816m of gross head to generate 5228MU of design energy annually in a 90% dependable year. Himalayan rivers are known to carry large quantity of silt especially during monsoon and therefore one no. Flushing cum spillway tunnel of 11m diameter with tunnel invert just 5m above river bed level has been provided to flush out sediment during Monsoon period and also to facilitate reservoir flushing.The silt management at Teesta-III project has been by way of effectively reducing the silt content in the water flowing to the power house by having two nos. desilting chambers in the water conductor system.

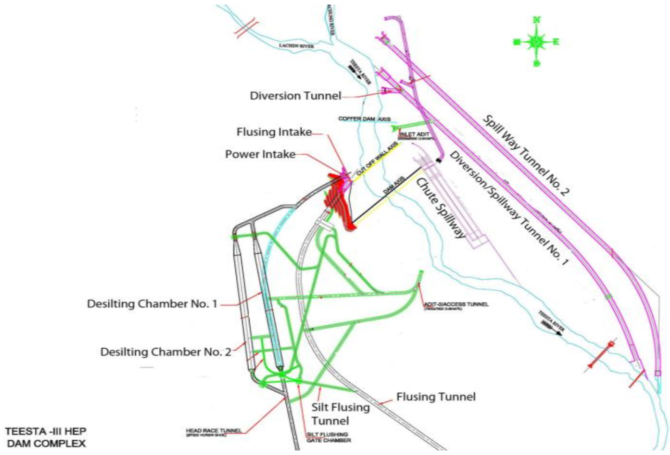

Fig-1

II. DESILTING ARRANGEMENT

An underground desilting arrangement comprising two parallel chambers each of size 320m long 17m wide and 23m deep fed by 7.5m diameter power intake tunnel 320m long is provided on the right bank of the Teesta river. The desilting chambers have been designed for removal of 90% of suspended silt particles above 0.2mm from the water discharging into the head race tunnel. The caverns are oval shaped with circular roof, curved walls and spaced 40m Centre toCentre leaving a rock pillar/wall of about 30m thick between the two chambers. A discharge of 202 cumecs enters in chambers with flow through velocity of 25cm/sec i.e. 13.5 cumecs is flushing discharge for each chamber for flushing the settled sediment from the chamber. The sediment in each chamber shall settle into the central cunnette 2m wide with depth varying from 0.5m to 2m wherefrom the flushing duct having width varying from 1m to 2m and depth varying 1m to 1.5m and having a slope of 1:108feeding into the common silt flushing tunnel which finally terminates into the Flushing tunnel for further discharging into Teesta river. The flushing discharge is controlled on the downstream of chambers by installation of bonnet type silt flushing gate on both ducts. Each conduit has been provided with opening of varying sizes to draw silt from the settling trench. On downstream of the silt flushing gate, the raw water passes into free flow Flushing tunnel through a suitably designed transition and silt flushing tunnel. Due to high velocity at downstream of the gates, weak hydraulic jump is envisaged and therefore the reaches where supercritical flow occurs are provided with abrasion resistant lining. The silt flushing tunnel 4.8mX4.5m D-shape and 100m long meets the 1360m long flushing tunnel at 300m from outlet portal. The silt flushing branches have been provided with high performance concrete(HPC)

M50A20 on the invert and side walls from silt flushing gates upto beginning of silt flushing tunnel to prevent abrasion action of silted water. Each of the desilting chambers has been provided with vertical lift gate on the upstream and downstream to facilitate isolation for cleaning and maintenance. The desilting chambers are connected to HRT through manifold type tunnel construction.

III. HYDRAULIC MODEL STUDIES

The dimensioning of the desilting chambers is decided by the principle of fall velocity being dependent mainly on the size and specific gravity of settling particle. The chamber was tested on model to analyse the efficiency of silt flushing system .The desilting chamber was tested on the model at 1:15 scale at Hydraulic Research Institute Bahadrabad, Roorkee, to test the hydraulic efficiency of the desilting chamber and adequacy of flushing system. Flow area i.e. width and the depth of chamber has been fixed creating a flow through velocity of 25cm/sec for removal of sediment coarser than 0.20mm. Based on model studies it was observed that 320m long and 23m deep desilting chamber was adequate for settlement of silt particles coarser than 0.02mm diameter with an overall silt trapping efficiency of more than 93%.for 2000ppm silt concentration discharge.

IV. GEOLOGICAL SET UP

The rock of this area belongs to Chungthang series comprising biotite schist, gneiss and quartzite and Calc Silicate and Darjeeling series of the Kanchenjunga group comprising central crystalline Gneissic Complex. The hard quartzite and calc gneiss with biotite-schist, amphibolites are exposed on the right bank. The Desilting chambers have been mainly excavated in class-II rock with patches of class-III rock exhibiting presence of predominantly quartz biotite gneiss. In both the desilting chamber area S1 (foliation) and S2 (160°-190°/30°-50°) are the dominant sets with high persistence whereas S4 (200°/45°) and S3 (°33/50°) exhibits very limited presence. The desilting chambers have been oriented across the prevalent foliation strike for better stability. The desilting chambers have a lateral cover of 300m to 400m and vertical cover of 300m to 350m with rock pillar averaging about 50m between the two chambers.

V. DESIGN CRITERIA FOR ROCK SUPPORT SYSTEM AND LINING OF DESILTING CHAMBER

The underground support design should be to strengthen the rockmass to support itself to the possible extent. Accordingly the shapes and layout and the support characteristics should be given due consideration. The conventional shape selected for a large cavern is usually a arch roof supported with vertical walls which are easier to excavate. But such tall walls have a tendency to deflect inwards inducing tensile failure and hence require higher rock stabilization measure. Therefore oval shaped chamber was finalized on geotechnical consideration for better cavern stability and reduced support requirement. The desilting chambers are large size pressure tunnels and could be drained out in sequence for maintenance during operation. Therefore the design criteria adopted for the chamber was when one chamber is depleted and other is running full and charging the surrounding rock mass upto full reservoir level (FRL). Keeping this into consideration lining of desilting chamber should cater to following requirement.

- Ensuring long term stability under varying internal pressure.
- Withstand external pressure of ground water during empty condition and also the external loads of the overlying rock.
- Prevent erosion of rock and its joint fillings.
- Limit water loss from the chamber.

The lining and rockmass around the emptier chamber is subjected to external water pressure because of saturation of rock upto full reservoir level due to seepage from other chamber. Each rockmass will have its own unique set of structural features and can introduce a directional pattern of weakness in the rockmass.The study was carried out using”UNWEDGE” to identify the unstable rock wedge formed by various combinations of four joint sets. Generally it was observed that few unstable rock wedges are formed but with pattern rock bolting and synthetis fibreshotcrete(SnFRS), sufficient factor of safety was achieved.

In the initial design the 30cm thick RCC lining was proposed for smooth flow and for taking care of internal and external pressures during operation stage. During construction it was opined that the concrete lining though hydraulically efficient involve construction difficulties. The construction sequences could have been the excavation from top to bottom upto top of hopper and then erecting a gantry for side walls and crown which would have a height of 18m and width of 17m. Erection of such large gantry, concreting and it’s sliding operation being difficult and time consuming ,was not found a desirable proposition. Also excavation of hopper and bottom flushing duct would have required extreme precaution as blasting might damage the concrete lining of walls etc. The concrete arch provide support for any rock that may loosen in the cavern roof but concrete arch being very rigid compared with surrounding rockmass, deformation induced as a result of excavation of the lower part of the cavern can cause excessive bending in the concrete arch and even cracking may occur. Repair of damage roof arch during construction is an extremely difficult and costly proposition. Experience has also shown that the use of a more flexible support system by way of longer bolts and additional layer of shotcrete provides a more satisfactory solution. Accordingly on balance of consideration the concrete lining was replaced with SnFRS excluding in hopper and flushing ducts where 500 thick reinforced concrete lining was done.

VI. CONSTRUCTION

Each chamber involved underground excavation of more than 100,000m3 each. The excavation of DC-I was started in Sept-08 and completed in May-12 i.e. in a period of 45 months and similarly excavation of DC-II was started in Jan-09 and completed in June12 i.e. in a period 42 months. 6.1 Excavation Sequence:The desilting chamber is excavated in the right bank hill in strong to very strong, moderately

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jointed to massive quartizites with gneissic laminations. Rock encountered has rockmass rating varying from 50 to 70 and thus grading the rock as fair (class-III) to good (class-II). Lateral cover of the order of 30m has been kept between two chambers to minimize stress of rockpillars. The pillar width is more than 1.5times the height of two caverns and hence there shall be very limited zone of overstress. This means that the stress fields surrounding the two caverns are almost independent of one another and hence a modest amount of support is required to stabilize the rockmass surrounding the caverns. Also the excavation in the two chambers was staggered and adequate support in the form of synthetic fibreshotcrete(SnFRS) and rock bolting was done to check wedge instability as a consequence of stress building. The structure was also monitored for stability during excavation by way of blast vibration measurement and monitoring the behavior of cavern after installation of support by way of load cells and multi point borehole extensometers.

The actual construction sequence has been

- Excavation of central gullet of size 8mX8.4m followed by 100mm thick SnFRS and 6m long rock bolts 1.5m c/c.
- Side slashing of crown gullet and application of rock support and initial SnFRS.
- Benching Excavation of side walls in six benches with concurrent application of support.
- The final SnFRS lining of 100mm thick was also applied from top downwards with every bench leaving some vertical lag with excavation.
- Excavation of hopper portion and flushing ducts .
- Concreting of flushing duct and hopper.

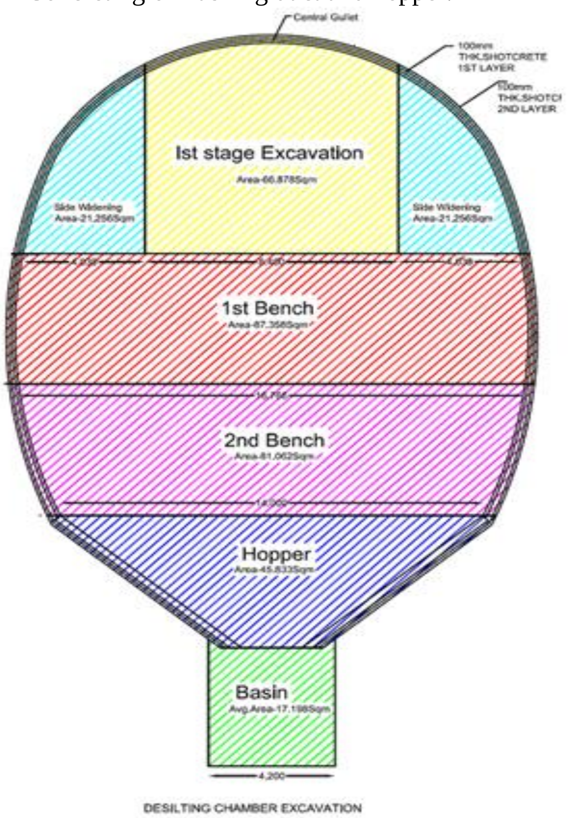


Fig-2

The systematic rockmass excavation with controlled blasting and concurrent support system during benching down in desilting chamber was adopted. Bench depths were fixed in the range of 2.5m to 3.5m depending upon the rock mass conditions. The drilling and blasting activity was taking more time with the use of electrical detonators as charging had to be done only after drilling activity had ended as all the operations involving electricity/welding had to be stopped during charging for fear of catching static charge. Since desilting chamber is quite long and excavation and rock support activity like shotcreting and bolting can be done together, it was decided to use Emulsion explosive (power gel 801) with Non Electric detonators (NONEL system) in place of conventional electric detonators and NGF based explosive compound. During benching excavation, wall support system was installed within 48hrs after excavation. The rockbolts were provided as 5° downward to facilitate grouting according to geological orientation. Instrumentation by way of installing load cells and multipoint bore hole extensometers were done to assess the effect of excavation on the stability of the cavern and also the efficacy of the support system.

6.2 Blast Vibration Monitoring: Blast control is not a part of cavern design but has the single most important influence on the outcome of the excavation process.The excavation sequence started with central gullet 7m wide by first making box cut 10m long and 3m deep with very controlled blasting. The central gullet followed by upstream and downstream benches was excavated across the free face by benching method. About 60 holes of 3m depth were drilled using crawler mounted pneumatic drills. The average explosive charge per production hole was kept between 1.5 to 4Kg and charge per delay used was in the range of 2-3.2 Kg per delay depending on the type and quality of rock. The non- electric detonators were used. The ground vibrations were measured and monitored regularly to assess and minimize the damage to the rockmass in the desilting chamber. High frequency vertical geophones (28Hz to 2 kHz) were used for monitoring vibration and monitoring station was established in the construction Adit. Ground vibrations having sufficient energy could damage the rockmass though the extent of damage is not only a function of vibration level but also dependent on rockmass characteristics and ground support system etc. The maximum observed peak particle velocity was 6.60mm/sec when using 150Kg of explosive (power gel) with 42 production holes using 12 series of non-electric detonator at a rate of 2.73Kg/delay. The vibration monitored during the excavation confirmed that the vibration levels were well within the threshold limit of 15mm/sec. The ground vibrations were monitored using InstantelsMinimate plus 4channel system capable of monitoring trigger level between 0.25mm/sec -127mm/sec and microphone trigger level from 106db-142db. The fig-3 shows the vibration monitoring of a sample blast.

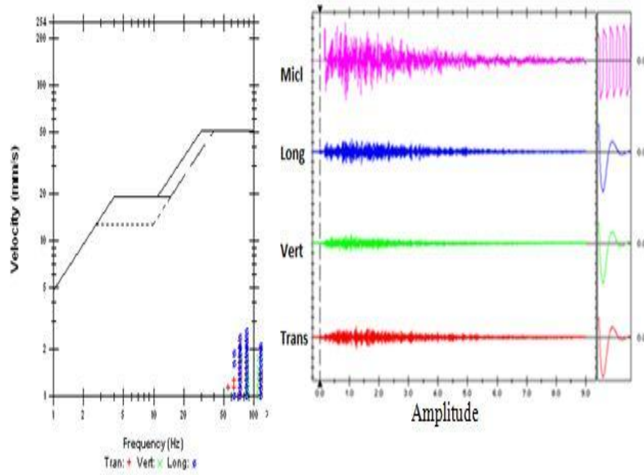


Fig-3

The stability of large cavern is dependent on the integrity of the surrounding rockmass and hence the controlled blast and monitoring vibration intensities proved very effective, and thus facilitating a trouble free and cavity free excavation of these caverns.

6.3 Rock support, grouting and drainage arrangement:

After detailed analysis through numerical modeling based on field and laboratory tests following rock support measures were executed.

- Rock bolts 6m long, 25mm dia have been installed at a spacing of 1.5m c/c and staggered. Fully grouted rock-bolts with resign end-anchorage with a pretension of 12T was provided 100mm thick SFRS was applied over the rock surface as initial support.
- Pre-grouting with Micro-fine cement was done before advancing the face by way of 5-10m holes 45 dia upto a pressure of 25-40Kg/cm². A total of 69 MT of Micro-fine cement was consumed in DC-I and 76MT in DC-II in the pre-grouting process.
- Consolidation grouting was also carried out through 5m long holes 45Φ at 3m c/c where excessive seepage was observed. The consolidation grouting was done with OPC cement and about 3200 bags were consumed in DC-I and 3500 bags in DC-II. However 30MT of Micro-fine cement was also used for grouting in DC-I in the area where seepage/wet condition was observed even after doing consolidation grouting with OPC.
- Final SFRS lining was done in stages from top downwards maintaining a suitable lag with the excavated bench and thus to ensure minimum damage due to blasting.
- Drainage holes 75mm diameter and 0.3m into the rock were provided in the crown, walls and the hopper zone.

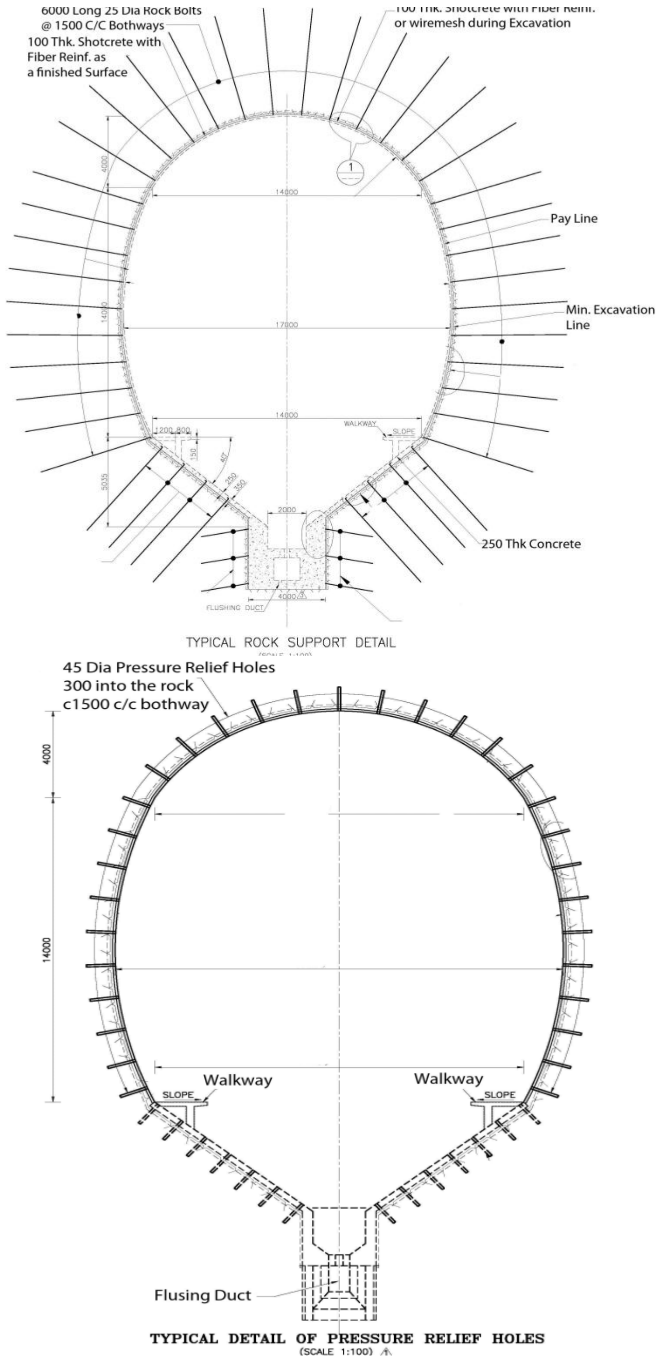


Fig-4

Each desilting chamber involved shotcreting and rock bolting to the tune of 3900m3 and 52000rm respectively. The concrete lining upto hopper top was about 5200m3 for each chamber. The polyfibre shotcrete was used instead of plain shotcrete to enhance quality and effectiveness of shotcrete. The micro-silica was used in the shotcrete mix for low permeability, better pumpability and increased strength characteristic. The structural macro polypropylene fibres of dia 8mm and about 50mm long (HICOPOLA) at a rate of 4Kg/m3 together with 1Kg/m3 of micro fibres (FIBRECAST) were used in shotcrete. The field performance of shotcrete was tasted by extracting cores from cast panels and results were monitored. Fig-5 shows the compressive strength of cores over a couple of month.

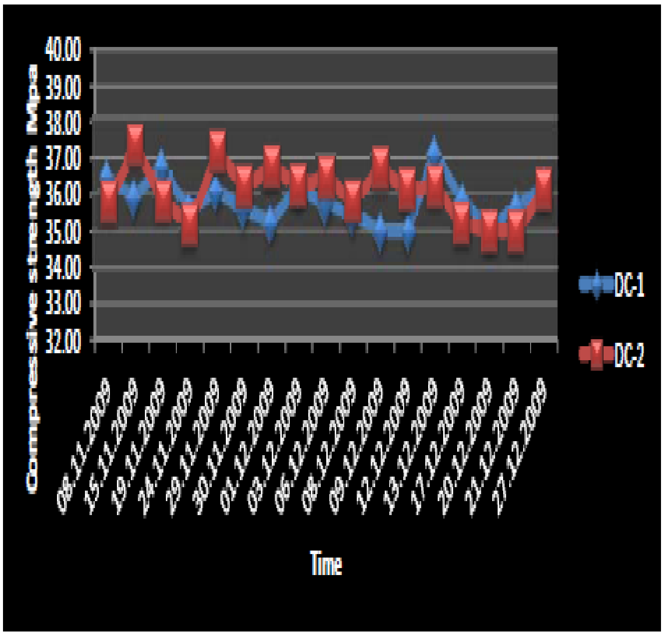
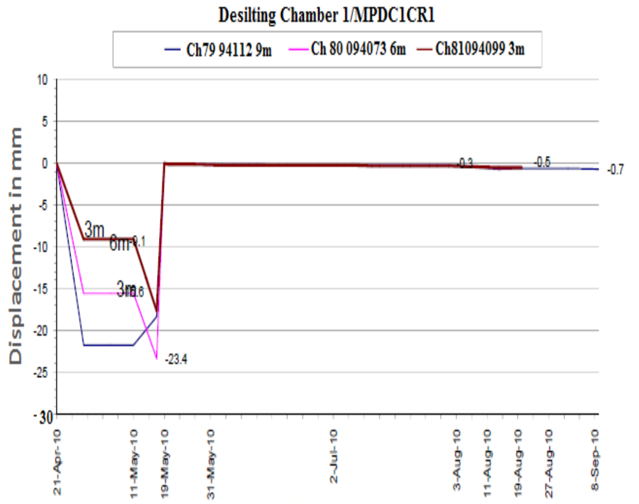


Fig.5 Field avg compressive strength of cores of fibre shotcrete in desilting chamber

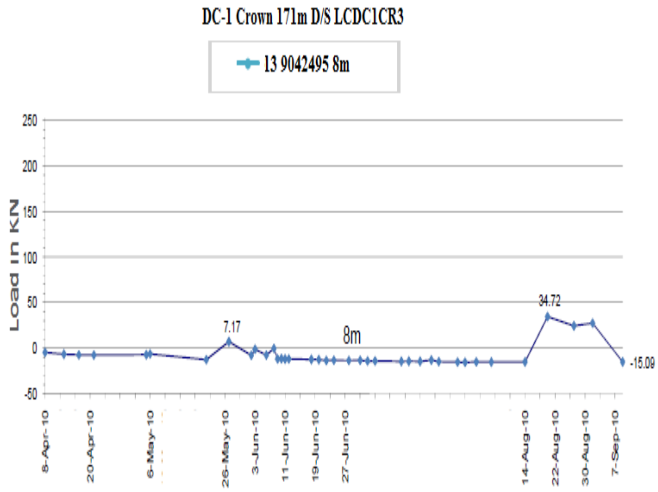
After the concreting of flushing duct at the bottom for about 40-50m, the hopper lining was started with 6m gantry. While doing lining under cuts were removed from hopper as well; as flushing duct portion through controlled blasting. In DC-II while the lining gantry was at RD:180m, about 70m from construction cum inspection Adit, the undercut removal was being under taken when peeling of shotcrete was observed from 3-4 locations in an areas varying from 10m2 to 40m2. The maximum depth of loosening was around 20-30cm. It was understood that after completion of the major part of excavation, there was a long hiatus in excavation activity. However due to vibration during enlargement of excavation for accommodating hopper and flushing trench minor stress arrangement may have taken place leading to such spalling of isolated patches of shotcrete. The area was reshotcreted by raising scaffolding after hopper lining was completed. All constructional activities including hopper lining and drainage holes etc. stand completed for both desilting chambers and water charging of DC-1 was done in september16.

VII. INSTRUMENTATION

Three arrays of instruments comprising 3-point bore hole extensometer and load cell were installed in both the desilting chambers. These instruments were installed in the crown and side walls to assess the magnitude of rock mass deformation during the excavation of the caverns. The anchor load cells were also installed in few bolts to estimate the magnitude of pretension and increase of load on the bolts. All the instruments were regularly monitored and the data collected were analyzed. Most of the instruments showed stable readings. The fig.6 shows the MPBX readings for DC1.



The maximum displacement in the MPBX was observed in DC1 to the tune of 9.1mm in the 3m zone and in 6m and 9m zone the displacement varied between 16mm to 23.4mm in the crown at RD 109.m. However the reading got stabilized in a week's time. The maximum load observed in the load cell to the order of 34.72KN at 171m d/s in the crown location which got stabilized in next ten days to 15KN as could be seen in fig. 7



In DC2, the maximum displacement observed was of the order of 8.8mm to 11mm in the 3mtr. zone in MPBX in crown at RD 66mtr. The load cell showed maximum load in the range of 12KN in the crown location. The instruments showed variation during early days only. The consolidation grouting and additional bolts were installed where instrument showed increasing trend and thus the readings became stable and benching down was a smooth affair.

VIII. CONCLUSION

The excavation and support measures adopted for desilting chamber has been quite innovative by way of introducing Pregrouting with OPC and Micro fine cement followed by extensive consolidation grouting of the cavern crown. Further monitoring the excavation process through blast-induced ground vibrations and use of NONEL system for blasting operation proved very effective and the excavation of such large twin caverns could be achieved successfully without undue over-break and cavities. Such blast monitoring studies for optimizing blast design etc should form integral part of excavation process of large caverns like desilting chambers where damage to rockmass in the vicinity cannot be permitted for its long-term stability.

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Graduated in Civil Engineering from Bhagalpur University Bihar in 1984 and joined National Hydro Electric Power Corporation (NHPC) a Govt. of India Enterprise in Feb 1985. The author has worked in NHPC Limited, the largest Hydropower Utility in India for more than 23 years. He worked as Chief Engineer for the construction and commissioning of Teesta Stage V (510 MW) HE Project (NHPC) and also as Project Director for the construction of Teesta Stage VI (500MW) HE Project(Lanco Infratech Limited) and Teesta III HE project (1200MW) in Sikkim. He has also worked as Owner’s Engineer for Govt. of Uganda for Karuma hydropower project (600MW) in North Uganda. He is a life member of Tunneling Association of India(TAI), INHA and ISRM and is a certified Chartered Engineer(India). He has more than dozens of publications in national & international journals and conferences.

