Geological and Geotechnical aspects and challenges during construction of the Tehri Hydropower Project

H L ARORA Former Director Technical THDC India Ltd.

Abstract

Tehri Dam is located in highly complex geo-tectonic environs of Himalayan region prone to seismic activity. Safe design and construction of Tehri Dam Project, posed immense technical challenges. In such complex environment, a 260.5 m high Earth and Rock-fill dam with slightly inclined clay core has been designed by utilizing locally available fill materials, with little or no processing. Design of the dam has been finalized by involving internationally renowned expert Institutes in the field viz. HPI, Moscow and DEQ, IIT, Roorkee. Extensive surface as well as sub-surface treatment of the core seat foundation i.e. grouting has been done for improving its properties. For monitoring the health of the dam during construction and operation period, a large number of instruments have been installed within the body of dam and its foundation. One of the unique features of this dam is the provision of inspection galleries within the core of the dam through which physical inspection and monitoring of the dam behaviour is possible during the operation period of the project. The paper describes in brief, about Geological and Geotechnical aspects and challenges during construction of the Tehri Hydropower Project.

1.0 Introduction

Multipurpose Storage Hydropower Projects play an important role in storing the surplus water in their reservoir during monsoon and releasing the same after monsoon as per requirement of irrigation and drinking water of downstream habitation. Tehri dam has been constructed on river Bhagirathi, a tributary of river Ganga, to serve the purpose of storing surplus water of river Bhagirathi during monsoon and releasing the same after monsoon (during lean season from Nov to June) for irrigation of 8.74 Lac Ha land of UP and to provide drinking water to about 45 Lacs population of Delhi and 30 Lacs of UP. Water stored in Tehri reservoir when released in river Bhagirathi and after travelling down about 42 kms, it reaches Devprayag where river Bhagirathi meets river Alaknanda and river Ganga is formed which after travelling down through Rishikesh reaches Haridwar. A barrage has been constructed across river Ganga just upstream of Haridwar at Bhimgoda. Water available in river Ganga is regulated at Bhimgoda barrage and distributed among Upper Ganga Canal (UGC), Eastern Ganga Canal (EGC) and natural river course (Ganga) by UP irrigation Department (UPID). Further, UP Irrigation department has developed a network of Canal System in river Ganga basin for distribution of water released into above three canals.

2.0 Tehri Power Complex

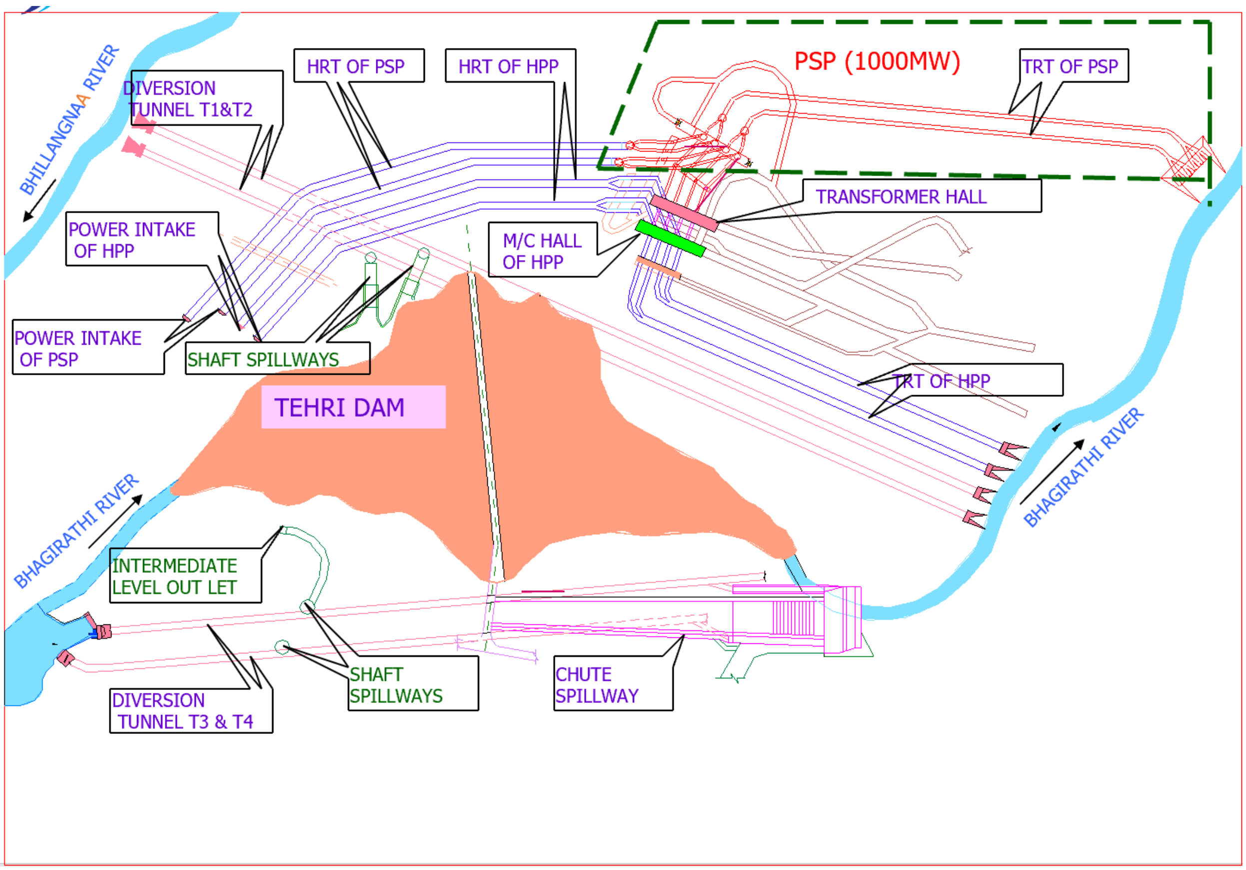
Tehri hydro power complex is a multipurpose scheme consisting of Tehri Hydropower Plant (4X250 MW), Tehri Pump Storage Plant (4X250 MW) and Koteshwar Hydroelectric Project (4X100 MW) designed for storing surplus water of floods during monsoon and there by moderate the floods. The water stored is utilized for irrigation and drinking throughout the year while generating 2400MW of peaking power.

2.1 Tehri Hydropower Plant (HPP) - 1000 MW

Tehri Dam Project conceived in 1949 as a major storage scheme on River Bhagirathi, was investigated by State of UP and Project was cleared for implementation by Planning Commission in 1972. The Project comprises the construction of a 260.5 m high earth & rockfill dam on river Bhagirathi at 1.5 km downstream of its confluence with river Bhilangana is located in a narrow S-shaped valley with steep side slopes. Left abutment of valley providing toe support to the dam.

Spillway system comprising of Chute Spillway on right bank, two no. Right Bank Shaft Spillways and two no. Left Bank Shaft Spillways to bypass the surplus water during monsoon and high floods has been designed to cater the probable maximum flood (PMF) discharge of 15540 Cumecs corresponding to a flood frequency of 1 in 10,000 years. The routed Design Discharge at PMF is 13043 Cumecs.

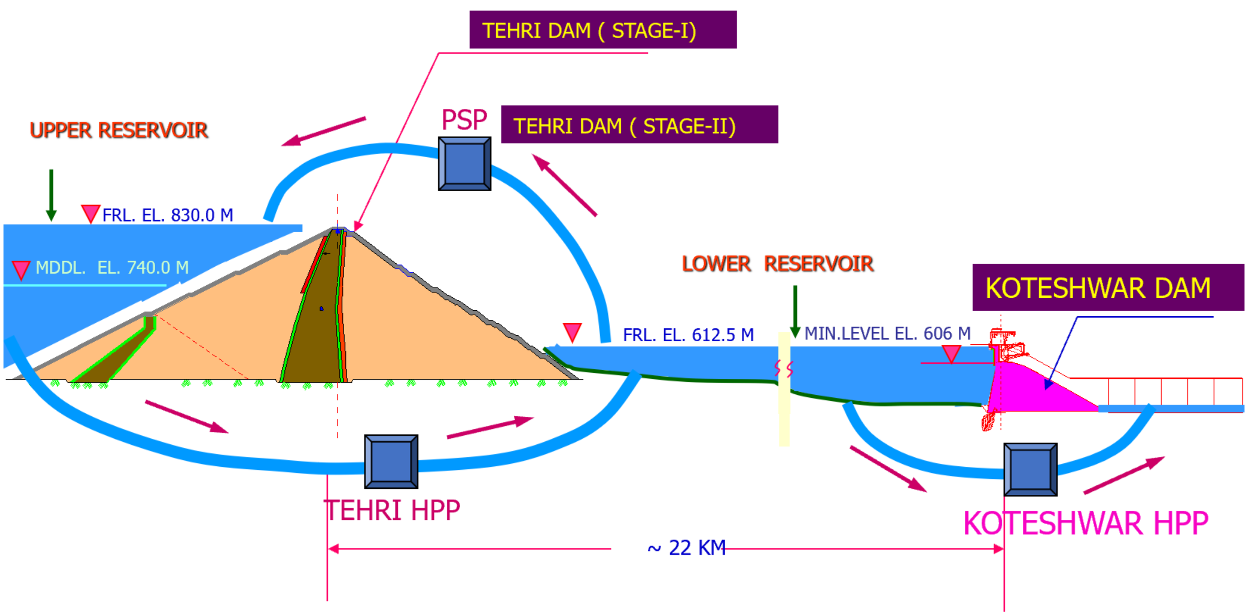
In addition to above an Intermediate Level Outlet (ILO) with discharge capacity of about 1125 cumecs at Full Reservoir Level (FRL) has been constructed on the right bank at an EI. 700m to control the initial filling of the reservoir and meet the irrigation requirement when all the power house machines are closed. In addition, in event of any eventuality, reservoir can be lowered through ILO.



Four head race tunnels of 8.5 m dia. each with intake and an underground power house on the left bank to accommodate four conventional power generating units of 250 MW each (Total installed capacity 1000 MW).

2.2 Koteshwar Hydroelectric Project (HEP) - 400 MW

Koteshwar HEP envisages construction of a 97.5m high concrete gravity dam across river Bhagirathi at Koteshwar (about 22 km downstream of Tehri Dam) and a surface power house with installed capacity of 4 x 100 MW capacity on the right-bank. The dam is provided with 4 spillway bays of size 18 x 16m with radial gates to pass the peak design flood. The overflow spillway and energy dissipation arrangement has been located centrally in the river channel. The reservoir created by Koteshwar dam shall act as a lower reservoir of Tehri PSP as well as reservoir for Koteshwar Hydel Scheme. Water released from Tehri Reservoir is regulated from Koteshwar HEP for maintaining continuous flow of water in the river for irrigation purpose and safety of population and pilgrims in the downstream area such as Deoprayag, Rishikesh and Haridwar. For this one machine of koteshwar HEP is running continuously for 24 hrs. This is also facilitating the functioning of Tehri Power Complex as a major peaking station in Northern grid.



L-SECTION OF TEHRI HYDRO POWER COMPLEX
(UPPER AND LOWER RESERVOIRS)

2.3 TEHRI PUMP STORAGE PLANT (PSP)- 1000MW

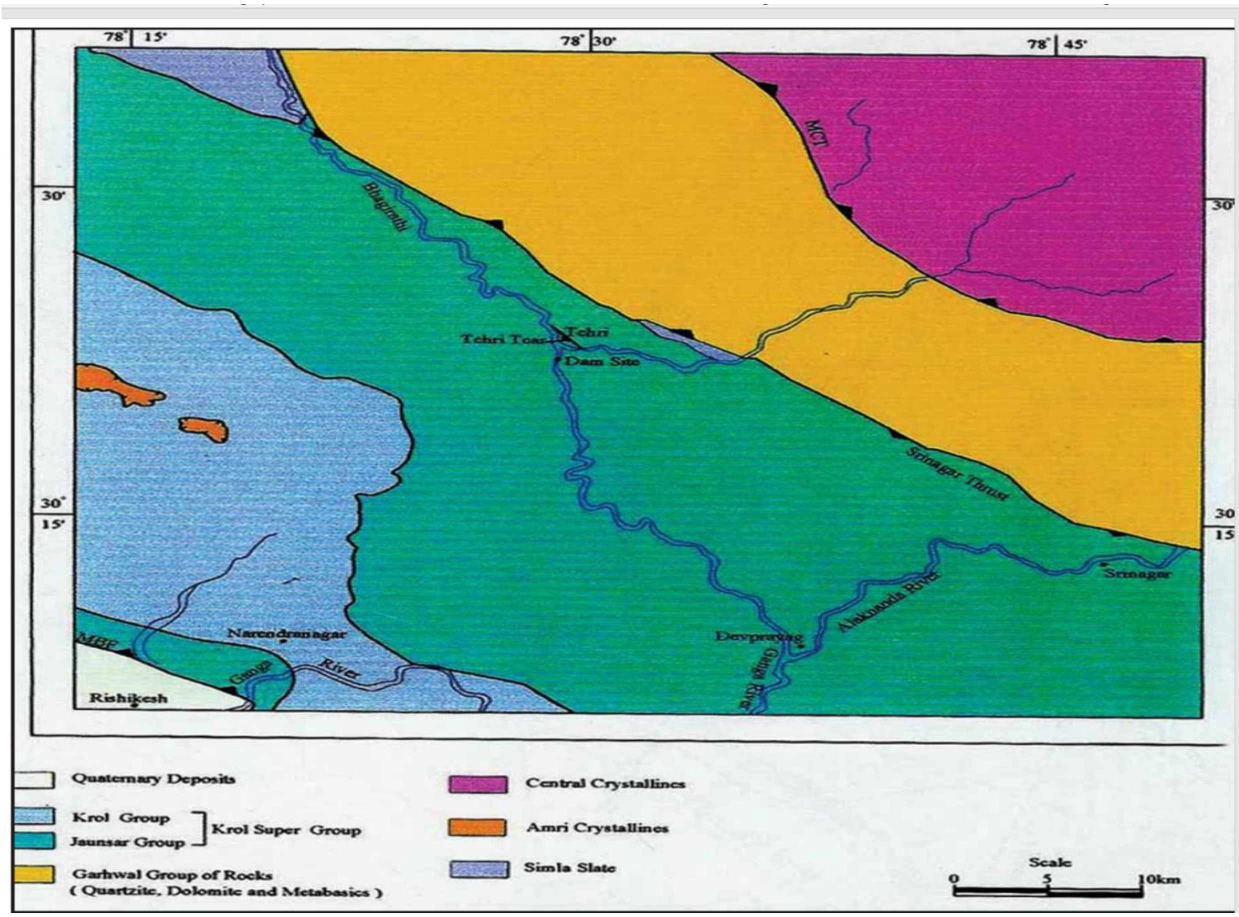
Tehri PSP has been envisaged to generate 1000 MW of peaking power for enhancing system reliability and also to provide balancing load to the thermal and renewable generation during off peak hours. The reservoir created by the Tehri Dam and Koteshwar Dam would function as upstream and downstream reservoir for Pump Storage Plant. This Plant will contribute in stabilisation a big way by meet the peak load and balancing requirement.

3.0 Geological and Geotechnical aspects of the Tehri Dam and Hydropower Plant

3.1 DAM

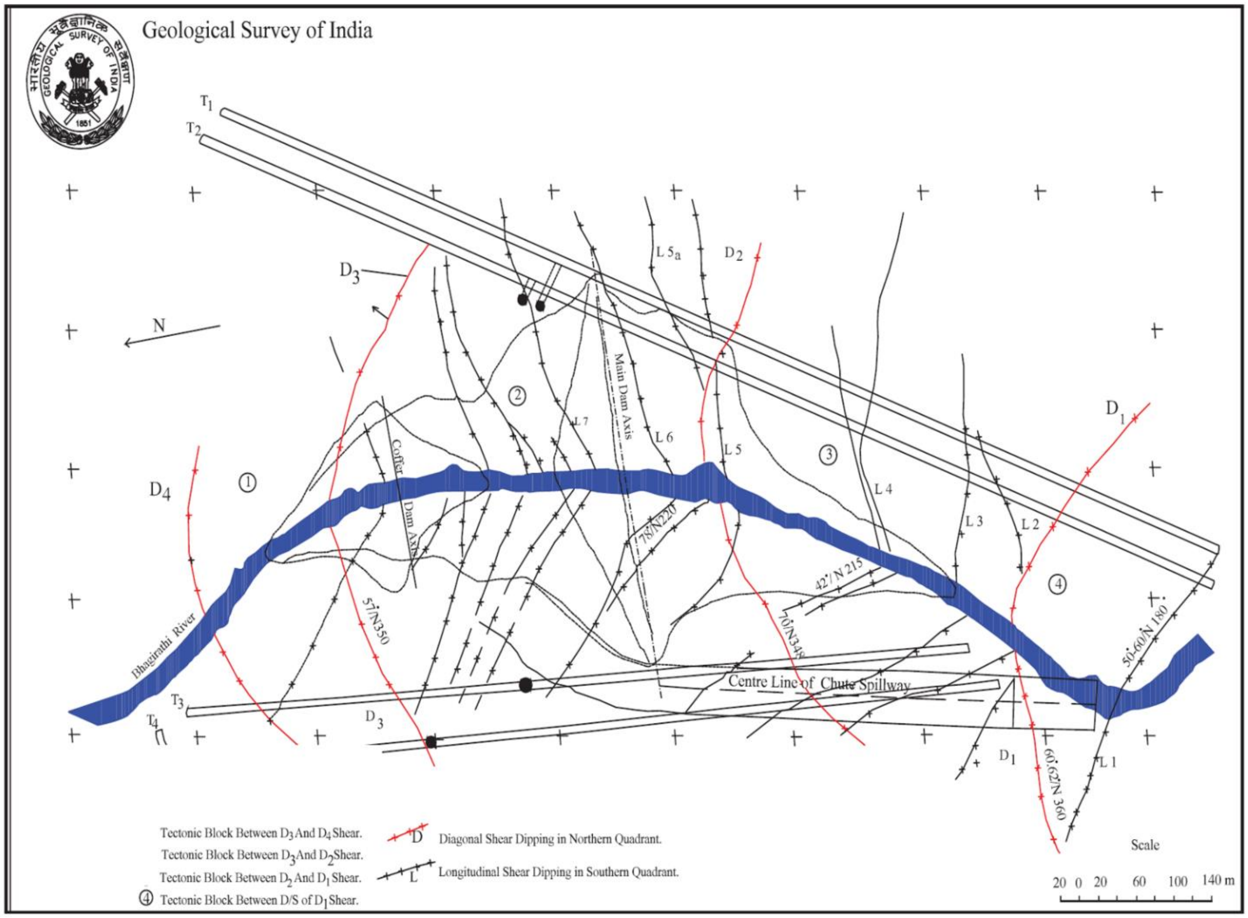
3.1.1 Geology

The Tehri dam project area is seismically active and falls in zone IV of seismic zoning map of India. The Tehri region falls between two main regional tectonic features of Himalayas–Main Boundary Fault (MBF) on the south-western side and Main Central Thrust (MCT) on the north-eastern site. Besides these, there are some other tectonic features in the vicinity of dam site, the important among them being Srinagar Thrust, which has a strike continuation of over 100 Km and lies at a distance of 6 Km towards north from the dam site.



Geological map of area around Tehri Dam

The rock formations at the dam site are phyllites of Chandpur series Tehri phyllites have been classified as phyllitic quartzites, thinly bedded (PQT), phyllitic quartzites massive (PQM), quartzitic phyllites (QP) and sheared phyllites (SP). PQM and PQT are the most competent rocks and have been geomechanically grouped into grade-I phyllites, while QP and SP have been considered grade-II and grade-III in decreasing order of strength and deformability respectively. These phyllites have been found to be traversed by numerous major and minor shear zones and joints. The major shears in the area have been classified as D (diagonal) and L (longitudinal) shears on the basis of their geometric relationship with the bedding and foliation, The geometry, orientation, frequency and interplay of D & L shears have considerably affected the geomechanical behaviour of rock mass and provided a scope of dividing the area into different tectonic blocks. Tehri dam is seated on single such tectonic block.



Tectonic Block Model of Tehri Dam Project

3.1.2 Geotechnical Assessment:

Based on data inputs generated during detailed engineering geological investigations and constriction stage, the following major geotechnical aspects were identified at the dam site.

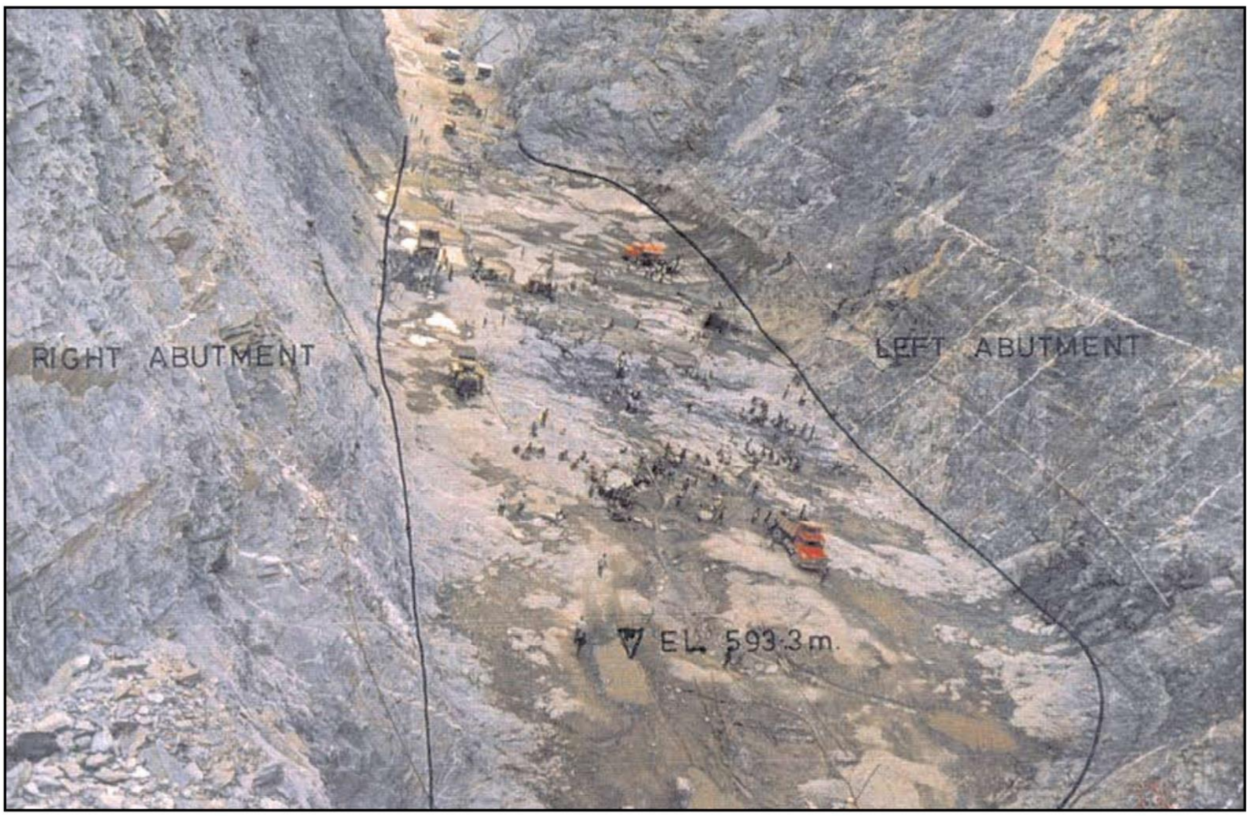
3.1.2.1 River Bed Shear Zone:

During the initial phases of investigation of foundation of dam by drill holes, a 3m to 20m thick major shear zone in the river bed slightly downstream of the core occupying a considerable portion of the gorge was apprehended. The possibility of a tectonic dislocation in the riverbed area was expressed.

In 1987, for exploration under the river bed, a 63.9m long exploratory drift was driven 30m below the river bed (El. 574.0 m) from the right bank to the left bank at a distance of about 50m d/s of dam axis. ***Detailed examination of the geological features in the drift revealed the absence of any major dislocation namely fault or shear zone along the river***. Further hydro-geological conditions in the drift, also indicated the tightness of planes of discontinuities in the rock.

3.1.2.2 Rock Mass Condition on the Abutments & Foundation:

After stripping to the designed levels, foundation of the core and shell areas was geologically mapped and geotechnically assessed. All important shear zones were given dental treatment and the areas occupied by sheared/deformed rock mass were lowered and back filled with suitable strength concrete. Clay core and transition zone (filter zones) was placed on fresh and sound rock mass devoid of slumping, open joints and fissures. This was ensured by stripping the seat of the structure which involves removal of overburden and weathered rock to improve shear properties of the foundation rock. For ascertaining the limit of stripping on the abutments and in the river bed section in the case of Tehri dam seat, explorations were done by geophysical method, drilling and drifting. Based on the sub-surface explorations, acceptable stripping limits were recommended. ***Excavations for laying the seat of the dam core and filter were limited to the fresh and unweathered sound rock (El 597.0m to 593.3m). The deepest foundation was achieved at El 593.3m contrary to the earlier proposed limit at El 579m, thus avoiding enormous excavation.***



Excavation of the main Dam core trench – JAN'91

3.1.2.3 Sub-Surface Treatment of Dam Foundation

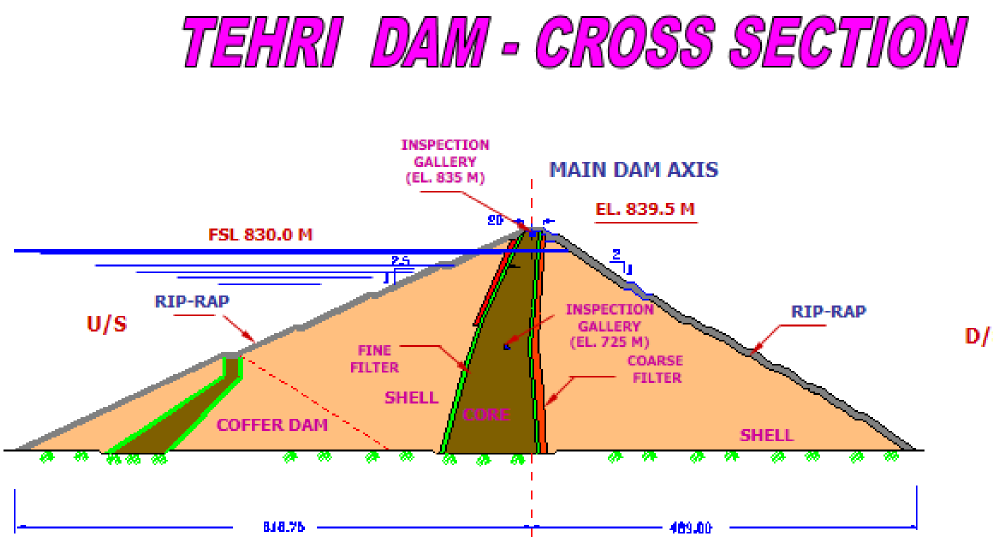
The scheme of grouting adopted at Tehri Dam is as follows:

i) 10 m deep consolidation grouting in the entire core-trench area in a grid pattern of 3mx3m at low pressures of 2 to 4 kg/cm2

(ii) Two row (spaced 1.5m apart) grout curtain with u/s row 30m and d/s row 60m deep has been provided along the centerline of the core. which has been extended further below the control structure of chute spillway on the right bank. The curtain grouting was carried out at high pressures restricted to 10Kg-12Kg /cm2, in order to avoid provoked grouting due to hydro fracturing at much higher pressure. These pressure ranges were specified after carrying out rock fracturing tests in the dam core area.

3.1.3 Design features of Tehri Dam- Fill Placement

The Tehri dam is a seismically designed structure with a sub-vertical thin core. The filters on either side of the blended clay core are the fine filter layer (on u/s side) consisting of fine to medium sand and coarse filter layer (on d/s side) comprising well graded mixture of sand and gravels (upto 80mm size). The upstream and downstream shells comprise a well graded sand-gravel-boulder mixture (size upto 600mm) with varying silt percentage. The designed upstream (2.5:1, H:V) and downstream (2:1, H:V) slopes covered with rip-rap layer (10m thick) of blasted rocks (quartzite) were found to be seismically safe.



SHELL	:	20.2 M CUM
CLAY	:	3.5 M CUM
FILTERS	:	1.5 M CUM
RIP RAP	:	2.8 M CUM
TOTAL QTY OF FILL PLACEMENT	:	28.0 MCUM

A number of nearby borrow areas were intensively investigated for getting the suitable clay material for the construction of Dam and investigations were carried out for assessing the suitability of clay material. It was seen that the deposit was reasonably homogenous in Koti Borrow area which was suitable for dam construction. For the shell material and aggregate, nearby source was identified which comprised sand, gravel and boulders up to 600 mm size. For the 1st layer of transition zone i.e. fine filter, the material obtained from Dobata borrow area was washed out for removing fractions lower than 0.1 mm in size and screened for removing the fractions over 20 mm in size. 2nd layer of transition zone, i.e. Coarse filter material was prepared by producing different fractions of filter by crushing and screening material available at Dobata shell borrow area. The material of different sizes were mixed in prefixed ratio for obtaining the material of specified gradation. For the protection of the dam slopes and increasing stability of slopes during seismic activity, riprap material consisting of well graded hard blasted rock upto1200 mm size has been provided on both the slopes of the dam. Quartzites and dolomites of 'Garhwal Group' located near Asena village along Bhilangana river at 17 km from Tehri town, were quite hard, compact and massive in nature. The rocks were mainly medium to coarse grained, dirty white colour massive quartzite, with thick interbands (5 to 10 m) of bluish gray massive dolomite and suited to the specifications required for blasted rock material.

3.2 Spillways:

Tehri dam spillways are designed for the probable maximum flood of 15540 cumecs estimated at Tehri dam site. Spillage of flood discharge from maximum reservoir level would involve a drop of about 220 m requiring well designed arrangements for energy dissipation. After prolonged analytical and hydraulic model studies, the scheme having a chute spillway, two ungated shaft spillways on right bank and two gated shaft spillways on left bank have been provided on techno economic considerations and greater flexibility & reliability in operation of Tehri dam spillways

The design of chute and shaft spillways, carried out in association with Soviet experts, suitably caters to the need of very high velocities that would generate at the heel of spillways. The scheme of imparting swirling motion to the flow for energy dissipation in shaft spillways is a unique feature adopted on Tehri dam spillways.

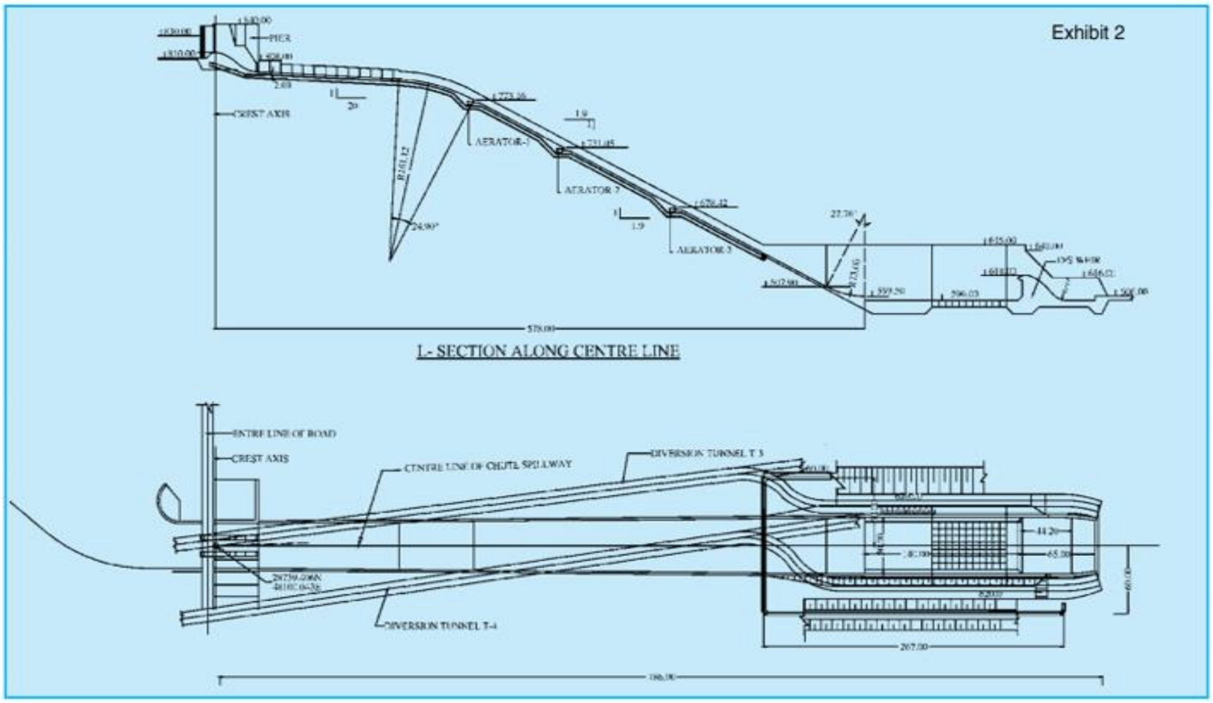
3.2.1 Geotechnical Aspects of Spillway structures

Spillway structures are located on different tectonic blocks of the block tectonic model of Tehri dam project. Construction of spillway structures, at different sites, posed certain geotechnical problems, which were tackled by detailed geotechnical assessment and by providing suitable foundation treatments and well designed support system as detailed below:.

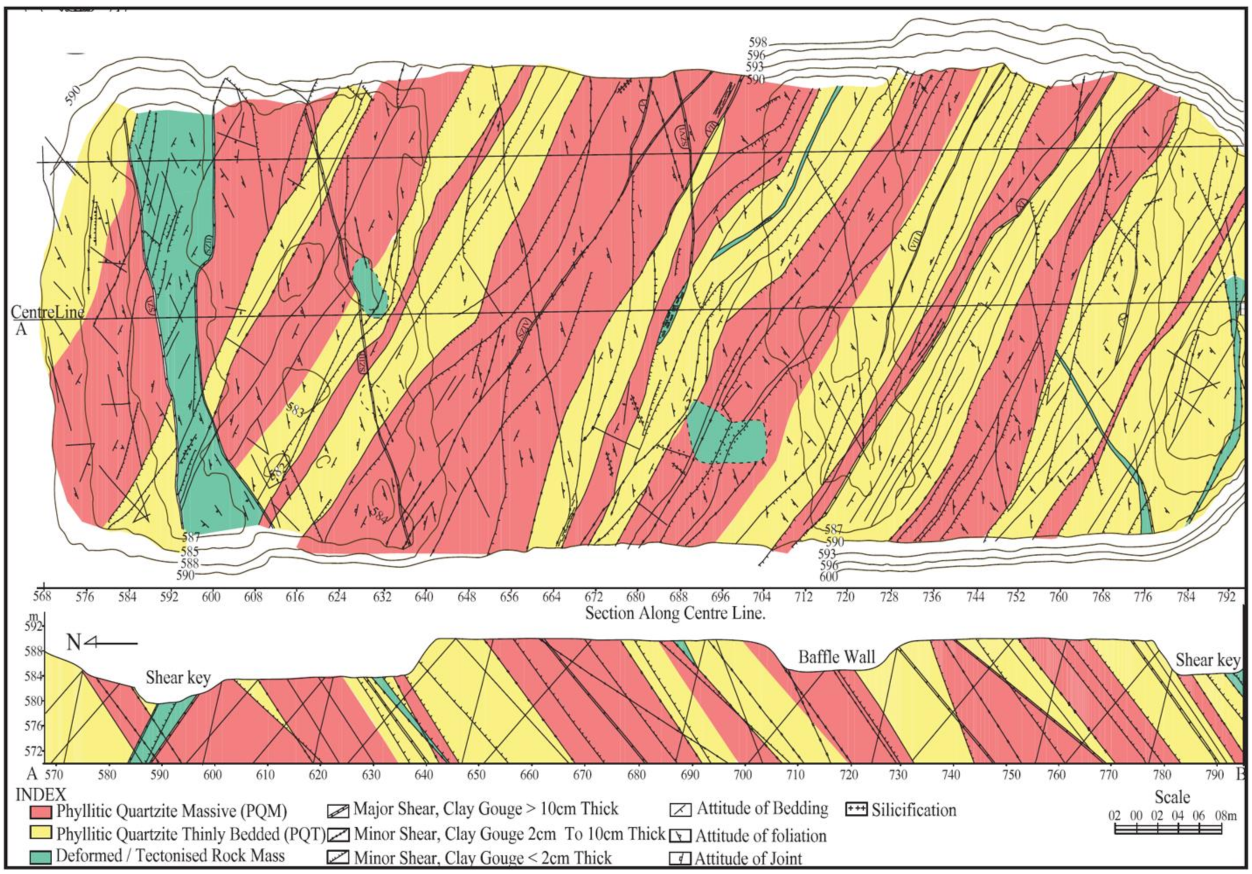
3.2.1.1 Chute Spillway

Along the chute spillway, different bands of phyllitic quartzite massive (PQM) and phyllitic quartzite thinly bedded (PQT) were encountered. Planar failure along the foliation joints and longitudinal (L) shears, due to day-lighting, were noticed and to arrest these planar failures, deeper rock bolts were provided. The junction of the chute spillway with the stilling basin was considered highly vulnerable because of the existence of sheared/deformed rockmass formed by the intersection of the major diagonal D1 shear (clay 12-14cm) and longitudinal L1 shear (clay 10-12cm).

Stilling basin, with a dead water pool of 140m x 50m x 22m, has been designed as a mode of energy dissipation for the high energy arising due to the high head of 225m through hydraulic jump formation. The stilling basin, extending from ch d/s 578m to ch d/s 795m and 25m either side of the centerline of the chute, is 217m long and 50m wide (including the downstream weir and cut off trench). Based on different design requirements the stilling basin had been divided into three reaches i.e Reach A of 60m length, Reach B of 66m lenth and Reach C of 91m length.



Details of the chute spillway

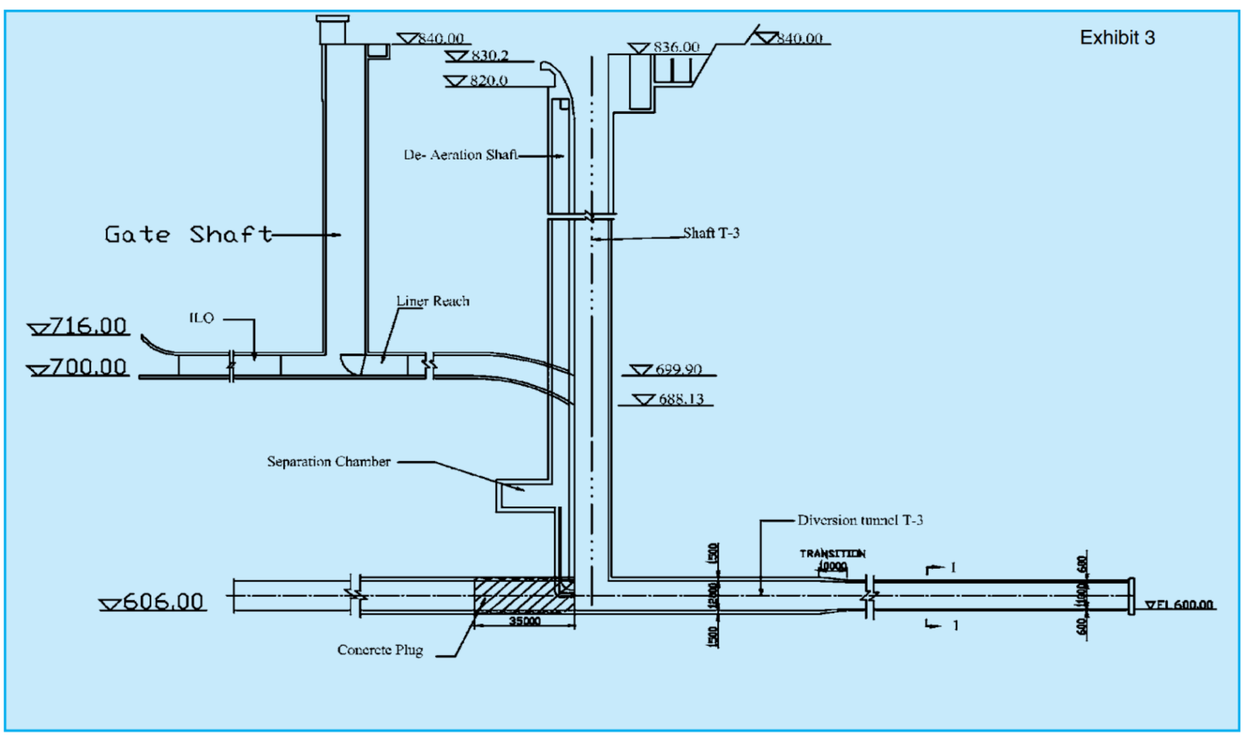


Geological map of stilling Basin Foundation

Due to existence of major shear in the u/s part of Stilling Basin near the junction, it was decided to provide u/s portion of Stilling Basin as thick slab monolithic with the walls involving placement of huge quantities of concrete of about1.0 lacs cubic metres. Accordingly, it was worked out to provide 12 m thick monolithic concrete floor in first 60 m length (Reach-A) to withstand the huge hydro dynamic forces occurring when the chute spillway operates at maximum discharge. Therefore, a shear key 7-15m wide, extending across the stilling basin and 4m below the general foundation level, was provided at the junction. During the excavation the deformed rock mass was removed and the entire trench was backfilled with highly reinforced cement concrete. Reach-B of stilling basin was also considered critical because of the hydraulic jump and the uplift pressure due to static and dynamic pressures. To counter these pressures, deep cable anchors of 110-120 tonne capacity @ 3m c/c of varying depth of 15.5m to 22m were provided in addition to the consolidation grouting.

3.2.1.2 Right Bank Shaft Spillways

The right bank shaft spillway area was assessed critical because of the poor rock mass around D-3 shear (near T-4 shaft) and multiple shear seams as projected along T-3 shaft. During the excavation of T-3 & T-4 shafts and their ducts, planar failures along longitudinal shears, wedge failure and problem of wall convergence was noticed. The junction of the main shafts with the de-aeration ducts was critical because of the reverse curve geometry. Rock bolting and chain link shotcreting were provided as the main stabilisation measures whereas spot bolting and steel rib support were provided in critical reaches. At the swirling device area of T-3 & T-4 shafts, planar failure along foliation joints were recorded due to day lighting of these joints and the area was stabilized by providing 8m long 32mm dia rock bolts along with two layers of shotcrete.



Right Bank Shaft Spillways and Intermediate level outlet

3.2.1.2 Intermediate Level Outlet

The intermediate level outlet (ILO) was driven through different bands of PQM & PQT. The excavation of the inclined glacis of ILO at the junction with T-3 shaft was very critical, and it was done by taking up smaller segments. Deeper rock bolts were provided on both the walls, and vertical steel channels were extended down to the increased wall sections, to prevent failure from the arch or wall sections, during the excavation at the junction.

3.2.1.3 Left Bank Shaft Spillways

No major geotechnical problem was encountered in the excavation of T-1 & T-2 shafts, and they were stabilized by providing rock bolts and shotcreting.

3.3 Power House Complex

Considering the topography of the narrow valley in the close vicinity of the dam site, an underground power house was envisaged. The water to the power house is being drawn from the reservoir into the head race tunnels through intakes at invert El 720 m. Two head race tunnels of 8.5 m diameter have been provided to serve the four machines each of 250 MW of conventional vertical shaft Francis turbines. There is a separate transformer hall cavity. The water from HPP is being conveyed back to the river through two tail race tunnels of 9 m diameter each.

3.3.1 Geotechnical Aspects of Powerhouse structures

3.3.1.1 Intake Structures & Maintenance Gate Shafts

The Intake area of Tehri HPP and PSP comprises the hill slopes from El 705 m to El 910 m. Important structures such as Intakes and platform at El 840 m developed for placing hoisting arrangements of maintenance gates and electrical control rooms. The slopes of the left bank in the area of water intakes towards gate shafts at EL 710.00-840.00 m are formed of phyllite and clay phyllite layers 5-30 cm wide with several beddings of quartzite phyllite. The rocks are schistic and crumpled into highly compressed folds dissected by longitudinal and diagonal faults and fractures. Near the daylight surface, the rocks are additionally weakened by the processes of de-stressing and weathering. The rock mass condition in the area U/S of Maintenance Gate Shaft MGS-2 and D/S of MGS-4 (more precisely on MGS 3 location), was found to be very poor , influenced by numerous major/minor D & L shears at different levels.

The slopes were stabilized with construction of deep cast-in-place concrete piles extending to the roof of relatively intact rock and integrated by massive pile caps. A system of concrete beam was provided between MGS-2 and MGS-3 at EL 835.0-840.0 m. These remedial measures stabilized the upper portion of the slide mass, which had positive effect on the overall stability of the slope.

Due to existence of D-3 shear and other longitudinal shears, MGS-4 collapsed below El.771m +/- and the muck from shear zone flowed into the shaft. For stabilising the collapsed portion, ehe shaft was filled with muck up to El. 746.5m for creating a platform for construction of drifts at the periphery of shaft. The shaft was supported by construction of multi-drift 2 X 2m size ensuring that the drifts are excavated at least 2.0 m deep into the sound rock. The drifts were filled with reinforced concreting. A total of 9 nos. drifts were constructed at EL-746.50 m. The construction of these drifts (functioned as shear keys) had virtually stopped any movement in the shaft due to presence of D-3 shear and stabilized the shaft. The shaft was backfilled with reinforced concrete to a depth of 1.5 m, thus protecting the entire shaft.

3.3.1.2 Butterfly Valve Chamber (BVC) and Penstock Assembly Chamber (PAC)

In this area, the axes of the caverns are sub-parallel to the strike of bedding which in this rock mass are steeply dipping at 52°. Zone of sheared & crushed quartzitic phyllite were exposed in the upstream wall of PAC (RD63 M. to 100 M) and extending to the d/s wall & roof of BVC.

Poor to very poor rock mass and unfavourable geological structure posed significant problems in the BVC & PAC excavations. Significant stability problems encountered in their roofs and the right walls. The BVC & PAC were of unfavourable orientation to the main geological structures and located sub parallel to the longitudinal fractures led to development of the technogeneous distressing zones. Inadequate Rock Pillar width and poor geotechnical properties of rock mass in and around BVC and PAC was serious concern for long term stability of these Chambers.

Both the chambers have been excavated by heading and benching method. The roof and walls were initially supported by rock bolts and shotcreting with welded wire mesh. During excavation of the roof of both PAC & BVC, heavy rock falls were experienced due to wedge formations and were subsequently supported by steel ribs of ISMB 300. In May 2001, in the downstream wall of the BVC at RD 100-122, between El 708 m to El 716 m the rock collapse of about 200 cum took place as a result of which the size of pillar between chambers was further reduced by 2-2.5 m.

Based on geophysical studies, visual observation and numerical analysis it was recommended to install pre-stressed cable anchors in rock pillars between PAC and BVC. The rock pillar between the chambers has been strengthened with 80 through pre-stressed cable anchors of design working load of 80 T. In addition, the opposite wall in the BVC was also strengthened with 94 blind pre-stressed anchors of design load of 80 T.

The instrumentation monitoring and the visual observations revealed that even the support system with prestressed cable anchors was not found to be adequate for the stability of these chambers due to poor rock mass condition, unfavourable orientation of these chambers with respect to disposition of the rock defects/joints, inadequate rock pillar e.g. Bifurcation chamber, penstock tunnel and adits etc between these chambers and excavation of multiple openings in around this area. In view of the above, the construction of four concrete buttresses in the BVC (from wall to wall) and four concrete buttresses (from wall to wall) in the PAC was taken up. After concreting the buttresses in PAC and BVC, the wall convergence reduced noticeably and stabilised thereafter.

3.3.1.3 Underground Power House

The Underground Power House consists of the three main cavities in stage I viz., Machine Hall, Transformer Hall, and Expansion Chambers of the complex are located in the available most competent rocks (PQM) and phyllitic quartzite thinly bedded (PQT) Grade–I. These cavities run parallel to each other and are aligned normal to the strike of rocks. In addition, there are four bus duct tunnels joining machine hall & transformer hall at right angle.

These caverns were stabilized by means of rock bolting and shotcreting barring a patch in the crown of machine hall where a 5m band of deformed rock mass, associated with a major longitudinal shear. As the strike of the deformed rock mass and the bus-duct alignment are sub parallel, serious stability problems were recorded in the crown portion of these openings (bus ducts) for a considerable length during excavations. Longer rock bolts (+ 15m length) followed by steel rib supports were installed in these critical zones.

Further, due to insufficient rock cover of 41m (less than 2D) between the Machine Hall and Transformer Hall, convergence was observed during the excavation of the bus ducts in the common wall. Multiple bore hole extensometers (MPBAX) and load cells were installed to monitor the rate and extent of convergence, and a number of deep cable anchors (blind and through) were installed to stabilize the area

(i) Blind Hole Cable Anchors

68 numbers of 18 m long (6 m fixed length) prestressed cable anchors of 110 tons load capacity (Locking force 77 tons) were installed in the blind holes of diameter 105-120 mm. on u/s wall of Machine Hall cavity particularly around the periphery of the bus duct tunnels.

(ii) Through Hole Cable Anchors

33 numbers of 21.7m long through prestressed cable anchors of 110 tons load capacity (Locking force 77 tons) were installed between bus ducts 1 and 2, 2 and 3, & 3 and 4 in the through holes of diameter 105-120 mm.

After installation of these cable anchors, the convergence reduced noticeably and stabilised thereafter.

4.0 Conclusion

Successful completion of India's highest dam across Bhagirathi is a landmark development in the history of high dam building in India. Geological and Geotechnical challenges have been adequately overcome by THDC and Project is functioning well since commissioning in 2006-2007. Tehri Hydropower complex not only met the drinking and irrigation water requirements of the command area and much needed peaking Power in the Grid but also mitigated floods during heavy rains in the year 2010, 2011 & 2013.

Acknowledgements

Author is grateful to THDC India Ltd and Geological Survey of India (GSI) for providing information about design & drawings, Geological & Geotechnical aspects and construction data relating to Tehri Power Complex.

References

i. B.L. Jatana, "Building Tehri Mega Dam In Himalayas- Challenges In Design and Construction" - Water and Energy International, Jan. to March 2007 Vol.64 No.1 published by Central Board of Irrigation and Power.

ii. P.C. Nawani, H.C. Khanduri and Harish Bahuguna, "Geotechnical Aspects of the Dam Foundation, Spillway Structures and Underground Caverns" - Water and Energy International, Jan. to March 2007 Vol.64 No.1 published by Central Board of Irrigation and Power.

iii. G.M. Prasad and R.K. Vishnoi, "Distinctive Features of Design and Construction of Stilling Basin of Tehri Chute Spillway" – Water and Energy International, Jan. to March 2007 Vol.64 No.1 published by Central Board of Irrigation and Power.

iv. R.K. Sharma, G.M. Prasad, Shailendra Singh and T.S. Routela, "Design of Underground Power House Caverns of Tehri HPP" Water and Energy International, Jan. to March 2007 Vol.64 No.1 published by Central Board of Irrigation and Power.

v. A.L. Shah, J.L. Narang, Shailendra Singh and Atul Jain, "Slope Stabilisation of Intake Area of Tehri HPP and PSP" – Water and Energy International, Jan. to March 2007 Vol.64 No.1 published by Central Board of Irrigation and Power.