2nd International Conference on

Sustainable Hydropower Development in the Himalayas

(India - Bhutan Partnership for Green Future)

20 - 21 November 2025. Dungkar Dzong, Paro, Bhutan

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Contents

FOUNDATION TREATMENT OF PARWAN DAM PROJECT - A CASE STUDY

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ABSTRACT

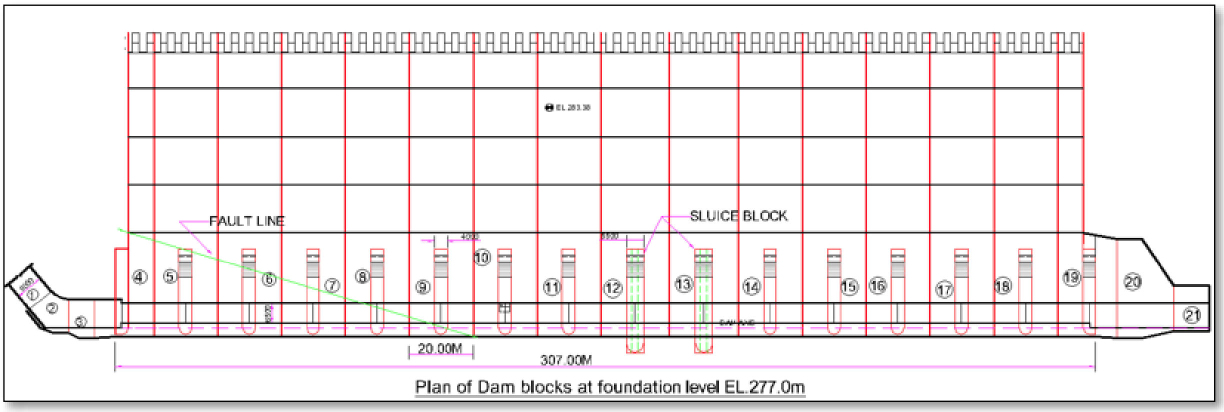
India has about 6628 specified large dams as per the National Register of Specified Dams (NRSD)-2025[1]. Most of the suitable sites for construction of dams have already been exploited and now new dams have to be constructed on technically challenging sites. The Parwan Major Multipurpose Project (PMMP), a 38-meter-high and 360-meter-long concrete gravity dam under advance stage of construction in Rajasthan, India, encountered significant geotechnical challenges due to a fault line mapped across dam blocks 5 to 9. Initial investigations by the Geological Survey of India (GSI) raised concerns about potential seismic activity, prompting advanced geophysical and geotechnical studies at the site. Comprehensive methods, including seismic refraction, Electrical Resistivity Tomography (ERT), and Multichannel Analysis of Surface Waves (MASW), revealed the fault as a narrow and inactive rock fracture/shear zone, tapering to a maximum depth of 20 meters in a stretch of 100 metres.

This paper illustrates the identification and delineation of a shear zone encountered in the dam foundation using geophysical methods, followed by an analysis of its seismic hazard potential and its impact on the dam. A 3-D Finite Element Model (FEM) of blocks 5 to 9 was developed using MIDAS FEA software to evaluate the effectiveness of the proposed treatment and the stresses induced in the dental concrete, thereby assessing the suitability of the reinforcement. Based on the FEM results, the foundation treatment was finalized and successfully implemented. The dam is now nearing completion.

Keywords: *Fault, Shear Zone, MASW, ERT, Seismic Resistivity, Dental treatment, FEM Analysis*

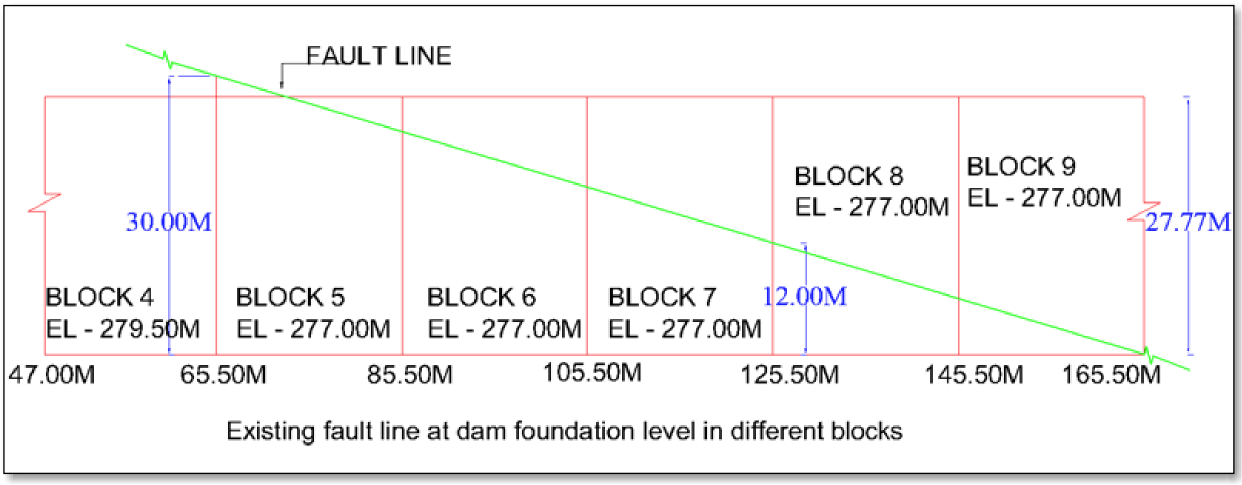
1- INTRODUCTION

The Parwan Major Multipurpose Project (PMMP) is a significant infrastructure initiative in Rajasthan, India, aimed at addressing regional water management challenges. The project features a 38-meter-high and 360-meter-long concrete gravity dam across the Parwan River, a tributary of the Kali-Sindh River, with a storage capacity of 490 million cubic meters (MCM). It includes 15 gated spillways (See fig.1), designed to manage design floods of approximately 29,000 cubic meters per second. The PMMP is primarily intended for pressurized micro-irrigation, drinking water and wild life. The foundation treatment of the PMMP presented unique challenges due to the identification of complex geological features within the dam site. During the foundation grade geological mapping of dam site by GSI, a fault line has been mapped in the dam foundation block Nos 5 to 9 (see fig-2). The trend of the fault is in N750W-S750E direction with sub-vertical dips. Crushed rock material along with fractured rock fragments have also been observed within the plane. The GSI apprehended that the blocks on either side of fault can move in any direction. In order to verify the above findings two inclined boreholes were drilled across the fault line and reported depth of fault was found about 39m with thickness varying from 0.15m to 0.60m. To evaluate the geotechnical and geophysical characteristics, as well as the seismological aspects of fault investigations, an integrated geophysical approach was applied. This involves the use of seismic refraction, shear wave measurement through Multichannel Analysis of Surface Waves (MASW), and Electrical Resistivity Tomography (ERT) methods. These techniques aim to determine the nature and extent of the fault and provide a comprehensive profile of the dam foundation.



Plan of Dam blocks at foundation level EL.277.0m

Fig-1 Dam Layout and Plan



Existing fault line at dam foundation level in different blocks

Fig-2 Plan of apprehended fault

3-Site geological set up and Sub-surface explorations

The project site exposes fine-grained, hard, and competent sandstones of the Upper Bhander Group (Vindhyan Supergroup). These thick- to thin-bedded sandstones, with near-horizontal to sub-horizontal dips, contain intercalations of silt, shale, and pebbly layers. Beds generally strike NW–SE with gentle dips (10°–20°) towards the southwest and northeast, reflecting broad warping. Sub-horizontal sandstone and silt/clay beds are prominently exposed on both flanks near the dam site, overlain by Quaternary loose sediments, soil, and drift deposits.

The subsurface explorations were undertaken to assess the continuity of faults, lithological characteristics, and rock-mass conditions. Two inclined boreholes, BH-1 and BH-2, were drilled across the mapped fault line to a depth of approximately 80m. These investigations focused on identifying fractures, measuring rock quality, and determining permeability values. Borehole BH-1 revealed fractured zones at various depths (1.72m to 31.08m) with a thickness of 0.10–0.85m. The rock was predominantly hard ortho-quartzite with red shale and silt bands. Recovery rates reached 100%, and rock quality designation (RQD) was 79%. Borehole BH-2 also encountered ortho-quartzite, intersecting fractured/weak zones at 12.55m to 37.33m with thicknesses ranging from 0.15–0.59m. RQD values in some zones were below 25%. Water percolation tests conducted at stages confirmed permeability values below 2 lugeons in fractured zones, indicating limited seepage risks. Both boreholes confirmed the fault line was a localized feature with no significant displacement or evidence of fault gauge.

5-Geophysical studies

5.1-Seismic refraction survey

The basic principles of the seismic refraction method have been described in detail by (Musgrave et al.1967) [2], (Dobrin and Savit et al. 1988) [3], (McCann and Forster et al.1990) [4] and (Telford et al. 1990) [5]. The seismic refraction survey conducted in the project area utilized a grid-pattern arrangement of survey lines to investigate the overburden and bedrock configuration, determine overburden thickness, and measure the compressional wave velocities of soil and bedrock. The survey employed a 24-channel high-resolution digital seismograph, capable of data stacking, frequency filtering, and various digital signal processing functions for optimal data acquisition. The sensors used were 10 Hz vertical geophones connected to the acquisition unit via specialized multiple take-out cables. A 10 kg sledgehammer served as the seismic source, with signals stacked 10–20 times at each shot point to enhance the signal-to-noise ratio. The field setup parameters included a channel spacing of 2–4 meters, a sampling interval of 250 milliseconds, and recording in SEG-2 format, with monochrome wiggle or similar displays. Data processing was carried out using Seis-Imager software. Seven sets of shots were recorded for each seismic spread, comprising three forward, three reverse, and one centre shot at different positions along the profile. Far-offset shots were recorded 30–40 meters from either end of the spread to ensure comprehensive data coverage.

5.1.1 Interpretation of Seismic Survey Results

The seismic data collected was of fair to good quality, with some interference from nearby vehicular and construction noise. The initial analysis utilized the reciprocal time method, refined through inversion in PLOTREFA software. A main refraction profile (L2) along the fault line (see fig-3), with six transverse profiles across dam blocks 4 to 9 i.e. (T1 to T6), was used to map P-wave velocity variations. The plot T2 in dam block 5 (see fig-4) has been shown. The velocity distribution indicated that the fault is a narrow, tight surface fracture with no significant width or depth. Subsurface P-wave velocities ranged from 800–2500 m/s in thin surface layers, increasing to 2500–3000 m/s up to 2–3 m depth, and exceeding 3500 m/s in harder rock below. Field observations and core data confirmed that the rock mass is relatively hard, with medium-velocity readings near the surface likely due to blasting-induced fractures.

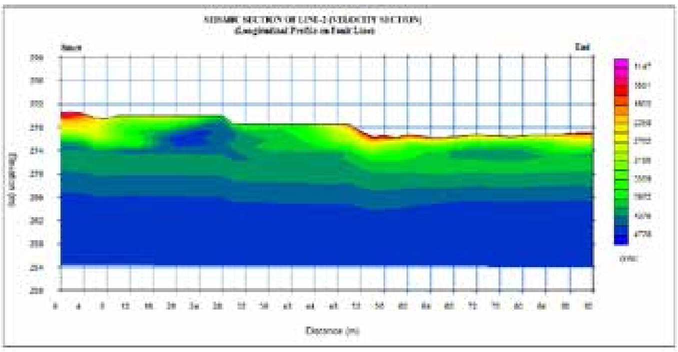


Fig-3-: Longitudinal Profile (L-2) on Fault Line.

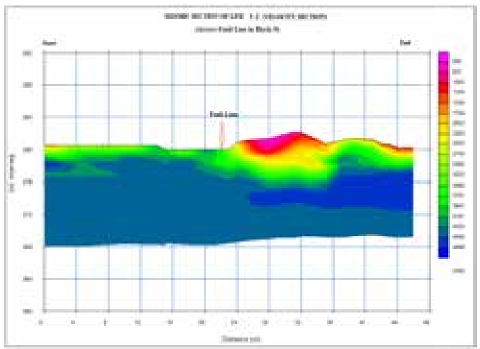


Fig. 4 Transverse Profile in Block-5

5.2 Electrical Resistivity tomography

2-D electrical imaging/tomography surveys for mapping moderately complex geological structures (Griffiths and Barker,1993) [6]. These surveys utilize systems with 64 or more electrodes connected via multi-core cables, enabling automated data collection through a switching unit that selects the relevant four electrodes for each measurement. A multichannel resistivity system with 60 electrodes, spaced at 1–2 meters, often uses a Schlumberger array for deeper investigations. Constant electrode spacing is maintained, and most fieldwork involves laying out cables and electrodes, with measurements automatically recorded. Data is saved in RES2DINV format for processing using finite difference modelling and finite element inversion methods, producing coloured contour plots for interpretation.

5.2.2 Interpretation of Resistivity Survey Results

Three ERT profiles L-2 (see Fig. 6) were conducted parallel to the fault line at 4-meter intervals. Additionally, six transverse refraction profiles (T-1 to T-6), with only T-2 in Block 5 shown (see Fig. 5), were carried out perpendicular to the fault line to measure the resistivity distribution and its lateral and vertical variations. The focus was on locations where these profiles intersect the fault to observe resistivity changes, which could indicate the fault's width and depth. The resistivity tomograms revealed no significant variations in the fault's width or depth, suggesting the fault is a narrow, tight fracture. The resistivity sections of L-1, L-2, and L-3 showed uniform strata with no considerable variation in resistivity. A thin cover layer with resistivity between 50 Ohm-m and 300 Ohm-m was identified, followed by a layer with resistivity ranging from 300 Ohm-m to 700 Ohm-m, and more than 700 Ohm-m up to 1000 Ohm-m. This suggests the rock mass is hard and the fracture is narrow and tight.

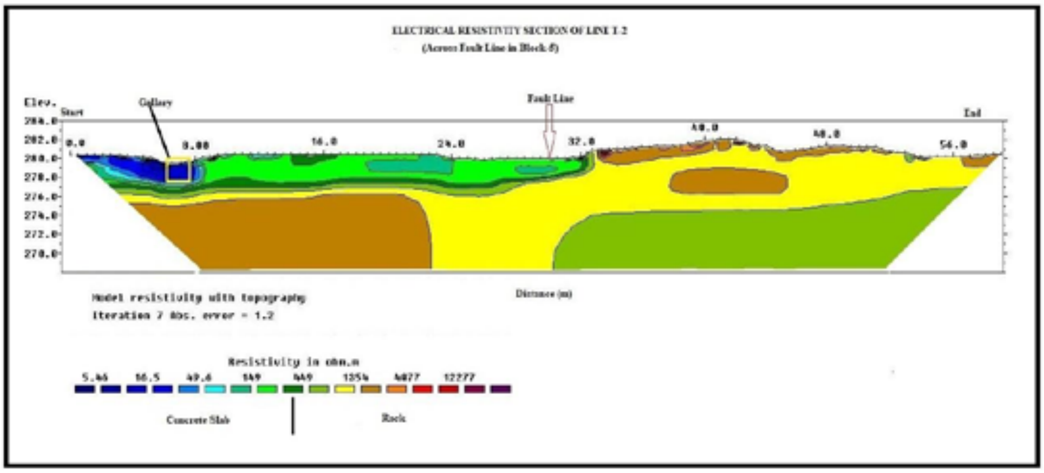


Fig.5 Transverse Profile (T-2) in Block-5

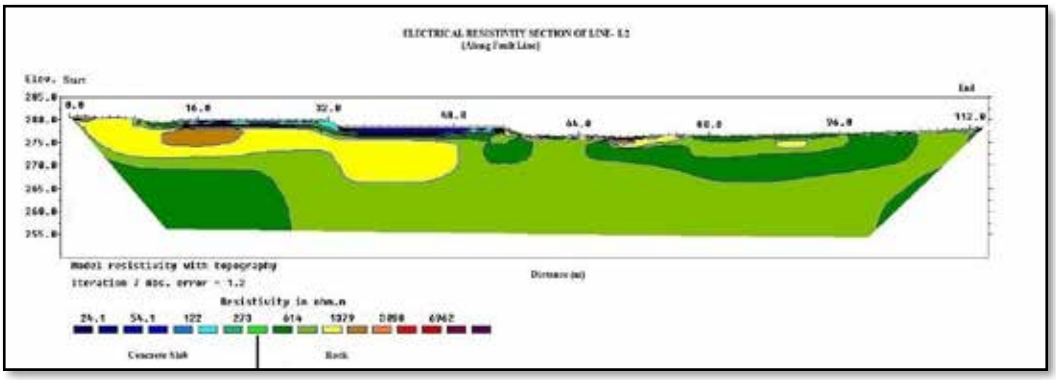


Fig.6 Longitudinal Profile L-2 Parallel to Fault Line.

5.3 Multichannel analysis of surface waves (MASW) surveys

Multichannel Analysis of Surface Waves (MASW) is a seismic exploration technique first introduced in geophysics by (Park et al. 1999) [7], is a non-destructive technique that uses the dispersion of Rayleigh waves to determine the vertical distribution of S-wave velocity in the subsurface, which reflects the elasticity, hardness, and stiffness of underground materials. The MASW survey involved generating seismic waves with a mechanical source and recording them using a 24-geophone array across 46m or 92m spreads. Data was processed to generate a dispersion curve, representing phase velocity versus frequency, which was then inverted to calculate the 1-D and 2-D distribution of S-wave velocity up to 30m depth. The results, processed with Seis-Imager SW software, provided subsurface images for site characterization and seismic hazard assessment. The technique's reliable and non-destructive nature makes it valuable for seismic site classification, with the average shear wave velocity at 30m depth (Vs30) being a critical parameter for earthquake engineering.

5.3.1 Results and interpretation

The MASW survey was conducted across six profiles transverse to a fault zone (blocks 4 to 9). The survey used 2.0 m shot and receiver spacing for the transverse profiles and one profile along the fault trace with 4.0 m spacing (see Fig. 9). The data quality obtained was reasonably good. The analysis was done following the SEISIMAGER software. It was seen that the anomalous shear velocity zone in each block is present near or in the vicinity of the surface exposure of fracture. The value of velocity in the anomalous velocity zones obtained in blocks 4 to 9 has velocity of more than 850 m/s. The range of obtained shear wave velocity for all profiles is from 800 m/s to 2500 m/s. The depth of anomalous velocity zone is maximum at block 4 and is minimum at block 9. The anomalous velocity zone in different blocks is overlaid by high-velocity layers, which are nearly horizontal. This indicates that the depth of fracture may be up to 20 m and decreases from block 4 to 9. Shear wave velocities ranged from 800 to 2500 m/s, with a maximum fracture depth of approximately 20 m in block 4, decreasing towards block 9. Based on the Vs30 values, the site is classified as very hard rock to rock according to the (National Earthquake Hazard Reduction Program NEHRP) [8] site classification (Building Seismic Safety Council BSSC, 2003), table-1.

Site Classification	Description	Average shear wave velocity up to 30 m (Vs30) ms-1
A	Very hard rocks	>1500
B	Rocks	760 < Vs30< 1500
C	Very hard soil and soft rock	360 < Vs30< 760
D	Hard soil (sands, clays and gravels)	180 < Vs30< 360
E	Soft clay of thickness about H in site profiles	Vs30< 180

Table-1 NEHRP site classes (BSSC, 2003)

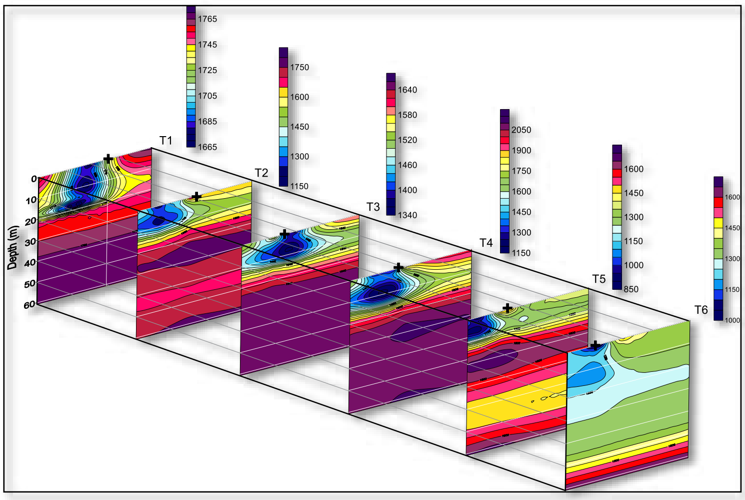


Fig-9 Interpreted section along block 4, 5, 6,7, 8, and 9 respectively. Point T1, T2, T3, T4, T5, and T6 are the traces of fracture obtained at the surface. The contours are drawn in each block differently on a relative scale.

6- Seismic status of the fault line

The Project site is located in Seismic Zone II, as per the Seismic Zoning Map of India (IS: 1893 Part 1: 2002) [9], with a Maximum Considered Earthquake (MCE) value of 0.193g and a Design Basis Earthquake (DBE) value of 0.117g horizontally and 0.078g vertically. The seismic coefficient for the dam design is αh=0.07 and αv=0.04. Historical and instrumental earthquake data show no significant events in the Jhalawar area, though fringe effects from larger earthquakes like the 1819 Rann of Kutch earthquake (M=8.0) have been noted. The site is categorized as a low seismic risk zone, and the fault line at the dam, approximately 100m in length, is too small to pose a significant seismic risk. The fault is not considered active or potentially active by ICOLD standards due to its limited extent and lack of surface manifestation. Examination of cores from two drill holes across the fault showed massive ortho-quartzite with minor shale/siltstone partings and no evidence of fault displacement, confirming no vertical bed displacement. The seismic design parameters for the dam account for all relevant seismic sources and tectonic features in the region.

7- Proposed Treatment Measures

Based on the geophysical studies it was concluded that the fault reported by GSI in dam blocks no.5 to 9 is not a fault, but a tight rock fracture/shear zone. The fracture is amenable to treatment for structural stability and seepage control and has no possibility of generating any movement. Following remedial measures were proposed for treatment of the rock fracture/shear zone.

7.1 Target grouting

To reduce the risk of seepage and piping, grouting along the fracture, in addition to consolidation grouting, was proposed. Initially, an inspection drill hole (I-Hole) would be drilled along the fracture line in block No. 5, to a depth of 20m, with water percolation tests conducted in 3m sections using cyclic pressure (1-2-3-2-1) up to a maximum of 3 kg/cm². This will determine the base permeability values of the fault line. The fault will then be grouted at 6m intervals using primary grout (PG) holes, with cement-water grout mixture applied in stages until refusal, and post-grout permeability will be measured in the I-Hole to achieve a target of 1 lugeon. If the desired permeability is not reached, the distance between holes may be reduced to 3m. The drill hole inclination will depend on site conditions and the fault orientation, potentially inclining upstream for better grout intake. This grouting procedure will be systematically applied along the fault line from Block 5 to Block 9.

7.2. Dental treatment

This treatment entails removing the weak material and backfilling the resulting excavation with concrete. A zone of the shape of trapezoid was excavated along the fracture by excavating a portion of the rock on either side of the fracture edge and having a depth of half of the top width, excluding fracture width, and base width equal to the width of the fracture. With the width of the fracture varying from 0.15m to 0.60m, a variable width of the trapezoid would result. To simplify the construction and design of the proposed treatment, a uniform width of 1.6m was kept with 0.8m on either side from the central axis line of the fracture. The depth of the V-notch was kept as 0.5m. This would facilitate uniform load transfer to the rock on either side of the fracture through compression. The excavated area was backfilled with shrink-resistant concrete of grade M40. Below this trapezoid section, dental treatment in the fracture region was undertaken as per standard (USBR 1976) [10] procedure commonly known as the Shasta formulae given below:

$$d = 0.002bH + 5 \qquad \text{for } H \geq 150 \text{ feet - (i)}$$

$$d = 0.3b + 5 \qquad \text{for } H < 150 \text{ feet - (ii)}$$

Where- H = Height of dam above general foundation level in feet;
b = Width of weak zone in feet; and
d = depth of excavation of weak zone below surface of adjoining sound rock in feet. (In clay gouge seams, d should not be less than 0.1H)

Since in the present case, the depth of the dam is less than 150 feet (45.72m), the formula (ii) would apply. The depth of the plug works out to 1.70m. The dental treatment can be applied below the base of the trapezoid plug. The total depth of treatment is 2.2m (see Fig-10). The concrete of grade M40 was used for dental treatment.

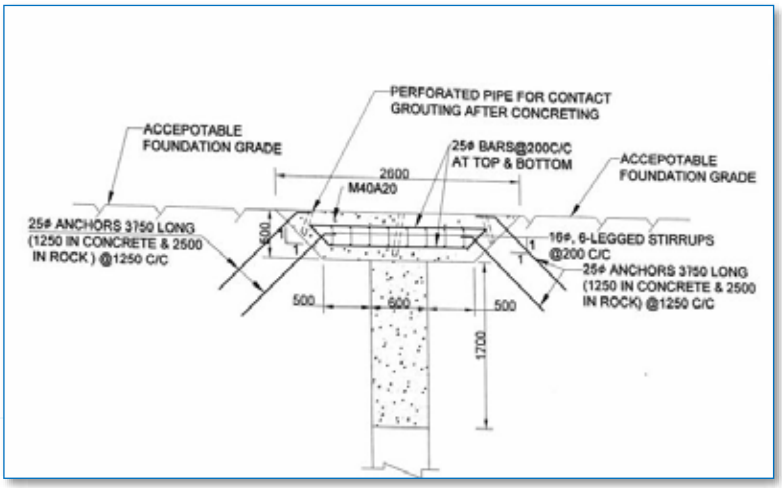


Fig. 10- Dental treatment of fracture/shear zone

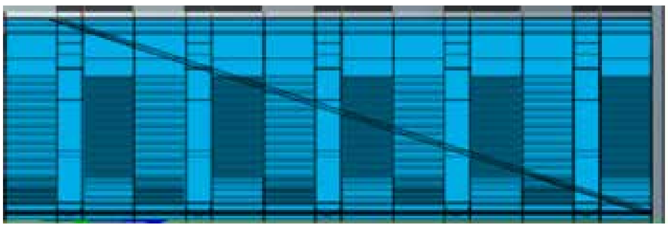
7.3 FEM Analysis

The foundation's behaviour with the proposed treatment was analysed using FEM to evaluate the resulting stress patterns at the contact points between the rock and dental concrete, particularly near the raft, due to differences in their moduli of elasticity. Based on this analysis, the reinforcement provided in the dental treatment was validated.

8- FEM modelling

3D Finite Element Model of blocks 5 to 9 was developed using MIDAS FEA software. Induced stresses, represented as major principal stresses on the bottom face of the spillway mass concrete, were analysed to assess the suitability of the proposed reinforcement in these blocks. The founding rock was modelled as a massless material to simulate the stable state of the excavated foundation under its self-weight. Deformations were induced solely by the weight and forces exerted on the dam blocks. A fractured zone, 600 mm thick (see Fig-11), was incorporated into the analysis. Additionally, interface elements were introduced to simulate contraction joints between the blocks and the rock-structure interaction at the founding level of the dam blocks.

The mass concrete structure and surrounding rock were modelled using a self-generating solid mesh in MIDAS. A mesh size of 1 m was used for the dam body, with a coarser mesh for the surrounding rock in the MIDAS model. A mesh size of 0.2 m was employed to model the shear zone. After conducting various sensitivity studies, it was found that the mesh size of the surrounding rock had no significant impact on the induced stresses in the dam block.



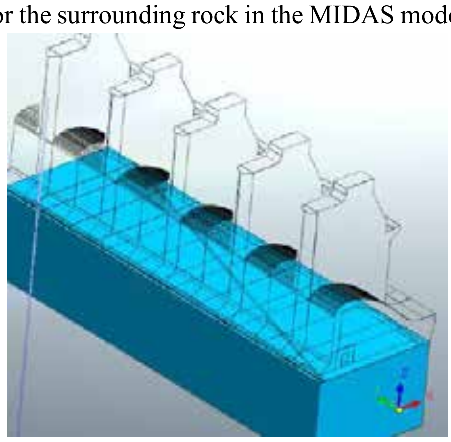


Fig.11: -Orientation of 600mm fractured zone

8.1 Material Properties: (Table-2)

Material Name	Property	Value	Unit
Isotropic Concrete	Grade of concrete, Fck	M25/M20	
	Static modulus of Elasticity of M25 concrete, Ec	2.5e+007	kN/m2
	Static modulus of Elasticity of M20 concrete, Ec’	2.236e+007	kN/m2
	Poisson's Ratio, m	0.2	
	Density, concrete	24	kN/m3
Rock (Mohr Coulomb)	Elastic Modulus	25000000	kN/m2
	Cohesion	110	kN/m2
	Friction Angle	46	degrees
	Poisson’s Ratio	0.3	

Table-2 Actual site-specific material properties

8.2 Load & Support conditions:

Design loads and load combinations were determined in accordance with BIS 6512[11] (see Fig-13). Site-specific seismic studies provided the parameters for seismic design. The horizontal inertial force was calculated by multiplying the seismic coefficient by the weight of the structure, with the inertia force assumed to act at the structure's centroid. Earthquake forces, both horizontal and vertical, were treated as static inertia forces and combined with other loads. A horizontal seismic coefficient of 0.12 and a vertical seismic coefficient of 0.08 were used. Lateral restraints were applied to the left bank, right bank, and downstream faces of the rock, while the bottom of the rock was restrained in all directions (see Fig-12).

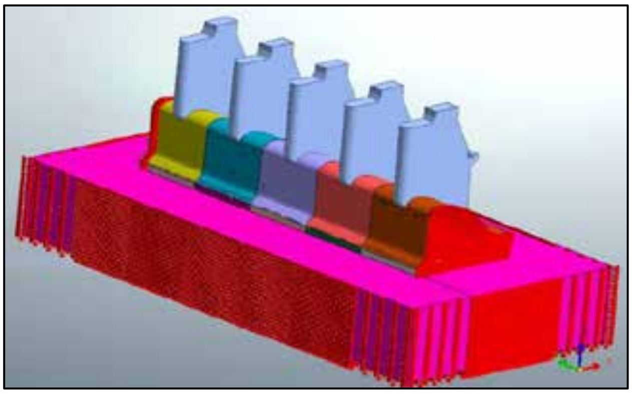


Fig. 12- Boundary condition

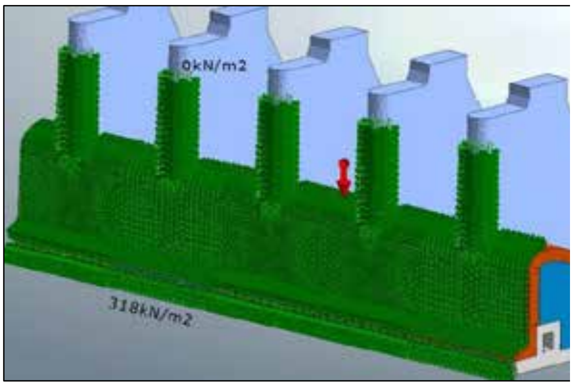


Fig.13-Design Loads

8.3 FEM Results

The distribution of principal stresses for load combinations B and E (see Fig. 14) at the bottom of the spillway block, including the 0.6 m wide shear zone concrete, is shown for dam block no. 5. he maximum induced stresses for all other blocks under all load conditions are presented in Table 2. Based on the FEM results, it can be concluded that the reinforcement provided in the dental treatment is adequate to withstand the tensile stresses generated.

Dam Block	Maximum induced compressive stress (-ve stresses are compressive)	Maximum induced tensile stress (+ve stresses are tensile)
Block-5	0.115MPa	0.20 MPa
Block-6	0.27MPa	0.25MPa
Block-7	0.24 MPa	0.26 MPa
Block-8	0.12 MPa	0.20 MPa
Block-9	0.18 MPa	0.26 MPa

Table-2

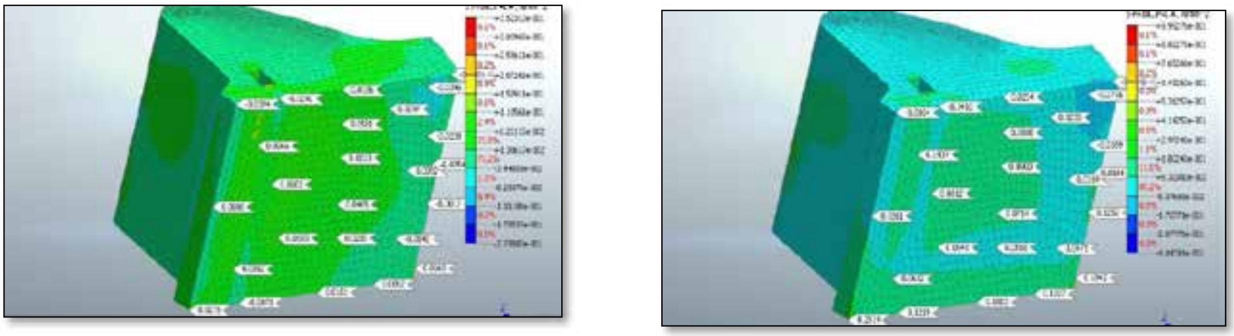


Fig. 14 Load B and Load condition E, Major principal stress distribution

9- Conclusion

Geophysical investigations, including Seismic Refraction, Electrical Resistivity Tomography (ERT), and MASW mapping, confirmed the presence of impermeable and stable rock beneath the fracture zones in the foundation of dam blocks 5 to 9. The observed surface fault was identified as a narrow, compact fracture with a maximum depth of 20 meters over a stretch of 100 meters, tapering across dam blocks 5 to 9. It was deemed inactive, eliminating the need for further seismic hazard assessments. Borehole data corroborated these findings, indicating the fault zone was a localized feature without significant broader implications.

Remedial measures, including dental concreting and targeted grouting, effectively addressed structural and seepage concerns, ensuring the foundation's stability. Subsequent structural analyses through Finite Element Modelling validated the adequacy of reinforcements and treatment strategies under operational and seismic loading conditions.

This study demonstrates the integration of advanced geological investigation, structural engineering analysis, and practical remediation techniques to overcome complex geotechnical challenges in dam construction, offering valuable insights for similar future projects. The dam is now in its advanced stages of construction, marking a significant achievement in addressing adverse foundation conditions.

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GEOTECHNICAL INVESTIGATION METHODS FOR EARTH DAMS

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ABSTRACT

This paper presents a structured methodology for geotechnical investigation and assessment of earth dams within hydroelectric installations. The objective is to characterize embankment retaining structures through systematic collection of geological and geotechnical information. The proposed approach integrates field investigations—including borehole drilling, trenching, riprap block size surveys, geological and topographic mapping, and piezometer installation–with laboratory testing of soil and rock samples. This process supports the evaluation of material properties, stratigraphy, and hydraulic behavior to inform maintenance planning and risk mitigation. All activities are carried out under rigorous health and safety protocols. The methodology results in a comprehensive geotechnical database and technical reports that enhance dam safety management and asset reliability.

Keywords *: Earth dams; Embankment retaining structures; Geotechnical investigation*

1. GENERALITIES

1.1 Object

The objective of this paper is to present a structured and comprehensive methodology for assessing the condition, severity, and extent of damage to earth dam elements in hydroelectric structures. This methodology focuses on geotechnical investigations of water retaining embankment and outlines the investigation methods, summarizes relevant evaluation criteria, and specifies deliverables to support maintenance planning, risk mitigation, and asset management.

As shown in photo 1 below, a typical hydroelectric complex includes the following main components:

- Retaining Dam
- Spillways equipped with mechanically operated gates designed to discharge excess water downstream during high-flow conditions
- Intake structure
- A powerhouse equipped with turbines

Photo 1 : General overview of hydroelectric installation-source Youtube, TheGaiaProject

2. GEOTECHNICAL INVESTIGATION PROGRAM

The purpose of the geotechnical investigation program is to collect the geological and geotechnical information required to characterize embankment retaining structures. Photo 2 below shows an example of a dam cross section.

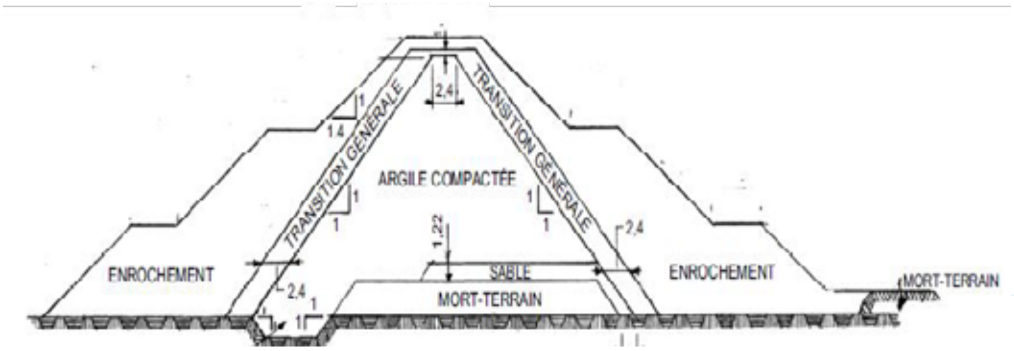

Photo 2 : Example of a dam cross section

Investigation program involves:

2.1 Boreholes:

- Determine the nature, density or consistency of soils and, where applicable, the depth and quality of the bedrock.
- Ensure good sample recuperation; verify that collected samples are representative of in-situ conditions.

2.2 Exploration Pits and Trenches:

- Identify stratigraphic horizons, nature and apparent density of each layer.
- Determine the level of the impermeable core.

2.3 Riprap Measurements:

- Assess block size distribution, shape, geological origin, degree of alteration, and thickness.

2.4 Geological Mapping:

- Map upstream and downstream slopes, identifying surface material types, changes, and extent of different riprap types.

2.5 Topographic Surveys:

- Record coordinates and ground elevations for borehole locations and mapped boundaries.

2.6 Piezometers:

- Measure the piezometric level within the embankment structure.

3. DESCRIPTION OF THE WORK AND SPECIFIC INSTRUCTIONS

3.1 Methodology

Geological and geotechnical investigations must follow technical specifications provided by the operator or owner. Results should be compiled into a database as per owner requirements. Riprap block dimensions and field measurements must be recorded in structured tables (e.g., Excel).

3.2 Safety Program, Work Method, Access, and PPE

The laboratory performing investigations must maintain a health and safety prevention program, schedule, work methods, and required equipment for the investigation campaign. Provide necessary personal protective equipment (PPE), including for work near water.

3.3 Underground or Overhead Infrastructure

For all boreholes, the laboratory must confirm with the owner that no underground or overhead infrastructure exists at the planned drilling locations.

3.4 Site Restoration

After drilling work, the laboratory must restore the site into the original conditions. Photos should be taken before work begins and after equipment is demobilized from drilling sites.

3.5 Topographic Surveys

The topographic survey must be performed according to the owner's instructions.

3.6 Boreholes, Location and Drilling

The location and estimated length of boreholes must be determined based on the type and nature of the structures under investigation. The location must be confirmed in advance by the owner.

Drilling must be conducted using conventional rotary or driven methods with a rig of adequate capacity:

- Drilling rig must have sufficient capacity for the required depth.
- Standard Penetration Tests (SPT) must be performed continuously, with energy measurement, to obtain normalized N-values and include sample descriptions.
- Record drilling fluid losses during drilling as a percentage of total flushing fluid, as well as other observations.
- Conduct grain size analysis on all representative samples; subdivide and analyze heterogeneous samples.
- Bedrock must be cored with NQ-size equipment, ensuring maximum recuperation in fractured rock zones.

Rock cores logged by a geologist should include:

- Petrographic unit nature
- Mineralogical composition, texture
- Description of fracture zones
- Run lengths, RQD, and recuperation percentage
- Structural description of discontinuities (depth, alteration, slope, etc.)
- If cohesive materials are encountered, have Shelby tubes and a field vane shear tester available
- Install open-tube piezometers
- Split spoon samplers of H, N, and B sizes must be available
- Grout boreholes with a cement-bentonite slurry from bedrock to surface.

3.7 Exploration Trenches

Trenches should be excavated using an excavator. They must reach a maximum depth of approximately 2.0 m or as needed to reach the core's impermeable element crest level. Representative samples must be collected from each stratigraphic horizon for laboratory testing. Material color and apparent density must be recorded.

The percentage of cobbles and boulders must be visually estimated and reported in ranges (e.g., 5–10% boulders, 5–15% cobbles). During excavation, materials must be sorted by composition. After excavation, backfilling must restore materials in reverse order, compacted in layers ≤300 mm to a density equal to or greater than adjacent undisturbed material.

3.8 Riprap Grain Size Survey

Surveys should be planned according to the owner's instructions. For each measurement point:

- Record block shape (cubic, rounded, flat, elongated, etc.)
- Record riprap thickness (if possible)
- Identify block geological origin
- Block degradation (cracking, delamination, etc.)
- Take representative photos.

3.9 Surface and Riprap Geological Mapping

Slopes without riprap:

- Identify and delineate surface materials on upstream and downstream slopes.
- Mark vegetated areas and types.
- Identify any specific observation.

Slopes with riprap:

- Mark representative area for grain size measurement.
- Delineate riprap types if more than one is present.
- Describe degradation (toe erosion, crest without blocks, erosion zones, etc.).

- Representative photographs must be taken during mapping.

3.10 Piezometers

- Install piezometers as directed by the owner.
- Measure stabilized water level to assess seepage conditions.

4. LABORATORY TESTING

A testing program must be defined based on field results. Sample selection is approved by the owner's representative.

Table 1 summarizes typical soil tests, and Table 2 summarizes rock tests.

Table 1 : Soil Sample Testing

Tests	Standard
Natural water content	BNQ 2501-170 or equivalent national standard.
Liquid and plastic limits	BNQ 2501-092 or equivalent national standard.
Undrained shear strength and sensitivity of cohesive soils using cone penetrometer	BNQ 2501-110 or equivalent national standard.
Grain size analysis of inorganic soils – sieving	BNQ 2501-025 or equivalent national standard.
Grain size analysis of inorganic soils – sedimentation	BNQ 2501-025 or equivalent national standard.
Volumique Density	NF P98-250-6 or equivalent national standard.

Table 2 : Rock Sample Testing

Tests	Standard
Absorption	LC 21-067
Apparent Density	LC 21-067
Los Angeles abrasion resistance	LC 21-400
Micro-Deval abrasion resistance	LC 21-070

5. REPORT

All collected data must be compiled into a structured geotechnical database defined by the owner. Investigation reports must integrate:

- Borehole logs, trench logs, and piezometer data
- Laboratory test results
- Mapping records and photographs
- Riprap surveys and topographic data.

This integrated dataset supports condition assessment, risk evaluation, and long-term maintenance planning.

6. CONCLUSION

This paper presents a structured methodology for geotechnical investigation of earth dams in hydroelectric facilities. By combining systematic field exploration with standardized laboratory testing and documentation, dam owners can develop a reliable understanding of material properties and hydraulic behavior. This approach enhances decision-making for maintenance, improves risk management, and supports long-term asset performance and regulatory compliance.

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4. BNQ 2501-110 – Determination of Undrained Shear Strength and Sensitivity (Cone Penetrometer)
5. BNQ 2501-025 – Grain Size Analysis of Soils (Sieving and Sedimentation)
6. NF P98-250-6 – Volumetric Density of Soils
7. LC 21-067 – Absorption and Apparent Density of Rock
8. LC 21-400 – Los Angeles Abrasion Resistance Test for Aggregates
9. LC 21-070 – Micro-Deval Abrasion Resistance Test for Aggregates

APPLICATIONS OF ROVS IN FLOODED TUNNEL INVESTIGATIONS FOR SUSTAINABLE HYDROPOWER DEVELOPMENT IN THE HIMALAYAS

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ABSTRACT

Sustainable hydropower development in the Himalayan region demands innovative, safe, and environmentally responsible approaches to managing critical water infrastructure. EyeROV's indigenous underwater robotic systems provide an advanced solution for the inspection and maintenance of submerged hydropower assets such as tunnels, penstocks, and intake structures-without dewatering or diver intervention. These Remotely Operated Vehicles (ROVs) integrate high-definition imaging, sonar mapping, and AI-based analytics to enable precise, real-time assessment of structural conditions under complex mountain environments. By improving inspection accuracy and operational safety, EyeROV's technology reduces downtime, mitigates environmental disturbance, and enhances long-term asset reliability. Aligned with the India-Bhutan vision for a green and resilient Himalayan future, this innovation supports sustainable hydropower development through cost-effective maintenance, reduced ecological footprint, and data-driven decision-making-advancing regional cooperation and technological self-reliance in hydropower modernization.

Keywords *: Sustainable hydropower, underwater robotics, tunnel inspection, AI analytics, resilience, India-Bhutan cooperation*

1. INTRODUCTION

Hydropower plays a vital role in achieving sustainable energy goals across the Himalayan region, providing clean, renewable power for both India and Bhutan. However, hydropower infrastructure-especially submerged tunnels and penstocks-faces degradation due to sedimentation, corrosion, and extreme hydraulic conditions. Traditional inspection techniques often involve diver operations or dewatering, posing significant safety risks and operational challenges. EyeROV's Remotely Operated Vehicles (ROVs) enable safe, non-intrusive underwater inspections, eliminating the need for dewatering and reducing downtime. The deployment of ROVs enhances inspection frequency and accuracy, ensuring better asset management and long-term sustainability for hydropower facilities in mountainous terrains.

2. METHODOLOGY AND DATA ANALYSIS

ROV-based inspections are carried out using EyeROV's indigenous systems equipped with sonar, high-definition cameras, and laser measuring tools. These ROVs are capable of navigating long tunnels and confined underwater spaces, transmitting real-time data to operators. Profiling sonar and imaging sensors capture tunnel geometry, while AI-powered analytics detect sedimentation, cracks, and deformation. The collected data enables 3D reconstruction of submerged tunnels, supporting predictive maintenance and engineering assessments. By using portable control stations and modular payloads, the systems are easily deployable in remote Himalayan hydropower sites.

3. RESULTS AND DISCUSSION

EyeROV's technology has been successfully implemented in multiple hydropower projects across India. The use of imaging sonar and laser measurement has provided engineers with actionable data on tunnel profiles and sedimentation trends. Compared to traditional methods, ROV-based inspections reduced inspection time by 40-60% and eliminated safety risks associated with diver-based operations. The technology's minimal environmental footprint aligns with sustainability goals by preventing unnecessary water discharge or disturbance to aquatic ecosystems. Such innovations highlight the importance of integrating robotics into sustainable hydropower operation and maintenance frameworks.

4. CONCLUSION

The application of underwater robotics, such as EyeROV's ROV systems, represents a key advancement in achieving sustainable and resilient hydropower development in the Himalayas. By ensuring safe, efficient, and environmentally responsible inspections, these technologies contribute to the longevity of critical hydropower infrastructure and support regional cooperation between India and Bhutan. The adoption of indigenous innovations like EyeROV aligns with national goals for green energy, self-reliance, and climate adaptation, reinforcing the shared commitment to a sustainable Himalayan future.

INTEGRATED MULTI-SENSOR APPROACH FOR GEOTECHNICAL SLOPE MONITORING: A CASE STUDY OF PUNATSANGCHHU-I HYDROELECTRIC PROJECT, BHUTAN

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ABSTRACT

Slope monitoring is essential for maintaining the safety and long-term stability of substantial infrastructure projects, particularly in geologically complex Himalayan terrain and high-risk areas. This study presents a comprehensive multi-sensor monitoring framework that integrates the sophisticated functionalities of the IBIS-FM Ground-Based Interferometric Radar (GB-InSAR) with other geotechnical instruments, such as Total Station, Load Cells, Piezometers, and Inclinometers. The GB-InSAR delivers high-resolution, real-time measurements of surface displacement across large regions, while the in-situ sensors record critical subsurface parameters including pore water pressure, internal deformations, and load responses. The amalgamation of these varied datasets allows for a comprehensive analysis of slope dynamics, promotes early identification of possible failure mechanisms, and supports informed risk mitigation strategies. Field implementation at the Punatsangchhu-I Hydroelectric Project Authority (PHPA-I), Bhutan illustrates the efficacy and reliability of this integrated method in regulating slope stability within intricate Himalayan geological settings.

***Keywords**: Slope Monitoring; Multi-Sensor Approach; IBIS-FM Radar; Geotechnical Instrumentation; PHPA-I Bhutan*

1. INTRODUCTION

Monitoring slope displacements adjacent to significant infrastructure projects is a global necessity, particularly in regions characterized by complex geological conditions, steep and rugged topography, where even minor displacements can compromise structural integrity and operational effectiveness (Fell et al., 2005; Petley 2012). The rapid growth of hydroelectric infrastructure, transportation networks, and urban projects in hilly regions has increased the demand for precise and continuous slope monitoring to ensure safety and long-term operational sustainability (Praveen et al. 2022). Conventional geotechnical investigations, while crucial for initial site evaluation, often exhibit limited spatial coverage and insufficient temporal resolution, reducing their effectiveness in identifying gradual or progressive slope deformations over extensive regions (Gili et al., 2000; Calà et al., 2016).

To mitigate these constraints, modern slope monitoring techniques increasingly utilize integrated frameworks that amalgamate surface based and subsurface sensors. Multi-instrument methodologies offer a more thorough characterization of slope dynamics, facilitating informed decision making, prompt intervention, and strategic long term planning for essential infrastructure.

Surface monitoring approaches encompass conventional instruments such as total stations, geodetic targets, and tilt meters, which offer excellent measurement accuracy but are limited to specific locations (Dini et al., 2020). Conversely, Ground-Based Interferometric Synthetic Aperture Radar (GB-InSAR) systems, exemplified by the IBIS FM radar, provide near real-time surveillance of slope deformations over extensive regions, attaining accuracy levels from millimeters to sub-millimeters (Carlà et al., 2018; Tarchi et al., 2003; Ferrigno et al., 2017; Su et al., 2022). GB-InSAR applications in open-pit mines, hydropower projects, and key infrastructure slopes have shown effective in identifying widespread deformation patterns and delivering early warnings, hence improving operating safety (Manconi et al., 2024).

Subsurface instrumentation enhances surface monitoring by directly measuring internal slope responses. Inclinometers assess lateral displacements, load cells evaluate stress redistribution in anchors or support structures, and piezometers monitor fluctuations in pore water pressure inside the slope mass (Gigli et al., 2011; Lowry et al., 2013; Frodella et al., 2017; Lhamo et al., 2022). Improvements in subsurface monitoring, particularly through data driven anomaly identification utilizing inclinometer time series, have enhanced predictive capabilities and early warning potential (Lowry et al., 2013; Lhamo and Chao, 2023). The integration of surface and subsurface measurements facilitates a comprehensive knowledge of deformation causes, enhances predictive modeling of displacement trends, and guides appropriate slope management techniques.

Integrated multi-instrument monitoring is increasingly recognized as the benchmark for precise and reliable slope surveillance allowing cross validation of measurements, enhanced spatial and temporal resolution, and the establishment of robust early warning systems (Antonello et al., 2004). Previous studies have demonstrated that combining surface measurements such as high precision total stations and ground based radar with subsurface instrumentation including inclinometers, piezometers, and load cells provides a comprehensive understanding of slope deformation and internal stress redistribution (Pecoraro et al. 2021; Debevec Jordanova et al., 2025).

This study focuses on the right bank slope of the PHPA-I, Bhutan, which is characterized by rugged topography and complex geology. The site-specific geological conditions, together with variable geotechnical properties, necessitate detailed and continuous monitoring to ensure safe construction and to guide the design of effective slope stabilization measures. In this context, integrated multi-instrument monitoring approaches have been implemented at the site to elucidate deformation patterns and support informed slope management. The slope is instrumented with a comprehensive network of surface target points, GB InSAR, inclinometers, load cells, and piezometers, allowing simultaneous tracking of surface and subsurface movements.

This research is innovative due to the implementation of a fully integrated, multi-instrument monitoring system in the geologically intricate Himalayan environment, which merges high-resolution surface and subsurface measurements to produce an unparalleled dataset of slope dynamics. This method facilitates a comprehensive examination of the temporal and spatial progression of slope deformation resulting from anthropogenic activities and environmental factors, yielding insights that may be directly applied to the design of stabilization strategies. The study's significance transcends the PHPA-I project, providing a rigorous standard for slope monitoring, risk assessment, and management in extensive hydropower projects situated in similarly challenging Himalayan terrains.

2. PROJECT SITE AND GEOLOGICAL SETTING

The PHPA-I in Bhutan is situated in the Wangdue Phodrang district and constitutes the country's largest hydropower project, serving as a vital portion of Bhutan's renewable energy framework. The location is roughly 115 km from Paro International Airport, 73 km from Thimphu, and 10 km from Wangdue Phodrang town, underscoring its accessibility and strategic significance. Figure 1 illustrates the layout of the dam, powerhouse complex, and related infrastructure.

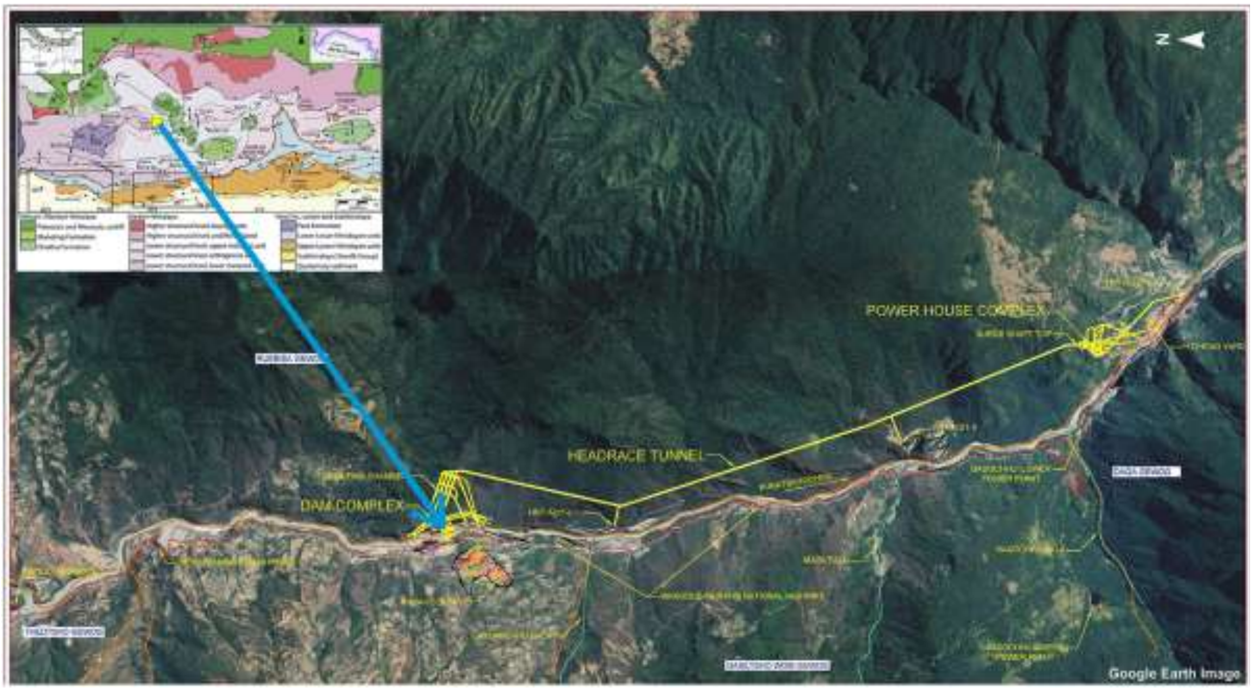

Fig. 1 : Layout of the Dam, Powerhouse Complex, and associated components of the PHPA-I, Bhutan.

The bedrock in the PHPA-I area consists mostly of garnet-bearing, foliated, and jointed quartz-biotite gneiss and quartzo-feldspathic gneiss, intermingled with bands of muscovite-biotite schist and veins of pegmatite and quartz. These medium to high grade metamorphic rocks are crystalline, with medium to coarse-grained textures, and display pronounced foliation and gneissosity, indicative of a polyphase tectono metamorphic history (Valdiya, 2010; Gupta and Tripathi, 2018). The lithological variability, along with intricate structural discontinuities like folds, mega-shears, and tectonic dislocations, emphasizes the necessity for comprehensive geotechnical characterization and ongoing monitoring in order to ensure the safe design and construction of hydropower infrastructure.

The regional foliation predominantly trends N10°–60°E to S10°–60°W, with a dip of 10°–40° towards N100°–N150°, underscoring the structural complexity of the Himalayan terrains (Gupta and Tripathi, 2018). Tight S-shaped folds and extensive warps in the foliation suggest several deformational episodes that significantly affect slope stability and rock mass kinematics.

3. MATERIALS AND METHODS

3.1 Instrumentation Network Design and Implementation

The methodological chart (shown in Figure 2) and the instrumentation details (presented in Table 1) were utilized in conducting the analyses for this study. Instrument sites and depths were established through comprehensive geological and geotechnical evaluations to ensure representative coverage possibly deforming and of stable zones (Figure 3). Depths were chosen to observe behavior in structurally significant regions, including overburden bedrock contacts, fractured and sheared zones. Data were frequently gathered and updated as needed to accommodate fluctuations in site activity and external influences such as precipitation or seismic events.

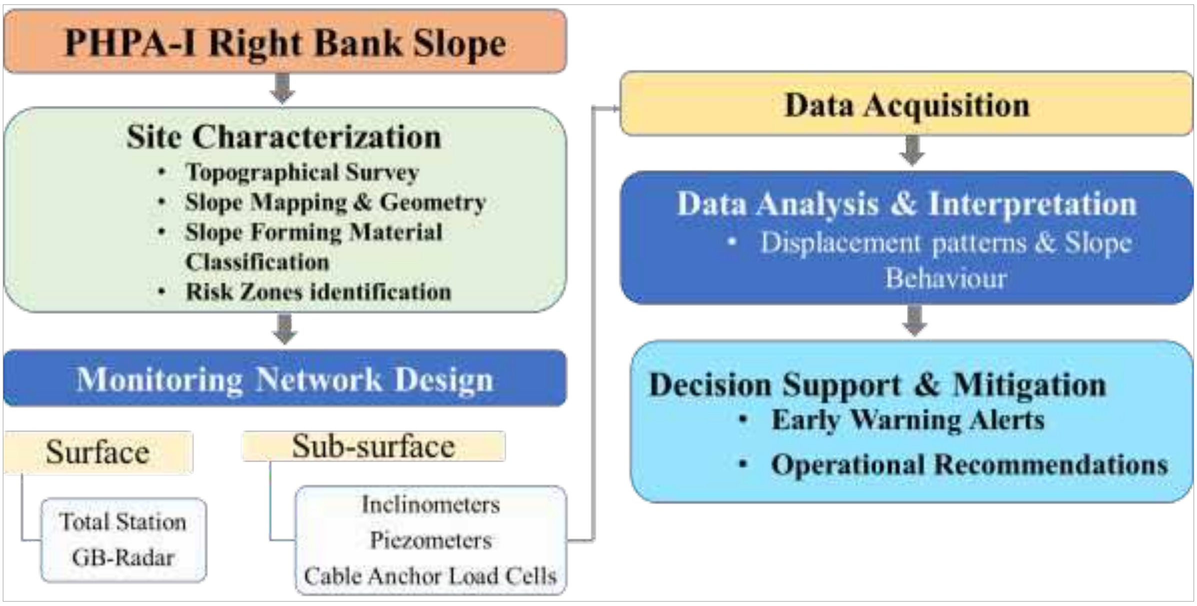

Fig. 2. Flowchart of the surface and subsurface monitoring framework implemented on the Right Bank Slope of PHPA-I.

Table 1 : Overview of instruments and their measurement characteristics.

Instrument	Measurement Parameter	Quantity	Measurement Interval
Total Station (survey)	Surface displacement through surface target points	50 surface target points	Periodic
GB Radar	Surface displacement	1	2 minutes
Inclinometer (borehole)	Subsurface lateral displacement	12	Periodic
Load Cell (anchor)	Stress in support/anchor	8	Periodic
Piezometer	Pore water pressure	5	Periodic

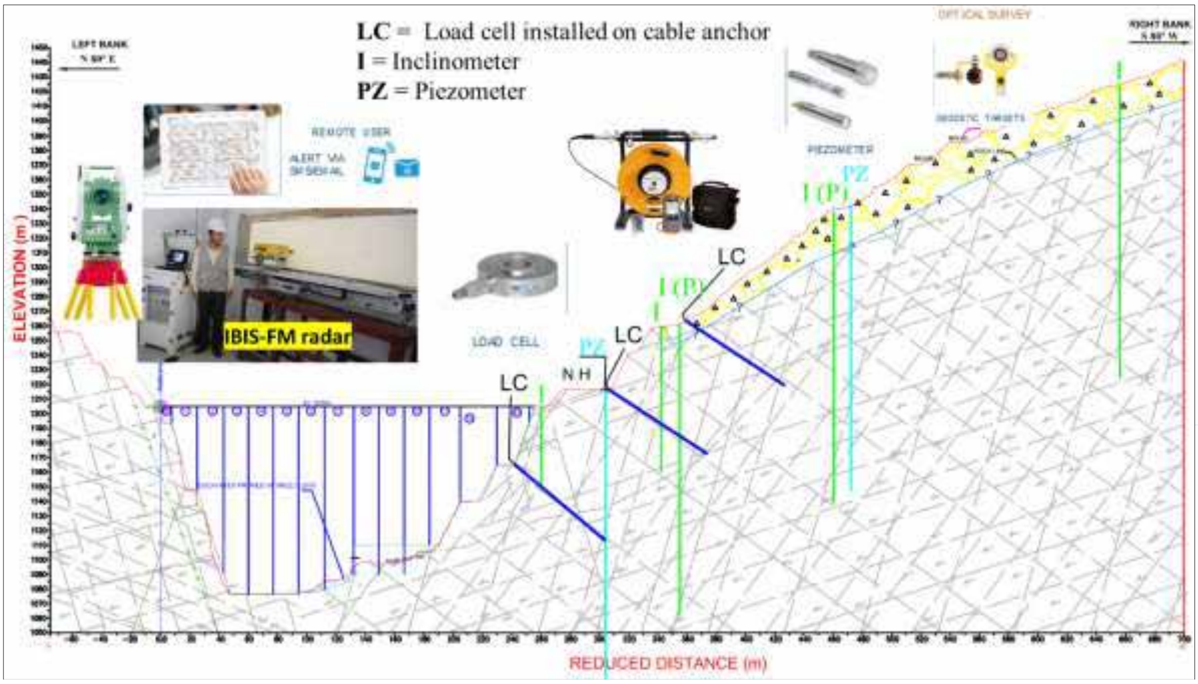

Fig. 3 : Typical cross-section of the right bank slope illustrating the approximate locations of surface and subsurface instruments installed for slope monitoring.

3.1.1 *Surface Monitoring*

Surface displacements were measured utilizing 50 Surface Target Points (STPs) surveyed with high precision total stations, supported by GB InSAR positioned on the opposing slope to ensure an unobstructed perspective (Figures 4 and 5). The GB InSAR system provided near-real-time displacement measurements along the line of sight (LoS) with sub-millimeter precision and a temporal resolution of 2 minute (Figure 5). Cumulative and differential displacements were calculated by radar interferometry, facilitating the early identification of progressive movement areas and general deformation patterns (Tarchi et al., 2003; Frodella et al., 2017; Su et al., 2022). Digital Terrain Models (DTMs) functioned as geospatial base layers for the integration of terrestrial radar data, providing the depiction of slope geometry with resultant displacement, velocity, and hazard maps, as well as associated time–displacement graphs. Hazard thresholds were designated by color: green (<0.30 mm/h), yellow (0.30–0.50 mm/h), and red (>0.50 mm/h), accompanied by automatic notifications through alarms in the monitored system, email, and SMS upon exceeding these limits.

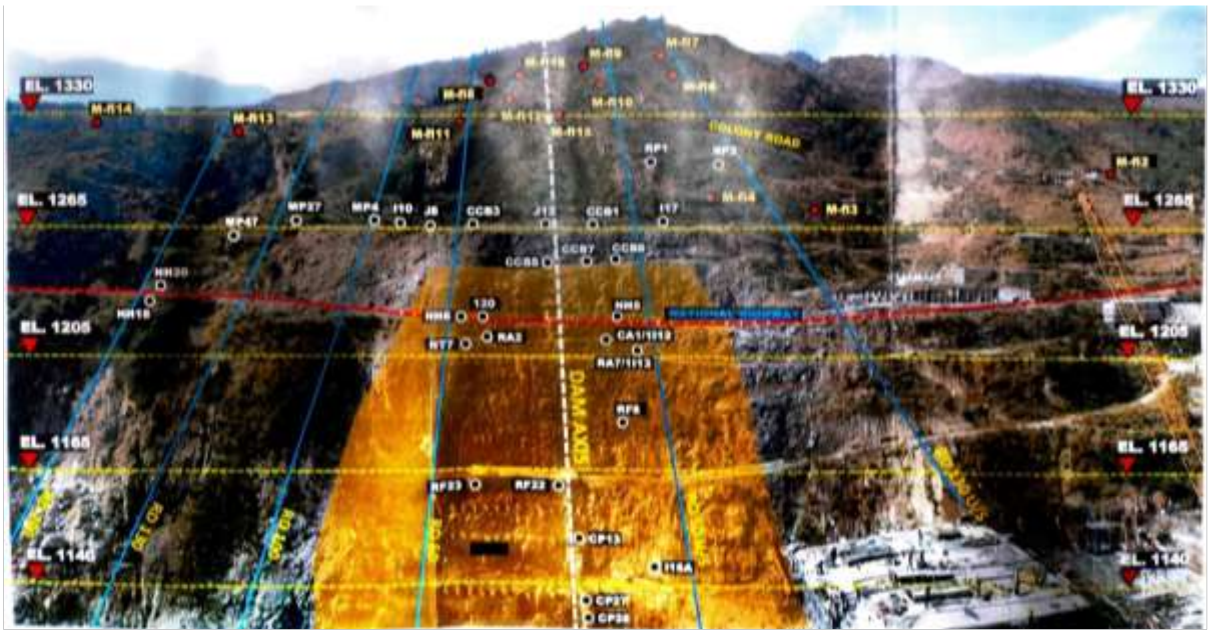

Fig. 4 : Spatial distribution of surface target points on Dam right bank.

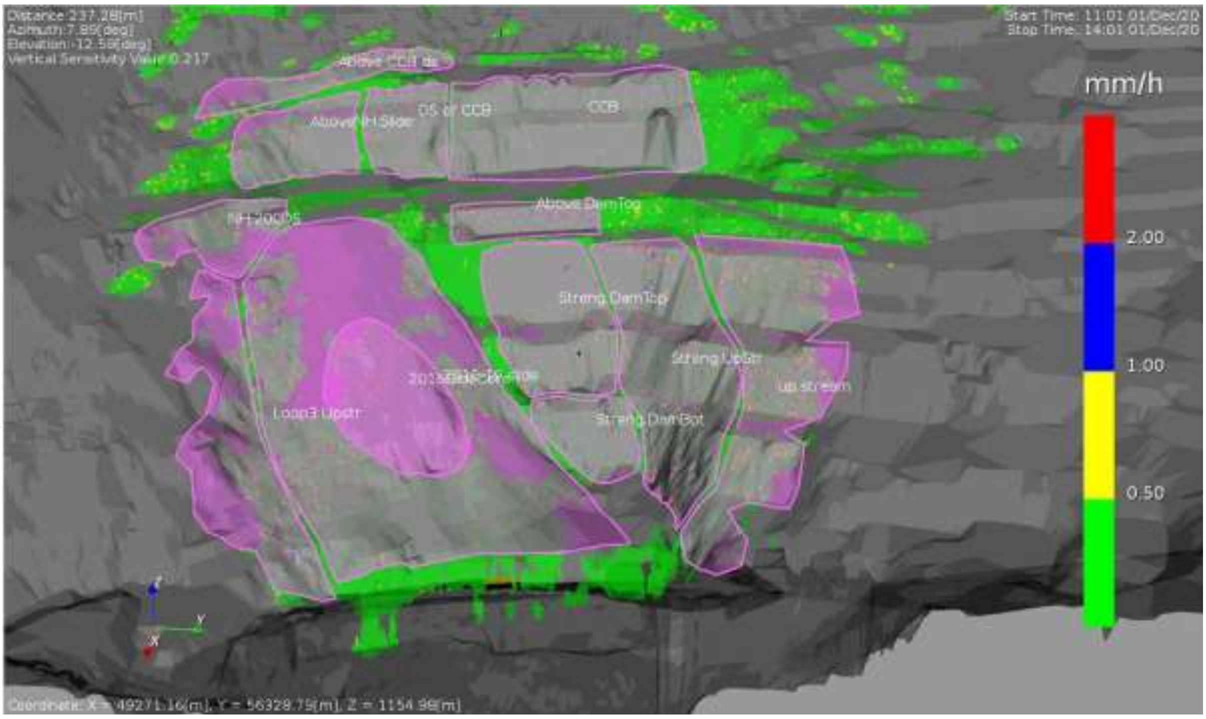

Fig. 5 : Radar image depicting the segmentation of Dam right bank slope into distinct zones for deformation assessment.

3.1.2 *Subsurface Monitoring*

Subsurface instrumentation included 12 inclinometers, 5 piezometers, and 8 cable anchor load cells (Figure 6). Inclinometers recorded lateral displacement profiles with depth, enabling identification of zones of active movement, creep, and relative stability. Piezometers measured pore water pressure fluctuations with ±0.5 kPa accuracy, providing insights into hydro-mechanical conditions influencing slope stability. Cable anchor load cells monitored stress redistribution within stabilization anchors, reflecting internal mechanical responses to environmental and construction induced loading (Liu et al., 2024). Measurements were performed weekly and after significant rainfall or seismic events to capture transient responses.

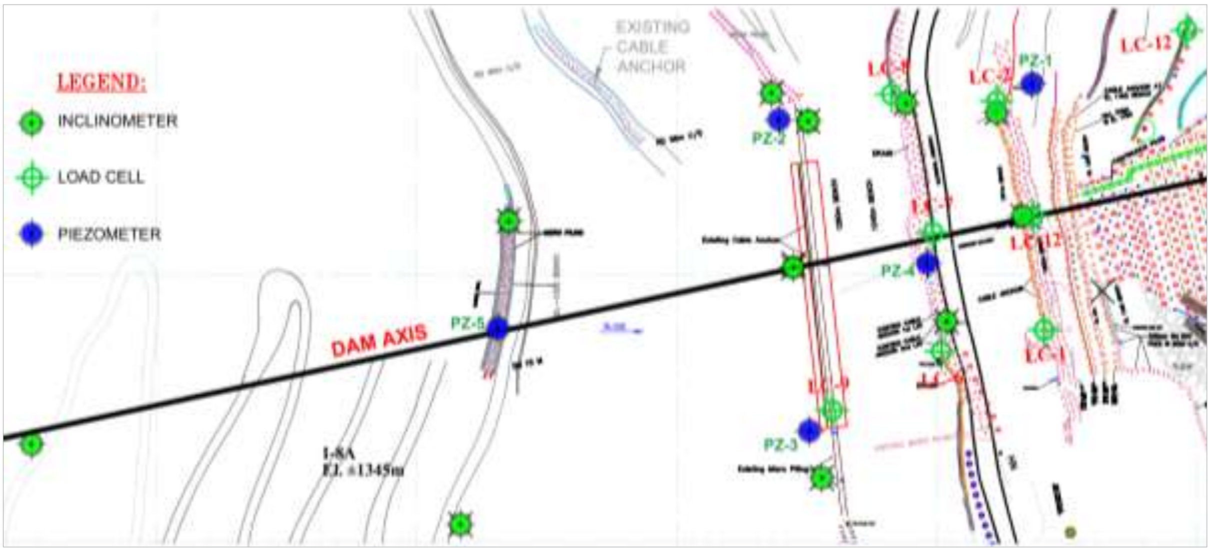

Fig. 6 : Layout showing the locations of inclinometers and piezometers installed on Dam Right Bank Slope.

3.1.3 *Data Acquisition and Processing*

All datasets were integrated to analyze slope behavior, with temporal trends in surface and subsurface measurements cross correlated to identify relationship between observable surface displacements, subsurface lateral movements, pore water pressures, and stress redistribution within anchors. This integrated approach enabled the sub-division of the slope into zones of active displacement, creep, and stability, supporting mechanistic interpretation, predictive modeling, and assessment of stabilization measures.

4. RESULTS AND DISCUSSION

The monitoring dataset for April 2025 was analyzed to understand the surface deformation behavior of the right bank slope at the PHPA-I dam. Ground-Based Radar observations showed that cumulative riverside displacements varied across the slope (Figure 7). The strengthen dam top slope area experienced minimal displacement of around 0.3 mm, while the strengthen dam bottom slope area recorded a displacement of about 1.0 mm. The strengthen upstream slope area exhibited displacement of approximately 0.9 mm, whereas the unreinforced upstream zone, which mainly consists of loose debris overburden and dumped material, showed the slightly more displacement of around 2.0 mm. These observations demonstrate the effectiveness of targeted reinforcement in reducing slope deformation, confirming previous studies that emphasize the importance of structural interventions in stabilizing vulnerable slope zones (Liu et al., 2024). Total station monitoring across 50 surface target points detected no measurable displacement during April 2025. The monitored areas included the strengthen slope, the cable car bench area, the region above the recessed National Highway and the downstream National Highway above Loop-3. There was no movement in any direction, whether riverside, downstream or vertical, indicating that the slope remained exceptionally stable during the observation period.

Inclinometers installed on the right bank slope revealed that displacements were primarily concentrated along the contacts between overburden and bedrock, as well as within fractured and sheared zones of the rock mass (Figure 8). These observations underscore the importance of subsurface monitoring for detecting deeper deformation processes that are not apparent from surface measurements alone (Xia et al., 2025). Piezometer readings from PZ-1 to PZ-5 showed no significant variation during April 2025. This indicates that groundwater conditions around the dam were stable and that the water table remained unchanged. The stability of pore pressures suggests that the slope was not experiencing hydro-mechanically driven deformation during the monitoring period. Load cells installed on cable anchors during April 2025 recorded minor variations in load across the slope. LC-5 and LC-7 exhibited slight increases of 0.20 Ton and 0.69 Ton, respectively, while LC-9 and LC-12 showed decreases of 9.10 Ton and 4.45 Ton, with LC-9 decreasing from 24.72 Ton to 15.55 Ton since 19 April, and LC-12 from 87.31t to 84.94t since 28 April. Inclinometers and piezometers in the vicinity did not indicate any corresponding slope displacement, suggesting that these changes were localized to the anchors. The observed reductions in load are likely related to minor stress redistribution within the slope mass, rather than any displacement in the slope itself. Field studies of pre-stressed anchors and cable-reinforced slopes have reported similar behavior, where load decreases can occur due to local stress adjustments, creep of overlying material, or minor anchor relaxation under stable slope conditions (Choi et al., 2013; Yang et al., 2023; Liu et al., 2024).

Comparison of surface target displacements with inclinometer displacements data at zero depth showed clear differences, demonstrating the importance of using multiple monitoring instruments. At I-17 (±53m U/S, Elevation 1260 m), the surface target recorded a cumulative displacement of 128.0 mm, while the inclinometer measured only 10.9 mm. At I-20 (±32m D/S, Elevation 1216 m), surface targets showed 127.0 mm of displacement compared with 58.8 mm recorded by the inclinometer. Monthly changes for both measurement methods were small, indicating that the slope remained stable during the monitoring period. Differences between surface and subsurface measurements suggest that surface targets are more responsive to near surface movements, whereas inclinometers capture deformation occurring deeper within the slope. These results highlight the value of integrating different monitoring techniques to gain a complete understanding of slope behavior, which is important for designing effective stabilization measures and early warning systems (Xia et al., 2025; Zhao et al., 2023; Su et al., 2022).

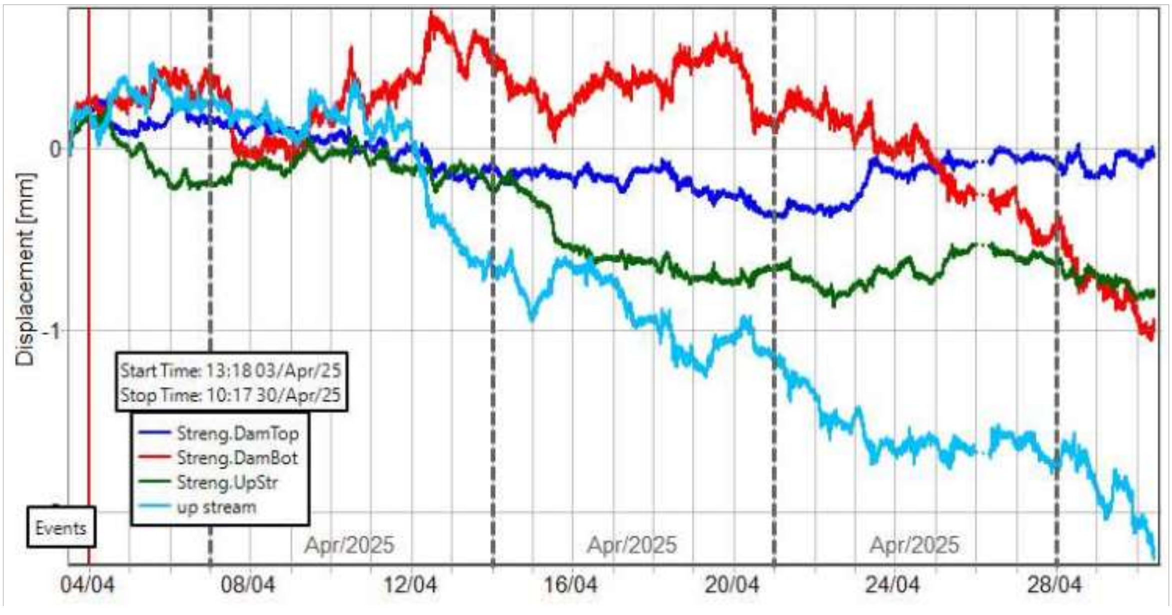

Fig. 7 : Cumulative riverside displacement over the month across different zones.

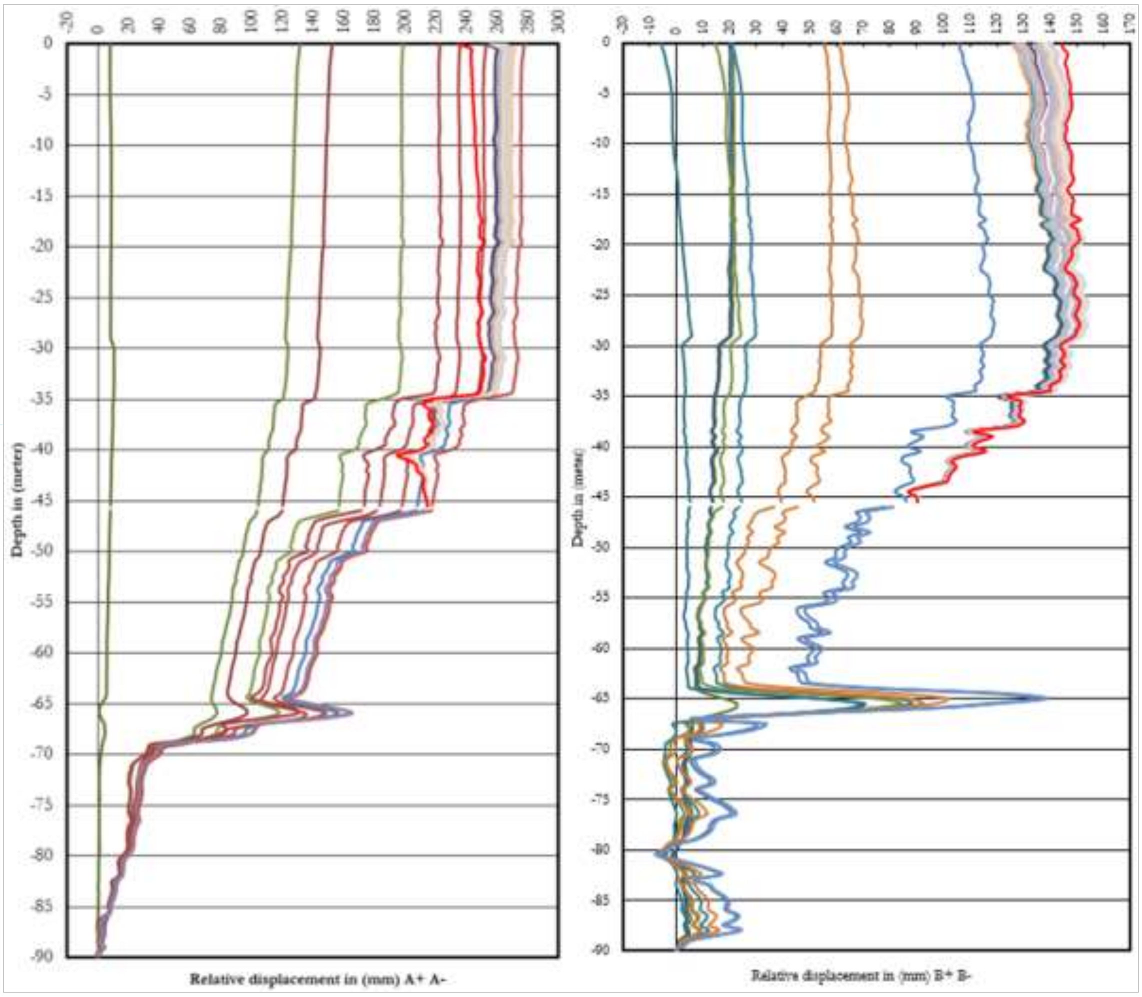

Fig. 8 : Inclinometer derived deformation profiles illustrating displacement patterns at different depth intervals.

CONCLUSION

Integrated surface and subsurface monitoring of the right bank slope at PHPA-I demonstrates the importance of using multiple instruments to accurately assess slope behavior in the complex geological environment of the Himalayas. At 0 m depth, Surface Target Points recorded significant near surface deformation, with displacements of 130.0 millimeters at location I-17 and 129.0 millimeters at location I-20. In contrast, inclinometer readings at the same depth indicated much smaller displacements of 6.55 millimeters at I-17 and 14.90 millimeters at I-20. Subsurface data from inclinometers, piezometers, and cable anchor load cells provide valuable insights into internal slope behavior.

Integration of all monitoring results enabled classification of the slope into active movement, creep, and stable zones. This classification offers direct guidance for the design and placement of stabilization measures such as anchors and piles. The findings highlight that high resolution, continuous monitoring is essential for reliable hazard assessment, timely early warning implementation, and effective long term slope management of large hydropower projects.

This integrated approach can be extended to other hydropower and critical infrastructure projects in Himalayan terrain, incorporating additional geotechnical instruments as necessitated by the specific site conditions.

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INNOVATIVE GEOPHYSICAL METHODS FOR PUMPED STORAGE PROJECTS

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ABSTRACT

Pumped storage projects (PSPs) have gained significant attention in India due to the country's increasing energy demands and the need for sustainable energy storage solutions. These projects play a pivotal role in balancing the grid, especially with the integration of renewable energy sources. However, the successful implementation of PSPs requires a comprehensive understanding of the subsurface, which is where geophysics comes into play. Geophysics offers a suite of non-invasive methods to investigate the subsurface conditions, crucial for the design and safety of PSPs. In the Indian context, where diverse geological formations exist, geophysical investigations become even more pertinent. These investigations help in identifying potential sites for PSPs, assessing the geological and hydrogeological conditions, and ensuring the structural integrity of the proposed infrastructure.

In India, for PSP investigations, seismic refraction and electrical resistivity imaging are two of the most commonly deployed geophysical methods. Seismic refraction offers insights into subsurface structures, enabling the identification of fault lines, fractures, and other geological intricacies that can influence the safety and efficacy of storage reservoirs. Concurrently, electrical resistivity imaging, an advanced form of the traditional electrical resistivity method, is instrumental in delineating the groundwater table, lineaments etc. It also pinpoints areas susceptible to seepage, a critical factor in the design and construction of reservoirs. The combined deliverables of these techniques underscore their significance in ensuring the robustness and reliability of PSPs in the region. MASW (Multichannel Analysis of Surface Waves) and GPR (Ground-Penetrating Radar) also have potential to be utilized for geophysical investigations of PSP projects. MASW aids in determining the shear wave velocity of the subsurface, providing insights into the ground's elastic properties and Vs30. On the other hand, GPR offers high-resolution imagery of the shallow subsurface, facilitating the identification of cavities, voids, and other anomalies that could jeopardize the project. Together, these methods furnish invaluable data about geological formations, playing a pivotal role in refining the design of the PSPs. Crosshole seismic tomography is another tool for PSP (Pumped Storage Projects) investigations. This advanced geophysical method involves generating seismic waves between boreholes to create detailed images of the subsurface. For PSPs, understanding the subsurface is paramount, as it directly impacts the safety and efficiency of the storage reservoirs. Crosshole seismic tomography provides high-resolution data on rock quality, fracture zones, and potential voids or anomalies. By offering a comprehensive view of the subsurface conditions, it aids in optimizing the design, construction, and operation of PSPs, ensuring their longevity and reliability. The geophysical investigations need to be complemented by geotechnical drilling and laboratory tests, ensuring a holistic understanding of the subsurface conditions. This integrated approach ensures that the design and construction *of PSPs are based on sound scientific principles, minimizing risks, and optimizing performance.*

This paper presents an in-depth analysis of the pivotal role geophysics holds in the investigations for pumped storage projects within the Indian context. Considering India's varied geological landscape and the growing focus on integrating renewable energy sources, the significance of geophysical investigations is underscored. The paper delves into how these investigations are instrumental in guaranteeing the safety and optimal performance of PSPs. Furthermore, to provide a practical perspective, the paper will encompass several case studies, highlighting their contribution to India's journey towards a sustainable energy future.

***Keywords**: Geophysical Methods, Pumped Hydro Power, Subsurface Characterization, Renewable Energy, Geological Assessment, Energy Storage*

INTRODUCTION

The increasing demand for renewable energy sources has intensified interest in pumped hydro power (PHP) systems, which serve as a crucial component in energy storage and grid stability. PHP systems utilize two reservoirs at different elevations to store energy by pumping water to the upper reservoir during periods of low demand and releasing it to generate electricity during peak demand. The feasibility and safety of these systems heavily depend on a thorough understanding of subsurface geological and hydrological conditions. Geophysical methods have emerged as essential tools for this purpose, providing non-invasive techniques to characterize subsurface properties and identify suitable sites for hydroelectric facilities (Ivanov et al., 2009; Mihevc & Stepišnik, 2012; Rehman et al., 2016).

Recent advancements in geophysical techniques, such as seismic refraction tomography, electrical resistivity imaging, and multichannel analysis of surface waves (MASW), have significantly enhanced the ability to assess geological formations and hydrological conditions. For instance, seismic refraction tomography is effective in delineating the velocity structure of subsurface layers, which is critical for understanding site stability and suitability (Bery, 2013). Similarly, electrical resistivity imaging allows for the mapping of resistivity variations that correlate with material properties, aiding in the identification of groundwater aquifers (Mihevc & Stepišnik, 2012). The integration of these methods improves the accuracy of subsurface assessments and supports the optimization of PHP systems, ensuring their reliability and efficiency in energy storage and generation (Ivanov et al., 2009; Bery, 2013). The integration of geophysical methods into the planning and execution of pumped hydro power projects not only enhances the feasibility of these systems but also supports the broader goals of renewable energy development. As the global energy landscape continues to evolve, the role of pumped hydro storage in facilitating the transition to a sustainable energy future will only become more pronounced. By employing advanced geophysical techniques, stakeholders can ensure that pumped hydro projects are designed and implemented with a comprehensive understanding of subsurface conditions, ultimately leading to more reliable and efficient energy solutions (Rehman et al., 2016; Bery, 2013).

SEISMIC REFRACTION TOMOGRAPHY

Seismic refraction tomography (SRT) is a sophisticated geophysical technique that enhances traditional seismic refraction methods by providing detailed subsurface imaging through the analysis of seismic wave travel times. This method is particularly effective in characterizing the velocity structure of geological layers, which is crucial for various applications, including environmental assessments and engineering site investigations (Ivanov et al., 2009). By deploying multiple geophones and seismic sources, SRT captures the propagation of seismic waves through different materials, allowing for the construction of two-dimensional (2D) and three-dimensional (3D) models of subsurface conditions. Recent advancements in SRT have improved its resolution and accuracy, making it possible to identify complex geological features that may not be detectable using conventional methods. The technique has been successfully applied in various contexts, such as landslide monitoring, where it helps to delineate the spatial heterogeneities in elastic properties of materials (Whiteley et al., 2020). Additionally, SRT has proven valuable in environmental studies, where it aids in identifying subsurface anomalies, such as voids or cavities, by correlating low seismic velocities with high resistivity zones. Overall, the ability of seismic refraction tomography to provide high-resolution images of subsurface structures makes it an essential tool in the planning and assessment of infrastructure projects, including pumped hydro power facilities.

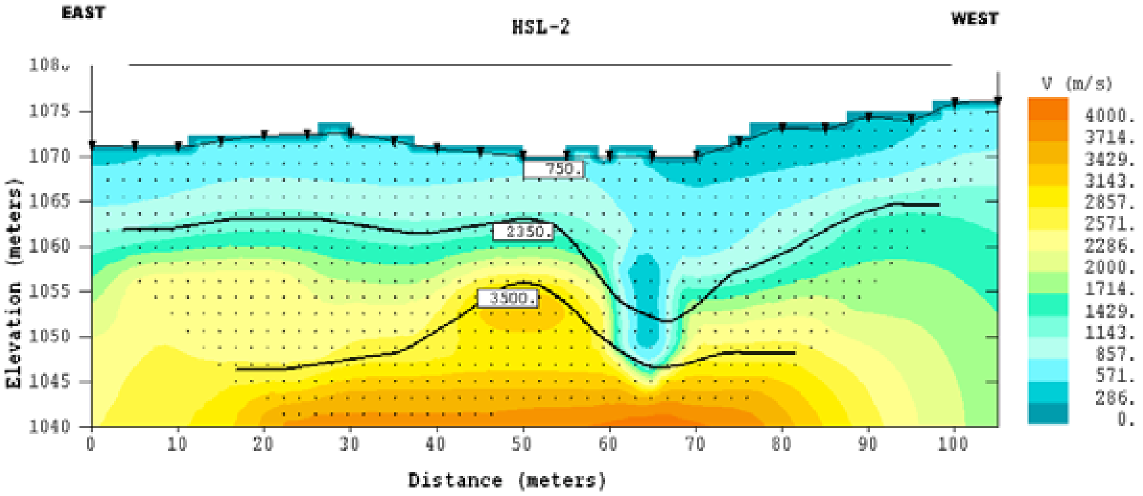

Fig. 1 : Seismic Refraction Tomography result- Ability to map lateral velocity contrasts and delineation of small shear zone.

ELECTRICAL RESISTIVITY IMAGING

Electrical resistivity imaging (ERI) is a geophysical technique that has gained prominence for its ability to provide detailed insights into subsurface structures and materials. This method involves the measurement of the electrical resistivity of the ground, which varies based on the geological composition, moisture content, and porosity of the materials present (Mihevc & Stepišnik, 2012). By deploying a series of electrodes in a systematic array, ERI allows for the generation of two-dimensional (2D) and three-dimensional (3D) images of subsurface resistivity distributions. The data collected can be processed using advanced inversion algorithms to create high-resolution models that reveal the spatial arrangement of different geological layers (Kemna et al., 2002). Recent studies have demonstrated the effectiveness of ERI in various applications, including the characterization of earth dams, where it has been used to identify potential seepage paths and assess the integrity of the structure. Furthermore, ERI has proven invaluable in hydrogeological investigations, as it correlates resistivity values with hydrogeological properties such as porosity and permeability, thereby aiding in the identification of groundwater aquifers. The development of computer-controlled multi-electrode systems and sophisticated resistivity modelling software has facilitated more cost-effective and efficient resistivity surveys, making ERI a valuable tool for subsurface investigations across diverse geological settings. Overall, the non-invasive nature, rapid data acquisition, and ability to visualize subsurface conditions make electrical resistivity imaging an essential method in the planning and assessment of pumped hydro power projects and other geotechnical applications.

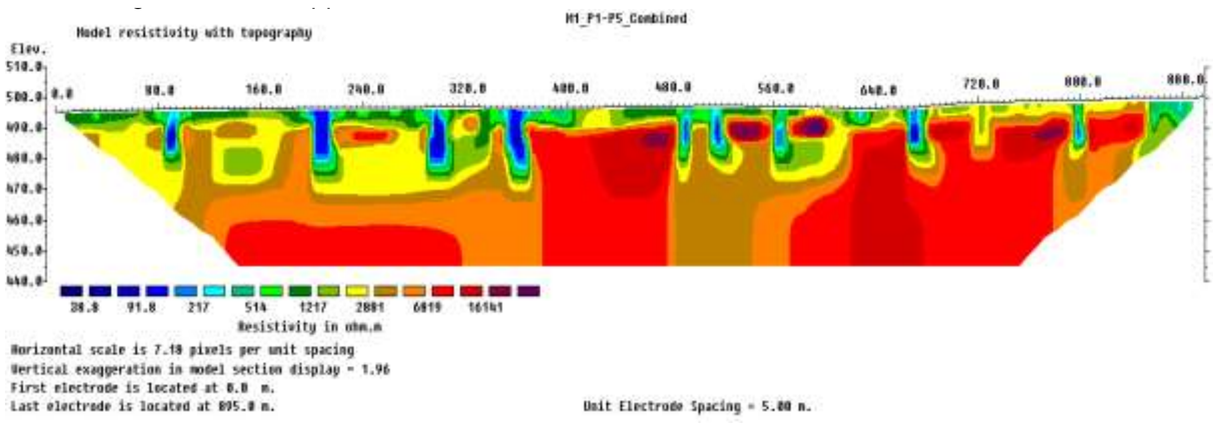

Fig. 2 : Mapping of Lineaments along Reservoir Periphery using Electrical Resistivity Imaging.

MULTI-CHANNEL ANALYSIS OF SURFACE WAVES (MASW)

The Multichannel Analysis of Surface Waves (MASW) method is a geophysical technique that utilizes surface wave propagation to assess subsurface shear wave velocities. This method has gained popularity due to its ability to provide high-resolution images of the near-surface geological structure, making it particularly valuable for geotechnical investigations, including those related to pumped hydro power projects. MASW operates by generating surface waves through impulsive sources, such as a sledgehammer or weight drop, and recording the resulting wavefield with an array of geophones. The recorded data is then processed to extract dispersion curves, which relate wave velocity to frequency. This allows for the inversion of shear wave velocity profiles, providing insights into material properties and stratigraphy (Ivanov et al., 2009). Recent studies have demonstrated the effectiveness of MASW in various applications, including the characterization of soft clays and peat, where it has been shown to accurately delineate subsurface layers and their mechanical properties (Zainorabidin, 2023). Furthermore, MASW has been successfully applied in assessing the dynamic behaviour of soils and sediments, which is critical for evaluating site stability and suitability for infrastructure development (Park et al., 1999). The method's non-invasive nature, combined with its capacity to produce detailed subsurface models, positions MASW as a key tool in the planning and execution of pumped hydro power projects, where understanding the geological context is essential for ensuring safety and operational efficiency.

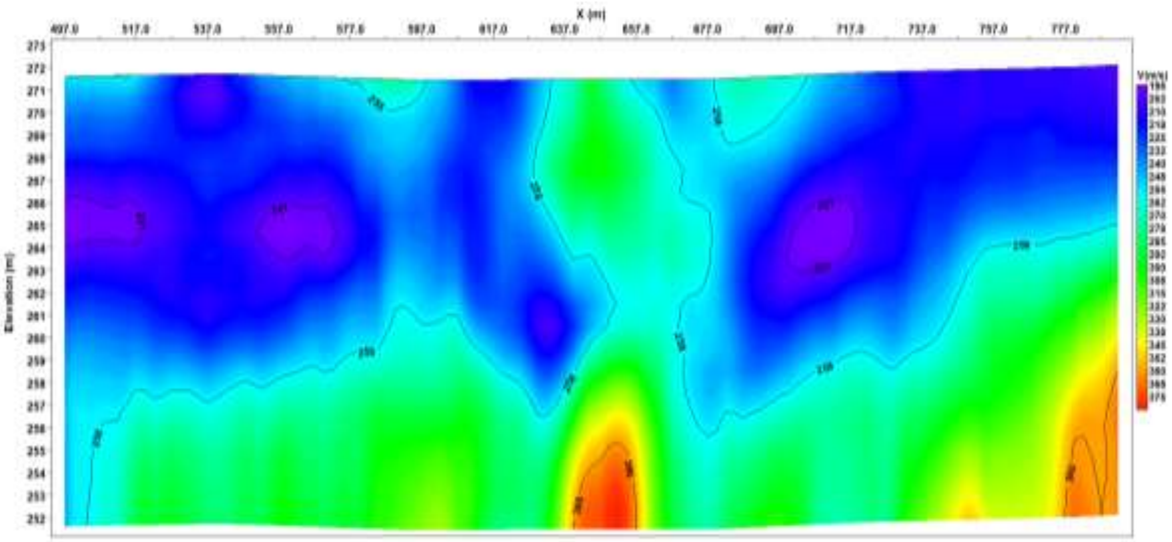

Fig. 3 : Results of MASW showing low velocity anomalous zones in an earthen dam

GROUND PENETRATING RADAR

Ground Penetrating Radar (GPR) is a non-invasive geophysical technique that utilizes high-frequency electromagnetic waves to obtain detailed images of the subsurface. This method is particularly effective for detecting and mapping buried structures, cavities, and geological features at shallow depths. GPR operates by emitting radar pulses into the ground and recording the reflected signals from subsurface materials, allowing for the identification of variations in material properties based on differences in dielectric permittivity. The high sensitivity of GPR to the presence of groundwater makes it a valuable tool in hydrogeological studies, where it is used to investigate near-surface aquifers and assess groundwater levels. Recent advancements in GPR technology have improved its resolution and efficiency, enabling rapid data acquisition and continuous imaging of subsurface conditions. As a result, GPR has found widespread applications in various fields, including civil engineering, archaeology, and environmental assessments, making it an essential method for subsurface investigations in the context of pumped hydro power projects.

CROSS-HOLE & CROSS FACE SEISMIC TOMOGRAPHY

Seismic tomography is a sophisticated geophysical technique that involves measuring the travel times of seismic waves between two or more boreholes to create detailed images of subsurface velocity structures. This method is particularly effective for characterizing geological heterogeneities at high resolutions, making it invaluable in various applications, including geotechnical investigations, groundwater studies, and environmental assessments (Holliger & Levander, 1992). By deploying seismic sources in one borehole and receivers in adjacent boreholes, cross-hole seismic tomography captures the propagation of seismic waves through the subsurface, allowing for the reconstruction of velocity profiles that reflect the elastic properties of the geological formations. Recent studies have demonstrated the utility of this method in assessing soil improvement for construction projects, as it provides critical insights into the effectiveness of ground treatment techniques (Ehosioke & Fechner, 2014). Additionally, cross-hole seismic tomography has been successfully applied in the characterization of heterogeneous aquifers, where it aids in understanding the spatial distribution of hydraulic properties essential for effective water resource management. The ability to obtain high-resolution images of subsurface conditions makes cross-hole seismic tomography a powerful tool for ensuring the safety and reliability of infrastructure projects, including pumped hydro power facilities, where understanding geological stability is paramount.

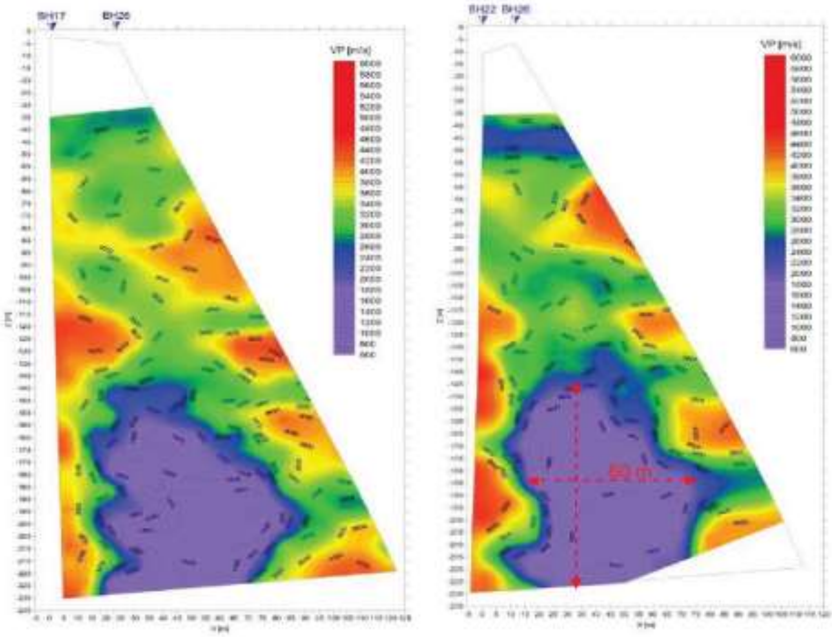

Fig. 4 : Mapping of a deep cavity in a hydro power project

Fig. 5 : Seismic tomography between two drifts of a hydro power project

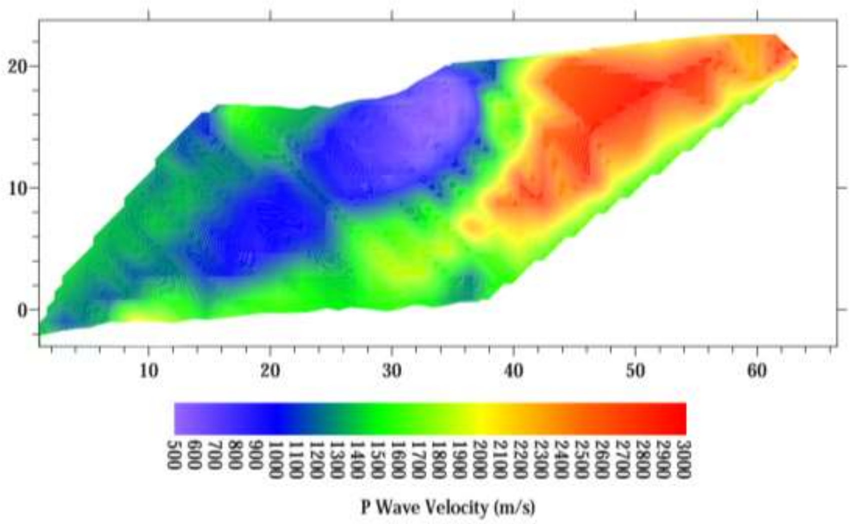

Fig. 6 : Results of seismic tomography between two drifts of a hydro power project

CONCLUSIONS

The integration of geophysical methods into the investigation of pumped hydro power projects is essential for ensuring the feasibility, safety, and efficiency of these renewable energy systems. This paper has highlighted the significant advancements in various geophysical techniques, including seismic refraction tomography, electrical resistivity imaging, multichannel analysis of surface waves (MASW), ground penetrating radar (GPR), and cross-hole seismic tomography. Each of these methods offers unique advantages for subsurface characterization, enabling engineers and geoscientists to assess geological formations and hydrological conditions critical for the design and operation of hydroelectric facilities. The non-invasive nature of these techniques allows for rapid data acquisition and detailed imaging of subsurface structures, which is crucial for informed decision-making in site selection and risk management. As the demand for sustainable energy solutions continues to grow, the role of geophysics in optimizing pumped hydro power projects will become increasingly important. Future research should focus on the continued development and integration of these geophysical methods, enhancing their applicability in diverse geological settings and contributing to the advancement of renewable energy technologies. By leveraging the capabilities of geophysical techniques, stakeholders can ensure that pumped hydro power systems are designed and implemented with a comprehensive understanding of subsurface conditions, ultimately leading to more reliable and efficient energy solutions.

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A CASE ILLUSTRATION OF THE PRIMARY INJECTION TESTING AS AN IMPORTANT AND EFFECTIVE TOOL FOR THE CONDITION ASSESSMENT OF POWER TRANSFORMERS

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ABSTRACT

Druk Green Power Corporation Limited (DGPC) is enhancing the performance of its power plants by implementing effective electrical asset management practices supported by advanced electrical testing. One of the vital electrical assets is power transformers, which have a pivotal role in power plants and are the key components of the power system and associated transmission networks. Their failure anytime can have a significant economic impact on the power plants, owing to long procurement lead times, manufacturing, installation and high equipment cost. The useful life of power transformers, if extended, altogether serves a single most important strategy for increasing the life of power transmission and distribution infrastructures, starting with generator step-up transformers (GTs), at the power plant itself. The required condition assessment of power transformers, if taken up periodically & timely, plays a vital role in analysing their risk of failure. Primary injection test is one amongst various electrical test techniques available that serves as a very effective tool in the area of electrical condition assessment of power transformers.

The paper, therefore, covers the electrical test diagnosis, internal inspection & rectification after a fault in a 3-Phase, 20MVA Generator Transformer to illustrate the importance of the primary injection test as an effective tool for condition assessment of power transformers. It was undertaken for the Electrical Maintenance Unit of 60MW Kurichhu Hydropower Plant, in Bhutan.

***Keywords**: Generator Transformer, Turn Ratio, Magnetizing Current, Winding Resistance, Magnetic Balance Test*

1. INTRODUCTION

DGPC's contribution to the nation by way of its hydro-generation earnings is a major source of revenues to the exchequer. In the fiscal year 2024-2025, DGPC is the highest contributor of Corporate Income Tax (CIT), contributing Nu. 3,841 million. Bhutan's CIT collection reached Nu. 14,757.82 million, reflecting a 27.46% rise from Nu. 11,578.65 million in the previous year. CIT made up 23.72% of the nation's domestic revenue, with the growth primarily fueled by increased contributions from hydropower plants and projects [1]. The revenue generated by the hydropower plants in Bhutan, therefore, constitutes a significant share of the country's economy. Downtime of any one hydropower plant's generating units, especially during the peak generating monsoon season, would therefore mean a huge impact on the country's revenue.

One of the important electrical components in hydropower plants is the GT, which actually steps up the generating voltage to a higher level, enabling the generated power to be transmitted at High Voltage (HV). The downtime of GT in a power station would mean disconnecting the associated generating unit from the grid, directly impacting the revenue. Therefore, conducting a condition assessment on the GTs on a regular basis is very important. The primary injection test is one of the most progressive electrical diagnostic techniques, conducted on the GTs to assess the internal health/

condition of the transformer. The ratio measurement, DC winding resistance measurement and magnetic/core balance test were taken up on the Unit-2 GT at Kurichhu Hydropower Plant (KHP) after the abnormal observation with the performance of the particular GT. Therefore, the GT was carefully examined with concern, in an anticipation of the impact it would have on the revenue if it broke down during peak generation.

2. PROBLEM STATEMENT

KHP has four numbers of 16.67 MVA synchronous generators, connected to its transmission network via individual 11kV/132 kV, 3-Phase, step-up GT units of 20MVA rating each. One of these GTs connected to the Unit-2 generator performed abnormally during its operation in 2018, for which the necessary diagnosis and rectification were called for. The abnormality, in a way, was vividly revealed as an internal one by way of a specialized preliminary test such as Dissolved Gas analysis (DGA) conducted on its insulation oil. The DGA test & analyses of the oil conducted indicated the definite presence of an internal arcing fault. It was further recommended for the fault localization, if any, through necessary electrical tests, its internal inspection and required rectification.

Therefore, the task involving tests, diagnosis, internal inspection and rectification of the particular GT was taken up by Centre of Excellence for Automation, Control & Protection (CoEACaP), Hydropower Research and Development Centre (HRDC), having mandates to provide such specialized electrical testing services, with the support from KHP.

3. TECHNICAL APPROACH

CoEACaP, based on the findings and recommendation, undertook to coordinate with KHP and satisfactorily complete the assignment. The approach to undertake the said task had the following overall objectives, but not limited to:

- Defining and outlining an overall approach and work plan to take up and resolve the task
- Performing the specialized transformer tests and professional interpretation of test results
- Diagnosing and locating of the faults, if any, indicated during the tests
- Internal inspection, fault rectification and transformer restoration

An in-depth assessment & engineering analysis of the actual problem, as referred from KHP, was made and drew a corresponding approach to resolve the task. A suitable approach was identified as applicable to detecting internal fault in a transformer along with engineering analyses and interpretation. The related industry best practices and standards. IEC 60076-1-2011 [2], IEEE Std. 62-1995 [3], and US Bureau of Reclamation document on Transformers – 2005 [4] were some of the many documents from which the reference was taken in determining the correct scientific approach to the problem.

The basic electrical investigation test and analysis techniques were utilized as applicable for a GT to resolve this task. The testing skill already possessed by CoEACaP in terms of human capacity, together with the state-of-the-art test equipment available, were utilized. The work plan, in sequence, first consisted of performing electrical and dielectric oil tests on the GT. It was followed by a plan to correlate any abnormalities indicated by electrical tests with the dielectric oil test findings, so as to confirm the problem. An electrical fault of any sort inside a transformer is equally confirmed by the tests performed on its dielectric medium (oil). Thereafter, the necessary recommendations were provided for internal inspection, upon confirming the internal fault. The approach in the final phase highlighted for execution of internal inspection, rectification and restoration of the GT.

4. IDENTIFIED INVESTIGATIVE TESTS

The transformer turn ratio test is a method of measuring the ratio of primary voltage to secondary voltage on the transformer. This test will assess the internal condition of the transformer, especially the shorted windings and mechanical deformation [5]. The magnetising current test will assess the transformers in terms of poor electrical connections, inter-turn short circuits, abnormal core faults and winding problems. The exciting currents are expected to be similar for the outer phases and slightly lower in the middle phase [6]. This ratio measurement and magnetising current test have been very effective to assess the internal conditions of the transformer; however, these test results have to be confirmed through other complementary diagnostic techniques. Therefore, to further validate the test results of the ratio measurement and magnetising current measurement, the other complementary tests, such as winding resistance measurement for the HV and Low Voltage (LV) sides and the magnetic core balance test, are also conducted.

The measurement of the resistance of the windings of the transformer is conducted mainly to check for any open winding turns and a loose joint, or a poor winding connection. Therefore, it helps in the inspection of the quality and healthiness of the windings. The magnetic balance test is performed on the three-phase transformers to assess the uniform distribution of the flux in the core. Therefore, it can check the imbalance in the magnetic circuit, any defects in the core and any inter-turn faults in the transformer [7].

As a part of the investigation plan, the following investigative electrical primary injection tests, along with the diagnostic tests on its solid and liquid insulation, were identified as essential for preliminary location of the fault in the GT.

- Turns Ratio and Magnetising Current Tests
- HV/LV Winding DC Resistance Test
- Magnetic or Core Balance Test
- Insulation Resistance (IR) and Polarisation Index (PI) tests
- Tan-Delta and Capacitance test

The insulation diagnostic tests, such as tan delta and capacitance test, and the IR measurement, have shown the truthful values indicating the insulation of the transformers is still intact and healthy. Therefore, these tests are not considered in this paper because of the above-cited reasons.

5. TEST EQUIPMENT

Figure 1 shows a multifunctional primary injection kit CPC 100TM from Omicron, Austria, that was utilized in performing the identified primary injection tests and inferring the corresponding results. The kit, with its multifunctional features, allows for automated testing of various power system primary equipment, including instrument transformers. It has an inbuilt capability to inject, measure and generate results in compliance with any standard industry best practice and international standards. The kit's capability to inject or measure AC voltages up to 2000V and AC currents up to 2000A is utilized in performing the measurements and analyses.

Fig. 1 : Multifunction Primary Injection Kit - Omicron's CPC100

Additionally, Doble's M4100 Tan Delta & Capacitance test kit, along with Simadzu's GC2030TOGAS1 Gas Chromatograph System, were used for performing the diagnostic tests on the transformer's solid and liquid insulations, respectively.

6. TEST RESULTS AND ANALYSES

Turns ratio and magnetising current tests, HV/LV winding resistance test, and magnetic or core balance test are the electrical investigative tests performed on the transformer prior to confirming the actual problem and taking up the transformer for internal inspection. The corresponding analyses of the results from each test are reported underneath each table. The results from the electrical tan delta and capacitance test, however, are not reported in the paper as they were found within limits for all the winding and bushings and therefore were not utilized in the investigation of the problem reported in the paper.

6.1 Turn Ratio and Magnetizing Current Measurement

The Table 1 shows the results for the turn ratio measurement and magnetizing current measurement at each tap position. The 300 V AC was injected at each tap of the HV side, and the corresponding induced voltage at LV side is measured. All the necessary details, such as vector group and the transformation ratio, were input in the test kit, CPC 100, and the ratio error and the magnetizing currents were measured and recorded.

Table 1 : Turns Ratio & Magnetizing Current Measurement Per Tap

Tap No.	Ratio Error Measured (%)			Magnetizing Current Measured (mA)		
	U	V	W	U	V	W
1	0.09	0.11	5269.20	3.5800	2.8750	0.6510
2	0.12	0.14	0.18	3.7570	2.9480	3.5140
3 (Nom)	0.15	0.17	5720.36	3.9180	3.1120	0.4760
4	0.18	0.20	0.24	4.0630	3.2320	3.8430
5	0.22	0.24	0.28	4.2620	3.3610	4.0110

The ratio measurements indicated the percentage ratio errors for all tap positions in three phases to be within the allowable IEC 60076-1 or IEEE Std. 62-1995 limits of ± 0.5 % with respect to the nominal ratio specified on their name plates. The ratio error readings, however, for the 1W phase tap no. 1 and 3 could not be correctly obtained, as highlighted in red, probably owing to their improper tap contact.

At the same time, the corresponding magnetizing currents at each tap position, compared between the phases, at all taps are found to be comparable. However, the same in the case of W phase could not be obtained for tap no. 1 and 3 for the probable reasons cited above.

6.2 DC Winding Resistance Measurement

The Table 2 shows the measurement of winding resistance of all phases of both HV and LV sides. The method employed here is the Voltmeter – Ammeter method, with 0.1 A DC injected and measuring the corresponding voltage across the HV windings. While 5 A DC is injected in the LV windings. The winding resistance measured is then corrected to the resistance value at 75°C.

Table 2 : HV/LV Winding DC Resistance Measurement

Tap No.	DC Resistance Measured & referred to 75 °C					
	HV Winding (Ω)			LV Winding (mΩ)		
	1U	1V	1W	2u-2v	2u-2w	2v-2w
1	2.5341	3.5804	10.464	25.23	24.99	25.40
2	3.6777	4.0419	3.6029			
3 (Nom)	3.7105	4.0018	10.448			
4	3.7129	3.9990	3.1883			
5	3.7852	4.0089	3.1106			

The HV side DC winding resistances measured and referred to 75°C at each tap position, compared among the phases, were found to be comparable, even among the taps, except for 1W phases (tap no. 1 and 3), which were measured high and abnormal and are reflected in red.

The LV side DC winding resistances measured and referred to 75 °C, compared among the three phases, were found within the allowable IEEE Std 62-1995 tolerance limit of ± 5.0 %.

6.3 Magnetic/Core Balance Measurement

The magnetic balance test is conducted to identify the defects in the cores and windings. This measurement requires the injection of a low voltage of 50 Hz at one phase and measuring the induced voltage at the other two phases. If there are no defects in the windings and core, then the injected voltage at one phase should be equal to the summation of the voltages of the two other phases [4]. The Table 3 shows the summary of the test results of the magnetic/core balance measurement.

Table 3 : Magnetic/Core Balance Measurement

Applied AC Voltages on HV Side (Volts)			Measured AC Voltages on HV & LV Sides (Volts)					
1U-1N	1V-1N	1W-1N	1U-1N	1V-1N	1W-1N	2u-2w	2v-2u	2w-2v
236.80	-	-	236.80	181.90	16.50	7.80	25.90	33.40
-	237.30	-	148.30	237.30	7.50	21.10	33.50	12.40
-	-	237.10	0.80	0.60	237.10	0.20	0.20	0.30

The transformer was observed not to be satisfying the required magnetic/core balance voltage equality criteria, both with respect to HV & LV windings, especially in relation to its W Phase, as highlighted in red. The equality criteria unfulfilled are as follows:

- (1U-1N) ≠ (1V-1N) + (1W-1N) & (2u-2w) ≠ (2v-2u) + (2w-2v)
- (1V-1N) ≠ (1U-1N) + (1W-1N) & (2v-2u) = (2u-2w) + (2w-2v)
- (1W-1N) ≠ (1U-1N) + (1V-1N) & (2w-2v) ≠ (2u-2w) + (2v-2u)

The voltage equality upon confirmation with measurement at another healthy tap (tap # 2) was observed OK

6.4 Overall Result Analysis

The overall test results indicated abnormal turns ratio, magnetizing current, winding dc resistances and magnetic balance for W Phase HV winding at tap positions 1 and 3, the latter being the GT's nominal tap. The results for other two phases were observed within standard limits. It was therefore inferred and reported for the likely existence of improper tap contacts at position 1 and 3 of 1W phase. This was resulting to continuous electrical arching fault in the GT and thus the abnormal performance of the GT. The GT was eventually to be undertaken for internal inspection of its tap contacts, prior to putting back to operation.

7. INTERNAL INSPECTION, FINDINGS & RECTIFICATION

Considering the confirmation of the faulty transformer tap contacts, specifically tap no. 1 & 3 deduced from the electrical investigative tests and the resulting recommendation for internal inspection, the transformer tank was drained out and opened up for inspection. The firsthand visual inspection of the tank's chamber internally indicated presence of electrical arcing that had been persisting, which was evident from the huge deposits of carbon dusts found all around the inside of the transformer tank. Upon closer inspection of the probable anticipated problem with the taps 1 & 3, indicated defective female tap contacts and pitting of the male tap contacts, that had actually resulted in the improper contact and corresponding arcing fault and internal carbonization. As shown in the Figure 2, Figure 3and Figure 4, these are illustrative of some of the physical observations and findings made from the thorough internal inspection of the transformer.

Fig. 2 : Thick Carbon Deposits Observed on the Transformer Winding Assemblies

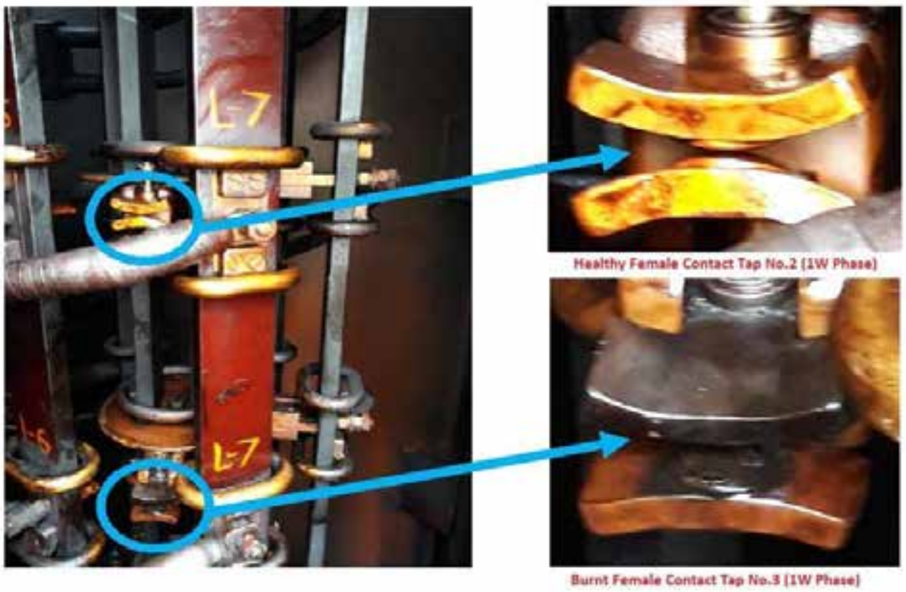

Fig. 3 : Tap Changer Female Contacts (Tap No. 2 & 3 - 1W Phase)

Fig. 4 : Pitted Male Tap Changer Contact (Tap No. 3 - 1W Phase)

7.1 Findings & Rectification

As implied by the electrical test results the transformer's tap changer's contact 1 & 3 of 1W phase were found damaged, resulting into improper contact of the taps, especially the nominal tap no.3. The improper tap contact at nominal tap no.3 was resulting into continual arcing and carbon deposits inside the transformer tank.

The faulty tap contacts 1 & 3 of 1W Phase were removed and replaced with new contacts. The transformer was boxed up, oil-filled with new oil and readied for testing and restoration.

8. POST-RECTIFICATION TEST FINDINGS

8.1 Turn Ratio and Magnetizing Current Measurement

The Table 4 gives a summary of the test results of the turn ratio and magnetizing current measurement obtained for the post-rectification of the tap contacts.

Table 4 : Turns Ratio & Magnetizing Current Measurement Post-rectification of Taps 1 & 3

Tap No.	Ratio Error Measured (%)			Magnetizing Current Measured (mA)		
	U	V	W	U	V	W
1	0.19	0.19	0.26	6.186	4.577	6.618
2	0.21	0.21	0.28	6.422	4.786	6.854
3 (Nom)	0.24	0.24	0.30	6.715	4.991	7.029
4	0.27	0.28	0.34	7.018	5.190	7.356
5	0.30	0.31	0.38	7.222	5.441	7.748

The turn ratio measurements indicated the percentage ratio errors for all tap positions in all three phases to be within the allowable IEC 60076-1-2011 or IEEE Std. 62-1995 limits of ± 0.5 % with respect to the nominal ratio specified in their name plates.

Moreover, the magnetizing current at each tap position, compared among the phases, at all taps was found to be comparable & within acceptable limits.

8.2 DC Winding Resistance Measurement

The Table 5 gives a summary of the test results of the DC winding resistance measurement obtained for the post-rectification of the tap contacts.

Table 5 : HV/LV Winding DC Resistance Measurement Post-rectification of Taps 1 & 3

Tap No.	DC Resistance Measured & referred to 75 °C					
	HV Winding (Ω)			LV Winding (mΩ)		
	1U	1V	1W	2u-2v	2u-2w	2v-2w
1	2.0142	2.0527	2.0495	25.20	25.19	25.22
2	1.9941	1.9868	1.9823			
3 (Nom)	1.9493	1.9357	1.9358			
4	1.8915	1.8860	1.8857			
5	1.8401	1.8360	1.8380			

The HV sides DC winding resistances measured and referred to 75°C at each tap position, compared among the phases/ taps, were all found comparable, and within the allowable IEEE Std. 62-1995 tolerance limit of ± 5.0 %.

Plus, the LV side DC winding resistances measured and referred to 75°C, compared among the three phases were found within the allowable IEEE Std. 62-1995 tolerance limit of ± 5.0 %.

8.3 Magnetic/Core Balance Measurement

The Table 6 gives a summary of the test results of the magnetic/core balance measurement for the post-rectification of the tap contacts.

Table 6 : Magnetic/Core Balance Measurement Post-rectification of Taps 1 &3

Applied AC Voltages on HV Side (Volts)			Measured AC Voltages on HV & LV Sides (Volts)					
1U-1N	1V-1N	1W-1N	1U-1N	1V-1N	1W-1N	2u-2w	2v-2u	2w-2v
242.80	-	-	**242.80**	204.00	38.87	**34.43**	29.06	5.47
-	242.60	-	128.80	**242.60**	112.80	18.42	**34.57**	16.11
-	-	242.90	40.80	200.90	**242.90**	5.84	28.70	**34.54**

The transformer was also observed to be satisfying the required magnetic/core balance voltage equality criteria, both with respect to HV & LV windings, for all the phases. The equality criteria fulfilled are as follows:

- (1U-1N) = (1V-1N) + (1W-1N) & (2u-2w) = (2v-2u) + (2w-2v)
- (1V-1N) = (1U-1N) + (1W-1N) & (2v-2u) = (2u-2w) + (2w-2v)
- (1W-1N) = (1U-1N) + (1V-1N) & (2w-2v) = (2u-2w) + (2v-2u)

The transformer, based on the post-rectification test results, was declared healthy for restoration and put back to normal operation.

9 CONCLUSION

The task of investigating the abnormal performance, identifying the cause, its rectification and eventual restoration of 20 MVA GT at KHP indicated the importance of the former electrical tests. The approach towards investigating the root cause by way of electrical and dielectric investigative tests resulted in a very positive outcome, which in turn helped in significantly narrowing down the search for the actual fault location during the internal inspection. The fault finding and restoration of the GT was achieved without having to involve any OEM expertise, thereby saving in terms of cost. The overall objective of rectifying and restoring the GT was fully accomplished. CoEACaP's role with all the electrical test and analysis skills possessed, contributed to the overall success of the task. Above everything, the case clearly illustrated the overall importance of the electrical primary injection tests as an effective tool for the condition assessment of power transformers.

10 RECOMMENDATION

The paper by way of the task undertaken to test, inspect, rectify and restore KHP's Unit-2 GT, precisely established the importance & effectiveness of the electrical primary injection tests on power transformers. It is therefore in view of this fact that the DGPC plants are recommended to give due importance to the value of primary injection testing and consider periodically & timely assessing the overall condition of the transformers across DGPC formations.

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HIGH-PRECISION SOLUTIONS FOR LEVEL, POSITION, AND FLOW MEASUREMENT

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ABSTRACT

Innovative solutions from Rittmeyer AG in the field of Flow, level and position measurement system is helping hydro power plants to run with better efficiency including Dam safety.

We would like to introduce Rittmeyer group as Swiss-based smart utility infrastructure and services company, involved in Hydro power and PSP in flow, level and position measurement. Since its establishment in 1904, Rittmeyer has put more than 20000 installations into operation and is represented worldwide with subsidiaries, sales offices, and a distribution network in over 25 countries, With more than 120 years of experience, technical expertise, passion, know-how, and skills we provide among other solutions ultrasonic flow measurement (turbine discharge measurement), with our intelligent applications such as sediment monitoring, turbine efficiency measurement, and penstock leak detection system. We are Leader in Flow, Level and Position Measurement.

In India we have more than 100 installations, our instruments play an important role for hydro and PSP projects for measuring efficiency of the power plant as well as safety of power plant.

Rittmeyer Ag innovative instruments provide flow measurement systems with intelligent applications like sediment monitoring, turbine overall efficiency and penstock leak detection system.

Our new generation of controller are specially designed for Hydro power plants. The instruments have all the cyber security features which help power plant operators to improve their plant efficiency by optimization and continuous monitoring. Rittmeyer level and position measurement system works for Dam safety by monitoring and feedback system. Our level measurement system is designed with advanced features which can work for dam safety at the time of high flood level.

For hydro power hydrology is the most important factor. We measure hydrology for plant optimization and plant safety. Rittmeyer Flow measurement system measures flow data with high precision. Since the flow data can be utilized for plant efficiency and decision making. Measurement of flow with high precision with durability is required. Every Hydro power plant is tailor made project because of hydrology unique geology of the project, accordingly rittmeyer have wide variety of sensors and measurement techniques to measure the flow for the different power plant.

FLOW MEASUREMENT

Integrated solutions from a single source Efficient and sustainable use of water resources is more important than ever before – now and for the generations to come.

With more than 100 years of expertise in high-end instrumentation solutions for demanding applications, Rittmeyer has contributed to a sustainable environment for generations and is an acknowledged partner in the worldwide industry for efficient, sustainable and economic management of water and energy resources. Rittmeyer is a leading provider of advanced and high-end solutions and is offering a state-of-the-art instrumentation portfolio alongside proven engineering, installation, commissioning and after-sales services. Rittmeyer flow measurement systems are extremely precise and durable also under though conditions. Their versatility allows for a broad range of smart applications. Flow measurement in pipes, tunnels, and channels (from head to tail race) for large- down to micro-scale hydropower plants including pumped storage and run-of-river.

Flow measurement enables efficient, ecological and sustainable management of water resources and increases safety for people and environment– topics ever more important on the agenda of humanity.

ULTRASONIC TRANSIT TIME METHOD

This method requires at least one pair of ultrasonic transducers to form an ultrasonic path which measures the flow velocity of water and calculates the flow and volume information. The average water flow velocity along the path is measured by emitting and receiving ultrasonic pulses, in both forward and reverse direction of the flow. Using more than one ultrasonic path substantially enhances the system accuracy.

Different kind of sensors are invented for hydro power plant like intrusive , inside sensors for the penstock which is fully buried under the concrete, clamp on sensors for the old penstock line, and special sensors for flow measurement in tunnel.

Various advanced applications are combined into one Rittmeyer solution. Thanks to that, hardware and instrumentation components can be reduced to a minimum – for maximum efficiency and cost savings.

ULTRASONIC TRANSIT TIME METHOD

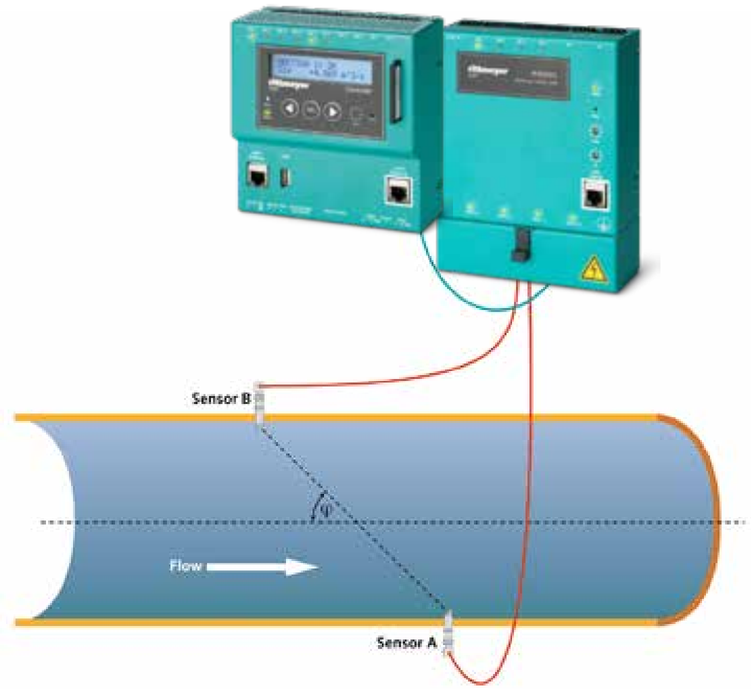
This method requires at least one pair of ultrasonic transducers to form an ultrasonic path which measures the flow velocity of water and calculates the flow and volume information. The average water flow velocity along the path is measured by emitting and receiving ultrasonic pulses, in both forward and reverse direction of the flow. Using more than one ultrasonic path substantially enhances the system accuracy.

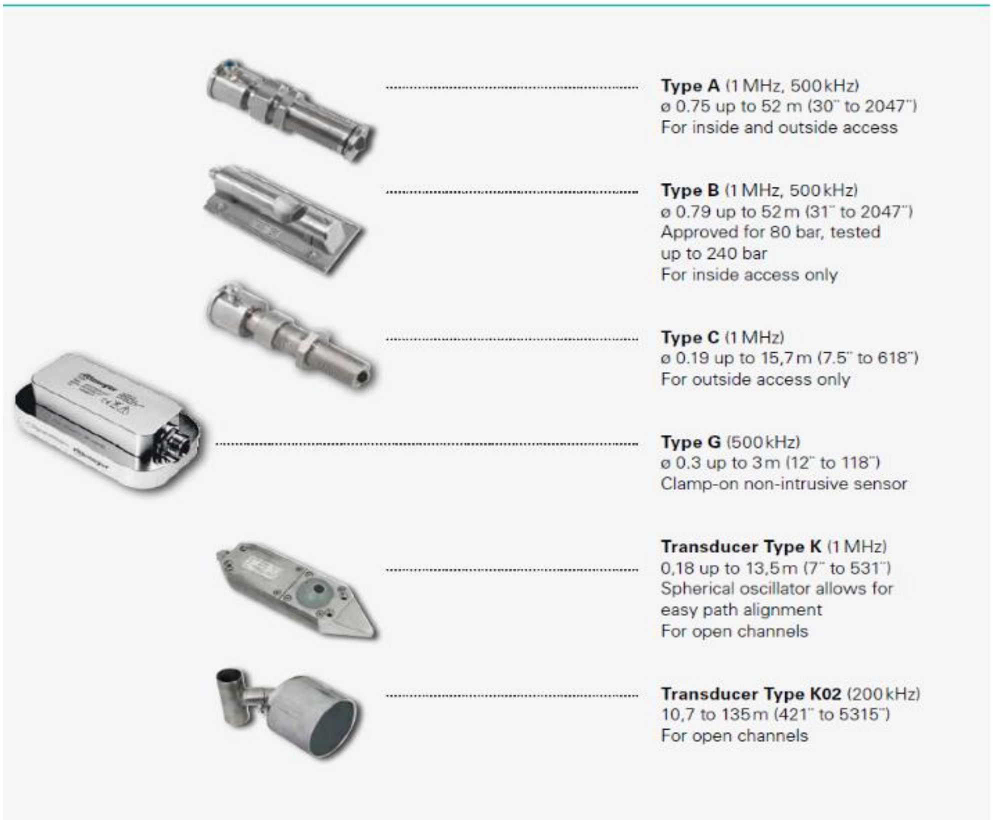
Two types of transducers are available:

- Intrusive transducers that are in direct contact with water
- Non-intrusive clamp-on transducers that are not in contact with water.

Benefits

- Wide range of pipe diameters of up to 52 meters
- Wide range of open channel widths of up to 130 meters
- High accuracy of up to 0.5 % in the field and 0.2 % under ideal conditions
- High flexibility for configurations such as crossed/non-crossed setups, up to 20 paths per measurement section, different plane angles, etc.
- Drift-free and long-term stability
- Easy to retrofit into existing, even third-party installations
- No moving parts





Transducers for Pipes

Type A (1 MHz, 500 kHz)
Ø 0.75 up to 52 m (30" to 2047")
For **inside and outside** access
External mount

Type C (1 MHz)
Ø 0.19 up to 15,7 m (7.5" to 618")
For **outside** access
External mount

Type B (1 MHz, 500 kHz)
Ø 0.79 to 52 m (31" to 2047")
For **inside** access only
Internal mount

Type G05, G1 and G02
(500 kHz, 1 MHz, 200 kHz)
Ø 0.3 to 6 m (12" to 236")
Clamp-on non-intrusive sensor
External mount

Intrusive Type – drilling / cable penetration on penstock

High Accuracy
Installation process is delicate
Longer lifespan

Non- intrusive Type – No drilling / cable penetration on penstock

Lower costs
Lower accuracy
Simpler installation process
Lifespan

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Flow Metering Capabilities

RISONIC *modular* is designed to measure:

- Flow measurement according IEC 60041 / ASME PTC 18
- Filled pipes
- Partially filled pipes
- Open channels

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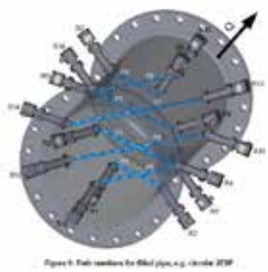
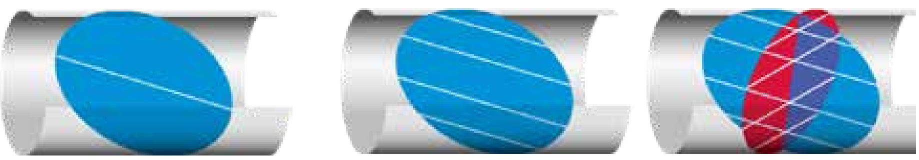
Multi-path Arrangements





1E1P 1E4P 2E8P

TRANSIT TIME FLOW MEASUREMENT ADVANTAGES:

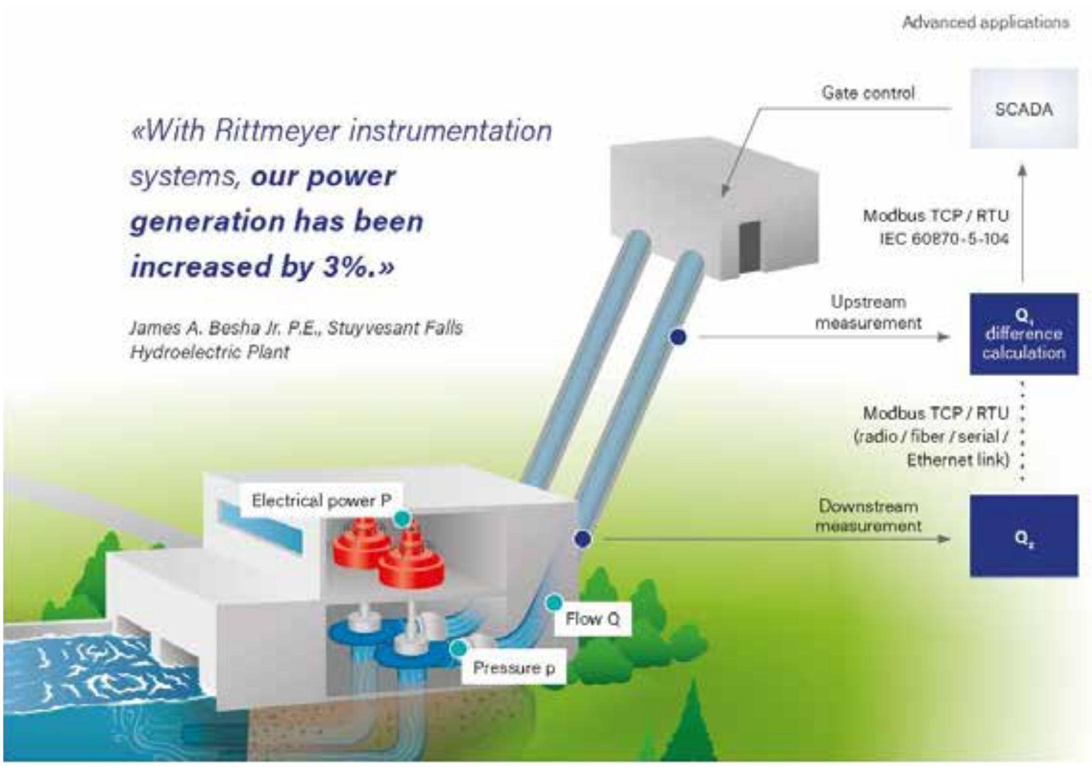
- Flow meters are "dry calibrated" based on precise as-built measurements taken at the time of transducer installation
- No recalibration is required
- Wide range of pipe diameters, up to 52 meters
- High accuracy class can be reached, up to 0.5 % in field applications and 0.2 % in laboratory environment
- Transit time method is accepted by IEC 60041 and ASME PTC 18 for turbine performance test – preferred method by major EPC of Hydroelectric power.

Rittmeyer combines 4 smart applications in 1 device:

- Flow measurement
- Turbine efficiency monitoring
- Penstock leak detection
- Sediment monitoring

TURBINE EFFICIENCY MONITORING

With the Rittmeyer turbine efficiency monitoring application, power plant operators can reliably determine temporal changes in turbine efficiencies by using high-precision ultrasonic transit time flow and pressure measurements. Predefined processing rules assure simplified evaluations. Thus, damages on turbine parts can be detected at an early stage. This reduces down-time as well as expensive and time-consuming replacement of turbine parts to a minimum.



Benefits:

- Maximizes productivity due to real-time efficiency information
- Excellent return on investment on the installed flow measurement system
- Preventive maintenance due to early-stage information about potential turbine damages
- Easy implementation due to predefined intelligent processing rules
- Monitoring of efficiency changes over time and creation of informed maintenance and CapEx plans
- Management of turbine life cycles
- Optimizes the refurbishment sequence to replace the least efficient units first
- Fine tuning of Kaplan and Pelton turbines which have additional variables and more opportunity for efficiency improvement
- Analyzes penstock losses to develop an optimized penstock replacement CapEx plan

PENSTOCK LEAK DETECTION

The unique concept of the Rittmeyer penstock leak detection system (PLDS) allows for early detection of leaks, thus maximizing public and environmental safety. With high-precision flow measurements at both ends of the penstock, leaks and ruptures of the penstock can be quickly identified. As an alternative to the ultrasonic measurement at the lower end of the penstock, the flow can also be measured using the Winter-Kennedy method at the turbine inlet.

Besides ruptures and larger leaks, the PLDS ensures that smallest leaks are reliably detected by accumulating the difference between the upstream and downstream flow over a longer period of time. Thresholds, time delays and corresponding actions are easily configured to match the specific requirements on site. With the integration of additional transducers, even more precise monitoring of critical locations is possible.

The entire control and processing unit can be fit into a compact wall-mounted cabinet and can operate fully independent of other plant process control systems.

The PLDS application can also be combined with the sediment monitoring application in parallel on the same control unit. The system can also be used for leak detection in open channels, i.e. for flood protection.

Benefits

- Minimizes potential damage through safe and early leak detection
- Taking account of public safety responsibility
- Meets regulatory safety requirements
- Easy to integrate into existing plant equipment
- Bidirectional monitoring possible (pump and turbine operation)
- Easy retrofit with

SEDIMENT MONITORING

When it comes to plant operation, maximum productivity comes first. Suspended solids and sediments in the water can lead to damages on installed equipment. Even the smallest impairment can severely decrease turbine efficiency. It is imperative to detect damages at an early stage to prevent high costs from expensive power generation outages and time-consuming replacements of damaged turbine parts. The amount of suspended particles in the water can be monitored with Rittmeyer ultrasonic measurements. Based on the signal attenuation, the sediment concentration is derived. In addition to filled pipe systems, sediments can also be monitored in open channels.

Benefits

- Protects plant and equipment against erosion
- Reduces wear and tear on turbines and pumps
- Reduces maintenance costs
- Maximizes turbine efficiency
- Assists to optimize maintenance intervals

PATH ARRANGEMENTS

Rittmeyer supports all path arrangements as per IEC 60041 / ASME PTC 18. Several scenarios are possible, from the measurement of one pipe with up to 20 paths, to the Measurement of four different pipes or pipe sections with five paths per measuring point. The more paths a measurement comprises, the higher the accuracy will be.

Monitoring sediment is important for the management of water resources, too. Sediment monitoring data can be used to determine effectiveness of sediment re-duction actions in the watershed and guide adaptive sediment management. Research at the Swiss Federal Institute of Technology in Zurich (ETH Zürich) and the University of Lucerne have shown that the ultrasound pulses can also be used to monitor the amount of suspended particles in the water. This feature has many technical, economical and ecological importance for the operation of a hydropower plant as it can reduce turbine scuffing.

Cost-efficient monitoring

When it comes to hydro power plant operation, maximum productivity and assured revenues come first. Therefore, it is essential to detect potential reasons for equipment damage at an early stage. This protects operators from the high costs due to power generation outage and the time-consuming and expensive replacement of damaged turbine parts. Sediments floating in the water are one of these potential risks for water turbines. They can scuff turbine parts and ultimately lead to a complete outage in the worst case.

In addition to closed pipe systems, this feature can also be used for sediment management programs for open channels such as canals and rivers.

DAM SAFETY

Rittmeyer smart (level measurement devices) can measure the dam level with the high accuracy. Radial gates position can measure using RIPOS & REVERT, During High Dam level Gates can be opened automatically. Radial Gates Position can be monitored in SCADA. Gates operation can be done automatically during very high level (can be customized as per site requirements). Level sensors of the MPx series are robust and durable. These advanced piezoresistive pressure transmitters deliver a variety of measurements, in addition to the Rittmeyer measurement systems. Any kind of data collection sensors, such as ultrasonic or radar sensors, can be connected to repress smart for gate control.

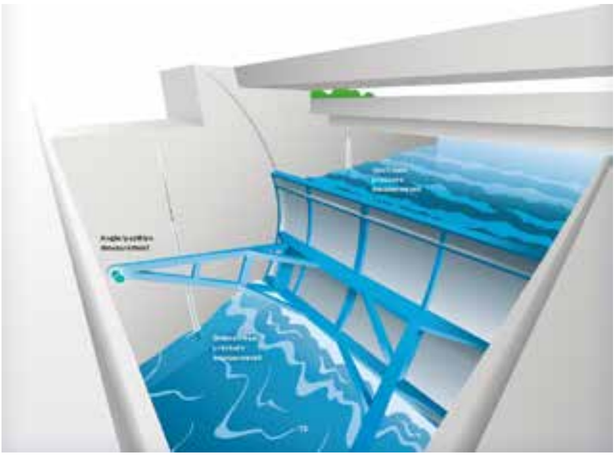
With the intelligent Ripos Smart and Revert Smart measurement systems from Rittmeyer, a reliable automatic weir control can be guaranteed. Furthermore, these robust systems also include comprehensive processing functions and a number of communication interfaces. In combination with a process control system, the optimal water level of a reservoir can be calculated and managed. Based on a Level Measurement and the measured position of the weir, its opening and closing can be independently controlled to maintain an optimal water level. Versatile, low maintenance, and cost-effective: RIPRESS smart can measure level using level sensors of the MPx series. These advanced piezoresistive pressure transmitters deliver a variety of measurements. The controller with galvanically isolated inputs and outputs, integrated web server can control the radial gates during high dam level. Gate position can be measured with RIPOS AND REVERT.

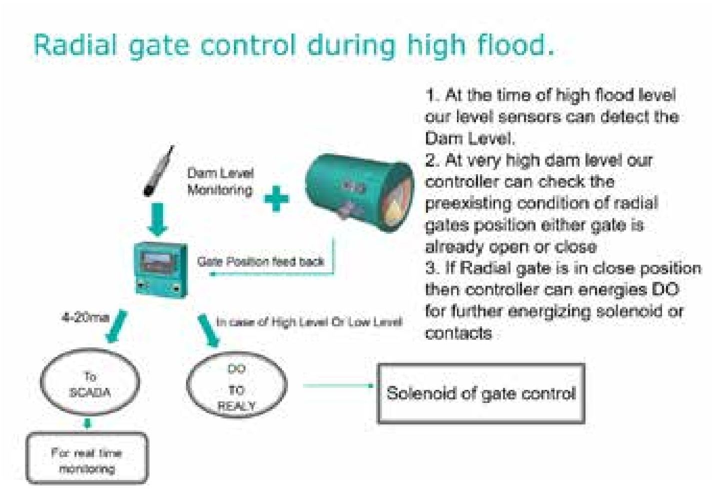
Ripress smart with Ripos or revert can open the radial gates during high dam level(abnormal condition) Ripress premium having DO which can further energies the relay and trigger the Gates solenoid valves(to open the gates) Ripress Smart have AO which can provide 4 -20 ma to customer SCADA for indication and control Ripress Smart can open the gate independently on high DAM level during abnormal rise in dam level. At the time of high flood level our level sensors can detect the Dam Level. At very high dam level our controller can check the preexisting condition of radial gates position either gate is already open or close. If Radial gate is in close position then controller can energies DO for further energizing solenoid or contacts.

- With the intelligent Ripos Smart and Revert Smart measurement systems from Rittmeyer, a reliable automatic weir control can be guaranteed. Furthermore, these robust systems also include comprehensive processing functions and a number of communication interfaces. In combination with a process control system, the optimal water level of a reservoir can be calculated and managed. Based on a Level Measurement and the measured position of the weir, its opening and closing can be independently controlled to maintain an optimal water level.

Radial gate control during high flood.

- Versatile, low maintenance, and cost-effective: RIPRESS smart can measure level using level sensors of the MPx series. These advanced piezoresistive pressure transmitters deliver a variety of measurements. The controller with galvanically isolated inputs and outputs, integrated web server can control the radial gates during high dam level. Gate position can be measured with RIPOS AND REVERT.
- Ripress smart with Ripos or revert can open the radial gates during high dam level(abnormal condition)
- Ripress premium having DO which can further energies the relay and trigger the Gates solenoid valves (to open the gates)
- Ripress Smart have AO which can provide 4 -20 ma to customer SCADA for indication and control
- Ripress Smart can open the gate independently on high DAM level during abnormal rise in dam level.





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I would like to thank Dr. Roger Wimmer who provided his expertise for all his support and information on the latest innovation in the field of floe and level measurement system.

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SMALL HYDROPOWER, BIG IMPACT: DESIGN INNOVATIONS IN A HIMALAYAN ROR PROJECT

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ABSTRACT

This paper presents a series of innovative and conceptual design modifications introduced into a small run-of-river hydropower development located in the Himalayas. The proposed scheme comprises a shallow diversion weir, a desander, a long water transfer tunnel, and a surface powerhouse situated downstream.

Following an extensive site visit - encompassing both upstream and downstream project locations, examination of core samples, and inspection of a neighboring hydropower tunnel under construction - key insights were gained into local geological conditions, rock formations and construction challenges. Drawing on these observations, and in collaboration with engineers from the Nepal Electricity Authority (NEA), the authors proposed several design refinements to the tunnel alignment, vertical shaft configuration, and weir structure.

These changes, rooted in practical design and construction experience, and informed by typical local unit (construction) rates, are expected to significantly reduce the overall project cost. Additionally, the revised tunnel and shaft design may help mitigate risks such as tunnel squeeze and optimize shaft support requirements.

The paper highlights how small-scale design innovations, founded in site-specific understanding and collaborative engineering, can yield substantial benefits in cost efficiency and constructability. The lessons learned are intended to offer valuable guidance for similar hydropower projects across the Himalayan region.

Keywords (and Concepts)*: Value Engineering, Innovation, Design and Construction Experience, Constructability*

1. INTRODUCTION

1.1 Small Hydropower in the Himalayas – The Innovation Beneficiary

Small-scale hydropower (SHP), with its inherent flexibility to adapt to local conditions and its suitability for deployment in remote rural areas, remains central to global development strategies (UNIDP 2022).

In the Himalayan region, SHP is particularly attractive due to favorable topography and relatively lower labor costs. These projects often benefit from reduced logistical burdens - especially in terms of transport and hoisting - compared to larger hydropower schemes. Moreover, SHP developments typically entail smaller environmental and social footprints. Importantly, even minor design adjustments or changes in construction methodology can significantly influence overall project costs, potentially enhancing project viability and attractiveness.

Run-of-river SHP facilities further compound these advantages. They generally require lower capital expenditure (CAPEX) than large-scale hydropower projects, owing to simpler civil works such as shallower dams or weirs and smaller turbine-generator assemblies[1]. However, the construction of long headrace tunnels can introduce substantial cost uncertainty, sometimes extending into the operational phase post-commissioning. Tunnel-related costs can disproportionately affect the feasibility of SHP projects, posing a significant challenge to their successful delivery.

Despite these risks, the Himalayas offer promising conditions for SHP - particularly run-of-river types - thanks to abundant glacial meltwater and high hydraulic head. While climate change[2], may temper water availability, SHP remains a compelling option for generating clean, renewable electricity with comparatively lower financial and technical risk. As noted by Bent Flyvbjerg (Bent Flyvbjerg 2004), smaller infrastructure projects, including tunneling works, tend to experience more manageable cost escalations, thereby reducing financial exposure and amplifying the benefits of innovative design interventions.

This paper presents several modest, yet impactful design innovations conceived by the design review team during their work on the Upper Modi SHP in Nepal. These include modifications to the water-retaining structures at the upstream weir, as well as design changes to the headrace tunnel and surge shaft. At the time of the review (and as reported herein), these concepts were considered to have a significant influence on the overall development cost.

Since the review was conducted during the feasibility stage - several years ago[3], the magnitude of projected savings and the perceived innovativeness may have evolved. Ultimately, the decision to implement any specific design change rests with the project designer, who remains accountable for the outcome. This decision-making process is typically guided by engineering judgment, experience, and the preferences of both the designer and the owner.

Regarding the status of the UMA project, financial close was achieved in January 2024, and preparatory site construction is underway. The Lot-1 contract (civil and hydro-technical works) has mobilized to site. According to a recent report by NEA, physical progress stands at approximately 5%, primarily associated with mobilization and camp construction. Meanwhile, bid evaluation for the electro-mechanical Lot-2 contract is ongoing. Construction is proceeding in a phased manner.

1.2 Innovate to Reduce Cost - Always a Necessity

Economies of scale are well established - large hydropower projects benefit from efficiencies in civil works and electro-mechanical equipment, resulting in lower unit costs ($/MW). In contrast, small hydropower projects (SHPs) offer a different set of advantages: lower upfront investment, shorter permitting timelines, reduced social and environmental footprints, faster construction schedules, simpler transmission interconnections, and earlier commercial operation dates. Collectively, these attributes help ease financial pressures on developers and governments.

A significant portion of hydropower development costs typically stems from civil and electro-mechanical components, along with early-stage expenditures such as feasibility studies, stakeholder engagement, environmental and socio-economic mitigation, and land acquisition - including Local Area Development Programs (LADP) and Resettlement Action Plans (RAP) (IRENA 2021). As such, design innovations in civil structures and electro-mechanical systems represent a key opportunity for cost reduction.

While SHPs generally require less capital expenditure than large-scale projects, they still demand rigorous planning and execution. Despite their smaller scale, SHPs encompass the full complexity of hydropower development. Therefore, it is essential that developers engage a broad and experienced team throughout design and construction. Maintaining adequate technical oversight and incorporating expert advisory input - where appropriate - can yield both immediate and long-term benefits, including cost savings and timely commissioning. In essence, addressing both known and unforeseen challenges requires foresight, planning, and the right resources - though a deeper discussion on this topic is reserved for a future presentation.

Finally, it is worth noting that SHPs may exhibit slightly higher levelized costs of energy (LCOE) and per-MW unit costs compared to larger plants (IRENA 2021). This further underscores the importance of continuous value engineering throughout the project lifecycle - from pre-feasibility through to construction[4] - to optimize design, manage risk, and enhance overall project viability.

1. Also, as we know, smaller equipment to be installed can also help reduce construction costs, e.g., a smaller stay ring is a less heavy mass to lift, and thus smaller lifting equipment is required.
2. Yet possibly exasperated by increased sediment accumulation (often associated with extreme weather events due to climate change) requiring a more robust sediment management design to accommodate increased accumulations of material, in front of the dam.
3. This material has not been reported or presented elsewhere. The authors are pleased to be able to share the material in this forum.
4. It is often during construction where actual conditions encountered (between drilled investigation holes) can require changes in the design, or construction methods. Sometimes, savings can be realized when decisions are made quickly, and the proper change is implemented. It also remains best to not conclude the "design" phase until after construction, and in fact, after commissioning - to ensure the experience gained during construction remains employed, and resources are available to address any surprises, say, for example, leakage and/or seepage from the dam (or weir) which is larger than anticipated.

1.2.1 Significant Innovations (as least in our opinion!)

1.2.1.1 *Weirs*

Water is the fundamental medium for hydropower generation and remains critical to all hydropower facilities[5]. Minimizing water loss - whether through seepage or leakage[6] - is essential. This priority is especially pronounced in the Himalayas, where river flow diminishes during colder seasons. Maximizing diversion of available river water[7] for energy production is therefore key to maintaining reliable power supply. In design terms, seepage reduction should be a primary objective, balanced against the outcome of a cost-benefit assessment.

1.2.1.2 *Tunnels*

As previously noted, the headrace tunnel is often a cost-critical component in SHP developments, not only due to its direct expense but also because of potential cost uncertainty and schedule delays - both of which represent financial risk.

More specifically, poor rock conditions - whether due to material properties or in-situ stress - can delay construction. In some cases, the excavation process itself may alter ground conditions, leading to undesired tunnel convergence. Such convergence increases head loss, thereby reducing energy output. Accordingly, minimizing or eliminating tunnel convergence should be a key design consideration.

1.2.1.3 *Shafts (Vertical Underground Cavities)*

Deep surge shafts constructed in challenging geological conditions can be particularly costly. High excavation expenses often stem from complex ground support requirements and difficult construction environments, of which, contractors can exasperate costs by moving slowly or using improper temporary support measures. As such, identifying innovative design approaches or construction methodologies to reduce shaft-related costs should be a priority for designers.

1.3 Content Qualification

The authors wish to remain clear and note that the case study and concepts presented in this paper are conceptual in nature and were developed by the authors during the design review process. As noted earlier, these ideas may or may not be incorporated into the final design. At the time of writing, one design innovation- related to the UMA headrace tunnel - was adopted into the final design.

The authors also emphasize that the views expressed herein are their own and may not reflect the official position of NEA or the project designer. These innovations are shared for consideration and potential application in other hydropower projects being conceptualized in the Himalayas or in regions with similar geological and operational conditions.

2. A SHP IN NEPAL - OVERVIEW

2.1 Scheme Location

The case study and associated design innovations presented herein were developed for the proposed Upper Modi "A" plant (UMA), part of the Upper Modi Cascade Hydroelectric Project located in Gandaki Province, Nepal (Figure 1). This development comprises two small hydropower plants: Upper Modi 'A' (42 MW) and Upper Modi (18.2 MW).

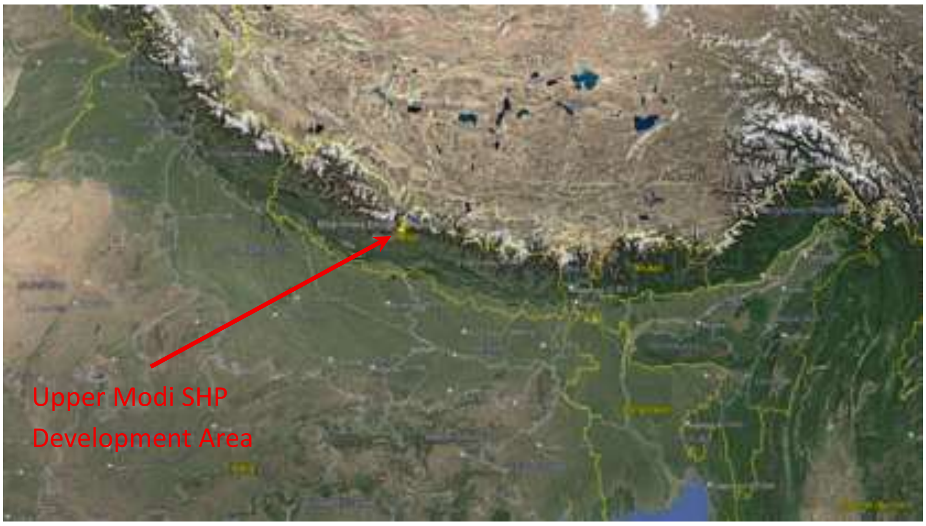

Fig. 1 : General location of UMA SHP in Nepal (Gandaki Province)

5. But not at any cost! Some projects require a proper cost/benefit, i.e., some short tunnels may best be replaced by an open cut channel.
6. Leakage occurring through concrete dams, versus seepage passing through an earthen dam.
7. Over and above what is required for compensation flow (where applicable).

Initial studies for the UMA hydroelectric project date back to 1997, led by the Nepal Electricity Authority (NEA). Pre-feasibility and Environmental and Social Impact Assessment (ESIA) work were completed around 2012. In early 2017, NEA refined the SHP layout to reduce construction costs, increase gross head, and improve overall project economics. Later that year, NEA engaged an independent consulting firm—AF Consult Switzerland, in association with the consortium of ITECO Nepal and Total Services Management - to carry out detailed design for both UMA and Upper Modi HEP, and to prepare tender documents for UMA's development (Modi Jalvidyut Company Limited 2023).

The UMA SHP is situated along the Modi River, whose basin is bounded by the Annapurna range to the north, the Kali Gandaki River to the west, the Madi River to the east, and the Middle Hills to the south. The Modi River flows through the midland region of the Himalayas - also known as the Lesser Himalayas - with elevation ranging from snow-fed peaks above 6,400 masl down to approximately 900 masl.

The headworks site for UMA, including the upstream weir, is located roughly 100 meters downstream of the confluence of Modi Khola and Kimrung Khola, near the New Bridge in Annapurna Rural Municipality. Key infrastructure elevations include the intake sill level at approximately 1,168 masl and the turbine axis at around 1,149 masl.

The water conveyance tunnel alignment passes beneath forested areas and settlements within Annapurna Rural Municipality. The powerhouse is located approximately 7 km downstream of the diversion weir, near Kilyu Village, about 8 km north of Nayapul. It lies roughly 7 km upstream of the Middle Modi generating station, whose headrace tunnel was under construction and visited during the design review.

Additional project characteristics - including geological conditions, installed capacity, and components subject to conceptual design review - are discussed below.

2.2 UMA - Hydropower Plant Overview (Typical Run-of- River Elements)

The UMA run-of-river small hydropower development comprises the following key components typical of Himalayan SHP schemes: upstream weir, low-level bypass gates, intake gates, desander basin, headrace tunnel, surge shaft, valve chamber, vertical shaft (penstock), horizontal high-pressure steel-lined tunnel (penstock), powerhouse, and tailrace channel. Many of these components involve underground openings that are subject to technical risks associated with complex ground conditions.

To mitigate the risk of collapse during construction and ensure long-term structural integrity, the design and timely installation of tunnel support systems remain paramount.

2.3 Geological Setting - A Significant Design Factor

2.3.1 *Upper Modi A*

The UMA project is situated within the Lesser Himalayan geological sequence of Nepal. The Modi Khola follows the axis of an anticline fold, with the Main Central Thrust (MCT) located approximately 5 to 8 km upstream of the weir/intake site near the New Bridge in Annapurna Rural Municipality. The weir foundation comprises river run materials - gravel, sand, siltstone fragments, and boulders- with an estimated depth to bedrock of up to 12 meters (Figure 2 - illustrates what is encountered at river level). The riverbed exhibits a high sediment load, suggesting a similarly permeable subsurface. Consequently, seepage beneath the weir is likely - posing a significant concern, especially during low-flow winter months when river discharge diminishes.

Two ground thrust features are expected to intersect the UMA headrace tunnel (HRT), while the powerhouse, being a surface structure, is not anticipated to be affected by ground stress.

Dominant rock types in the project area include phyllite, quartzite, intercalated phyllite-quartzite sequences of Precambrian age, and granitic gneiss of Paleozoic age. Figure 3 illustrates the exposed and destressed rock mass near the proposed upstream weir/intake portal.

The HRT alignment traverses the right bank hill slope of the Modi Khola and is expected to encounter moderately strong rock masses interspersed with weak slope zones. A significant portion of the tunnel will pass through phyllite with quartzite, with remaining sections encountering quartzite, gneiss, schist, and phyllite, and based on engineering assessments and estimates, approximately 6% of the HRT may pass through extremely poor rock mass conditions, 12% through very poor conditions, and the remainder through poor to fair rock masses.

Drilling investigations suggest that the surge shaft at Kilyu Village will encounter surficial colluvial deposits and highly fractured bedrock, with rock quality designation (RQD) values around 19%. The rock mass in this area is classified as poor to very poor (Modi Jalvidyut Company Limited 2023).

Given these geological conditions - and similar observations from the Middle Modi HRT (see Figure 4) - the installation and timing of tunnel support systems are critical to the success of the UMA development and other hydropower projects in Nepal.

Excavated materials (from the tunnel) such as crushed phyllite, schist, and quartzite from Himalayan tunnels typically exhibit low porosity, moderate to high density, variable strength, and good thermal durability. These properties may render them suitable for use in water-retaining embankment fills, provided careful material selection is undertaken. However, schist and phyllite may pose stability risks due to foliation and weathering.

Fig. 2 : Upstream weir/intake area looking downstream but over the weir axis (at time of site visit)

Fig. 3 : Typical geological conditions - anticipated in the development area (Modi Jalvidyut Company Limited 2023)

2.3.2 *Neighbouring Development (Middle Modi HRT Site Visit) Observations - The Analogue Plant*

The Middle Modi Hydroelectric Project is a similar run-of-river facility located downstream - yet in proximity - to UMA.

During the UMA design review, the authors visited the Middle Modi HRT, which was under construction at the time. This tunnel, located along the right bank of the Modi Khola, was progressing through rock mass types and conditions like those anticipated for UMA. The site visit provided valuable insights for the UMA project.

As observed near station c. 1+140 (approximately halfway along the tunnel alignment - Figure 4 - and typical of many locations along the Middle Modi HRT), the permanent tunnel liner had buckled, reducing tunnel width by over 10%. Similar instances of tunnel wall convergence and invert heave were noted at multiple locations.

The observations (e.g., Figure 4) confirmed that the rock mass encountered in the Middle Modi HRT - and expected in UMA - can be classified as poor to exceptionally poor. Combined with strong lateral ground stresses typical of the Himalayas, these conditions present significant excavation challenges and necessitate robust long-term support strategies (Banjade 2019).

Fig. 4 : Tunnel Liner Failure - Middle Mode HRT Conditions (observed during the site visit)

2.4 Capacity and Size of Plant

The Upper Modi Cascade Project harnesses the hydropower potential of the Modi River by leveraging both its flow and topographic gradient to generate clean, renewable electricity. Water is collected at the upstream weir and diverted through the intake into the UMA powerhouse via a 5,968 m long headrace tunnel (Modi Jalvidyut Company Limited 2023).

To illustrate the potential cost savings associated with the design innovations proposed in this paper, some features regarding the capacity of the proposed UMA, and salient features of the plant, are provided. Refer to Table 1 (noting the date listed in the table is subject to refinement as the design is further optimized).

Table 1 : Salient Features of Plant -UMA

Number of units	2
Type of turbine	Vertical Axis Pelton
Max turbine discharge (m3/s)	17.4±
Rated head (m)	274.5±
Rated Generator Output (MW), i.e., per unit.	21.5±

2.5 Key Components (i.e., those pertaining to this paper) - Feasibility Design and (Suggested) Conceptual Change

The design review focused on three critical components: the upstream weir (specifically its impervious elements), the headrace tunnel (HRT), and the surge shaft.

The review focused on these key elements and anticipated to address the geological challenges anticipated, which were deemed to reduce construction (CAPEX) costs (at the time of the review).

With the geological context previously outlined, readers - particularly dam engineers and developers - will better appreciate the rationale behind the proposed design changes. In summary, three significant (at least in the authors' view) innovations were postulated, engineered, and deemed more cost-effective to construct:

(i) the introduction of an upstream impervious blanket using tunnel "muck" - thereby helping to reduce seepage or water loss around and under the upstream dam (weir), but also reducing tunnel spoil management, i.e., reducing the cost to transport the material to the spoil area(s) and to manage the spoil area.

(ii) tunnel support mechanisms, and

(iii) a conceptually new surge shaft configuration.

More details are provided in the sections which follow.

2.5.1 *Weir*

The upstream weir across the Modi River is designed ((Modi Jalvidyut Company Limited 2023) to impound water and pass up to 836 m³/s over its crest. A portion of the impounded water is diverted for power generation through a desander and intake structure (comprised of four gates, each 3 m × 3 m). The weir will span the river and is 30 m in width and includes upstream concrete training walls to guide flood flows. A typical cross-section is shown in Figure 5.

At this location the average riverbed elevation is near elevation 1448.5 masl, with the overflow crest positioned about 6 m above the riverbed. Sediment management is facilitated by a 6 m × 6 m radial sluice gate, designed to flush bedload during the monsoon season and reduce sedimentation in the small reservoir.

The gated weir - comprising two 2.5 m × 6.2 m gates - is a concrete gravity structure approximately 30 m wide and 10 m high. It is designed to divert 17.4 m³/s of flow into the right-bank intake and desander. Water cutoff is achieved via a short upstream concrete apron and a shallow concrete key.

During the site visit to the neighboring HRT, the authors observed that the tunnel muck - stockpiled near the portal - was relatively impervious and potentially suitable for use as a water-retaining element. Given the proximity of the HRT portal to the weir, repurposing this material to construct an upstream impervious blanket could reduce seepage losses and lower spoil management costs. The conceptual layout of this blanket is illustrated in Figure 6.

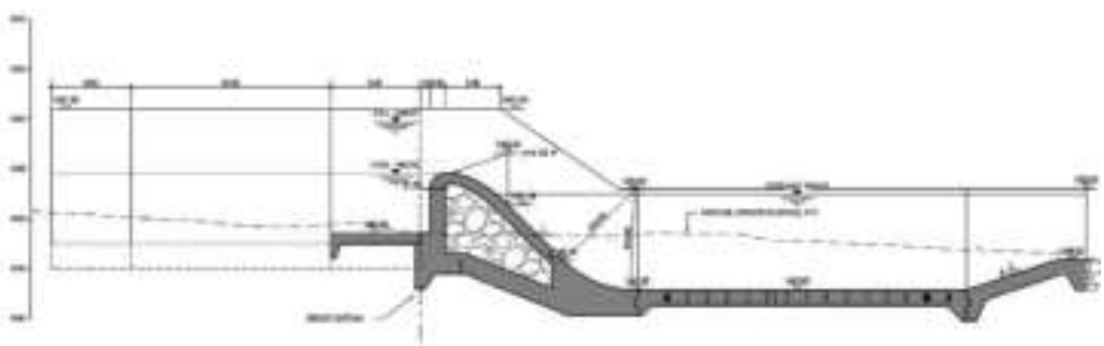

Fig. 5 : Upstream weir configuration - feasibility design (Modi Jalvidyut Company Limited 2023)

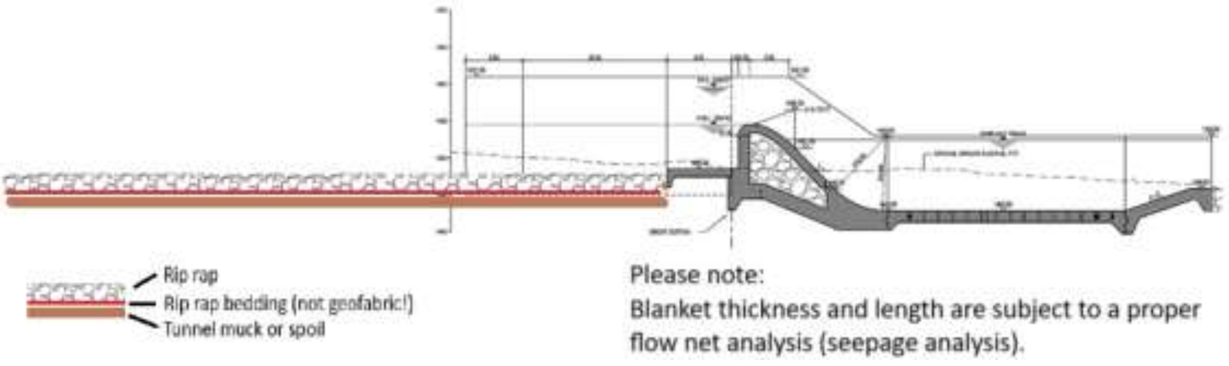

Fig. 6 : Upstream blanket (composed of select tunnel muck)

2.5.2 *Headrace Tunnel*

At the time of the design review, the proposed HRT was approximately 5,950 m long with an internal diameter of 3.4 m, originally designed with an inverted D-shaped cross-section (see Figure 7). However, based on observations from the neighboring HRT and supported by numerical modeling and closed-form analysis, the authors concluded that a design change was necessary to better manage ground stress and accommodate the anticipated poor rock mass conditions.

A parametric study of alternative tunnel geometries was conducted, and the modified horseshoe shape - combined with ribbed support in select zones - was found to be more suitable for the Himalayan context. The revised tunnel geometry for Class II rock (Q = 10–40) is shown in Figure 8.

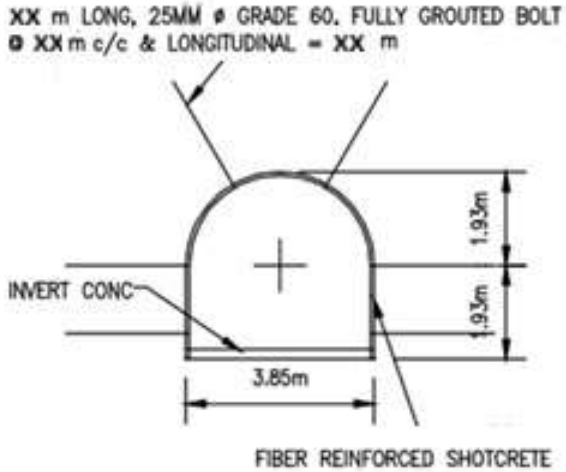

Fig. 7 : Feasibility Design - Headrace tunnel (HRT) standard “D” shape configuration (typical Good Rock/Class II or $10 < Q \leq 40$)

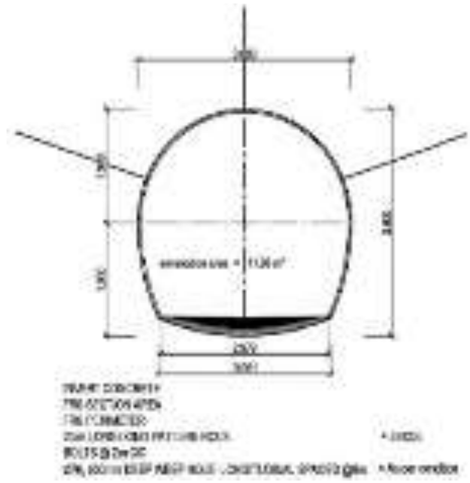

Fig. 8 : Design Review (and final design) - HRT modified horseshoe for Good Rock/Class II or $10 < Q \leq 40$ (Modi Jalvidyut Company Limited 2023)

2.5.3 *Surge Shaft*

The restricted-orifice surge shaft is designed to rise approximately 44 m above the HRT obvert and has a diameter of 10 m. The final design, including typical ground support, is illustrated in Figure 9. While the engineer of record opted for a traditional vertical shaft configuration, the authors proposed an alternative geometry aimed at improving constructability and reducing support requirements - and should be investigated for use on other hydropower developments in the Himalayas, and with similar geological ground conditions.

The suggested configuration (Figure 10) incorporates a telescoping profile, which allows for staged excavation in very poor ground. This approach provides annular space for installing vertical (perforated) steel piles, which can be used for consolidation grouting or drainage - enhancing ground stability during and after excavation.

Although the final design issued for tender followed the traditional vertical shaft layout, the authors emphasize the practical merits of the telescoping concept, particularly for similar geological conditions in the Himalayas or elsewhere. The proposed system was numerically tested to validate its structural and geotechnical feasibility.

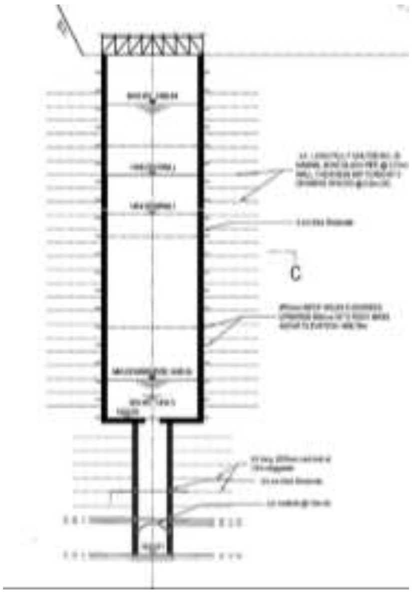

Fig. 9 : Final Design - Surge shaft geometry and illustrative support (Modi Jalvidyut Company Limited 2023)

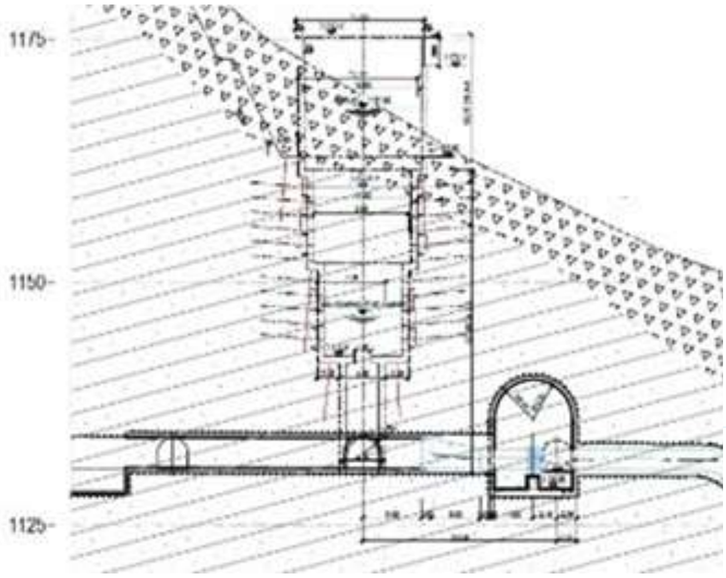

Fig. 10 : Plausible shaft design - for very poor rock conditions

3. CONCLUSIONS

This paper presents a case study of a small run-of-river hydropower development in Nepal, where targeted value engineering was introduced during the feasibility-stage design review. The review focused on three key components: the upstream weir, headrace tunnel geometry, and surge shaft configuration.

Based on the tender documents (with the process still ongoing), NEA has adopted the revised tunnel cross-section proposed by the authors during the review. However currently, the conceptual changes to the upstream weir and surge shaft were not incorporated into the final design - due to various project-specific considerations.

Nonetheless, these design innovations remain technically relevant and merit further discussion. They may offer practical benefits for other hydropower developments in the Himalayas or in regions with similar geological and construction challenges.

4. RECOMMENDATIONS

Based on the findings and observations presented in this paper, the following recommendations are offered for future small hydropower developments in the Himalayas and other geologically complex regions:

(a) Incorporate Value Engineering Early: Design reviews during the feasibility stage can yield significant cost-saving opportunities, particularly when informed by site visits, analogue projects, and interdisciplinary input

(b) Adapt Tunnel Geometry to Ground Conditions: In regions with poor to very poor rock mass and high lateral stress, modified tunnel geometries - such as horseshoe profiles with staged support - should be considered to reduce convergence and long-term maintenance risks.

(c) Explore Reuse of Excavated Material: Tunnel muck, when properly characterized, may be repurposed as impervious fill or embankment material, reducing both spoil management costs and environmental impact.

(d) Consider Alternative Surge Shaft Configurations: In weak ground conditions, telescoping shaft geometries with provisions for consolidation grouting and drainage can improve constructability and long-term stability.

(e) Document and Share Lessons from Analogue Projects: Observations from neighboring or similar hydropower developments can provide invaluable insights into ground behavior, construction risks, and support strategies, an

(f) Maintain Design Flexibility: Not all innovations may be adopted in a given project, but documenting and testing them ensures a growing knowledge base for future applications.

These recommendations aim to support more resilient, cost-effective, and context-sensitive hydropower development in mountainous regions.

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APPRAISAL OF ROCK MASS PROPERTIES AND THEIR INFLUENCE ON PROBABILISTIC SLOPE STABILITY ANALYSIS FOR A HYDROELECTRIC PROJECT

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ABSTRACT

Hydroelectric projects in the Himalayas typically feature high slopes composed of geological materials such as rock, soil, or debris deposits. Rocks encountered in this region are generally jointed, making it a challenging task to assess their engineering properties. The present article presents an appraisal of rock mass properties for analyzing and designing a high rock slope for a hydroelectric project located in the higher Himalayas. The rocks encountered are predominantly mica schists with bands of quartzites. A concrete gravity dam was constructed at the site. The excavation was done to accommodate the dam. The height of the slope was more than 70m. Instabilities were observed after excavating to a depth of about 15m from the top. Immediate support measures were required to keep the slope stable.

Detailed analysis of the slope is done, and the probability of unsatisfactory performance of the slope is evaluated. Rock mass properties for the slope section were derived from the outcome of field and laboratory investigations. The rock failure was found to be governed by joints. Non-linear shear strength criterion was considered for simulating shear strength behavior of the joints. Since uncertainties are involved in the properties, a probabilistic framework was adopted using statistical distributions.

Results demonstrate that incorporating statistical evaluation of joint shear strength behavior results in more realistic behavior of unsatisfactory behavior of compared the slope. The study also demonstrates the importance of non-linear failure criteria in assessing rock-slope stability.

Keywords*: Joint shear strength, Rock mass properties, Probabilistic analysis, statistical analysis, Hydroelectric project, Factor of safety, Probability of failure.*

1.0 INTRODUCTION

Hydroelectric projects are inherently sensitive to geological and geotechnical conditions. The construction of major project components such as dams, reservoirs, spillways, tunnels, and access roads often necessitates extensive excavation in complex rock masses, where the mechanical behavior of the rock directly governs the safety, performance, and cost of the structures. Geological features such as unfavorable jointing, weathering, foliation, schistosity, and groundwater inflow can significantly reduce the strength and deformability of the rock mass, thereby compromising slope stability. Hence, a rigorous appraisal of rock mass properties is a prerequisite for reliable design and risk management in such projects.

The conventional slope stability assessments primarily employ deterministic approaches, in which fixed representative values are assigned to rock mass parameters. These analyses yield a single Factor of Safety (FoS), representing the ratio of resisting to driving moments under specified design conditions. However, rock masses are inherently heterogeneous and spatially variable; as a result, deterministic methods often fail to capture the uncertainty associated with natural variability and may lead to either conservative or unsafe designs [1].

To overcome these limitations, probabilistic slope stability analysis has gained increasing attention in recent years. Probabilistic approaches explicitly incorporate uncertainties in input parameters and quantify the probability of failure, providing a more realistic measure of slope reliability. Methods such as Monte Carlo Simulation [2] [3], First and Second-Order Reliability Methods [4], and Random Finite Element Method [5] enable the propagation of parameter variability through numerical models, allowing a more comprehensive evaluation of risk. For instance, in the Baixo Iguaçu Hydroelectric Power Plant (HPP), probabilistic analysis was employed to evaluate the influence of foundation heterogeneity and hydrological variability on sliding failure probabilities under normal and extreme flow conditions [6].

In rock slope engineering, probabilistic methods have proven effective in quantifying the influence of uncertainty in rock mass classification, discontinuity orientation and spacing, weathering, and strength reduction. A study on Turkish highway cut slopes using the Slope Stability Probability Classification (SSPC) approach demonstrated that incorporating parameter uncertainty, especially in discontinuity orientation and material variability, significantly reduced the predicted stability probability of steep rock slopes [7].

Slope instability poses major risks to the safety, operation, and economic performance of hydroelectric projects. Integrating detailed rock mass characterization with probabilistic slope stability analysis provides a reliable framework for identifying failure mechanisms and addressing parameter uncertainties. The Nikachhu Hydroelectric Project in Bhutan, commissioned in 2024, encountered significant geological and geotechnical challenges, including instability at the dam's left bank abutment, which caused delays and cost overruns. This study highlights the need for comprehensive rock mass evaluation and its integration into probabilistic analysis to improve design reliability, reduce construction risks, and achieve safer and more cost-effective hydroelectric project outcomes.

2.0 STUDY AREA, GEOLOGICAL SETTING AND PROBLEMS ENCOUNTERED

The study has been conducted for left bank slope of the Nikachhu Hydroelectric project. The project is located on river Nikachhu in district Trongsa of Bhutan (Fig. 1). The dam area is primarily an asymmetrical valley characterized by a steep slope on the right bank, while the left bank has a moderate slope, terraced with fluvial and tectonic features (Fig.2). The average riverbed elevation at the dam axis is El. 2260.52 m. The dam height from the deepest foundation (El. 2253 m) is 42 m (dam top El. 2295 m). The length available at the crest is 104 m. The geology of Bhutan is shown in (Fig 3). The main rock mass at the Dam site comprises mica schists with intermittent quartzite and gneissic bands, which are intruded by granite-leucogranite, pegmatite, and mafic. The sporadic amphibolite bands have also been encountered. The rocks belong to Thimphu Group of Higher Himalayan Tectonic Unit. Thimphu Group has been divided into Sure, Naspe, and Taktsang Formations. There is a perceptible increase in the metamorphic grade from south to north.

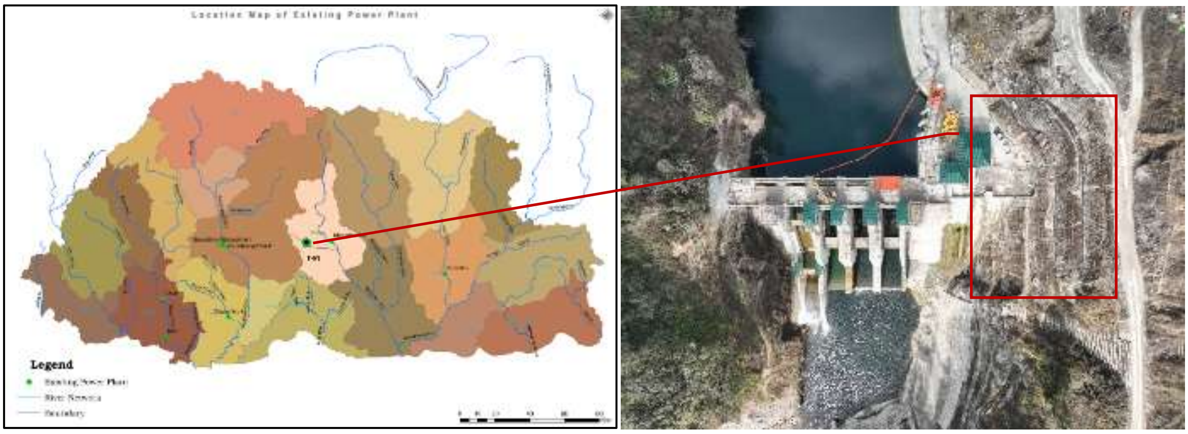

Fig. 1 Geographical map of Bhutan showing the project location

The overburden thickness ranged from 15 to 20 m. To accommodate the dam section, the excavation of the left bank cut slope was initiated from road level El. 2317 m in a gradient 2H:1V. In the excavation, encountered in-situ rock masses are mainly Mica-schist/ Quartz-mica-schist and highly degraded granitic/quartz-feldspathic rock (Fig. 4). The low-grade schists are highly fractured and closely jointed, and alternate inter-folial phyllites. The overall grade of weathering is moderate to high with prominent sericitization and mylonitization. Foliation joint dipping into the hill is traceable in comparatively lesser weathered zones. Foliation joints are very closely to closely spaced with joint surfaces smooth to slicken-sided. The instabilities were observed while excavating the bank at El, 2298 (Fig. 5). To mitigate instabilities, 25mm diameter, 6m-12m long SDR fully grouted with double-layer were used with wire mesh. A bench of 3m width was created at El. 2309, 2302, and 2295 m with an overall slope gradient of 2H:1V. Water seepage was observed at the left bank, which caused deterioration in the properties of the mica schist.

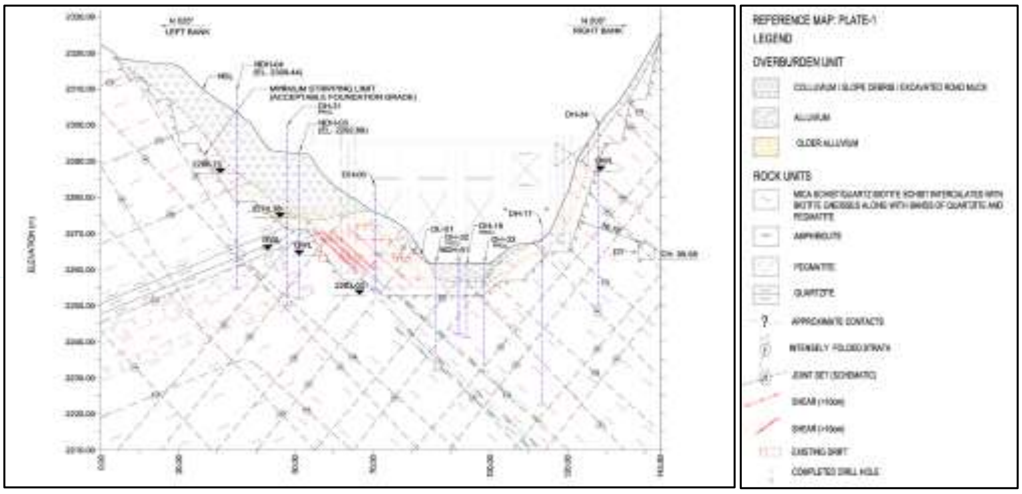

Fig. 2 Natural cross section of the river before construction of dam

Fig. 3 Geological Map of Western and Central Bhutan, adapted from [8]*Showing sample localities. STD = South Tibetan Detachment, MCT = Main Central Thrust, MBT = Main Boundary Thrust, MFT = Main Frontal Thrust.* [9]

Fig. 4 Mica Schist/Quartz Mica Schist intruded by Quartz-Feldspathic veins along foliation with intense schistosity encountered during excavation

3.0 ROCK MASS CHARACTERISATION

Stability assessment of rock slopes involves assessment of rock mass properties followed by stability analysis based on kinematic, analytical, and numerical analyses. An adequate appraisal of rock mass involves identifying and quantifying key geological and geotechnical characteristics such as rock type, strength, deformability, groundwater conditions, and the rock mass classification systems [10]. The main purpose of the rock mass appraisal is to determine the engineering behavior of the rock mass, to select suitable design parameters for slopes, tunnels, and foundations, to enable a plan for appropriate support measures, and essentially to identify the geological hazards.

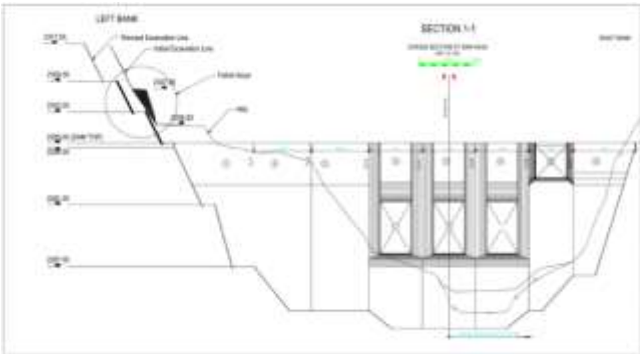

Fig. 5 Excavation profile and instability at left abutment

The laboratory test data utilized in this study were obtained from the Detailed Project Geological Report [11] and the Revised Geological Baseline Report (2017) of the Nikachhu Hydroelectric Project. The available dataset includes results from uniaxial compressive strength (UCS) tests, triaxial compression tests, direct shear tests on rock joints, Brazilian tensile strength tests, point load tests, and plate load tests. These test results were subjected to statistical evaluation to characterize the variability of the rock mass properties and to identify appropriate statistical distributions for each parameter. The derived statistical parameters were subsequently incorporated to account for inherent material uncertainty. Four probability distributions namely normal, log-normal, Weibull and Gamma functions, were fitted to the test data (Table 1), and goodness-of-fit evaluations were conducted using the D-index test and Chi-square test to determine the most representative distribution for each geotechnical parameter [12].

Table 1 Probability distribution used in analysis (Reported from Kottegioda and Rosso 2008)

SL. No	Details	PDF	CDF
1.	Normal	$F(x) = \frac{1}{\sigma\sqrt{2\pi}} exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right]$	$F(x) = \frac{1}{\sigma\sqrt{2\pi}}\int_{-\infty}^{x} exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right]dx$
2.	Log Normal	$F(x) = \frac{1}{x\sigma\sqrt{2\pi}} e^{-\frac{(lnx-\mu)^2}{2\sigma^2}}$	$F(x) = \frac{k}{\sqrt{2\pi}.\sigma_y}\int_{-\infty}^{x}(1/x)\exp[-\frac{1}{2\left((\frac{log_e(x)-\mu_y}{\sigma_y})^2\right]}dx$
3.	Weibull	$PDF = \frac{\alpha}{\beta}\left(\frac{x-x_0}{\beta}\right)^{\alpha-1} exp\left[-\left(\frac{x-x_0}{\beta}\right)^{\alpha}\right]$	$P = 1 - exp\left[-\left(\frac{x-x_0}{\beta}\right)^{\alpha}\right]$
4.	Gamma	$F(x) = \frac{1}{\lvert\alpha\rvert\Gamma(\beta)}\left(\frac{x-x_0}{\alpha}\right)^{\beta-1} e^{-\left(\frac{x-x_0}{\alpha}\right)}$	$F(x) = \int_{x_0}^{x}\frac{1}{\lvert\alpha\rvert\Gamma(\beta)}\left(\frac{x-x_0}{\alpha}\right)^{\beta-1} e^{-\left(\frac{x-x_0}{\alpha}\right)}$

3.1 Intact Rock Uniaxial Compressive Strength (σ_{ci})

Intact rock strength defines the ability of the parent rock to resist stress under operational loads. The overall strength behavior of the rock mass is governed by several factors, and the intact rock contributes to a maximum of about 30% of the overall strength behavior of the rock mass. The UCS is determined through laboratory tests or indirect methods like Schmidt hammer test and Point Load Strength index tests. The UCS of intact rock, (σ_{ci}) was measured following ISRM (1979) standards. Statistical analysis of UCS data was done using Normal, Log Normal, Weibull and Gamma distributions and Probability Density functions and Cumulative Distribution Functions were obtained (Fig.6). The mean and standard deviation for UCS were observed to be 50.82 MPa and 27.95 MPa respectively.

Figure 7 infers that all the distributions fit well and show their characteristics of trend and variability. However, based on goodness-of-fit test, it can be concluded that Weibull distribution fits better than other distributions. It is also observed that the THREE parameter Weibull distribution provides the optimised probability distribution over the TWO parameter Weibull distribution, where the latter doesn't consider the effect of the location parameter.

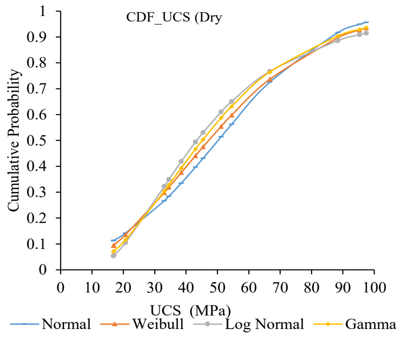

Fig. 6 CDF Plots for UCS of the intact rocks

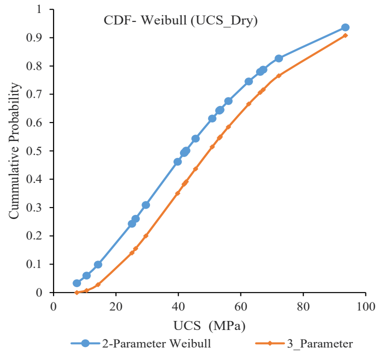

Fig. 7 Comparison plot of 2&3 parameter Weibull distribution

3.2 Intact rock shear strength parameters

Triaxial tests were performed at the intact specimens[14]. The confining pressure was varied up to a maximum of 11.768 MPa. The linear Mohr Coulomb shear strength parameters were obtained, and their distributions are presented in Fig. 8. The mean and standard deviations of the Mohr Coulomb parameter are 2.53 MPa and 1.14 MPa for cohesion, c and 42.47° and 7.79° for friction angle (ϕ_i) respectively. Moreover, Hoek Brown material constant (m_i) was obtained from triaxial test data, and its distribution is shown in Fig. 8.

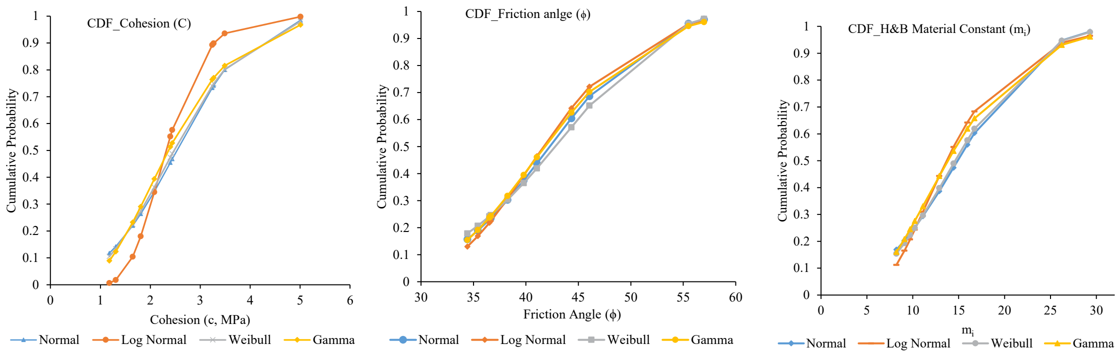

Fig. 8 CDF Plots of parameters obtained from Triaxial Test data

3.3 Intact Rock Modulus (E_i)

Intact rock modulus (E_i) has been taken equal to tangent modulus computed at 50% of UCS from axial stress-strain plots of UCS tests. The available data is analyzed statistically to determine the variability of the data. The fitted distributions are shown in Fig. 9. The analysis indicated that log normal & gamma distributions fit well considering the skewed data.

3.4 Rock Mass Modulus from Plate Load Tests (E_{rm})

Cyclic Plate load tests were performed in the field using plate size of 600 mm as per [15]. The modulus of the rock mass was obtained, and their distributions are shown in Fig. 10. The mean and standard deviation obtained from statistical analysis are 617.29 MPa and 292.10 MPa respectively.

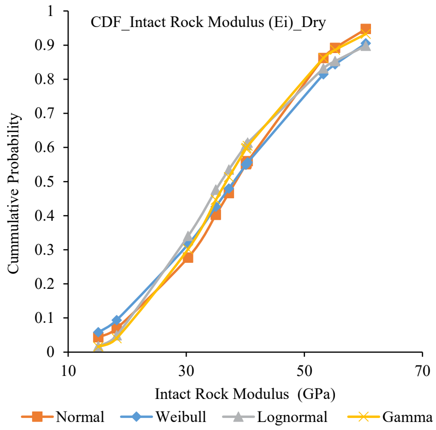

Fig. 9 CDF plots for intact rock modulus

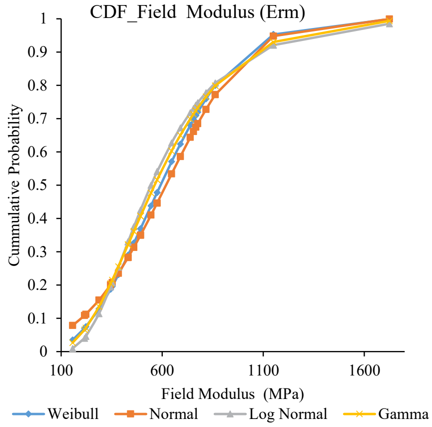

Fig. 10 CDF plots for field modulus

3.5 Joint Shear Strength Model

Joint shear strength parameters define the resistance of rock joints to sliding and are essential for assessing slope and foundation stability. Barton's JRC-JCS model is the most widely used shear strength model for rock joints [16]. The model represents non-linear variation of shear strength represented in the form of residual friction angle (ϕ_r), joint roughness coefficient (JRC), and joint wall compressive strength (JCS), and normal stress (σ_n). The shear strength is represented as:

$$\tau_f = \sigma_n tan\left[JRC \log_{10}\left(\frac{JCS}{\sigma_n}\right) + \phi_r\right] \quad (1)$$

Where τ_f = Shear stress at joint plane, σ_n = normal stress applied, JRC = Joint roughness coefficient, JCS= Jointwall compressive strength, and ϕ_r= residual friction angle.

The laboratory test data on direct shear tests on joints was available from the geological base line report. A total of 14 sets were available, and in each set 3 normal stress values were used and shear stress at failure was reported. Fig.11 shows the entire data base in the form of shear strength vs normal stress values. The figure also shows upper and lower bound failure envelopes. The parameters ϕ_r, JCS and JRC were treated as unknowns. The upper bound value of JCS equal to 50 MPa was assigned, and parameters were optimized by least square method. Fourteen sets of parameters were thus obtained. The statistical analysis was performed for the joint parameters obtained from the back analysis and distributions are determined as presented in Fig.12. The mean and standard deviations of JCS and ϕ_r obtained are 26.14 MPa, 12.40 MPa and 30.22°, 1.19° respectively. Moreover, the JRC obtained was 11.00, almost constant for all the tests.

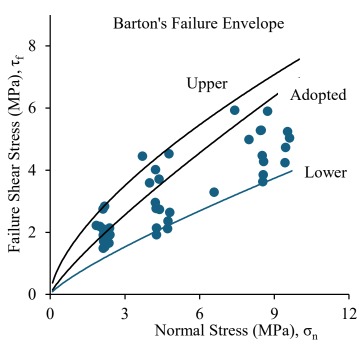

Fig. 11 Failure envelopes from laboratory tests on joints

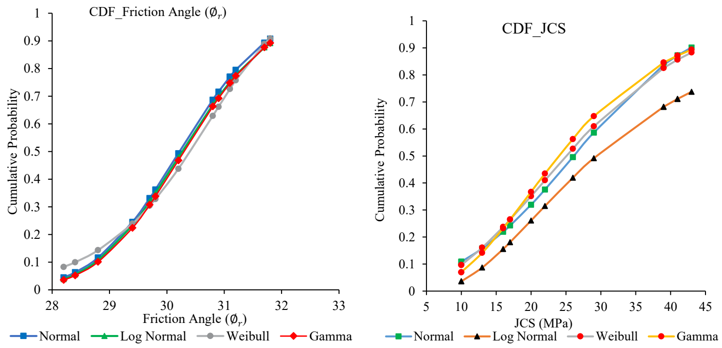

Fig. 12 CDF plots for joint shear strength parameters obtained from direct shear test

4.0 SLOPE STABILITY ANALYSIS

Slope instabilities were observed during excavation of left bank slope. An attempt is made in the following to simulate the observed behavior of the slope. For this purpose, two different scenarios are considered, and stability analyses are carried out using the shear strength parameters from rock mass characterization as available. Finally, the shear strength parameters were fine tuned to match the failure conditions of simulated slope with the actual slope in the field. The analysis was carried out using software SLIDE 2 (RocScience, 2022).

4.1 Scenario 1: Natural slope with overburden

This scenario represents the slope before excavation. Colluvium overburden was present at that time and was stable. The section is shown in Fig. 13. To back-calculate the shear strength parameters, the analysis was carried out by using different combinations of cohesion and friction angle of the colluvium. The ϕ values were set at 20, 25, …45° and for each ϕ, the cohesion was varied from 20 to 100 kPa. The pore water pressure was simulated by setting Ru=0.25. The saturated unit weight of the colluvium was assumed to be 18KN/m^3. The FoS obtained are plotted in Figs. 14. From the plots the shear strength parameters for the colluvium are decided as c = 35 kPa and ϕ=35°.

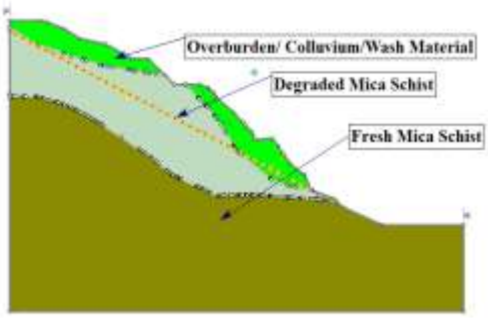

Fig. 13 Initial profile before excavation with colluvium up to certain depth

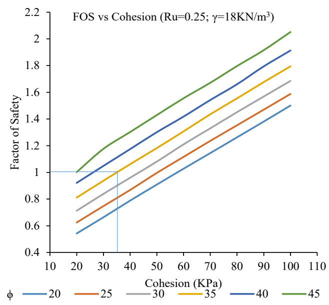

Fig. 14 FOS plots to obtain c & ϕ combination

4.2 Scenario 2: Natural Slope without overburden/colluvium

In this case, the colluvium/overburden was removed from the model, as was the case at the time of instability occurrence. An excavation was made up to El 2298m. The model is shown in Fig. 15. Failure occurred in rock at this stage. The stereographic projection of the joints is shown in Fig. 16. It is seen that there is possibility of planar failure along joint plane J1b. Few weak layers were introduced in the SLIDE model to simulate the joint plane (Fig. 15). Barton's JRC-JCS model was used with the following parameters: ϕ_r = 30.22±1.19°, JRC=11, JCS=26.14±12.4; Ru=2.0. The rock masses were assigned GSI of 40 (Degraded Mica Schist) and 50 (Fresh Mica Schist). Rock mass properties are presented in Table 2. Analysis was performed for dry as well as wet conditions for different Ru values. The results are presented in the Table 3.

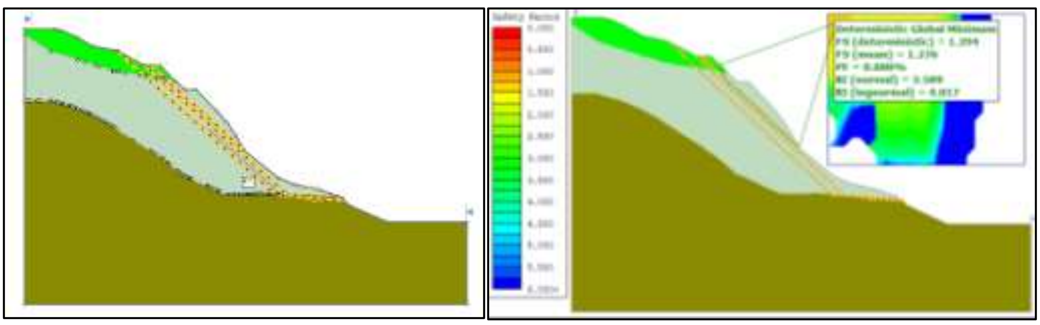

Fig. 15 Slope geometry and joint details for degraded mica schist without overburden and probabilistic analysis results

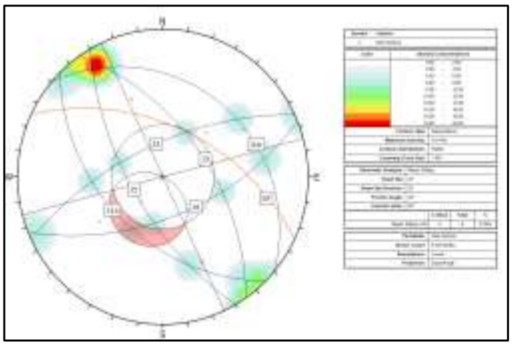

Fig. 16 Stereographic projection of joints

Table 2 Statistical parameters derived from the field data

Sl. No	Rock Parameters	Mean (μ)	SD (σ)	CoV	R^2	Best Fit Distribution
1	Intact rock tensile strength (MPa)	7.31	3.20	0.44	0.9556	Normal
2	Intact rock UCS (MPa)	50.82	27.95	0.55	0.9607	Log Normal &Weibull
3	Intact rock modulus (Ei) (GPa)	38.36	13.61	0.35	0.9409	Weibull
4	Rock mass or field modulus (Erm/Ee) (MPa)	617.29	292.1	0.47	0.9688	Normal & Weibull
5	Cohesion, c (MPa)	2.53	1.14	0.45	0.9456	Log Normal &Weibull
6	Friction angle, $\phi°$	42.27	7.79	0.18	0.7873	Log Normal &Weibull
7	Hoek Brown m_i	14.9	6.98	0.47	0.8420	Weibull & Gamma
8	JCS (MPa)	26.14	12.4	0.04	0.9349	Weibull
9	Residual friction angle (ϕ_r °)	30.22	1.19	0.47	0.9456	Normal &log Normal

Table 3 Probabilistic analysis results with varying Ru Values

Ru	0.0	0.1	0.2
FOS	1.232	1.048	0.869
Probability of failure	0.0	23.20%	100%

It is seen that the slope fails for the condition represented somewhere between Ru= 0.1 and 0.2. This condition agrees with the field condition and the failure occurred just after a rainfall.

The Fig. 11 shows the complete data set of direct shear tests performed on joints. The upper and lower bound failure envelopes are also given in the plot. The failure envelope corresponding to the shear strength parameters used in this analysis is found to be in between lower and upper bounds. This failure envelope indicates that statistical analysis provides a better understanding of the shear strength parameters. The shear strength parameters, should, therefore be obtained from statistical analysis and probabilistic analysis should be performed for simulating realistic situations.

5.0 CONCLUDING REMARKS

Rock masses in Himalayas, in general, are inherently inhomogeneous and show large scatter in their shear strength characteristics. In present article, a comprehensive appraisal of rock mass properties for a project located in greater Himalaya, was carried out. Instabilities were observed at the left bank slopes which were attributed to presence of joints in the rock mass. The instability conditions were simulated by considering statistically analysed properties and performing probabilistic slope stability analysis. Back analysis was done to arrive at the properties of the colluvium. Barton's non-linear JRC-JCS shear strength model for rock joints was used to model the shear strength of the joints. It was observed that statistically analysed shear strength parameters simulated field conditions close to what was observed in the field. It is highly recommended that statistical analysis of the rock mass properties be done using adequate field and laboratory testing. Probabilistic methods of slope stability analysis should be used for analysing the slopes. The back analysis should be integral part of the design and analysis as the construction progresses. The design parameters should be updated continuously based on analysis of actual ground conditions.

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REPAIR AND STRENGTHENING OF MASONRY AND CONCRETE STRUCTURES IN SUBMERGED OR UNDERWATER CONDITIONS – AN ENVIRONMENTALLY AND SOCIALLY SUSTAINABLE APPROACH TO HYDRO DEVELOPMENT

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ABSTRACT

Hydraulic structures such as dams, barrages, bridges, jetties, and tunnels are vital assets in hydro infrastructure, often exposed to partial or complete submergence. Over time, they suffer deterioration due to cavitation, erosion, and chemical attack. Conventional repair methods involving dewatering or cofferdam construction not only increase project costs and downtime but also impose significant ecological and social impacts.

This paper presents a sustainable framework for environmentally responsible underwater repair and strengthening of concrete and masonry structures, developed through field experience and practical case studies. By employing in-situ underwater rehabilitation—without reservoir drawdown—projects have achieved 35–45% reduction in downtime, up to 25% cost savings, and substantial mitigation of environmental disruption.

The use of eco-friendly polymer-modified micro-concretes, low-toxicity epoxy systems, and performance-enhancing admixtures has achieved compressive strengths in the range of 40–60 MPa with superior bond characteristics under submerged conditions. These materials and methods significantly reduce carbon-intensive processes and construction waste.

The study concludes that sustainable underwater rehabilitation not only extends the service life of hydraulic assets by 15–20 years but also aligns with the UN Sustainable Development Goals (SDGs) by minimizing ecological disturbance, conserving water resources, and safeguarding community livelihoods dependent on hydropower and irrigation infrastructure.

INTRODUCTION

Hydro development projects are crucial for renewable energy, irrigation, and water resource management. However, their longevity and sustainability depend on maintaining the integrity of submerged structures exposed to harsh aquatic environments.

Concrete and masonry structures in submerged or tidal zones face accelerated deterioration due to mechanical, chemical, and biological factors. Repairing such structures underwater demands not only technical precision but also environmental sensitivity—ensuring interventions cause minimal ecological disruption and social inconvenience.

1. Key challenges include:

- Maintaining material cohesion and bond integrity under water flow,
- Ensuring the safety of divers and aquatic ecosystems,
- Adhering to sustainability principles in material selection and methodology.

Through innovative underwater rehabilitation techniques, it is possible to achieve durability and safety while promoting circular resource use, reducing carbon footprint, and preserving aquatic biodiversity.

2. CAUSES OF DETERIORATION

Common Factors Contributing to Submerged Structure Damage

- Hydraulic Forces: High-velocity flow, turbulence, and cavitation leading to material erosion.
- Chemical Attack: Sulphate, chloride, and carbonic acid reactions degrade cementitious matrices.
- Biological Growth: Algae, mollusks, and microorganisms induce bio-deterioration.
- Aging of Joints: Open or degraded joints cause seepage and progressive weakening.
- Construction Defects: Honeycombing, improper compaction, and poor-quality materials.

These degradation mechanisms not only affect structural performance but also jeopardize water resource efficiency, indirectly impacting local communities and ecosystems dependent on these assets.

3. CHALLENGES IN UNDERWATER REPAIR

- Restricted Visibility: Turbid water conditions limit diver visibility, requiring specialized lighting and sonar mapping.
- Accessibility: Deep or confined working spaces demand safe diver operations.
- Material Washout: Conventional mortars disperse in water, necessitating anti-washout formulations.
- Curing Conditions: Continuous water exposure affects setting and bonding.
- Safety & Environment: Diver safety and minimal aquatic disturbance are paramount.

An integrated approach balancing technical precision, safety, and sustainability is essential.

4. SUSTAINABLE UNDERWATER REHABILITATION FRAMEWORK

A four-phase methodology has been developed to ensure durability, safety, and environmental sustainability in underwater rehabilitation.

Phase 1: Inspection & Assessment

Comprehensive underwater surveys form the foundation of sustainable repair planning.

- Visual & ROV Inspection: Diver and ROV-based visual mapping of cracks, cavities, and scour zones.
- Acoustic Sonar Mapping: Used in low-visibility zones to detect hidden voids and surface wear.
- Non-Destructive Testing (NDT): Ultrasonic pulse velocity, rebound hammer, and corrosion potential mapping.
- Core Sampling & Laboratory Testing: Evaluation of strength, permeability, and chemical contamination.
- Geo-Technical Investigations: Electrical resistivity, streaming potential, and seismic tomography to locate seepage paths and weak zones.

This data-driven assessment avoids unnecessary structural dismantling, thereby minimizing waste and environmental disturbance.

Phase 2: Diagnosis & Technique Selection

Inspection data are analyzed to identify the root cause of damage and to select appropriate, eco-compatible repair techniques.

Decision Parameters:

- Type and extent of deterioration
- Flow and depth conditions
- Accessibility and safety
- Environmental sensitivity

Technique Selection Matrix:

Type of Damage	Recommended Sustainable Solution
Surface Erosion & Cavitation	Polymer-modified micro-concrete, epoxy putty, High strength cementitious mortar
Cracks & Joints	Low-VOC epoxy injection or cementitious grout
Section Loss	FRP or glass-fiber jackets with low-shrink grout
Leakage Control	Cementitious curtain grouting and polyurethane sealants

These methods minimize the use of high-carbon materials and avoid dewatering, thus conserving aquatic life. The materials also reflect a balance between structural efficiency and environmental responsibility

Phase 3: Execution

Execution involves precision-controlled operations ensuring material efficiency and reduced ecological impact.

- Surface Preparation: Removal of loose material and marine growth using hydro-demolition and water jet cleaning—without chemical contaminants.
- Material Placement: Application of anti-washout admixtures, tremie concrete systems, and diver-assisted placements to prevent segregation.
- Structural Strengthening: Installation of fiber jackets or wraps filled with eco-friendly grout or micro-concrete to restore integrity and load capacity.

This approach promotes resource optimization and minimal intervention repair, reducing carbon emissions and environmental footprint.

Phase 4: Monitoring & Performance Evaluation

Continuous monitoring ensures long-term sustainability.

- Post-Repair Inspection: Visual and ROV-based verification of repair quality and alignment.
- Performance Testing: Strength and permeability tests to ensure durability.
- Sensor-Based Monitoring: Embedded sensors track structural performance and water ingress, enabling predictive maintenance.

These measures enhance resilience and reliability of hydro assets while enabling data-driven sustainability reporting.

CASE STUDIES:

1. HIRAKUD DAM:

Fig. 1 : Hirakud Dam Image

1.1 Introduction

The Hirakud Dam, constructed across the Mahanadi River, is located approximately 10 kilometers from Sambalpur in Odisha, India. Recognized as the longest earthen dam in the world, it has a main dam length of 4.8 km (3.0 miles) and a vast catchment area of 83,400 km² (32,200 sq. miles). As a critical multipurpose structure, the dam supports irrigation, flood control, and hydropower generation, contributing significantly to regional water and energy security.

Given its age and continuous exposure to hydraulic forces, it is essential to maintain the dam's integrity through sustainable maintenance and strengthening practices that minimize ecological disturbance and extend the asset's life cycle.

1.2 Need for Strengthening

During an underwater inspection of the upstream substructure of the Hirakud Dam, significant cavities were identified beneath the sluice barrels/gates at the joints between the substructure and foundation, at a water depth of approximately 40 meters. Additionally, the left and right spillway blocks showed major defects, including cracks, honeycombing, and surface spalling.

These defects raised concerns regarding structural stability, seepage risk, and potential water loss, which could compromise dam safety and operational efficiency.

Following a detailed assessment, the Expert Dam Safety Committee recommended immediate repair and strengthening of the affected areas. Importantly, the committee emphasized conducting all activities underwater, maintaining the existing water level to ensure environmental sustainability and continuous reservoir operation.

This sustainable approach avoided large-scale dewatering, which would have caused ecological disturbance, affected aquatic life, and disrupted water supply and power generation.

1.3 Scope of Work

The project scope was designed to ensure precision-based, environmentally responsible rehabilitation of the dam's submerged components. The major activities included:

- Conducting detailed underwater precision mapping and high-definition video recording of critical zones exhibiting cracks, cavities, and other structural defects.
- Underwater repair and filling of large cavities, maintaining the integrity of the original structure.
- Underwater treatment of cracks, spalled areas, honeycombing, and smaller voids using eco-friendly and durable materials.
- Post-repair inspection to verify performance, quality, and adherence to safety and sustainability standards.

This methodology ensured minimal disturbance to the aquatic environment while achieving high-quality structural restoration.

1.4 Execution Methodology

The work was executed strictly as per the approved methodology and technical specifications, following sustainable and safe underwater engineering practices.

Underwater Inspection and Mapping

A systematic underwater inspection was conducted using a combination of divers and Remotely Operated Vehicles (ROVs).

- The mapping was performed on a 3 m × 3 m grid pattern, covering all critical areas of concern.
- The inspection data were accurately plotted on AutoCAD drawings, with 3D mapping of cavities and defects referenced to the dam's coordinate system.
- The cavities were found to extend up to 10 meters in length and 3 meters in depth.

This digital mapping approach reduced the need for intrusive physical exploration, thus supporting non-destructive, low-impact assessment.

Underwater Repair of Cavities

The underwater cavity treatment was executed using sustainable repair materials and low-impact construction methods, ensuring long-term durability and minimal environmental footprint.

Key Activities:

(a) Drilling:

51 mm diameter holes were drilled underwater up to 4 meters deep into the rock foundation, following precision alignment and minimal disturbance to surrounding areas.

(b) Anchoring:

High-strength 25 mm diameter steel bars were installed and anchored with corrosion-resistant materials up to 4 meters deep in the rock bed to restore structural continuity and load transfer.

(c) Reinforcement & Filling:

A reinforcement cage with helical steel was fixed within the cavity. Graded hard granite boulders were placed inside the cavity to reduce material usage and improve interlocking.

The cavity was then filled using anti-washout cementitious grout blended with eco-compatible admixtures, mixed and pumped through a controlled tremie process to ensure uniform filling, zero washout, and environmental safety.

This repair methodology effectively restored the foundation's strength and integrity while aligning with the principles of sustainable dam maintenance — reducing material waste, preventing contamination, and preserving the aquatic ecosystem.

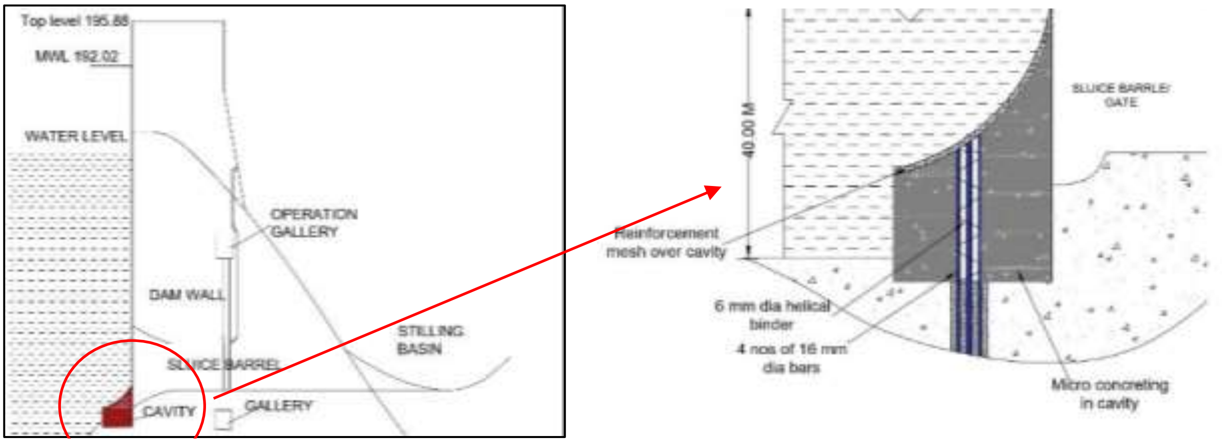


Fig. 2 : Underwater repairing of cavity

Underwater Repairing of cracks, spalls/ small cavities/ honeycomb areas:

(a) Cleaning and opening of cracks and other defects

(b) Drilling and fixing of 20 mm dia shear anchors

(c) Drilling and fixing of nozzle for Underwater epoxy grouting

(d) Sealing of defects with Underwater epoxy putty

(e) Underwater Epoxy Grouting through pre-fix nozzles.

Post inspection:

A post-inspection was conducted to assess the effectiveness of the treatment. This involved a visual Underwater inspection, a hammer test, and the extraction of core samples from the repaired area to evaluate the quality of the repair materials and workmanship. The core holes were then sealed with epoxy putty.

2. ALAKNANDA HYDRO POWER CHANNEL

Fig. 3 : Alaknanda Hydro Power Channel Image

2.1. Introduction:

The Alaknanda hydroelectric project is a 330MW hydropower generating station developed on the Alaknanda River in Uttarakhand, India.

2.2. Need of Waterproofing and strengthening:

Water leakage was observed on the downstream side of the embankment of hydropower channel with erosion of soil material. Underwater inspection revealed that the contraction joints had opened. Sealing these open joints was essential to minimize water loss and to stop erosion.

2.3. Scope of work:

The scope of Underwater works involves:

- Underwater inspection for identification of suction points, cracks and other defects.
- Underwater sealing of joints.

2.4. Execution:

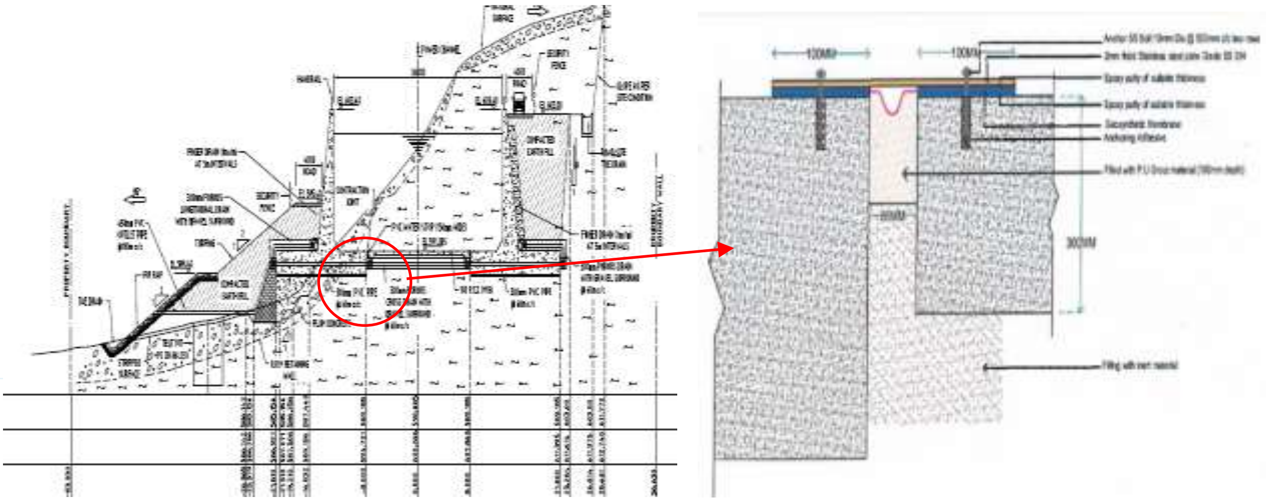


Fig. 4 : Section and Schematic representation

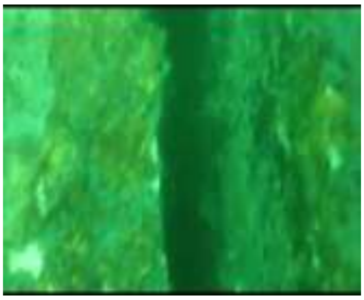
(a) Cleaning and removal of loose of foreign materials from the joints.

(b) Pouring of graded aggregates into the widened contraction joints.

(c) Injecting two component semi-flexible grout

(d) Fixing of geosynthetic membrane with epoxy putty

(e) Fixing of stainless steel over the geosynthetic membrane for protection.



Existing expansion joints

Sand and aggregate filling

Filling semi-flexible grout

Fixing geosynthetic membrane with epoxy putty

Drilling and fixing stainless steel plate

Fig. 5 : Repair Treatment Picture

2.5 Result:

- The work was carried out successfully, resulting in a substantial reduction in leakage.

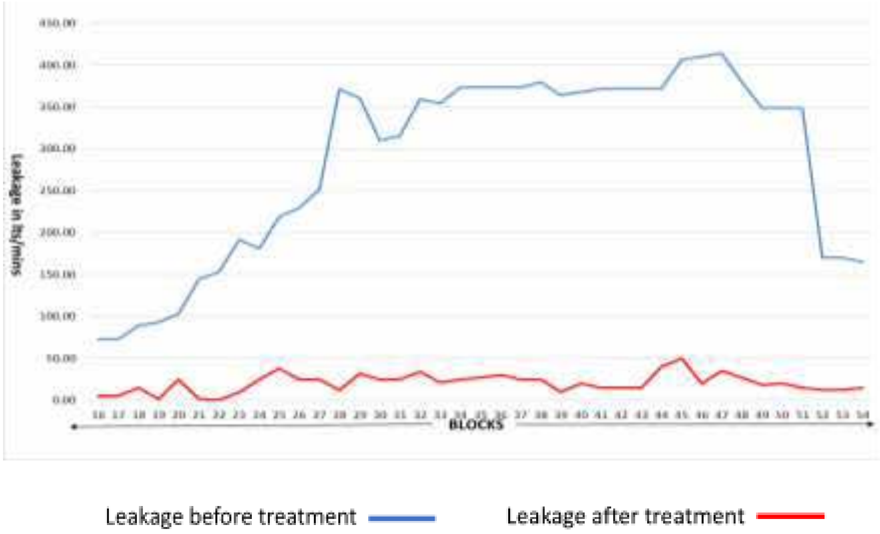


Fig. 6 : Graphical representation of reduction in leakage

3. NARAYANPUR DAM:

Fig. 7 : Narayanpur Dam

3.1 Introduction

The dam is constructed across the Krishna River in the Yadgir District of Karnataka. It has a height of 29 meters and extends 10 kilometers in length, with a catchment area of 47,850 km². This structure plays an important role in the region's water management system, contributing to irrigation, hydropower generation, and local ecosystem balance.

3.2 Purpose

The dam forms a vital component of the Upper Krishna Project, designed to support the agricultural and economic development of surrounding arid regions by providing a reliable source of irrigation water. In addition, it contributes to sustainable energy generation through hydropower, ensuring an efficient and renewable use of natural water resources.

3.3 Need for Waterproofing and Strengthening

During an underwater inspection conducted by professional divers, leakage was detected in the drainage gallery of the dam. Further, the masonry and concrete surfaces of the upstream face and spillway exhibited visible defects such as honeycombing, surface spalling, and deteriorated joints.

These defects were potential sources of seepage and strength reduction, which could affect the dam's long-term performance and safety. Therefore, the Dam Authority initiated a comprehensive waterproofing and strengthening program to:

- Arrest the observed leakages,

- Restore surface integrity, and
- Enhance the overall durability of the dam structure.

The approach focused on sustainable rehabilitation, ensuring structural safety while minimizing environmental impact.

3.4 Scope of Work

The scope of the project was planned to address structural and hydraulic issues through precision-based underwater and surface interventions, including:

- Underwater survey using ROV (Remotely Operated Vehicle) and diver-assisted inspection to locate and document defects.
- Underwater pointing of masonry joints using durable and fast-setting cementitious materials.
- Underwater resurfacing of damaged concrete areas to restore a smooth, watertight finish.
- Construction of a reinforced concrete retaining wall to provide additional stability and erosion protection.

This integrated approach ensured both structural restoration and environmental sustainability by avoiding large-scale dewatering and maintaining reservoir operations throughout the process.

3.5 Execution Methodology

The strengthening and waterproofing work was executed in accordance with approved methodology and technical specifications, ensuring quality, durability, and ecological balance.

(a) Underwater Masonry Joint Pointing

All defective masonry joints were raked out and cleaned to remove loose material and bio-growth. The joints were then filled with a fast-setting, polymer-modified cementitious mortar, specifically designed for underwater application. The joints were finished smoothly to ensure effective sealing against seepage.

(b) Underwater Resurfacing

Damaged or uneven concrete surfaces were prepared and cleaned before applying a polymer-based cementitious repair mortar up to 10 mm thickness. This layer restored the smoothness and waterproofing performance of the upstream surface, improving hydraulic efficiency and reducing erosion.

(c) Grouting for Strengthening

For deeper waterproofing and consolidation, holes were drilled, cleaned, and fitted with packers at pre-determined intervals. A high-strength, non-toxic, anti-washout, fast-setting grout mix—consisting of cement and crystalline admixtures—was injected through the packers.

This process effectively filled internal voids, reduced porosity, and improved the overall density and strength of the concrete and masonry sections, ensuring a long-term, sustainable repair.

(d) Construction of Retaining Wall

An RCC retaining wall was constructed as per the approved structural design, using RMC M30 grade concrete. This wall provided additional support to prevent soil erosion, improve drainage control, and enhance the dam's structural resilience against hydrostatic forces.

3.6 Materials Used

Material	Purpose / Application	Specification / Properties
Underwater Cementitious Mortar	Used for resurfacing and masonry pointing	Conforming to EN 1504, polymer-modified, fast-setting, anti-washout
Underwater High-Strength Cementitious Grout	Used for internal strengthening and waterproofing	Non-toxic, anti-washout, crystalline additive-based
RMC M30	Construction of RCC retaining wall	Durable, dense, high-strength concrete mix

This sustainable rehabilitation approach not only restored the dam's structural and hydraulic performance but also prolonged its service life with minimal ecological disturbance. By employing environmentally compatible materials and underwater techniques, the project ensured continued water management efficiency while upholding environmental stewardship.

3.7 Repair Treatment Pictures:

Underwater pointing

Construction of Retaining wall

3.8 Result:

(a) The leakage in the drainage gallery caused by suction points on the upstream face has been arrested.

(b) Strengthening of the masonry joints and concrete surface.

(c) Protection of Soil Erosion through construction of Retaining wall.

CONCLUSION

The underwater rehabilitation of hydraulic structures, when implemented using sustainable materials and advanced methodologies, plays a vital role in enhancing structural integrity while protecting the environment and community welfare.

By eliminating the need for dewatering, minimizing material waste, and adopting eco-friendly repair technologies, such projects exemplify the principles of sustainable hydro-infrastructure development.

The demonstrated outcomes a 35–45% reduction in project downtime, approximately 25% cost savings, and an extended service life of 15–20 years, clearly establish underwater strengthening as a technically efficient, economically viable, and environmentally responsible approach to dam maintenance and rehabilitation.

ENHANCING CYBER RESILIENCE IN INDIA'S HYDRO POWER SECTOR: GOVERNMENT OF INDIA'S APPROACH IN SAFEGUARDING AUTOMATED AND DIGITALIZED HYDROPOWER PLANTS THROUGH CSIRT-POWER, MINISTRY OF POWER

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ABSTRACT

India has set ambitious targets for expanding renewable energy portfolio with hydro power expected to play a pivotal role. As of August 2025, India's installed capacity under Hydro domain stands at 50.1 GW with a renewable installed capacity of 242.6 GW and total installed capacity of 495.5 GW. This figure underscores the importance of hydropower which is supporting one-fifth of total renewable capacity. At a humble 10.11% of total installed capacity, Hydro power sector contributes significantly to India's installed capacity, helping the country with both energy security and sustainable economic growth. This ratio of hydro energy in total energy mix is closely following the global figure of 14%. India's position in this sector is further strengthened because of its strategic initiatives and strong partnerships, especially with neighbouring nation of Bhutan.

India and Bhutan have established a strong cross-border electricity trade framework through successful joint hydropower ventures. With prominent projects like Tala (1,020 MW) and Mangdechhu (720 MW) already in place, Indo-Bhutan partnership boasts a commissioned capacity of around 2262 MW. New projects such as Punatsangchu-I & II and Kholongchhu amounting to capacity of around 2820 MW, are also planned to increase cross-border clean energy exchange.

Hydropower sector in India encompasses a diverse mix of large and small hydroelectric stations, pump storage plants, and connected dams, turbines, and control systems. However, geographical location of these plants in remote, hilly areas, presents operational and management challenges. Also, these plants are spread out across wide river basins, therefore centralized control, regular maintenance, and ensuring security are not easy tasks.

To tackle these issues, the sector has been continuously undergoing rapid automation and digitization. The advanced technologies such as IoT sensors, automated SCADA systems, and remote-control features, early warning system, the operation of hydro plants has become safer, more efficient and reliable. This adaptation of technology has enabled 24x7 monitoring, predictive maintenance, and optimized management of hydro energy with ease.

However, the digital transformation of hydro power plants has also brought significant cyber risks. Legacy control systems lacking robust security, gaps in patch management, and convergence of OT and IT environments have expanded the attack surface, exposing vital assets to threats such as ransomware, Advanced Persistent Threats (APT), data tampering, and unauthorized remote access. Notable incidents globally—like the targeted malware attacks on dam SCADA systems—highlight the sector's vulnerability. The nature of operational technology (OT) makes these risks more acute, as cyber incidents can result in physical consequences including grid instability and public safety hazards.

To address these emerging challenges, the Government of India has established a dedicated Computer Security Incident Response Team for Power Sector (CSIRT-Power) under Ministry of Power in September 2024 to act as a nodal cybersecurity agency for India's power sector. CSIRT-Power coordinates incident response, disseminates threat intelligence, develops sector-specific cybersecurity frameworks, and spearheads capacity building initiatives tailored for power sector utilities, including those in the hydro domain.

CSIRT-Power has implemented both preventive/proactive and responsive/reactive measures to bolster the security posture of hydro plants. These include assessment of risks targeting hydro facility OT systems, standard operating procedures for incident detection and containment, advisories on critical vulnerabilities in hydro-digital systems, and simulations of attack scenarios to build operator capacity. The team also fosters collaborative initiatives at the national level, such as sector specific reference architecture, information-sharing platforms such as Information Sharing and Analysis Centre (ISAC) platform for emerging threats, and continuous evaluation of emerging digitization risks. The complexity and strategic importance of India's hydro sector make it imperative for CSIRT-Power to remain at the forefront, anticipating both opportunities and challenges posed by ongoing digitization.

This paper presents an overview of India's hydro power sector, analyses the unique cyber risks associated with digitization and IT-OT integration, and articulates CSIRT-Power's holistic approach to safeguarding critical hydropower infrastructure now and into the future.

***Keywords**: CSIRT-Power, Automation, Digitalization, Information Technology (IT), Operational Technology (OT), Cyber Resilience, early warning system, APT, ISAC, Incident response, Grid security, Reference Architecture*

1.0 INTRODUCTION

Hydro power plants are a backbone for India's electricity grid and a major player in global renewable energy. They provide clean, reliable, and flexible electricity, helping to balance supply and demand, especially as more solar and wind energy are added to the grid. Pumped storage plants are also very important because they act as large-scale water batteries: storing excess electricity and supplying it when needed. India's hydroelectric generation is predominantly driven by the public sector (≈92.5%), led by major utilities such as SJVNL, NHPC, NEEPCO, THDC, and NTPC-Hydro. The private sector also plays a growing role, especially across the Himalayan and North-Eastern regions.

1.1 Hydro Power Installed Capacity

- India: As of June 2025, India's installed hydropower capacity is about 54,480 MW, including both large and small hydro projects.
- Global: Worldwide, hydropower has over 1,250 GW installed capacity (which is 14% of global electricity supply). Pumped storage accounts for around 189 GW globally.

1.2 India-Bhutan Hydropower Partnership

India and Bhutan have built a strong partnership in the hydropower sector, setting an example for international energy cooperation in the region. Over decades, this collaboration has led to major capacity additions and successful bilateral projects.

- Five major hydropower plants, including Chukha, Kurichhu, Tala, Mangdechhu, and Punatsangchhu-II, have been jointly constructed, contributing more than 3,150 MW to Bhutan's installed capacity.
- New projects, such as the 1200 MW Punatsangchhu-I, are under active development, and private sector collaborations are expanding into hydropower and solar.
- Electricity trading between India and Bhutan is dynamic, Bhutan imports power during lean seasons and exports surplus to India, strengthening energy security for both countries.

1.3 Importance and Benefits

Hydropower offers dependable, round-the-clock electricity, energy storage, and rapid response to grid fluctuations. Pumped storage plants (PSPs), in particular, act as water batteries, enabling excess renewable energy to be stored and swiftly released during peak demand, essential for a grid with high shares of intermittent renewables. PSPs enhance grid stability, reduce curtailment of renewables, and provide ancillary services such as frequency regulation and black start capability.

Hydropower's significant economic and social benefits include:

- Long plant lifespans (often 40+ years)
- Low operational costs after initial investment
- Job creation (the sector globally supported 2.3 million jobs in 2024)
- Regional development and water management.

1.4 Measures taken by Ministry of Power, India to Promote Hydropower

To strengthen India's hydropower sector and encourage new investments, the Ministry of Power (2019) announced multiple policy measures, which are now shaping project development and integration with the national renewable energy mix.

- Recognition as Renewable Energy: Large Hydropower Projects (LHPs, above 25 MW) are officially classified as renewable energy sources, helping them contribute to India's clean energy targets.
- Hydropower Purchase Obligation (HPO): HPO is now a separate category in the Renewable Purchase Obligation. State utilities must include hydropower in their renewable purchases, improving market demand for this sector.
- Tariff Rationalization: Tariff policies are updated to make hydropower competitive. Project life for tariff calculations extended to 40 years, debt repayment to 18 years, and tariff back-loading introduced, allowing flexibility and affordable rates for DISCOMs and developers.
- Budgetary Support for Infrastructure: Direct government grants support flood moderation components for new storage projects, and partial grants for essential infrastructure (roads, bridges), accelerating project timelines especially in remote/hilly regions.

1.5 Components of a Hydro Power Plant

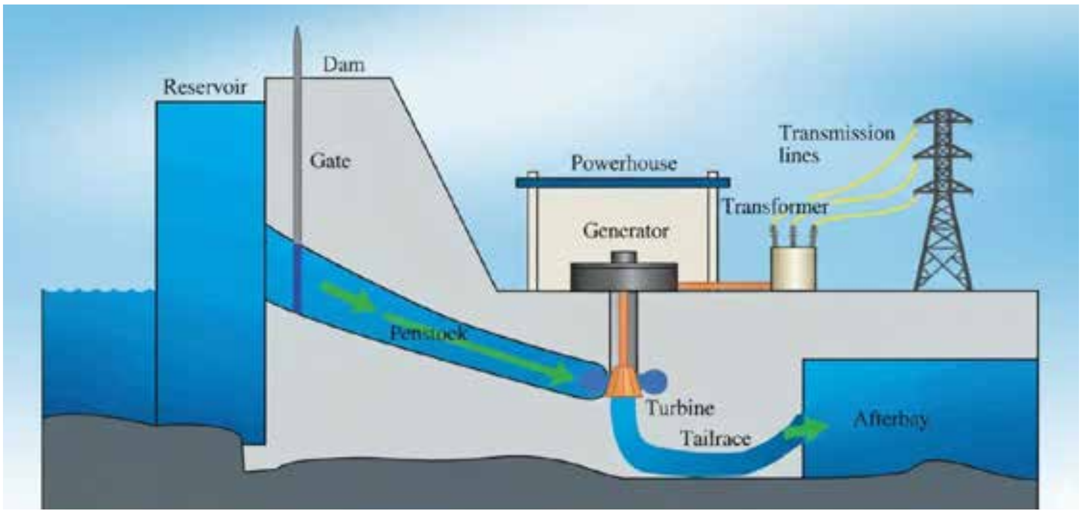


Fig 1 : Components of Hydro Power Plant

A hydro power plant works by using the potential energy of water collected behind a dam. The dam blocks the river to build a reservoir and stores water at a height. Water flows from the reservoir through intake gates and is temporarily held in the forebay before going through the penstock, a strong pipe built for holding water under high pressure. The fast-moving water rotates the turbine blades, converting water's energy into mechanical motion, which is then converted to electricity by the generator connected to the turbine shaft. All main machines are housed in the powerhouse, which also has cranes for repairs and switchboards for control. The generated electricity is sent to transformers, which raise the voltage so power can travel long distances. Used water is sent back to the river via the tailrace, and a draft tube helps improve efficiency for reaction turbines. Hydro plants use SCADA and automation for monitoring, while auxiliaries like batteries, lighting, lifts, and ventilation keep everything running smoothly.

1.6 Pumped Storage Hydro Plants

Pumped storage hydropower is like a big water battery, and it's the most used energy storage technology in the world. Nearly all of the world's long-term energy storage, about 200 GW, comes from this method. The best part is, the same water can be used again and again, making it easy to charge whenever needed.

These plants can store and supply a large amount of electricity and run for many hours. This is important because they can give steady power during busy times when everyone needs more electricity. They are also very flexible, quickly increasing or decreasing their output as required. When we use more solar and wind power, which sometimes gives unpredictable electricity, pumped storage helps keep things stable. It balances the supply and demand, which becomes even more important as the use of electric power grows everywhere.

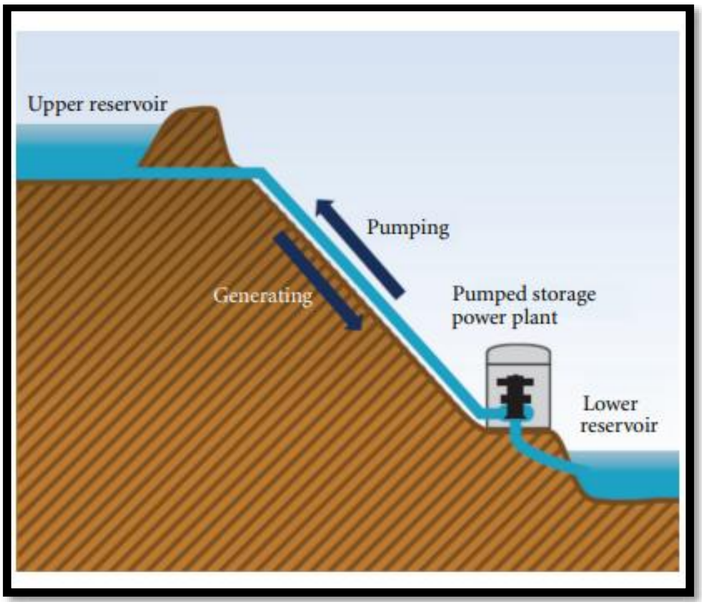


Fig 2 : Components of Pump Storage Plants

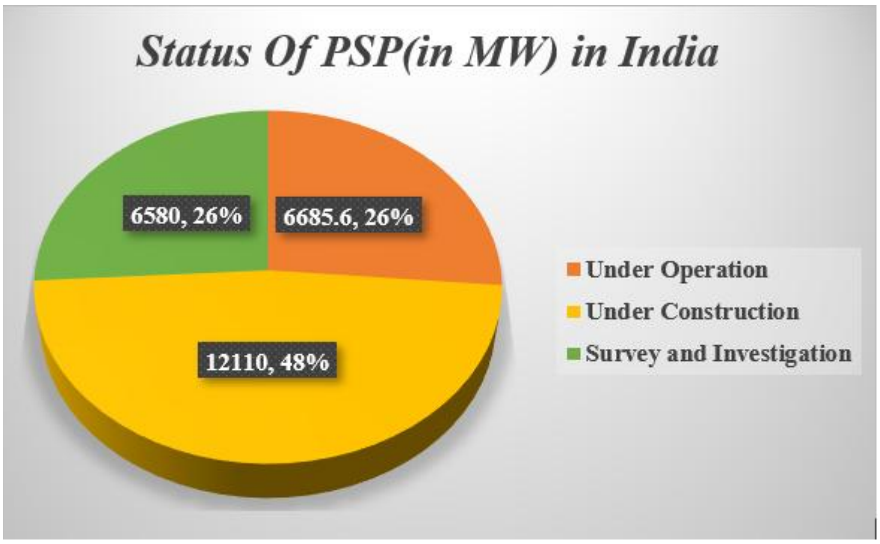


Fig 3 : Status of PSP (in MW)

Table 1 : In Operation PSPs in India (Above 25 MW Capacity)

Sr No.	Utilities/Stations	Developer	State	Capacity (MW)	Year of commissioning
1	Ghatgarh	MAHAGENCO	Maharashtra	250.00	2008 (250 MW)
2	Bhira	Tata Power Company	Maharashtra	150.00	1927 (125 MW) 1949 (25 MW)
3	N J Sagar	TSGENCO	Telangana	705.60	1980 (100.8 MW) 1981 (100.8 MW) 1982 (100.8 MW) 1983 (100.8 MW) 1984 (100.8 MW) 1985 (202 MW)
4	Srisailam LBPH	TSGENCO	Telangana	900.00	2001 (300 MW) 2002 (450 MW) 2003 (150 MW)
5	Kadamparai	TANGEDCO	Tamil Nadu	400.00	1987 (100 MW) 1988 (200 MW) 1989 (100 MW)
6	Purulia	WBSEDCL	West Bengal	900.00	2007
7	Kadana	GSECL	Gujarat	240.00	1990 (120 MW) 1998 (120 MW)
8	Sardar Sarovar RBPH	SSNNL	Gujarat	1200.00	2005 (800 MW) 2006 (400 MW)
9	Tehri	THDC	Uttarakhand	500.00	2025 (500 MW)
10	Pinnapuram	Greenko AP01 IREP Private Limited	Andhra Pradesh	1440.00	2025 (1440 MW)
			Total	**6685.60**	

2.0 DIFFICULTIES OF HYDRO POWER PLANTS

Hydro plants in India and globally often face several operational and logistical difficulties mainly due to their remote locations and decentralized nature. These difficulties impact the development, running, and maintenance of hydro power projects, especially in hilly or forested terrains.

2.1. Remote Locations: Challenges and Impact

- **Site Accessibility**: Most hydro projects are located mountainous regions. Difficult terrain and poor road or rail infrastructure makes transportation of heavy equipment and skilled manpower tough. Building transmission lines for power evacuation is expensive and time-consuming due to mountainous or forested land.
- **Geological Surprises & Natural Hazards**: Hilly regions, especially the Himalayas and North-East India, frequently face earthquakes, landslides, and floods. Unpredictable geological conditions (unstable soil, shifting mountain slopes) slow construction and increase costs.
- **Limited Local Infrastructure**: Lack of hospitals, schools, and markets in remote areas makes it difficult to attract skilled workers and contractors. Setting up basic facilities increases costs and extends project timelines.
- **Transmission Issues**: Evacuating power from isolated sites needs long transmission lines, adding financial and operational burdens.
- **Security & Local Disputes**: Projects sometimes face blockades or unrest from local communities or neighbouring states during equipment transport or land acquisition.

2.2. Decentralized Operations: Challenges and Impact

- **Spread Out Assets**: Hydro plants are often built on river networks with assets spread across wide areas (dams, tunnels, canals, powerhouses). Operating such decentralized infrastructure needs more skilled staff, regular patrolling, and greater coordination.

- **Grid Integration**: Managing multiple small hydro plants or mini-grids is complex. Decentralized feed-in can cause voltage fluctuations and requires modern control systems to maintain stable frequency and grid quality.
- Technical Challenges: Sudden changes in river flow, seasonal water variability, and unexpected silt or debris can affect plant efficiency and require advanced planning and automatic monitoring.
- **Cybersecurity Risks**: Increased digitalization and wide area management mean more network connections, making decentralized hydro assets more vulnerable to cyberattacks or communication failures.
- **Operations and Maintenance**: Decentralized operation makes routine maintenance and emergency response tough, as vehicles, materials, and manpower must travel longer distances, sometimes during floods or landslides.

2.3. Cost and Delays

- **High Upfront Investment**: Because of remote location, complex design, tough terrain, and the need to build extra infrastructure (roads, bridges, transmission lines), hydro projects have greater capital costs than most other power sources.
- **Time Overruns**: Projects face long delays in land acquisition, resettlement, environmental clearances, and local agitations, leading to cost escalations and financial risks. Environmental issues (like flash floods or rehabilitation protests) can add years to implementation.

3.0 ROLE OF DIGITIZATION IN HYDRO POWER PLANTS

Digitization is making hydro power plants smarter, safer, and more efficient. It involves the use of advanced technologies such as sensors, automation, central control systems (like SCADA), artificial intelligence (AI), and data analytics. These digital tools help plants operate smoothly, quickly spot problems, and enable remote, centralized operation—bringing many improvements in reliability, maintenance, and cost savings.

3.1 Centralized Operation and Remote Monitoring

- **SCADA Systems**: A modern hydro plant uses Supervisory Control and Data Acquisition (SCADA) systems to monitor and control turbines, gates, generators, and safety alarms from a central control room, even if the plant is far away or assets are spread out. Operators don't need to be everywhere; instead, they get real-time updates, alerts, and control options on their screens. SCADA enables remote operation, fast decision-making, and safer conditions, especially for decentralized or difficult-to-reach sites.
- **Sensors and IoT**: Sensors are installed on machines, pipes, turbines, and even dams to track wear and tear, temperature, flow rates, and more. All these readings are collected and analysed to keep the plant running at its best and alert staff before any breakdowns happen.
- **Digital Twins and Predictive Analytics**: Advanced hydro plants use digital twins, virtual models that simulate the entire plant's operations. Operators can run simulations, predict maintenance needs, and optimize electricity generation based on data trends, weather, or water conditions.

3.2 Impact on Operations and Efficiency

- **Improved Asset Management**: Operators can track the health of every turbine, generator, and vital asset. Maintenance schedules move from "fix after failure" to "predict and prevent" because digital systems can foresee possible faults, helping avoid sudden outages.
- **Operational Efficiency**: Digital controls help maximize electricity output, balance river flows, and manage reservoir levels even when conditions change quickly. Software tools optimize when plants run to make the most money or supply energy at the time of highest need.
- **Reduced Cost and Downtime**: Automation and remote monitoring reduce the need for staff onsite, especially in sites with risks of landslides or floods. Such systems speed up fault detection, cut down on unnecessary repairs, and reduce the cost of operations and maintenance.
- **Grid Integration and Flexibility**: Real-time data helps grid operators match supply with demand and balance electricity coming from other renewables like wind and solar.

3.2 Digitalization in Indian Hydro Sector

India's hydro sector is rapidly adopting smart SCADA systems, AI, and IoT to streamline nationwide hydro plant operations and reduce transmission losses. Remote monitoring has become vital for plants in remote and hilly regions. Government schemes like "Smart Cities Mission" and "Digital India" are encouraging the shift to central control and automated management.

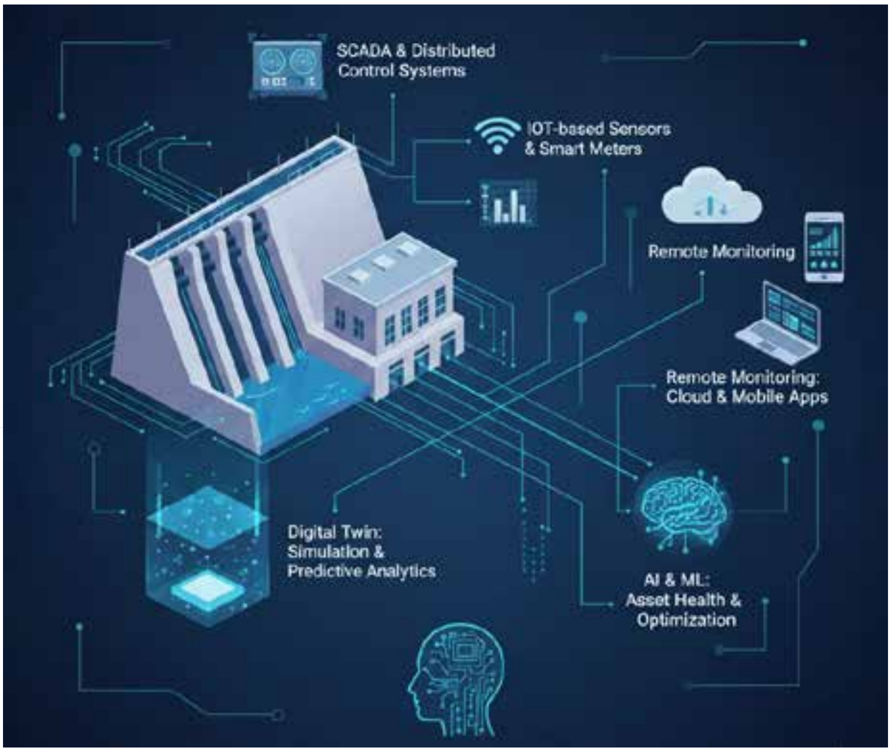


Fig 4. Digitization in Hydro Sector

4. CONS OF DIGITIZATION IN HYDRO POWER PLANTS

Digitization brings modern tools, efficiency, and remote control to hydro plants—but it also increases exposure to cyber risks. As old systems get digital upgrades and more devices are connected, the risk of cyber incidents grows, especially in operational technology (OT), which manages turbines, gates, and electricity supply.

4.1 Key Risks from Digitization

- **Expanded Attack Surface**: Every new sensor, IoT device, or automated system connected to the network opens up more paths for cyber criminals. If not protected, hackers can exploit devices, software flaws, or weak passwords.
- **Legacy Systems**: Many hydro plants have old control systems not designed for today's cyber threats. Updating these systems is complicated, and sometimes security measures are left out or ignored. Default passwords, missing patches, and unsecured remote access are common problems.
- **OT-IT Convergence**: Before digitization, OT systems were separate (air gapped) from IT networks, making remote hacking difficult. But as plants integrate data, remote monitoring, and cloud tools, attackers can target OT via IT pathways, potentially disrupting turbines, gates, or even dam safety mechanisms.
- **Data Tampering**: Cyberattacks may target sensors or control signals, change readings and cause unsafe decisions about water flow, equipment operation, or grid frequency. Such fake data can lead to dam breakdowns or unnecessary shut-offs.
- **Remote Access Threats**: Centralized control allows remote operations, but if login credentials are weak or systems unpatched, hackers can hijack plant controls, lock out operators, or trigger shutdowns.
- **Ransomware and Malware**: Malware can encrypt plant databases, lock out SCADA controls, or plant fake alarms, crippling operations until a ransom is paid. Example: In 2020, a ransomware attack hit a U.S. hydropower plant, forcing an operational shutdown and causing massive losses.
- **Real World Consequences**: Cyber incidents in hydro plants can have serious impact—blackouts, loss of water supply, flooding, environmental hazards, and financial costs. The May 2021 ransomware attack on the Colonial Pipeline led to millions in damage and widespread disruption.

4.2 Case Studies: Cyberattacks on Hydropower

Digitized hydropower plants face increasing risk from cyberattacks targeting their Operational Technology (OT) systems, such as SCADA controls, turbines, and dam gates. Here are some examples drawn from real-world incidents:

Table 2 : Cyber-Attacks on Hydro Sector

Incident Name	*Description*	*Reference*
Norway Lake Risevatnet Dam Breach	In April 2025, unidentified hackers breached the hydroelectric dam's control system (HMI) by exploiting a weak password on a web-accessible interface. The attackers gained real-time process control and remotely opened a water discharge valve to 100% capacity for approximately four hours. No physical damage occurred, but the event demonstrated successful manipulation of physical processes.	https://www.radiflow.com/radiflow-labs/inside-norway-2025-dam-cyberattack-radiflow/
Norsk Hydro Ransomware Attack	In March 2019, this global renewable energy company (which operates significant hydropower plants) was hit by the LockerGoga ransomware. The attack crippled the IT systems across 170 locations in 40 countries, forcing production lines in various facilities (including its Extruded Solutions division) to revert to manual, pen-and-paper operations for weeks. The estimated cost was around $70 million.	https://www.dnv.com/cyber/insights/articles/frontline-insights-the-norsk-hydro-cyberattack-a-reflection-on-the-importance-of-securing-digital-identities/
Korea Hydro & Nuclear Power (KHNP)	In December 2014, KHNP, which operates major power facilities including hydropower, suffered a breach initiated by spear-phishing emails sent to thousands of employees. The attackers stole non-critical but sensitive data, including designs and manuals for nuclear reactors, and posted them online. While physical operations were not impacted, the incident exposed the vulnerability of employee systems as an initial access point for critical infrastructure.	https://www.controleng.com/throwback-attack-korea-hydro-and-nuclear-power-highlights-the-vulnerability-of-critical-systems/

4.3 Key Takeaways

- Cyberattacks forced plant operators to run plants manually for weeks, highlighting the risk when OT systems are disrupted.
- SCADA system vulnerabilities (default passwords, remote access) can enable attackers to directly manipulate hydro plant controls.
- Adequate segmented network security, anomaly detection, and regular cyber drills are critical for OT resilience in hydropower.

5.0 MINISTRY OF POWER MEASURES AND APPROACHES

5.1 MoP/CEA Approach

To mitigate these cyber security challenges some of the major steps taken by MoP/CEA.

(a) CEA has issued "CEA (Cyber Security in Power Sector) Guidelines 2021."

(b) CSIRT-Power has been established by MoP and fully functional w.e.f September 2024. This is first sectoral 24*7 SOC of any civil sector in Govt of India. CSIRT-Power is having Security Operation Centre, Data Centre equipped with cutting edge technology and expert manpower, along with a dedicated cyber forensic lab which helps to prevent, detect, handle, and respond to respond to cyber security incidents in Power sector utilities.

(c) MoP has also created 6(six) Sectoral CERTs namely Thermal, Hydro, Transmission, Grid Operation, RE and Distribution for ensuring cyber security in Indian Power Sector.

(d) Capacity building exercises in collaboration with academia, industry, premier cyber security organizations of India.

(e) CEA is coming out with Cyber Security Regulations, Model Contractual Clauses and Trusted Vendor Scheme in Power Sector.

5.2 CSIRT-Power

In response to the escalating cyber threats targeting India's critical infrastructure, the Ministry of Power (MoP) established the Computer Security Incident Response Team for the Power Sector (CSIRT-Power). This initiative was formalized through MoP's notification F.No. 1/32/2021/IT&CS (258359) dated April 5, 2023, following the recommendations of the Empowered Committee on Cyber Security constituted in 2021 under the chairmanship of the Secretary (Power).

Key Activities of CSIRT-Power: -

- Laying down the Cyber Security framework and protocol for the Power Sector
- Laying down Standard Operating Procedures for Cyber Security
- Reviewing the Cyber Security arrangements in the different wings from time to time and strengthening such arrangements
- Implementation of Trusted Vendor system
- Drafting of Model Contractual Clauses for Cyber Security
- Analysis of Cyber Security Incidents
- Cyber Security Testing
- CISO Workshop
- Analysis, follow-up & action on Alert and Advisories, issued by NCIIPC, CERT-In & MHA
- Capacity development for using Vulnerabilities Scanning tools.
- Security Control Selection & Tailoring Process
- Vetting of Cyber Test bed Proposals.

5.3 Initiatives/ Activities of CSIRT-Power

5.3.1 *CEA (Cyber Security in Power Sector) Guidelines, 2021*

CEA has come up with Cyber Security in Power Sector) Guidelines in October 2021, since many cyber security directives and guidelines existed in Indian scenario, but none of them were power sector specific. After the issuance of CEA cyber-Security guidelines 2021, the basic cyber security framework of power sector utilities has enhanced such as appointment of CISO & Alt CISO, onboarding on CSK of CERT-In, regular cyber security audit by CERT-In empanelled auditors, Cyber security trainings, ISO 27001 certification etc.

Table 3 : 14 Articles of CEA Cyber Security Guidelines

Article No.	Description	Use of Article
1	***Policy for Cyber Security***	Establishes the foundational principles and regulations for formulating a cyber security policy.
2	***Appointment of Chief Information Security Officer (CISO)***	Mandates the designation of a CISO to oversee and implement cyber security measures.
3	***Identification of Critical Information Infrastructure (CII)***	Outlines procedures for identifying assets critical to national security and ensuring their protection.
4	***Electronic Security Perimeter (ESP)***	Provides guidelines for setting up ESPs to safeguard critical cyber assets from unauthorized access.
5	***Cyber Security Requirements***	Specifies the necessary conditions and standards to establish robust cyber security frameworks.
6	***Assessment of Cyber Risk and Mitigation Plan***	Details the process for evaluating cyber risks and developing corresponding mitigation strategies.
7	***Phasing Out of Legacy Systems***	Encourages the replacement or upgrading of outdated systems to enhance security and compatibility.
8	***Cyber Security Training***	Emphasizes the importance of regular training programs to build cyber security awareness among personnel.
9	***Cyber Supply Chain Risk Management***	Provides guidelines to manage and mitigate risks associated with third-party vendors and supply chains.
10	***Cyber Security Incident Reporting and Response Plan***	Establishes protocols for reporting cyber incidents and outlines response strategies to address them effectively.
11	***Cyber Crisis Management Plan (C-CMP)***	Mandates the development of a comprehensive plan to manage and recover from cyber crises.
12	***Sabotage Reporting***	Details procedures for reporting and addressing acts of sabotage within cyber systems.
13	***Security and Testing of Cyber Assets***	Outlines requirements for regular testing, including vulnerability assessments and penetration testing.
14	***Cyber Security Audit***	Specifies the frequency and standards for conducting cyber security audits to ensure compliance and effectiveness.

5.3.2 Incident Response & Forensic

CSIRT-Power Team has been 24*7 actively involved in doing the analysis of the cyber security incidents reported to Utilities by Central Agencies like CERT-In/ NCIIPC/ Intelligence Bureau/TSOC/CSK etc.

Incident response team and Forensic team has been established at CSIRT-Power.

The CSIRT Power Team collects the logs, does the forensics analysis for the purpose of RCA (Root Cause Analysis) using limited resources available with it. Incident wise report is also prepared with recommendations made for closure of identified gaps. The involvement of CSIRT-Power brings in sector specific insight thus helps in better analysis of incident.

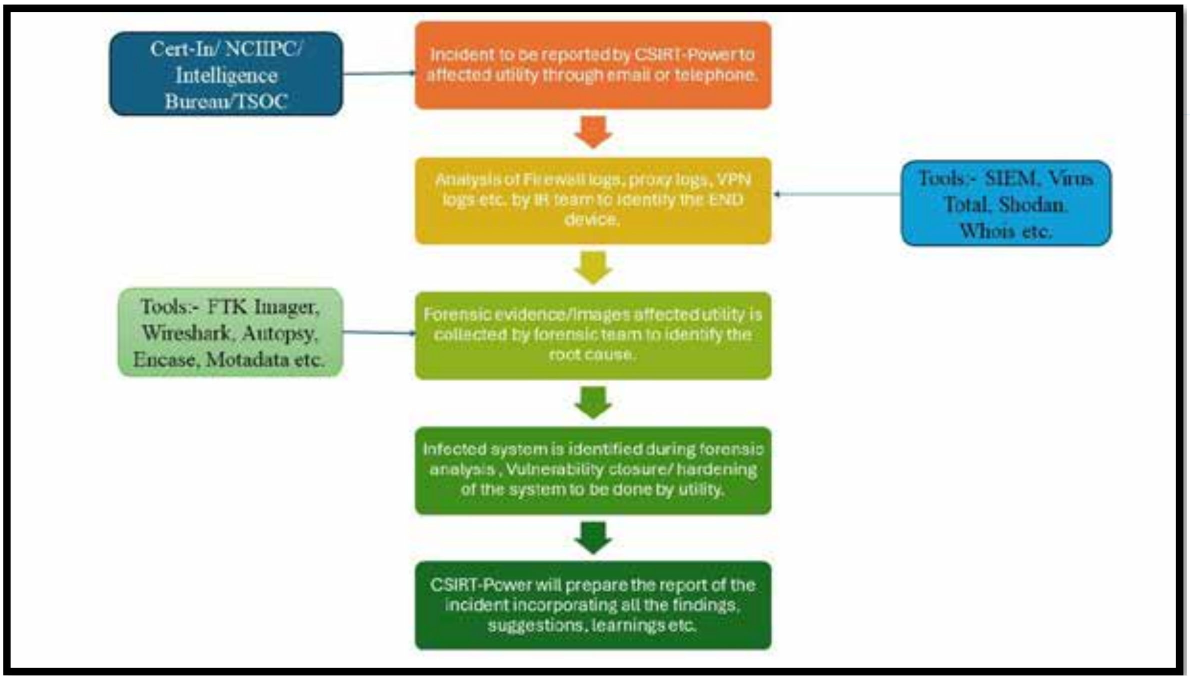


Fig. 5 : Incident Response and Forensic Team workflow

5.3.2.1 *Use Case: Website Defacement – A Real-World Cyber Incident Response*

Power sector utility reported a website defacement incident on its public-facing website.

Initial Investigation & Coordination

- The defaced webpage was immediately taken offline to prevent further damage.
- Injected files and a full website dump were asked for forensic analysis.
- Firewall, proxy, and application logs were asked for investigation.

Technical Analysis of CSIRT-Power

- Output of a malicious web shell (or injected PHP file), commonly referred to as a “backdoor”, which may be executed via a PHP file (door.php) uploaded to a web server. It’s an interactive command-line interface (CLI) through a browser. This was an HTML + JavaScript frontend that interfaces with a PHP backend (door.php). It allows the attacker to execute shell commands on the server via a web browser, effectively giving them remote command-line access.
- Malicious Script Injection

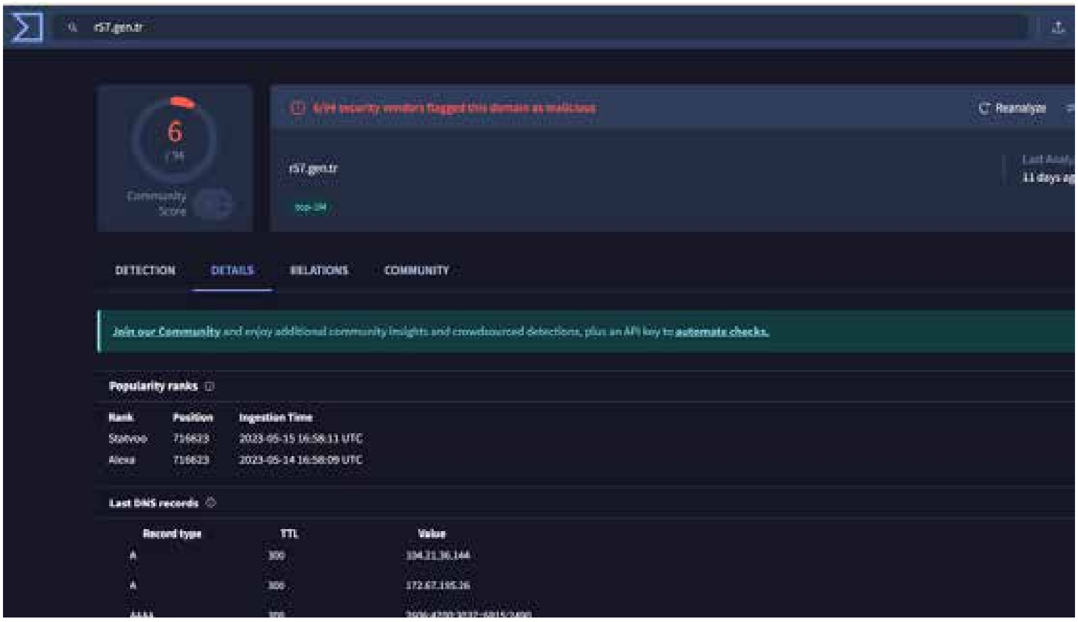
  <SCRIPT SRC=”http://r57.gen.tr/yazciz/ciz.js”>

  Threat: Loads external malicious JavaScript from a known web shell source.

  Impact: Enables command execution, file manipulation, privilege escalation.
- In-Browser CLI Interface Delivered via HTML+JS, enabling full terminal emulation inside a browser tab.
  – Threat Actor TTPs (Tactics, Techniques, and Procedures): -
    (a) Use of R57, p0wny, and custom PHP web shells.
    (b) Attempted uploads of r57shell.php, r57eng.php, etc. from IP: 103.108.174.62.
    (c) Persistent backdoor via door.php.

- **Attack Vectors**
  - File upload vulnerability with weak MIME/type validation.
  - CMS misconfiguration (WordPress/Joomla).
  - Credential theft (FTP/SSH).
  - SQLi chained with file write.
  - Exposed admin interface or insecure eval/assert functions in PHP code.
- **Recommendations & Remediations given by CSIRT-Power**
  - Block malicious IOCs in perimeter devices.
  - Search and remove files such as r57shell.php, door.php, etc.
  - Patch all CMS/core web frameworks.
  - Reset all access credentials (CMS, FTP, SSH, DB).
  - Scan for lateral movement in adjacent subnets.
  - Conduct regular penetration testing and security audits.
  - Implement and tune a Web Application Firewall (WAF).
  - Monitor website file integrity and maintain frequent secure backups.
  - Segment networks to restrict web server exposure.
- **Hardening Measures given by CSIRT-Power**
  - Disable these PHP functions: eval(), shell_exec(), system().
  - Enforce strict MIME-type & file extension validation for uploads.
  - Prevent executable uploads to webroot.
  - Disable remote script loading.
  - Implement multi-factor authentication for admin access.

Reputation of url www.r57.gen.tr/yazciz/ciz.js (found in HTML code) is malicious as shown below in Virus Total.



List of IPs associated with Malicious R57 web shell

- 172.67.195.26
- 104.21.36.144
- 188.114.97.3
- 188.114.97.7
- 188.114.97.12
- 104.21.7.55

- 172.67.135.200
- 104.28.19.124
- 104.28.18.124
- 104.31.69.74

5.3.3 *Trusted Vendor System*

The CSIRT Power has been vigorously involved in establishing a Trusted Vendor System to counter cyber-security supply chain threats in the Indian Power Sector. Here Procurement is envisaged to be done from trusted sources-countries/vendors whose cyber integrity can be relied upon; for supplying equipment to the Power Sector in India. This system is also envisaged as a self-disclosure-based system, on which suppliers would be assessed through an appropriate inter-disciplinary institutional mechanism, based on an established set of criteria.

5.3.4 *Drafting of Cyber Security Regulations:*

The CEA (Cyber Security in Power Sector), Regulation 2025 have been successfully drafted, which is in advanced stages of approval after disposal of public comments and is going to be published in due course of time.

Cybersecurity Regulation is much more comprehensive. Additional areas apart from Cyber security guidelines being added in regulations: -

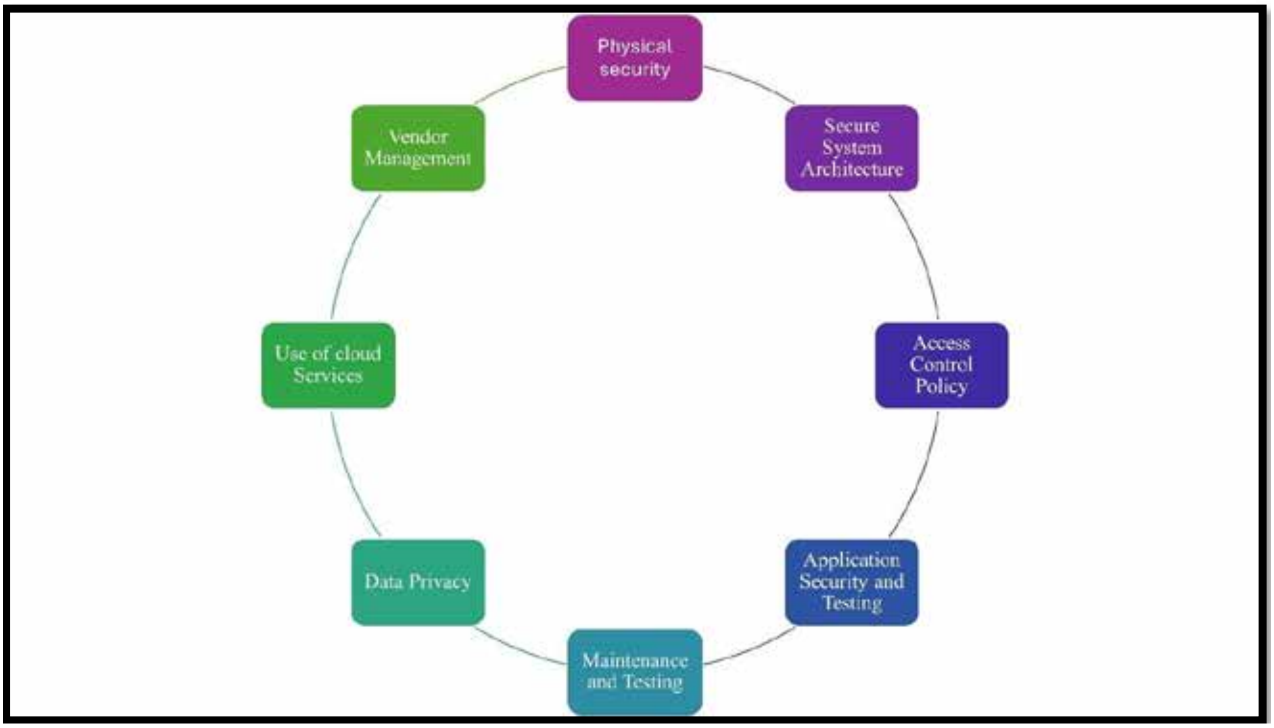


Fig. 6 : Upcoming Regulations additional areas

5.3.5 *Guideline on Scope of Cybersecurity Audit in the Power Sector:*

CSIRT-Power has recently issued Guideline on Scope of Cybersecurity Audit in the Power Sector. The specific purpose of this guideline is to provide a template for scope of conducting cybersecurity audits in the power sector against a set of requirements/standards/baselines, through CERT-In empanelled auditors. Power sector utilities are mandated to conduct periodic cybersecurity audit of both Information Technology (IT) and Operational Technology (OT) assets, through auditors empanelled by the Computer Emergency Response Team - India (CERT-In). In alignment with the directions provided, these cybersecurity audits go beyond Vulnerability Assessment and Penetration Testing (VA/PT) of applications and systems.

5.3.6 *Cyber Security Framework*

For assessment of Cyber security preparedness of Power sector utilities,

CSIRT-Power is following below mentioned parameters as shown in Fig 8.

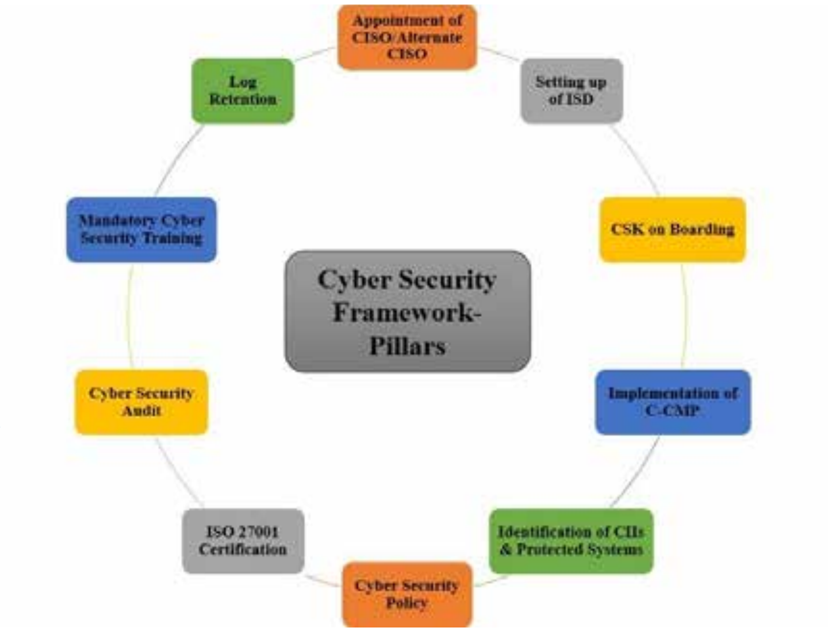


Fig. 7 : Cyber Security Framework

5.3.7 *Capacity Building*

(1) Regional Workshops: In line with directions issued during the Regional Power Ministers' Conference in June 2025, five workshops were organized across the Eastern, North-Eastern, Southern, Northern, and Western regions. Conducted with host utilities such as DVC, OPTCL, PGCIL, NTPC, and GSECL, these workshops brought together over 700 professionals from state utilities, central PSUs, renewable developers, and private players for awareness and sensitization.

(2) Capacity building exercises (tailor made) conducted in collaboration with academia (i.e. IIT Kanpur, Rastriya Raksha University, PGCOE IISC Bengaluru, and IIT Kharagpur), industry, premier cyber security organizations of India, more than 400 power professionals trained through these programs.

(3) Workshops, trainings, awareness sessions, conferences and cyber security exercises are being organized regularly by various agencies including NCIIPC, CERT-in and CSIRT-power, utilities, industry and academia to spread awareness among the stakeholders.

(4) Cyber Jaagrukta Diwas is organized on first Wednesday of every month to create cybersecurity awareness among all employees, power sector employees are also encouraged to obtain certification and attend courses offered by agencies such as National Power Training Institute (NPTI), CBIP, Rastriya Raksha University etc.

5.3.8 *Cybersecurity Capability Maturity Model (C2M2)*

CSIRT-Power has been working on the generic Cybersecurity Capability Maturity Model (C2M2) which is used to conduct cybersecurity programmatic self-evaluations. It addresses the implementation and management of programmatic cybersecurity practices associated with information technology (IT) and operations technology (OT) and the environments where these assets operate.

6.0 DRAFT REFERENCE ARCHITECTURE FOR THE HYDRO SECTOR

CSIRT -Power has prepared sector wise Reference Architecture which aims to provide guidance to the power sector utilities for designing their secure network architecture. The reference architecture enlists the minimum requirements to be inbuilt in the network for implementation of appropriate controls to be applied by each sub-Sectoral utilities of the power sector. Currently the same is under review.

At the core of hydro power operations lies the Supervisory Control and Data Acquisition (SCADA) system, the digital backbone that monitors and controls the entire generation process.

1. Multi-Layered Control System

Hydro plants use a hierarchical control setup for safe and efficient power generation:

- Field Level: Sensors, switches, and actuators monitor physical parameters like pressure, flow, and gate movement.
- Controller Level: Intelligent controllers (Governor, Excitation, Dam, GIS/Switchyard, Auxiliary, etc.) process data from field devices and issue control commands.

- Control Room Level: Operator Workstations (OWS) and servers connected through redundant LAN networks allow real-time monitoring and control.

Each controller also includes a local HMI (Human-Machine Interface) for manual operations in case of communication loss with the control room.

2. Interdependence on SCADA

The hydro plant's safe and stable operation depends heavily on SCADA. While manual operation is possible if SCADA software is offline, data acquisition, monitoring, and automation are halted — highlighting the need for robust cybersecurity and network reliability.

3. Secure Communication & Data Flow

- Telemetry: Data from hydro plants is transmitted to Load Dispatch Centres (LDCs) using IEC-104 protocol via Optical Ground Wire (OPGW) links.
- The flow is one-way, ensuring operational data moves only from the plant to RLDC, safeguarding against remote tampering.
- Firewalls and gateway PCs isolate SCADA networks from enterprise IT systems, with only filtered data shared through DMZ (Demilitarized Zone) servers for corporate use.

4. Automatic Generation Control (AGC)

AGC enables centralized real-time control by the National Load Dispatch Centre (NLDC):

- A dedicated Remote Terminal Unit (RTU) with redundant controller's interfaces with the plant's control system through hardwired links, independent of the SCADA LAN.
- NLDC can send setpoints for power generation, while plants provide feedback such as load, voltage, and operational limits via IEC-104 protocol.

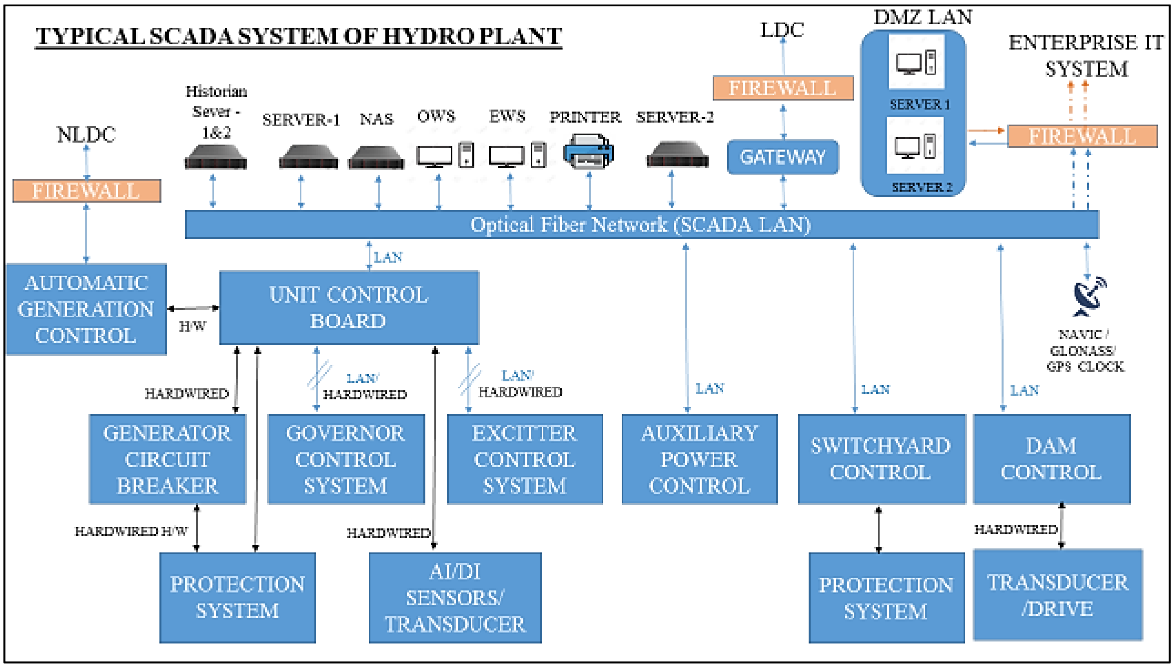


Fig 8 : Draft Reference Architecture for Hydro Sector

7.0 SUMMARY

India's hydropower sector is vital for the country's renewable energy goals, supplying a significant portion of its clean energy mix while fostering cross-border collaboration, especially with Bhutan. The deployment of advanced digital technologies such as SCADA and IoT has improved efficiency, safety, and reliability in hydro plant operations. However, this digital transformation has also introduced new cyber risks, making hydro infrastructure increasingly vulnerable to sophisticated cyber threats. To address these concerns, the Government of India established CSIRT-Power, a specialized cybersecurity agency for the power sector. CSIRT-Power strengthens sector resilience through comprehensive risk assessments, proactive threat intelligence, and ongoing capacity-building initiatives. CSIRT-Power has also developed sector-specific frameworks and incident response strategies tailored to the operational needs of hydro plants. Ultimately, this paper emphasizes the need for continuous vigilance and robust cybersecurity measures to secure India's critical hydropower assets in an evolving digital landscape.

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IMPLEMENTATION INSIGHTS FROM THE 336 MW CHUKHA HYDRO ELECTRIC PROJECT, BHUTAN

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ABSTRACT

India and Bhutan have been partnering in infrastructure development since Bhutan's First Five-Year Plan in 1961, with India serving as its primary collaborator. Recognizing Bhutan's vast hydropower potential, the Government of India (GoI), through its national institutions, initiated joint development efforts. The Royal Department of Power, Thimphu, under the Royal Government of Bhutan (RGB), served as the local executing authority.

The Chukha Hydroelectric Project (336 MW)—Bhutan's first large-scale venture—marked a milestone in Indo-Bhutan energy cooperation. Initiated in the mid-1970s and commissioned in phases beginning 1986, the project featured a run-of-the-river diversion dam, a headrace tunnel (HRT), surge shaft, and an underground powerhouse housing four 84 MW units and 220kV outdoor substation. The transmission system included three 220 kV circuits for power export to India and 132 kV, 66 kV, and 33 kV lines for domestic supply to Thimphu, Paro, Phuntsholing, and Pasaka.

Drawing from direct involvement (1980–1987), the author highlights engineering challenges and corrective actions during execution and commissioning. These include HRT alignment faults , and damage & rectification to the first generating unit during trial run.

The paper offers implementation insights and invites further contributions to document and strengthen hydropower development in fragile mountain ecosystems.

***Keywords**: Bhutan, India, Chukha Hydroelectric Project, diversion dam, HRT, desilting chamber, surge shaft, underground powerhouse, transmission lines, grid integration.*

1. INTRODUCTION

1.1 Strategic Partnership in Hydropower Development

India and Bhutan share a longstanding partnership in hydropower development, formalized through intergovernmental agreements and sustained through decades of collaboration. India has invested in Bhutan's hydropower infrastructure, receiving clean energy in return, while supporting Bhutan's broader development goals. Financing models gradually shifted from grant-based to loan-supported structures, reflecting evolving economic cooperation. Recent agreements between Druk Green Power Corporation (DGPC) and PTC India further reinforce this mutual commitment to cross-border energy exchange. [1] for more details.

1.2 Chukha Hydroelectric Project – A Milestone

The Chukha Hydroelectric Project (336 MW), Bhutan's first large-scale venture, marked a turning point in Indo-Bhutan energy collaboration. Initiated in the mid-1970s and commissioned in phases from 1986, the project features a run-of-the-river diversion dam near Chimhakoti, a headrace tunnel with an integrated desilting chamber, a surge shaft and a valve room for water flow control and penstock maintenance at Tsimilakha. Water is conveyed thru steel penstock to an underground powerhouse housing four 84 MW Pelton units, with tailrace tunnels—initially one, later expanded to two—discharging into the Wangchu River.

The transmission system includes three 220 kV circuits (one double-circuit and one single-circuit) from Chukha to Birpara (India), approx. 36km out of total length 70km in Bhutan thru Phuntsholing for power export to India, and 132 kV/66 kV/33 kV TLs supplying power within Bhutan (Thimphu, Phuntsholing, Pasaka, and Paro.

Indian Prime Minister and Bhutan King have a interactive meeting at New Delhi deciding to step up its support to Bhutan Up-coming 13th Five-year Plan, agreeing for an upward revision of Tarif of Chukha Hydro-electric project, etc. [--Times of India: 'India to step up support for Bhutan's five year plan, work for expediting rail link project', ANI, 4th April,2023] for mor details

1.3 Emerging Renewable Initiatives

Complementing hydropower, Juniper Green Energy commissioned a 100 MW solar project in Rajasthan to supply electricity to Bhutan under a cross-border agreement. Founded in 2018, Juniper operates over 1.1 GW of renewable capacity in India. [2] refer for more details

1.4 Future Expansion – Wangchu Project

In a recent development, Adani Power and DGPC signed a Shareholders Agreement to develop the 570 MW Wangchhu Hydroelectric Project. Construction is scheduled to begin in early 2026, with completion targeted within five years. [3] refer for more details

2. CHUKHA HYDRO ELECTRIC POWER PROJECT (336 MW) – IMPLEMENTATION SUMMARY

2.1 Foundational Agreement (GoI –RGB, March 23, 1974)

The 1974 agreement between the Government of India (GoI) and the Royal Government of Bhutan (RGB) initiated Bhutan's first large-scale hydroelectric project, aimed at exporting surplus power to India. The Chukha Hydro Electric Power Authority (CHP), chaired by Her Royal Highness Ashi D.W. Wangchuk—Representative of His Majesty in the Ministry of Development—was designated as the implementing agency.

Scope of Works under Article-1:

- **Diversion Dam**: 40 m high, located 1.6 km upstream of the Ti-Chu–Wong Chuk confluence; riverbed at +1689 m, pond level at +1716.20 m
- **Head Race Tunnel**: 6560 m long, 71.5 cumecs capacity, left-bank intake to underground powerhouse
- **Powerhouse**: 4×84 MW units (including standby), operating under 466.34 m average gross head
- **Transmission Systems:**
  - o 220 kV line (1xDC & 1xS/C): Chukha to Phuntsholing (India border)
  - o 66 kV links: Chukha to Thimphu and Phuntsholing, with substations and staff colonies
- **Additional Works**: As mutually agreed during execution

Supplementary Transmission Additions:

Following bilateral reviews by CEA (India) and the Royal Department of Power (Bhutan), the scope was expanded to include:

- 132 kV D/C lines: Chukha–Phuntsholing and Chukha–Thimphu

These 132kV TLs served Pasakha Industrial Estate(Phuntsholing), Paro, and Thimphu regions

Implementing Institutions:

The Chukha Hydro Electric Power Authority (CHP), Chimakothi

India: Central Electricity Authority (CEA), Central Water Commission (CWC), Water and Power Consultancy Services (WAPCOS) site office at Chimakothi as project coordinator

Bhutan: Royal Department of Power, Thimphu

The 220kV circuits and configuration (D/C & S/C) changed quite often. Initially, it was two 220kV, S/C, lines changed to one 220kV D/C Line and finally changed to one 220kV D/C and one 1x220kV S/C, line from Chukha to Phutsholing (India border) based on reliability and sustainability of mountainous terrain

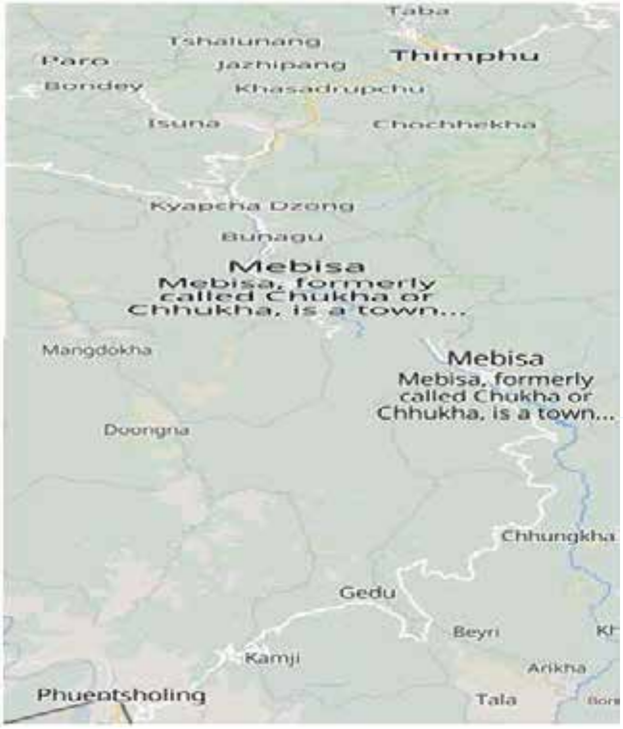
[4] for more details

2.2 Bhutan Highway Linkages to CHP

The Bhutan Highway connecting Phuntsholing–Gedu–Chukha–Chimakothi–Chapcha-Simtokha-Thimphu (with a branch to Paro) has been vital for project logistics

Vulnerable segments include:

- Kamji (Sorchen Curves) - Phuntsholing
- Gedu–Canteen stretch (Highway ascend)
- Before Bridge on Highway proceeding further towards Chimakothi and branch road to Chukha Dzong,



Landslide susceptibility mapping along the Thimphu-Phuentsholing highway

2.3 Field Engagement and Transmission Corridor Planning

- In the latter half of the 1970s, I (the author) visited Phuntsholing, Chukha Dzong (Powerhouse Adit), Chimakothi (CHP Headquarters), and Thimphu for the first joint meeting with the Royal Department of Power. The objective was to finalize the preliminary Right of Way (RoW) for the 220 kV transmission lines from Phuntsholing to Chukha, ensuring alignment through geologically stable zones and avoiding damage-prone areas.
- During my tenure with the Chukha Project (1980–1987) for project work implementation, frequently coordinated with project and transmission line contractor teams (SAE (India) & Tashi, Phuntsoling/ Tsimilakha to address highway blockages—particularly along vulnerable segments of the Bhutan Highway—ensuring timely restoration of access and continuity of works.
- Coordination with West Bengal State Electricity Board, Siliguri and Eastern Region, Calcutta (Kolkata)
- **Bhutan Highways outline maps pertaining to Project Area**

Bhutan Highway Map

Indian Regional Power Grids Bhutan is linked to Eastern Region

2.4 Chukha Dam Site Complex – Layout and Components

2.4.1 *Details of Encardio Rite's Role at Chukha*

Encardio Rite provided Instrumentation at the diversion dam, powerhouse, and associated tunnels for Chukha site to investigate Geotechnical and structural health monitoring. The installed system comprises Foundation piezometers, Strain meters, Anchor bolt load cells, Borehole extensometers, Crack meters, Temperature meters and Automatic data acquisition system (ADAS)

These systems were critical for monitoring structural integrity, water pressure, and stress during both construction and operation phases.

Fig.1: Conceptual layout of the Chukha Diversion Dam, located on the Wangchu River in Chhukha Dzongkhag, Bhutan. Designed as a run-of-river structure with pondage, the dam is ~40 m high and 105 m long, equipped with four radial spillway gates. The leftmost gate includes a fish ladder for safe downstream transit. The dam and powerhouse lie ~6 km from Tsimasham and ~3 km from Chhukha Zero Point on the Thimphu–Phuentsholing highway. The site was selected for optimal head utilization and geological stability within the Lesser Himalayan Sequence.

Fig. 1 : Chukha Dam Site Complex

Chukha Power Plant

Key Components of the Chukha Hydro Electric Project (CHP)

S.No	Components	Description
1	Diversion Dam	Concrete gravity dam, 43 m high, 105 m crest length; 4 radial spill way gates including fish ladder gate; crest elevation 1645 m MSL; instrumentation included piezometers, extensometers, load cells, and automated monitoring.
2	Desilting Chambers	Twin underground chambers (~92 m each) downstream of intake; connected to flushing tunnels for sediment evacuation and turbine protection.
3	Head Race Tunnel (HRT) (2Nos)	6560 m long, 71.5 cumecs capacity; fault zones stabilized with steel supports and prefabricated slabs; route diverted to bypass unstable geology. One more added.
4	Surge Shaft	Vertical shaft, 12.2 m diameter, 76.47 m height; absorbs water hammer and stabilizes flow between HRT and penstocks.
5	Under Ground Powerhouse	Cavern: 24.5×37.5×141.25 m; 4×84 MW units under 466.34 m gross head; elevation 1384.68 m MSL; rock anchored with 26m anchor bolts and shotcrete; includes service bay and transformer installation.
6	Cable Tunnels and Switch yard	220 kV oil-filled cables (AG German make) laid through two tunnels to outdoor switchyard built on muck-filled ground; Dzong wall and hill face used for bus bar layout.
7	Tail Race Tunnel (TRT) 2nos	Original TRT limited output to 320 MW; secondary TRT added in 1995 to enable full 336 MW capacity with 10% overload margin.
8	220kV Transmission System	Chukha–Phuntsholing–Birpara (India); longest spans 1.8 km near IMTRAT Canteen; tower foundations concrete weight only stabilized foundations due to poor soil composed of leaves of meters depth.
9	132kV Transmission System	Chukha–Simtokha (132/33 kV) line for Thimphu and Paro; longest span 2.2 km near Chapcha.
10	66kV Transmission Links	Chukha–Thimphu and Chukha–Phuntsholing lines with substations and staff colonies at both ends.

3. COMMISSIONING OF GENERATING UNIT AND STABILITY OF TLS PRIOR TO COMPLETION OF PROJECT

The successful commissioning of 84MW Generating units phased between 1986 to 1987. However, before final commissioning, the 1st unit completed in early eighty six failed and caused extensive damages (Pilot exciter burnt, Unit entered into runaway speed, brake jet not commissioned failed to stopping of unit, Zero-Zero dowels crushed causing unbalanced air gap and sever vibration, leading to splitting of concrete from concrete barrel for housing generator, etc.). The base box supporting Lower Arm bracket was provided with about 1inch thin shims which failed to provide designed surface load carrying base and the base plates were not interconnected to barrel concrete. The Project Team visited BHEL works and observed and discussed rectification in the units. The machine modifications were rectified, civil works repaired and gap between base box and concrete barrel were filled with resin concrete affording rigidity. 1st unit was commissioned successfully under strict supervision (Project & Manufacturer).

The subsequent units were rectified and commissioned successfully.

The TLs successfully commissioned and there was no hold up except two minor incidents, one each in 220kV, and 132kV TLs.

One tower location enroute 220kV TL (downside of Gedu towards Chukha), underground water flow from uphill and washed soil base material below one of the footing and same stabilized by mass concreting.

Another incident enroute 132kV Chukha- Simtokha TL (Thimphu) where settlement of hill caused below the two legs on the tower transverse face due to earthquake and tower remained stable on two legs. The tower suitably anchored, and hill depression was filled with borrowed soil and brought to original level and footings were re-casted without change in location and line charged.

4. DGPC (DRUK GREEN) RENOVATION AND MODERNIZATION

Over the years, DGPC has initiated major rehabilitation and modernization of the plant. To overcome problems associated with the ageing and obsolescence of spares and support services and to improve efficiency and reliability of the power plant, refurbishment works began in 2006 with the excitation and governor systems and generator brackets. This was followed by the upgradation of Unit I and III generators, protection system, switc gear and dam radial gates hoisting mechanism and most recently the replacement of Unit IV generator was completed. The replacement of Unit II generator and the auxiliary equipment of all four units is currently taken up. A modern SCADA system to be incorporated by 2025-2026. These R&M works are taken up in a phased manner during the lean periods to avoid generation loss.

DGPC experience in improving the reliability of the system be incorporated into future upcoming project documents / requirement.

Further, in the above observation, no mention is made on follow up on monitoring of Encardio Rite instruments installed at Chukha site.

5. EASTERN REGION MEETINGS

As Bhutan power system is inter-connected to Eastern Region, Druk Green Power Corporation (DGPC) be attending the business & operational meetings regularly conducted at Calcutta (Kolkata)

The committee has advised PowerGrid to coordinate with Bhutan for synchronized reconductoring of the 220 kV Birpara–Chukha lines. Timeline sharing is critical to ensure uninterrupted power flow and regional grid stability.[5&6] may refer for more details

This is vital issue and DGPC leverages interaction with CEA / Eastern Region and PowerGrid (India)

4. CONCLUSION

- Chukha Hydro-electric Power Project (336MW) exemplifies successful Indo-Bhutan collaboration.
- The 1974 agreement between GoI and RGB initiated Bhutan's first large-scale hydroelectric project, aimed at exporting surplus power to India.
- Frequent Highway disruptions enroute Phutsholing –Chukha entail stabilization.
- Field-tested solutions offer guidance for future projects.
- Encourage Druk Green Power Corporation (DGPC) to interact in Eastern Region Meetings to delve development in the Region viz. Change of Conductor of Chukha TLs.
- Invitation to DRUK Green to document and share further insights with ER/ CEA/PowerGrid/ NHPC

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FINANCIAL MODEL AND PUBLIC – PRIVATE PARTNERSHIP (PPP) IN BHUTAN'S HYDROPOWER SECTOR

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INTRODUCTION

Hydropower remains the largest source of renewable electricity worldwide. In Bhutan, it serves as the backbone of national economic growth, providing the main source of domestic energy and foreign exchange earnings. The sector contributes significantly to Bhutan's GDP, government revenues, and bilateral energy trade with India.

(Source: National Atlas of River Basins and Water Infrastructure in Bhutan, ADB, March 2016)

According to the 13th Five Year Plan, hydropower's GDP contribution is projected to grow from BTN 30,534 million in 2023 to BTN 55,999 million in 2029, with an average annual growth rate of 10.6%. Hydropower debt accounts for 73.3% of Bhutan's total external debt, making it the largest component of the country's debt profile. By 2029, Bhutan's total public debt is projected at BTN 474,419 million, about 94.8% of GDP, with the electricity and energy sector remaining one of the largest contributors to national output. Hydropower exports—especially to India—are a major source of foreign exchange and government revenue, supporting fiscal stability and funding for social programsi. Bhutan has five main river basins (Amochhu, Wangchhu, Punatsangchhu, Mangdechhu, and Drangmechhu) and some small river basins:-

OVERVIEW: BHUTAN'S HYDROPOWER SECTOR

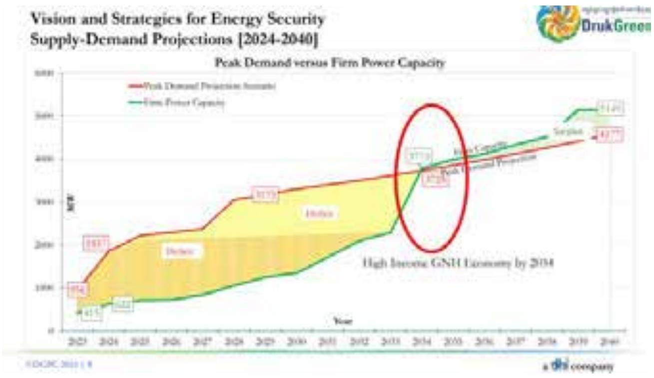
Bhutan's Hydropower generation is primarily managed by the Druk Green Power Corporation (DGPC), with a total installed capacity of approximately 3,464 MW as of August 2025.ii Major operational projects include:

- Chukha (336 MW)
- Kurichhu (60 MW)
- Tala (1,020 MW)
- Mangdechhu (720 MW)
- Punatsangchhu-II (1,020 MW)
- Dagachhu (126MW)
- Basochhu (64 MW)
- Nikkachhu (118 MW

A substantial share of Bhutan's Hydropower generation is exported to India through long-term Power Purchase Agreements (PPAs)iii, which form the foundation of the country's external revenue stream.

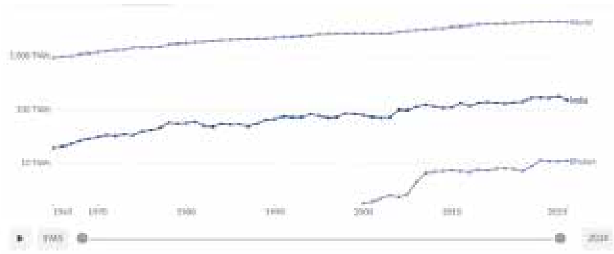
Project Under pipelines of around 2370 MW such as

- Punatsangchhu 1 (1200MW)
- Wangchhu(570MW)
- Khorlochhu (600 MW).



BHUTAN, INDIA, AND GLOBAL HYDROPOWER CONTEXT

Globally, installed Hydropower capacity reached around 1,443 GW in 2024iv, reflecting steady growth in both conventional and pumped-storage systems. India accounts for approximately 52 GWv, ranking among the world's leading Hydropower producers.



Although Bhutan's installed capacity of 3.46 GW is modest in absolute terms, it is exceptionally high on a per-capita basis and strategically significant due to the country's vast untapped Hydropower potential.

PUBLIC–PRIVATE PARTNERSHIP (PPP) MODEL FOR HYDROPOWER DEVELOPMENT IN BHUTAN

The Public–Private Partnership (PPP) model offers a transformative opportunity to accelerate Hydropower development in Bhutan.

Currently, most projects are implemented directly by the Government through the state-owned DGPC. While this approach ensures public control, it demands significant fiscal investment and involves multiple layers of regulatory and compliance processes—often resulting in project delays and cost overruns. Consequently, public funds remain tied up in project financing, limiting fiscal space for other developmental priorities.

As per the Bhutan's Sustainable Hydropower Development Policy,2021 and the Public Private Partnership Policy,2016, Bhutan seeks to promote sustainable, inclusive, and commercially viable Hydropower projects through private sector, inter-Governmental participation. The present PPP framework encourage collaboration between the public and private sector to leverage private capital, technology, and managerial expertise while maintain public ownershipvi.

ACCESS TO FINANCING

Equity:

- Operating Cash Flows
- Public Private Partnerships
- Bonds and IPOs
- Bilateral Equity Partnerships
- Intergovernmental funding

Debts:

- Multilateral funds
- Bonds
- Exim Financing
- Green Bonds

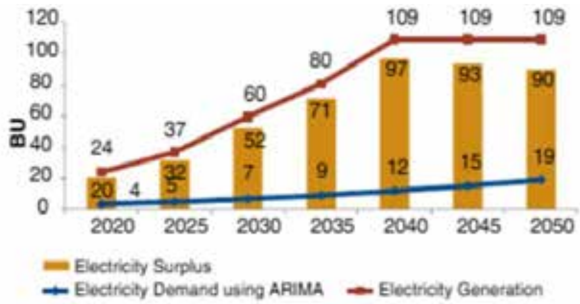
Challenges:

- Sovereign Guarantee Constraints
- Credit Rating
- Size of economic
- Financial Closure
- Access to climate funds

To address these challenges, Bhutan should consider adopting a PPP-based policy framework for future Hydropower projects. Under this model, the government would identify and assess potential sites, then auction them to private developers through a transparent, tariff-based competitive bidding process. A dedicated government-owned entity could be established to:

- Manage the auction process
- Sign PPAs with selected developers
- Facilitate the sale of power through bilateral contracts or regional energy exchanges

This entity could also serve as a single-window clearance mechanism, providing all required approvals—such as environmental, construction, and safety permits—for a nominal fee. Such a framework would substantially reduce time, cost, and uncertainty for private investors, thereby encouraging wider participation.



The PPP model would yield multiple benefits: the Government of Bhutan would reduce its financial burden, earn regulatory service revenue, and gain access to additional clean energy capacity without large upfront capital commitments. Private developers, in turn, would benefit from assured PPAs, streamlined regulatory processes, and opportunities for domestic and cross-border power trading.

Overall, a well-structured PPP framework would enable Bhutan to accelerate clean energy generation, attract private and foreign investment, and enhance energy security and GDP growth. By shifting from a purely government-led model to a collaborative public–private approach, Bhutan can achieve faster, more efficient, and sustainable Hydropower development.

FINANCIAL MODELING FOR HYDROPOWER PROJECTS

The financial model for any infrastructure project, including hydropower, typically comprises two main stages:

1. Capital Cost Phase

This stage covers all installation, construction, and development expenses. Capital costs are financed through a mix of equity, quasi-equity, and debt, with typical equity-to-debt ratiosvii ranging from 10:90 to 30:70, depending on project strength, risk profile, macroeconomic conditions, interest rates, and the developer's financial capacity.

The capital cost remains within planned limits only if the project progresses according to schedule. Any delay leads to Interest During Construction (IDC) accumulation on both debt and equity, significantly escalating total costs—sometimes to the point of financial unviability. These overruns not only erode profitability but can also result in higher tariffs for consumers.

In Hydropower development, project delays often result in substantial cost escalations due to the cumulative impact of IDC, rising material and labor costs, and extended overhead expenses. Implementing robust project management systems, proactive risk mitigation measures, and continuous monitoring of construction progress is essential to ensure cost control, maintain financial viability, and preserve investor confidence.

Hence, timely execution is essential to maintain financial viability and tariff competitiveness.

2. OPERATIONAL PHASE

Once the project achieves its Commercial Operation Date (COD), the operational phase begins — during which the project's financial viability is primarily determined by its ability to generate consistent revenue, service debt, and provide satisfactory returns on equity. The success of this phase depends on achieving and maintaining the projected generation levels while controlling operating costs and ensuring high plant availability.

Hydropower projects derive their revenues mainly from the design energy and capacity availability defined in the Power Purchase Agreement (PPA). Any deviation from these projections — due to changes in hydrology, sedimentation, reservoir siltation, or equipment inefficiencies — directly affects generation and, consequently, revenue realization. Factors such as seasonal variability in river discharge, alterations in glacial melt patterns, and long-term climate change effects can cause fluctuations in annual generation. These parameters should be regularly monitored, and necessary operational or design modifications should be implemented in a timely manner to adapt to evolving hydrological and environmental conditions.

To maintain operational efficiency and financial stability, effective Operation and Maintenance (O&M) strategies are critical. This includes preventive and predictive maintenance schedules, optimized turbine and generator performance monitoring, and regular inspections of dam safety and hydraulic structures. Equally important is sediment management, as excessive siltation can erode turbine components, reduce efficiency, and shorten equipment life.

Modern Hydropower projects increasingly rely on digital monitoring systems, SCADA-based controls, and real-time hydrological data analytics to optimize generation and respond quickly to variations in inflow or equipment performance. Adaptive reservoir operation — balancing power generation with flood control and ecological flow requirements — further enhances reliability and long-term sustainability.

In addition, cash flow management plays a critical role during the operational phase. The cash generated from operations must be sufficient to meet debt repayment, operational and maintenance expenses, and capital expenditure requirements that may arise due to activities such as machine maintenance, sedimentation works, reservoir strengthening, and other geological interventions identified through continuous monitoring. Proper cash flow planning should also ensure that adequate surplus remains for returns on equity, supporting the overall financial health, organizational growth, and future expansion of the project.

Efficient management of these financial and operational factors ensures sustained performance, a high Plant Load Factor (PLF) and Plant Availability Factor (PAF), leading to stronger Debt Service Coverage Ratios (DSCR) and improved overall returns for investors.

Key financial indicators for Project analysis (Based on financial results) :

S. No.	Key Ratio	Formula	Purpose / Interpretation
1	Debt–Equity Ratio	Total Debt / Total Equity	Indicates the project's leverage level; typical range for Hydropower is 70:30.
2	IDC to Total Cost Ratio	(Interest During Construction / Total Project Cost) × 100	Time overrun indicator — shows the proportion of financing cost accumulated during construction; lower is better.
3	Capital Cost per MW	Total Project Cost / Installed Capacity (MW)	To benchmark project cost against similar Hydropower projects.
4	Payback Period	Time required for cumulative cash inflows = Initial Investment	Indicates liquidity and recovery speed of the investment.
5	Internal Rate of Return (IRR)	Discount rate at which NPV = 0	Assesses the overall profitability of the project over its lifecycle.
6	Net Present Value (NPV)	Σ (Cash Flow$_t$ / $(1 + r)^t$) – Initial Investment	Measures value creation after considering time value of money;
7	Debt Service Coverage Ratio (DSCR)	Net Operating Income / Total Debt Service (Interest + Principal)	Measures the project's ability to service debt from cash flows; >1.25 is desirable.
8	Plant Load Factor (PLF)	(Actual Energy Generated / (Installed Capacity × 8,760 hrs)) × 100	Indicates capacity utilization and operational efficiency.
9	Plant Availability Factor (PAF)	(Hours Available / Total Hours) × 100	Reflects reliability and availability of generation assets.
10	Cost per kWh (Unit Cost)	Total Cost of Generation / Units Generated	Core indicator for tariff determination and cost competitiveness.
11	Return on Equity (ROE)	(Profit After Tax / Shareholders' Equity) × 100	Measures actual return earned by equity investors.
12	Energy Availability Ratio	(Saleable Energy / Design Energy) × 100	Evaluates generation reliability after accounting for auxiliary and transmission losses.
13	Auxiliary Consumption %	(Auxiliary Consumption / Gross Generation) × 100	Indicates internal energy use efficiency within the plant; lower is better.
14	Transmission Loss %	(Transmission Loss / Energy Sent Out) × 100	Determines effective saleable output after grid losses; used in revenue assessment.

ILLUSTRATIVE CASE STUDY: FINANCIAL RESTRUCTURING – DANS ENERGY PVT. LTD.

A practical example of Hydropower financial restructuring is the DANS Energy Pvt. Ltd. project, which developed a 96 MW Hydropower plant at Jorethang, Sikkim. The project achieved commercial operation in September 2015 after multiple COD extensions due to construction delays and cost overruns. The final project cost increased from the original estimate of ₹625 crore to ₹1,508 crore.

A long-term PPA was signed on 30 May 2022 at a levelized tariff of ₹4.34 per unit, valid until FY 2050, ensuring long-term revenue visibility. However, in the absence of a PPA during initial years, coupled with cost escalation and reduced generation caused by technical and siltation issues, the project became a Non-Performing Asset (NPA) in September 2017.

To restore financial viability, lenders implemented a comprehensive debt restructuring plan. The outstanding debt of approximately ₹1,403.13 crore was split into:

- Sustainable debt: ₹930.66 crore
- Unsustainable debt: ₹472.47 crore, converted into Optionally Convertible Debentures (OCDs) at a nominal interest rate of 0.01% p.a.

The repayment period was extended from 18 to 28 years to align with the PPA duration, ensuring sufficient cash flow for debt servicing. The restructuring revived project operations and reduced financial stress for both promoters and lenders.

Takeaways

Bhutan's Hydropower sector stands at a pivotal juncture. Strategic reforms in project structuring and financing can unlock its next phase of growth. While the state-led model under DGPC has been effective, future expansion will require a more diversified, collaborative, and efficiency-driven approach.

The PPP model offers a practical pathway to attract private investment, accelerate capacity addition, and reduce fiscal pressure while preserving national ownership of critical assets.

The DANS Energy case demonstrates that even technically sound projects can face financial stress without accurate forecasting, timely execution, and well-structured financing. Hence, a robust financial and policy framework—anchored in transparent tariffs, predictable PPAs, and single-window clearances—is essential.

By integrating PPP structures with sound financial modeling, Bhutan can achieve faster commissioning, improved cost control, and sustainable cash flow management, strengthen investor confidence and reinforcing its position as a regional leader in clean energy.

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EMISSION OF INFLAMMABLE GASES DURING DEVELOPMENT OF HYDEL TUNNELS IN COMPLEX HIMALAYAN TERRAIN OF EASTERN BHUTAN

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ABSTRACT

Countries around the world are exploring clean energy technologies and hydropower projects are one of the best options for generating energy without harming the environment and ecosystem. Bhutan is known for being carbon neutral in terms of greenhouse gas (GHG) emissions. Bhutan is part of the greater Himalayas, characterized by complex terrain and a rich river system. The hilly river network features high-gradient, fast-flowing and perennial rivers favours the construction of hydropower projects. Bhutan aims to reach a total electricity generation capacity of 25 gig watts (GW) by 2040. This includes 20 GW from hydropower and 5 GW from solar. The current hydropower capacity ~ 2.4 GW and under construction ~ 3.1 GW. The hydel projects needs extensive tunneling in the complex hilly terrain comprising the classic Himalayan subdivision into four main zones: the Sub-Himalaya, Lesser Himalaya, Greater Himalaya and Tethyan Himalaya, each bounded by major faults. Out of these, the Lesser Himalayan Sequence, located between the Main Boundary Thrust (MBT) and the Main Central Thrust (MCT), comprises 8 - 12 km of Paleoproterozoic to Permian metasediments. This sequence is divided into the lower Paleoproterozoic Shumar and Daling Formations and the upper Cambrian Baxa Group and Permian Gondwana Sequence. The Baxa Group includes quartzites and dolomites, while the Gondwana Sequence features coal-bearing strata. The Gondwana sediments mainly contain carbonaceous shale and coal seam lenses, which have a significant amount of organic carbon. Gases generated primarily methane (CH4) and hydrogen sulfide (H2S) are stored in the coal and shale deposits. These gases can be released during tunneling operations and may accumulate along the tunnel roof, posing an explosion risk if ignited.

There are established safety norms for methane and other flammable gas emissions during tunneling or mining activities. Safety is a key concern, especially because tunnels are often developed using blind headings and forced ventilation systems, which take time to fully flush long tunnels with fresh air. In light of the above, a case study was conducted on the measurement of methane and other gases in the Nyera Amari II Powerhouse at Martshala tunnels in eastern Bhutan. A systematic method was proposed to measure methane and H2S concentrations within the tunnel. The minor methane layering and negligible methane accumulation was observed in general air body of the tunnel. The methane concentration in 1.5 meter borehole drilled at advance face, roof and floor of the tunnel after plugging for 1 hour ranges from 3.78 to 4.37 %; whereas, the methane concentration after plugging the borehole for 48 hours varies from 6.43 to 12.76. Similarly, small amount of ethane (C2H6) *concentration after plugging the borehole for 1 hour ranges from 0.18 to 0.35 %; whereas, the ethane concentration after plugging the borehole for 48 hours varies from 0.25 to 0.32 %. It also indicates minor changes in ethane percentage with time in any of the boreholes on keeping them plugged for 48 hours. The traces of H2S were observed, probably emitted from intersected shale beds in tunnels. Also, the air samples were collected both before and after rock blasting and analyzed for their composition shown slight increase in methane and H2S concentration in the general body air. The sources of methane and H2S were identified through a study of the local geology, the thermal maturity of the shale and coal formations encountered in the tunnels and stable isotope analysis of methane. The range values of stable isotope (δ13C1) of methane in the studied borehole samples indicating possible thermogenic origin signatures (δ13C1< -40 ‰). This has been confirmed through plot of C2+ and δ13C1, which indicates methane and ethane in borehole gas are of thermogenic to late thermogenic in origin. Conventional electrical equipment (such as cables, bulbs and pumps) was replaced with flame-proof alternatives to ensure safety. Moreover, the methane concentration should be maintained below 10% LEL (0.5% by volume) for safe working. A structured set of safety procedures was implemented to raise awareness among workers operating in the tunnels. Regular monitoring using portable, handheld multi-gas analyzers is recommended to detect and manage gas accumulation. Further, it is recommended that regular check on general body concentration of methane, return air velocity at the tunnel portal and duct discharge end near working face may be made to detect any presence of methane even it be low, in view of small amount of methane observed in the boreholes as a preventive measure against any possible gas hazards. This paper presents the sources of methane and H2S generation, accumulation and emission in tunnels during excavation. It also outlines the detailed methodology used for their measurement and suggests safety measures to prevent potential explosions in tunnel environments.*

Keywords: *Emission, inflammable gases, measurement, tunnels, hydropower, safety measures.*

INTRODUCTION

Hydropower projects often involve extensive underground tunnelling for water conveyance, such as headrace, tailrace and diversion tunnels. During the excavation of these tunnels, particularly in areas with organic-rich sedimentary formations or coal-bearing strata, there is a possibility of encountering methane and other toxic gases trapped within rock layers. Methane is a highly flammable gas and, when mixed with air in concentrations of 5-15%, can form explosive mixtures that pose serious safety hazards to workers and infrastructure. Besides methane, other toxic gases such as carbon dioxide (CO2), hydrogen sulphide (H2S) and carbon monoxide (CO) may also be emitted during tunnelling operations. The accumulation of these gases in confined underground environments can result in oxygen deficiency, respiratory hazards, poisoning or explosion. Such conditions compromise occupational safety and demand strict control measures throughout excavation activities (Rohdin and Moshfegh, 2011, Sa et al., 2012, Fabiano et al., 2014; Haque et al., 2016). To ensure safe working conditions, hydropower tunnelling must incorporate robust gas monitoring, ventilation and emergency response systems. Continuous gas detection, efficient ventilation design and controlled excavation sequencing are essential to prevent gas build-up and maintain adequate air quality (Pena-Garcia, 2022). Pre-construction geological and geochemical investigations are equally important for identifying gas-prone zones and planning appropriate mitigation strategies (Shao et al., 2016). From an environmental standpoint, methane emissions from tunnelling contribute to greenhouse gas accumulation and elevate the carbon footprint of hydropower projects. Integrating methane capture and utilization systems, wherever technically feasible, can help mitigate these impacts while supporting sustainable energy objectives (Etheridge, 2000; Ji et al., 2011). In conclusion, tunnelling for hydropower projects in gas-bearing formations demands a comprehensive safety management framework. Early detection, effective ventilation, periodic risk assessments and emergency preparedness are vital for ensuring worker safety and environmental protection, thereby promoting the sustainable and responsible development of underground hydropower infrastructure.

In this study, a case study has been undertaken for the Nyera Amari-II Powerhouse site in Martshala, Bhutan. The Druk Green Power Corporation (DGPC), Bhutan, has initiated exploratory tunnel (drift) excavation at the Nyera Amari-II Powerhouse site in Martshala, Samdrup Jongkhar, Bhutan. During the course of excavation, the exploratory drift encountered challenges related to poor ventilation and the presence of methane gas within the tunnel. In view of these safety concerns, DGPC sought technical assistance from the CSIR-Central Institute of Mining and Fuel Research

(CIMFR), Dhanbad, to conduct a scientific investigation aimed at identifying the source of methane emissions and devising measures for the re-establishment of an effective ventilation system. The objective of this study is to ensure the safe resumption of excavation activities while maintaining strict adherence to safety standards and environmental considerations.

Tunneling for Hydropower Projects

Tunneling is a crucial part of hydropower development, facilitating the conveyance of water from the intake to the powerhouse and from the turbine to the tailrace. The main types of tunnels involved are headrace tunnels, pressure shafts and tailrace tunnels, all designed to minimize head loss and ensure efficient water flow. The choice of construction method depends on geological conditions and tunnel length, with common techniques including drill-and-blast, tunnel boring machines (TBMs) and the New Austrian Tunneling Method (NATM). Important factors during design and construction include rock stability, groundwater management, lining design and safety measures. A well-planned tunneling system is essential for ensuring the overall performance, safety and economic viability of a hydropower project. Tunneling for hydropower projects in the Himalayan region poses significant challenges due to its complex geological and climatic conditions. The terrain consists of young, fragile and highly folded rock formations, with frequent fault zones and high in-situ stresses, making tunneling operations technically demanding and risky. Typical challenges include rock bursts, landslides, water ingress, squeezing or collapsing ground and highly deformable rock masses. To address these issues, comprehensive geotechnical investigations, real-time monitoring and flexible construction methods are essential for ensuring tunnel stability and worker safety. Emission of gases like methane, carbon dioxide and carbon monoxide poses another serious hazard (Colella et al., 2009). Regular air quality monitoring, installation of gas detectors and provision of adequate ventilation systems are essential to maintain safe working conditions. Workers should be trained in emergency response procedures and equipped with personal protective equipment (PPE) and self-rescue devices. Implementing these safety measures ensures a safer working environment and enhances the stability and longevity of tunnels in the challenging Himalayan terrain.

PURPOSE AND OBJECTIVES OF THE STUDY

The study undertaken by CSIR–CIMFR, Dhanbad focused on addressing two major issues, such as re-establishment and adequacy assessment of the ventilation system in the tunnel and identification of the sources of methane emission. The detailed objectives and scope of work of the study are outlined below:

- To provide detailed guidelines and procedures for rectifying the existing ventilation system in the accessible or approachable sections of the tunnel and ensuring that it is safe and explosion-proof. This also includes the replacement of normal lighting with flame- and explosion-proof lighting systems.
- To recommend and specify essential monitoring equipment required for safe operation in potentially explosive environments, along with their technical specifications and usage protocols.
- To develop a detailed methodology for the recovery of inaccessible tunnel areas, including a step-by-step procedure and precautions to be followed during progressive access, until the entire tunnel is declared safe for further excavation.
- To guide and supervise the recovery operations up to RD 550 m to ensure safe and systematic restoration of working conditions.
- To assess the adequacy of the tunnel's ventilation system under maximum working load or extent and to recommend strengthening measures, if required.
- To measure dust generation levels during tunnelling operations, evaluate their dilution through ventilation and assess potential safety risks associated with dust during excavation.
- To impart basic awareness and training to the workforce regarding the risks and safety requirements associated with tunnelling through carbonaceous rock formations.
- To conduct scientific investigations on the sources of methane emission after the restoration of proper ventilation within the tunnel.
- To analyze the carbonaceous slate samples encountered from RD 550 m in the drift for their chemical properties, stable isotopic composition, gas storage capacity and diffusion characteristics.
- To identify the source rocks of methane emission following the establishment of the ventilation system, through the drilling of short-length boreholes (approximately 1.25 m deep) in the sidewalls and roof of the tunnel. Methane emissions from selected boreholes are to be measured at intervals of 1 hour and 48 hours to assess gas build-up patterns.
- To analyze collected gas samples for their molecular composition and stable isotopic characteristics to determine the type and origin of the gas whether biogenic, thermogenic or transitional.
- To process and interpret field and laboratory data, including borehole gas composition and related analyses, to

estimate the gas potential and evaluate associated safety hazards in the drift. Based on these findings, the study aims to provide recommendations on appropriate preventive and control measures.

FIELD INVESTIGATION

The exploratory drift of the Nyera Amari-II Powerhouse encountered serious challenges related to inadequate ventilation and methane gas emissions within the tunnel workings. A tragic methane explosion on 15th June 2018 resulted in the loss of five lives, underscoring the critical nature of the problem. Recognizing the gravity of the situation, the Druk Green Power Corporation (DGPC), Thimphu, assigned the CSIR-Central Institute of Mining and Fuel Research (CSIR-CIMFR), Dhanbad, to undertake a comprehensive scientific study. The objective was to examine the existing ventilation system, identify the sources of methane emission and recommend corrective measures to prevent such incidents in the future. In response, CSIR-CIMFR deployed a team of scientists who conducted an on-site investigation from 28th September to 2nd October 2018. The study involved air quantity and methane concentration measurements, gas and carbonaceous slate sampling, temperature monitoring and performance evaluation of ventilation fans. The data collected during this investigation were critically analyzed to assess the ventilation efficiency and gas emission characteristics. The findings of this investigation are presented and discussed in this report. The study includes the problem definition and scope of work, site particulars, ventilation system description and duct layout and the results of ventilation and gas analyses. It further covers hydrocarbon gas composition, stable isotope analysis for determining gas origin, adsorption studies to evaluate gas storage capacity and chemical and petrographic analyses to assess organic content. Based on these results, the report provides conclusions and recommendations for enhancing the ventilation system and ensuring improved safety in future exploratory drift operations.

DETAILED METHODOLOGY OF TUNNEL INVESTIGATION FOR GAS EMISSION

A comprehensive program of airflow measurements was undertaken in the tunnel to assess air quantity distribution, air losses due to duct leakage and to identify leakage points. For this purpose, calibrated vane anemometers were employed to measure the average air velocity inside the ventilation ducts. The airflow rate at each section was calculated as the product of the corrected air velocity and the corresponding duct cross-sectional area. At each duct extension point, three to four sets of velocity readings were recorded and the average of all acceptable readings those within ±5% of the mean was used to determine the final flow rate. Airflow measurements were conducted at nine stations along the tunnel intake. The observations indicated significant air leakage from the ventilation ducts, which necessitates repair or replacement to improve air delivery to the tunnel face and ensure effective methane dilution. The required air quantity for safe operation was estimated in accordance with Article 160(1) of the Coal Mines Regulations (Government of India), which stipulates that in auxiliary ventilation systems where methane is present, the minimum air velocity at the face should be 0.5 m/s to prevent methane layering at the roof. Given the exploratory tunnel dimensions of 1.8 m × 2.4 m (cross-sectional area = 4.32 m2) and deducting the duct area of 0.2869 m2, the effective tunnel area is approximately 4.04 m2. Accordingly, the required airflow at the face is calculated as Air Quantity = 0.5×4.04×60=121.2 m3/min. The airflow measurements recorded at various points along the tunnel such as, RD-150: 183 m3/min, RD-425: 173 m3/min, RD-550: 68 m^3/min and RD-575: 60 m3/min. After the ventilation system was re-established, it was found that the air quantity delivered at the face was only about 22% of the total fan capacity, confirming substantial leakage losses in the ducting system. Hence, duct repair or replacement is strongly recommended to enhance ventilation efficiency. Measurements of humidity and temperature at all duct extension points using a whirling hygrometer showed values within acceptable limits, indicating a comfortable working environment. To assess the effectiveness of ventilation and methane accumulation, the fan was stopped for 16 hours. Observations revealed that the tunnel remained free of methane up to RD-350, while beyond this point, methane concentration increased from 0.5% to 3% near the face. After restarting the fan, methane concentrations were reduced to 0.5% or less within about 30 minutes of operation. Air samples collected at RD-410, RD-470 and RD-585 following the re-establishment of the ventilation system indicated methane concentrations up to 0.5%, which are within permissible safety limits.A list of preparatory arrangement for taking up ventilation restoration work has been prepared and sent to the management of DGPC Bhutan for its compliance before taking up restoration work. Accordingly, a minimum essential preparatory arrangement was ensured before taking up ventilation re-establishment work in the tunnel. The detailed methodology for re-establishing ventilation system in the tunnel were considered as, a fan at the tunnel was started 24 hours before the start of the recovery operation, only certified LED cap lamp based lighting (intrinsically safe) were allowed to be used during ventilation re-establishment operation and before commencement of extension of ventilation in the tunnel, inspection of ventilation system inside tunnel up to RD-320 was to be done to ensure. Some of the suggestions and recommendations were made for safe tunnel advancement as below:

(i) Duct to be properly aligned. Hanging almost same level in the tunnel.

(ii) Ensure replacing of damaged segments with new one or repairing of damaged one (if the damage is very less) to ensure minimal leakage or no leakage in the auxiliary ventilation system.

(iii) Measured the air quantity, temperature and humidity at the entry of tunnel and at RD-320.

(iv) Measured the CH_4, O_2 and CO level at RD-320.

(v) Travel towards the inbye of the tunnel and measure the lead distance i.e. the distance between air discharge at duct and the place along the tunnel where the CH4 concentration reaches to 100 LEL (5%). It is assumed that other gases are zero.

(vi) After ensuring and setting right the existing ventilation, the engineers were advised to add one segment of 25 m of the fresh duct.

(vii) Tunnel get ventilated for sufficient time to ensure dilution of methane in the 25 m tunnel length and stabilise the ventilation system upto increased length.

(viii) Measured the air velocity/quantity, temp/humidity, CH_4, CO and O_2. If CH_4 and CO are about zero and O2 is more than 19%, at duct discharge, lead distance is almost same as previous, ask the tunnel engineer to add another fresh duct segment of 25 m.

(ix) The additional ducts were kept on adding till the face by repeating the activity 5 and 6 in series and extending the ventilation by another 25 m every time.

MEASUREMENT OF TOXIC AND INFLAMMABLE GASES IN THE TUNNEL

As a safety measures during field investigation and re-installation of ventilation system, approaching face of tunnel (RD595) and drilling of boreholes for source evaluation of methane in tunnel at different interval the concentration of toxic and inflammable gases like methane (CH_4), carbon monoxide (CO), carbon dioxide (CO2), oxygen (O2) etc. were measured using handheld multigas analyser Altair 5X (Make MSA, USA). The general body air samples were collected over the entire cross-sectional area of the tunnel moving the sampling tubes. The air samples were collected and analysed using portable gas analyzer (Altair 5X (Make MSA, USA) and also in the laboratory at CSIR-CIMFR Dhanbad. A number of boreholes of minimum 1.25 m depth and 0.04 m diameter were drilled by air compressor drill in the left and right side of the tunnel and advancing face in the carbonaceous slate encountered in the tunnel. Gas samples were collected from inside the boreholes after keeping them plugged for 1 hour and 48 hours. Methane (CH_4), carbon monoxide (CO), carbon dioxide (CO_2) and oxygen (O_2) gases were measured using handheld multigas analyser Altair 5X (Make MSA, USA) during the field investigation. Also, the borehole gas and general body air samples were collected during field investigation at different time interval in gas sampling tube by water displacement method under saturated saline water to avoid any likely solubility. These gas samples were analysed by gas chromatography system at CSIR-CIMFR laboratory using Chemito, Model: 1000 for hydrocarbon and non-hydrocarbon distribution like CH_4, C_2H_6 and CO, CO_2 using Flame Ionisation detector (FID) and Thermal Conductivity Detector (TCD) respectively.

The results of molecular gas composition of general body air samples collected at different locations in the tunnel indicated that on 29/09/2018 in the absence of proper ventilation system a maximum of 2.28% methane observed in general body air sample collected at RD-470. Further, on 30 Oct. 2018, during laying of ventilation duct the maximum methane concentration in before advance of duct was observed to be 2.16 at RD-475. However, it has been reduced to less than 1 or negligible after 10 minutes of discharge of air. Moreover, a complete dilution of methane occurred after running the ventilation fan of about 1 hour 30 minutes on 30 Oct. and 31 Oct., 2018. No or negligible accumulation of methane in the general body air and no layering in the vicinity of roofs of the tunnel after stopping the fan for 16 hours and running the ventilation fan for 30 minutes on 31st Oct. 2018 signifies that methane concentration in the tunnel can be maintained through the existing ventilation system. Hence, it is recommended, as a safety measures, to establish ventilation circuit and flush out air for about 1 hour and 30 minutes before entering of personnel into tunnel whenever there is a long breakdown in fan or ventilation system due to power failure or otherwise.

The molecular gas composition and stable isotope study of the borehole gas samples collected from RD-550 and RD-595 indicated the methane concentration after plugging the borehole for 1 hour ranges from 3.78 to 4.37 %; whereas, the methane concentration after plugging the borehole for 48 hours varies from 6.43 to 12.76. Similarly, small amount of ethane (C2H6) concentration after plugging the borehole for 1 hour ranges from 0.18 to 0.35 %; whereas, the ethane concentration after plugging the borehole for 48 hours varies from 0.25 to 0.32 %. It indicates minor changes in methane and ethane percentage with time in any of the boreholes on keeping them plugged for 48 hours. The traces of H2S were observed, probably emitted from intersected shale beds in tunnels. Also, the air samples were collected both before and after rock blasting and analyzed for their composition shown slight increase in methane and H2S concentration in the general body air. The sources of methane and H2S were identified through a study of the local geology, the thermal maturity of the shale and coal formations encountered in the tunnels and stable isotope analysis of methane. However, the carbonaceous slate contains small amount gases mainly held in free state in its silken plane, bedding planes and fractures, which gets released in the tunnel or working through the fracture network.

STABLE ISOTOPE ANALYSIS OF BOREHOLE GAS SAMPLES

The isotopic analyses for δ13C-CH4 values were carried out by Mass Spectrometer of Finnigan, Model: Mat 251 to evaluate the gas genesis pattern in carbonaceous slate. The methane and ethane were oxidized in separate CuO ovens in order to prevent cross-contamination. The combustion products CO2 and H2O were frozen into collection vessels and separated. The water was reduced with zinc metal in a selected tube to prepare hydrogen for isotopic analysis (Meier-

Augenstein, 1999; Henning et al., 2007). The isotopic values of methane (δ13C1) in gas samples after plugging the borehole for 1 hour range from -31.48 to -33.16 ‰; whereas, the isotopic values of methane (δ13C1) after plugging the borehole for 48 hours varies from -31.79 to -33.48 ‰. The value of δ13C1 of borehole gas of organic or inorganic is an important parameter in assessing information about CH4 genesis (Schoell, 1980; Mendhe et al., 2017). The range values of δ13C1 of methane in the studied borehole samples indicating possible thermogenic origin signatures (δ13C1< -40 ‰). This has been confirmed through the plot of C2+ and δ13C1, which indicates methane and ethane in borehole gas is of thermogenic to late thermogenic in origin.

SOURCE EVALUATION OF METHANE AND OTHER GASES IN TUNNEL

The molecular composition and isotopic analysis of the borehole gas samples indicate that the gas is of thermogenic origin, generated during the thermal alteration of organic matter that occurred prior to the silicification of the host rock. Such gases are typically formed under high temperature and pressure conditions, where buried organic materials undergo decomposition and chemical transformation. The presence of hydrocarbons with specific isotopic signatures supports this inference of thermal genesis rather than a biogenic source. A significant portion of the gas is retained in fractures, joints, bedding planes and slickensided surfaces within the carbonaceous slate, existing in a free or adsorbed state rather than being stored within the rock matrix. Since the carbonaceous slate lacks a well-developed pore system or interconnected micro-fractures, the diffusion and long-term storage capacity for gases are minimal. Consequently, any gas released from such formations is expected to be of a short-term and localized nature, primarily associated with mechanical disturbance during excavation. Therefore, methane emission from the carbonaceous slate is considered episodic and transient, posing limited long-term accumulation risk but requiring careful ventilation management during tunnelling and excavation operations to ensure safety.

SUMMARY AND CONCLUSIONS

The tunnel ventilation system comprising two auxiliary fans operating in forcing mode, each capable of delivering an air quantity of 330 m3/min through ducts of 550 mm diameter joined by zappers. The effective air intake into the tunnel is about 270 m3/min, though significant leakages are observed along the ducts due to physical damage. Wet and dry bulb temperature measurements taken at each duct connection point were found to be within permissible comfort limits. Following a 16-hour stoppage of the fan and subsequent operation for 1.5 hours, negligible methane accumulation and no roof layering were observed, confirming the adequacy of the existing ventilation system. It is therefore recommended that, after any extended fan shutdown or power failure, the tunnel be ventilated for at least 1.5 hours before personnel entry. Borehole gas analysis indicated methane concentrations ranging from 3.78-4.37% (1 hour plug) to 6.43-12.76% (48 hours plug), with minor variation over time. Ethane levels remained low (0.18–0.35%), suggesting minimal gas retention. The traces of H2S were observed, probably emitted from intersected shale beds in tunnels. Also, the air samples were collected both before and after rock blasting and analyzed for their composition shown slight increase in methane and H2S concentration in the general body air. The sources of methane and H2S were identified through a study of the local geology, the thermal maturity of the shale and coal formations encountered in the tunnels and stable isotope analysis of methane. Isotopic and compositional data ($\delta^{13}C_1 < -40‰$) confirmed a thermogenic origin of gases derived from the thermal alteration of organic matter. For safe operations, ventilation ducts must be repaired or replaced and air quantity increased to 121 m3/min at the face. Ducts should be suspended along the tunnel roof, with discharge within 10 m of the working face. The lighting system must be upgraded to LED cap lamps and flame-proof fittings and methane maintained below 10% LEL (0.5% v/v). Regular monitoring of methane concentration and return air velocity is strongly recommended to prevent gas hazards.

Acknowledgement

We express our sincere gratitude to the Director, CSIR–Central Institute of Mining and Fuel Research (CSIR-CIMFR), Dhanbad and to the Druk Green Power Corporation (DGPC), Bhutan, for providing the opportunity to undertake this investigation on the probable causes of gas emissions and the associated safety measures required for ensuring safe tunnelling operations during the construction of the hydropower project.

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HIMALAYAS, A NATURAL RESEARCH LABORATORY FOR TUNNEL ENGINEERS AND GEOLOGISTS

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ABSTRACT

Being tectonically active, Himalayas exhibits very complex behaviour in terms of frequent seismic activities and changing geology. Most of the hilly terrains are inaccessible and remain unexplored by the geologists. Any underground construction beneath the rock cover in such regions, require a good geological and geotechnical investigation to assess the rock mass behaviour at the selected depth for the construction. That is why, sometimes, it becomes nearly impossible to record signature of all desired properties of rock mass and thus leading to various geological surprises during underground construction. The entire rock mass of the Himalayan region is fragile in nature because of the continuous tectonic activities and weathering environment. Due to the reasons, the Himalayan region has been a great field research laboratory for the tunnel engineers and the geologists.

On the other hand, to harness increasing demand of power, various hydroelectric projects are also being constructed wherein many tunnels for water transportation must be constructed. Most of such tunnel constructions are housed in Himalayan region. The high in-situ stresses resulted by tectonic activities pose severe tunnelling problems in the region. For example, squeezing problem encounters in case of weak rock mass and rock burst problem takes place in strong rock mass under the influence of high in-situ stresses. Even site customised precise drilling, adaptive charge placement and controlled blasting plays an important role in efficient and safe tunnelling in geologically complex environments.

The present keynote highlights some of the key factors and associated problems, which have been influencing the geo-constructions in the Himalayan regions turning it a research laboratory for the geologists and engineers. To tackle the above-mentioned constructional problems, New Austrian Tunnelling method is adopted. Its philosophy is based on "build as you go".

1.0 INTRODUCTION

Due to increasing population growth (especially in India) and development of new highways and railways to connect strategic hilly regions, tunnelling activities are increasing nowadays. For example, Tunnelling projects are expanding globally, with India leading this growth, having 75 projects worth Rs. 49,000 Cr under construction and more worth Rs. 3 lac Cr planned over the next 10 years. (Business Standard, 2024; The Economics Times, Dec 2024). Most of the Indian tunnelling projects are housed in the Himalayan region.

The Himalayas is young mountain, and it is still under the process of orogeny resulting from plate movement and collision (Molnar 1998; Bollinger et al. 2006; DiPietro and Pogue 2004). Due to mountain building activities, high stresses are often generated and usually cause folding, faulting and shearing of rock mass. The in-situ stresses originated from plate tectonics result in geological instability in rock mass leading to development of high seismic zones and complex geological conditions.

In the past, many hydel, highways and railways projects housed in Himalayan region were considerably delayed due to adverse geological conditions characterized by high in-situ stresses and other unpredicted swelling conditions and water channels in the tunnels. For example, Chhibro-Khodri hydro-electric project, Dul-Hasti (3 x 130 MW), Teesta VI (500 MW) and Tapovan-Vishnugad (520 MW) were delayed by for more than 6 years, 10 years and 5 years respectively due to squeezing and rock burst problems during tunnelling (Sharma and Tiwari 2015). Udhampur-Katra railway tunnels delayed by many years due to swelling conditions and unpredicted water channels. The tunnelling projects suffer very high economic loss due to such time long time delays. Further, installed supports get deteriorated and their designed capacity reduces due to lack of maintenance in trapped stretches of the affected tunnels because of absence of any access to those points.

In view of the above-mentioned huge losses to the projects due to tunnelling problems arisen by high in-situ stresses, the ground condition is required to be extensively investigated prior to start of the underground construction. Thus, the information about ground behaviour would be available prior to excavation, and the site engineers can plan their strategy regarding excavation method and the supporting elements for the tunnel for safe and time efficient tunnelling (Dwivedi, 2015).

In-situ stresses, the main factor affecting tunnelling projects in Himalayan region are briefly described in following paragraphs in terms of the geology, in-situ stress conditions and other problems encountered during tunnelling.

2.0 IN-SITU STRESSES

Most of the major tunnelling problems are due to high in situ stresses, especially horizontal stresses caused by tectonic activities in the region. The in-situ stresses are one of the main input parameters used in stability analysis of the underground structure like tunnels. In India, particularly, in the lesser Himalayas, where the geology is highly complex and fragile, such problems have encountered in the past. The rock masses undergo intense tectonic activity giving rise to major faults, folds and other discontinuities (Fig. 1 & 2). Such problems are rarely encountered in the peninsular (southern part) India, where rocks are hard, rigid, strong and less disturbed.

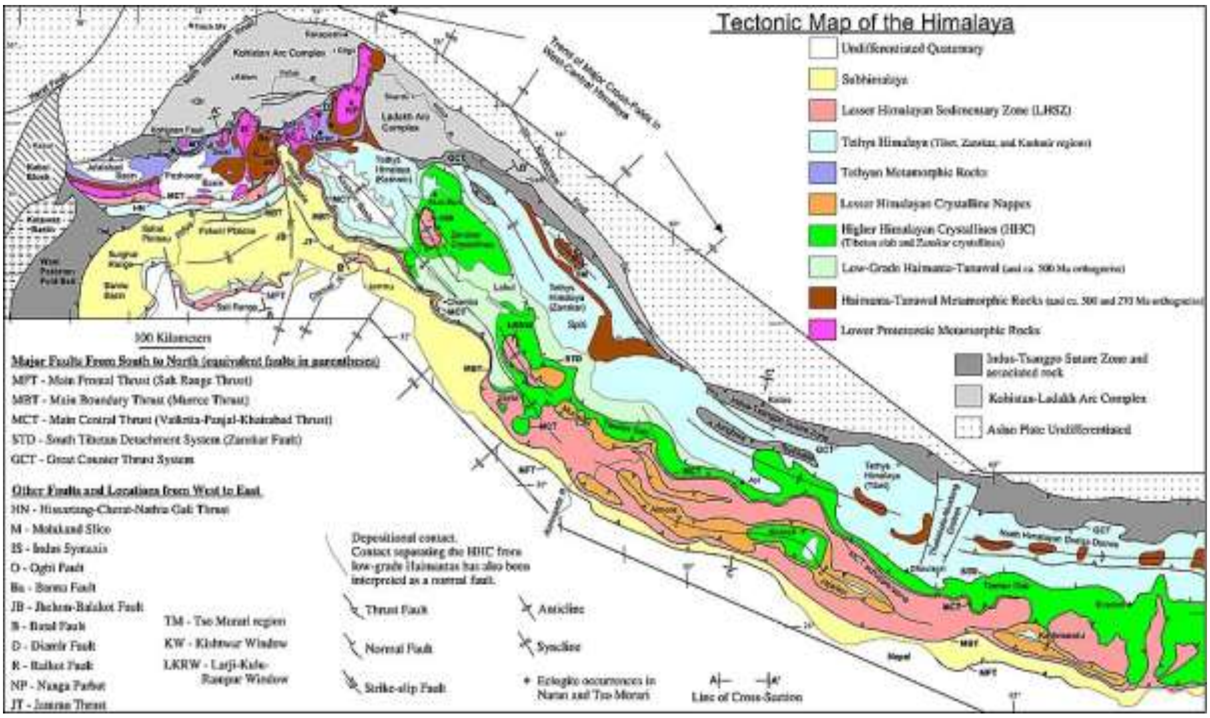


Fig. 1 : Tectonic map of Himalaya (Di Pietro and Pogue, 2004)

Several hydroelectric projects in India are in the lower Himalayas. Some of these projects lie in India (states of Jammu & Kashmir, Himachal Pradesh, Uttarakhand and Manipur) and Bhutan (Table 1). The projects listed in the Table 1 at Sl. Nos. 1-8 and 9-10 experienced squeezing ground condition and rock burst condition respectively. Squeezing ground condition is characterized by large tunnel diameter, weak rock mass quality and high in situ stress state, whereas rock burst takes place in the presence of high in situ stress state in large size tunnels excavated through strong rock masses like granites, quartzites, basalts etc. Squeezing phenomenon is time dependent and rock load go beyond control, if adequate supports are not timely installed. On the other hand, rock burst takes place on sudden release of stored elastic strain energy in the rock mass due to high induced stresses. Shape of openings also plays an important role especially in squeezing ground. It is therefore proper geotechnical investigations are highly recommended before starting tunnelling, so that the ground behaviour can be predicted and preparatory arrangements can be done timey.

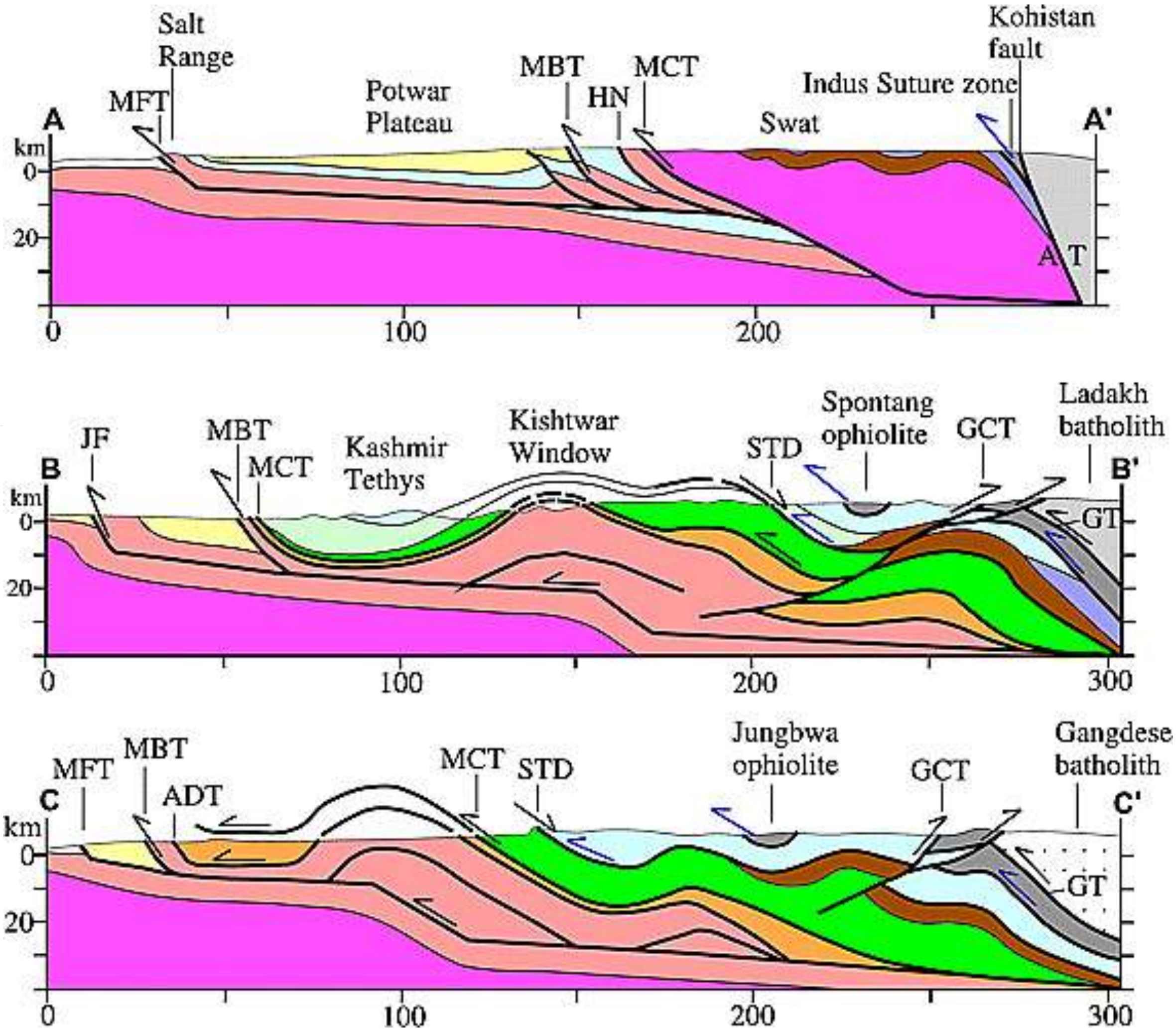


Fig. 2 : Sections A, B and C as indicated in Figure 2 (Di Pietro and Pogue, 2004)

Table 1 : Some affected tunnelling projects briefly (Dwivedi, 2015)

Sl. No.	Name of Project	Type of Project	Place
1.	Udhampur Tunnels	USBRL project	Udhampur, J&K, India
2.	Salal	Hydroelectric project	Reasi, J&K, India
3.	Giri-Bata	Hydroelectric project	Himachal Pradesh, India
4.	Chhibro-Khodri	Hydroelectric project	Uttarakhand, India
5.	Loktak	Hydroelectric project	Imphal, Manipur, India
6.	Nathpa Jhakri	Hydroelectric project	Shimla, HP, India
7.	Tala	Hydroelectric project	Chukha Dzongkhag, Bhutan
*8.	Chenani-Nashri	Highway (NH) project	NH-44 (Patnitop), J&K, India
9.	Tapovan Vishnugad	Hydroelectric project	Uttarakhand, India
10.	Parbati II	Hydroelectric project	Himachal Pradesh, India

Notation: *-The tunnel is now known as Dr. Shyama Prasad Mukherjee tunnel

Following projects encountered squeezing problems while tunnelling:

3.0 PROJECTS AFFECTED DUE TO HIGH IN-SITU STRESSES

3.1 Udhampur Railway Tunnel

Indian Railways are linking the State of Jammu & Kashmir through the Kashmir valley in Himalayas, with a broad-gauge railway line which is below the snow line making it an all-weather route. The total route length is 342 km, out of which about 100 km is passing through the tunnels. The ruling gradient of these tunnels / railway tracks is 1 in 100 and the maximum degree of curvature is restricted to 2.75º. Udhampur - Katra section is the 1st phase of Udhampur-

Srinagar - Baramula Rail (USBR) Link Project, which is 25 km long and involves construction of 7 tunnels aggregating to 10 km. Tunnel No. 1 is D-shaped and having 6.5 m diameter and a height of 8.25 m. It is the longest tunnel of this section with a length of 3.1 km

The total length of the railway line between Jammu and Kashmir Valley is 342 km, with approximately 100 km running through tunnels. These tunnels and tracks have a ruling gradient of 1 in 100, and the maximum curvature is limited to 2.75°.

The Udhampur–Katra section represents the first phase of the Udhampur–Srinagar–Baramulla Rail (USBR) Link Project. This 25 km stretch includes the construction of seven tunnels, totaling around 10 km in length. Among these, Tunnel No. 1 is the longest, extending 3.1 km. It features a D-shaped cross-section with a diameter of 6.5 meters and a height of 8.25 m (Goel et al., 2004).

The tunnel falls in Shiwalik Group and Pleistocene Formation and traverses through unconsolidated or poorly consolidated sediments with rocks of upper/middle/lower Shiwaliks and Murree formations. It passes through thickly bedded, moderately soft, sparsely jointed sandstones, sheared clay stones, siltstones and the overburden comprising of boulders / pebbles in sandy /silty matrix. Claystone/siltstone beds have 3 sets of closely spaced joints with random joints dipping at 60-70o. Strike of the joints makes an angle of 30o with the tunnel axis. In the stretch from 270 m to 313 m, which is comprised of weak rock formation (claystone and siltstone), the tunnel experiences squeezing condition. Rock Mass Quality, Q-values of claystone and siltstone vary between 0.041 and 0.2 and the stand-up time is approximately 1 day (CIMFR, 2007). In-situ stresses were measured using flat jack technique and the ratio of horizontal to vertical in-situ stresses (k) was found out as equal to 1.2 at a depth of 300 m (Dwivedi et al., 2014).

The tunnel was excavated by drill & blast method. Squeezing behaviour of rock mass was experienced during tunnelling. Steel rib supports (ISHB 200) were installed at a spacing of 0.75 m centre to centre. Load cells and closure studs were installed up to 3 m behind the tunnel face. Support pressure was observed to be 0.30-0.52 MPa, whereas the radial deformation was recorded to be 1.5 - 3% (CIMFR, 2007).

3.2 Salal HEP Tunnel

The Salal Hydro Electric Project (HEP) is located on river Chenab near Reasi in Udhampur district of J&K, India. The total installed capacity of the project is 690 MW with 6 units of 115 MW each. Two tail race tunnels, each of 2.5 km in length, were constructed with a finished horse-shoe section having a width of 11.0m. The excavation was carried out with the conventional method and supported by steel ribs with concrete backfill.

The tunnel alignment crosses the hill under a maximum cover of about 630 m. The longitudinal geological cross-section along the tunnel alignment is presented in Fig 3. The tunnel is excavated through dolomites of the great limestone series of Precambrian age. The dolomites are moderately jointed, and the joint walls are weathered. At some places, dolomites are highly jointed and sheared. Three regularly spaced prominent joint sets are present. The bedding joints dip at 40-55o due North. The transverse joints dip at 70-80o due West and the cross joints dip at 30-40o due South. The strike of these joints makes an angle of 65o with the tunnel axis at the outlet end and 20o at the inlet end.

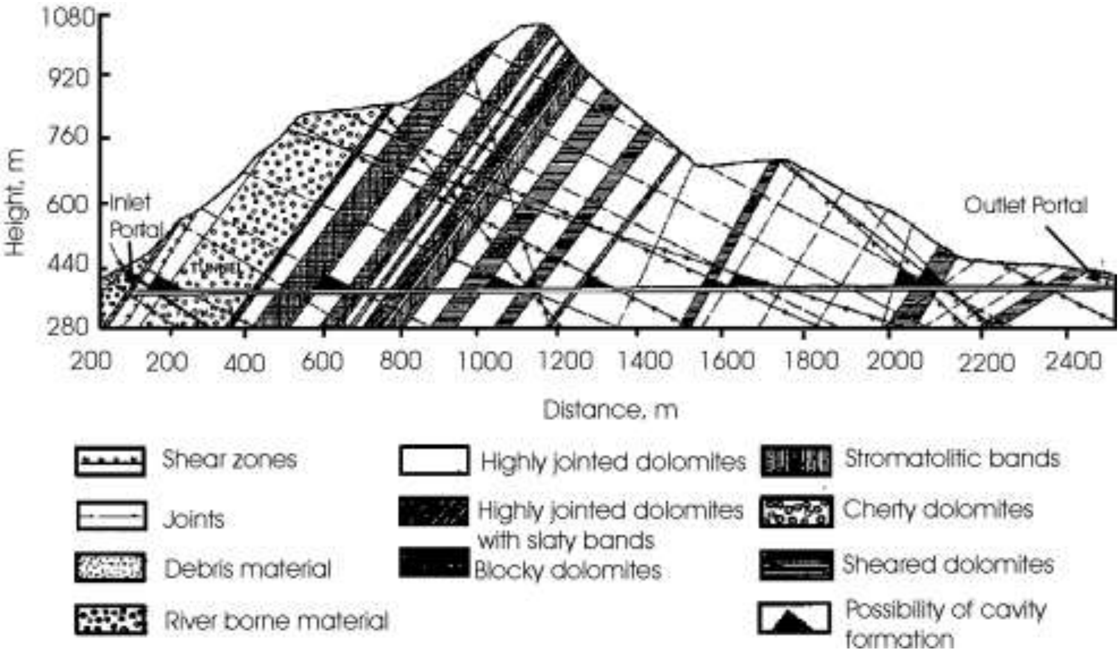


Fig. 3 : Geological cross-section along Salal hydro power project tunnel (Jethwa, 1981)

Several shear zones were also encountered. These shear zones vary in thickness from a few centimetres to a couple of meters and are filled with crushed calcareous matter, a product of crushing of dolomites due to shearing. The Q and RMR (Rock Mass Rating) values of jointed dolomites encountered in the tunnel are 1.2-1.7 and 41 respectively.

The dolomites are classified into three categories depending upon the joint spacing and the degree of disintegration and their possible extent into the tunnel is given in Table 2 (Dwivedi, 2015).

Table 2 : Classification of dolomites of Salal hydro power project (Dwivedi, 2015)

Sl. No.	Joint Spacing (cm)	Type of Dolomites	Length of Tunnel (%)
1.	30-100	Cherty and blocky	5-10
2.	5-30	Highly jointed	80
3.	<5	Crumbly and sheared	10-15

Presence of three joint sets and shear zones at the tunnel grade has created many wedges above the tunnel crown (Fig. 3). These wedges have become unstable and have given rise to the instability problems when installation of the supports was delayed. Squeezing problem was encountered between chainage 660 m and 1300 m, where the overburden is up to 300 m.

3.3 Giri-Bata HEP Tunnels

This project with an installed capacity of 120 MW, was constructed on Giri river, a tributary of Himalayan River Yamuna. It is located near Girinagar in Sirmour district of the state of Himachal Pradesh in India. A 7.1 km long head race tunnel with a finished diameter of 3.60 m was driven through a ridge separating the valleys of Giri and Bata rivers (Dube, 1979). The tunnel was excavated by drill & blast method and was supported by steel ribs. Plain cement concrete lining of 300 mm average thickness was applied as final support.

The tunnel traverses through Blaini series rock formations of carboniferous age for a length of about 1500 m and through highly jointed clay stones, highly crushed phyllites and siltstones for the remaining length. The Blainis are dark grey to black quartzitic slates containing angular to rounded pebbles and boulders firmly embedded in a clay-silt matrix. The rock formations showed extensive jointing and shearing at places and the strike generally remained parallel to the tunnel alignment. Blaini's slates, near the fault at a chainage 1350 m also posed serious problems during construction because of high tunnel closures, which were of the order of 10%. Joints were spaced at 45-50 mm and dipping at 60-70o. These formations were highly crushed exhibiting a low angle of internal friction lying between 20^{o} and 26^{o} (Dube, 1979). In-situ stresses were measured using flat jack technique and the ratio of horizontal to vertical in-situ stresses (k) was determined to be equal to 2 (Dwivedi et al., 2014).

Most of the tunnelling problems were faced in zones of phyllites and slates. Load cells and closure studs were installed up to 3 m behind the face. Support pressure was observed in the range of 0.2-0.5 MPa. Plain cement concrete lining of 300 mm average thickness was applied as final support.

3.4 Chhibro-Khodri HEP Tunnels

The project was constructed on river Tons, a tributary of Yamuna River located about 45 km North of Dehradun in the state of Uttarakhand, India. Tunnel with a finished diameter of 7.5 m was constructed between Chhibro and Khodri to utilise discharge of the Chhibro powerhouse for generating 120 MW of power through a surface powerhouse at Khodri.

The Chhibro-Khodri tunnel passes through three geological formations namely, Mandhali series (Paleozoic), Subathu-Dagshai (Lower Miocene) and Nahan series (Upper Tertiary). Mandhali series consists of boulder slates, graphitic & quartzitic slates and Bhadraj quartzite unit with 5-10 m thick crushed quartzite along Krol thrust (Fig. 4). Subathu-Dagshai series is comprised of (i) 1-3 m thick plastic black clays, (ii) red & purple coloured crushed, brecciated & sheared shales and siltstones, (iii) minor grey and green coloured crushed quartzites, (iv) 20-22 m thick black clays with thin bands of quartzites, and (v) 5-10 m thick soft and plastic black clays along the Nahan thrust (Jain et al., 1975). Nahan series is comprised of greenish grey to grey micaceous (Upper Tertiary) sandstones, purple siltstones, red, purple, grey and occasional mottled blue concretionary clays. General strike of these litho units is nearly perpendicular to the tunnel axis and the dip ranges from 20o to 60o in NNW to NNE direction (Shome et al., 1973). There are two main boundary faults running from the state of Punjab in the North to the state of Assam in the East along the foothills of the Himalayas. The faults are known locally as the Nahan and the Krol thrusts. The dip of the Nahan and the Krol thrusts varies from 27o to 30o due N10oE to N10oW and 26o due N26o W respectively. The strike of joints makes an angle of 40-50o from the tunnel alignment.

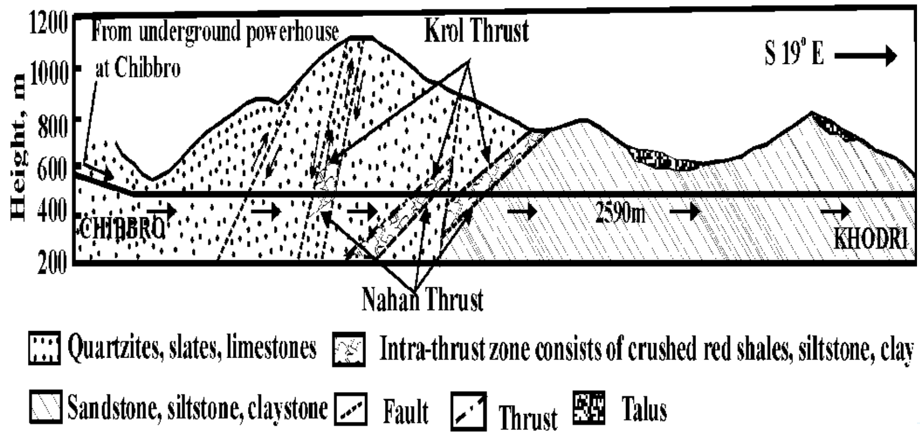


Fig. 4 : Geological cross section along Chhibro-Khodri tunnel (after Jain et al., 1975)

3.5 Loktak HEP Tunnel

This project lies 39 km south of Imphal, the capital city of Manipur State in northeast India. It diverts 58.8 m3/s of water from Loktak lake to supply 16.8 m3/s for irrigation. The remaining 42 m3/s of water with a gross head of 312 m is used to generate 105 MW of power from three units. The finished diameter of 6.5 km long head race tunnel is 3.65 m.

Loktak tunnel traverses through lake deposits, terrace deposits and shales with thin bands of sandstones and siltstones. In the first stretch of about 830 m, the tunnel passes through lacustrine deposits. Terrace deposits were encountered in the next stretch of 420 m and the remaining part of the tunnel traverses through splintary shales, sandy shales with variation of slaty and phyllitic types and some sandstones under the rock cover of 300 m. The sandstones were bedded and flaggy in nature, whereas the shales were thinly laminated (Fig.5). The general trend of the rock masses was in N-S direction i.e., perpendicular to the tunnel axis.

In-situ stresses were measured using flat jack technique and ratio of horizontal to vertical in-situ stresses (k) was estimated as 1.0 (Dwivedi et al., 2014).

The tunnel was excavated by drill & blast method. The Loktak tunnel is the first tunnel in India, where NATM was used in some stretches to tackle the squeezing ground (Malhotra et al., 1982). Serious difficulties were experienced during excavation due to excessive deformation and the high squeezing behaviour of laminated shales. Steel ribs (150 mm x 150 mm) at 1 m c/c spacing were used in conjunction with - (i) 4 m long rock bolts which were provided at 1 m c/c spacing, and (ii) 150 mm shotcrete to support the tunnel.

Load cells and closure studs were installed up to a distance of 4 m behind the face. Support pressure of 0.4 - 0.6 MPa and large tunnel deformations of about 7% of tunnel diameter were observed. Conventional tunnelling was adopted to go ahead in the squeezing section of the tunnel. The diameter of the excavated tunnel was increased to accommodate the excess deformation.

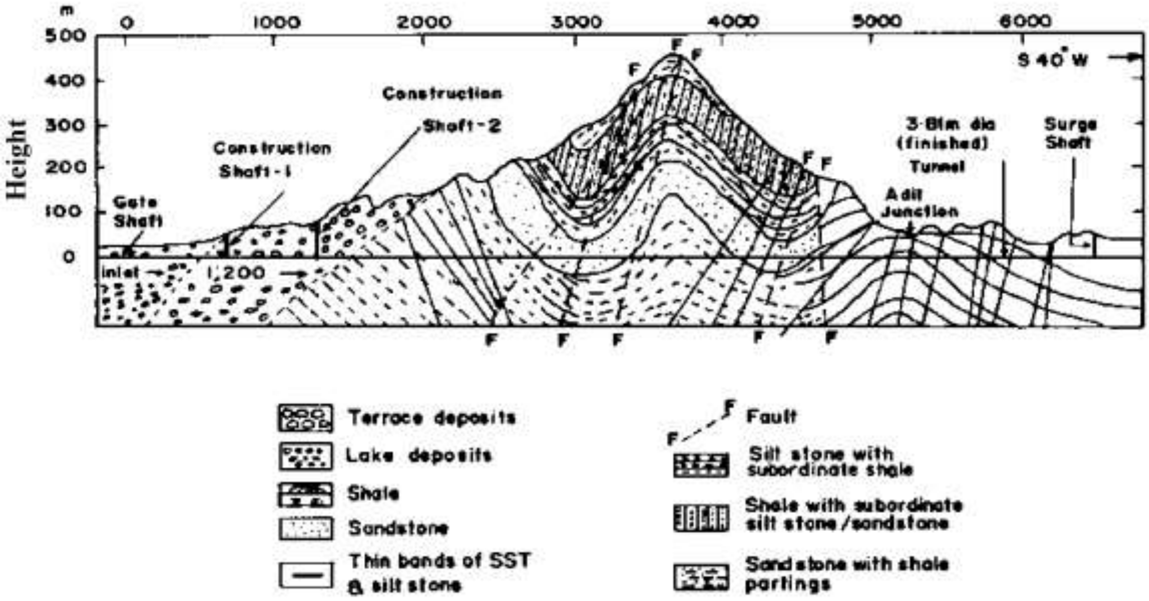


Fig. 5: Geological section along Loktak head race tunnel (Malhotra et al., 1982)

3.6 Nathpa-Jhakri HEP Tunnels

The Nathpa-Jhakri hydro power project is located between 77o and 78o longitude and 31o and 32oN latitude in the northern state of Himachal Pradesh (India), on the downstream of Wangtoo bridge and derives its name from the names of two villages in the project vicinity - Nathpa in district Kinnaur and Jhakri in district Shimla (Bhasin et al., 1995). The project was conceived as a run-off-river type hydro power development, harnessing hydroelectric potential of the middle reaches of the river Satluj, one of the principal tributaries of the river Indus in the southwestern Himalayas. The head race tunnel, with 10.15 m excavated diameter and 27394.5 m long, was constructed to carry a design discharge of 405 m^3/s.

The major rock types of the area are augen gneiss, quartz-biotite schist, amphibolites and some pegmatite lenses at places (Fig. 6).

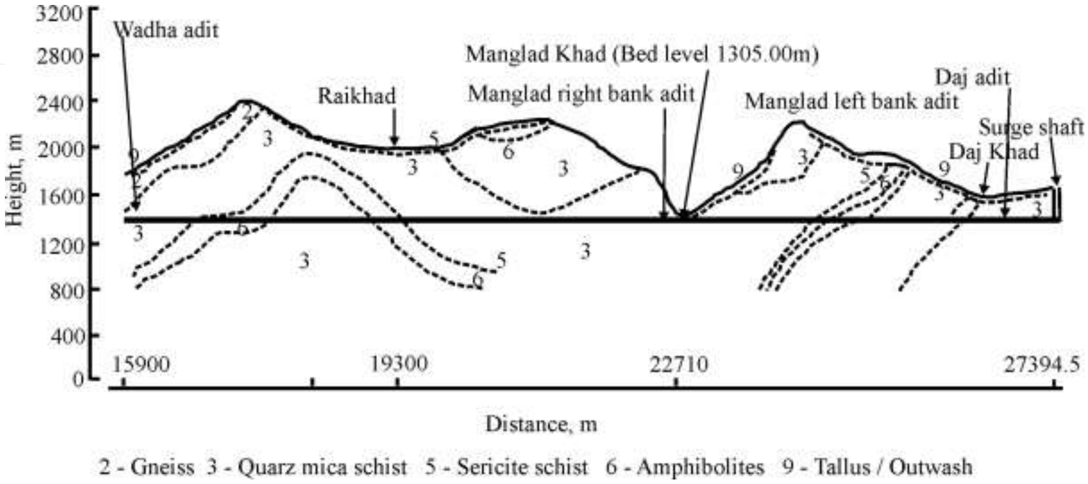


Fig. 6 : Geological section along Nathpa-Jhakri head race tunnel (Kumar, 2002)

Augen gneiss consists of two feldspars, two micas (mainly biotite), gneiss with a porphyro-blastic texture, which at places are mylonitic. The foliations are defined by the micaceous layers, which flow around the augens. The elongation direction of the augens defines a strong stretching lineation. The shape of the augens varies from nearly round to lensoidal at places, showing well drawn out porphyroclast tails. Schistocity of quartz-biotite-schist has a strong dominant character with well-defined quartzose and micaceous layers. The layers are tabular to lensoidal. At places some biotite rich lenses are also seen. Strong stretching lineation on the foliation plane is marked. At places, the biotite altering to sericite, indicated by crumpling and high fissility, is also noticed. The amphibolites are massive weakly foliated with a prominent amphibole lineation, which appears to be a primary igneous flow structure. The quartz, feldspar content is very low, and the rock is especially a biotite rich amphibolite. The amphibolites occur as narrow linear belts in the outcrop and generally unparallel to the foliation of the country rocks except at places where they are at an angle to the country rocks. Pegmatite occurs both as concordant and discordant bodies and are commonly associated with the gneisses. These are present as tabular laths. Quartz and feldspar exhibit a graphic texture and show two sets of fractures (Kumar, 2002).

There are three sets of joints, two of them are at right angles to each other and the third, oblique to them, is sub-vertical and results in wedge shaped block or rocks. Foliations were observed dipping with 30^o-70^o, 40^o-75o, 15o-55o, and 30^o-85^o towards North-East in various sections of the tunnel.

The head race tunnel (HRT) traverses through augen gneiss, gneiss, quartz mica schist, biotite schist, sericite schist, amphibolites, granite gneiss, and pegmatite (Fig. 3.10). In-situ stresses were measured using hydro fracturing technique and ratio of horizontal to vertical in-situ stresses (k) was found to be equal to 1.3 (Dwivedi et al., 2014).

The excavation of tunnel was carried out through seven adits by heading and benching method and using drill & blast technique. The geological section along the head race tunnel from chainage of 15900 m to 27394.5 m (Fig.7) at various locations posed squeezing problems during tunnelling. Large tunnel deformations were observed due to high ground stresses and poor quality of rock mass between chainage 24438.0 m and chainage 24745.0 m (rock cover of 600 m-700 m) where quartz-mica schist was striking sub-parallel to the tunnel.

The type of support varied with the category of rock mass encountered. Steel ribs (300 mm x 140 mm) were installed at a spacing of 1.25 m c/c to support the squeezing sections of the tunnel. Concrete lining of 400 - 600 mm thickness was applied as final lining (Kumar, 2002). Load cells and closure studs were installed up to 5 m behind the advancing face. Tunnel deformation and support pressure were measured and were found to be 3.5% - 6% and 0.26 - 1.02 MPa respectively. To get the required clear space, the tunnel has been over excavated by 300 mm.

Fig. 7 : Steel ribs failed in Nathpa Jhakri HRT due to squeezing ground (Hoek, 1999)

Fig. 7 : Steel ribs failed in Nathpa Jhakri HRT due to squeezing ground (Hoek, 1999)

3.7 Tala HEP Head Race Tunnel

The Tala hydro-electric project is situated in South-West Bhutan in the Eastern Himalaya. The project is in the district of Tala 3 km downstream of the existing 336 MW Chukha Hydroelectric project on river Wangchu (Sripad et al., 2007). The Tala Hydroelectric Project area falls within the central crystalline belt of Thimphu Formation and meta-sediments of Paro Formation. The Thimphu Formation comprises of a variety of granitoid rocks, such as megmatite, augen-gneiss, banded-gneiss, granitic-gneiss, and schistose rocks with subordinate quartzite and marble bands. The Paro Formation consists of high-grade calcareous rock and meta-sedimentaries such as marble, calc-silicate rock, quartzite, quartz-garnet-staurolite-kyanite-silmenite schist, graphite schist etc. with subordinate felspathic schist and gneiss bands (Khushlani, 2013). It has a 22 km long head race tunnel (HRT) of 6.8 m excavated diameter.

The head race tunnel traverses through highly weathered biotite schist associated with banded gneiss amphibolites and quartzites. In-situ stresses were measured using flat jack technique and ratio of horizontal to vertical in-situ stresses (k) was found to be equal to 0.6 (Dwivedi et al., 2014).

The tunnel was excavated by drill & blast method. During tunnelling, excessive deformation was encountered at many tunnel sections due to squeezing behaviour of the poor rock mass) (completely sheared & highly weathered biotite schist associated with banded gneiss, amphibolites and quartzites in thin bands) present around the tunnel periphery (Fig.8).

Fig. 8 : Steel ribs buckled due to squeezing ground behaviour (Dev et al., 2013)

The support system provided to the tunnel is in the form of 5-6 m long rock bolts of 25 mm diameter at 2.0 m c/c spacing in combination with 175 mm steel fibre reinforced shotcrete (SFRC) as a temporary lining and steel ribs (ISMB 200 or SMB 250 at 0.5m centre to centre), along with concrete lining as a permanent support system (Tripathy et al., 2000). Load cells and closure studs were installed up to 3.5 m behind the face of advance. On measurement, tunnel deformation was observed as 2 - 3.8 %, and the support pressure was in the range of 0.61 - 0.94 MPa.

3.8 Dr. Shyama Prasad Mukherjee Tunnel

This 9km long tunnel was formerly known as Chenani-Nashri tunnel. This main tunnel is four laned facilitating bi-directional traffic highway tunnel constructed on NH-44 between km 89.00 to km 130.00. The tunnel has reduced the previous route of highway from 41 km to 9 km only (Fig. 9). The traffic diverted towards the tunnel avoids the heavy snow on the highway route near Patnitop during peak winter seasons.

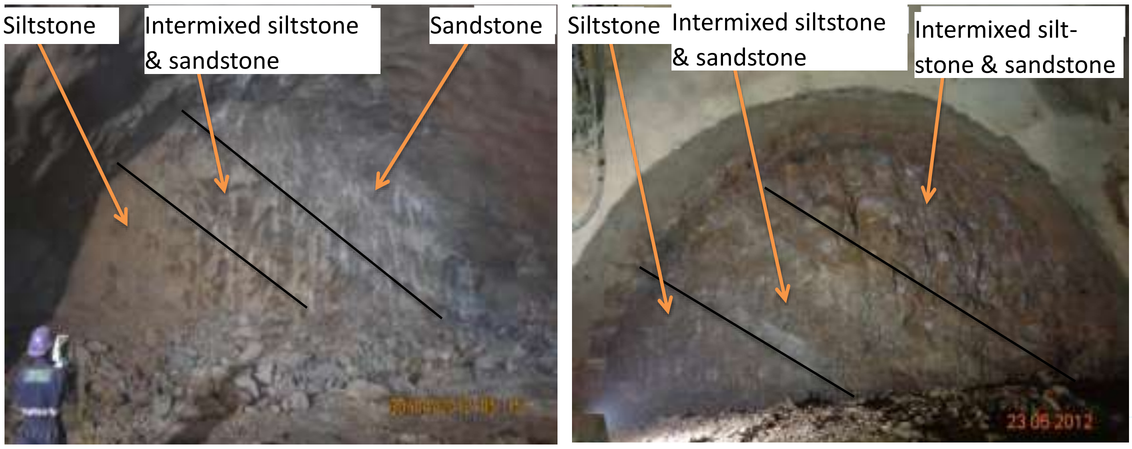
Fig. 9 : Inside view of Dr. Shyama Prasad Mukherjee tunnel

The tunnel passes through Murree formations of the Himalayas, which are influenced by the regional and the local faults and shear zones as well. Design and construction of tunnels through such a complex and varying geology with a rock cover of more than 1000 m was a difficult and challenging task.

The project area lies in Western Himalayan region in a sector of collisional belt known as sub-Himalayas. This tectonic domain is bounded towards the south by the Himalayan Frontal Thrust (HFT) or the Main Frontal Thrust (MFT) and by the Main Boundary Thrust (MBT) towards the North. The rock masses along the Chenani-Nashri tunnel project belong to the Lower Murree formation. This sedimentary succession is classified as the “Lower Tertiary Sediments” of the “Murree Structural Belt” and are bounded on the south by the MFT and on the north by a complex of thrusts, which are regionally referred to as the MBT. The Murree formation is characterised by a sequence of argillaceous and arenaceous rocks which includes a sequence of inter-bedded sandstone, siltstone and claystone beds with thickness ranging from a few metres up to 10 m (Goel et al., 2012).

Rock mass of the area has three sets of joints, i.e., bedding planes dipping at 30o-45o towards N 90o-110o, second joint set dipping at 50o-75o towards N 250o-300o and the third joint set dipping at 50o-80o towards N 200o-255o. The strike of the joints makes an angle of about 8o-33o with the tunnel axis. The bands of sandstone, siltstone, and claystone of varying thicknesses are frequently encountered during the tunnel excavation. There is no fixed pattern of the bands of these rocks.

In fact, the bands of mixed rocks, for example, intermixed siltstone and sandstone and intermix siltstone and claystone are also encountered frequently (Fig.10a & b). The uniaxial compressive strengths of freshly obtained rock samples of sandstone, siltstone and claystone are 50-60 MPa, 30-40 MPa and 20-30 MPa respectively. Squeezing problem was encountered in siltstone, claystone and intermixed rock sections. The intermixed rocks have a uniaxial compressive strength of 30-35 MPa.



(a) (b)

Fig. 10 : View from main tunnel, south end showing bands of various rocks of Murree formations

In addition to the above projects encountered with squeezing behaviour, head race tunnels (HRT) of Tapovan Vishnugad HEP and Parbati II HEP experienced rock burst at about 900m in quarts mica gneiss and 1100m in quartzites respectively. Both the tunnels also encountered with heavy inrush of water.

4. TACKLING STRESS INDUCED TUNNELLING PROBLEMS

The squeezing and rock burst conditions may be either avoided or well tackled during tunnelling by adopting following common guidelines:

4.1 For Squeezing and Rock Burst Ground

- In-situ stress measurement should be conducted at the tunnel grade along the tunnel alignment.
- Proper geotechnical investigation should be done by drilling bore holes or by other techniques suitably at various locations along the tunnel alignment so that the accurate L-geological section can be prepared.
- On the basis of values and direction of the in-situ stresses, if possible, tunnel shape should be designed and alignment should be fixed. In hydrostatic stress condition (k = 1), circular shape; for k > 1, elliptical shape with horizontal major axis; and for k < 1, elliptical shape with vertical major axis would be suitable. Intermediate principal axis is good for tunnel alignment. Here 'k' is the ratio of horizontal in-situ stress to vertical in-situ stress.
- Workmanship at the tunnelling site should be religiously monitored by the site engineers, as this parameter cannot be measured and analysed by any method.

4.2 For Squeezing Ground

- In case of squeezing ground condition, New Austrian Tunnelling Method (NATM) should be adopted as this method recommends continuous monitoring (stress and deformation), which indicate the degree of accuracy of the installed supports (Fig. 11).

Fig. 11 : Measurement of radial tunnel deformation

- In case of large sized tunnels, benching shall be done as near to the heading face as possible. So that the supporting rings can be closed by installing inverts in time to avoid bottom heaving (Fig. 12).

Fig. 12 : Bottom heaving

- Depending upon the trend of tunnel deformation observed during monitoring, the reading-frequency should be decided. The frequency may be increased to once in a day, if deformation trend is very high.
- In squeezing grounds, Attention and Alarm limits should be defined as per the design and the site engineers should strictly follow the timely counter measures in such a way that the additionally installed supports should become active before the tunnel deformation exceeds the alarm limit.
- Three probe holes are required to assess the geology ahead of the tunnel face. It especially helps in case of presence of any water aquifer in syncline of folded rock mass. Proper action can be taken to drain out the water if it is assessed by probing.
- The steel supports (lattice girders or ribs) shall be properly fixed at the bottom either by installing wall beam in the case of steel ribs or elephant footing in both the cases. This technique transfers load to the ground properly. Actually, in most of the cases, this arrangement is not done may be in view of keeping increasing tunnelling rate. But failing of this arrangement, especially in squeezing ground, attracts undesirable excessive tunnel deformation which may lead to failure of a long stretch of the affected tunnel.
- The tunnel deformation measuring targets (bi-reflex targets) should be installed as close to the tunnel face as possible so that minimum tunnel deformation would be missed to measure. Otherwise measured less deformation may be deceptive in indicating actual rock load (Fig. 13).

Fig. 13 : Bi-reflex targets (T1, T2,…) installed on the supporting rings

- Any water received from the tunnel strata shall be drained using plastic or steel pipes along the tunnel avoiding water seepage towards bottom of the tunnel. This seeped water may attract excessive deformation due to settlement of the heading support (lattice girder/ steel rib), which may lead to stability problem.

4.3 For Rock Burst Ground

- Core disking should be checked while coring through bore holes. This is clear cut indication of possible rock burst case (Fig. 14). However, this phenomenon can be predicted by checking the case if tangential stresses exceed 0.6 times uniaxial compressive strength (UCS) of the rock.

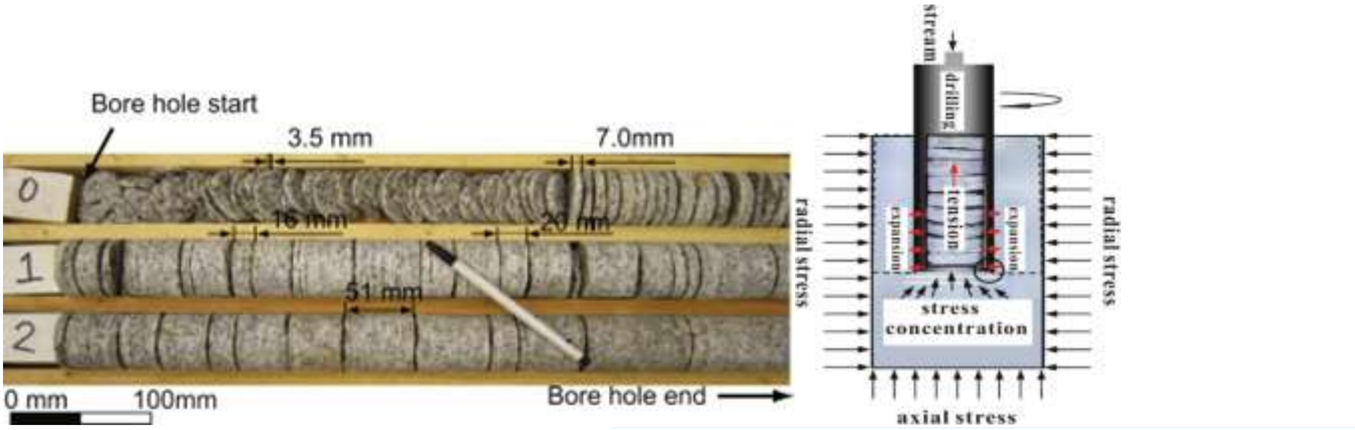


Fig. 14 : Core disking in case of rock burst

- Try to go for stable shape, if possible.
- Release stresses by destress blasting and reinforce the rock mass to increase the ductility to dissipate some strain energy.
- Hold with frictional or yielding (cone) rock bolts and retain with mesh or deformable shotcrete panels.
- Experienced tunnel engineers or geologist may try to hear or measure acoustic emission at quite far from the tunnel face. Acoustic emission close to audible range indicate that rock burst may take place very near in future so the prompt action should be taken to evacuate the tunnel face immediately.

5.0 CASE STUDY OF CONTROLLED BLAST DESIGN

In another case, a large diversion (DT) was excavated using V-cut with stab holes pattern of tunnel blasting at Pakal Dul Hydroelectric project (1000MW). The Dam site of this project is located on the Marusudar River, a main tributary of the Chenab River, in the Kishtwar district (the union territory of J&K). The DT has been driven along the left bank of the dam that has an overburden ranging between 28 to 235 m. The dimension of DT tunnel is 12.2 m excavated span and 847.7 m of a total 859.7 m is excavated in hard rock. Approximately 12 m length of inlet portal of DT was made using reinforced concrete. The general view of Dam package of the project is shown in Fig. 15. Regional geology of the project area as per the Geological Survey of India, involves predominantly rock of Kishtwar group Kibar/Pipran formation (Pre Cambrian). The Kibar is the oldest Formation of Kishtwar Window zone and it comprises granitic gneiss, quartzite, and mica schist. Seismicity of area, as per seismic zonation map of India, falls in the Zone-V where events of magnitude more than equal to 7 are anticipated.

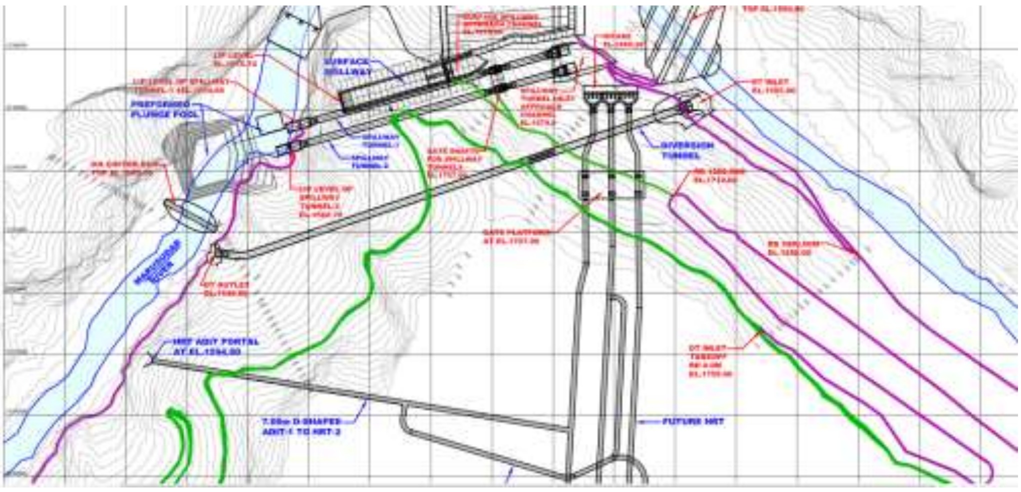


Fig. 15 . General layout of Pakal Dul HEP (1000 MW), Bhagat et al. 2025

The diversion tunnel is horseshoe-shaped with excavated span of 12.2 m. The reference profile areas of the diversion tunnel for Class I, II, and III were 126.658, 128.207, and 129.764 m², respectively. Elevation level of diversion tunnel outlet bottom is 1539 m and inlet bottom is 1555 m. The sequence of excavation of diversion tunnel is as follows:

1. Excavation of top heading of about 7.2 to 8.3 m height followed by rock bolting and 50 mm thick steel-fiber-reinforcement (SRFS).
2. Benching of remaining 3.9 to 5 m tunnel bottom portion with concurrent application of supports.

3. Excavation of inlet structure (transition length of 12 m) having larger cross-sectional area (approximately 226 m² at chainage (CH) 847.7 m to 129.764 m² at Ch 836 m) and low cover zone using multiple sections (Singh et al., 2012).

The rocks are mostly weathered grade of up to W-3, and moderately jointed. During the excavation of top heading, the geological formations traversed by the tunnel are gneissic quartzite and gneissic granite of the Kibar gneisses, interspersed with subordinate schist bands. Gneissic quartzite dominates the areas in the vicinity of the inlet and outlet portals, while the middle stretch primarily consists of gneissic granite. Quartz veins (mainly crushed material) and shear seam (<10 cm) appeared all along the full cross-sectional area regularly throughout the tunnel. The transition between these two litho-units is gradual. The rocks are characterized by medium strength and jointed rock mass, with four to five sets of joints dissecting the rock, including foliation joints that intersect the tunnel alignment almost perpendicularly. The major orientation of joint sets, rock mass rating and rock class at different chainage sections are shown in Table 3.

Table 3 : Major orientations of joint sets encountered at different chainage sections in diversion tunnel.

Ch (m)	Joint sets (°)					RQD (%)	RMR	Rock class
	J1	J2	J3	J4	Random			
0-15	350-015/15-20	200/35	195/30	065/75-80	310/60-70	50-75	68	II
100-120	035-040/55-60	230/30	170/35	035/55	-	65-70	73	II
335-345	030-050/45-65	235-250/65-70	170-190/70-85	090/45	340-350/30-50	75-85	85	I
497-514	050-055/65-80	250-280/70-85	180-190/70-85	120/80	340-350/65-80	50-70	75	II
837-847	060-110/70-80	235-290/70-80	165-170/35-80	310/75	340-355/45-80	25-50	46-53	III

The excavation sequence involved driving the D-shaped top heading with a width of 12.2 m and a height ranging from 7.2 to 8.3 m. The cross-sectional areas of the top heading portion varied between 73 and 83 m². A V-cut pattern with three horizontal central stab holes (1.6 m depth) between the first set of V-cut holes was planned for routine blasting (see Fig. 16). Drilling was performed using a double boom hydraulic drill machine with a 45 mm drill diameter. Since the depth of holes often exceeded 3.5 m, shorter stab holes were drilled perpendicular to tunnel face to ensure proper breakage in the central cut area by creating an initial cavity before detonating the first set of V-cut holes. This helps in reducing the confinement and maximizing the advance per round. In the present case, emulsion explosive of 40 mm diameter along with non-electric detonator (NED series of No. 1 to 15), i.e., shock tube initiation system, were used to excavate the rock. The packaged emulsion explosive, weighing 390 gm per cartridge and 300 mm in length, was detonator-sensitive and highly water-resistant. It had a nominal density of 1.20 ± 0.05 g/cc. The tested detonation velocity of the explosive was 4.787 km/s. It offered high energy output with reduced post-blast fumes which was ideal for tunneling. Specification of NED, along with nominal firing time (NFT) and tested actual firing time (AFT) is shown in Table 4. The velocity of detonation (VOD) of shock tube ranged between 1776 and 1834 m/s. The scattering in firing timings of various non-electric shock tube detonators (1 to 15 nos.) was within 5%. This did not cause overlapping of the sequence of designed firing timings of holes. Further, use of a minimum delay window of 300 milliseconds (ms) between charges in blasting operations helped in managing the vibrations by dividing the total charge into segments, corresponding to the number of NEDs used. This segmentation reduced the magnitudes of vibrations by preventing the simultaneous detonation of all charges, thereby avoiding their cooperative effect (Bhagat et al., 2024).

In most of the cases, drilling was conducted using 3.5 to 3.8 m hole depth with 45 mm drill diameter. Holes were charged with 40 mm diameter packaged emulsion explosives, except for perimeter holes in the crown portion where a 25 mm explosive charge was used to control overbreak. Explosive charge per hole varied between 0.78 to 4.23 kg depending upon the depth of holes. Stemming of blast holes were carried out using coarse sand filled in polythene tube of 40 mm diameter. Smaller hole depths per round were used mainly in the inlet and outlet portions when strata conditions were not favorable, at curves and in low cover zone. Excavation began through the outlet portal due to outcropping and lack of access from the inlet portal end. Approximately 300 blasts were conducted for excavating approximately 837 m length of the heading portion of tunnel, with blast hole depths varying from 2.5 to 3.8 m depending on rock mass class, discontinuities, and cover. The number of holes per round was 144 and average total charge was 430 kg. The average advance per blast obtained was 2.9 m for an average hole depth of 3.5 m. The powder factor for the top heading portion ranged from 1.1 to 3.16 kg/m³, averaging at 1.95 kg/m³.

The benching for the bottom portion of the diversion tunnel, with a width of 10.2 to 12.2 m and a depth of 3.9 to 5 m, was carried out using the horizontal hole drilling method. This involved drilling holes with a depth of 3.8 to 5.5 m using a drill diameter of 45 mm. The maximum charge per hole was 2.73 kg for production holes. The wall side hole was charged with 25 mm diameter explosive and maximum 1.25 kg was used for each hole. The powder factor varied from 0.75 to 0.9 kg/m3 with a total charge varied between 125 and 200 kg.

Table 4 : NED series nominal and actual firing time used in study.

NED Nos.	NFT (ms)	AFT (ms)	Scattering (%)	Shock tube VOD (m/s)
1	300	290.4	3.20	1812
2	600	580.2	3.30	1811
3	900	892.4	0.84	1798
4	1200	1150	4.17	1822
5	1500	1433.6	4.43	1800
6	1900	1970.9	-3.73	1821
7	2300	2385.7	-3.73	1782
8	2700	2775.8	-2.81	1834
9	3100	3065.2	1.12	1826
10	3500	3493.6	0.18	1840
11	4000	4049.9	-1.25	1830
12	4500	4496.4	0.08	1776
13	5000	4872.7	2.55	1805
14	5500	5488.4	0.21	1842
15	6000	5923.1	1.28	1800

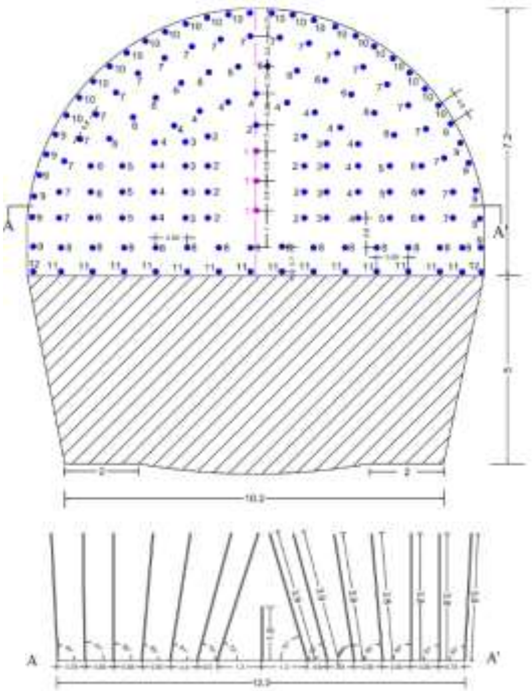


Fig. 16 : Layout of V-cut pattern with stab holes for heading potion (All dimension in m)

The 12 m section of the rock plug in the inlet structure transition was excavated in multiple stages due to its height of 15 m and a cross-sectional area ranging from 226 to 130 m2, with only 28 m of cover. The rock mass consisted of gneissic quartzite with schist, shear seams (less than 10 cm), and five sets of joints. The rock mass rating (RMR) and RQD percentages varied between 46 to 53 and 25 to 50%, respectively. The excavation sequence for the inlet portal began with the top right heading, followed by the left heading, maintaining a 2 m barrier between them. After supporting the top heading portion, benching was completed in two stages. The depth of holes varied between 2 and 2.5 m, with a charge per hole ranging from 0.79 to 1.58 kg. The maximum charge per delay ranged from 6.24 to 15.6 kg, while the total charge varied between 23 and 50 kg. Fig. 17 illustrates the initial left side opening and subsequent daylighting of the diversion tunnel.

The V-cut pattern, characterized by shorter stab holes, offers several advantages in tunnel blasting. It reduces the requirements for drilling and explosives, as well as the time needed, particularly when dealing with complex geological challenges such as quartz veins and jointed rock masses. This makes it more suitable than the parallel cut pattern for excavating a 12.2 m wide diversion tunnel. The V-cut pattern is compatible with mechanized double boom drilling machines, enhancing efficiency and cost-effectiveness compared to wedge and parallel cuts. The shorter stab holes create initial cavities that facilitate optimal breakage of the first set of V-cut holes. The advance per round ranges from 2 to 3.5 m out of a hole depth of 2.5 to 3.8 m, resulting in an advance per round of 60.6% to 95% across different rock mass classes. However, the appearance and disappearance of quartz veins throughout the tunnel length and extremely hard rock mass, coupled with four sets of joints, posed significant geological challenges that affected the desired advance rate of 95%. Despite these constraints, the average advance per round was 80.7%, demonstrating the effectiveness of the blast designs in achieving targets, especially considering the project's remote location.

Fig. 17 : View of initial opening of inlet portal (left) and daylighting of diversion tunnel (right)

The overbreak profile of the diversion tunnel was assessed using the Amberg Laser Tunnel Profiler TS16 A 1" R1000 at intervals of 1 to 5 m. The overbreak, calculated at 10-m intervals, is depicted in Fig. 18. The overall overbreak for the DT was 9.26%. In different rock classes, the overbreak was as follows: Class I (105 m) had 9.87%, Class II (563 m) had 9.34%, and Class III (179.7 m) had 8.95%. It was noted that overbreak during benching operations was greater than during top heading. The relatively higher overbreak was attributed to several factors:

1. Interaction of four joint sets and random joint forming natural blocks that slid along the joints. Presence of quartz vein also worsen the conditions.
2. Low-skilled drill operators, frequent changes of trained drillers, and inadequate supervision, leading to improper drilling of perimeter holes according to the cross-section of different rock class. This is also evident from the average percentage of overbreak in Class III rock mass which is less than Class I & II.
3. Use of a higher powder factor than the required for proper breakage specially in perimeter holes by using 40 mm diameter explosives instead of the 25 mm diameter explosives.

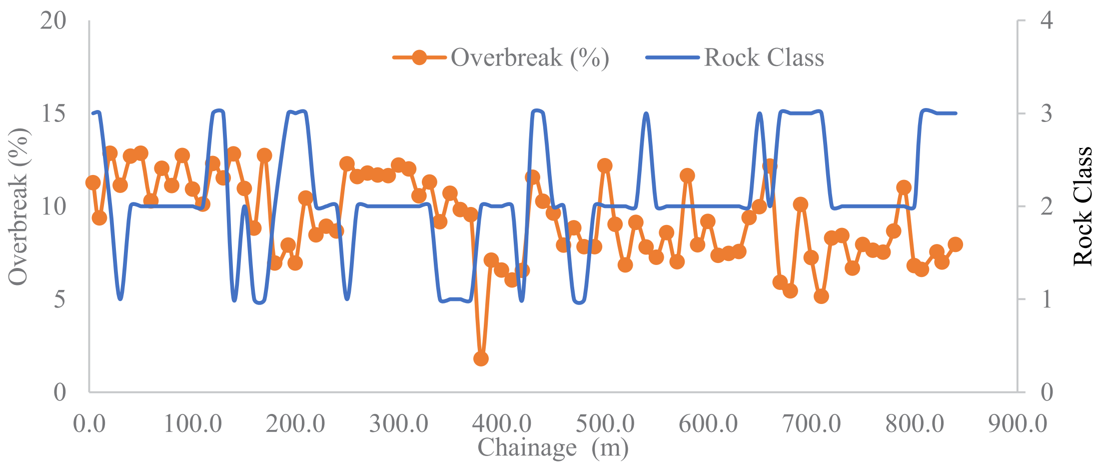


Fig. 18 : Overbreak along the length of Diversion Tunnel.

The magnitude of vibration generated during the tunnel blasting was monitored on vertically above on ground surface near public structures as well as approachable locations and lower cover zone. The maximum magnitude of vibration recorded at 28 m distance due to blast at inlet portal was 9.68 mm/s with a maximum charge per delay of 15.6 kg and total charge of 50 kg. Whereas, the minimum magnitude was 0.508 mm/s at 180 m due to top heading blasting at

RD 608.5 m with 139 number of holes having maximum charge per delay of 54.7 kg with total charge 525 kg. The regression plot of recorded data using cube root scaling law is shown in Fig. 19. The recorded vibration never crossed 1 mm/s near the public residential structures which were situated more than 200 m distances.

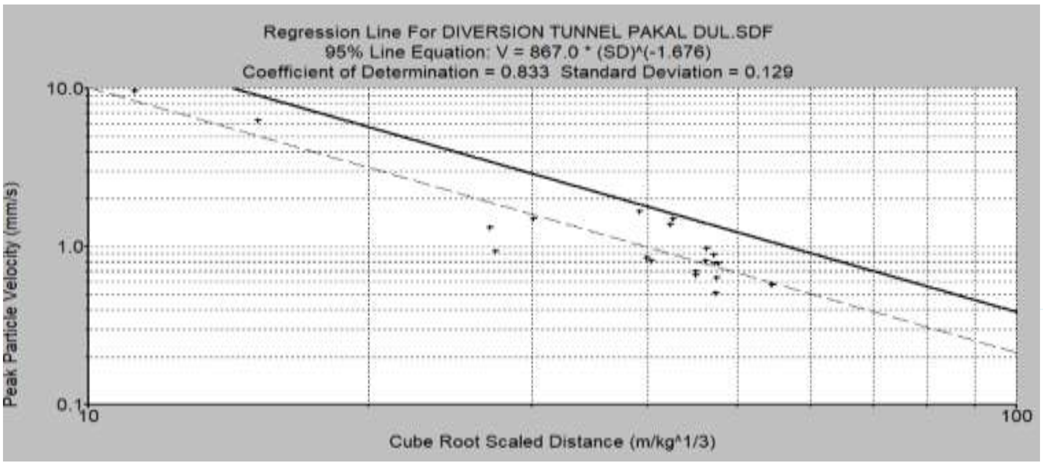


Fig. 19 : Regression plot of vibration recorded at surface ground at Pakal Dul Dam Site.

5.0 CONLUDING REMARKS

The described case projects in the present keynote indicate that the tunnelling problems associated with high in-situ stresses may result in huge time and money loss to the respective tunnelling projects and the problems can be either avoided or atleast minimized to a certain extent by proper geotechnical investigations including in-situ stresses measurement. In case of squeezing grounds, probe holes at the tunnel face are recommended to know the geology ahead of the tunnel face. It gives especially prior knowledge of presence of an aquifer so that heavy inrush of water can be avoided. Uncontrolled water inrush multiplies the squeezing phenomenon by many folds. It is because water reduces the shear strength of the rock joints considerably giving rise in increased plastic radius of the broken zone around the tunnel periphery.

In case of rock burst distress blasting and application of support having high energy absorbing capacity like cone rockbolts considerably help in reducing the stability problems. In addition to this, the experienced tunnel engineers are suggested to keep an eye on acoustic emission so that the tunnel can be evacuated in time to minimize the damage to deployed man and machineries.

The successful tunneling in the Himalayan region demands a multi-dimensional approach tailored to highly fractured rock masses, varying hydrogeological conditions, and extreme loads. By rigorously analyzing we may conclude that (i) Customizing blast design to suit local geology, including precision in cut hole spacing, charge concentration, timing, and feed pressure, effectively addressed operational inefficiencies caused by jointed and wet rock masses. (ii) Continuous monitoring, and drill operator skills and strict blasting protocols and integrated engineering controls resulted in improved safety, reduced vibrations, and robust tunnel performance in complex environments.

Overall, this work provides valuable technical guidance that will aid tunneling professionals in designing and executing safe, efficient, and economically sound projects in similar complex geological environments as tunnel construction expands in the Himalayas and comparable terrains worldwide.

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RESILIENT AND SUSTAINABLE HYDROPOWER IN THE HIMALAYAS: THE ROLE OF ADVANCED PVC GEOMEMBRANE WATERPROOFING IN THE INDIA-BHUTAN GREEN ENERGY PARTNERSHIP

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ABSTRACT

The development of hydropower resources in the Himalayas represents a strategic imperative for the energy security and economic growth of India and Bhutan. However, this potential is constrained by an environment characterized by extreme geotechnical, seismic, and climatic vulnerabilities. Conventional waterproofing methods, such as concrete and asphalt facings, have demonstrated significant inadequacies in these high-stress conditions, frequently failing under structural movements and leading to chronic seepage, operational inefficiency, and accelerated deterioration of critical assets. This paper presents an advanced, flexible polymeric geomembrane waterproofing technology, specifically the CARPI PVC-P system, as a technologically superior and sustainable alternative. The system's core attributes–near-absolute impermeability with a hydraulic conductivity of approximately 10-14 m/s, high flexibility with elongation exceeding 200% to accommodate seismic and settlement stresses, and rapid installation protocols—directly address the primary failure modes of rigid linings. The efficacy of this technology is validated through an in-depth analysis of six diverse case studies from India: the Kadamparai, Servalar, and Upper Bhavani dams; the Tanakpur power channel; the Bajoli Holi pressure tunnel; and the Pinnapuram Pumped Storage Project. These examples demonstrate dramatic and sustained reductions in leakage, enhanced operational longevity, and significant construction timeline advantages. Critically, the paper highlights the system's profound sustainability credentials, including a documented reduction in embodied CO_2 of up to 70% compared to conventional methods and a fundamental contribution to water security. These benefits position advanced PVC geomembrane technology as an indispensable tool for realizing the "Green Future" envisioned by the India-Bhutan partnership, offering a pathway to de-risk investments, ensure long-term asset resilience, and build a truly sustainable energy infrastructure.

1.0 Introduction: The Hydropower Imperative in a Fragile Environment

The pursuit of clean, renewable energy is a global priority, and for the nations of South Asia, the vast, glacier-fed river systems of the Himalayas represent one of the world's most significant sources of untapped hydropower potential. The partnership between India and Bhutan stands as a model of cross-border cooperation aimed at harnessing this resource for mutual economic development and regional energy security. However, the very geological forces that created the region's immense hydropower potential also render it one of the most challenging engineering environments on Earth. The success of future development is therefore contingent not on ambition alone, but on the adoption of advanced technologies capable of ensuring the long-term resilience and sustainability of critical infrastructure

1.1 The Strategic Importance of Himalayan Hydropower for India and Bhutan

The Himalayan mountain range is the source of river systems that hold an estimated 80% of India's total hydropower potential of 148,700 MW.1 For both India and Bhutan, developing this resource is a cornerstone of their respective strategies to transition away from fossil fuels, meet escalating energy demands, and drive economic growth.[2] Unlike intermittent renewable sources such as solar and wind, the perennial, snow-fed nature of Himalayan rivers provides a reliable, baseload power supply capable of stabilizing national grids and supporting industrialization.[3] The India-Bhutan partnership, in particular, leverages Bhutan's immense per-capita hydropower potential and India's vast energy market and technical expertise, creating a symbiotic relationship that is crucial for achieving regional renewable energy targets and enhancing energy security.[2]

1.2 Inadequacies of Conventional Waterproofing in High-Stress Environments with Geotechnical, Seismic impacts

The viability of hydropower projects in this region is perpetually tested by a formidable combination of geological, seismic, and climatic hazards. These challenges are not discrete risks but form an interconnected "threat matrix" where one hazard often intensify another, demanding a holistically resilient engineering approach

- **Geology:** The Himalayas are the world's youngest and most tectonically active mountain range. This geological dynamism results in rock masses that are severely fractured, deeply weathered, highly folded, and traversed by numerous shear zones and faults translating to high rock mass permeability, creating persistent and severe risks of water ingress
- **Seismicity:** The entire Himalayan belt is classified under the highest-risk seismic categories, Zones IV and V, posing a severe threat to the structural integrity of large, rigid civil structures like concrete dams, spillways, and tunnels.
- **Climate and Hydrology:** The challenges are further compounded by the accelerating impacts of climate change.
- **Logistics and Construction:** The remote and rugged terrain imposes severe logistical constraints on construction. Difficult access, the need to build extensive road networks through fragile slopes, and a very short working season dictated by heavy monsoons and winter snows all contribute to extended project timelines, increased costs, and heightened construction risks.[14]

For decades, the standard for waterproofing hydraulic structures has been the use of rigid facings, primarily concrete and asphalt. While functional in stable geological settings, these conventional methods are fundamentally ill-suited for the dynamic and high-stress environment of the Himalayas. Their primary limitation is their rigidity and brittleness. Under the combined effects of hydrostatic pressure, thermal cycling, and, most critically, the differential settlement and seismic ground shaking endemic to the region, these materials are prone to cracking.

Once cracks form, the waterproofing function is compromised, initiating a cycle of degradation. Seepage through these fissures not only represents a direct loss of valuable water—a chronic issue in India, where losses in irrigation canals alone can reach 30-50%—but also accelerates the deterioration of the underlying structure. This leads to escalating operation and maintenance (O&M) costs and can ultimately threaten the structural integrity of the asset. The performance gap of these traditional methods underscores the urgent need for a more resilient and adaptable solution.

2.0 The CARPI PVC Geomembrane System: A Technologically Advanced Solution

In response to the limitations of conventional rigid facings, advanced geosynthetic engineering has produced flexible polymeric geomembrane systems. The CARPI system, based on a high-performance plasticized polyvinyl chloride (PVC-P) formulation, represents a paradigm shift from a simple barrier to an integrated, engineered waterproofing and monitoring system. Its design philosophy directly counters the failure modes of traditional materials by prioritizing flexibility, absolute impermeability, and long-term durability, creating a level of performance that no single component could achieve on its own.

2.1 Material Science and System Components

The CARPI system is not a monolithic product but an engineered assembly of several, synergistic components: majorly the geomembrane, a protective geotextile, a drainage layer, and a robust anchoring system.

- **Geo-Membrane:** The core of the system is a high-performance PVC-P geomembrane, specifically engineered for extreme durability and flexibility in hydraulic applications. Its material properties are designed to withstand the immense stresses encountered in large dams and tunnels. Key performance specifications include a tensile strength exceeding 15 MPa and, crucially, an elongation capacity of over 200%. This high elongation is the fundamental property that allows the membrane to stretch and deform without rupturing, accommodating the structural movements that would cause a rigid facing to crack. The membrane thickness is tailored to the application, ranging from 1.5 mm for lower-head structures like canals to 4 mm for high-head dams and pumped storage projects.
- **Geotextile:** Placed directly against the substrate, beneath the geomembrane, is a heavy nonwoven geotextile layer with a mass typically ranging from 500 to 2,000 g/m². This layer serves a critical dual function. Firstly, it acts as a protective cushion, safeguarding the geomembrane from puncture or abrasion by irregularities in the underlying concrete or rock surface. Secondly, and more innovatively, it functions as an integrated drainage layer. This design transforms the system from a passive barrier into an active, monitorable asset. In the unlikely event of a breach in the membrane, any seepage is intercepted by the geotextile and safely channelled to collection points and drains. This allows for early detection, quantification, and localization of leaks, a "leak-before-break" philosophy that is a significant advancement in dam safety protocols.
- **Drainage Arrangement :** Carpi's geomembrane system is predominantly the first and the only waterproofing system accompanied by a drainage system that helps to discharge any kind of drained water behind the membrane and also allows to discharge any negative water entry due to pressure difference between the water stored body and the subgrade pore pressure.
- **Anchoring:** The system's integrity is secured by a robust anchoring mechanism composed of stainless steel or marine-grade aluminum profiles. These profiles are mechanically fixed to the structure at the crest, abutments, and other peripheries. The geomembrane is then clamped into these profiles, creating a continuous, mechanically secured, and fully watertight seal. On steep slopes or vertical faces, intermediate anchors are used to distribute the immense weight of the membrane and prevent slippage under gravity and hydrostatic loads. This modular design also facilitates targeted repairs without requiring major dewatering of the structure.

2.2 Key Performance Characteristics: Impermeability, Flexibility, and Durability

The synergy of these components delivers a set of performance characteristics that are demonstrably superior to conventional methods in high-stress environments.

- **Impermeability:** The PVC-P geomembrane possesses an extremely low hydraulic conductivity, on the order of 10^{-14} m/s.[16]
- **Flexibility:** The system's capacity for elongation greater than 200% is its defining advantage in the tectonically active Himalayas. This inherent flexibility allows the geomembrane to absorb and accommodate significant structural movements—whether from thermal expansion and contraction, differential settlement of foundations, or, most critically, seismic ground shaking—without compromising its watertight integrity. It is a direct countermeasure to the primary failure mode of rigid linings: brittle fracture under strain.
- **Durability:** The engineered PVC-P material, combined with the protective geotextile, is designed for exceptional longevity. The system has a projected service life of 30-50 years in fully exposed applications and over 100 years when covered by soil or rockfill. This long-term durability reduces the need for frequent and costly rehabilitation cycles, lowering the overall lifecycle cost of the asset and ensuring sustained performance over decades.

3.0 Applications in Himalayan Hydropower Infrastructure

The technical merits of the CARPI geomembrane system translate into profound practical advantages when applied to the specific context of Himalayan hydropower development. Its features create a virtuous cycle of risk reduction, addressing not only the technical challenges of waterproofing but also the critical construction, operational, and economic risks that can determine a project's success or failure. This multi-faceted risk mitigation makes projects more resilient, bankable, and insurable—a crucial consideration for the capital-intensive investments required by the India-Bhutan partnership.

3.1 Versatility in Rehabilitation and New Construction

The technology's adaptability makes it equally suitable for upgrading existing assets and for incorporation into new builds, providing a standardized solution across a project portfolio.

- **Rehabilitation:** For the many aging dams and tunnel in the region facing deterioration of their original concrete facings, the geomembrane system offers a permanent, modern solution. It can be installed directly over the existing surface, often with minimal preparation, avoiding the need for costly and time-consuming demolition. This retrofitting approach not only restores watertightness but also protects the underlying structure from further degradation, significantly extending the asset's service life.
- **New Construction:** In new projects, specifying a geomembrane lining from the outset provides a more resilient and sustainable primary waterproofing layer compared to conventional designs. As demonstrated in the Pinnapuram PSP project, choosing a geomembrane over a traditional asphalt facing can be driven by superior technical performance (e.g., accommodating settlement risk) and significant

environmental benefits, positioning it as the modern best practice for new hydraulic structures.

3.2 Addressing Himalayan Specifics: Rapid Installation, Seismic Resilience, and Underwater Capabilities

The system's unique features directly counter the most severe challenges posed by the Himalayan environment.

- Rapid Installation
- Seismic Resilience
- Underwater Installation

4.0 Validated Performance: Case Studies from Various Indian and Global Hydropower Projects

The theoretical and technical advantages of the CARPI PVC geomembrane system are substantiated by a long and successful track record of performance in real-world applications across India. The following six case studies of India demonstrate the technology's platform-like versatility, proving its efficacy across the entire spectrum of hydropower assets—from high-head dams and low-head canals to high-pressure tunnels and new pumped storage reservoirs. This diversity validates the system not as a niche solution for a single problem, but as a comprehensive and robust waterproofing platform. This versatility offers a strategic advantage for large-scale development programs, enabling standardization of design, training, and maintenance protocols, which leads to significant long-term efficiencies and cost savings.

4.1 High-Head Dam Rehabilitation: Kadamparai, Servalar, and Upper Bhavani

These three projects highlight the system's effectiveness in rehabilitating large, critical dam structures.

- **Kadamparai Dam:** The Kadamparai project stands as a landmark for geomembrane application in India. Faced with a staggering leakage rate of 38,000 liters per minute, the dam's operational efficiency and long-term stability were severely compromised. The installation of the CARPI PVC geomembrane system over an area of 17,000 square meters resulted in an immediate and dramatic reduction in leakage to just 100 liters per minute. This solution was implemented in a remarkably short period of 16 weeks, six weeks ahead of schedule. More importantly, this high level of performance has been sustained for over two decades without any maintenance, a testament to the system's durability. The project's success was recognized with the prestigious India Power Award in 2008.

Before Geomembrane

After Geomembrane

- **Servalar Dam:** The rehabilitation of the Servalar masonry dam was a critical intervention for a structure that had deteriorated significantly since its construction in 1986. With leakages exceeding 7,200 l/min, conventional repair methods had repeatedly failed. The project's significance is underscored by its status as the first dam rehabilitation in India to be funded under the World Bank's Dam Rehabilitation and Improvement Project (DRIP), signaling the acceptance of geomembrane technology by major international financing institutions for critical dam safety upgrades. The installation reduced leakage to less than 20 l/min, conserving water for power generation, irrigation, and public supply, and earning the Dam Safety Excellence Award in 2018.

Before Geomembrane (wet downstream)

After Geomembrane (dry downstream)

- **Upper Bhavani Dam:** This case illustrates the system's capability in very high-head applications. The 80-meter head dam is a crucial component of a cascaded power system generating nearly 600 MW. A specialized, thicker 3 mm geomembrane (SIBELON CNT 4400) bonded to a 500 g/m² geotextile was selected to withstand the immense hydrostatic pressure. The project also demonstrated logistical flexibility, with installation work programmed in phases during the lean seasons to align with the operational constraints of the power utility, TANGEDCO, thereby minimizing disruption to power generation.

4.2 Power Channel and Pressure Tunnel Lining: Tanakpur and Bajoli Holi

These cases demonstrate the technology's robustness in dynamic, high-flow environments.

- **Tanakpur Power Channel:** The 6.9 km long Tanakpur canal suffered from both significant water loss due to seepage and geotechnical instability, with frequent collapses of its side-slopes. These issues led to power generation losses and problems for nearby communities. In 2007-08, a 400m vulnerable section was lined with the CARPI system. The solution not only eliminated seepage but also stabilized the slopes, allowing the channel to operate at its full capacity. The lining has remained fully functional for over 18 years without any maintenance, proving its long-term reliability in exposed, water conveyance applications.

- **Bajoli Holi HEP:** This project located in the Himalayan region posed a serious challenge to the owner GMR Energy. The Head Race Tunnel (HRT) lined with concrete had several cracks and due to hydrostatic pressure during operation the cracks widened and extreme leakage was reported causing public outrage.. The project was an extreme challenge considering the hydraulic head of 118 m and flow velocity of 3.3 m/sec/. Post-commissioning in 2021, a major leak of around 12,000 l/min emerged, and conventional grouting methods failed to resolve it. Carpi's geomembrane solution was chosen by the client and was designed to withstand a hydraulic head of 118 meters and a high flow velocity of 3.3 m/s. A robust system featuring the SIBELON® CNT 4400 composite and watertight stainless-steel perimeter seals was installed. The result was a fully watertight tunnel, demonstrating the system's capacity to perform under the most demanding dynamic conditions found in pressure tunnels.

4.3 New Construction in Pumped Storage: The Pinnapuram PSP Project

The Pinnapuram project is a forward-looking case study that showcases the benefits of incorporating geomembrane technology at the design stage of a major new infrastructure project.

- **Pinnapuram PSP:** For one of India's largest pumped storage projects, the original design specified conventional asphalt facings for the vast reservoir embankments. However, a detailed engineering review identified significant risks associated with differential settlement, which could compromise an asphalt lining, and noted the long curing times required. The CARPI geomembrane system was selected as the superior alternative. Its flexibility inherently mitigates the risk of settlement, while its rapid installation allowed for faster completion of the waterproofing works, contributing to earlier project commissioning milestones. Critically, this choice delivered a major sustainability benefit: the substitution of geomembrane for asphalt avoided an estimated 8,000 tonnes of CO_2 emissions. This project serves as a powerful example of how selecting advanced materials in new construction can simultaneously enhance technical performance, accelerate project delivery, and achieve significant environmental goals.

Lower Reservoir

Tail Race and Tail Pool

5.0 International Validation Across Diverse Applications

The principles and performance validated in the Indian context are mirrored by the technology's extensive and successful implementation in a diverse range of international projects. The system's scalability, adaptability, and robustness have made it the solution of choice for complex new-build and rehabilitation challenges worldwide. The following international case studies, categorized by application, demonstrate the platform's versatility in solving critical infrastructure problems under varied geological and operational conditions.

5.1 Hydropower Canals (New Zealand & Canada)

- **Tekapo Canal (New Zealand):** This 26 km long canal, part of a major hydropower scheme, was suffering from significant water loss due to leakage, creating large swamps and raising stability concerns. A massive-scale rehabilitation project involved lining the canal with 358,000 m² of SIBELON® CNT 3750. The installation was strategically phased using cofferdams, allowing the canal to remain operational and minimizing power generation losses.

Left: Deployment of the SIBELON® geocomposite rolls. Temporary ballasting with sand bags was necessary to prevent uplifting of the SIBELON® geocomposite sheets caused by wind. **Right:** seaming of SIBELON® geocomposite sheets.

Kootenay Canal (Canada): This asset required a phased rehabilitation, beginning with an emergency repair in 2009 to address failing joints. Following the success of this initial intervention, the solution was extended in 2014 to cover larger areas of the forebay to mitigate widespread leakage risks. The exposed SIBELON® geomembrane system, secured with tensioning profiles, substantially reduced leakage and restored asset integrity.


Installation of support layer (SIBELON® geocomposite) at vertical joints, and installation of drainage geonet (black material). The first component of the tensioning profiles assembly is under placement in horizontal lines, at regular spacing.


The SIBELON® geocomposite is placed over the geonet and permanently anchored with the tensioning profiles. Installation, fastening and waterproofing of tensioning profiles with SIBELON® geomembrane cover strips

5.2 Concrete Dams (France & USA)

- **Chambon Dam (France):** This 136-meter-high concrete gravity dam was suffering from Alkali-Aggregate Reaction (AAR), a condition that causes concrete to swell and crack. The CARPI system (SIBELON CNT 3450) was installed on the upper 40 meters to ensure watertightness and provide stability during a series of critical slot-cutting operations designed to relieve stress in the dam. The system performed successfully for over 20 years and was re-specified and re-installed in 2013-2014 after new structural reinforcement works were completed.

Chambon (France,1991 and 2012/2014), 136 m height

- **Lost Creek Dam (USA):** This 36-meter-high concrete arch dam had severe concrete deterioration and spalling due to frequent freeze-thaw cycles. The CARPI SIBELON CNT 3750 system was selected for a full-face rehabilitation. The project pioneered a patented underwater installation method, allowing specialized divers to install the liner in the dam's lowest sections and achieving a remarkable leakage reduction to less than 0.013 l/s.

Lost Creek (USA, 1997), 36 m of height. UW installation.

5.3 Rockfill & Earthfill Dams (USA & Czech Republic)

Upper Blue Dam (USA): This 22-meter-high rockfill dam is located at a high altitude of 3,580 meters. Its original asphalt concrete facing had completely deteriorated under the harsh environmental conditions. The SIBELON® CNT 4400 geocomposite was installed over the damaged face using tensioning profiles, demonstrating the system's ability to be rapidly deployed in a single short work season in remote, high-altitude environments.

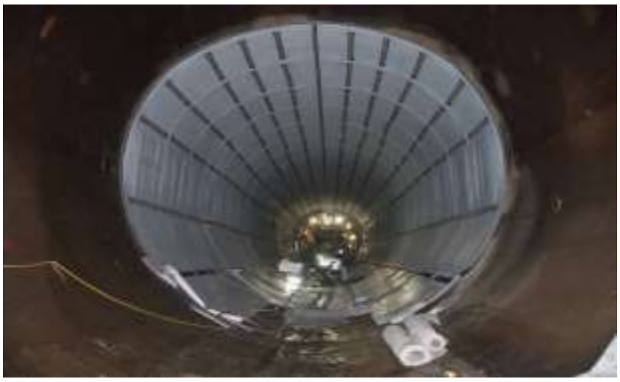
At left, the surface of the asphalt concrete facing as it appeared from an inspection in 2005,

The system of tensioning profiles (left) during and (right) after completion of the installation

5.4 Tunnels & Shafts (USA, Colombia & Bolivia)

- **Helms Pressure Tunnel (USA):** This 8-meter-diameter tunnel operated under extreme pressure (over 8.27 MPa). Increasing leakage from a highly sheared rock zone posed a significant safety risk. A heavy-duty SIBELON® CNT 5050 geocomposite was anchored with 16 longitudinal lines of solid stainless-steel profiles after the system design was validated by full-scale testing at 8.52 MPa.


At the beginning and end of the waterproofed stretch, the SIBELON® geocomposite is fastened by a watertight perimeter seal that avoids water infiltration behind the liner

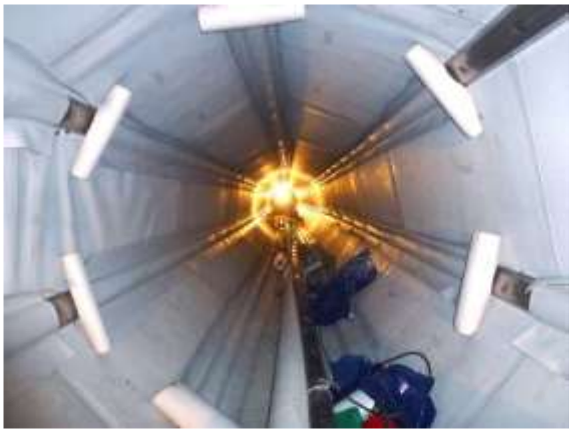



Installation of the geomembrane system started in September 2017, and the Helms Pumped storage scheme was placed back into operation in early November 2017.

- **Tunjita Surge Tank, Shaft And Tunnel (Colombia):** This 320-meter-long concrete-lined water conduction system, comprising a surge tank, shaft, and tunnels, was found to have widespread cracking and detachment of its concrete lining. A CARPI exposed geomembrane system was installed along the entire 320-meter length to restore integrity. The solution involved installing a high-resistance TENAX geogrid composite to support the liner over the cracked areas , followed by the SIBELON® CNT 4400 (3.0 mm) waterproofing geocomposite. The system was secured using 8 to 10 parallel longitudinal lines of stainless-steel profiles.


- **Chojilla Pressure Tunnel (Bolivia):** This 200-meter-long reinforced concrete tunnel suffered from radial cracks around its full circumference, causing significant leakage that threatened slope stability by raising the local groundwater table. After a 2010 repair attempt failed , a multi-layer CARPI system was installed to handle high external (12 bar) and internal (6.7 bar) pressures. The system included a geogrid layer to bridge the cracks , a geonet drainage layer to manage the high water table , and the SIBELON® CNT 4400 geocomposite. This was anchored using patented tensioning profiles and sealed directly to both the concrete tunnel and the steel penstock.



5.5 Joint & Crack Waterproofing (Colombia, Canada, Greece & USA)

- **Porce II Dam (Colombia):** On this 118-meter-high RCC dam, the original design for an external rubber seal on the contraction joints was found to be unbuildable. The CARPI external waterstop system was selected as the alternative. This multi-layer system, featuring a SIBELON® CNT 5050 liner and a steel-plate support structure, was laboratory-tested to successfully withstand a 2.4 MPa head and a 35 mm joint opening.

- **Daniel Johnson Dam (Canada):** This 214-meter-high multiple arch dam, the world's largest, was experiencing leakage through joints on the downstream buttresses. After previous injection attempts failed, the CARPI external waterstop (SIBELON® CNT 3750) was applied to the *downstream* face, restoring watertightness and protecting the structure's appearance from seepage stains.

- **Platanovryssi RCC Dam (Greece):** After the first impounding, a 20-meter-long crack appeared on the upstream face of this 95-meter-high dam. Draining the reservoir for a dry repair was operationally impossible, as it would have halted the critical Thissavros pumped storage scheme. The CARPI external waterstop system was successfully installed underwater by specialized divers to permanently seal the crack, adapting the same technology previously used on the dam's main contraction joints.

At left, the crack. On its left can be seen the holes for the anchors of the perimeter seal. At right, the template used to lower the sacrifice and waterproofing layers down to their placement position.

Below at left the sacrifice layers have been placed at their final position. At right, rubber gasket is prepared at the perimeter seal.

At left, completed seal. At right, detail of the bottom perimeter seal at the plinth.

At left, the crack spilling water. At right, a few hours after underwater installation of the external waterstop, the crack had completely dried up

- **Occoquan Powerhouse (USA):** Water was seeping through cracks and horizontal construction joints on the powerhouse, an appurtenant structure, threatening to cause concrete deterioration, rebar corrosion, and dangerous uplift pressure. As draining the reservoir was not an option for the water supply facility, a SIBELON® CNT 3750 geocomposite was installed over a drainage geonet by divers. This complex underwater installation covered the entire structure, including the successful waterproofing of 218 existing anchors for trash racks.

5.6 Underwater Applications (Italy & Venezuela)

- **Olai Dam (Italy):** This 54.6-meter-high concrete gravity dam suffered from severe leakage (over 9 l/s) from its vertical dilatation joints. As the dam could not be dewatered for water supply reasons, the CARPI external waterstop was installed by specialized divers at depths up to 49 meters. The installation, using a double-layer SIBELON® CNT 3750 system, successfully reduced the critical leakage to negligible "drops."

- **Turimiquire Dam (Venezuela):** This 113-meter-high CFRD had a long history of multiple failed repairs using various methods. A permanent solution using the SIBELON® CNT 4600 system was installed while the dam remained in full operation. This challenging project required divers to work at extreme depths, at times exceeding 60 meters, to install the geomembrane over a specially placed geogrid support layer.

6.0 A Low-Carbon Pathway: Sustainability and Climate Resilience

The adoption of advanced PVC geomembrane technology is not only a decision for enhanced technical performance and resilience but also a strategic choice for sustainability. In an era where infrastructure development is increasingly scrutinized for its environmental impact, this technology offers a compelling "triple bottom line" of economic, social, and environmental benefits. These quantifiable advantages in carbon reduction and climate adaptation align perfectly with the goals of international development banks and climate funds, potentially helping India and Bhutan unlock preferential "green financing" for their hydropower projects. The technology thus becomes a catalyst for securing the capital needed to build the "Green Future" central to the conference's theme.

6.1 Quantifying the Reduction in Embodied Carbon Footprint

The most significant direct environmental benefit of the geomembrane system is its substantially lower carbon footprint compared to conventional methods.

Lifecycle Emissions Reduction: Comprehensive life-cycle assessments (LCAs) have demonstrated that PVC-P geomembrane systems can reduce cradle-to-grave embodied carbon emissions by up to 70% when compared to traditional cement or asphalt facings. This figure accounts for the entire lifecycle, from raw material extraction and production through transportation, installation, and end-of-life disposal.

Sources of Carbon Savings: These savings are derived from multiple factors. While the production of PVC is energy-intensive, it is significantly less carbon-intensive than the production of cement. Furthermore, the lightweight nature of the geomembrane rolls drastically reduces transportation-related emissions compared to hauling massive quantities of aggregate, cement, or asphalt. The most substantial savings, however, often occur during the construction phase. Installation requires far less heavy machinery—such as concrete mixers, pavers, and rollers—leading to a dramatic reduction in onsite fuel consumption and associated emissions.

Project-Level Validation: The Pinnapuram PSP case study provides a concrete, project-level validation of these savings. The decision to use a geomembrane lining instead of the originally planned asphalt facing resulted in an estimated avoidance of 8,000 tonnes of CO_2 emissions, a tangible contribution to the project's sustainability profile.

6.2 Water Conservation: Enhancing Efficiency and Climate Adaptation

In the context of the Himalayas, where climate change is altering hydrological cycles and increasing the variability of river flows, water conservation is a critical component of climate adaptation.[13]

Eliminating Seepage: By providing a virtually impermeable barrier, the geomembrane system effectively eliminates the chronic seepage losses that plague many older dams and canals. This directly enhances water security. Every cubic meter of water saved from leakage is a cubic meter that remains available for its intended purpose: generating electricity, irrigating crops, or supplying municipal water systems.
Maximizing Storage Efficiency: As glacial melt patterns shift and rainfall becomes more inconsistent, the ability to efficiently store water during periods of surplus for use during dry spells becomes paramount. The geomembrane technology ensures that the water captured in reservoirs is not squandered through leakage, thereby maximizing the value and resilience of existing and future water storage infrastructure. This contribution to resource efficiency is a powerful, albeit indirect, climate adaptation benefit.

6.3 Lifecycle Advantages: Longevity, Low Maintenance, and Reduced Environmental Impact

The sustainability benefits of the geomembrane system extend throughout its operational life.

Longevity and Reduced Consumption: With a service life that can exceed 50 years in exposed conditions and 100 years when covered, the system's durability is a key sustainability feature. Fewer replacement and major rehabilitation cycles over the asset's lifetime translate directly into less consumption of raw materials, energy, and labor, and less waste generation.

Low-Impact Maintenance: The low O&M requirements further reduce the system's environmental footprint. The ability to perform localized repairs, often without dewatering the reservoir, minimizes the operational carbon footprint associated with mobilizing large crews and equipment. It also avoids the significant disruption to downstream river ecosystems that can be caused by emptying and refilling a large reservoir.

Circular Economy Potential: Looking to the future, the materials science of geosynthetics continues to evolve. Post-service recycling programs for PVC membranes are becoming increasingly feasible, offering a pathway to recover and repurpose the material for secondary, non-critical applications. This points toward a more circular economy model for infrastructure, further enhancing the long-term sustainability of the technology.

7.0 Conclusion: A Strategic Imperative for the India-Bhutan Partnership

The development of hydropower in the Himalayas is at a critical juncture. The immense potential for clean energy is counterbalanced by a formidable array of interconnected geotechnical, seismic, and climatic challenges that render conventional construction and waterproofing methods inadequate and, in many cases, unacceptably risky. The evidence presented in this paper demonstrates that advanced flexible polymeric geomembrane technology offers a direct, proven, and sustainable solution to these challenges.

7.1 Summary of Findings: A Direct Solution to Himalayan Challenges

The analysis has established that the CARPI PVC geomembrane system, through its unique and engineered combination of near-absolute impermeability, high flexibility, and long-term durability, directly and effectively mitigates the primary risks associated with Himalayan hydropower infrastructure. Its ability to accommodate the seismic and settlement stresses that cause rigid linings to fail is not a theoretical advantage but a core performance characteristic validated by over two decades of successful operation in demanding Indian projects. The case studies—from the dramatic leakage reduction at Kadamparai and Servalar to the high-pressure resilience at Bajoli Holi and the low-carbon construction at Pinnapuram—provide irrefutable evidence of the technology's versatility and reliability across the full spectrum of hydropower applications. The system is not a novel experiment; it is a mature, field-proven technology that has consistently delivered superior performance, enhanced safety, and significant environmental benefits.

7.2 Recommendations for Future Hydropower Development in the Region

The weight of the technical evidence and the long-term performance data is compelling enough to warrant a fundamental shift in engineering practice for the region. The conclusion of this analysis transcends a simple technical summary to become a firm policy and design recommendation.

It is strongly recommended that flexible polymeric geomembranes be adopted as the **primary, first-line waterproofing option** for all new and rehabilitation hydropower projects undertaken as part of the India-Bhutan partnership and across the wider Himalayan region.

Integrating this technology into national design standards and project specifications from the earliest stages of planning will be a transformative step. It will systematically de-risk investments by building in resilience to the region's inherent hazards. It will improve the financial viability of projects by reducing construction timelines and long-term maintenance liabilities. Most importantly, it will accelerate the achievement of the region's clean energy goals in a manner that is both environmentally responsible and sustainable for generations to come.

The adoption of this technology should not be viewed as a mere component choice, but as a strategic decision. It is an investment in asset longevity, operational security, and climate resilience. For the India-Bhutan partnership, embracing this new best practice is essential for building a truly sustainable and green energy future, ensuring that the immense promise of Himalayan hydropower can be realized safely, efficiently, and responsibly.

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INNOVATING TO REDUCE GLACIAL LAKE OUTBURST FLOOD RISK: HOW DATA IMPROVES WHOLE-OF-RIVER SYSTEM PREPAREDNESS

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ABSTRACT

In development and disaster risk settings, difficult decisions must be made to protect infrastructure, secure economies, and save lives. While other sectors – from telecommunications to transportation – iteratively improve through making use of abundant data, asset-level disaster risk management has been slow to adjust. This paper explores the issue of inconsistent use of risk data, the incompatibility of risk data sources, and the lack of data availability beyond specific infrastructure projects. It discusses how these challenges create a fragmented, often misleading view of risk that can burden government and business with significant costs. Through the lens of hydropower development, the paper explores how understanding risk consistently throughout a river basin leads to better decisions, improved social license, and safer operating conditions. The paper discusses how recent innovations enable data-driven, whole-of-river risk assessment that considers all aspects of risk, including hazard exposure, social vulnerability, critical infrastructure, cultural heritage, and the strength of emergency preparedness and response systems. Finally, the paper will address how such innovations can be used by a variety of government programs and infrastructure owners to optimize investment activity while improving resilience throughout the river basin.

1.0 INTRODUCTION

The increasing frequency and intensity of natural disasters, particularly in the context of climate change, have elevated the significance of disaster risk management (DRM) across various sectors. Among these challenges, glacial lake outburst floods (GLOFs) have emerged as a critical hazard in many mountainous regions, posing threats to lives, infrastructure, and economic stability. GLOFs occur when glacial lakes, which have formed due to melting glaciers, experience sudden and catastrophic outflows, often triggered by climate-related factors such as ice dam failures, landslides, or seismic activities (Haeberli & Beniston, 1998).

In development and disaster risk settings, decision-makers face difficult choices regarding resource allocation and risk mitigation strategies to protect vital infrastructure, secure economic activities, and ultimately save lives. Despite the availability of diverse data sources, asset-, community-, and landscape-level disaster risk management has been slow to adopt a data-driven approach compared to other sectors, such as telecommunications or transportation, which have effectively utilized data for iterative improvements (IPCC, 2012).

This paper explores the challenges associated with the inconsistent use of risk data, the incompatibility of various data sources, and the limited availability of data beyond specific infrastructure projects. These issues often lead to a fragmented and potentially misleading understanding of risk that can impose significant costs on governments, businesses, and communities. By examining the case of hydropower development, this paper illustrates how a comprehensive understanding of risk within a river basin context can facilitate better decision-making, enhance social license, and ensure safer operating conditions. The paper discusses recent innovations that enable data-driven, wholeof-river risk assessments that consider various risk factors, including hazard exposure, social vulnerability, critical infrastructure, cultural heritage, and the robustness of emergency preparedness and response systems. Finally, the paper addresses how these innovations can be leveraged by government programs and infrastructure owners to optimize investment activities while fostering resilience throughout the river basin.

2.0 DISCUSSION

2.1 The nature of GLOF risk and data challenges

Glacial lake outburst floods (GLOFs) represent a growing threat in deglaciating environments worldwide. For instance, a multi-decadal inventory of GLOF events in High Mountain Asia identified 697 events, causing 906 deaths, and revealed trends hitherto under-recognised (Shrestha et al., 2023). Harrison et al. (2018) shows that many proglacial lakes are expanding as glaciers retreat, increasing the potential volume of water release and thus the flood hazard. Cook et al. (2016) found that in the Bolivian Andes proglacial lakes were developing rapidly, prompting identification of 25 lakes that posed potential GLOF threat to downstream communities. More recently, studies using remote sensing and hydrodynamic modelling show the complexity of the hazard chain: glacier retreat, lake formation, moraine stability, avalanche or mass-movement triggers, dam breach dynamics, downstream inundation. For example, Yang et al. (2023) used multi-sensor remote sensing (SAR, DEM, optical) and HEC-RAS hydrodynamic modelling to better model a moraine-dammed lake and its outburst flood scenario. These studies underscore that hazard exposure is dynamic, uncertain, and often not captured fully in conventional risk assessments.

Yet despite this growing body of hazard-focused research, data availability and use at the level of infrastructure investment and operations remains fragmented. Several specific challenges stand out:

Inconsistent use of risk data. Many infrastructure development programmes embed site-specific risk assessments (for example a hydropower dam may commission a GLOF hazard review for its immediate upstream glacial lake). However, these assessments often use different modelling approaches, different inputs, different units of analysis, and seldom feed into a broader basin-wide risk picture. The methodology applied to one lake may differ from that applied to another, reducing comparability. For example, a multi-criteria decision analysis (MCDA) approach applied to potentially dangerous glacial lakes in Bolivia used a consistent desk-based method yet acknowledged that further work is required to determine relative weights of risk criteria and thresholds (Kougloulos et al, 2018). When decision-makers rely on such heterogeneous assessments, it can lead to misleading prioritisation.

Incompatibility of data sources. The data that inform GLOF risk-assessments come from disparate domains: glacier mass balance, lake bathymetry, moraine geometry, remote sensing time-series, downstream river catchment topology, infrastructure exposure, social vulnerability. These datasets often reside in different formats, with different spatial and temporal resolution, generated by different organisations with different standards. For example, Shrestha et al. (2023) emphasised the need for integrated, version-controlled GLOF event databases to enable comparative risk assessment. Without interoperability, efforts to integrate hazard, exposure, and vulnerability across a river basin are hampered.

Lack of data availability beyond specific infrastructure projects. Many hydropower, irrigation or flood-control developments commission risk modelling for the segment of river, lake or reservoir they are responsible for, but seldom for the entire river-basin cascade of risks or for downstream exposure beyond the footprint of their own infrastructure. The result is that the broader reservoir of risk — for example the cumulative effect of multiple glacial lakes, potential cascade of lake-breach flood into river-basin systems, downstream towns, transport corridors, other infrastructure — remains opaque. For instance, Chen et al. (2025) highlighted limited historical data on GLOF impacts (casualties, spatial redistribution, infrastructure damage) and emphasised that while hydrodynamic models exist, they are rarely applied consistently across all lakes in a region.

The cost of fragmentation. This fragmented risk-data landscape leads to several adverse consequences. First, infrastructure investors, governments and insurers may hold distorted views of risk: either over-concentrating on one lake or one site while ignoring systemic downstream risk, or conversely under-estimating cumulative risk cascades. Second, fragmented data and incompatible methodologies make it more difficult to compare, prioritise, and optimise investment decisions across the river basin. Third, lack of basin-wide data inhibits building social licence: downstream communities or governments may perceive the developer or operator as only managing local risk, while remaining blind to broader system risk. Fourth, the absence of consistent data undermines emergency preparedness and response planning: if each infrastructure owner treats only its immediate upstream lake and dam, but not the downstream network of hydropower, irrigation, municipal water supply, roads and bridges, the operator may under-estimate the potential consequences of a GLOF event cascading into its river system. In short, fragmented risk data impose longerterm latent costs in resilience, which may manifest as damaged infrastructure, stranded assets, reputational harm and social loss.

2.2 The potential of data-driven, whole-of-river system assessment

Faced with these challenges, a shift in thinking is required: from isolated project-centric risk assessments toward a whole-of-river-basin, system-based risk-management approach that places hazard, exposure, vulnerability, critical infrastructure, social systems, cultural heritage, and emergency preparedness under a unified data-driven framework. Several strands of research support the viability of this shift:

From asset-level to system-level resilience. The infrastructure resilience literature emphasises the interdependence of assets, the need for network-wide assessments, and the role of system-level metrics. For example, Balakrishnan & Cassottana (2022) developed the open-source simulation platform “InfraRisk”, which enabled asset-level resilience analysis in interdependent infrastructure networks. Although focused on water, power and roads, the conceptual parallels for a river-basin with hydropower, dams, lakes and downstream infrastructure are evident: the need to simulate hazard propagation, component failures, interdependencies, and recovery sequences. Data-driven decisionmaking in this context is critical.

Data-driven approaches in disaster risk management. The International Society for Integrated Disaster Risk Management (IDRiM) special issue on "Data-Driven Approaches to Integrated Disaster Risk Management" provides a useful orientation. The volume emphasises that data science and analytics can inform prevention, preparedness, response and recovery phases of disaster risk management, but highlights that heterogeneous sources, difficulties in data curation and sharing remain major obstacles. Similarly, in the built-environment domain, So, E. (2023) argued that reducing disaster risk requires standardisation and collecting exposure-vulnerability data — "what we measure matters." Recent sector-leading, data-driven risk visualization applications such as Resilience Engine enable millions of validated, geo-referenced data points to be consolidated into a detailed picture of risk across a land base with high detail. These themes translate directly to GLOF risk and river-basin infrastructure: effective decision-making requires coherent, interoperable data across hazards and the communities they threaten.

Systemic risk and data in other domains. The finance sector provides useful analogies. Flood et al. (2011) pointed out the importance of data integration, metadata management, cross-institution comparability, and elimination of silos if systemic risk is to be managed effectively. The technical and organisational challenges of risk-data integration in the finance domain mirror those in infrastructure risk and GLOF risk management: multiple heterogeneous silos, variable standards, high uncertainty. These insights reinforce that for river-basin infrastructure and GLOF risk we should adopt similar architecture and data-governance mindsets.

Innovations in data and monitoring for GLOFs. In recent years, GLOF-specific advances have begun to emerge that enable more system-wide data use. For example, a recent dataset "GLOFNet" generated by Fatima, Z, et al (2025) integrates multispectral imagery, glacier-velocity products, land surface temperature trends, and offers a structured foundation for monitoring and predictive deep-learning approaches in the Shisper Glacier region. Recently, Mir, R. A., et al (2025) developed a strategic framework for GLOF risk reduction in the Himalayas, emphasising five steps: scientific evaluation, real-time monitoring, structural mitigation, stakeholder participation and adaptive governance. These advances point to the possibility of moving from isolated lake-level assessment to basin-wide risk monitoring and decision-support.

2.3 Hydropower development as a lens for whole-of-river preparedness

Hydropower development is a domain in which whole-of-river system preparedness is both highly relevant and increasingly challenging. Hydropower assets – dams, reservoirs, diversion structures, penstocks, downstream powerhouse plants, grid connections, access roads, sediment management regimes – are embedded in the river-basin hydrology, sediment transport, and environmental and social systems. In glaciated mountain regions, the hydropower cascade may link with multiple glacial lakes, moraine-dams, friable slopes, avalanche risk, river-valley settlements, critical downstream infrastructure (roads, bridges, towns) and cultural heritage. A GLOF triggered upstream may cascade through one or more hydropower reservoirs or diversion structures, impacting not only the hydropower asset but also downstream infrastructure, communities and ecosystems.

From a hydropower-owner or investor perspective, using a basin-wide data-driven risk assessment offers numerous benefits:

Improved decision-making for site selection, design parameters and reservoir/dam safety. If the upstream glacial-lake threat, moraine-dam condition, glacier retreat dynamics and downstream exposure of communities and infrastructure are understood, developers can target mitigation measures (lake-lowering, dam reinforcement, early warning systems) more precisely. For example, Cuellar & McKinney (2017) applied a decision-making methodology to Imja Lake in Nepal, evaluating options of lake lowering by 3, 10 or 20 m. The methodology considered flooding scenarios, consequences and cost–benefit under uncertainty. Even if the immediate economic benefit (based on agricultural land value) suggested limited attractiveness, the approach exposed how varying input assumptions shift outcomes. That illustrates how better data (on exposure, downstream value, social vulnerability) could change decisions.

Enhanced social licence and stakeholder confidence. Hydropower projects often face community concerns about safety, downstream risk, environmental and cultural heritage (Zhao et al., 2018). A transparent, basin-wide data-driven risk assessment can support engagement with downstream communities, regulators and insurers by showing that risk has been evaluated in a system-wide, integrated way, rather than simply at the local dam site.

Optimised investment and maintenance decisions. If a hydropower owner understands the whole-of-river risk chain (glacial lake risk, dam breach path, downstream infrastructure exposure, evacuation capacity, community vulnerability, emergency preparedness), then capital investment (e.g., in early warning, physical mitigation, lake-lowering, improving downstream evacuation routes) can be prioritised effectively. Rather than each asset owner acting in isolation, the cumulative benefit of system-wide resilience investment can be optimised.

Safer operating conditions and business continuity. For a hydropower plant operating in a glaciated basin, a GLOF event can create extreme discharge waves, debris flows, inundation of intakes or tunnels, damage to infrastructure, and risk to lives. Having basin-wide monitoring of upstream lake volumes, glacier velocity, slope deformation, and downstream river response enables real-time situational awareness and triggers for operational decision-making (e.g., shutting down diversion intake, isolating penstocks, community evacuation). The novelty lies in integrating upstream glacier–lake–moraine monitoring with downstream infrastructure and social systems.

In effect, hydropower developers and operators can be a catalyst for shifting from project-by-project risk mitigation to whole-of-river system preparedness. The domain of GLOF risk is especially suited to this shift because the hazard chain often crosses site-specific asset boundaries and because the downstream consequences can be cross-sectoral (hydropower, municipal water, irrigation, roads, bridges, local towns). Thus, a data-driven, basin-wide risk framework and data-set is not only desirable but increasingly necessary.

3.0 FINDINGS

From the literature review and analysis, several key findings emerge regarding how data improves whole-of-river system preparedness for GLOF risk, and the systemic benefits of shifting to integrated, data-driven approaches.

Data-integration is foundational for whole-system risk visibility. The fragmentation of hazard, exposure and vulnerability data means that many infrastructure owners lack a full picture of systemic risk across the river basin. As Shrestha et al. (2023) emphasised, the absence of integrated, version-controlled databases limits comparative risk assessment and downstream exposure modelling. By contrast, when data from glacier retreat, lake bathymetry, moraine condition, remote sensing, infrastructure exposure and community vulnerability are brought together, the risk picture becomes more complete, enabling better prioritisation.

Standardisation and interoperability of data matter. One of the impediments to basin-wide assessment is incompatible data formats, variable methodologies, and lack of standard metadata. This echoes findings from systemic risk literature in finance: Cook et al. (2011) noted “multiple heterogeneous silos, the data quality gap, lack of standards” as barriers to systemic risk monitoring. Schröter et al. (2025) similarly emphasises the need for data-sharing standards and interoperable platforms to allow cross-domain integration. For river-basin GLOF risk, data governance (who owns the data, who updates it, how it is shared) becomes critical.

A shift from asset-specific to system-wide exposure and vulnerability modelling enables better investment decisions. Historically, hydropower and other infrastructure owners have commissioned lake-level or dam-level risk assessments, e.g., Cuellar & McKinney 2017 on Imja Lake. These are necessary but insufficient. By modelling the downstream flood wave, cascading impacts, exposure of infrastructure, communities, evacuation routes and response capacity, decision-makers can prioritise mitigation not only upstream physical measures (lake lowering, dam reinforcement) but also downstream preparedness (early warning systems, evacuation planning, road and bridge resilience). For example, Rawat, M. et al. (2023) describe a study of a lake in the Satluj basin used multi-temporal Landsat imagery and MCDA to compute a susceptibility index and emphasised the need for regular monitoring and risk-reduction plans.

Data-driven monitoring and real-time/near-real-time systems enhance preparedness and response. The emergence of datasets like GLOFNet (Fatima et al., 2025) integrate multi-sensor glacier, lake and environment variables to support predictive modelling. In addition, remote sensing and hydrodynamic modelling studies, e.g. Yang et al., (2023), show the value of monitoring dam deformation, avalanche triggers, lake volume changes over time. For hydropower operations, this means early warnings, trigger thresholds, and integration with operational decision frameworks (e.g., reducing diversion flows, evacuating downstream, accessing emergency shut-down). The risk lifecycle can thus be better managed: prevention, preparedness, response and recovery.

Social, cultural and institutional layers must be integrated into the data framework. Effective whole-of-river system preparedness is not purely technical. Social vulnerability, community exposure, emergency preparedness, cultural heritage, downstream critical infrastructure are all part of the risk system. The risk data must therefore include socioeconomic indicators, community capacity, emergency services readiness, and cultural assets. The DRM literature highlights that strongly engineered hazard analysis alone is insufficient: vulnerability and governance matter. For example, So, E. (2023) argues for collecting vulnerability-relevant exposure data. In GLOF basins, knowledge of downstream community capacity, early warning system effectiveness, evacuation route redundancy, cultural heritage values, and critical infrastructure (roads, bridges, power lines) is indispensable.

Whole-of-river system preparedness pays off in avoided costs, enhanced resilience and improved stakeholder engagement. By adopting basin-wide, data-driven risk frameworks, infrastructure investors, governments and communities can optimise risk mitigation investments, reduce stranded-asset risk and improve preparedness. This improves social licence: communities see that risks beyond the immediate dam site are being addressed. Furthermore, by aligning downstream infrastructure, evacuation planning, community vulnerability reduction and upstream hazard mitigation, the entire system becomes more resilient. Indeed, in the infrastructure asset-risk domain, Chang & Hossain (2024) recent research shows how system-of-systems and data-driven methods can prioritise investment and track key performance indicators.

Governance, data ownership and institutional coordination remain major challenges. In many basins, multiple agencies – hydropower developers, glaciologists, disaster-management authorities, municipal operators, road and bridge authorities, community emergency services – hold pieces of the puzzle. Without governance frameworks that enable data sharing, model interoperability, responsibility for triggers and thresholds, and coordinated downstream preparedness, the data-driven system will not fully deliver. Schröter et al. (2025) highlights that governance of data, metadata, access rights, interoperability, and cross-domain linkage is as important as the technical modelling and emphasises the value of federated data infrastructure and open-source platforms that link multiple institutions and provide common data baselines.

4.0 CONCLUSION

Reducing the risk of glacial lake outburst floods (GLOFs) in glaciated basins demands innovation in data use and system-scale thinking. The traditional project-centric approach — where infrastructure owners commission lakespecific or dam-specific risk assessments in isolation — is no longer sufficient. The complexity of GLOF hazards, their dynamic drivers (glacier retreat, lake expansion, mass movements), and the downstream cascades through river basins means that better data, integrated across hazard, exposure, vulnerability and response, is essential.

For hydropower developers, governments and infrastructure investors operating in glaciated basins, key steps are: develop asset-inventory and exposure databases across the river basin; integrate remote sensing, glacier monitoring, lake bathymetry, moraine/dam condition, and downstream infrastructure and community vulnerability data; adopt interoperable data architectures and standards; deploy real-time or near-real-time monitoring and early-warning systems; link hazard-monitoring triggers to operational decision-making; engage downstream communities and agencies in basin-wide preparedness planning; and track key performance indicators for resilience, preparedness and system-wide risk reduction.

Such a data-driven, whole-of-river system approach offers multiple benefits: it supports more accurate and transparent assessment of risk, enables better prioritisation of investment in mitigation and preparedness, enhances social licence and stakeholder confidence, improves emergency preparedness and resilience, and ultimately contributes to safer operating conditions for infrastructure and communities alike.

Innovation in data use is not just a technical improvement; it is a transformation of how infrastructure, disaster risk management and basin-wide natural systems are governed, financed and planned. As climate change accelerates glacier retreat and expands glacial-lake hazard potential, the imperative to adopt whole-system, data-driven preparedness cannot be deferred. Infrastructure owners and government regulators who make this transition stand to safeguard investment, protect downstream lives and livelihoods, and build a more resilient river-basin future.

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BEST PRACTICES IN O&M OF HYDRO PLANTS

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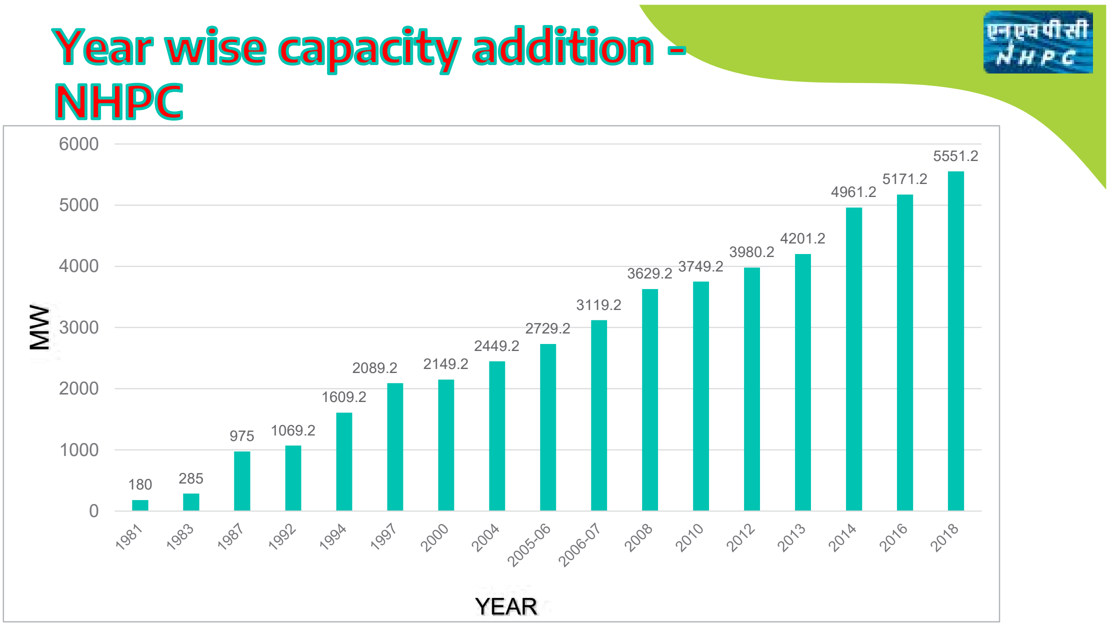
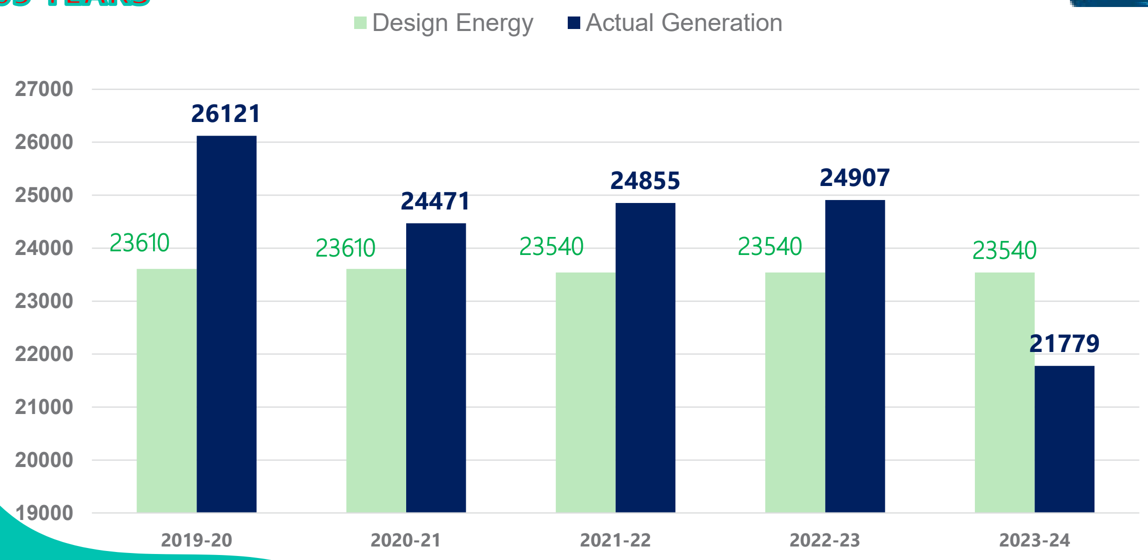
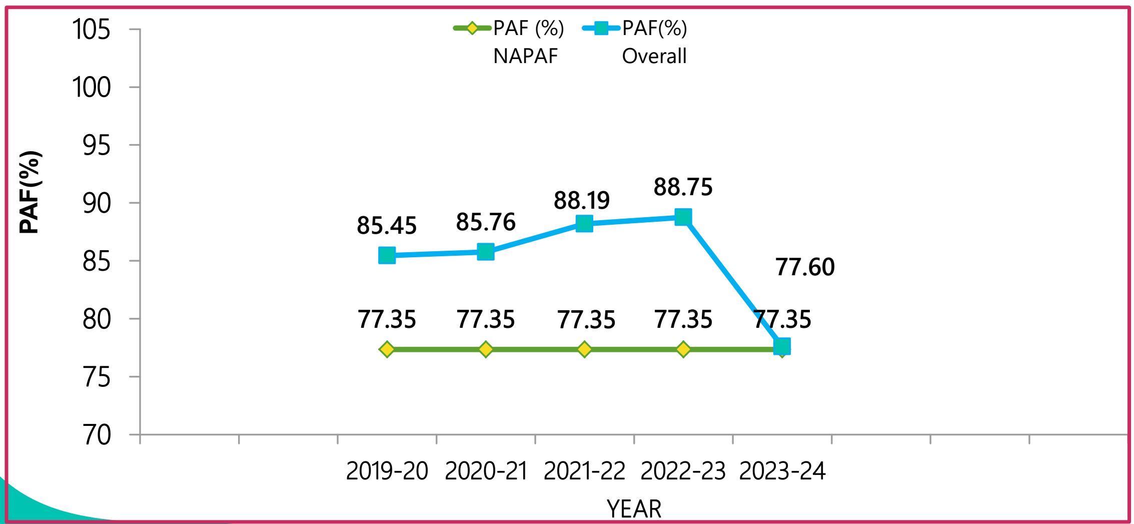
GM (O&M), NHPC Ltd.



GENERATION (MU) DURING LAST 05 YEARS



PLANT AVAILABILITY FACTOR (PAF%) DURING LAST 05 YEARS



O&M BEST PRACTICES –

Some of the foremost O&M practices includes the following:

- Operational Analysis of all Plant on daily basis.
- Tripping & Breakdown analysis.
- Silt/ Sediment Management
- Critical spares & Inventory Management
- Annual /Capital Maintenance
- Adoption of New Technologies
- Ancillary Services to Grid
- Training of Manpower

O&M BEST PRACTICES – OPERATIONS ANALYSIS

Operational Analysis of all Plant on daily basis

Operation of generating unit needs to be analyzed with reference to Inflow, Water Utilization, Reservoir Level, Spillage, Operational parameters, etc. and in compliance to IEGC.

MVAR Analysis: Injection/Drawal of MVAR as per the Generator Capability Curve and grid requirement by the generating unit/plant is to be analyzed.

AGC Implementation: The AGC command to various Power Stations is given by NLDC and response of the same is to be analysed.

Mock Black Start : Mock Black Start needs to be carried out in compliance to IEGC, mock black start exercise is to be done by capable Power Stations as per schedule and procedures given by RLDCs.

O&M BEST PRACTICES –

Outage Management - Tripping & Breakdown Analysis

- Tripping / Breakdown Analysis of each event is to be carried out.
- Identify the root cause of fault for early restoration.
- Corrective action, as required, are to be taken to avoid/minimise repetition of tripping/breakdown.

Compliance of Grid Code

- IEGC 2023 has been effective since 01.10.2023.
- Following compliances are to be done:-

Annual protection audit:

- As per clause 15 of IEGC 2023, all users shall have to conduct internal audit of their protection systems (220 kV and above) annually and any shortcomings identified shall be rectified and informed to their respective RPC.
- Further, all user shall also conduct third party protection audit once in 5 years.

Protection performance indices

- Users shall submit the following protection performance indices of previous month to their respective RPC and RLDC on monthly basis

  The Dependability Index (D)

  The Security Index (S)

  The Reliability Index (R)

Compliance of Grid Code

Event Reporting

- As per IEGC clause#37.2 (c), tripping report along with EL/DR are to be furnished within 24hrs of occurrence of the event.

Reactive Energy Compensation

- All generating stations connected to the grid shall generate or absorb reactive power within the capability limits.
- The regional entity pays for VAr drawal when voltage is below 97%
- The regional entity gets paid for VAr return when voltage is below 97%.
- The regional entity gets paid for VAr drawal when voltage is above103%.
- The regional entity pays for VAr return when voltage is above 103%.
- ***The charge for VArh shall be at the rate of 5 paise/kVArh.***

Compliance of Grid Code

The frequency response characteristic (FRC)

- The frequency response characteristic (FRC) calculation shall be carried out by each control area for any load or generation loss incident involving net change of more than 1000 MW of load or generation or a frequency change involving 0.1 Hz or more.
- All regional entity generating stations shall also assess the FRC for their respective stations and submit the same to respective RLDC within six (6) working days alongwith high resolution data (1 second or better resolution) of active power generation and frequency.

Compliance of Grid Code

Mock Black Start Exercise

- As per Indian Electricity Grid Code (IEGC) clause 34.3, The user shall carry out a mock trial run of the system restoration procedure for different subsystems including black-start of generating units.
- Hydro plants are capable of self-black-start, shall have to provide schedule for mock black exercise.
- ***Any entity extending black start support by way of injection of power shall be paid for actual injection @ 110 % of the normal rate of charges for deviation in accordance with DSM Regulations for the last block in which the grid was available***.

Monsoon Preparedness

Followings are required to be ensured before start of monsoon:

- Daily checking of DG sets for its proper working.
- Ensuring Diesel Stock.
- Operational healthiness of Drainage & Dewatering System, Dewatering/Flood Pumps
- Operational readiness of Spares.
- Healthiness of CCTV Cameras installed at strategic locations
- Healthiness of Communication system between Power House , Dam & Demanded other strategic locations.
- Readiness of G&D site/Sensors/AWS for early forecasting of high inflow/flood
- Availability of suitable manpower at Dam/Power House to meet any exigency.
- Operational Healthiness of Gasoline for emergency operation of gates.

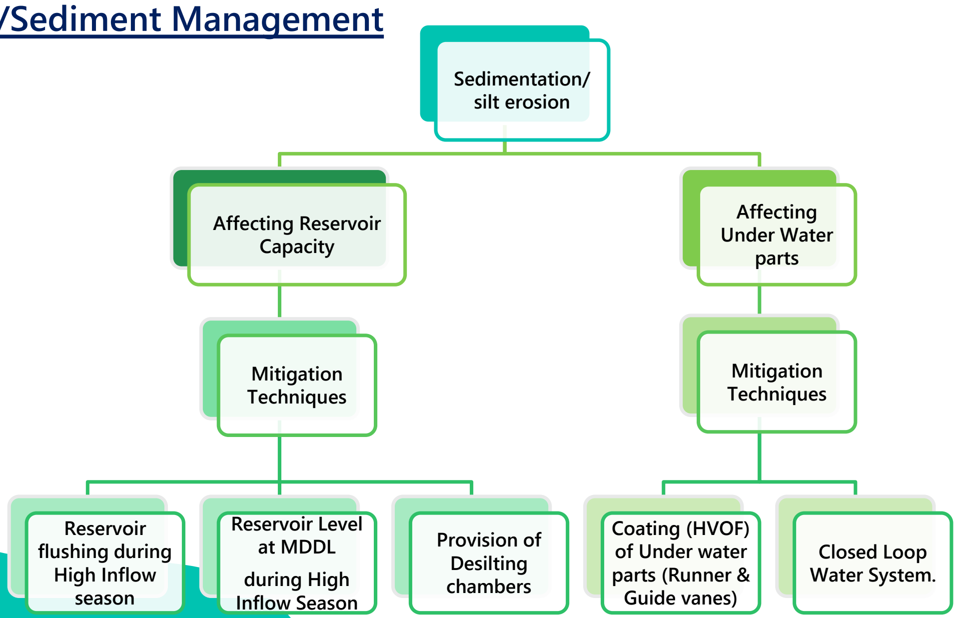
O&M BEST PRACTICES –

Silt /Sediment Management

- Himalayan rivers carry huge sediment during monsoon season.
- More than 80% of average annual sediment comes during the monsoon season
- Efficient sediment management system is needed in monsoon season to protect the economic and useful life of the reservoirs.
- There is correlation between discharge and silt and same needs to be identified and in case of alarming values , the frequency of silt measurement through fast method needs to be increased .

O&M BEST PRACTICES –

Silt /Sediment Management



O&M BEST PRACTICES –

Silt Erosion in under water parts

Problem: Hydro abrasive erosion is a common damaging mechanism in many plants of NHPC **(e.g Salal Dulhasti, Chamera-II, Dhauliganga, Teesta-V, Baira Siul, etc.)** situated on the rivers carrying excessive hard particles such as quartz and mica along with other minerals such feldspar, muscovite etc. These hard particles with high velocity, damage the underwater components of hydro turbine.

Measures taken by NHPC to mitigate/limit damages in under water parts due to silt:

- The efforts require avoiding or delaying the abrasion with either using hard material to match the hardness to that of quartz particles.
- HP HVOF coating of underwater parts – runner, guide vanes, cheek plates, etc has been undertaken at these effected power stations to mitigate abrasion/erosion problem.

SEDIMENT EROSION EXPERIENCES

Damage uncoated Guide Vanes after one year of operation (Teesta-V PS)

Damages in Coated Guide Vanes after three years of operation (Teesta-V)

SEDIMENT EROSION EXPERIENCES

RUNNER:

Severe loss of material along the length of the blades resulted in weight loss of up to 3% of original runner weight after every season.

Profile of *repaired runner* altered within 2 years of operation leading to reduced efficiency.

A repaired & *coated* Runner last three seasons, whereas an *uncoated* Runner does not last more than one season.

EFFECT OF SILT ON AUXILIARIES :

- Frequent choking of coolers : Generator, Guide bearings, Generator Transformer Coolers.
- Wearing of Shaft seal
  - Use of seepage water/Clean water instead of drawing from Draft Tube for shaft seal requirement. Its implementation at Dulhasti & Teesta-V power station has minimized the failure of shaft seal.
  - In Tanakpur power station, a separate cooling pond has been made which reduces the effect of silt during monsoon.
  - Also the shaft seal box needs to be pressurized with clean water during silt flushing and pneumatic seal applied.

O&M BEST PRACTICES –
Critical Spares & Inventory Management

Power stations performance sometimes are affected by non-availability of critical spares of high value. These high value spares have more lead time due to manufacturing cycle. Remote location of Hydro power plants further aggravates the problem.

NHPC has identified the critical spares for each power station along with Min/Max inventory and Re-order level. Each power station is required to maintain minimum critical inventory level.

The procurement/contract process – initiation of purchase, award and delivery is to be monitored to ensure critical spares availability to carry out annual / capital maintenance as per schedule and also to meet any exigencies.

O&M BEST PRACTICES –

Preventive / Annual Maintenance

Preventive / Annual Maintenance:

Maintenance schedules are framed based on past history, Operational aspects, Technical Inspection & recommendation of OEM.

It is to be ensured that the schedules are implemented thereby minimizing Breakdowns/Forced Outages.

HP HVOF coating of underwater parts – runner, guide vanes, cheek plates, etc is to be undertaken at effected power stations to mitigate silt abrasion/erosion problem.

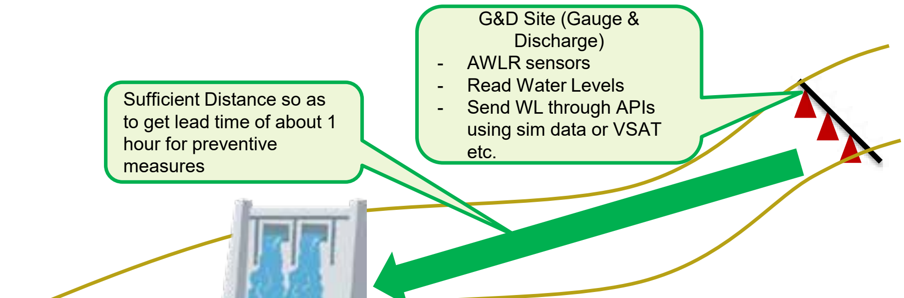
Replacement viz-a-viz Repair of Major components:

With experience its has been established that "replacement" of major underwater parts is less time consuming method than "repair & reuse" method. This strategy has led to reduction in the maintenance period and increased the Operational Availability of the Plant.

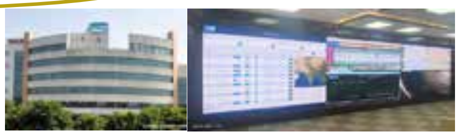
EARLY WARNING SYSTEM

- Hydroelectric projects in hilly regions are more vulnerable to the occurrence of Cloud Burst, Flash Flood etc which causes losses to life and property.
- It is necessary to set up Early Warning System (EWS) to avoid losses due to these disasters. Early warning systems give people time to react during floods, enabling local administration and people to evacuate or shelter before a flood.
- 47 nos. Projects / Power Stations (in which 10 nos. are from NHPC and 2 nos. are from CVPPPL) have been identified as vulnerable.
- **The task of setting up a Master Control Room was entrusted upon NHPC by MoP for all the hydro power generating companies of the country.**
- NHPC has established Master Control Room at NHPC Faridabad office with 24X7 real time monitoring and alert generation mechanism.
- A software named "eAABHAS" for this purpose have been developed and a Master Control room with advanced technologies including video wall is now fully functional.

THE PLAN



- Discharge is above Long Term Max Discharge
- Discharge is above 75% of Long Term Max Discharge
- Discharge is above Long Term Average
- Discharge is between Long term Min & Average



Master Control Room Software at NHPC CO

1. Water levels will be received in the software at regular intervals
2. Forecast the discharge at Dam using rating Curves and discharge at G&D
3. Issue Alerts as per warning levels through Hooters, SMS, emails etc.

O&M BEST PRACTICES –

Condition Monitoring Test

- Periodic off line condition monitoring test is be done on electrical equipment.
- Various On-Line condition monitoring systems to be kept in healthy condition and action to be taken on any abnormality .
- Preventive Maintenance/ replacement to be planned accordingly.

ASSET MANAGEMENT

Future Outlook

- Asset management procedure shall be implemented in new analytical software solution with new age ERP.
- The process flow will be automated and no intermediate human judgment will be required.
- Real Time Monitoring (RTM) is being implemented to monitor operation from centralized location.
- Provision for relay data (EL/DR) management from central location is being implemented.
- A centralized portal is proposed to be developed, to store all breakdown , analysis reports and restoration procedure for equipment, which will be accessible to all Power Station.

THANK YOU

PIONEERING UNDERGROUND TOGETHER – ADVANCEMENTS IN TBM TUNNELING IN HIMALAYAS WITH A CASE STUDY

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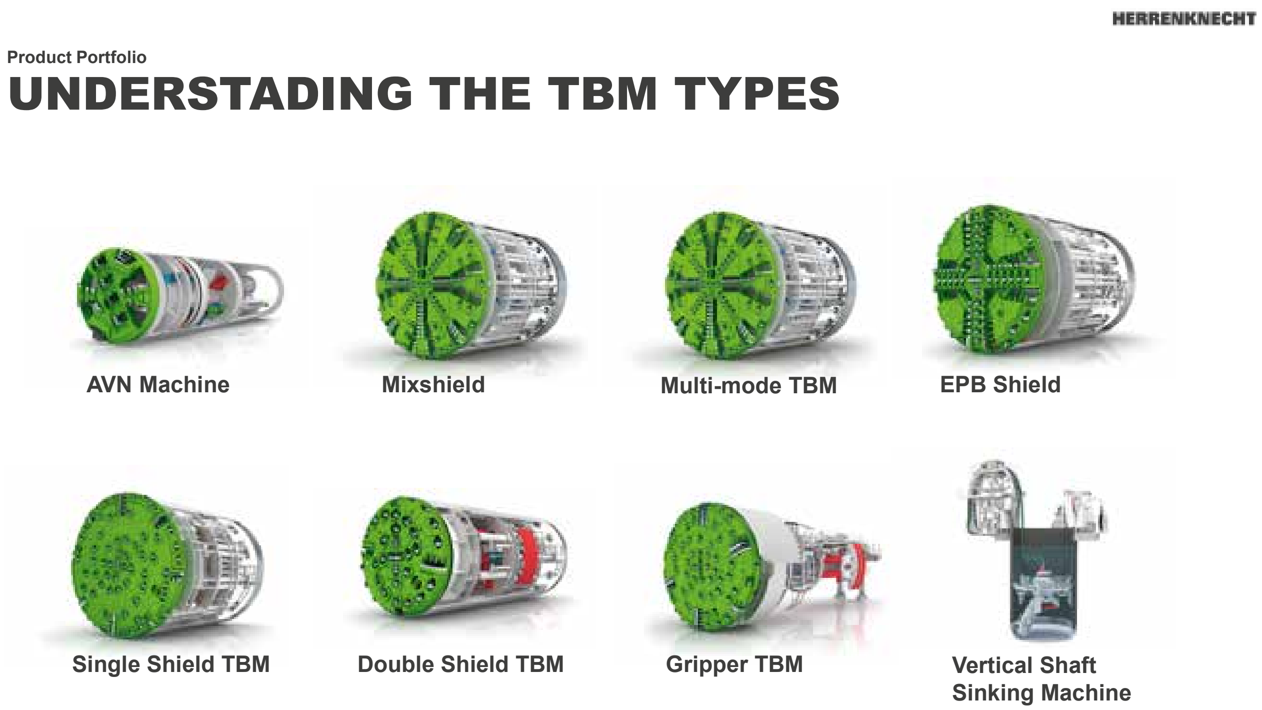
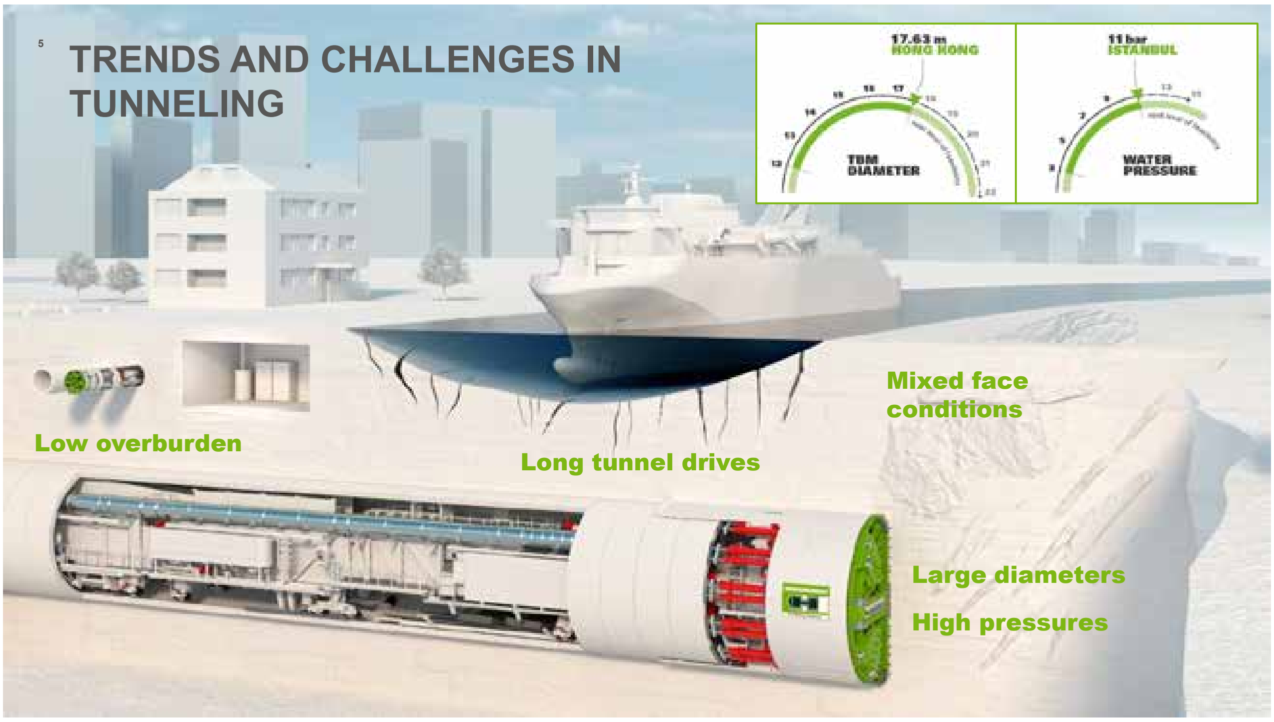



4
Increasing the diameter
GOING BIGGER
STEP BY STEP
The world's largest Variable Density TBM
Hampton Roads Bridge Tunnel, Virginia, USA
Ø 13,990 mm
Currently the largest TBM in the southern Hemisphere
Melbourne West Gate Tunnel, Australia
2x EPB Shield
Ø 15,550 mm
HERRENKNECHT
Ø
> 14 m
15 m
1.80 m
14.20 m
15.20 m
15.43 m
15.55 m
15.87 m
17.63 m
19.25 m
22.00 m


5
TRENDS AND CHALLENGES IN TUNNELING
17.63 m HONG KONG
TBM DIAMETER
11 bar ISTANBUL
WATER PRESSURE
Low overburden
Long tunnel drives
Mixed face conditions
Large diameters
High pressures

HERRENKNECHT

TBM TUNNELING EXPERIENCES
HIMALAYAN TERRAIN

Sl No	Project Name	Purpose	Tunnel Length /TBM Tunneling (m)	Overburden (m)	TBM's	TBM Type	Status
1	Parbati II, Himachal Pradesh	HEP	1 HRTx31,500 / 9,050	100-1300	2x6,800 mm	DSH/Gripper	Completed in 2023
2	Tapovan-Vishnugad, Uttarakhand	HEP	1 HRTx12,100 / 8,600	1100	1x6,570 mm	DSH	Stopped after Alpine Bankruptcy/ Restarted
3	Vishnugad-Pipalkoti, Uttarakhand	HEP	1 HRTx13,400 / 13,400	800-1500	1x9,860 mm	DSH	Stopped after Alpine Bankruptcy/ Restarted
4	Lambadug, Himachal Pradesh	HEP	1 HRTx4,600 / 3,500	~200	1x3,500 mm	Gripper	Completed
5	Kishanganga, J&K	HEP	1 HRTx23,650 / 14,750	1300	1x6180 mm	DSH	Completed
6	Rishikesh-Karnprayag Package-4, Uttarakhand	Railway	2x15,100 / 2x10,400	450-816	2x9,100 mm	SSH	Completed in 2025
7	Pakal Dul, J&K	HEP	2 HRTx9,700 / 2x7,700	Upto 2000	2x8,330 mm	DSH	Ongoing (70% Complete)
8	Neelum Jhelum, PoK	HEP	2 HRT x 11,000	Upto 1200	2x8530 mm	Gripper	Completed
9	Beri Babhai, Nepal	Irrigation/HEP	1 x 12208	Upto 820	1x5.06m	DSH	Completed before time
10	Sunkoshi Marin, Nepal	Irrigation/HEP	1 x 13,300	Upto 1320m	1x6.4m	DSH	Completed before time

7

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CASE STUDY 1
RAIL VIKAS NIGAM LIMITED – RISHIKESH KARNPRAYAG RAILWAY TUNNEL

Package 4, Uttarakhand, INDIA

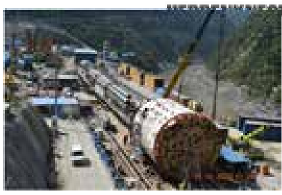
HERRENKNECHT

RISHIKESH KARNPRAYAG RAILWAY TUNNEL
PACKAGE – 4 T8 PROJECT INFORMATION

- Rishikesh – Karnaprayag Railway Line project is a 125.20 km single-track railway line under construction in Uttarakhand by Rail Vikas Nigam Limited (RVNL)
- Location 230km north-east Delhi, Himalaya foothills besides Ganges River
- Along its route, 105.47 km (84.24%) of it will consist of 35 bridges and 17 tunnels including a 15.1 km tunnel, one of the longest in the country
- Project is divided in to 10 packages, out of which Package 4 has **Two (2) parallel tunnels excavated by TBM method using precast tunnel lining + precast invert elements**
- TBM Tunneling: Attributing to the long length of tunnel with no intermediate access along the mountains with rock cover of upto 800m

8

Rishikesh – Karnaprayag Link (Railway)



- Owner = RVNL, Contractor = L&T
- S-1309A / S-1310A / 2x Single Shield TBM dia 9.11 m, EXW Schwanau
- 2x 10.5 km Tunnel / min. to max. overburden 22 to 819 m
- Additional Herrenknecht Supplies:
  - 4x MSV (TMS)
  - 11km Conveyor Belt (H+E)
  - Grout Plant (HAG)
- Both TBM's 100% Drive completed in April 2025

9

RISHIKESH KARNPRAYAG RAILWAY TUNNEL
PACKAGE – 4 LONGITUDINAL C/S

Quarzitic Phyllite	
Chainage	**47+584 to 51+000**
Characteristics 	• Medium strong to strong • Fine to medium grained • Fresh to slightly weathered • Massive, slightly jointed, tight foliation

Schistose Phyllite	
Chainage	**51+000 to 58+730**
Characteristics 	• Medium strong to weak • Fine grained • Fresh to slightly weathered • Jointed, foliated

RISHIKESH KARNPRAYAG RAILWAY TUNNEL
PACKAGE – 4 T8 GEOLOGICAL ISSUES/CHALLENGES

- **FACE STABILITY** ~ challenging for the project due to rock type and rock mass quality along the tunnel alignment
- **ROCK STRENGTH** ~ rock strength along the tunnel alignment varies from 25 to 114 MPa
- **ABRASIVENESS** ~ CAI values of up to 4.93 make the anticipated geology along the alignment highly abrasive
- **HIGH ROCK COVER** ~ 450m – 530m between chainage 48+500 and 48+700, 816m between chainage 51+600 and 53+900.
- **WATER INFLOW ~** joints and fractured zones / fault zones bear the risk of high inflows; necessary to provide dewatering systems on proposed TBMs.
- **CLOGGING ~** risk of clogging is present in interbedded layers of shale and fault zones with completely weathered rock / soil.
- **MIXED FACE CONDITIONS** ~ likely to occur in fault zones featuring a mix of soil and rock
- **GROUND CONVERGENCE** ~ very high overburden in combination with schistose phyllite, thinly interbedded with sandsone and silststone with UCS in the order of <25MPa; High overburden in combination with fault zones tectonic zones, weak rock with repeated discontinuities.
- **GEOLOGICAL STRUCTURES** ~ The main geological structures in the bedrock are foliation, joints, shear zones, slight folding and small faults.

11

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Rishikesh Karanprayag P4 T8

Technical Specification Hard Rock Single Shield TBM 9,050 mm Diameter

Herrenknecht. Pioneering Underground Technologies

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RISHIKESH KARNPRAYAG RAILWAY TUNNEL
TBM DESIGN CONSIDERATIONS

- To Combat Squeezing
  - Additional Overcut / Vertical Shifting of TBM Main Drive Unit
  - Lifting of the TBM Drive Unit
- To Prevent High Water Ingress
  - High Performance Drills for probing
- To Reduce Geotechnical Uncertainty Risk
  - Exploration Drilling in Regular Intervals
  - Geophysical instruments such as ISP
- To Deal with Potentially Large Amounts of Water / Fines
  - Dewatering System/Concept & Emergency Dewatering
  - Design Considerations for Abrasive Geology such as Wear Protection/Detectors

13

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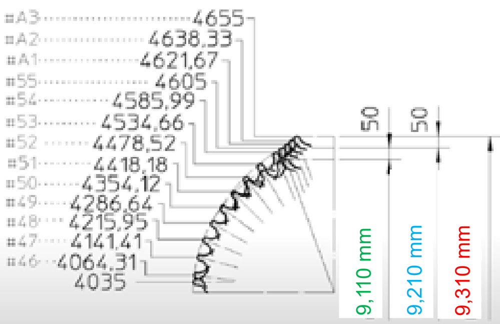
Overcut

- Regular gauge profile for nominal bore diameter 9,110 mm
- Extension in radius 50 mm by shifting / use of 20” cutters
- Extension in radius 100 mm by installation of three (3 x) special overcutters
- → maximum overcut in crown 250 mm (invert: 10 mm)



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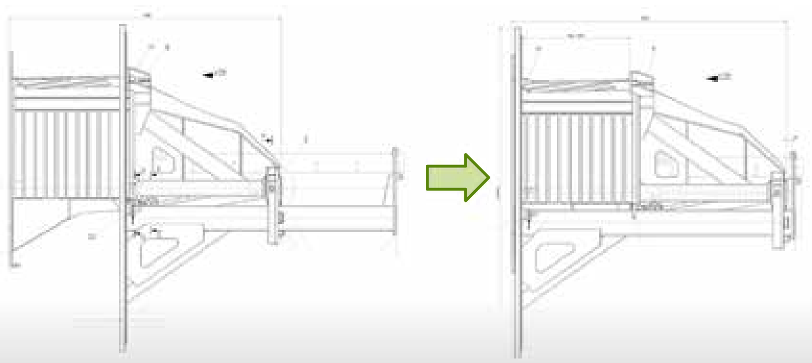
Retractable Muck Ring

- Reduction of the risk of ground inflow into the TBM by isolation of cutterhead chamber by retraction of muck ring.



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Electric Main Drive

- 4,200 kW (12 x 350 kW) VFD controlled electrical drive unit
- Main bearing diameter 5,086 mm
- Torque box
- DUAL STAGE planetary gearboxes
- 3 inner / 3 outer seals
- Automatic lubrication including HBW

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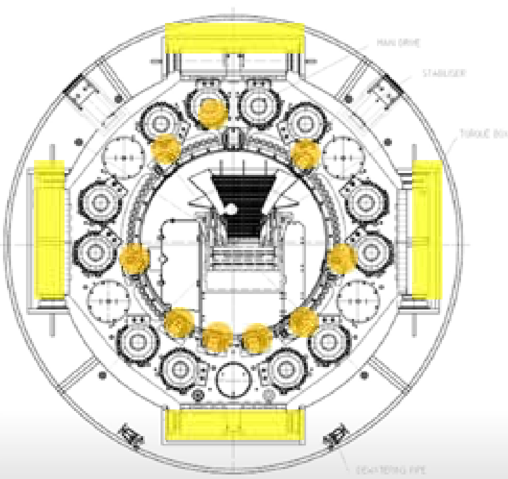
Electric Main Drive ~ Torque Box

- Horizontal and vertical drive hydraulic displacement during TBM operation permits to shift max overcut in any direction necessary to free the head (WITHOUT BOLTING)
- Axial drive displacement to make the cutterhead retractable for tool changes



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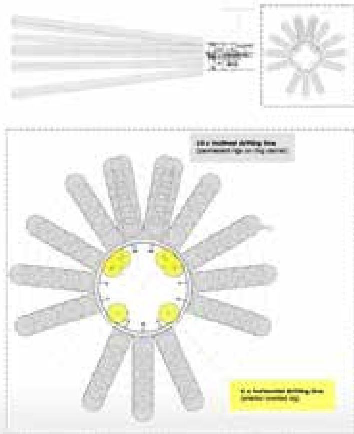
Rock Drills & Drilling / Grouting Pattern

- 2 (TWO) DOOFOR 560L drills permanently mounted on an independent drill carrier to serve 15 (FIFTEEN) inclined drilling ports for inclined drilling / advance grouting
- 1 (ONE) erector mounted DOOFOR 560L drill to serve 6 (SIX) horizontal drilling ports for face drilling / grouting
- All drills are capable in excess of 50 – 60 m drilling ahead of the face.
- 1 (ONE) core drill adapter included (can be used on either of the drill rigs)
- 1 (ONE) MWD system per drill rig (total 3)



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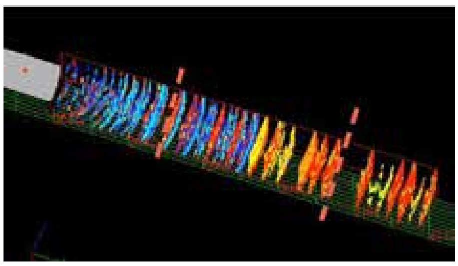
Herrenknecht Integrated Seismic Prediction (ISP)

Detection capabilities – Fault Zones & Water / Air-filled Cavities



Detection range ahead of tunnel face 20 - 80m (120 m in very good conditions)

Dependencies: *impact energy*, *noise level*, *number of repetitions*

Detectable structures front face of cavities, weaknesses / fault zones

Dependencies: *seismic impedance contrast*

Size of detectable objects ~ 5 – 15 m

Dependencies: *recorded wavelengths*, *signal quality*, *continuous measurements*

Position accuracy of detectable objects ~ 5 – 15 m

Dependencies: *acquisition geometry*, *number of receivers*, *continuous measurements*

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Pre-Excavation Grouting, Advance Grouting, Secondary Grouting

- Probe holes / ground treatment equipment and provisions detail drawings
- Equipment 3 x Hāny IC310

INJECTO - COMPACT		IC	310
Capacity, max.		m³/h	1.0
Pressure, max		bar	100
Pressure, min.		bar	3
Particle size, max.		mm	5
Water connection			3/4"
Unsable content		l	100
Circulation capacity		l/min	540
Unsable content		l	150
Power consumption	400V, 50Hz	kW	7.5
	460V, 60Hz	kW	9
Weight		kg	570

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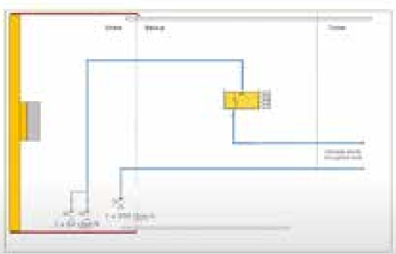
Dewatering System

- **Total dewatering capacity 328 m³/h**
- Pump in shield area Pump 2 No.
- capacity (each) Standby 64 m³/h
- pump in shield area Pump 1 No.
- capacity 200 m³/h



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CHALLENGES DURING MINING

EXCESSIVE GROUNDWATER



During probe drilling operations, excessive groundwater was encountered in the ground. The proactive implementation of regular **probe drilling enabled early detection of the water-bearing zone.**

To manage the groundwater inflow, the affected area was treated using **PU grouting, which effectively reduced the water ingress** to a manageable level. Following successful pressure grouting and confirmation of reduced water flow, the Tunnel Boring Machine (TBM) was safely advanced through the treated section.

CHALLENGES DURING MINING

Bentonite Lubrication System



Due to challenging geological conditions, increased TBM thrust was required to maintain forward progress. In response, the **TBM's inbuilt bentonite lubrication system** was activated to pump bentonite through the shield lubrication ports. This created a lubricating layer between the TBM shields and the surrounding ground, effectively **reducing friction**.

Additionally, the **TBM's capability to generate high thrust,** combined with the support of a high-pressure pump, enabled successful advancement through the difficult terrain.

CHALLENGES DURING MINING

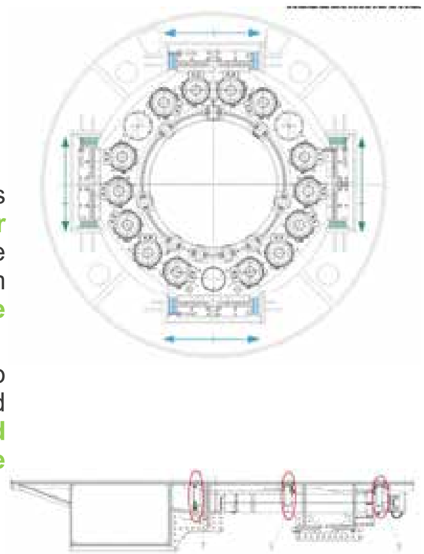
TBM Recovery from Fractured Zone and Shield Obstruction

The TBM encountered a fractured ground zone accompanied by ground settlement, which led to the machine becoming severely stuck. A large **rock fragment became wedged between the shield and the surrounding ground,** preventing further movement.

To overcome this obstruction, the TBM's in-built high-pressure hydraulic system was activated. The **high-pressure pump generated a thrust force of 130,000 kN,** which enabled the TBM to resume advancement through the challenging section.

CHALLENGES DURING MINING

TORQUE BOX AND VOID MONITORING SYSTEM



The TBM is equipped with a Torque Box system, which allows for controlled **overcutting to increase the boring diameter (R+100mm).** During extended stoppages—such as those required for maintenance—this system was utilized to perform an overcut. This helped **relieve ground pressure around the TBM shields and minimized the risk of ground settlement**.

In parallel, the void monitoring system was actively used to continuously track and assess the gap between the TBM shield and the surrounding ground, ensuring **ground stability and enabling timely intervention if abnormal voids were detected**.

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CHALLENGES DURING MINING
CUTTERHEAD DRIVE SYSTEM PERFORMANCE IN FRACTURED GROUND CONDITIONS

The TBM is equipped with 12 dual-stage planetary gearboxes, designed to ensure high torque transmission and reliable operation, particularly in hard rock and fractured ground conditions.

Level 1: Cutterhead rotation speed up to 2.5 rpm with a nominal torque of 24,304 kNm.

Level 2: Cutterhead rotation speed up to 6.3 rpm with a nominal torque of 10,230 kNm.

This two-level drive system allows for flexible adjustment between torque and speed depending on ground conditions. **In fractured zones, the high torque capability at low RPM (Level 1) ensures continuous cutterhead rotation, preventing it from getting stuck.** Maintaining cutterhead rotation in such conditions is critical, as prolonged stoppages could lead to excessive overburden pressure on the TBM shields, increasing the risk of the machine to get stuck.

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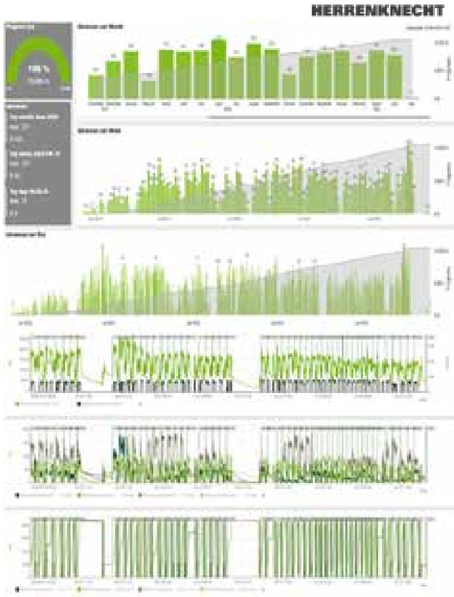
CHALLENGES DURING MINING
HERRENKNECHT CONNECTED DATA MANAGEMENT SYSTEM



The Connected Data Management System developed by Herrenknecht is an advanced digital platform designed for **real-time monitoring, analysis, and management of Tunnel Boring Machine (TBM) operations.** It integrates all critical machine and project data into a centralized system, enabling enhanced transparency, operational safety, and efficiency throughout the tunnelling process. By continuously ensuring that the **TBM operates within recommended parameters, the system supports proactive decision-making, contributing significantly to the successful and timely completion of the project.**

Journey of “Shakti” TBM from Factory to Site

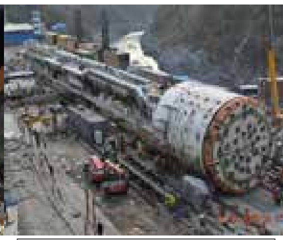

Factory Inspection at Germany


TBM Transportation on Hills


TBM Transportation on Bridges


TBM Assembling Area at T8 P1


TBM Shield assembling


Completion of TBM Assembly


Pushing of TBM inside Tunnel


Twin Tubes for UP and DN Lines

Tunnel Boring Machine (TBM) - Work Site

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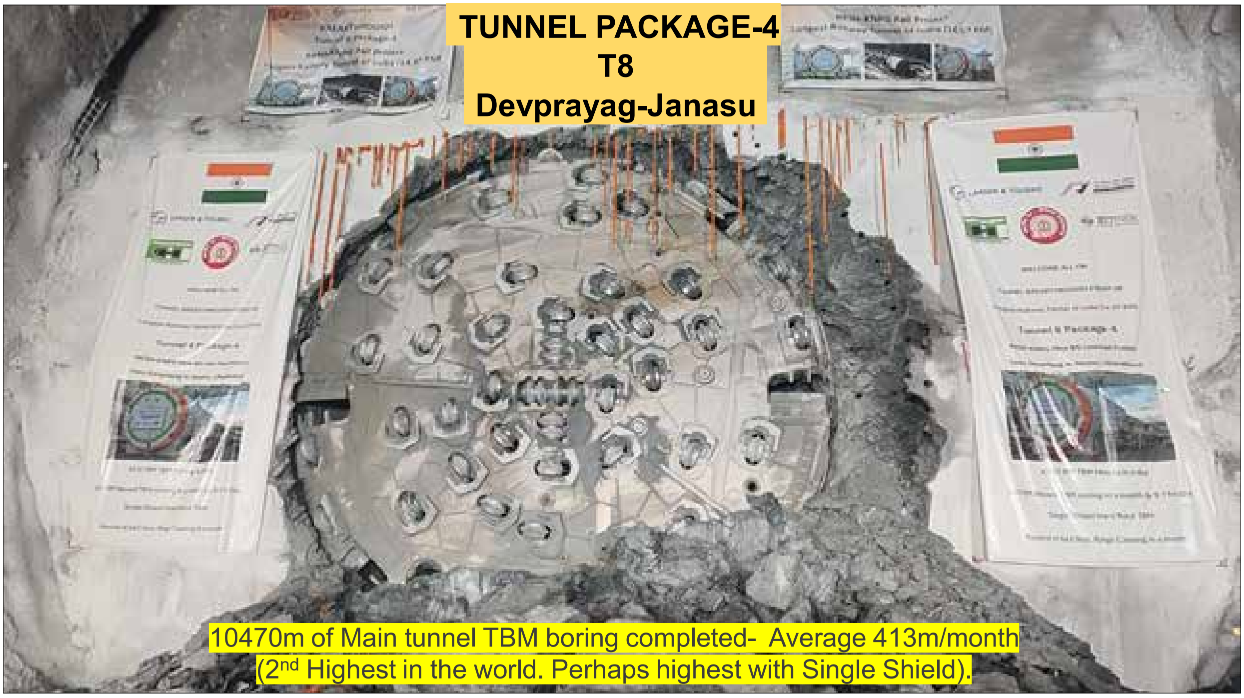

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CASE STUDY 2
PAKAL DUL HEP

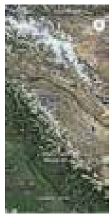
KISHTWAR, J&K, INDIA

Pakal Dul HEP

2022-today

- Excavation of 2 Nos HRTs of 7.35km length @ +0.735%
- Location at Kishtwar, Jammu & Kashmir (Himalaya), approx. 1,500 to 1,600m above sea level
- Min. / Max. overburden of 400 to 2,000m
- 2 New DSH TBMs of Dia 8.33m to excavate from 2 construction adits in the surge shafts
- Geology:
  - quartzite/gneissic granite
  - RMR Classification: 30% - I /II good rock, 50% - III fair rock, 20% - IV/V poor rock

- Geo issues:
  - Fault zone, vertical shear zones, Very to extremely abrasive rock
  - Sudden ingress of water under high head (cover ~400m)
- TBM S-1294 – Tunnelling, TBM S-1293 – Tunneling



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Pakal Dul Hydroelectric Power Project

Summary of Geological and Hydrogeological Issues

- **Geological condition:**
  - **Heterogeneous rock mass with varying compressive strengths 20MPa up to high rock strength of 250 Mpa,**
  - **Medium to very abrasive rocks (CAI > 4),**
  - **Frequent occurrences of fractured, sheared, finely foliated, & faulted zones.**
- **Local condition:**
  - **Remote location & condition of access roads**
  - **Mountainous environment**
  - **Limitations for transportation – in size and weight, narrow roads**
  - **Limited space on jobsite**
  - **Logistic & Supply to the jobsite can affect the TBM performance**

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Pakal Dul Hydroelectric Power Project

Summary of Geological and Hydrogeological Issues

- **Geo-hazards:**
  - **High cover (~400m to ~2000m) cause rock stress issues**
    - **in hard rocks like gneissic granite/quartzite – rock bursting, spalling, popping**
    - **& in soft rocks like phyllite, schists – moderately to high convergence (squeezing)**
  - **Expected high water ingress due to rainfall & snow cover at higher elevations**
  - **Instance of debris / silt flow under high water ingress**
  - **Overbreak and cavity formations in poor to very poor rock conditions in Schist, Phyllite & Slate, in proximity of fault and intense folding; over break owing to slabbing**
  - **High geo-thermal gradient under high cover zones**

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TBM DESIGN FEATURES

- **Largest possible overcut**
- **Shortest possible shield**
- **Conicity of the shield, Col**
- **Retractable Muck Ring**
- **Dewatering System**
- **3 Probe Drills, Grout Ports**

TBM PERFORMANCE PAKAL DUL S-1294A

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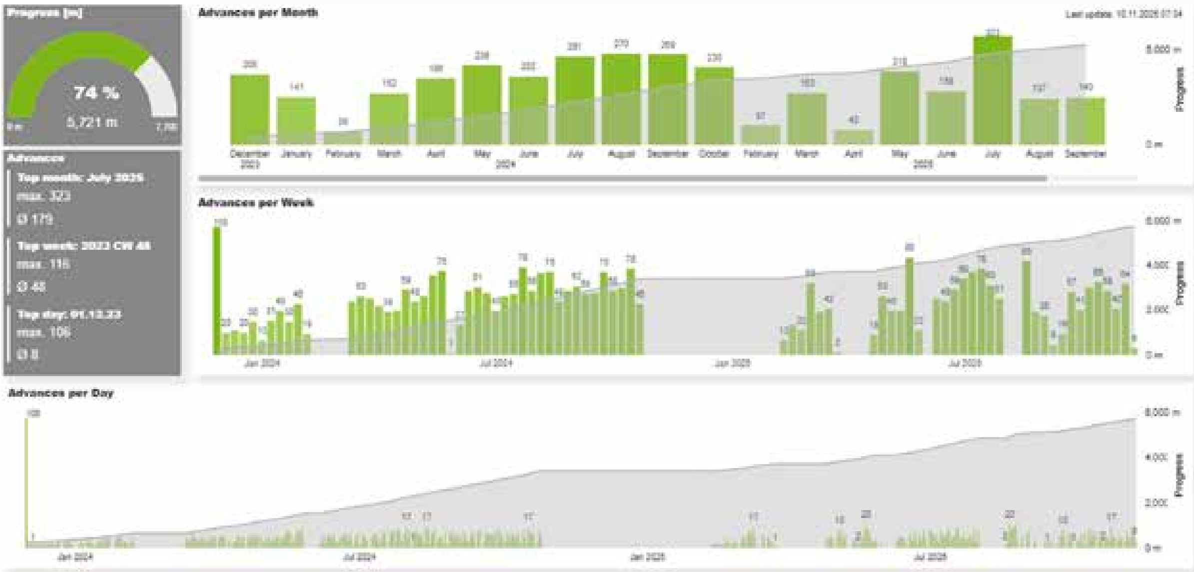

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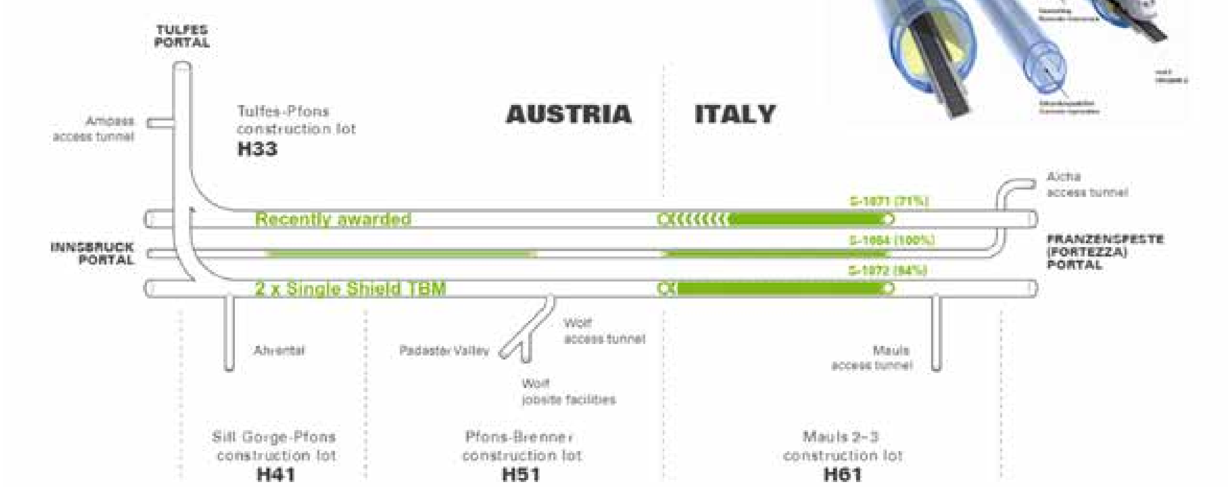
TBM TUNNELING EXPERIENCES IN HIGH MOUNTAINS AROUND THE WORLD

BRENNER BASE TUNNEL
THE LONGEST UNDERGROUND

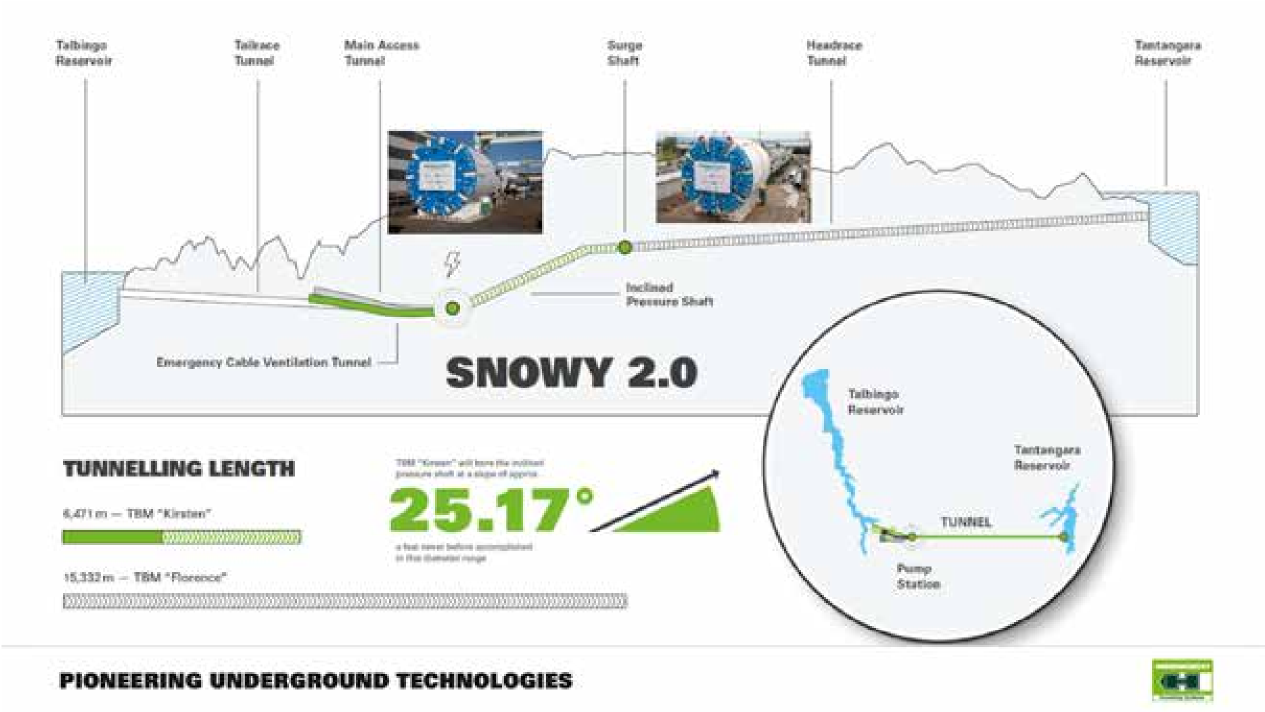


PIONEERING UNDERGROUND TECHNOLOGIES

BRENNER BASE TUNNEL
THE LONGEST UNDERGROUND RAILWAY CONNECTION IN THE WORLD

- **Gripper TBM at tunnelling since September 2015 for the 15 km long exploratory tunnel Tulfes-Pfons**
- **Current tunnelling status: more than 10 km advanced**
- **South: 3x Double Shield TBM sold for lot "Mauls"**
- **North: 2x Single Shield TBM for lt "Pfons"**

PIONEERING UNDERGROUND TECHNOLOGIES



Talbingo Reservoir
Tailrace Tunnel
Main Access Tunnel
Surge Shaft
Headrace Tunnel
Tantangara Reservoir
Inclined Pressure Shaft
Emergency Cable Ventilation Tunnel
SNOWY 2.0
Talbingo Reservoir
Tantangara Reservoir
TUNNEL
Pump Station
TUNNELLING LENGTH
6,471 m — TBM "Kirsten"
15,332 m — TBM "Florence"
25.17°
PIONEERING UNDERGROUND TECHNOLOGIES

TAILOR-MADE SOLUTIONS FOR SPECIAL CHALLENGES.
DECLINED AND INCLINED TUNNELS.
Limmern, Gripper TBM, Ø 5.20 m
40° incline, tunnel length 2 x 1,023 m
Shafts for pumped-storage power plant
30°
TBM
St. Petersburg, EPB Shield, Ø 10.
30° decline, tunnel length 120 m
Escalator shaft for metro station
PIONEERING UNDERGROUND TECHNOLOGIES

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SHAFT SINKING FOR U-PARK®

World premiere in Nanjing

- Vertical Shaft Sinking Machine VSM12000 (Ø 12,800mm)
- 2x 66m deep shafts for 100 parking lots on 25 levels each

PIONEERING UNDERGROUND TECHNOLOGIES

RAISE BORING RIG RBR. RAPID, SYSTEMATIC AND SECURE SHAFT CONSTRUCTION.

- **Precise construction of shafts in rock to 2,000 meters in depth**
- **High flexibility even under space constraints due to compact design**
- **Safer, less personnel-intensive and more cost-effective compared to conventional shaft sinking**

PIONEERING UNDERGROUND TECHNOLOGIES

SUMMARY - TBM PROJECTS IN HIMALAYAS

Herrenknecht AG / Herrenknecht India

Tunnelling Challenges in the Himalayas

(Carter 2014)

- In mountainous terrain when considering a decision on whether or not to utilize a TBM, and which type of TBM to use for a planned deep tunnel, it must be appreciated that historically, three types of ground conditions have proved the most problematic from the viewpoint of halting tunnel advance. In order of severity, case records suggest:
  - (i) bad faults
  - (ii) heavy water and (iii)
  - (iii) major stress
- individually, and/or in combination, constitute the most problematic ground conditions.

Drill & Blast Tunnelling vs TBM Methods
Planning

- The design of a tunnel should at a very early stage evaluate the excavation method
- Consideration of only D+B tunnelling during the planning stage may exclude most of cost saving features for TBM methods
- Contract Type plays a key Role in Success of the Project
  - EPC
  - BOQ
- A good Geological investigation and a well prepared GBR is a key success factor for Tunneling in Himlayas
- Risk Sharing Matrix will result is cost effective tender

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Drill & Blast Tunnelling vs TBM

Cost Saving Features of TBMs	
DIRECT	**INDIRECT**
Reduced number of adits	Reduced requirements for construction of access roads, communication, power supply etc
Electric Operation	Reduced ventilation requirements
circular cross section	Less rock support, more stable
Higher advance rates	Less shotcrete, continuous mining possible

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Advantages of Longer TBM Bored Tunnels

- Possibility to chose different route for improved geological conditions
- Possibility to chose more direct route when there is no need to connect with several adits
- Less environmental impact
- Safer tunnelling
- Higher speeds of tunnelling

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Drill & Blast Tunnelling vs TBM

Project Specific Factors	Size of Cross Section		Form of Cross Section		Tunnel Length	
	uniform	variable	uniform	variable	short	long
Drill & Blast	🟩	🟩	🟩	🟩	🟩	🟥
TBM Tunnelling	🟩	🟥	Circular	🟥	not usual	🟩

🟩 well suited
🟥 not suited

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Comparison of Drill & Blast and TBM Tunnling	DRILL & BLAST	TBM
Long Distances	More working faces	Less working faces
Requirements for Adits	Higher	Lower
Number of Working Sites	Higher	Lower
Tunnel Cross-Section	Easily varied	Cannot be changed
Lining Thickness	Easily varied	Cannot be changed
Overbreak	Much higher	Lower
Working Flexibility	Higher	Lower
Crew Safety	Lower	Higher
Working Environment	Lower Quality	Higher Quality
Ventilation Requirement	Much higher	Much Lower
Personnel Number	Higher	Lower
Construction Time	Higher	Lower
Construction Cost	Higher	Lower
Construction Planning	Early start-up possible	Long delivery for TBM

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SUMMARY – TBM UTILIZATION IN HIMALAYAS

DECISION ON TBM VS NATM / FACTORS INFLUENCING SUCCESS OF CHOSEN METHODOLOGY

- **Geographical/ Geological Aspects**
- **DPR/ Pre-feasibility Reports**
- **Detailed Geo-technical & Geo-physical Investigations (Geotechnical Baseline Reports)**
- **CAPEX & Project Timelines**
- **Accessibility of Project Sites – Logistics Possibilities of major items/components**
- **Availability of Local Facilities & Services**
- **Skilled/Experienced/Dedicated Manpower who can undertake** Meticulous planning, Precise & timely decision – Main reason for success of TBM tunneling in Himalayas so FAR
- **Mitigation Measures and Readiness to Encounter Major Challenges** to overcome all hurdles
- Backup Support from Contractors/Vendors/Suppliers

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IMPROVING RESILIENCE OF HYDROPOWER INFRASTRUCTURE FROM SEISMIC RISKS IN BHUTAN: A CASE STUDY

DR DOWCHU DRUKPA
Geotechnical Specialist, Druk Green Power Corporation (DGPC)

ABSTRACT

Hydropower infrastructure represents a critical cornerstone of Bhutan's economic development, yet these assets face daunting seismic threats due to active geodyamics of the Himalayan collision zone. Here we examine the challenges and opportunities presented towards improved seismic resilience through comprehensive site-specific seismic hazard analysis performed for major hydropower projects in the country.

The gradual and steady stress build-up during inter-seismic periods in the Himalayan region due to convergence of the India and Eurasia plates pose significant seismic risks for hydropower infrastructure development in Bhutan, which necessitates stringent enforcement of seismic risk reduction measures. A review of site-specific seismic hazard analysis performed for five major hydropower projects namely Bunakha, Sankosh, Dorjilung Jeri and Khorlochhu reveals significant methodological evolutions over the last one and half decades. Earlier assessments relied on limited Ground Motion Prediction Equations (GMPEs) and simpler analytical frameworks, while recent studies employ advanced Probabilistic Seismic Hazard Analysis (PSHA) with logic-tree approach to systematically reduce epistemic uncertainties. This modern assessment methods incorporates multiple models including the use of NGA-West2 model and longer safety evaluation return periods of 10,000 years representing significant improvement in the standards over the years.

Despite these improvements, critical challenges still remain which impedes our effort to develop a comprehensive seismic risk reduction strategy in the country. The synthesis of seismic hazard parameters across different projects in the country indicate substantial variability with Peak Ground Acceleration (PGA) values for 475-year return periods ranging from 0.14g to 0.34g and drastically increasing to 1.53g for 10,000-year return periods at sites close to the main seismogenic sources. This variability stems mainly from three interconnected factors: the lack of high quality local seismic data leading to reliance on global models, inconsistent application of evolving methodologies across project timelines, and important regulatory gaps between engineering practice and national safety standards.

A significant discrepancy exists between the Safety Evaluation Earthquake (SEE) specified in the Dam Safety Guidelines of Bhutan that advocates 1:25,000 annual exceedance probability and the current engineering practice that limits to 1:10,000 return period for calculations of MCE/SEE of site-specific seismic analysis of major hydropower projects in the country. Furthermore, recent regional seismic hazard studies indicate significant PGA value variations across the country with the maximum PGA reaching values of 0.77g for 475-year return periods contrary to the uniform PGA of 0.36g adopted in national building code derived from Indian standards.

To improve the resilience of critical hydropower infrastructure, three key inventions are proposed. First, it is crucial to standardize the methodologies for conduct of site-specific seismic hazard analysis for all new and existing assets with strict adherence to national and international standards. Second, there should be targeted effort to improve data quality to support the development of region-specific GMPE, thereby reducing dependency on global proxies. Third, strong emphasis should be accorded to review seismic safety of existing dams in the country using the most updated safety benchmarks.

The inconsistencies associated with infrastructure risk assessments to cope with the latest advances in geosciences represents a notable challenge for asset management and regulation. The case study from Bhutan demonstrates that ensuring infrastructure resilience requires not only technical excellence in hazard and risk assessments but development of harmonized national frameworks that translates evolving scientific knowledge into tangible safety standards. Concerted implementation of these measures provides an important opportunity to safeguard critical energy infrastructure and secure its foundation for long term sustainable development in seismically vulnerable regions.

1.0 INTRODUCTION

Bhutan is endowed with huge renewable hydropower potentials estimated to be around 36,900 MW with annual production capacity of over 154,000 GWh (Department of Hydropower & Power Systems, Ministry of Economic Affairs, 2021). However, the geographic location of being situated in one of the world's seismically active zones (Drukpa et al., 2006; Drukpa et al., 2012) dictates a strategic imperative to ensuring the seismic resilience of its hydropower infrastructure, which is of paramount importance not only for national energy security and economic stability but for safety of downstream communities.

This manuscript presents a review and critical analysis aimed at improving the seismic resilience of Bhutan's hydropower sector. The primary objective is to critically examine the seismic hazard assessment methodologies currently being applied to major projects, benchmark these practices with established international standards and identify key discrepancies and propose a strategic framework for an improved resilience of hydropower infrastructure from seismic risks.

2.0 SEISMOTECTONIC CONTEXT OF BHUTAN FOR HYDROPOWER DEVELOPMENT

A comprehensive understanding of the regional geodynamics and seismotectonic characteristics is the fundamental step in any credible seismic hazard computation. The seismotectonic characteristics of Bhutan region is governed by the continual active convergence between the Indian and Eurasian tectonic plates (Simoes et al., 2021; Singer, 2017). This continent-continent collision zone has created a complex fold and thrust zone, with major thrust faults such as Main Himalayan Thrust (MHT), Main Frontal Thrust (MFT), Main Boundary Thrust (MBT) and Main Central Thrust (MCT) that stretches along the entire Himalayan arc posing significant seismic challenges to the build-up areas in the Himalayan region (Figure 1). The presence of these major active thrust faults is of critical concern for dam engineering as these tectonic faults are capable of generating not only large magnitude earthquakes (Drukpa et al., 2006) but significant crustal deformation and widespread earthquake induced landslides that can disrupt reservoir stability and appurtenant structures (Zhao et al., 2023). Aside from these well-known thrust fault systems, there are also other faults (Figure 1) systems such as the Yadong (Long et al., 2019), Dubri-Chungthang Fault (DCF; Diehl et al., 2017), Kopili (Singh et al., 2017) and Oldham (England & Bilham, 2015) which represents significant seismic threat capable of generating large magnitude events that can devastate the Bhutan region. These structures display a vastly different geological setup in contrast to other parts of the Himalayas. The Bhutan region is dominated by large number of strike-slip faulting mechanisms (Drukpa, 2006) and the possible segmentation of MHT by DCF appears to play an important role in the geodynamics, structure and seismicity signature of the region (Diel et al., 2017).

The Bhutan region is marked by significant historical seismic events (Figure 2) that serve as a stark reminder of its destructive potential. The 1714 Bhutan earthquake (M8.0±0.5), 1897 Shillong earthquake (M8.0), 1934 Bihar-Nepal earthquake (M8.2) and 1950 Arunachal Pradesh-China earthquake (M8.7) caused widespread damages and demonstrated the region's vulnerability to such mega-seismic shaking from time to time. More recently, the 2009 earthquake centered in eastern Bhutan and the 2011 Sikkim-Nepal earthquake caused significant destruction reminding the need for holistic risk reduction and mitigation strategies. This active and complex tectonic settings of the Bhutan Himalaya region demands a rigorous, consistent and standardized approach to tackle seismic hazards for critical infrastructure.

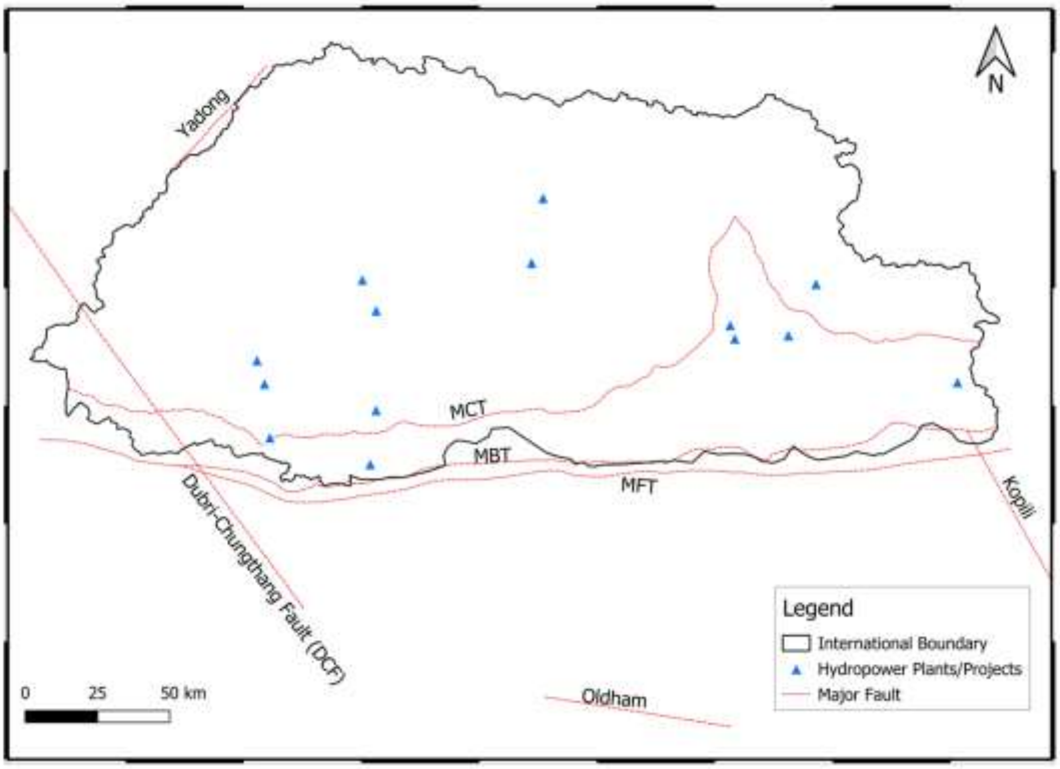

Fig. 1 : Major tectonics faults and locations of hydropower projects/plants in Bhutan

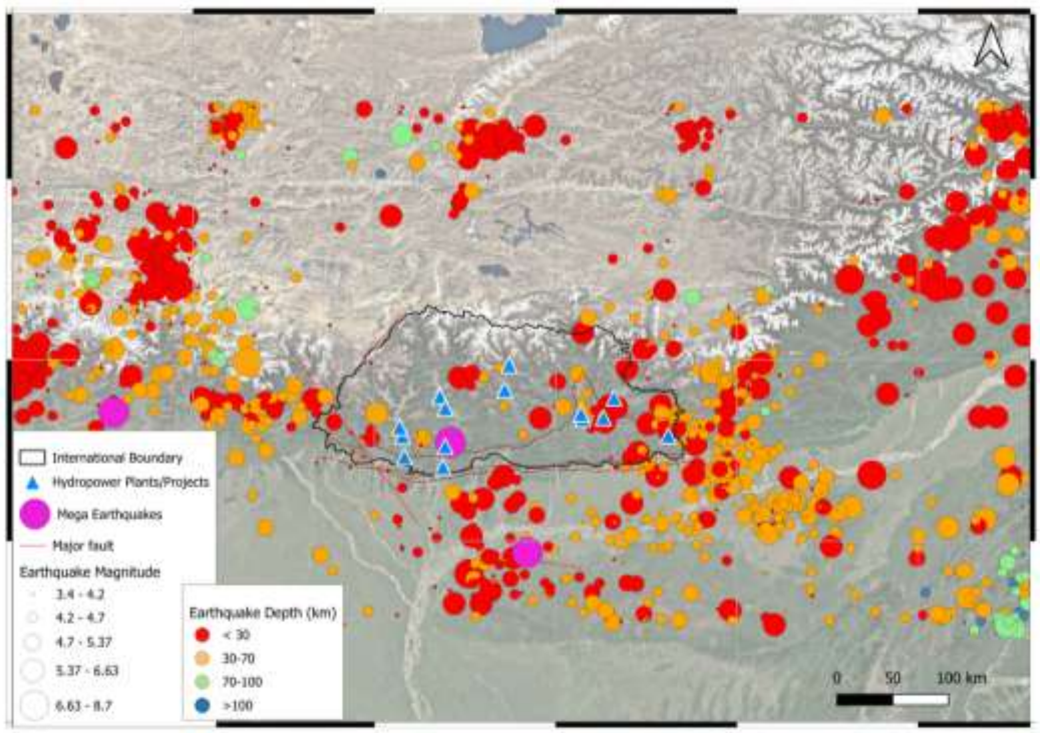

Fig. 2 : Historical seismicity of Bhutan and neighboring areas; seismicity data from NEIC, USGS (1900-2025) and mega-earthquake information from USGS significant earthquakes

3.0 SEISMIC HAZARD ASSESSMENT FRAMEWORK: A REVIEW OF APPLICABLE STANDARDS

The robust seismic hazard assessment for critical infrastructure, particularly dams, is not an arbitrary process. It is guided by decades of research and operational experiences, which has been codified in globally recognized practices. Past studies (Stevens et al, 2020; Ghione et al., 2022) focused on the characterizing the seismic hazard of the Bhutan region using probabilistic methods show high level of seismic vulnerability with PGA values in the range of 0.77g corresponding to 475 year return period. This is significantly different from the Zone V PGA value of 0.36g for 10% annual probability of exceedance according to the Indian Seismic Zonation 2002(BIS-1893 in Indian standard criteria for earthquake resistant design of structures, Part 1—General provisions and buildings, New Delhi, 2002). In absence of a national seismic guideline, Bhutan adopted the Zone V of the Indian Seismic Zonation for the entire country. This section outlines some of the important practices to establish a clear benchmark for seismic hazard analysis methodology applied to hydropower projects in Bhutan.

3.1 The ICOLD Guidelines

The International Commission on Large Dams (ICOLD) provides globally recognized best practices for selecting seismic parameters for dams. Over the decades significant evolution in methodology is evident in the updates from 1989 First Edition of Bulletin 72 to the 2010 Second Edition, published as Bulletin 148, which incorporates advances and improvement in seismic hazard analysis techniques. The ICOLD defines two key design earthquake levels for ensuring dam safety:

- **Safety Evaluation Earthquake (SEE)**: This is described as the level of ground shaking for the dam must be able to resist catastrophic failure (no uncontrolled release of water from the reservoir). This modern terminology replaces the older classifications such as Maximum Design Earthquake (MDE) and Design Basis Earthquake, used in some context.
- **Operating Basis Earthquake (OBE)**: This is less severe level of shaking during which the dam and its appurtenant structures should experience no or insignificant damage and can continue to operate.

ICOLD advocates for complementary use of both Deterministic Seismic Hazard Analysis (DSHA) and Probabilistic Seismic Hazard Analysis (PSHA). While the former focuses on worst-case scenarios from specific faults, the latter integrates uncertainties across all known seismic sources to calculate the probability of exceedance of certain levels of ground motion over time. The integrated use of both DSHA and PSHA methods provides a reliable understanding of the seismic hazard.

3.2 Indian Standards and Methodological Practices

The seismic hazard assessment for many of the hydropower projects in Bhutan, particularly prior to 2020, were performed in accordance with the Indian standards and practices. The detailed seismic assessment reports for Khorlochhu and Sankosh projects clearly show that the studies were guided by frameworks such as the Indian Standard IS: 1983 and recommendations of the National Committee on Seismic Design Parameters (NCSDP). The Indian framework uses the following terminology:

- **Maximum Credible Earthquake (MCE)**: This is defined as the largest earthquake that could reasonably be expected to occur on a given seismic source (fault). It is generally calculated using deterministic method.
- **Design Basis Earthquake (DBE)**: This is defined as the earthquake that can reasonably be expected to occur at least once during the life of the structure. This level is determined using PSHA method.

The Indian derived standards and guidelines allow DSHA to determine the MCE, while PSHA is the preferred method for estimating parameters for DBE (return period of 475 years).

3.3 Bhutan Natinal Dam Safety Guidelines

The Royal Government of Bhutan, in May 2020, established a comprehensive national framework with publication of the Dam Safety Guidelines for Hydropower in Bhutan. This document marks a significant step in aligning the standards of hydropower construction with the international standards which will improve safety, functionality and longevity of the country's critical hydropower infrastructure. The guideline outlines clear national requirements for design earthquakes based on dams classified as Large, Intermediate, or Small. Accordingly, the different seismic design levels are categorized as:

- **Operating Basis Earthquake (OBE)**: This is the level of shaking for which little or no damage occurs, allowing dam operation to continue without interruption. The OBE is typically set at 1:145 Annual Exceedance Probability (AEB).
- **Maximum Design Earthquake (MDE) or Safety Evaluation Earthquake (SEE)**: This represents the maximum level of ground motion which the dam should be designed; damage may be accepted, but there must be no uncontrolled release of water from the reservoir.

The MDE or SEE is allowed to be set equal to deterministically derived MCE or to an earthquake determined by the probabilistic method. For dams classified under Large category, the required MDE or SEE is specified as MCE or 1:25,000 AEP. For structures and components that do not have a critical dam safety functions such as penstocks and power stations, the DBE is allowed to be represented by ground motions of 1:475 AEP which corresponds to a return period of 475 years.

4.0 COMPARATIVE ANALYSIS OF METHODOLOGIES AND PARAMETERS OF MAJOR HYDROPOWER PROJECTS

Systematic comparison of site-specific seismic studies for Bunakha (180 MW), Sankosh (4000 MW), Dorjilung (1125 MW), Khorlochhu (600 MW) and Jeri (1800+740 MW) projects (Figure 3) reveals important inconsistencies mainly across three key technical definition of seismotectonic sources, the selection of ground motion prediction equations (GMPEs), and the resulting seismic hazard parameters. These differences suggest the challenges associated with seismic hazard assessment to perform uniform seismic safety evaluation of nation's critical infrastructure and highlight the evolution of methodological application over time.

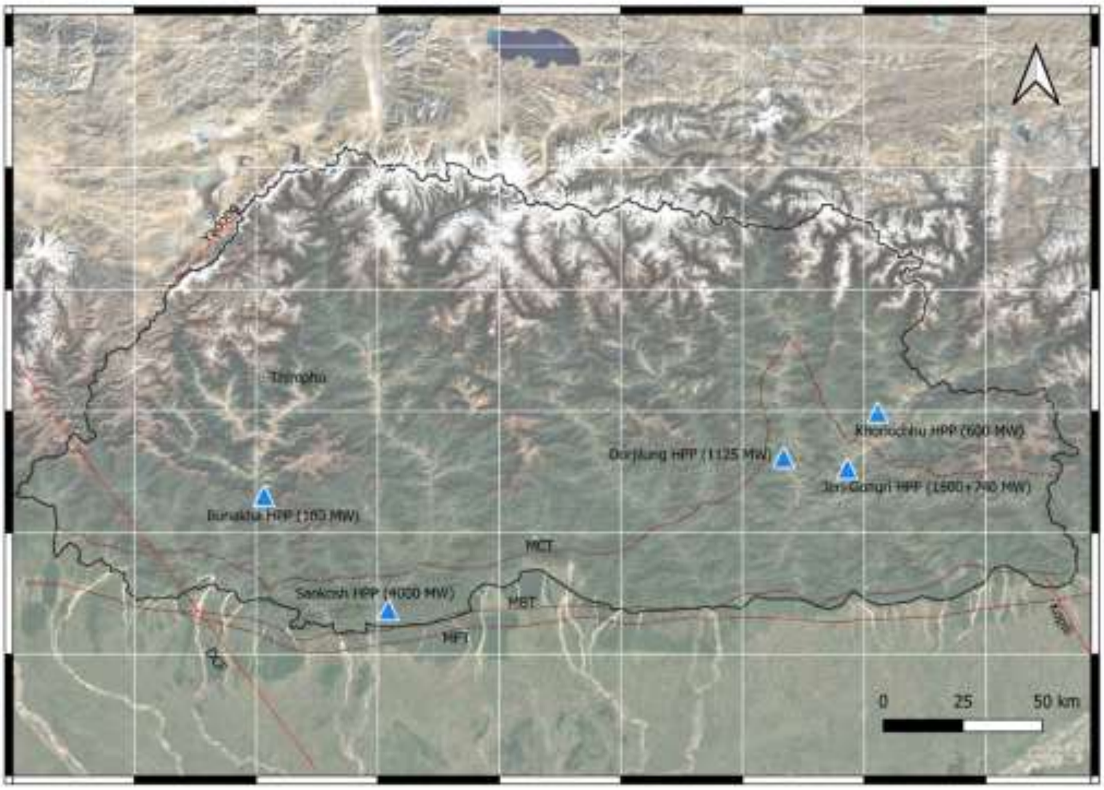

Fig. 3 : Map showing location of major hydropower project sites were detailed seismic hazard analysis were conducted and major fault lines

4.1 Seismotectonic Source Modeling

The initial fundamental step in performing site-specific seismic hazard analysis is the delineation of seismogenic sources, which includes identification and characterization of faults and areal zones with potential to generate significant earthquakes. A review of analysis of seismic hazard studies for these projects show that the delineation of these sources is an important driver of inconsistency in the final hazard outputs.

For instance, the 2025 Bunakha Hydropower Project study identified 6 area seismogenic zones, and 2 specific linear faults. These broad zones consider structures and individual faults as generalized fault zones. The remaining area was treated as background seismicity divided into 25 km x 25 km blocks. On the contrary, the 2024 study for Dorjilung Hydropower Project identified 8 areal zones and linear fault sources including MCT and MBT. The recent seismic hazard analysis study for Jeri Project show the decollement lying beneath the Himalaya as the governing seismogenic source for the site, especially for DSHA and for determining the Mmax in case of PSHA. In terms of areal seismogenic zones, 9 distinct seismic source zones were identified covering major tectonic domains. The examples of seismic assessment for Khorlochhu and Sankosh which were conducted in 2011 and 2008, respectively, took a broader approach focusing on the Main Himalayan Thrust zone as the dominant regional feature.

As mentioned above, discrete linear fault sources were considered in case of Bunakha, Dorjilung, Jeri and Khorlochhu to model known, significant tectonic structures. However, in case of Bunakha and Dorjilung, 3D modeling of the major faults was considered for PSHA calculation, while in case of Jeri the closest structure was taken to account for the highest magnitude in the PSHA computation. The older studies of Khorlochhu and Sankosh used MCT/MBT as fixed scenario inputs for DSHA to define MCA magnitude and distance, while PSHA calculation was done employing broad geographic source areas. Therefore, in absence of a unified, national level seismotectonic source model it poses huge challenge to compare the hazard outputs obtained from different projects that uses different seismic source models.

4.2 Ground Motion Prediction Equations (GMPES)

The choice of GMPEs to estimate the level of ground shaking at a site based on earthquake magnitude, distance and site conditions is an important determinant of the calculated hazard levels. The Case Studies employed different suites of GMPEs suggesting a significant methodological divergence between the earlier and current studies.

The 2011 Khorlochhu assessment relied on Boore and Atkinson (2008) attenuation relationship, while the 2008 Sankosh report adopted the Abrahamson and Silva (1997) ground motion model. On the contrary, the more recent seismic studies for Bunakha, Dorjilung and Jeri hydropower projects opted for the most recent state-of-the-art model, the NGA-West2 (Next Generation Attenuation) models, which were developed for active shallow crustal region similar to tectonic settings in the Himalayas. The models used for Jeri, including Abrahamson et al., 2014, Boore et al., 2014, Campbell and Bozorgnia, 2014, are a different suite of NGA-West2 models than those considered for Bunakha and Dorjilung. These models are based on a significantly larger and more comprehensive global strong motion database, which provides improved constraints on ground motion variability and near source effects. However, even the use of the state-of-the-art models for assessment exhibits variability. Therefore, in absence of GMPE that perfectly describes the Himalayan environment, the selection of which modern models to include in the logic trees is a key source of epistemic uncertainty. This dependence on global proxies construe a major ongoing challenge to perform a robust seismic hazard computation of the region.

4.3 Hazard Parameters and Outputs

The methodological inconsistencies in seismic source modeling and GMPE selection translates to a significant variation in the final peak ground acceleration (PGA) for the DBE level, which corresponds to a 475-year return period. For Khorlochhu the deterministic determined PGA for DBE level is estimated at 0.14g, while for Sankosh the DBE level PGA is 0.19g. Accordingly, the DBE level PGA for Bunakha, Dorjilung and Jeri is estimated to be 0.31g, 0.26g and 0.343g, respectively. This variation for the same design level is a direct consequence of differing analytical inputs and models used in each study and the way epistemic uncertainties are weighted. Furthermore, it is important to emphasize that the divergence is even more pronounced at longer return periods with the PGA for SEE level for Jeri touching 1.53g for a 9,975 year event, while the MCE PGA level for Jeri, Bunakha and Dorjilung are 0.784g, 0.59g, 0.69g, respectively. The PSHA PGA level for Bunakha and Dorjilung corresponding to 10,000 year return period is estimated at 0.68g and 0.63g, respectively. This clearly shows the pressing need to conduct a thorough evaluation of the studies against national and international safety standards.

4.4 Conformity With National and International Standards

To ascertain a consistent and high standard of safety, it is important to benchmark site-specific studies against both national regulations and international best practices. This study reveals that while recent studies generally align with modern procedural standards, there exists significant deviations in the application of specific design criteria, particularly concerning Bhutan's national dam safety guidelines.

4.4.1 *Alignment with Bhutan Dam Safety Guidelines (2020)*

The Dam Safety Guidelines for Hydropower in Bhutan (2020) provides an important framework for classifying dams and specifies the design earthquake levels. Accordingly, the guideline emphasizes that all seismic assessment studies must adopt the use of both probabilistic and deterministic methods. Based on the present Case Study of reviewing the site-specific studies for 5 hydropower projects, this requirement is fulfilled by all projects expect Khorlochhu. A notable divergence also appears in the choice of the return period for SEE, the most extreme seismic event that a dam must endure without uncontrolled release of water from the reservoir. For dams falling under the "Large" category, the guideline stresses an SEE corresponding to the MCE or a probabilistic event with 1:25,000 Annual Exceedance Probability. On the contrary, the recent site-specific studies for Dorjilung, Bunakha, and Jeri adopted a 10,000 year return period or a 1:10,000 AEP for the SEE. This either shows a potential underestimation of the seismic resilience required for critical structures as per the national dam safety guideline, or possible over prescription of the guidelines that may be too stringent for realistic engineering application.

4.4.2 *Adherence to International Standards*

The general methodologies adopted in the recent studies, including the combined use of PSHA and DSHA as well as the application of logic trees to account for epistemic uncertainties, are clearly in accordance to the international best practices, such as those outlined in ICOLD Bulletin 148. The consultants that performed seismic hazard computation for the Bunakha, Dorjilung and Jeri projects have stringently followed the latest internationally accepted procedures for seismic hazard analysis. However, the effectiveness of these standards and practices is largely dependent on the quality and reliability of the inputs. As previously explained, the lack of standardized inputs such as a unified seismotectonic model and a realistic GMPEs appropriate for the Himalayan region leads to an inconsistent application of these practices. This results in highly variable outcomes that likely compromise on overall integrity of the national dam safety portfolio. Thus, the gap between the procedural adherence and input standardization is a critical constraint that need to be reviewed and addressed appropriately.

5.0 STRATEGIC FRAMEWORK FOR IMPROVING SEISMIC RESILIENCE IN BHUTAN

To ensure a uniform, transparent and high standard of seismic safety for all current and future hydropower projects in Bhutan, a coordinated national effort is required to address the deficiencies identified in this paper. The following are measures proposed to establish a robust and consistent national strategic framework for seismic hazard assessment.

- **National guideline for site-specific seismic hazard assessment**

The Royal Government of Bhutan, through its designated authorities, should commission the creation of a detailed technical guideline for site-specific seismic hazard assessment. This guideline must standardize key inputs and methodologies, including a pre-approved, vetted suite of modern GMPEs appropriate for the Himalayan active shallow crustal tectonic settings and a consistent methodology for using logic trees to systematically account for epistemic uncertainties. In the long run, it will be of vital importance for Bhutan to develop a GMPE model relevant to the region. To realize this, concerted effort should be made by relevant organizations to install strong motion networks in the country and conduct scientific research.

- **Establish a unified national seismotectonic model**

A national level project should be initiated to develop a single, authoritative seismotectonic source model for Bhutan and its surrounding regions. This model developed through a national and international expert would serve as the mandatory baseline for all future site-specific studies, eliminating primary sources of inconsistencies.

- **Mandate periodic hazard re-evaluation**

There should be a policy provision requiring all major hydropower projects to update site-specific seismic assessment based on availability of new information to ensure that seismic design parameters are periodically revisited to incorporate new advances, latest scientific consensus on tectonic processes and updated international best practices and standards.

6.0 CONCLUSION

The comparative analysis carried out in this study confirms that while all reviewed site-specific seismic hazard assessment for major hydropower projects in Bhutan adopts a standard dual-analysis approach employing deterministic and probabilistic methods, there are significant discrepancies in the selection of specific GMPEs and the reconciliation of results reflecting evolution of methodological practice over time. In most cases, a generalized Vs30 value of 760 to 800 m/s were used to characterize the near-surface geology, but the hazard parameters obtained show variation across projects underscoring the site-specific nature of seismic hazard but also highlights the impact of methodological choices on the final design values.

The high hazard levels identified in most of the recent assessment are consistent with the high seismic vulnerability of the Himalayas; however, the divergence between current engineering practice and national guidelines for the most extreme design events for SEE level poses a challenge to ensure a uniform safety standard. To address these constraints, a harmonized national technical standard for site-specific seismic hazard assessment is an essential step.

As evidenced in the detailed, site specific seismic hazard studies for major hydropower projects, Bhutan has demonstrated a proactive approach to ensuring the safety of its hydropower assets. This practice reflects a clear commitment to engineering excellence and public safety that aligns well with international best practices. To further secure Bhutan's hydropower future from geohazard risks, it is important to adopt a strategic framework built on the following: 1) Standardize the seismic hazard assessment methodology through a unified national guideline, 2) Integrate geohazard risks beyond site specific seismic studies through adoption of a basin wide Integrated Geohazards Assessment (IGA) framework that analyzes cascading multi-hazard risks (earthquakes, GLOFs, floods, and landslides) exacerbated by climate change (Reynolds, 2023), and 3) Invest in national data infrastructure and technical capacity to enhance national geohazards resilience and risk reduction. Through implementation of this strategic framework, Bhutan will be in better position to ensure that its hydropower sector remains secure, resilient and sustainable engine for national development for generations to come.

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INNOVATIVE TOOLS FOR SUSTAINABLE DAMS AND HYDROPOWER DEVELOPMENT

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ABSTRACT

A number of tools and processes are available to improve the sustainability profile of hydropower projects and facilities. This paper presents examples of processes that result in more sustainable projects, influence project outcomes and contribute to enhancing relationships with stakeholders. Such processes include basin level or regional Strategic Environmental Assessment (SEA), which assesses policies, plans and programs, ideally before project siting and project level approvals are made. Another example is the Hydropower Sustainability Standard, a certification scheme that assesses the sustainability performance at the project or facility level and can be applied to projects in the preparation, construction and operation stages. Finally, the paper will address more specific processes embodied in Technical Guidance or Technical Bulletins on emergency management, public safety around dams and other topics developed by entities such as ICOLD and the Canadian Dam Association, which emphasize processes that lead to community resilience.

1. INTRODUCTION

A number of international standards have been used and continue to be used in the hydropower to ensure good practice with respect to environmental, social and governance practices. These tools such as the World Bank Group's or the Asian Development Bank's Environment and Social Framework and Environmental and Social Standards (ESS), IFC's Performance Standards, used widely around the world and across sectors. Sector specific entities such as ICOLD and member country entities (such as the Canadian Dam Association), IHA and HSA have been instrumental at developing guidelines and standards that are specific to the dam and hydropower sectors. When used together these tools lead to better projects and more positive outcomes from an environmental, social, governance and infrastructure and dam safety points of view. This paper will describe these tools and provide references for further information, as well as some of the results of the implementation from such tools.

2. STRATEGIC ENVIRONMENTAL ASSESSMENT

Strategic Environmental Assessment (SEA) aims to improve strategic decision-making on the sustainability of government policies, plans and programs. It differs from Environmental Impact Assessment which is a project-level planning and approval process.

By their nature, hydropower projects are developed on river systems and at times, in clusters. As a result, they can have wide scale impacts on communities and ecosystems. Planning and management of hydropower projects is therefore best carried out at the watershed level, rather than individually or in isolation. SEA allows for a basin-wide view of cumulative impacts on ecosystems, environmental flows, sediment transport, biodiversity and community and human well-being, which cannot be done through project level environmental impact assessment (EIA). SEA can help align both governments and individual hydropower project developers in multiple ways:

- Water resource sustainability: SEA aligns hydropower planning with integrated water resource management (IWRM) and basin management plans ensuring that ecological flows and downstream needs are maintained.
- Promotes multipurpose hydro development: SEA supports the integration and optimization of hydropower with other types of power generation or multi-purpose reservoir uses (e.g. irrigation or flood control).
- Strategic siting and sequencing: SEA identifies which river stretches or regions are most or least suitable for hydropower development, reducing environmental and social conflict with other land, resource and/or water uses.
- Promotion of biodiversity conservation: SEA assesses risks and impacts to biodiversity to ensure no net loss and ideally net gain, including design offsets and protected areas strategically, e.g., conserving intact upstream ecosystems while developing lower-priority areas.
- Alignment with national energy strategies: SEA ensures that hydropower supports a diversified, low-carbon energy transition and complements other renewable sources, such as wind and solar.
- Integrates consideration of climate change: SEA identifies potential climate issues for both short term and long-term hydropower development and proposes mitigation and adaptation strategies.
- Institutional capacity building: SEA identifies capacity gaps in the management of environmental flows, resettlement, and regulatory oversight, and recommends strategic investments in institutions. It can help improve governance, inter-institutional coordination and decision making in the hydropower sector.
- SEA streamlines individual hydropower project EIAs: By considering environmental and social issues at a strategic watershed level, SEA can provide an improved planning context for the assessment and approval at the project EIA level.
- Improved dialogue around hydropower: SEA brings people and diverse interests together in a conversation about hydropower development and benefit sharing. It can improve social acceptability and improve public trust and confidence in decisions relating to hydropower development.
- Reduction of conflict and delays: SEA can incorporate stakeholder concerns and mitigate disinformation early on to minimize opposition, legal challenges, or community unrest, which are common risks in large hydropower projects.
- Finally, SEA can provide critical information to support better decision-making for hydropower master plans, including where there might be significant environmental and/or socio-economic risks across the watershed.

In summary, if done early and coordinated together with project level EIA, SEA can assist both governments and hydropower project developers to promote top-down planning from the policy, plan and program levels with bottom-up planning from the individual hydropower project level. This nexus between these two planning processes can lead to enhanced environmental and social sustainability balancing multiple interests across an entire watershed, promoting standardization of operating procedures for hydropower operators and creating financial certainty for lenders and investors.

3. HYDROPOWER SUSTAINABILITY STANDARD AND OTHER TOOLS

The International Hydropower Association (IHA) released hydropower-specific sustainability guidelines in 2004 and the Hydropower Sustainability Assessment Protocol (HSAP) in 2010 and the environmental, social and governance (ESG) gap analysis tool in 2018. These tools describing good and best industry practice have been used by organizations worldwide to guide the sector towards more sustainable outcomes, covering a wide range of sustainability topics relevant to the hydropower industry. One of the strengths of these tools lies in their development by a multi-stakeholder forum which integrated all stakeholder voices including industry, civil society, governments and financial institutions. The need for a sector-wide sustainability certification and labeling scheme led to the development of the Hydropower Sustainability Standard (HSS) and the creation of its independent management body, the non-profit Hydropower Sustainability Alliance (HSA) governed by a council with representation from governments, industry, civil society and financiers. The HSA manages the HSS and certification process and operates a training academy aimed at fostering "healthy ecosystems, prosperous communities, resilient infrastructure, and good governance".

Since the Standard's inception in 2020, HSS assessments have been carried out worldwide with various objectives, some leading to certification, some leading entities on a sustainability journey to improve internal practices and relationships with external stakeholders. The standard covers 12 ESG topics with hydropower-specific requirements over two levels of performance, good practice (minimum requirements) and best practice (advanced requirements).

Countless organizations have used the suite of hydropower sustainability tools with different objectives in mind: to build internal capacity, to improve the ESG performance of their projects, for project finance, to better demonstrate sustainability of their operations to stakeholders, financiers or electricity customers. The tools have also been endorsed and used by multilateral financing entities to benchmark ESG performance in the sector in various regions and jurisdictions. The tools help organizations and financiers focus improvement efforts where basic good practice and best practice gaps are.

These tools have been used in the Himalayan region since their inception with uptake over the years in India, Nepal and Bhutan and include the official and unofficial application of HSAP, the ESG data gap analysis tool and now the Standard. The HSAP was used in Bhutan in 2016 on the Mangdecchu Hydropower Project early in the construction stage with the support of the World Bank, and also on numerous projects in India, and Nepal. The ESG Tool has been used to assess operating projects in due diligence for potential asset acquisitions, and the Standard has been applied on a number of projects across the region, with many having achieved certification (official Hydropower Sustainability Standard assessment reports are published on the HSA's website). An example of an HSS results pie chart diagram is shown in Figure 1 below, showing all 12 sections of the Standard. This particular example is from Hydro-Quebec's Eastmain-1 Hydroelectric Development in Quebec, the first (and only) project in Canada to undergo an HSS assessment, in 2023. 10 years prior, Manitoba Hydro's Keeyask project in northern Manitoba had carried out an HSAP assessment (2013). Both assessments highlighted how partnerships with indigenous communities improve the outcomes of hydropower projects by including them in the planning process, environmental and social monitoring activities and more. HSS assessment results and action plans included in the assessment reports can help focus on improvements that are achievable. The use of these tools has led to many companies and countries updating or broadening their policies. Over the past few years, the HSA has been building capacity and deploying "user-certified" and "accredited assessor" training in Nepal and as a result many assessments have been carried out on projects in Nepal that are under development or in operation.

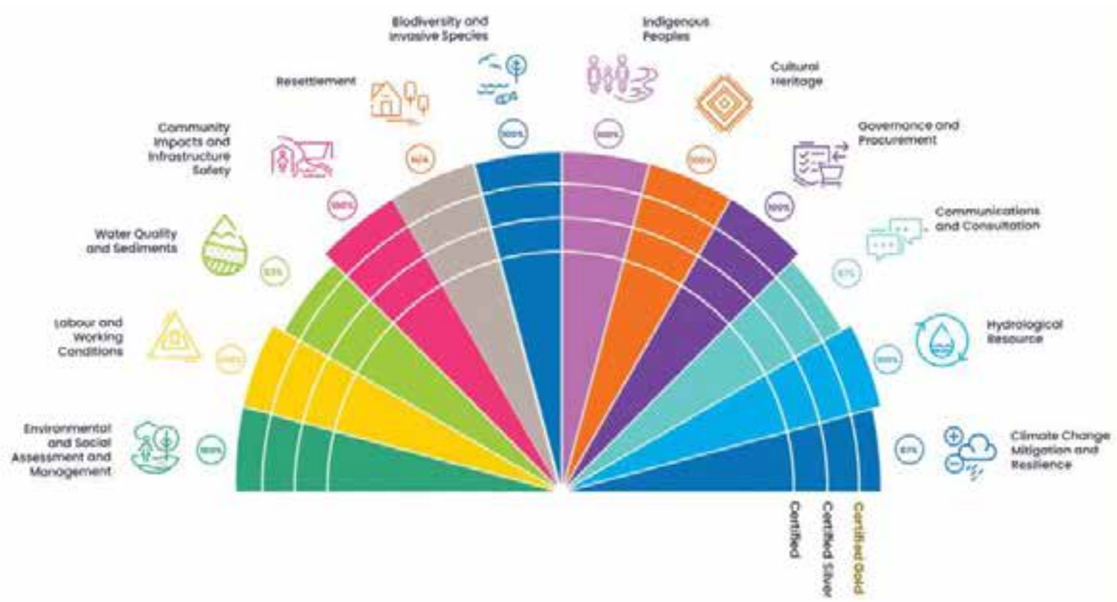

Fig. 1 : Results Diagram in an Assessment currently published for public consultation.
Source: Eastmain 1 Hydroelectric Development, HSS Assessment Report, 2023.

Figure 1 above provides a snapshot in time of a project in the preparation, implementation (construction) or operations stage and Certifications must be renewed periodically to ensure ongoing performance.

In addition to these individual project assessment tools, and similar to the SEA process described in the previous section, the HSA has also developed an early planning stage decision support tool: "HydroSelect", that can help assess different hydropower project options being considered, or to assist in the identification of risks of a single hydropower project, in the effort to build better, more sustainable hydropower and to ensure that all new hydropower projects are sustainable hydropower projects.

HydroSelect produces a heat map type table that enables early decision making with respect to hydropower project options or individual projects and is available for download on-line for free and is a welcome addition to the suite of HSA tools. Figure 2 below shows where this tool fits in the development process.

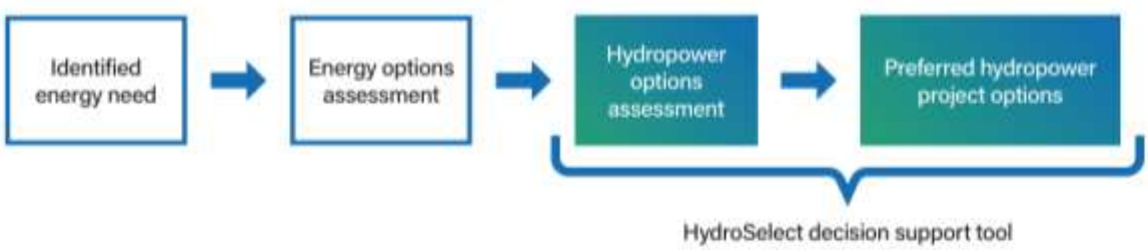

Fig. 2 : Where HydroSelect fits in the early stage energy planning sequence
Source: Hydropower Sustainability Alliance, HydroSelect, The Hydropower Sustainability Early Stage Decision-making Tool, 2025.

4. INTERNATIONAL GUIDANCE FOR GOOD PRACTICE

4.1 International Commission on Large Dams (ICOLD)

International organizations such as the International Commission on Large Dams (ICOLD) have long focused on establishing guidance on practices of dam engineering. They have produced over 100 Bulletins developed by technical committees with representatives from many countries. In recent years, ICOLD has increasingly moved beyond the engineering/technical focus to address dam safety, public safety, and sustainability of dams.

In May 2025 at the 28th Congress in Chengdu, China, ICOLD approved a Declaration on Dams and Reservoirs for Energy transition and Adaptation to Climate Change. Prior declarations have addressed Dam Safety (2019), Water storage for Sustainable development (2012), and Dams and Hydropower for African Sustainable Development (2008).

In 2023, ICOLD launched a Dams and Sustainability (D&S) initiative to bring together the efforts of several technical committees with the main objective of raising awareness among dam professionals about the need to enhance sustainability and life cycle considerations for existing and new dams. The group has organised two webinars. The first took place in May 2024 and presented the main sustainability goals and objectives of this new ICOLD initiative. The second webinar was held in May 2025, introducing Life Cycle Sustainability Assessment (LCSA) methods (ICOLD 2025). Both webinars are available on a link from the ICOLD website. A third webinar #3 will focus on calculating greenhouse gas (GHG) emissions throughout the life cycle of dams. (ICOLD 2025)

Thus, membership in ICOLD brings benefits in terms of access to guidance publications, participation in development of that guidance, and networking opportunities for sharing of knowledge among countries or regions with similar challenges. Information about ICOLD, its bulletins and committee initiatives is available on the website icold-cigb.org.

Many of the national committees of ICOLD produce guidance that has a particular focus on the conditions of their countries. Generally, the principles are similar and there is much to be learned from looking at the different approaches. Excellent guidance publications are available from several national committees. This paper will discuss some examples the authors are most familiar with – from the Canadian Dam Association.

4.2 Canadian Dam Association (CDA)

The Canadian Dam Association (CDA) was formed as a non-governmental not-for-profit organization to provide a national perspective and guidance for the dam community, since there is no federal regulation of dam safety in Canada. Rather, dam safety falls under the jurisdiction of the 10 provinces and 3 territories, resulting in different regulations in different provinces. Furthermore, the Province of Quebec has a different legal system from the other provinces. Some dams must meet the requirements of more than one province and legal system. CDA first published *Dam Safety Guidelines* and related Technical Bulletins in 1995, revisions in 1995, 2007 and 2013. The Guidelines have become the common reference and guide to good practice in Canada, with wide acceptance by owners, regulators and consultants for both water retaining and tailings dams. Another revision to the Guidelines is currently in progress. An interesting aspect of the CDA guidance is that it is produced through extensive consultation in order to reach a broad consensus within the dam community.

Looking beyond the engineering design and construction aspects of dams, CDA is also seeking approaches and methods to enhance the sustainability, environmental and social impacts of dam developments. The Dam Safety Guidelines include environmental and cultural values in the dam classification used in dam safety assessments. The method for Environmental Consequence Classification that is currently applied to tailings dams, is being updated to provide a systematic and evidence-based methodology to assess the environmental consequences that could result from breach of a water dam or tailings facility.

As further examples to illustrate the interface between dam engineering and sustainable development of hydropower, two other CDA publications are discussed below.

In 2011, CDA published *Guidelines for Public Safety Around Dams* after extensive work carried out by Ontario Power Generation (OPG) to develop innovative procedures for management of public safety around its dams (Bennett, pers. comm.) The CDA publication outlines a detailed methodology for assessing the safety risks around dams, focusing on hydraulic behaviour, environmental conditions, human behaviour, and design limitations. Tools are provided, including a set of spreadsheets that take the assessor through a methodical and comprehensive assessment of risks to the public, and then document corresponding risk reduction measures. The CDA methodology is presented in 1-2 workshops for managers, engineers, and those who carry out public safety risk assessment. Figure 1 is an illustration from the CDA workshop.

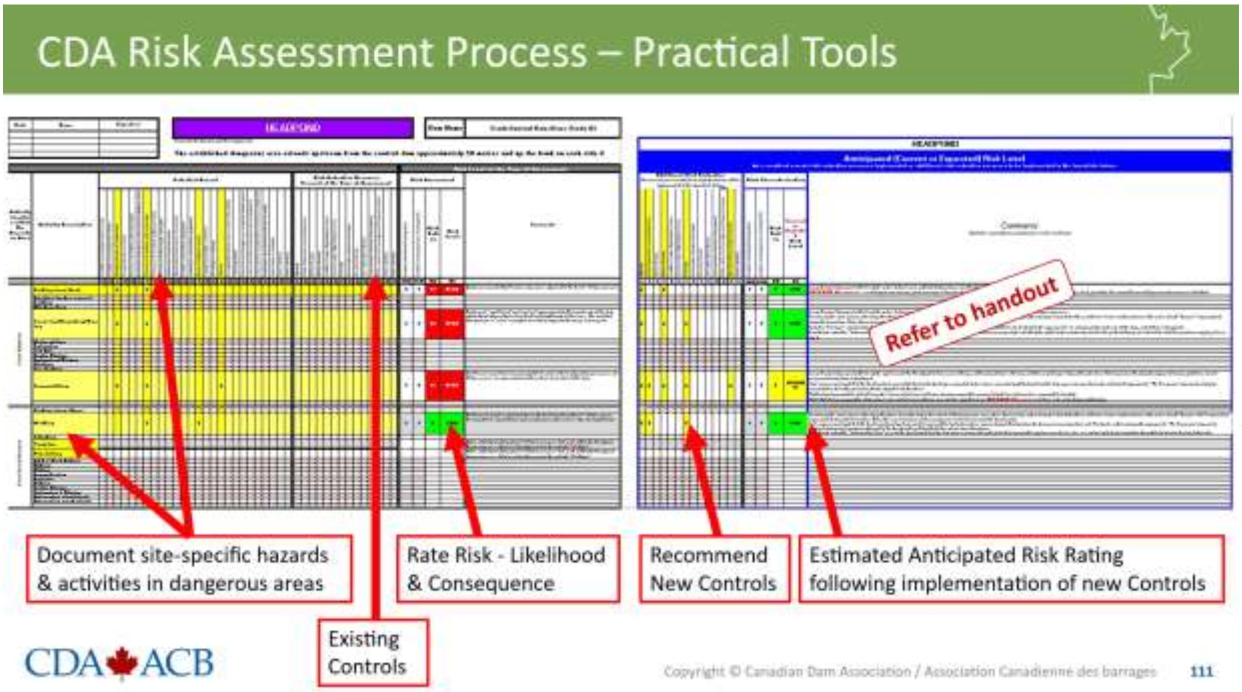

Fig. 3 : Illustration of CDA Process for Public Safety Risk Assessment

Effective safety management requires an integrated approach combining public education, clear signage, engineering design of barriers and enforcement. Strategies such as physical barriers, exclusion zones, automated warning systems, and targeted awareness campaigns are evaluated for their effectiveness in reducing risk. The role of community engagement, especially with first responders who may be called upon to perform rescues in dangerous waters, is also emphasized as essential for sustainable safety outcomes.

By improving public understanding of how dams and weirs operate and the dangers they present, dam owners can take effective actions to reduce accidents, allowing members of the public to make informed choices and reduce exposure to life-threatening situations. There is a need for ongoing risk assessments, incident reporting and consistent safety standards to protect the public while maintaining the essential functions of these vital water infrastructure assets.

Since publication of Guidelines for *Public Safety Around Dams* and dissemination through CDA workshops and other programs, the number of fatalities around dams in Canada has been reduced significantly. Figure 2 shows the reduction from an average of about 6 fatalities per year to only 2, within a year or two after the CDA guidelines were published and generally adopted as good practice for dam owners in Canada,

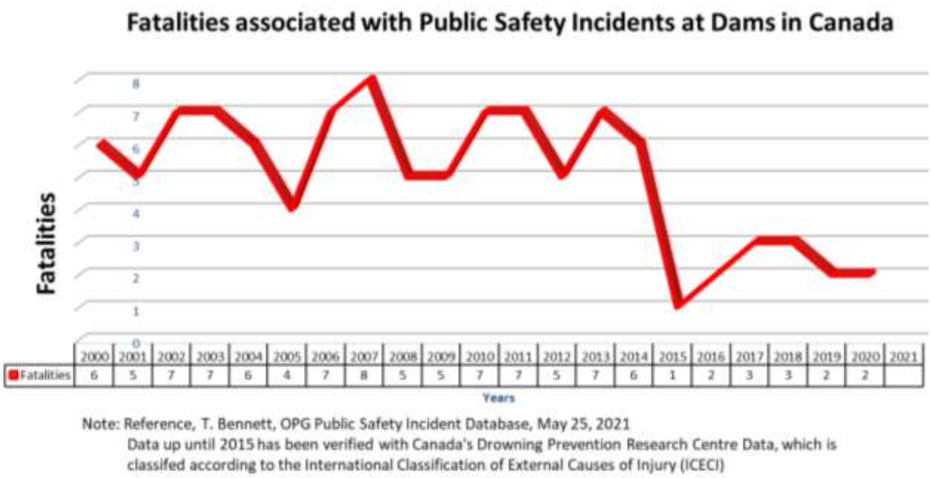

Fig. 4

Another CDA publication with particular relevance to sustainability of dams and hydropower facilities, is the Technical Bulletin on Emergency Management for Dam Safety. For many years, dam safety engineering guidance has included development of flood and dam failure inundation maps and reference to the need for dam owners and affected communities to have emergency response plans in place. The CDA publication takes the subject a step further by treating emergency management for dam safety with a view to all phases of emergency management – from preparedness through risk assessment and implementation of controls, to coordinated response actions by both the dam owner and the affected parties, and finally to recovery from dam safety events. This approach includes ongoing engagement between the dam owner, the responsible response agencies, and the affected communities. This collaboration through transparent planning and public participation enhances the development of trust, and it leads to improved community resilience.

5. CONCLUSIONS

In this paper, the authors have made reference to several sources of information and guidance on tools and approaches to increasing the sustainability of dams and hydropower developments. These are examples of innovations that can be adapted and applied to developments around the world. The organizations that produce the guidance are continuing to develop tools and improve understanding of the potential implications of hydropower development.

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MAJOR SHEAR ZONE IN DAM FOUNDATION - A CASE STUDY OF THE PUNATSANGCHHU-II (1020MW), HYDROELECTRIC PROJECT, BHUTAN

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ABSTRACT

The Punatsangchhu–II Hydroelectric Project (PHEP-II) is a run-of-the-river project that includes the construction of a concrete dam with a height of 91 m and a length of 224.90 m across the Punatsangchhu River. The dam axis is oriented at a fore bearing of N39°E. The riverbed level at the dam site is 784.0 m, and the dam top level is 846.0 m. The dam and intake structures are situated downstream of Uma village on the right bank of the Punatsangchhu River, along the Wangdue–Tsirang National Highway in Wangdue Dzongkhag. All major project components are located on the right bank of the river, except the diversion tunnel.

The Dam foundation excavation started in 2016. Geologically, the dam site lies within the gneissic terrain of the Thimphu Group, which comprises quartzo-feldspathic gneiss and biotite gneiss, with intrusions of leucogranite and pegmatite. These rock units belong to the Sure Formation and are of Proterozoic age. During the excavation of the foundation of dam, a major shear zone was encountered in overflow blocks foundation.

The major shear zone exposed during excavation exhibited variable thickness and orientation. The affected zone ranged from 3.5 m to 5.6 m in width and consisted of clay gouge with crushed rock. The shear zone was oriented obliquely to the river flow direction, extending from upstream to downstream (heel to toe of the dam). It was bounded by two prominent shears one on the left boundary and other on the right boundary. The paper presents the detailed account of various remedial measures taken for treatment of shear zone in the Dam Foundation like confirmatory drill holes within the shear zone area, preparation of geological sections to delineate the shear zone disposition, trench excavation and placement of PCC concrete, followed by RCC raft with heavy reinforcement, anchorage of foundation rock by providing fully grouted rock anchors and consolidation grouting in shear zone area.

***Keywords**: Excavation, Coffer dam, Shear Zone, Reinforcement, Grouting*

1. INTRODUCTION

Punatsangchhu–II Hydroelectric Project (PHEP-II) is a run-of-the river scheme which envisages construction of a 91m height and 224.5m length concrete gravity dam across Punatsangchhu River at about 2 km downstream of Tail Race Tunnel (TRT) outfall of Punatsangchhu-I Hydroelectric Project (Fig.1). The dam and intakes are located below the Uma village on the right bank of Punatsangchhu river along the Wangdue-Tsirang national highway. The water from Dam Reservoir diverted through a 8.5 km long 11m diameter Head Race Tunnel (HRT) to an underground power house located near Kamichhu village to generate 1020MW (6X170 MW) of power.

1.1 Location and Accessibility

All modes of transport within Bhutan are mostly by road. The motorable roads are well maintained and connected with most places. The International Airport is located in Paro and Thimphu, the capital town of Bhutan is located 55km away from Paro. From Thimphu to Wangdue Phodrang, it takes two hours to cover 76km by road. The Punatsangchhu-II Hydroelectric Project is located about 20 km downstream of Wangdue Phodrang Bridge.

1.2 General Layout Plan and layout of the dam complex of Punatsangchhu–II Hydroelectric Project

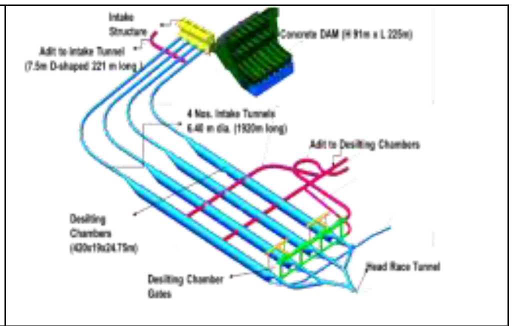


Fig2. General layout of the Punatsangchhu–II Hydroelectric Project	Fig3. General layout of the dam complex of Punatsangchhu–II Hydroelectric Project

1.3 Regional and Dam Site Geology

Regionally the PHEP-II area is located within a part of the Tethyan Belt of Bhutan Himalayas and at the dam site, rocks of Sure Formation of Thimphu Group of Precambrian age are exposed *(Bhargava, O.N. 1995).* The rocks of Thimphu Group in general is characterized by coarse-grained quartzo feldspathic biotite gneiss with bands of mica schist, quartzite and concordant veins of foliated leucogranite. The bedrock exposed in the project area (reservoir and dam) is represented by garnetiferous, biotite bearing quartzo feldspathic gneiss showing a general foliation trend N10°E to N40°E and dips 20° to 40° towards ESE to SE and at places (Fig 4).

Greater Himalayan Zone

GHlmu: Upper metasedimentary Unit (Neoproterozoic-Ordovician(?)- Variable metamorphic grade: dominantly amphibolite facies(Gansser,1983), quartzite and biotite-muscovite-garnet schistin central west-central Bhutan.

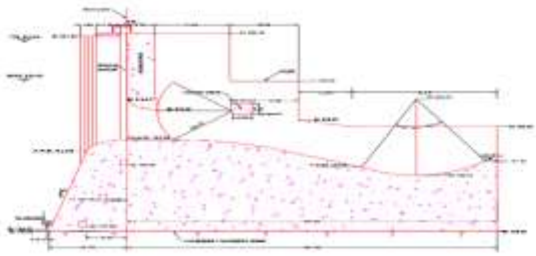
Ghlo: Orthogneiss Unit (Cambrian-Ordovician)- Cliff- forming, massive-weathering, granite-composition orthogneiss: generally exhibits leucosomes and abundant feldspar augen.

GHlml: Lower metasedimentary Unit(Neoproterozoic-Cambrian(?)- Dominantly amphibolite-facies(Gansser,1983: Grujic et al., 2002: Daniel et al, 2003) metasedimentary rocks, including quartzite, and biotite-muscovite-garnet schist.

Fig 4. Regional and dam site geological overview of the Punatsangchhu–II Hydroelectric Project

1.4 Key features of dam

Type	Concrete Gravity Dam
Top of Dam	El 846 m
Length of Dam (top)	224.5 m
Max. height of Dam	91 m
MDDL	EL. 825 m
FRL	EL. 843 m

Gate Crest Level	EL. 797.0 m	*View of Dam*
Gross Storage Capacity	7.0 MCM	
Live Storage Capacity	4.64 MCM	
Design Flood & GLOF	11723 m³/s PMF + 4300 m³/s	
Type & Nos.	Radial Gate,7 Nos.	
Size of Gate	Width 8 m x Height 13.2	*Sluice Section*

2. DAM FOUNDATION EXCAVATION

During the excavation of the dam foundation, the foundation mapping was carried out on 1:100 scale of each overflow and non- overflow spillway blocks to identify any adverse geological features and suggest proper remedial measures before laying the concrete. These are the main geological steps taken during construction of the spillway.

a. Geological foundation mapping
b. Pre and Post Water Permeability tests
c. Consolidation Grouting and
d. Curtain Grouting

2.1Excavation methodology

The construction of 91m high & 224.5m long concrete gravity dam started with preparation of approach road to the top of the excavation profile as per construction drawing. Works for the excavation were done by conventional excavation method which involves drilling and blasting. Pre-splitting blast was carried out first followed by the conventional blasting which breaks the unwanted rock mass. Bench were lowered by at least 3m before the slope stabilization works (Photo 5(a) & 5(b).

	
Photo 5 (a) Excavation of dam left bank	Photo 5 (b) Excavation of dam left bank

2.2 Foundation Mapping and geotechnical assessment of Dam

The foundation mapping of overflow blocks, 4 to 10 (Photo 10), left non-overflow blocks (1 to 3) and right non-overflow blocks (11 to 12) were carried out on various scales 1:100/200 scale. The bearing of the dam axis is S39°W (left to right abutments). The excavation for dam foundation was carried out upto various levels as per designed level. The original ground level was ±784m and the average excavation foundation levels were excavated between block 1 and 3 was RL ±765m, 4 to 6 was upto RL ±752m (Shear zone blocks) and between 7 and 10 was at RL ±755m (Photo 6).The excavation of dam about 29m deep excavated from the OGL ±784m. The mapped area is predominantly occupied with fresh to slightly weathered (W_0 - W_1) Quartzo Feldspathic Biotite Gneiss (QFBG) with minor pegmatite and leucogranite intrusions (Photo 9 & 10).

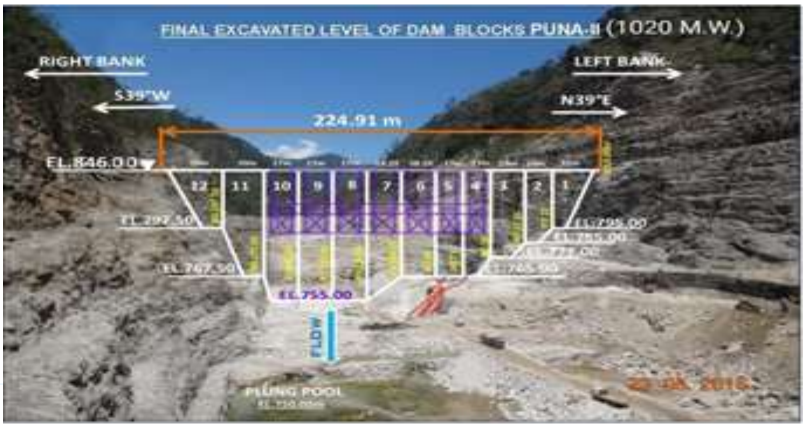


Photo 6: Final excavation level of dam blocks of PHEP-II

The foundation grade rock except block 4 to 7 was devoid of any major shear/weak zones, however, minor staining/weathering (W_1), along joint planes and pegmatite veins were noticed. The rock mass was intersected by four set of prominent joints and two set of random joints. The recorded joint sets were 15^0-20^0/N070^0- N110^0; foliation joint, 55^0-80^0/N230^0 - N260^0. Variation in dip and direction of foliation joints is due to swirling and warping. Rock is fresh (W_0), hard (as per field estimates strong to very strong; R_4 to R_5 Grade) category. The UCS value (as measured in the field by blow of geological hammer) of rock would be 100 to 200Mpa.The rock mass qualities and classifications of the dam site is assigned using the rock mass rating (RMR). As per RMR it was classified as fair to good rock mass (RMR = 55 - 75).

Photo.7
Right flank of the dam showing Intake and drift at different El.

Photo.8
Construction & excavation in Dam Pit area

Photo.9
Rock type Quartzitic feldspathic gneiss (QFG) exposed in the Dam foundation

Photo.10
Dam concreting work of Overflow block nos.4 to 10

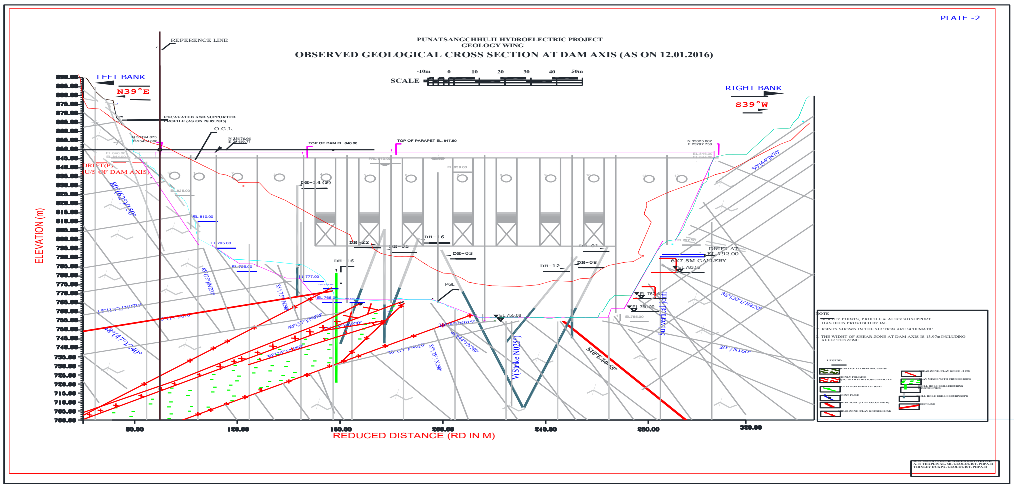


Fig 5. Geological cross section along dam axis showing subsurface strata, joint attributes, shear zones.

2.3 Rock support systems installed for Dam abutments

Rock Anchor: Fully cement grouted rock anchor of 32mm dia & 7.5m long were placed at 1.75m c/c. 32mm dia cement grouted rock anchors of length 12m were also inserted at certain places.For places where rock was not encountered as anticipated in DPR, cable anchor was provided, 7 layers of 100 Ton capacity cable anchor of 30m long at 2m c/c were placed (Photo 11).

Shotcrete: 150mm thick plain shotcrete with mix proportion of M25A10 grade was used. Shotcrete was not done on the slope area forming the part of Dam Abutment, instead, chain link fabric was installed, which was later removed during Dam Concreting (Photo 12).

		
Photo 11. Cable anchors installed at dam left bank.	Photo 12. Left Abutment Drift at U/s El. 843m	Photo 13. Left Abutment treated with Wire mesh shotcrete

Chain Link Fabric: In addition to shotcrete and rock anchors, chain link fabric wire mesh was also provided in the excavated surfaces to protect surfaces from which loose pieces of rock or cobbles may fall during benching down (Photo 13). Galvanized steel chain fence fabric of size 50mm x 50mm with wire diameter of 3mm was used to stabilize the excavated surface. Steel plates of size 100 x 100 x 5mm with nuts were used to fasten the fabric with the rock anchors.

3.SHEAR ZONE ENCOUNTERED DURING DAM FOUNDATION EXCAVATION

Shear Zone at the Dam overflow blocks

A thick shear zone containing clay mixed leucogranite and pulverized pegmatite rock was encountered at the foundation of the dam block 4 and continues in block 6 and 7 from upstream to downstream direction at El.765m (Photo 14&15). The top and bottom contacts of the shears zone having variable orientation i.e. 20^0-55^0/N070^0-100^0. This zone is bounded by two major shears one on left side boundary and other on right side). The width of this zone varies from ±7m, at u/s side near dam axis, to ±2m at d/s side i.e. near the toe of the dam.

	
Photo 14. Disposition of shear zone in the dam block no. 5.	Photo 15. U/s view of shear zone in the left abutment dam foundation block no. 6 to7

3.1 Geological Investigations carried out to assess the extent of shear zone

i. Geological foundation mappings were carried out in different stages of excavation as the attitude of shear zone changed abruptly. Based on the site inputs and design requirement for the treatment of shear zone, excavation to the extent of 74m width on the upstream and minimum 20m on the downstream Dam blocks to a depth of 13.5m to 18.5m was carried out as per design requirement. The top and bottom contacts of the shears zone having variable orientation i.e. 20^0-55^0/N070^0-100^0. The thickness of this zone also varies from 7m at the dam axis to 2m at 90m downstream i.e. near the toe (106m d/s) of the dam. The problems anticipated for this zone related to stability problem of the dam and secondly seepage /leakage problem below the foundation of the dam.

ii. A longitudinal section along center line of the dam axis and cross sections at 20m interval were developed from the geological plan, which indicate that the shear zone pass as below the block no. 5 and its depth will increase towards d/s side. The profile has been developed from the spot height shown at the plan and sections are developed as per actual orientation of the shear zones measured at particular places. The excavation of shear zone was started from EL765m to EL752m although, excavation quantity of the shear zone is very less i.e.56000 cum only but more than 5 months' time was taken to excavate as it was carefully carried out with and without blasting in phased manner as per design requirement.

iii. Exploratory Drilling: After the geological mapping four nos. of exploratory holes (DH-13 to DH-16) were drilled (Fig. 6) at different locations to confirm actual depth/thickness and sub-surface behavior of shear zone. The disposition/depth of the shear/weak zone in the dam block nos. 5, 6 and 7 was delineated on the basis of drilling.

iv. Drill core log abstracts were prepared for all the four holes and projected sections were made (Fig. 7,8 &9). Shear material mainly comprises highly pulverized rock mass, granular rock flour, fractured rock mass, clay gouge with slush, and broken rock fragments. The drill core logging revealed that the shear zone depth varies from 3.1m to 11.22m and affected/fractured rock mass zone below the shear zone varies from 6.0m to 20.0m in depth.

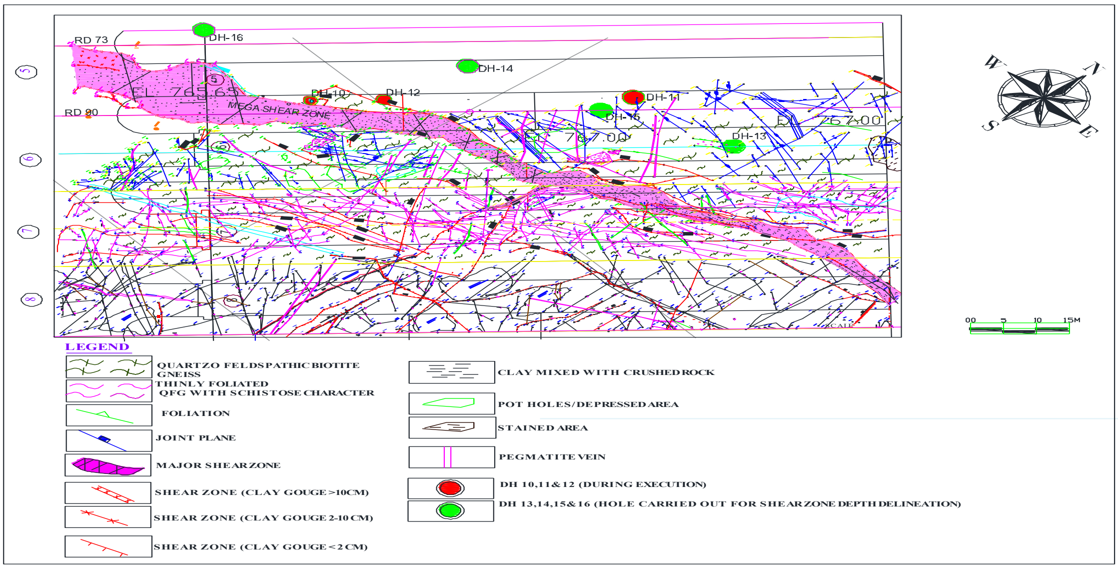


Fig. 6 Geological plan of shear zone (EL 765.0m) and adjacent blocks.

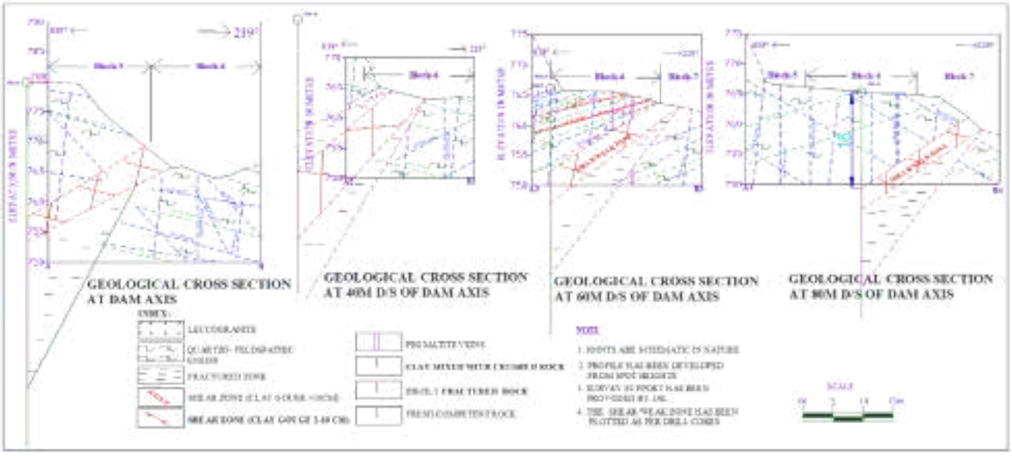


Fig. 7 Geological cross sections depicting shear zone projection at Dam Axis, at CH 40.0m, 60.0m and 80.0m D/s of Dam Axis.

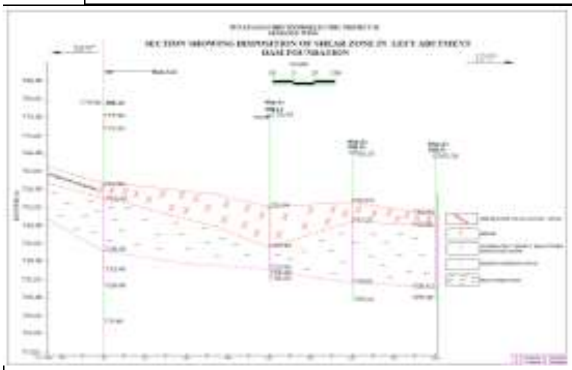


Fig 8. Geological section at dam foundation showing orientation of the shear zone

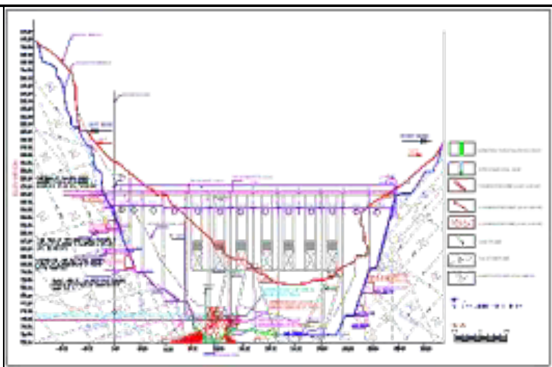


Fig 9. Prepared geological section at dam axis showing orientation of the shear zone

3.2 Treatment of Shear Zone was worked out to mitigate for anticipated problems during dam construction

Stability problem of the dam: Settlement of the dam foundation along the shear zone is anticipated due to poor bearing capacity of the sheared material and high load of the concrete dam. Dental treatment i.e. V-shaped excavation of the weak zone equal to the depth of thrice the width of the zone will be required to transmit the load of dam on the hard rock.

Seepage /leakage problem: Seepage /leakage problem below the foundation of the dam was anticipated as the shear zone will act as shorter avenue for leakage. As it was suggested for the depth of the cut off may be worked out and to decipher the nature (thickness, orientation, continuity etc.) of the shear zone/weak zone.

3.3 Treatment of the Shear zone

3.3.1 Trench excavation and placement of concrete:

The bottom of the shear zone area was treated with Plain Cement Concrete layer of about 6.2m to 11.5m depth followed by the Consolidation and Contact grouting. Further the top of the shear zone was treated with the heavily reinforced concrete raft of 6.8m thick (Fig. 10). Total of 56,000 M^3 of concreting and 2250 MT of steel reinforcement was provided. Therefore, a total of 679 days were spent for excavation and treatment of shear zone areas. However, in between overtopping of upstream Coffer Dam took place during 2016 and 2017 monsoons period which has affected the work plan of excavation and treatment works of shear zone areas. After cleaning of these blocks, foundation treatment like dental treatment, rock covering & consolidation grouting were taken up for execution. After that, main concreting work was taken up.

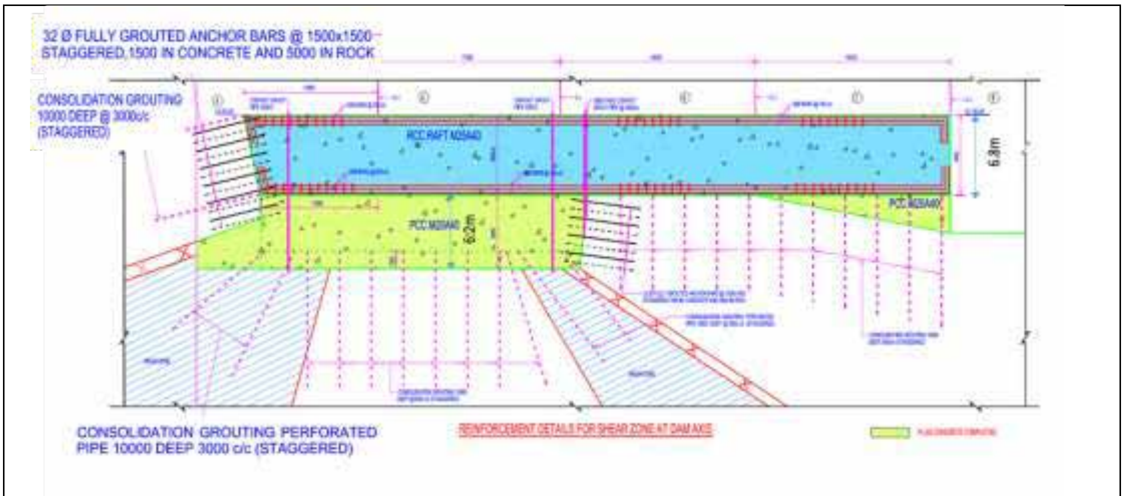


Fig 10. Reinforcement details for shear zone at dam (source: WAPCOS/CWC: Design Drawings of Shear Zone

Photo 16.
Shear Zone trench excavation

Photo 17.
Rock anchor support on trench walls

Photo 18. Reinforcement in shear zone at dam	Photo 19. PCC in shear zone at dam foundation

Table 2. Details of shear zone treatment

S.No.	Particular's	Quantity	Instruments installed in Shear Zone	Block	Elevation
1	Excavation	53000 M^3	MPBX	4 & 5	752.0m
2	Consolidation Grouting	428 Nos.	Piezometer	4,5,6&7	752.0m
3	Cement grout	161.75 MT			
4	32mm dia rock anchors	12209			
5	Concrete	56000 M^3			
6	36mm and 20mm dia Steel reinforcement	2250 MT			

3.3.2 Anchorage of concrete with foundation rock

The RCC Raft was anchored with 32 ∅ fully grouted anchor bars @ 1500x1500 staggered,1500 in concrete and 5000 in rock.

3.3.3 Consolidation grouting

Grouting consists of drilling series of grout holes and injecting grout under pressure, which eventually sets joints and voids in the rock. Grouting can be carried out to full depth either in one operation or in successive depths by stage grouting. Full depth grouting is generally preferred in short holes up to 10m having small cracks and joints. Stage grouting is done in the surface where there is large variation of rock discontinuities. It permits the treatment of different zones individually either in ascending or descending order. A total of 504 Nos. of grout holes were grouted with a cement water ratio in various depths.

Photo 20. Anchoring on both wall of the trench	Photo 21. Consolidation grouting in the shear zone

Fig 22. MPBX instrument installed in the block nos. 4 & 5.

3.3.4 Seepage and settlement monitoring

To assess the behavior, stability of the treated shear zone, instrumentation was installed to monitor the settlements, strain, stresses and displacements in dam foundations, well-planned networks of instruments were installed in dam galleries and are being monitored regularly. Even after filling of reservoir, there is no abnormal behavior noticed. Seepage is also negligible in foundation galleries.

4. CONCLUSIONS

- The construction of major dams in the Himalayan region demands a foundation free from adverse geological conditions such as shear zones, weak zones, and other adverse structural features.
- A stable geological foundation is crucial for maintaining project schedules and minimizing cost overruns. However, when such geological complexities arise during construction, they pose significant challenges that must be addressed through appropriate engineering and construction methodologies. The excavation, cleaning, and treatment required for these issues extended the project timeline by nearly two years. The shear zone posed potential stability concerns due to its low bearing capacity relative to the significant loads imposed by the concrete dam.
- Thorough understanding of the subsurface geological conditions during the preliminary investigation stages is crucial. Adequate and accurate investigations enable the design of safe and stable structures, reduce construction uncertainties, and help prevent unexpected project delays and cost escalations.

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DESIGN AND GEOTECHNICAL CHALLENGES FOR SEEPAGE CONTROL IN POWER HOUSE – A CASE STUDY OF 1020 MW PUNATSANGCHHU HYDROELECTRIC PROJECT, BHUTAN

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ABSTRACT

The Punatsangchhu–II Hydroelectric Project (PHEP-II) is a run-of-the-river hydroelectric project that includes a 91 m high concrete gravity dam across the Punatsangchhu River and an Underground powerhouse with an installed capacity of 1020 MW located on the left abutment of the Kamichu Nala, a right-bank tributary of the Punatsangchhu River in Wangdue, Bhutan. The Underground Powerhouse Complex includes three large caverns: the Power House cavern (241 m(L) × 23.5 m(W) × 51 m(H)), Transformer cavern (216 m × 14 m × 26.5 m) and Downstream Surge Chamber cavern (314 m × 19.4 m × 58.5 m). All project components like Head Race Tunnel (HRT), Surge Shaft (SS), Butterfly Valve Chamber (BVC), Pressure Shaft (PS), Downstream Surge Chambers (DSSC), Additional Surge Tunnels (ASTs), and Tail Race Tunnel (TRT) were completed and thoroughly inspected by experts. Initial filling of the Water Conducting System (WCS) began on 27 June 2024 and was completed on 16 July 2024 up to elevation of EL 825.0 m, enabling mechanical spinning of Units I & II on 15 August 2024.

After reaching the Surge Shaft filling level at MDDL, minor seepage was first observed in the upstream wall (D-Line) of the Machine Hall, gradually extending to the AHU crown area and Auxiliary Bay. Additional seepage locations subsequently developed along the upstream wall of the Power House from the MIV floor to the crown as well as in the top adit to the Machine Hall and Transformer Hall. Within few days, seepage had extended to the entire length of the Machine Hall, with the volume of dripping water increasing exponentially in certain areas. The dripping water from cavern walls on multiple E&M components, including control panels, UPS, stator, and rotor assemblies, caused significant disruption to ongoing works.

Several remedial measures were taken to arrest the seepage in Machine Hall including complete dewatering of the HRT, Surge Shaft, and Pressure Shaft, inspection to identify seepage sources, drilling of drainage holes ranging from 35 m to 130 m in length from the adit to pressure shaft bottom towards the Machine Hall crown, grouting at the bottom of the Surge Shaft, Penetron coating in the HRT and Surge Shaft bottom and walls etc.

The primary objective of this paper is to present the seepage challenges encountered in the Machine Hall and describe the remedial measures successfully implemented to mitigate and control seepage within the Underground Power House complex.

Keywords: WCS Initial filling, Seepage, Drainage Hole, Grouting, Machine Hall

1. INTRODUCTION:

Punatsangchhu–II Hydroelectric Project (PHEP-II) is a run-of-the river scheme which envisages construction of a 91m height and 225m length concrete dam across Punatsangchhu River at about 2 km downstream of Tail Race Tunnel (TRT) outfall of Punatsangchhu-I Hydroelectric Project (Fig.1).

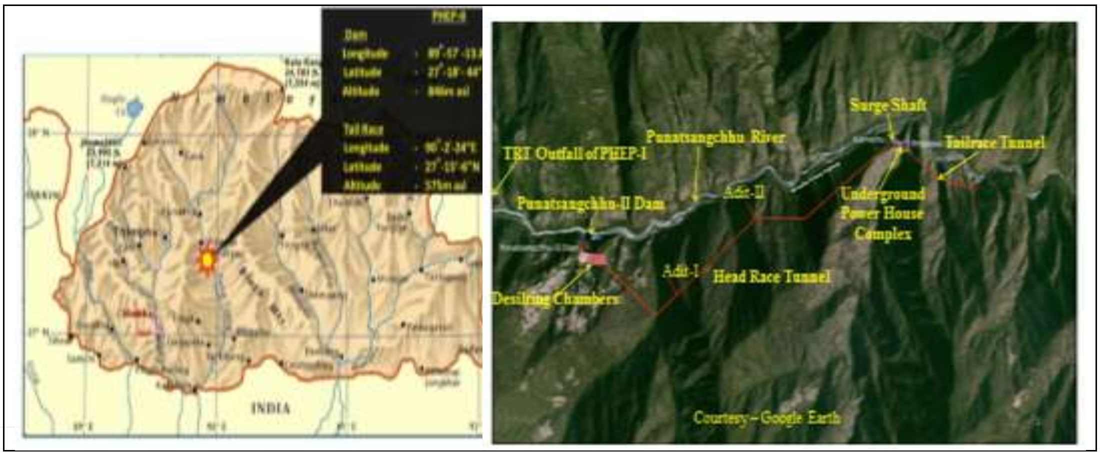


Fig1. Location map of the PHEP-II project site, Wangdue, Bhutan

The dam and intakes are located below the Uma village on the right bank of Punatsangchhu river along the Wangdue-Tsirang national highway. The water will be diverted through 8.58 km long having 11m finished diameter Head Race Tunnel to an underground power house located near Kamichhu village to generate 1020MW of power (Fig 3).

1.1 General Layout Plan of Punatsangchhu–II Hydroelectric Project

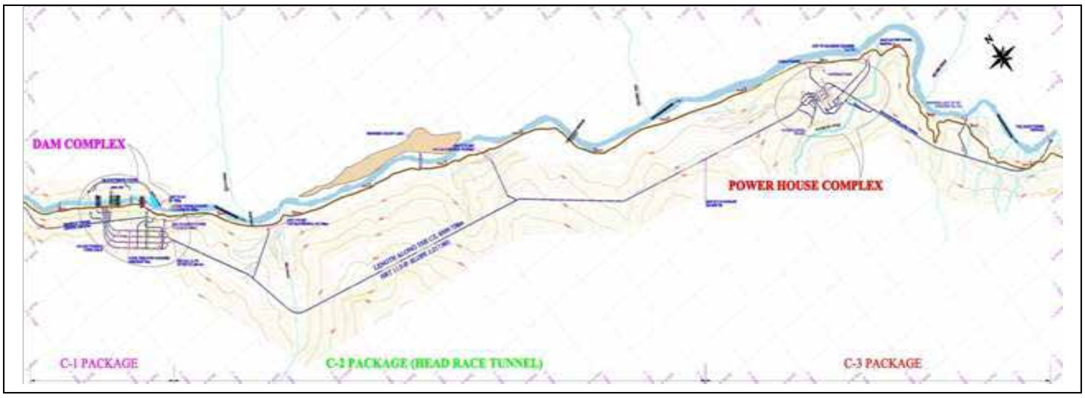


Fig 2. General layout of the Punatsangchhu–II Hydroelectric Project

1.2 Layout Plan of Power House Complex

Layout of Power house complex is in 3D view is depicted below:

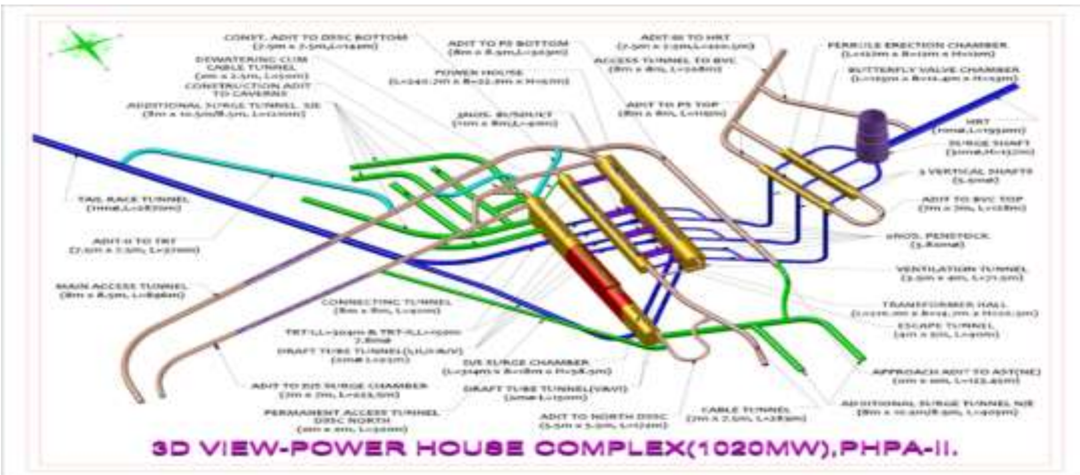


Fig 3. 3 D view of Power House Complex

1.3 Regional Geology and Geomorphology of the project:

Regionally the area is located within a part of the Tethyan Belt of Bhutan Himalayas and at the dam site, rocks of Sure Formation of Thimphu Group of Precambrian age are exposed *(Bhargava, O.N. 1995).* The rocks of Thimphu Group in general is characterized by coarse-grained quartzo feldspathic biotite gneiss with bands of mica schist, quartzite and concordant veins of foliated leucogranite. The bedrock exposed in the project area is represented by garnetiferous, biotite bearing quartzo feldspathic gneiss showing a general foliation trend N10°E to N40°E and dips 20° to 40° towards ESE to SE and at places, the rocks exhibit broad warps as evidenced from the swing in foliation from N40°E to N-S and even upto N10°W to S10°W (Fig 4).

Bhutan Himalaya is divided into a number of geomorphic regions, ravines by major southerly flowing rivers, Amochu, Wangchhu, Punatsangchhu, Mangdechhu and Manaschhu, within its lesser and Central Himalaya tract.

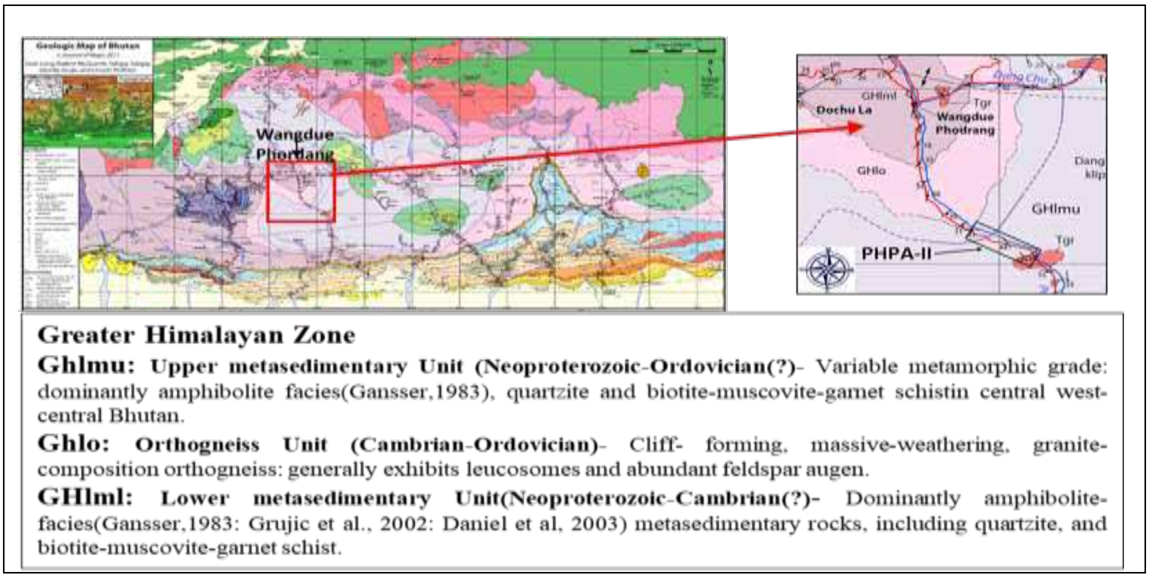


Fig 4. Regional and dam site geological overview of the PHEP-II

1.4 Geology of Power House Complex

The power house complex comprises of two major rock types. Dark grey, fine to medium grain, thin to moderately foliated biotite schist with random lateral intrusions in form of leucogranite, pegmatite bands and quartz veins. Leucogranites are generally pale white to white in color, medium to coarse grained with flaky mica patches. Leucogranites fresh, hard, massive with UCS ranging from 150-175 MPa (Avg. Hammering index) when compared to biotite schist i.e. biotite gneiss and leucocratic granite constitute the hill. Biotite gneiss exhibits persistent foliation. Biotite gneiss is usually weathered except those places where it has more quartz content. This is also traversed by pegmatite veins where it is in immediate contact with granite.

1.5 Key features of Power House Complex

Power House Cavern	
Location	Right Bank
Type	Underground
Size	240.7 m(L) x 23.5 m(W) x 51 m(H)

Installed Capacity	6 x 170 MW (1020 MW)
Service bay level	EL. 582.0m
Type of Turbine	Francis type
Design Head	241 m
Design Discharge /Unit	77.67 m³/s (85.43 m³/s, with provision of 10% overloading)
Transformer Hall Cum GIS Cavern	
Type	Underground
Size	215.7(L)x14m(W)x26.5m(H)
Downstream Surge Cavern	
Type Size	Underground 314m (L) x 18.80 m (W) x 58.5/48.5m (H)
Surge Tank (Surge Shaft)	
Type	Orifice Type (Open to Sky)
Number	One
Size	31.0 m Diameter
Height of Surge Shaft	137.00 m

Support System Provided in Machine Hall of Power House

- SFRS- 200 Mm Thick Crown and Both Walls
- Rock Bolts
- Resin End Anchored/FMI Fully Grouted 36 Dia, 20m/12m Rock Bolts @ 1.5m x 1.5m (Staggered)
- Cement Grout in both U/s & D/s Walls Through 20m long Grout Holes @ 3mx3m spacing at pressure of 3-5 kg/cm2
- Cement Grout in Crown Through 15m long Grout Holes @ 3mx3m spacing at pressure of 3-5 kg/cm2. The length of grout (15m) is inside rock.
- ISMB 300 @ 0.5/0.75 m c/c staggered

2. GEOLOGICAL INVESTIGATIONS OF SEEPAGE IN POWER HOUSE

2.1 Initiation of Seepage in Power House Cavern

The HRT, Surge Shaft, BVC, PS, DSSGs, ASTs, TRT were completed and thoroughly inspected by external inspection committee. The initial filling of the water conducting system (WCS) began on 27.06.2024 and was completed on 16.07.2024, up to elevation 825.0m for targeted commission of Unit-I & II on 15th August 2024.

On reaching of Surge Shaft filling level at MDDL (EL. 825 m), minor seepages were observed in Adit to pressure shaft bottom (located u/s of Machine Hall Cavern). In first week of August seepages were observed in AHU Room, Auxiliary Bay (RD 0 to RD 23 m). Further new seepage locations were observed in upstream (u/s) wall of Power House right from MIV floor to crown, top Adit to Machine Hall and Transformer Hall. Following are the photographs of the power house during seepage occurrence:

		
MH u/s Wall & Crown	MH u/s Wall & Crown	MH u/s Wall & Crown
		
MH u/s Wall	Seepage in AHU Room, Auxiliary Bay	Covering of Unit-1 with Tarpaulin Sheets
		
Seepage at MIV floor	Seepage between Auxiliary Service Bay Area	Heavy Seepage in APS bottom

2.2 Immediate remedial measures

Prima facie immediate necessary decisions were taken to minimize the impact of seepage viz.

i. Drains installed in the machine hall and transformer halls were cleaned.
ii. Additional drainage holes are provided in Adit to Pressure Shaft Bottom and Adit to Transformer Hall on major dripping points.
iii. False ceiling installed in the Machine Hall to prevent water dripping on the machines.
iv. V-notches installed in the drains at suitable locations along the length of machine hall and transformer hall to ascertain quantum of seepage
v. Continuous pumping out from machine hall sump and Adit to PS Bottom

The seepage was showing increasing trend while reservoir filling of last 7m was still balance. It was decided for long term solutions to arrest seepage in Machine Hall and further recommended to take up the following measures:

i. Dewatering of complete HRT and Surge Shaft.
ii. Inspection for identification the source of Seepage.
iii. Continuous seepage measurements, monitor the instrumentation data, correlate, and analyze the seepage data with actual geological profile.
iv. Remedial measures to arrest the source of seepage.
v. Detailed documentation i.e 3D geological sections of MH, TH, Surge Shaft and HRT to correlate all weak zones/foliation planes of the power house and simultaneously carry out physical inspection thoroughly in the HRT, PS and SS.

vi. Recorded every seepage data of machine hall with water levels in surge shaft and dam.

2.3 Geological Investigations carried out in Power House Complex

For the proper correlation and targeting weak zones/area in subsurface components of the power house, prepared detailed 3-D geological sections of Machine Hall to HRT, Surge shaft, vertical pressure shaft (Fig 7,8 &9). Held technical discussion with expert's time to time before providing the treatment in the power house. Accordingly decided for re-drilling of the surge shaft bottom, grouting, WPT and applied penetron for nearby structure of the Machine Hall.

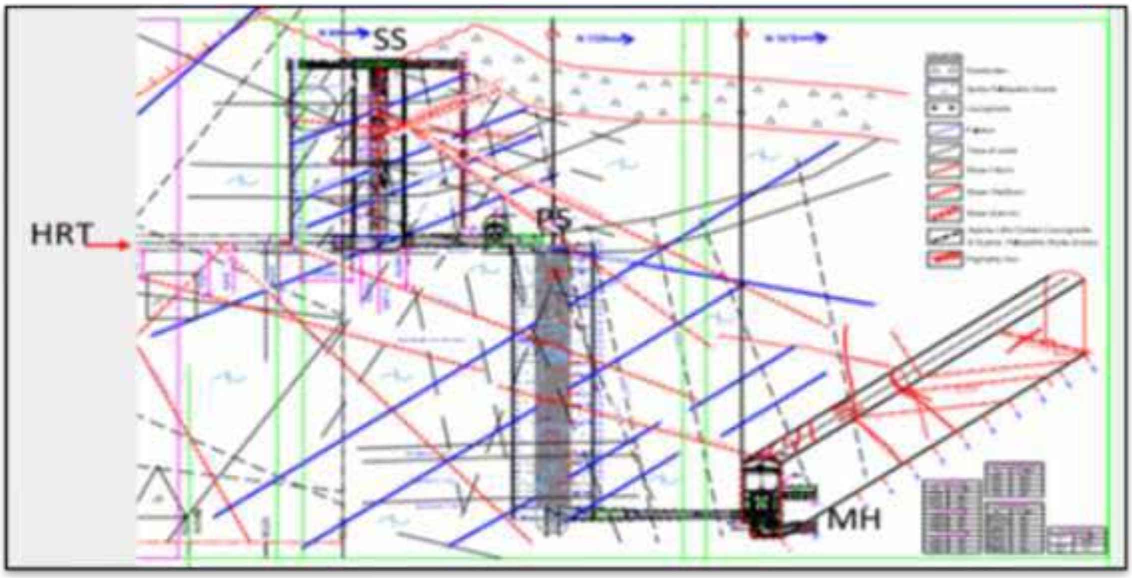


Fig.5 Geological L-section showing HRT, Surge shaft, Pressure shaft and machine Hall.

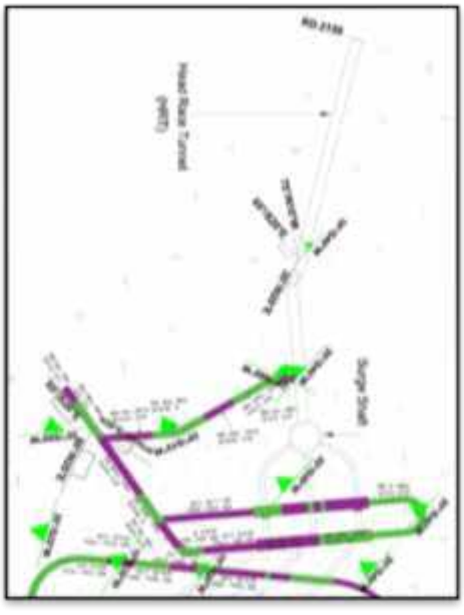


Fig. 6: Rock mass of HRT, Surge shaft and Pressure shaft	Fig. 7: Rock mass of MH, TH and DSSG

3. REMEDIAL MEASURES ADOPTED FOR CONTROL OF SEEPAGE IN POWER HOUSE

i. Grouting at bottom of Surge Shaft through existing grout holes by re-drilling them to greater depth (1m -1.5m below the previous depth of 7 m).
ii. Penetron painting in HRT up to weak zone which appears up to RD 200 m. The extent of penetron painting to be reviewed based on detailed geological sections/model to be submitted by PHPA-II at the earliest.
iii. Penetron painting in Surge Shaft bottom and walls up to FRL especially at every vertical lift concrete joint.
iv. 90 mm dia. drainage holes in Adit to pressure shaft bottom up to crown of machine hall (Approximately 35-40 m long) at the approximate spacing of 2m x2m.
v. Inspection of complete HRT, inspection of Pressure Shaft with emphasis on the portion where the weak zone of Class IV and V crosses the pressure shaft.
vi. The filling post remedial measures may be taken up with pressure shaft, and then the seepage trend in the power house may be observed. NRV to be installed in HRT as far as practicable in places where water ingress was found from the PVC pipes left behind after concrete lining.
vii. Proper videography of HRT and Pressure Shaft inspections for baseline data.
viii. Preparation of Detailed Geological Sections from Machine Hall to HRT/Surge shaft for identification of any weak zone.
ix. Systematic measurement of seepage in the power house, reach wise to be carried out.

Accordingly following measures were taken to arrest the seepage in the Power House Cavern:

a) Redrilling and extension of grout holes and Penetron painting in Surge Shaft Bottom portion

Total 10 nos. of holes were re-drilled at bottom of Surge Shaft to greater depth of 1m to 1.5m below the previous depth of 7 through existing grout holes, After that the Water Permeability Tests were carried out for all re-drilled holes and it was found under permissible limit (< 3 lugeon), no water intake has been observed. After a successful WPT tests, all re-drilled holes were grouted and plugged. These are the photographs of drilling , WPT and treatment is tabulated below:

		
Drilling at the bottom of Surge Shaft	WPT carried out at Surge Shaft bottom	Flushing of re-drilled hole before WPT at Surge Shaft bottom

Penetron painting in the Surge Shaft bottom and walls especially at every vertical lift concrete joint. These are the photographs of the penetron treatment are given below:

		
Penetron treatment near junction of HRT and Surge Shaft	Penetron treatment in Surge Shaft	Penetron treatment of Surge Shaft bottom.

b) Re-drilling of grout holes and Penetron treatment in HRT

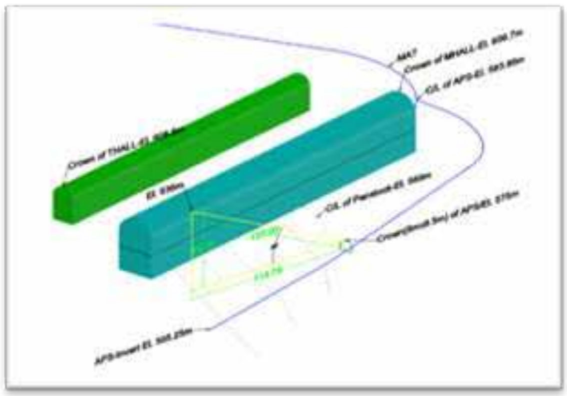
In the HRT section near to the surge shaft between 0m and 500m was re-checked thoroughly and targeted the PCC invert lining reach and adit plugged zone of HRT. Drilled the invert lining and proper investigate the weak zone subsurface rockmass. Simultaneously, carried out post Water permeability tests.After completed the drilling and WPT, all gantry joints, minor cracks and adit plugged area near to the surge shaft were repaired with penetron treatment.

		
Grout hole at HRT invert lining in PCC reach near to the Surge Shaft	Grout hole at HRT invert lining in PCC reach near to the Surge Shaft	Water Permeability test in HRT near to the surge shaft

The treatment works of HRT and Surge Shaft were completed and around 6.00 MT of penetron were consumed while applying the water proofing materials in HRT/SS area. WPT on 10 Nos. of grout hole at invert of Surge Shaft and 02 Nos. of grout hole at HRT invert lining were carried out and was found within the limit.

c) Additional drainage holes in the Adit to Pressure Shaft

To arrest the seepage directly from the machine hall, prepared a 3D model of the area. Initially 90 mm diameter drainage holes in Adit to pressure shaft (APS) bottom up to crown of machine hall (Approx. 35- 40 m long) was drilled (Fig.11). The seepage in Machine Hall, Auxiliary Bay and service bay area upto Unit-III was observed to be in decreasing trend with the progressive drilling of drainage holes in APS.

	
Fig.8 Layout of Adit to pressure shaft bottom	Fig.9 3D Model of drainage holes in Machine Hall

d) Extension of drainage holes in the Adit to Pressure Shaft

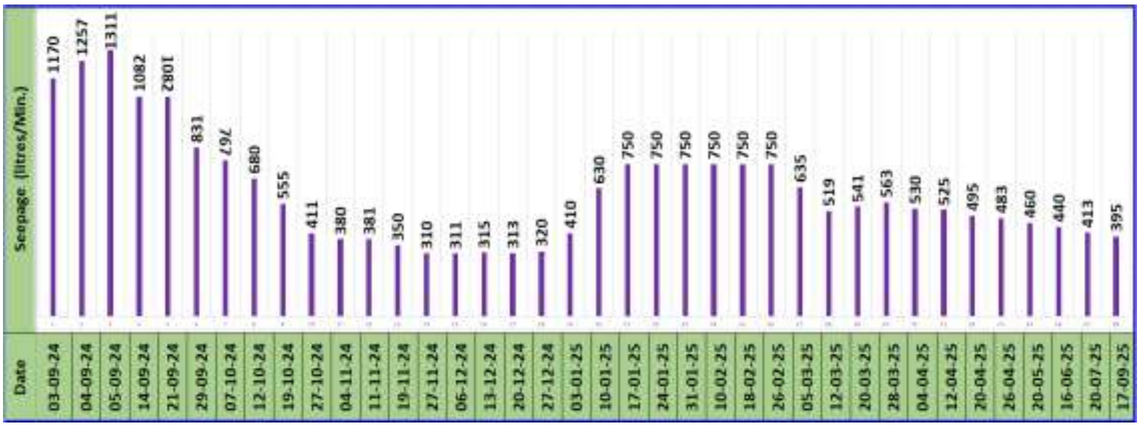
Despite above measures, continuous seepage was observed between the Unit IV to VI. The site was re-assessed in 2nd week of December to see the further possibilities of arresting the seepage between Unit IV to VI. Accordingly, longer holes of 100m to 130m from RD 220m to 240 m were planned in inclined direction to arrest the seepage towards the far end of Machine Hall (Fig 12). Further, it was recommended to extend the ongoing drainage holes

upto 40-50m in APS bottom as per the geological condition and drilling of inclined holes upto 100-130m to cover machine hall area upto Unit - IV. 3D Model of drainage holes in Machine Hall is depicted below:
The drilling of 68 nos. drainage holes ranging from 35 m to 130 m between RD 40m and 240m has been completed. The details of drainage holes are tabulated as below:

		
Drainage hole at left SPL in APS	Drainage hole at crown in APS	Drainage hole at right SPL in APS
		
Drainage hole released maximum seepage at RD 107 (LHS)	Drainage holes releasing water from RD 70 to 180 m in APS	Seepage in MH Service Bay Crown after drainage Holes

e) Monitoring of Seepage in Powerhouse

Seepage measurement was started from first week of September 2024 and discharge quantity was 1050-1100 L/min but with the time it was gradually increased upto 1311 L/m w.r.t. water level in surge shaft and in dam was EL 827m.
In the first week of September 2024, the maximum seepage measured in the Machine Hall was 1311 lpm, while the minimum seepage recorded was 302 lpm during the Water Conductor System (WCS) empty condition in the last week of November 2024. A gradual decreasing trend in seepage was observed from the first week of March 2025 onward. Following the completion of the drainage holes, the present seepage in the Machine Hall has reduced to less than 395 lpm in the September 2025, achieving a stable and acceptable seepage level for the powerhouse complex. The detailed seepage pattern during initial filling, dewatering, refilling, and the post-drainage-hole stage in the APS is illustrated in the graphs below.



Graph A- Seepage Trend in Machine Hall during and after remedial measures

Seepage measurement from the drainage holes in the Adit to the Pressure Shaft bottom commenced in the last week of December 2024, with an initial recorded value of 78 lpm after the completion of 13 drainage holes. Continuous monitoring was carried out thereafter, and the current seepage reading has come down from 500lpm to 100 lpm following the drilling of 68 holes.

The seepage pattern in the Machine Hall shows a consistent decreasing trend. The seepage observed earlier between the Auxiliary Bay and Unit-III up to the Annexes Building has almost completely dried up, and at present, no minor or major seepage is occurring in the Machine Hall.

		
Machine Hall after treatment	Dry Bus Duct 3	Dry Gable end wall of the MH

f) Provision of False Ceiling in Machine Hall

As per the recommendation, additional provision of installation of false ceiling in the Machine Hall was installed to prevent any water dripping on the machines in near term. A design was prepared by the WAPCOS and it has been installed within two months.

g) Monitoring of Instrumentation data

Instrumentation data from the power house, particularly the piezometric data from the Machine Hall, is essential for assessing seepage trends. We are monitoring this data on a daily basis to correlate seepage behavior with the performance of various power house components. The piezometers and load cells installed at the crown and upstream wall of the Machine Hall have been functioning satisfactorily and have not indicated any abnormal readings. NIRM is undertaking the monitoring and analysis of instrument data of Power House Complex.

4. CONCLUSIONS:

- The construction of an underground power house in the Himalayas is inherently challenging, and the occurrence of major seepage in the machine hall prior to commissioning the first two units posed a critical hurdle for the successful completion of the hydropower project.
- During initial filling of Water Conductor System, seepage was detected in the Powerhouse cavern with an increasing trend and mitigating this seepage became a significant challenge. It was critical for the project team to identify the source of the seepage quickly and implement corrective measures to prevent further issues, with the goal of commissioning the first two units for power generation at the earliest.
- A thorough re-inspection of the entire Water Conductor System, from the intake to surge shaft was carried out and data was recorded. Treatment work was undertaken to reduce seepage in the powerhouse, particularly in the Machine Hall area, to ensure the safety of the structure and prevent water ingress into equipment.
- Continuous high-level technical meetings were held, and treatment measures were recommended periodically based on updated observations. Subsequently, nearby components of the Machine Hall were targeted for detailed inspection and remedial measures were taken in HRT, Surge Shaft, and Vertical Pressure Shaft areas.
- The drainage holes drilled from the adit to the pressure shaft bottom have proven effective in arresting seepage into the Machine Hall. With the improved and now stable seepage conditions, the objective of channelizing Machine Hall seepage toward the APS has been successfully achieved.
- Furthermore, the testing and commissioning of Units I & II were successfully completed on 17 December 2024. And within a span of less than 9 months, all six units of the project have now been fully commissioned.

The geological challenges faced by the Project has been successfully handled with extensive remedial measures as per expert guidance of Agencies of GoI and RGoB coupled with the hard work and dedication of Indian and Bhutanese Engineers and Geologists.

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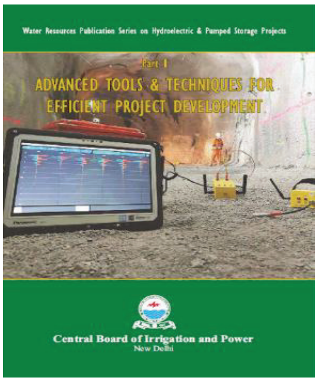

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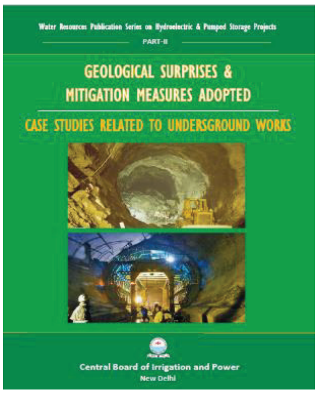

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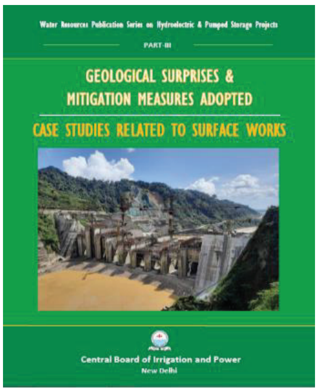

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