PROCEEDINGS

International Conference

DAM SAFETY-2025

Latest Technologies for Rehabilitation and Dam Safety

20-22 March 2025, Hotel Peterhoff, Shimla (H.P.), India

Workshop on

Grouting Technologies in Rehabilitation of Dams

19th March 2025, Shimla

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FOREWORD

A.B. Pandya

Dams are increasingly occupying our attention as a reliable measure of water management and energy transition all across the world. The importance of the reservoir storage created by the dam increases over the times and the resultant water supplies form the kingpin of the economy and well-being of the beneficiary communities. Dams, therefore have to retain their designed operations over multiple generations. It is in this context that dam safety has emerged as a discipline in its own right to ensure that our water resources assets continue to provide the benefits to the society for a long period of time.

In view of the innovative technological development in the field of dam safety, there is a need to update the knowledge of dam safety professionals, dam owners, contractors, consultants and the agencies involved in dam safety discipline. It has been recognized that dam safety aspects particularly of the existing dams, are not receiving much attention as they should be, especially in view of the fact that a number of these old/existing dams are ageing, leading to gradual natural degeneration. Even safety of some of the dams which have been constructed in the recent past may become questionable, if the flood characteristics or seismicity of the area has changed. These old dams may need a research under today's technology.

The Dam Safety Society will provide a forum to deliberate on the unique challenges posed by existing dams for evolving unique solutions in order to ensure their safety. Dam safety touches many unique aspects of technology, economy and disaster management. There are multiple stakeholders involved in the dam safety activities namely, regulators, dam owners, service providers and researchers to name a few. However, there is no common platform where the mutual concerns of various stakeholders can be articulated. The Dam Safety Society will focus on such unique aspects and attempt to generate common knowledge base utilizable by the professionals.

The International Conference DAM SAFETY 2025 on the theme "**Latest Technologies for Rehabilitation and Dam Safety"** is being organized on 20-22 March 2025 at Shimla preceded by one day Workshop on **Grouting Technology in Rehabilitation of Dams** on 19th March, 2025 by Dam Safety Society in association with leading organizations CWC, DRIP, NDSA etc. in the field. The conference will provide a forum for exchange of knowledge and from leading professionals, an exhibition of latest products and processes.

This Conference would bring together the policy makers, senior functionaries and technocrats from Central/State/UT Governments, academicians, to focus on the best global practices; technological advancements, emerging dam safety challenges in addressing dam safety concerns. The 2nd in the series, the International Conference Dam Safety - 2025 being held during 20-22nd March 2025 at Hotel Peterhof, Shimla, is a joint initiative of the Dam Safety Society, Directorate of Energy, DRIP, Central Water Commission, Government of India;, Govt. of Himachal Pradesh, SJVN Limited.. The Conference received an overwhelming response from dam professionals from both within the country and abroad.

More than 200 dam professionals from India and abroad are participating in the deliberation of the conference and 36 technical papers are being presented by Indian and overseas experts during the 8 technical sessions from Indian and overseas experts. 12 presentations have been received from the experts working in the field of grouting, to focus on the methodologies, technologies, products, and services in the workshop as well as in the exhibition being organized concurrently at the conference venue. Many organizations came forward to project the contemporary developments in technology, instrumentation, materials, and services for the dam rehabilitation activities.

We are thankful to all the sponsors, authors, delegates and exhibitors for their active participation in the conference and workshop. The technical papers brought out as Proceedings Volume is a valuable reference material for the decision-makers as well as dam professionals engaged in various aspects of dam design, construction, operation, maintenance, and rehabilitation.

This Proceedings Volume contains the accepted technical papers and is available for download from the Dam Safety Society website www.damsafetysociety.net; by all the interested dam professionals for their reference and use. We hope all concerned will make best use of the information contained in the Proceedings Volume in shaping their dam safety initiatives.

The rich exchange of knowledge and experience, to address the dam safety issues culminating to prepare the set of recommendations for addressing the dam safety issues faced by the dam professionals. Ten (10) national and overseas organizations will show case their technologies, products, and instruments to the vast assemblage of dam professionals and decision makers in an exhibition being organized concurrently as part of the conference.

Careful planning and untiring efforts of several individuals from the collaborating organizations with the guidance from the Organizing and Technical Committee contributed to the success of this major event.

We compliment all the persons and organizations for their valuable contributions.

A.B. Pandya
President, Dam Safety Society &
Former Chairman, Central Water Commission

2nd International Conference on
"Latest Technologies for Rehabilitation and Dam Safety"
19-22 March 2025 at Shimla

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Proceeding of International Conference on Dam Safety-2025

Latest Technologies for Rehabilitation and Dam Safety, 20-22 March 2025, Shimla

Autonomous Technology for Dam Safety, Maintenance and Rehabilitation

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Alok Mukherjee, ex Ground Head (Robotics) R&DE(Engrs), DRDO, Pune and CEO,
Prakash Khanzode, Director, Raja Mahbubani, Directo and Sunny Sebatian, Director
i4 Marine Technologies P. Ltd., Pune

Abstract

Autonomous systems on land, air and for water applications are playing important and ever-increasing roles in every aspect of life. This is particularly true where skilled manpower is hard to come by and is not cost and time effective. These systems conduct repetitive jobs very efficiently and without errors. At i4 Marine technologies, Autonomous Surface Crafts and Remotely Operated Underwater Vehicles have been developed and demonstrated in applications involving dam bathymetry, structural health monitoring and dam wall inspection. These platforms are electrically powered and hence utilise clean energy without polluting the waters. Further, they are being optimized for other application for mapping waste and pollution in river and water bodies.

This paper brings out an example of both bathymetry and dam wall inspection carried out at two dams in Pune and the results. The results demonstrate the efficacy of the systems and with regard to the precision of measurement and repeatability to scan exactly the same location which is essential- to study the yearly effects of sedimentation in the dams, as well as the status of the crack propagation in dam walls with time.

Introduction

To date, many surface crafts and underwater vehicles, both remotely operated as well as autonomous, have been developed in last decade or so. Primarily designed for research purposes, this technology has been adapted for many applications in various industries like survey, petroleum for oil extraction wells, pipeline laying and inspection and maintenance of related equipment.

The autonomous surface vehicles (ASV) have applications mainly for survey of water bodies by providing a path for traversing and collecting scientific data. The sensors on-board allow for real-time data collection of various parameters as the platform carries out a pre-programmed mission. The ASV both saves the data or the parameters being measured as well as transmits that to shore to the Ground Control Station for live view and analysis where required. This allows for exact mapping of a water body and providing data of depth in case of bathymetry as well as pollution mapping where corresponding sensors are provided.

In the underwater domain, the technology has proliferated to many other applications like inspection of underwater mechanical and civil structure, pollution monitoring, reach and rescue as well as even underwater tourism. The platforms realized are equipped with many different types of sensors that allow visualisation, mapping, identification and other related areas. The use of Artificial Intelligence and Machine Learning find immense use to extract the information from both the visual as well as numerical data being collected during a mission.

The underwater systems are also equipped with robotic arms having multiple degrees of freedom and tools for cutting, lifting, hooking etc. mainly underwater. These hazardous operations underwater which used to be performed by trained and experienced divers and being off-loaded to machines. Many such ROV's have been used from shallow depths to full ocean depths though usually they are more tuned for oil industry applications currently to depths of 2000 meters water. With new sensors emerging their application range has diversified in many directions.

The Technology

At i4 Marine Technologies, a Start-up at Pune, with a vision to design and develop both Autonomous Surface Vehicle and Remotely Operated Vehicles, systems have been designed initially for Dam Inspection and Rehabilitation which is the necessity today. The firm is co-founded by ex- scientists from DRDO and NIO and industry experts in Electronics, Ship building and Industrial Product design to bring out state-of-the-art systems. The amalgamation of such experienced personnel with expertise in design of Robotics and Autonomous platforms on Ground, Air, Surface and Underwater Vehicles has resulted in the realization of an Autonomous Surface Vehicle – ***AquaScanner*** and Underwater Remotely Operated Vehicle – ***AquaNaut-30*** which have been trial evaluated in actual field to verify their performance. The details of the technology in the systems are explained to bring out the usefulness of the system and their effectiveness.

AquaScanner

The Autonomous Surface Vehicle (ASV) – Aqua Scanner (fig 1) can be programmed to traverse over or scan a water body measuring different parameters in real-time. AquaScanner was used to demonstrate its capability in bathymetric mapping of Panshet dam reservoir near Pune. The platform fitted with an Echo Sounder, GPS, 3 axis accelerometer measuring roll pitch and yaw, heading sensor measuring the direc11tion with respect to North, was preprogrammed to undertake waypoint navigation in a lawn mower fashion over the water surface. The mission was loaded onto the platform controller and commanded to be executed. The intelligence provided in the platform carried out the traversal autonomously over the path collecting data on real time. The data is stored on-board as well as transmitted using Radio Frequency on real time to the Ground Control Station placed 2km away.

Figure 1: AquaScanner – Autonomous Surface Vehicle

Figure 2 shows a snap-shot of the screen during mission execution. The yellow lines are programmed lines while the actual platform movement is shown in red colour superimposed on the lines. It was noted during the mission execution that the drift from the pre-defined line is less than 2m during operation. The path defined during mission planning takes into consideration the current state of the reservoir by initially defining a boundary around the area or carrying out geo-fencing. Once commanded to carry out the mission the platform effortlessly scans the defined path accurately by the AI built into the platform which provides the PID correction to the servo mechanism on real time ensuring minimum drift. The same exercise or path was repeated over an area demonstrating the unique feature of this Autonomous System to exactly and accurately go over the same path. This feature is extremely useful to study the periodic variation of the sedimentation of the reservoir either monthly or annually to aid in prediction studies. This data could be effectively used for planning the rehabilitation program for each dam depending on its rated and current capacity.

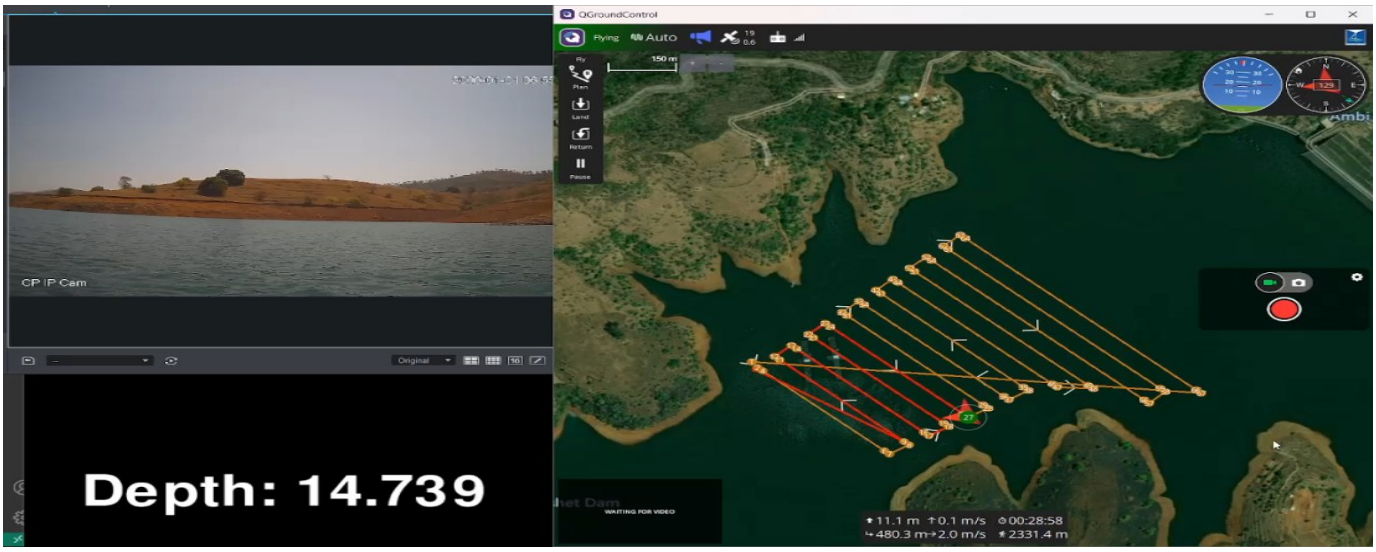

Figure 2: Bathymetry survey mission at Panshet Dam.

On completion of data collection, the same was processed to generate counter maps as well as 3D terrain of the resulting area. Figure 3 shows one such plot processed from the data. Dam volume capacity, water spread area and processed numbers generated. The details of the same have been outlined in the report prepared (Ref. 1)

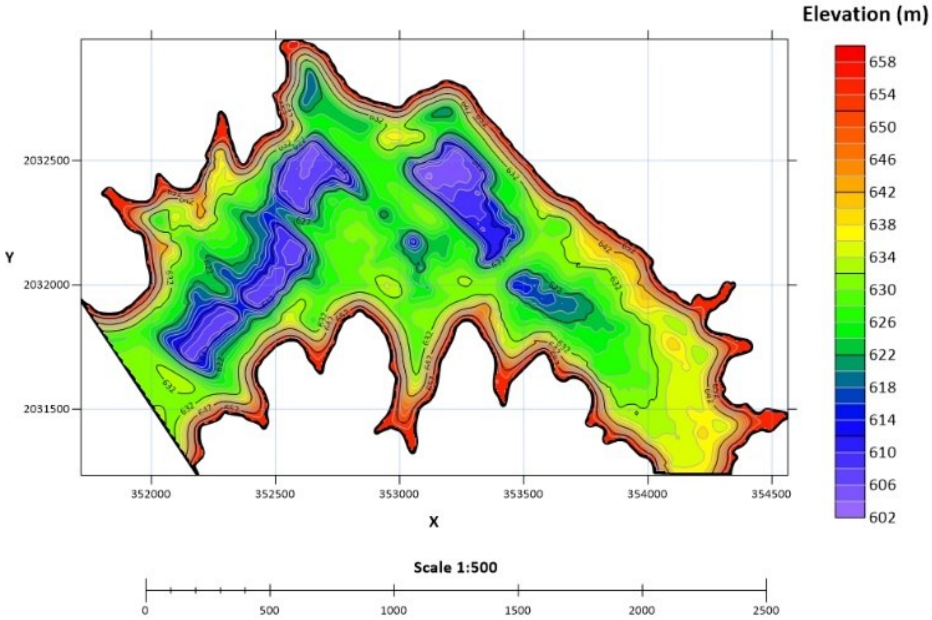

Figure 3: Underwater Terrain map of the area scanned

The Aqua Scanner can also be fitted with the Flow sensor to measure the flow in a canal by lowering the sensor to different depths. The Aqua Scanner can also be sent to different locations in the canal remotely and put on Position Hold Mode. The flow while the boat is stationary, can be measured at different depths. This data is both transmitted to shore where a Ground Control Station displays the same to the User as well as stored on-board for further analysis. The canal discharge can be measured on real time at different locations to detect leakage or pilferage etc.

AquaNaut-30

The Underwater Remotely Operated Vehicle AQUANAUT-30, Figure 4, has been designed and developed as a platform for underwater visual inspection. It is an Underwater Remotely Operated Vehicle that carries sensors for navigation and propulsion as well as a controller to provide the user with the feedback of the path being traversed in real time by measuring the depth and heading of the platform on real time. The platform can be enhanced with other sensors for diverse applications aimed at marine geology, chemistry, living resources and for constructions in seas and for inland applications towards pollution mapping, aquaculture amongst many more. Another area is the development of deck side hardware and software for higher level AquaNaut's versions.

The Aquanaut-30 consists of an aluminium chassis which is the outer frame of the platform thereby providing long life. The Aquanaut-30 is self-contained with its power source, control electronics, communication and navigation hardware placed in its central hull. The Aquanaut 30 has six thrusters placed in such a plane and geometry, thereby allowing four Degree of Freedom (DoF) to the platform. The thrusters are placed within the frame thus protecting them from mechanical damage during operations or mission. The rechargeable batteries provide good endurance necessary during survey.

Figure 4: Aqua Naut 30 at site.

The platform was launched at Varasgaon Dam near Pune to inspect the dam wall for any anomaly. The ROV was tethered and lowered at different pillar location of the dam and the platform dived vertically down to a depth of 30m from the water surface. The resting video is subjected to Artificial Intelligence algorithms for removal of turbidity in the water. This unique feature provides the Dam Safety officials a clear vision of the location being surveyed. One such snap shot from the video at Pillar No 6 at a depth of 23.6m is provided below in Fig 5.

Figure 5: Anomaly Observed at depth of 23.6m at 6th pillar

The ROV was taken to different pillar location along the dam and lowered to yield a complete picture of the dam was. Another anomaly (Ref. 2) observed at the 8th pillar as a depth of 7.8m is shown is Fig 6. below.

Figure 6: Anomaly Observed at 7.8m depth at 8th pillar

Conclusion

The latest technology being adopted by using the Autonomous Surface Vehicle - Aqua Scanner for Bathymetry has demonstrated that the resulting analysis of the Dams from the point of view of sedimentation studies and Area-Capacity Curves can be easily and accurately generated. The use of the Aqua Scanner makes the operation clean due to the use of electrically powered systems and safe as no humans are necessary to traverse over the dam waters. In addition, the precision of the data and the cost effectively of the method provides a new and efficient manner to be useful for the Irrigation Departments of the States and Dam Rehabilitation planners.

The usefulness of the Aqua Naut or the Underwater Remotely Operated Vehicle (ROV) has been tested and trial evaluated at an actual dam site to provide real-time video and still images to detect anomaly in the civil structure. This system is extremely cost effective and fast as it has a good speed of dive as it captures imagery on real time. The in-built AI and ML algorithms carries out video enhancement and hence providing the Dam Maintenance personnel to plan the necessary follow up action.

In conclusion, both these equipment specifically designed and developed for Dam Safety, Maintenance and Rehabilitation are invaluable tools in the hands of the concerned personnel to aid in future planning.

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2. Internal Report: Structural Health Monitoring of Dam wall, Varasgaon Dam, Pune. March 2024.

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UNVEILING THE UNKNOWN-ROV INSPECTION OF FLOODED TUNNEL - A CASE STUDY

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ABSTRACT

The hazardous underwater environment of flooded tunnels makes inspection and maintenance extremely difficult. The innovative technique for using a remotely operated vehicle (ROV) to check flooded tunnels is presented in this paper. For evaluating these tunnels, underwater remotely operated vehicles (ROVs) or underwater drones have proven to be a vital instrument for gathering vital information on the structural health conditions and for making prompt judgments and remedial activities. The suggested method facilitates accurate and efficient examination in submerged tunnel environments by combining sophisticated sensors, live data transfer, and cutting-edge ROV technology.

To navigate, record imagery, and evaluate the structural strength, the system comprises software and hardware aspects. The paper seeks to reduce threats to humans while improving the reliability and security of flooded tunnel examinations using an innovative method.

1. INTRODUCTION

Flooded tunnels are essential elements in infrastructure systems because they are commonly utilized for hydroelectric power generation. The safety and dependability of these systems depend heavily on maintaining their structural soundness and operating effectiveness. However, because of their inaccessibility, changing surroundings, and possible risks, flooded tunnels provide special obstacles for inspection and maintenance. Water pressure, sedimentation, and structural wear are just some of the stresses that these tunnels must withstand; therefore, routine inspection is crucial to preserving their durability, safety, and effectiveness.

This paper presents a case study where ROV-based inspection was effectively used for the inspection of a 5.2km long tunnel to understand its structural details and furthermore, the potential for thorough and effective data collecting is highlighted by the incorporation of cutting-edge instruments like sonar, high-resolution cameras, and environmental sensors into ROV systems.

In order to ensure long-term sustainability and resilience, the paper hopes to further knowledge of how cutting-edge technologies might improve the administration and upkeep of vital water infrastructure.

2. SCOPE OF WORK

The scope of the work was to conduct the ROV- based videography and SONAR survey to capture the internal profile of the tunnel of 5.2km length to clearly understand the interior of the tunnel and to evaluate the structural condition of the tunnel.

3. ROV BASED INSPECTION

Considering the importance of these structures, regular inspections as well as appropriate maintenance and repair work are required. Dewatering the entire structure and performing the structural evaluation are not feasible. The fact that there are limited and smaller entrances for these tunnels makes diving for inspections quite hazardous. The divers will find it challenging to enter the tunnel because of its restricted accessibility. Diver-based inspections may have poor data precision and accuracy. For underwater structural examinations, remotely operated vehicles, or ROVs, are an invaluable asset.

Tunnel Specialist ROV specially designed for tunnel inspections can be used through the manholes for the tunnel inspection. A joystick is used for controlling the ROV during navigation. HD cameras on ROVs assist in obtaining visuals of the interior of the tunnel. When the water is murky, visibility may be low and the images may not be adequate in clarity. Acoustic sensors like imaging SONARs which produce acoustic images, can be used in these circumstances. Each collection of data that the ROV acquires is gathered by the control station.

The data is delivered to the control station via a tether system that is attached to the ROV. The client can obtain the processed data after it has been collected.

Figure1: Tunnel Specialist ROV

4. EXECUTION METHODOLOGY

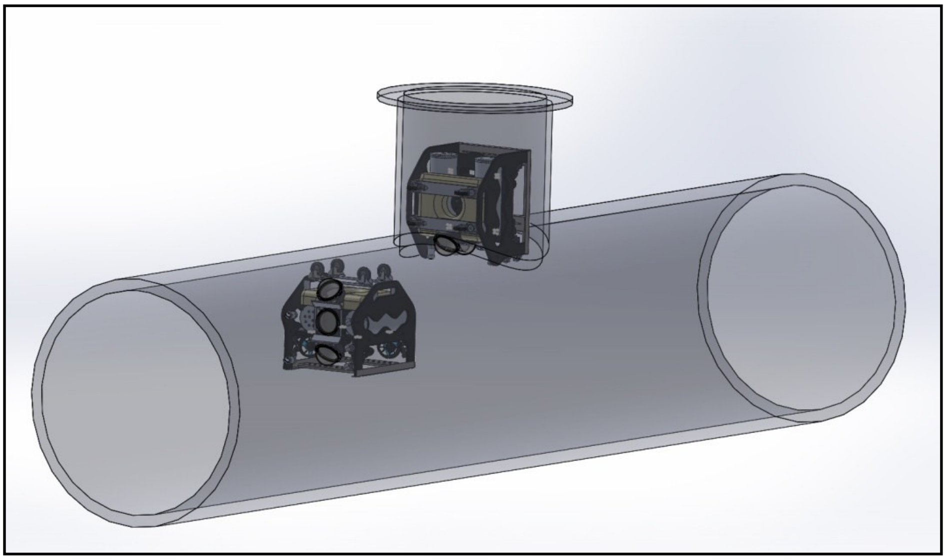

Figure2: ROV Traversing through the Tunnel

A vehicle was used to house the control station. The vehicle was placed near the system for Tether Management. The control station's power supply setup was situated roughly 50 meters

from the van. Inside the van are the specially designed monitors for the live sonar data outputs. During the ROV operation, the client representatives could have a look at the inspection along with the ROV pilot, copilot, and data recorder from the control station.

ROV carried out a methodical underwater examination of the tunnel's interior sections in accordance with the work's scope. The internal survey was conducted using an ROV equipped with three specialized cameras and a system of acoustic sensors placed to map the internal profile of the tunnel. To operate and transmit data from the ROV, a specialized 10-kilometer neutrally buoyant optical fiber line was utilized. This Tunnel specialist ROV can effortlessly inspect tunnels up to 10km in one stretch, first of its kind globally.

The ROV was lowered into the dam's upstream/inlet side. The pilot took command of the ROV as soon as it touched the water, conducted a trial run, and determined that the equipment was operating satisfactorily. After the trial run was over, the equipment was moved towards the outlet. Certified lifting equipment was utilized for the deployment procedure.

The ROV was placed along the center line of the tunnel where the tunnel diameter was 5.2m. This could map the tunnel profile in one go. The acoustic sensor systems and an additional forward-looking sonar was installed on the ROV during the survey. To determine the ROV's trip distance, a tether counter was positioned atop the Tunnel Outlet. In order to track the real-time position during the survey, the ROV system was also equipped with an underwater positioning system (DVL).This could give geotagging of the captured data.

5. RESULTS

The tunnel has a D-shaped profile with an internal diameter of 5.2 meters, containing both lined and unlined sections. The tunnel's interior was thoroughly evaluated by the acoustic sensor or SONAR and videography study, which covered the entire length of the tunnel. The survey precisely documented the tunnel's inside profile using a Tunnel Specialist ROV equipped with HD cameras and the latest Acoustic sensor/ SONAR imaging. Crucially, no notable irregularities, silt, or debris were found, suggesting that the structure is mostly undamaged. A 3D model of the tunnel , profile cross sections at 1m intervals and detailed data, such as bay region profiles, were recorded and examined, providing important information about the tunnel's state.

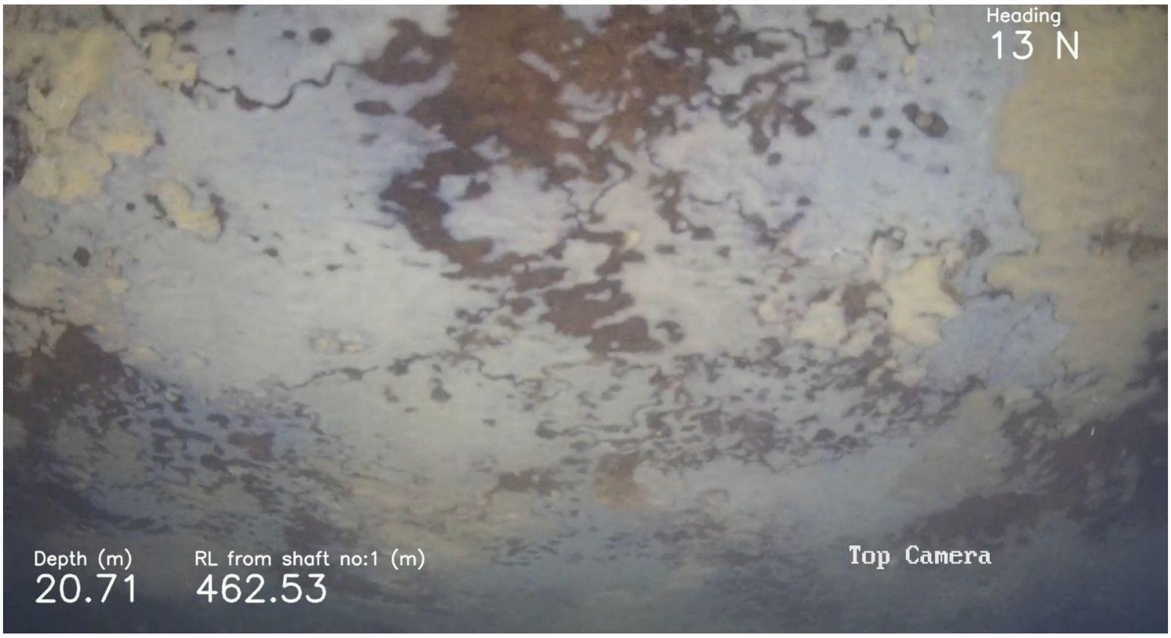

Figure 3: Videography Snap of tunnel interior captured by ROV

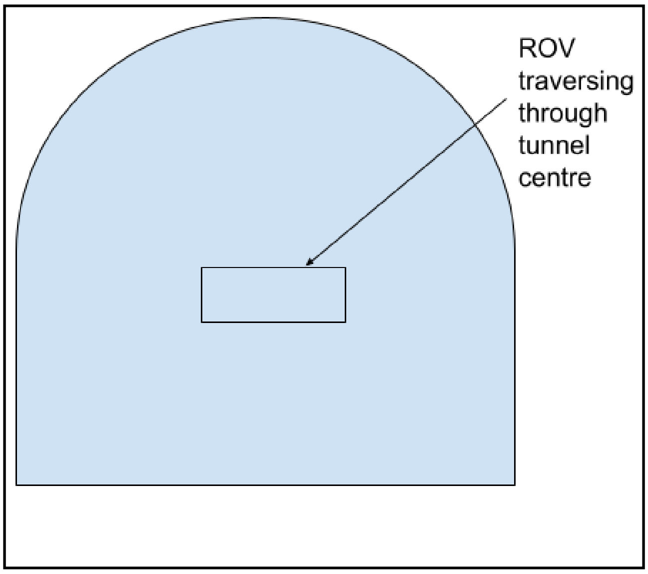

Figure 4: Schematic diagram of ROV traversing through tunnel centre

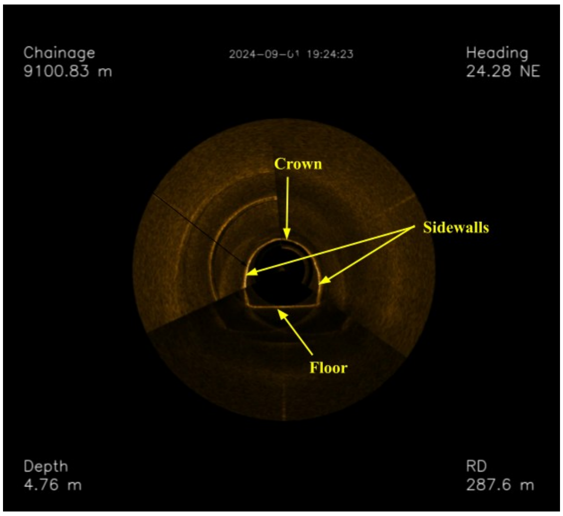

Figure 5: Acoustic SONAR image of Tunnel with marking of Crown, Floor and Sidewalls

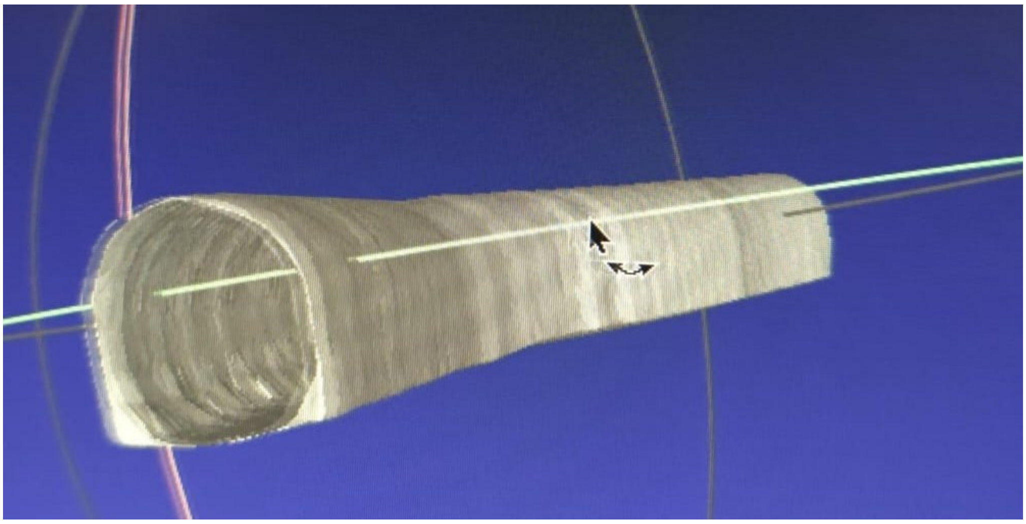

Figure 6: 3D Model of Tunnel -generated from SONAR data

6. CONCLUSION

Using remotely operated vehicles (ROVs) for tunnel inspection is a revolutionary method of maintaining vital infrastructure. Compared to conventional techniques, ROVs improve the precision, effectiveness, and safety of tunnel inspections by combining sophisticated robotics, high-definition photography, and data analytics. These technologies reduce human exposure to dangers while guaranteeing comprehensive assessment because they can function in dangerous situations, enter restricted areas, and give real-time data.

The thorough ROV-based survey helps in understanding the flaws and weak zones of the tunnel in the early stage itself. This technology allows predictive maintenance, minimizes inspection expenses, and cuts downtime. This proactive strategy guarantees the structural integrity and user safety of tunnels while also extending their service life. An important step toward more intelligent and sustainable infrastructure management is the use of ROVs in tunnel inspection.

The end results of the ROV survey can be made into a user-friendly manner and which helps the authorities to get a clear understanding about the structure and make necessary decisions to protect them.

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Advanced geomembrane systems in underwater rehabilitation

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ABSTRACT:

Underwater geomembrane technologies started being studied at the beginning of the 1990s in Europe, where the largest experience of geomembrane systems installed in the dry had been acquired since the beginning of the 1960s. Further research led to the development of a system that could be installed underwater, in still water conditions, to restore watertightness at the upstream face of dams. After the first full-face underwater repair on a dam in 1997, underwater geomembrane technologies prospered, evolved, and at present allow restoring watertightness, and hence safety, of ageing dams, in any condition and at any depth, either to repair the whole or leaking parts of the upstream face, or to perform repair in a staged manner, or to repair failing joints or cracks. Underwater geomembrane systems have the unique advantage of allowing the owner to normally exploit the dam/reservoir during rehabilitation works, and have shown to be durable, able to resist large deformations, joints opening, and differential settlements. The first section of the paper presents the different approaches and solutions to underwater rehabilitation of dams, on wide surfaces or on localised defects, via significant case histories. The second section of the paper addresses the subject of underwater repair in flowing water conditions, i.e., geomembrane technologies that have been developed in the last decade, and allow repairing canals underwater, without stopping the canal's operation. Geomembrane systems for underwater installation in fully operating canals are available in two versions, the Sibelonmat® and the Sibelonzip®. The Sibelonmat® is a prefabricated mattress formed by double-geomembrane panels, deployed and watertight joined underwater, and then filled with cement grout, to provide anchorage by ballast, with a method that does not affect the quality of the water in the canal. The Sibelonzip® is a single-geomembrane system that can provide a similar performance as the Sibelonmat®, at lower costs. The design, placement, and performance of the Sibelonmat®, which has already been installed on trial sections in 3 canals are discussed via the latest installation in a fully operating navigation canal in France. The Sibelonzip® is presented through its concept and future advantages.

1 INTRODUCTION

Geomembrane systems have been used to repair ageing dams since the beginning of the 1960's. They were mostly used to restore upstream watertightness that had decreased over the years, with the objective of bringing the dam back to efficient and safe operation. A large number of successful projects, now decades old, testify the reliability of geomembrane systems adequately designed and installed.

Relining a dam with an upstream geomembrane system required dewatering the reservoir, which in some cases is impossible, or is possible only at high financial, and/or operational, environmental, and social costs, which may not be acceptable. In such a case, the alternatives were to carry out repair with traditional methods such as internal grouting with chemicals or microfine cements, internal drainage by drilling from the crest, or construction of a new downstream facing/buttress. These solutions were not always effective and durable, and/or were very expensive and with high environmental impact.
Europe had the largest experience in rehabilitation of ageing dams with geomembranes, and at the beginning of the 1990s the state-of-the-art system almost exclusively adopted for dry installation was used as a starting point for in-house study and testing carried out by Carpi, the company that had developed the patents for the dry geomembrane system, later widely used also in India (Subramanian, 2025). The outcome was the design of a prototype geomembrane system suitable for underwater installation. The United States Army Corps of Engineers (USACE), owner of a great number of dams and hydraulic structures that could not be dewatered, had in the meantime become interested in finding a technology to install geomembrane systems underwater. Starting in 1995, the USACE Waterways Experiment Station in Vicksburg supervised a 2-year, two-phase research jointly performed by Carpi and by Oceaneering, a large US company providing subsea engineering, high-tech solutions, and underwater services. The research program studied, developed, tested in full scale underwater in a tank, and validated, an exposed geomembrane system suitable for underwater rehabilitation of dams. The evolution of underwater geomembrane technologies and the options now available for installation on dams, i.e., in still water conditions, are presented in Chapter 2.

The next frontier was to perform underwater installation in flowing water conditions, to offer an alternative to those owners whose canals cannot be dewatered, or can be dewatered only with great inconveniences and at high costs. The available traditional underwater technologies required using divers, who can safely operate only in still water, or in water flowing at a speed not superior to 0.5 m/s; therefore, such technologies could not be applied in the majority of canals. Carpi had since the 1970s acquired a wide experience in waterproofing of canals with geomembrane systems in dry conditions, and in 2003 had carried out joint research with TUM, the Technical University of Munich, concerning anchorage techniques in canals. Based on these experiences, at the beginning of the 2010s Carpi addressed the issue of installation of geomembrane systems under flowing water. After developing the preliminary concepts and design, throughout 2013 and 2014 full-scale testing was performed on different configurations of the various components, in still water and in flowing water. The outcome of this effort was the development of the Sibelonmat® and Sibelonzip® systems, which are discussed in Chapter 3.

2 UNDERWATER GEOMEMBRANE SYSTEMS IN STILL WATER

Underwater installation of a geomembrane system requires more time than dry installation and is more expensive, but on the other hand, the dam can continue operating without disruptions so that time is no more an issue, and the uninterrupted revenues/services can offset the higher underwater costs. Selecting an underwater geomembrane system is sometimes an inevitable necessity, but in other cases it may be a choice based on the assessment of the financial, environmental, and social costs of dewatering, and of the benefits deriving from the installation of a geomembrane system that will protect the dam for decades to come.

2.1 *Concepts and developed applications*

A geomembrane system designed for underwater installation is conceptually similar to a system for dry installation. The geomembrane is placed upstream and is sealed at periphery by a continuous seal watertight against water in pressure at submersible peripheries, and against rain waves and snowmelt at crest, out of the spillway area. A face anchorage system is generally required to keep the geomembrane liner stable on the dam face against wind and waves uplift, and when possible a drainage system is placed at the upstream face under the liner, to provide a first monitoring system by measurement of drained water, and to avoid unbalanced under-pressure in case of lowering of the reservoir.

Unlike in a dry system, where joining of adjacent geomembrane sheets is done by heat-seaming, in underwater installations where seaming is not possible, mechanical stainless-steel profiles are used to join the sheets. For the same reason, such profiles are not waterproofed by a geomembrane strip seamed unto the liner, but are made intrinsically watertight by adequate gaskets and watertight fittings. The anchors securing the profiles are mechanical instead of chemical, and greater use of synthetic materials is made to minimise time consuming surface preparation works underwater.

The characteristics and quality of an exposed geomembrane system installed underwater in still water conditions are comparable to those of a system installed in the dry, provided that a sound design based on a preliminary underwater inspection is developed, that installation is performed by experienced professional divers operating in a secured environment (intakes and outlets closed, strong suction areas eliminated or alleviated), and that execution is controlled from the dry by specialised technicians familiar with the systems. If such requirements are met, the water barrier provided by the geomembrane will be durable end effective, accommodating settlements, differential displacements, opening of joints and cracks in the subgrade, thanks to tensile properties that allow an elongation largely exceeding that of traditional remedial measures.

The first underwater installation was made in 1997 and consisted in applying the system developed and validated in 1996 to repair the full face of a dam, as further described. The experience acquired in repair of failing joints and cracks in the dry with a patented external waterstop was the basis for developing a similar system suitable for underwater placement, eventually first adopted in 2002 to waterproof underwater a crack that formed at the first impoundment of Platanovryssi 95 m high RCC dam in Greece. In 2009, the range of available options was completed by adapting the full-face repair system so that it could be installed in stages, following the operational or financial needs of the owner, as further discussed.

2.2 *Face repair: different approaches and examples*

Face repair can concern the entire face of the dam, or parts of it. Full-face and partial-face repair are practically identical in components; hence the extent of the underwater works is a site-specific choice: full-face repair minimizes the possibility of leakage coming from unlined upstream portions of the dam, but may be unpractical, especially when large surfaces, great depths, hence high diving costs, are at stake, or when leakage is limited to a small area.

The full-face repair approach is evidently the most efficient one from a technical point of view, especially in some conditions, such as when the amount of sediment is small and can be removed to place the bottom perimeter seals close to the foundation, or when diving depths are shallow. Lost Creek, a concrete arch dam 36 m high in the mountains of Northern California, is an example. The construction techniques, design and materials used at Lost Creek had produced a somewhat porous concrete, through which water seeped; in a thin arch dam subject to freeze-thaw as Lost Creek, seeping water travels through the dam, it freezes on the downstream face and, because of the expansion caused by freezing, spalling occurs. In 1985, a thickness of 30 cm of concrete had been lost in some areas, and in many places the remaining concrete had very low strength for a more than 30 cm depth. Following extensive investigations from 1985 through 1994, among seven seepage control rehabilitation alternatives considered three were retained because compatible with the lowering of the reservoir allowed during rewinding of the generator at the powerhouse: upstream geomembrane installed mostly underwater, downstream drainage system covered with gunite or shotcrete, downstream buttressing with RCC. The geomembrane system was selected because it had the most favourable cost/benefits ratio, and involved the least environmental impact. Full-face repair was imperative in this project, due to the large loss of concrete that had already occurred at the dam.

The waterproofing liner is a flexible composite material consisting of a 2.5 mm thick Sibelon® geomembrane providing watertightness, and of a 500 g/m^2 nonwoven geotextile for puncture protection and dimensional stability. The geomembrane is placed on a full-face drainage geonet, doubled at bottom to form a longitudinal drainage collector conveying water to a transverse pipe discharging downstream. The face anchorage system for the geomembrane consists of an assembly of two stainless-steel profiles, which also act as free-flow vertical drainage pipes conveying water to the bottom collector, and are detailed further on.

The geomembrane works coincided with the rewinding of the generator at the powerhouse, which allowed drawing down the reservoir about half the maximum water depth without suffering an additional loss of energy production. Installation was accomplished for the dewatered part using suspended platforms anchored on the crest of the dam, and underwater using barges moored in the reservoir. Four standard 2.05 meters wide geomembrane sheets were assembled to prefabricate 8 m wide panels, which allowed to expedite installation underwater. The spacing between vertical anchorage lines is 3.7 m in the section above water, and 7.4 m in the submerged section by water. Closer spacing in the top area (Fig. 1 at right) where the geomembrane is always exposed is meant to maintain the liner stable on the upstream face, resisting dead loads, wind, and wave action. In total, 2,800 m^2 of geomembrane installed.

Waterproofing works were executed in 1997. Acceptance criteria were drainage flow < 1.58 l/s, and a maximum 3 m height of water standing behind the geomembrane liner. Drained water and piezometer readings, and a dry downstream face, show that the geomembrane continues to perform as designed. The seepage flows have remained < 0.02 l/s since 2002. More details on the project and on its performance have been published by the owner (Zancanella et al., 2013).

Figure 1. Lost Creek. At left, the grey geocomposite sheet under placement over the black drainage geonet, and the first components of the vertical anchorage assembly. At right, the anchorage profiles waterproofed in the dry section. The closer spacing of profiles in the top area can be seen.

When full-face lining is unpractical for operational or economic reasons, or when leakage comes only from a portion of the dam, a different approach can be used. This was the case at Turimiquire 113 m high Concrete Face Rockfill Dam (CFRD) in Venezuela, used for potable water supply. Already one year after impoundment, in 1988, the dam started leaking and, despite regular repair works carried out over time, leakage kept increasing, eventually reaching 9,800 l/s. Underwater investigation ascertained as main causes of leakage diffused cracks and two craters in the concrete slabs, located in a large intermediate area of the dam that was always submerged. The owner decided to carry out rehabilitation works in a staged manner, starting with the most leaking parts. The geomembrane system was designed specifically in view of future additional areas to be lined. The first stage took place in 2010/2011, maximum diving depth exceeded 50 meters only occasionally, which allowed surface supplied diving. The first-stage rehabilitation works, marked in dark grey in Fig. 2 at left, covered about 20% of the dam face, and allowed decreasing leakage to 2,400 l/s, well below the target leakage for this first intervention, which was 3,000 l/s. This leakage reduction allowed to maintain the dam in service and to gather new funds for continuing the rehabilitation. The second stage works in 2016/2017, marked in red in Fig. 2, covered an additional 5% of the upstream face.

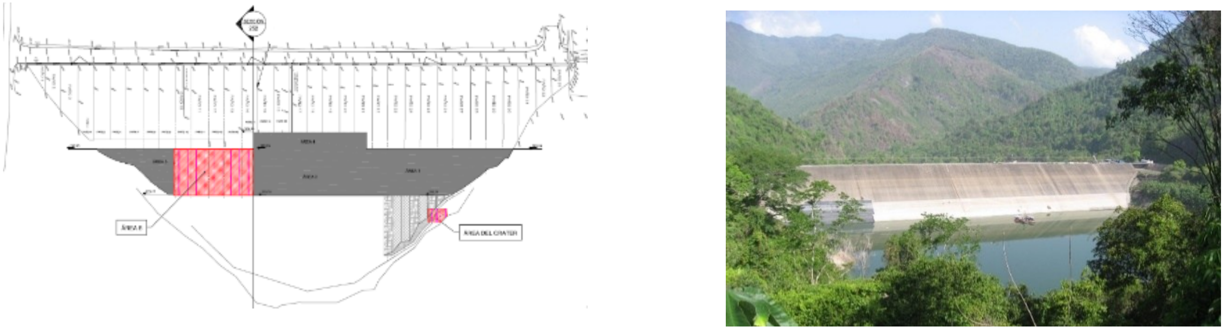


Figure 2. Turimiquire. The grey (2011) and red (2017) areas object of waterproofing works.

The political situation in the country impedes to have updated information on the evolution of the project. To the best of the authors' knowledge, no additional areas have been lined.

Studena buttress dam in Bulgaria is an example of another approach to underwater repair, that of maximising the efficiency of the geomembrane system while meeting the operational needs of the owner. The dam, 55 m high and 259 m long, located in a seismically active area, is used for potable and industrial water supply, for irrigation, for regulating the water of the Struma River and of its tributaries, for flood protection, and for hydropower. The harsh climate, with snowfalls beginning mid-October, snow cover persisting from 100 to 200 days, snow depth from 1 to 3.4 m, frequent ice formation and freeze/thaw cycles, required protecting the concrete face with a shotcrete layer. Nevertheless, after about 50 years of operation the dam and its appurtenant structures were badly deteriorated, with blistered shotcrete no longer attached to the concrete, cracks on working joints, vertical cracks, and disturbed structure of the concrete, visible in the zones where the shotcrete was detaching. To prevent a critical situation that could later threaten water supply and require more expensive works, and to extend the functional life of the dam by at least 50 years, the Bulgarian Government decided to implement a complete rehabilitation project, financed by the World Bank. The works had to be carried out in conditions that could guarantee the safe and proper technical operation of the dam and of appurtenant structures, while providing continuous water supply without affecting the quality of the water. This meant that most of the works had to be carried out underwater, avoiding working in the sediment layer and creating turbidity. The geomembrane system was installed from the crest, at elevation 843.3 m, to elevation 814.0 m, with underwater works from elevation 838 m downwards (Fig. 3 at left).The geomembrane liner, of the same type used at Lost Creek, was deployed on a 2,000 g/m^2 cushion geotextile placed over excessively aggressive rough areas. The complex geometry of the dam required a sophisticated anchorage system (Fig. 3 at right).

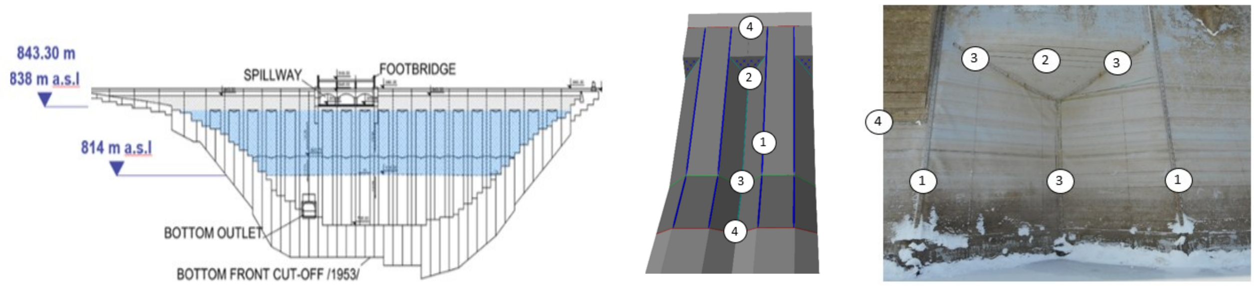


Figure 3. Studena. In grey, area lined in the dry, in blue, area lined underwater. Tensioning profiles (1, in blue), point anchors (2, in black), batten strips (3, in green), and perimeter seals (4, in red).

The tensioning profiles adopted at Studena (Fig. 4 at top) are a new patented system: while at Lost Creek (Fig. 4 at bottom) the tensioning effect was essentially achieved by the installation procedure, at Studena the tensioning effect is achieved by the geometry of the two profiles, like what happens with the tensioning profiles used in dry installation.

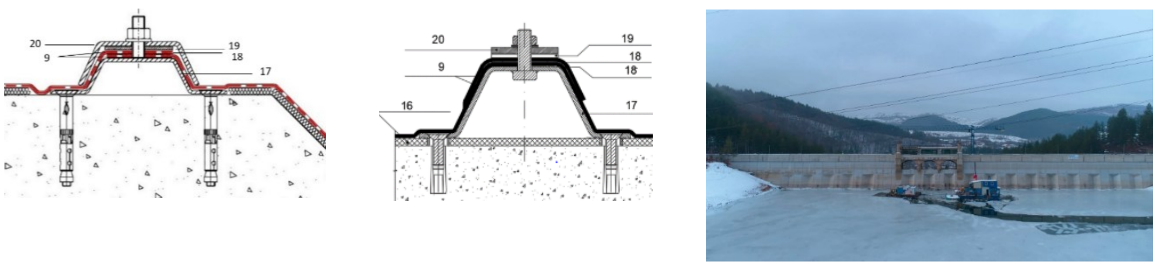
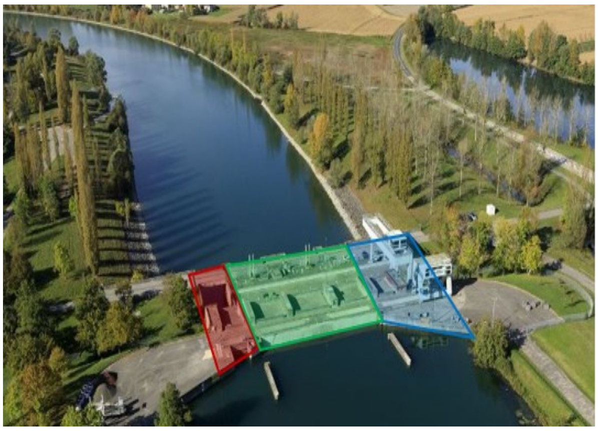


Figure 4. At left, the tensioning profile adopted at (top) and Lost Creek (bottom. At right, works under completion at Studena.

2.3 *Local repair*

Repair of failing joints or cracks or small defective areas is made with a patented external waterstop. The waterproofing liner is supported over the joint/crack/cavity by one or more flexible or rigid layers, and sealed at the periphery by a perimeter seal watertight against water in pressure, of the same type adopted for face repair. The most recent project of this type was carried out in 2022 at Brégnier Cordon, a 17 m high dam on the Rhône River in France, a composed structure consisting of a left block housing the outlet for floating debris (in red in Fig. 5), of a central block with the powerhouse (in green), and of a right block housing the annex building (in blue). The external waterstop was installed underwater at joint BC3, between the powerhouse and the annex building, from the top of the parapet wall (elevation 219.50 m) down to the bottom (elevation 208.20 m), after removal of the rockfill channel bottom protection and silty fill. The challenges of this project were a complex geometry with 90° and 45° corners and a large opening, the speed of the water and the suction ascertained by a previous inspection, the presence of large floating debris with branches weighing up to one ton; these conditions called for ad hoc precautions before and during installation, for robust materials, and for a suitable cover layer to protect the geomembrane from possible heavy impacts



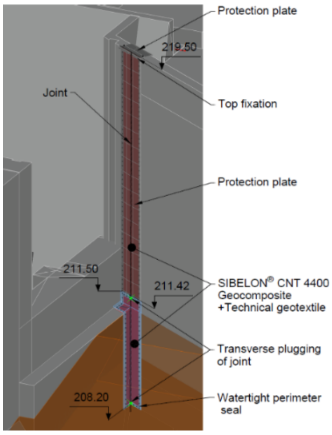


Figure 5. Brégnier Cordon. The blocks of the dam, and joint BC3.

Very flexible materials were needed for the sacrifice and waterproofing layers to adapt to the geometry and to the possibility of future structure movements; at the same time, the sacrifice layer had to be strong enough to support the waterproofing liner and avoid it collapsing into the large opening. The waterproofing liner is of the same type adopted at Turimiquire, the sacrifice layer is a high-tech geosynthetic with a high elasticity modulus. From crest to the elevation where the geometry at the joint changes, the waterproofing liner is covered by 50 cm high stainless-steel plates, designed, connected and anchored to allow opening/closing of the joint; from there down to bottom, the waterproofing liner is covered by a nonwoven geotextile, to provide protection in the event of strong impacts or puncture by the rockfill subgrade. Most of the installation was carried out underwater by surface-supplied divers, after securing of the underwater work area, necessary to carry out works concurrent with hydropower production.

3 UNDERWATER GEOMEMBRANE SYSTEMS IN FLOWING WATER CONDITIONS

Flowing water conditions require a robust anchorage system to resist the water speed, and the under-pressure due to water flowing under the geomembrane liner in case of accidental damage. Installing many mechanical anchorage lines involves relatively long diving times and high costs, and may be impossible because of the water speed. The alternative is a geomembrane system incorporating a ballast anchorage, the Sibelonmat®. Sibelonmat® is a double-geomembrane mattress, prefabricated in a factory, with a bottom robust geomembrane providing watertightness, and a top geomembrane confining the grout that, injected at site into the mattress, anchors the mattress by ballast, keeping it stable on the slopes/invert of the canal. Each mattress incorporates an internal device to connect the two geomembranes, watertight hoses to inject the grout from the dry, and heavy-duty zips, attached in a flexible way at the edges to allow adapting to the irregularities of the canal and compensating possible misalignments between mattresses. Joining of adjacent mattresses is made underwater by pulling the zips with bespoke equipment and unmanned procedures; divers are employed when the water speed is low, for control of underwater placement if needed. Different from existing geotextile mattresses whose watertightness is granted by the concrete filling, the watertightness of the Sibelonmat® is granted by the bottom geomembrane, capable of adapting to irregular subgrade and resisting settlement and differential displacements. The top geomembrane impedes water pollution.

After a few years of study, testing in the laboratory and full-scale testing in real structures, both in still and flowing water conditions, the Sibelonmat® was adopted at three pilot projects, in Egypt and Italy for irrigation canals, and most recently, in 2020, at the Kembs embankment, which is part of the Grand Canal d'Alsace, a 150 m wide, 8-10 m deep and 52 km long navigation canal in France. The Sibelonmat® at Kembs was installed on the existing deteriorated concrete facing of one embankment, over a length of 50 m, from crest down to about elevation 236.3, i.e., on about 28 meters of slope, covering 80 slabs and spanning 26 vertical and horizontal joints, while the canal was in full operation. Five panels, each 10 m wide and 28.4 long, were designed and prefabricated to waterproof the area, in total 1,445 m^2.

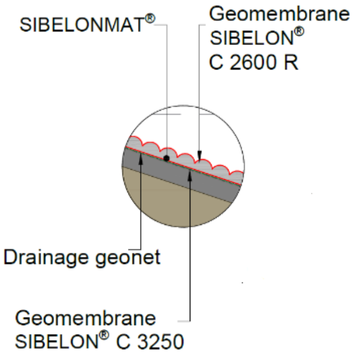


Figure 6. Kembs. Cross section of the Sibelonmat®, rolled void mattresses ready for installation (the drainage geonet is the black material, and the integrated zips is visible along the edge of the mattresses), and the Sibelonmat® deployed on the slope with navigation ongoing during underwater works.

The Sibelonmat® mattresses were fixed at top by a stainless-steel seal watertight to rain and waves. The side peripheries have a standard watertight stainless-steel perimeter seal, the bottom perimeter seal is an L-shaped stainless-steel profile that also acts as support for the filled mattress. Adjoining mattresses were joined by pulling the zips from the dry. After the panels had been joined, via the integrated grouting hoses the hollow space between the two geomembranes was injected from the crest with cement grout, thus preventing loss of cement in the water, providing a solution totally respectful of the environment. The project was awarded the 2022 Golden Innovation Award by ICOLD, the International Commission on Large Dams. The first results for the Sibelonmat®, as monitored by the owner EDF – Electricité de France, are encouraging and confirm the interest in developing this kind of technique.

A recent development of the Sibelonmat® is the Sibelonzip®, a single-geomembrane panel equipped with heavy-duty watertight zips along the edges and ballasted with an external ballast; Sibelonzip® has been conceived with the objective of providing a geomembrane system for underwater installation in canals with a performance similar to the one of the Sibelonmat®, at lower costs. Sibelonzip® better adapts to the uneven shape of riverbeds, anchorage against the dragging force of water flow is made by ballast with soil or stones placed on the bottom of the canal and on the top part of the slopes. The limited area of the ballast leaves a large surface of the geomembrane exposed, thus increasing the water flow. The Sibelonzip® panels, connected underwater with the zips, form a continuous liner compliant with health and safety standards and environmentally friendly: no water pollution, no impact on fauna, the soil or stones used for ballast allow to maintain a very natural "green" aesthetical appearance. Extensive performance testing carried out on all components of the system testifies its great potential for future successful applications.

4 CONCLUSIONS

Dam safety will become increasingly important issue as dams age and climate changes. Underwater geomembrane systems help ensuring the safety of dams by restoring and maintaining long-term watertightness even when the dam cannot be dewatered. Their efficiency and reliability, and their capability to adapt to different problems, situations and conditions, at any depth, and even in flowing water, has been proven by successful projects worldwide.

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Application of Environmental tracers in Reservoirs Leakage/Seepage studies

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Abstract

Identification of leakages in large water reservoirs is of great importance from view-point of both safety and economy of their exploitation. Large variety of water reservoirs encountered in hydro-engineering and industry call for adaptation of investigation methods to their specific features. Any attempt to eliminate leakages, has to be preceded by precise location of its occurrence and estimating the leakage magnitude.

While solving such problems, the use of environmental tracers substantially facilitates and speeds up both the identification of location and relative comparison between leakages in question. Therefore, number of methods based on the tracer techniques have been developed throughout the world and successfully implemented for this purpose. These are useful in locating leakages occurring both in bottoms or walls of water tanks as well as in embankments and dams.

In this study, isotopes and tracers were used to identify the leakages in the Shehjad Dam which is a major Irrigation Project, located at Dangbaroda village Gram Panchayat in Talbehat block of Lalitpur District of Uttar Pradesh (India). The project is under the control of the Rajghat Nirman Khand, Irrigation and Water Resources Department (I&WRD), Lalitpur, Government of Uttar Pradesh. Leakages were noted on the left and right flanks of the reservoirs.

About 27 water samples were collected from the reservoir at multiple depths at an interval of 2 m. Samples were also collected from the river inlet, canals, leakage points, seepage gallery and groundwater in the surrounding area for the stable isotope analysis. It was observed that the isotope signatures of the reservoir water and leakage samples matched, which confirmed the leakage from the reservoir. The seepage water isotope values matched with the groundwater isotope values. Artificial tracer test using salt (NaCl) was conducted and the leakages were categorized into 2 classes: (i) Class 1- highly porous/connective (ii) Class 2- less porous/ weakly connective. After various investigations, it was observed that the leakage of water takes place through masonry block joints after rising of reservoir level, mainly at the location of joints of control room and masonry block.

Keywords: Environmental tracers, isotopes, reservoirs, dams, Shahjad, Lalitpur, UP

5 INTRODUCTION

From the view point of safety and economy, it is crucial to find out the location of leakages in large water reservoirs. For this, the precise location has to be marked. The tracer techniques have been developed throughout the world and successfully implemented for this purpose. These are useful in locating leakages occurring both in bottoms or walls of water tanks as well as in embankments and dams.

In such cases, interconnection tests are an integral portion of leak investigations from dams and reservoirs for providing proof of hydraulic connection between the point of injection and the measuring point(s). In addition to providing the arrival time, the interconnection tests allow determination of the passing curve, transit time, amount of tracer recovered, characteristics of the preferential flow path(s), and an estimate of the volume of fissures or cavities along the flow path. These tests required detailed planning and execution. These tests conducted after performing some of the other prelimanary observations.

The stable isotopes of water, deuterium (^{2}H or D) and oxygen18 (^{18}O), offer unique possibilities for investigating leaks in dams and reservoirs. The isotopes of a given chemical element are represented by AX, where X is the symbol of the element and A is the mass number. The atomic number of the element is incorporated on the element. The heavy isotopes of hydrogen and oxygen, ^{2}H (deuterium) and ^{18}O oxygen18), represent an integral part of the water molecule. This is a special characteristic, which differs from the dissolved tracers. From the chemical standpoint, water molecules containing any of these heavy isotopes behave as any other water molecule containing the most abundant isotopes. As a result, their behavior is ideal when compared with other tracers whose properties are different from those of water molecules. Water molecules containing these heavy isotopes are perfect tracers to assess interaction of water bodies during the hydrologic cycle.

A common way of expressing the abundance of certain isotope is by using the isotopic ratio, which is defined as:

R = Number of molecules of $^{1}HD^{16}O$ or $^{1}H_2{}^{16}O$ / (Number of molecules of $^{1}H_2{}^{16}O$)

The isotopic ratios are modified by processes during the water cycle, which provides a characteristic labeling of the water involved in the process. This natural labeling is used as a fingerprint of water molecules that allows the study of the origin and dynamics of different water bodies.

Isotopic Deviation: The isotopic ratios R of a given sample W are not reported as absolute number but rather they are expressed as a deviation of the same ratio from a standard S using the following equation:

$\delta S (W) = (RW-RS)/ RS *1000$ (1)

The delta value δ is adopted as a dimensionless number and expressed as per mil. Delta values can be negative or positive, depending on whether the isotopic ratio of a given sample is greater or lower than the standard. However, negative δ-values of waters are commonly found because the standard is the seawater. The isotopic deviation for deuterium and oxygen ^{18} is denoted as δD and $\delta^{18}O$, respectively.

The water at the leak downstream of the dam may have different origins, such as:

1) local groundwater;

2) water from the reservoir; or

3) a combination of reservoir and groundwater. There are cases when the presence of runoff

from recent rains combined with emerging groundwater makes the determination more complex.

Dam Safety Review Panel (DSRP) vide its inspection report of Shahzad Dam dated 21.12.2020 reported that leakages are observed at many places along downstream (D/s) face of right and left flank of Non-Overflow (NOF) block (Masonry dam) of Spillway. It has been observed by DSRP that on the non-overflow block, the pointing has been damaged on upstream (U/s) side and that may be the cause of the seepage through the masonry dam block. Overall condition of the dam was marked as poor under Category-II (Major deficiencies requiring prompt remedial measures) by the DSRP. If such leakages are left unchecked, may result in dam failure by various mechanisms. There is enormous pressure on dam operators to repair leaks without significant delays. Frequently, the need to reduce the risk of failure or control water loss has led to costly remedial repairs that are planned and executed without a complete understanding of the problem. A lack of appropriate leakage investigation and monitoring can result in repairs that are unsuccessful in controlling or reducing leakage.

It is in this context, a study has been undertaken to assess the leakage points of the reservoir using various natural tracers such as temperature, conductivity and environmental isotopes.

6 STUDY AREA AND METHODOLOGY

The study has been carried out for Shehjad dam. The dam is located in Lalitpur District, Uttar Pradesh (India) at 8031'22” East latitude and 24056'47” North longitude. The Google Earth map of the dam showing its water spread area is provided in Figure 1.

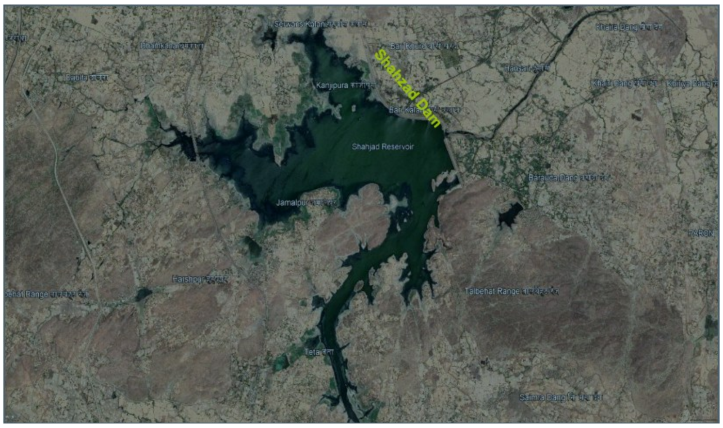

Figure 1: Google earth map showing Shahzad Dam and its reservoir

Shehjad dam, is a major Irrigation Project, located at Dangbaroda village Gram Panchayat, Talbehat block of Lalitpur District. The project is under the control of the Rajghat Nirman Khand, Irrigation and Water Resources Department (I&WRD), Lalitpur, Government of Uttar Pradesh. The construction of project was started in year 1973 and completed in the year 1992. The project is providing assured irrigation to 14577Ha area during Rabi crop and 3863Ha during Kharif crop and delivers irrigation to the 55 villages on the downstream of the dam through main branch, distributaries, minor and sub-minor etc. The project is built across the Shahzad River, intercepting catchment area of 955 km^2. Shahzad river is a tributary of Jamini River which finally joins the river Betwa. The catchment is mainly rain fed and has mixed land use pattern. The Govind Sagar Dam is located upstream of Shahzad dam at about 32 Km. The project comprises of homogeneous earth dam having the total length of 4160 m. The maximum height of the dam is 18.00 m. The dam has ogee crested radial gated spillway and the two head regulators exist near the spillway with a service gate and emergency gate. There are 80-80 m masonry structures on the left and right flanks where there is high leakage and same is also true for the seepage gallery.

The L-section and gallery RL of the Shahjad dam has been given in Figures 2 and 3.

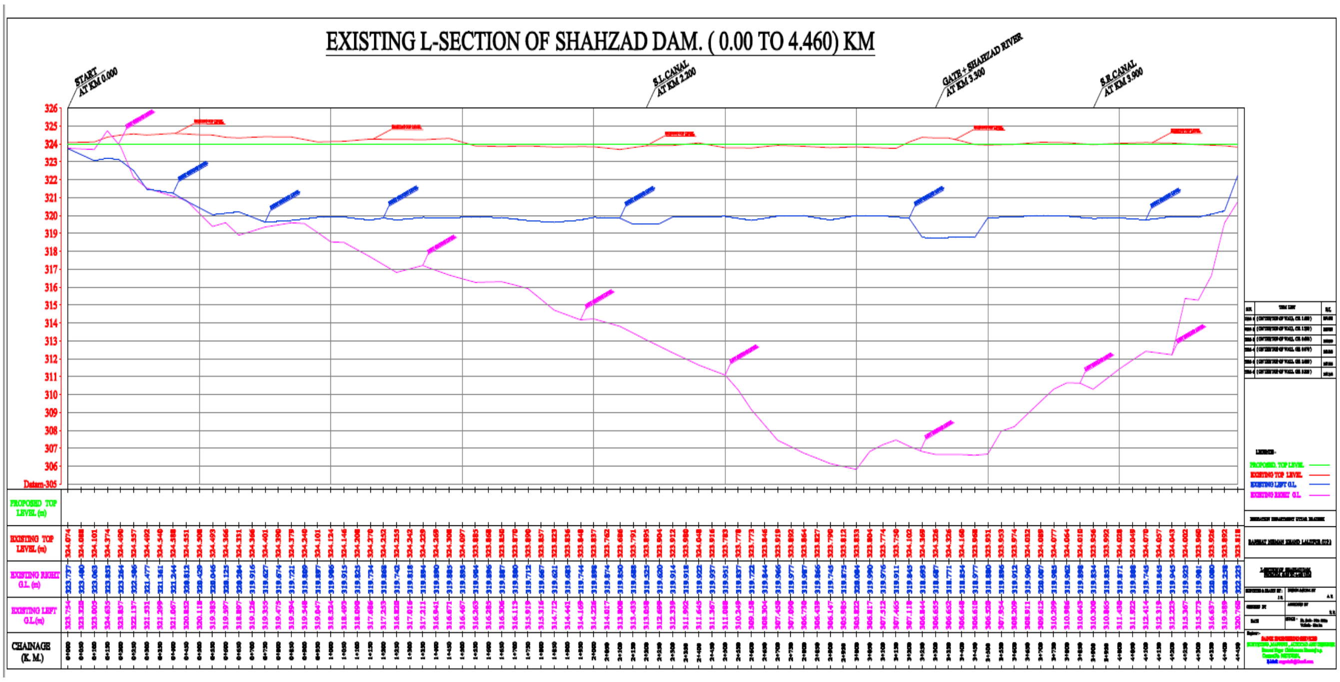

Figure 6.: L-Section of Shahzad Dam

The major part around the project site occupied by Banda alluvium, which is fine to medium grained and yellowish in color. The spillway has been placed in the river portion where pink colored, medium to coarse grained granite which are porphyritic in nature, are exposed in the immediate downstream of the spillway. The rock mass is very hard, compact and very dense in nature. The foliation is feeble, however four sets of joints, are distinctly visible. The rock mass belongs to Bundelkhand Granitoid Complex (BGC). Physiographically the entire area is almost flat, accordingly no major instability is expected around the reservoir periphery. The geological data such as foundation map, geological L-section along dam axis, weak features encountered during construction and grouting details could not be provided by the project.

6.1 Methodolgy

Based on DSRP observations on leakage condition, environmental tracer technique (interconnection technique) was proposed for the assessment of probable seepage zones/cavities. The Site was inspected with an objective to review potential leakage and seepage areas of the various structures of Shahzad dam project to ascertain possibility. During the site visit, project officials informed that the leakage is only prevalent in both the Right/Left flanks of Masonry NOF blocks (Masonry dam) of Spillway and the seepage gallery.

A seepage/foundation gallery of size of 1.5m x 1.8m x 2.1m has been constructed throughout the masonry NOF (masonry dam) blocks covering spillway structure. Curtain holes of 51mm dia and drainage holes of 75 mm dia have been provided for grouting and drainage purposes. During the inspection on 17 May, 2024, the gallery was observed to be completely filled with water up to the approach shaft (EL 311.2M) owing to which inspection of the same could not be carried out. But during the visit of 10-12 June, 2024, seepage gallery could be inspected and in-situ measurments for EC were taken and simulataneously samples were collected for isotope analysis. However, inspection was limited to right and left flanks of Masonry NOF blocks (masonry dam) of Spillway only. Some U/s portions of both the flank and entire D/s of both the flanks were thoroughly assessed and measurements for EC carried out and samples were collected for isotope analysis.

Seepage was reported as below: Left flank- 1452.60 ltr./min; Right flank-424.80 ltr./min *Isotope analysis*

Stable isotope analysis of water provides information about groundwater and its source of recharge. Similar to ^{1}H and ^{16}O forming water molecule as a $^{1}H_2{}^{16}O$, the other isotopes of hydrogen and oxygen namely ^{2}H and ^{18}O also form a water molecule. An excess value in the relationship between δD and $\delta^{18}O$ during water evaporation is observed, which was defined by Dansgaard (1964) as deuterium excess (*d*-excess = δD – 8.$\delta^{18}O$), with the global average *d*-excess value being 10. Because *d*-excess decreases during evaporative processes, it can be used to investigate the effect of evaporation as samples are collected from surface water sources like ponds/dams. Furthermore, *d*-excess is unrelated to the isotopic composition of rainwater, and waters with different δD and $\delta^{18}O$ values having the same *d*-excess value. When precipitation undergoes evaporation, the slope of the evaporation line will be less than 8, and consequently the *d*-excess will be much lower. To identify the possible interaction and recharge sources of groundwater, *d*-excess can be considered.

In the present study, a total of 27 water samples were collected for the isotope analysis from various sources consisting of surface water reservoirs, leakage, seepage gallery and groundwater (hand pumps and dug wells) during May- June, 2024. The samples were collected in acid washed LDPE (Low-Density Polyethylene) tarson bottles using the standard methodology.

The ratios of heavy stable isotopes ($\delta^{18}O$ and SD) were measured using a Dual Inlet Isotope Ratio Mass Spectrometer-DI IRMS (Isoprime GV instruments, UK) with automatic sample preparation units at the Nuclear Hydrology Laboratory of National Institute of Hydrology, Roorkee.

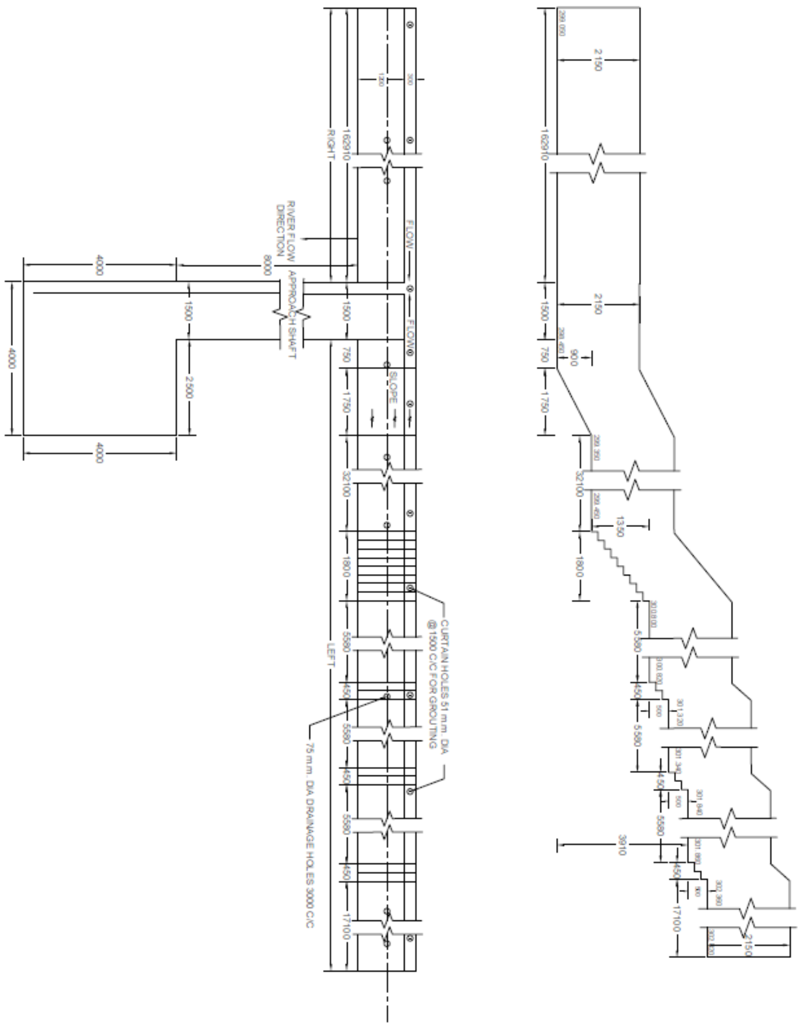

Figure 3: Gallery RL of Shahzad Dam

Temeprature and conductivity profiling

In addition to stable isotopes, water has two other natural tracers, which can be used to obtain information on the origin and location of the infiltrating points in the reservoir. In particular, these natural tracers are used for determining the origin of the groundwater emerging at the discharge points located downstream of the dam as well as groundwater flowing through the foundation and abutments, and investigating the dynamics of the groundwater emerging downstream. The natural tracers most commonly used for this purpose and discussed herein are the temperature and conductivity of water.

6.2 Water temperature

Water temperature is a good natural tracer in the investigation of leakages in dams and reservoirs. This is because most reservoirs (depths greater than 10 m) develop thermal stratification. In shallow reservoirs (i.e. less than 10 m), the thermal stratification does not take place and they exhibit uniform temperature because of the mixing effect derived from wind action. The thermal stratification in deep reservoirs is the result of the heat exchange between the reservoir surface and the atmosphere. During the summer time, the water in upper most layer of the reservoir heats rapidly and this absorbed heat energy is transmitted to the deeper layers. On the other hand, during the wintertime, upper water in the reservoir becomes colder causing overtun resulting in a more or less uniform vertical temperature profile.

Comparison of the temperature of the water emerging at the leaks, emerging downstream of the reservoir and the vertical temperature profile measured in the reservoir provides valuable information on the depth at which the water emerging in the leak is leaving the reservoir. In the initial stages of the leak investigation, field campaigns should include systematic measurements of temperature and conductivity in the reservoir and leaks. These simple parameters can provide valuable information, which can be used to define other complementary techniques to be utilized. In those cases when the transit time of the water between the reservoir and the boreholes or leaks is less than a few days, the temperature of the water does not change

significantly, as it flows through the ground. This is because the specific heat capacity of water is much higher than most natural materials found in the ground. As a result, the water associated with the flow path tends to maintain the same temperature as the reservoir at the infiltration zone. This is the case when the flow rates involved in the leak are high and transit times are short. However, water temperature may change when water infiltrated in the bottom of the reservoir mixes with groundwater of different origin.

6.3 Conductivity

Similar to water temperature, conductivity is a good natural tracer that can provide valuable information when evaluating leaks in dams and reservoirs. This is because, as in the case of temperature, reservoirs also develop saline stratification and therefore, deep waters exhibit higher salinity contents than water in upper layers. Measurements of conductivity in the reservoir, boreholes, and leaks are strongly recommended during the first stages of dam leakage investigations. It is common to obtain temperature and conductivity measurements simultaneously during the same campaign because both parameters are usually measured using the same instrument. The thermoconductivity probe is typically used to measure both the temperature and conductivity.

The measurements of conductivity are performed in the reservoir, boreholes, and leaks located downstream of the dam. In the reservoir, the measurements of vertical profiles of conductivity should be included in the program to investigate the salinity stratification. When submerged springs discharge high salinity waters, they will probably accumulate in the deeper parts of the reservoir. However, the opposite situation can also be found. These possible scenarios are described in detail by Plata and Araguás (2002).

Conductivity of the reservoir waters shows great variation with time and space. Frequently, reservoir waters show seasonal variations of conductivity with lower salinity water during the rainy period and higher salinity during the dry periods. These seasonal variations are often related to the origin of the water. During rainy periods, most of the water entering the reservoir is associated with surface runoff water and thus the salinity is lower. On the other hand, during dry periods the relative contribution of groundwater into the river will be higher and therefore, the salinity of the water entering the reservoir can be much higher. These variations can be of interest in the investigation of hydraulic connections between reservoir, boreholes, and leaks downstream of the dam. This is obtained by correlating the peaks in conductivity in the reservoir, boreholes, and leaks.

6.4 Salt Tracer

Tracer tests have been used for detecting the dam leakages since last 40 years and have a number of advantages, including low-cost, high efficiency, and provide direct hydraulic connectivity of the leakage points and provide data on the seepage. These tracers can be natural- based on inherent physical or chemical characteristics of the fluid, such as temperature, electrical conductivity (EC), total dissolved solids, and stable isotopes and artificial tracers-. salt, radioactive isotopes, and dyes (e.g., fluorescein). On the basis of an artificial tracer and its subsequent data analysis, help in (1) calculating flow velocity; (2) confirming the hydraulic connectivity, and (3) estimating relative hydraulic properties based on the concentration curves.

Strong ions are used as ionic chemical tracers. This includes elements present in ionic forms as cations and anions. One substance of great use in this group is salt NaCl. Salt has the advantage of having an almost ideal behavior and being easily encountered. The solubility of salt in water is about 350 g/l at 20°C. It can be injected in any water body as a saturated solution depending on the experiment. The main limitation for the use of salt as a tracer is that the chloride is always present in natural waters and sometimes in high concentration, which does not allow the use of salt. In general, salt can only be used with its concentration is less than 50 or 100 mg/l and the water volume to be labeled less than 1,000 m^3 but can be used at specific points to detect leakages or change in original salt concentration. Typical applications of the use of salt as tracer include tests measurements in leakages, boreholes, determination of small flows rates, and identification of groundwater discharge in closed reservoirs.

7 RESULTS AND DISCUSSION

From the results shown in Table 1, it is found that in dam/reservoir water, the $\delta^{18}O$ values range from -2.6 to -0.9‰ with an average of -1.7‰; in groundwater (handpumps), $\delta^{18}O$ values range from -6.1 to -4.1‰ with an average of -5.1‰; in leakage, $\delta^{18}O$ values range from -6.2 to -3.3‰ with an average of -5.2‰; in seepage, $\delta^{18}O$ value is found to vary between -2.8 to -1.9‰ with an average of -2.3‰ .

The values of δD in dam/reservoir water range from -26.6 to -15.8‰ with an average of -19.5‰; in groundwater (handpumps) δD values range from -49.0 to -33.6‰ with an average of -41.3‰; in leakage δD values range from -49.9 to -26.8‰ with an average of -41.5‰; in seepage δD value range from -26.8 to -20.2‰ with an average of -23.5‰ .

The values of $\delta^{18}O$ and δD clearly show that the groundwater (hand pumps) values and seepage water values are highly isotopically depleted as compared to other water source samples. This may be due to the processes involved before the water reaching to the aquifer and the recharge from the combination of sources like precipitation and surface waters. On the other side, the values of the reservoir, leakage water are isotopically enriched. The surface water bodies in arid/semi-arid regions are isotopically enriched in heavier isotopes due to continuous evaporation process.

The values of d-excess in dam/reservoir water range from -10.6 to -3‰ with an average of -6.2‰; in groundwater (hand pumps) d-excess values range from -1.1 to-0.4‰ with an average of -0.8‰; in leakage water d-excess values range from -2.6 to 1.5‰ with an average of 0.1‰; in seepage d-excess value found -4.6 to -4.9‰ with an average of -4.8‰. The low d-excess values suggest the effect of evaporation of the surface waters.

Table 1: Statistical summary of the results of the isotopic analysis of water samples

Water Source	**Reservoirs (‰) n= 9**			**Groundwater (‰) n=3**			**Leakage (‰) n=10**			**Seepage (‰) n=2**		
	$\delta^{18}O$	**δD**	**D-excess**	**$\delta^{18}O$**	**δD**	**D-excess**	**$\delta^{18}O$**	**δD**	**D-excess**	**$\delta^{18}O$**	**δD**	**D-excess**
Min	-2.6	-26.6	-10.6	-6.1	-49.0	-1.1	-6.2	-49.9	-2.6	-2.8	-26.8	-4.6
Max	-0.9	-15.8	-3.0	-4.1	-33.6	-0.4	-3.3	-26.8	1.5	-1.9	-20.2	-4.9
Avg	-1.7	-19.5	-6.2	-5.1	-41.3	-0.8	-5.2	-41.5	0.1	-2.3	-23.5	-4.8

D-excess of individual sample (any reservoir, leakage water, groundwater, seepage water etc.), estimated using the equation d-excess = δD – 8-$\delta^{18}O$, can be used to understand the original source of water (recharge source in the case of groundwater).

E- It is clear from the Figure 4 that a low to medium correlation exists between *D*-excess and $\delta^{18}O$ values which suggests that evaporation shifted the enriched $\delta^{18}O$ and reduced the *D*-excess values in case of surface water. The same has been observed in case of the water samples from the dam water. The reduced *D*-excess in surface waters is resulted from kinetic isotopic fractionation during phase changes, which could be ascribed to the differences in diffusion rates among H_2O molecule isotopologues. There is overlapping in the samples of leakge water, surface water and dam water which suggest a possible same source and groundwater recharge in seepage water.

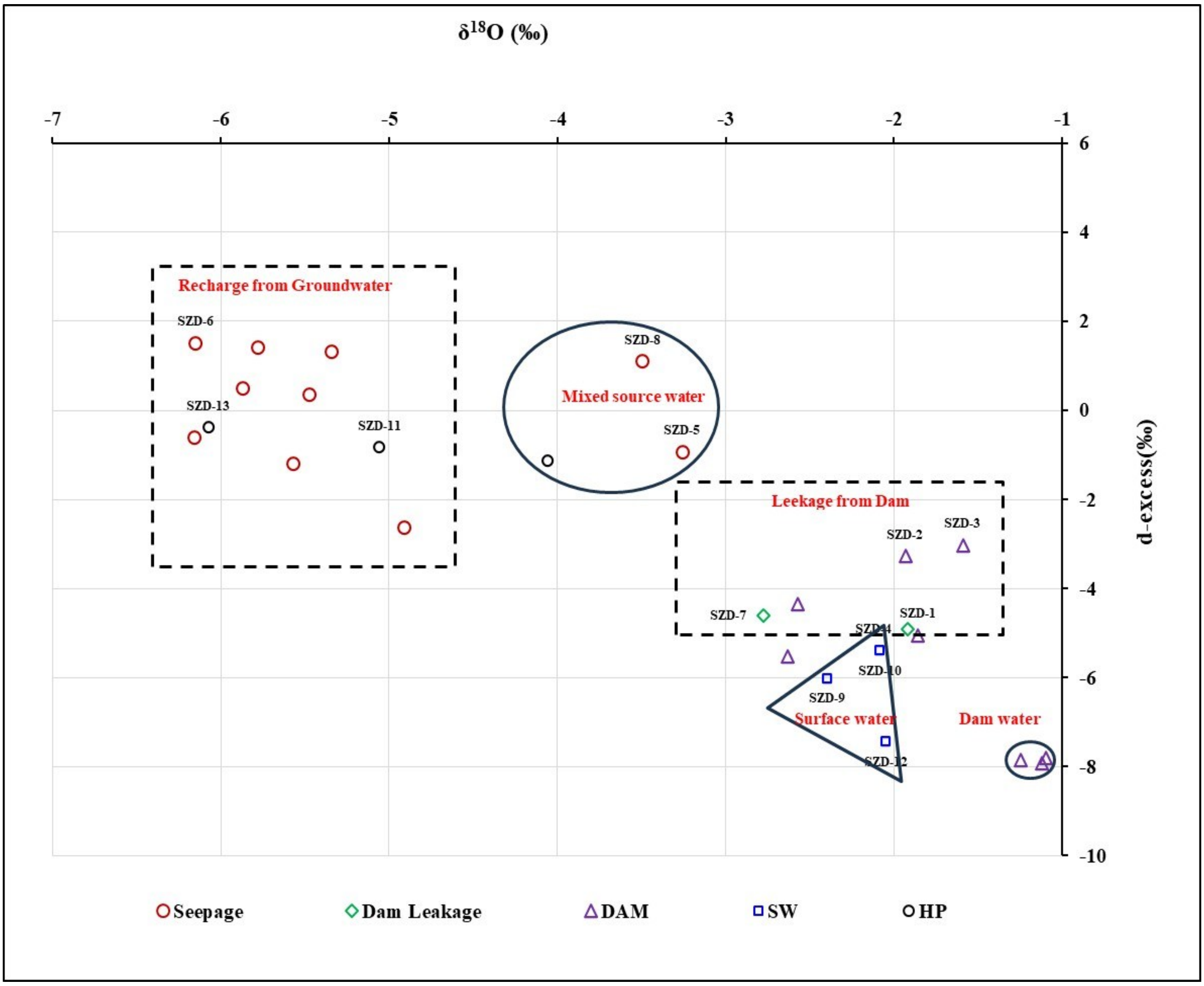

Figure 4. Plot between $\delta^{18}O$ and d-excess of water samples from various sources

Electrical conductivity (EC) that depends on the presence of dissolved ions, their concentration, mobility, valence and temperature with greater ± 0.01 precision as compared to density analysis where precision is ± 0.004 for ± $3x10^{-6}$ g/cm^3. The SI unit of conductivity is Siemen (1/Ohm) mS/cm or µS/cm at 25^{0}C and for change in temperature, compensation is applied as follows;

$$\text{EC } (\mu\text{S/cm}) \text{ at } 25^0\text{C} = \frac{\text{EC measured } (\mu\text{S/cm})}{1+ 0.019\ (t - 25)}$$

Hydrochemistry plays a vital role in assessing the dissolved salts and geochemical processes while environmental isotopes help in providing the information on the source and mechanism of groundwater recharge, interconnections between water bodies. In the present study, isotopes and EC have been correlated to find the related with various water sources.

Correlation between $\delta^{18}O$ v/s EC is shown in fig. 5 which indicated that the more enriched values of $\delta^{18}O$ resulted due to evaporative enrichment as dissolution does not affect the isotopic values (only EC value increases). It has been seen that groundwater has higher EC and depleted $\delta^{18}O$ values while reservoir samples have less EC and enriched $\delta^{18}O$ values. The signature of the reservoir water and leakage water matches as both have almost same EC and $\delta^{18}O$ values.

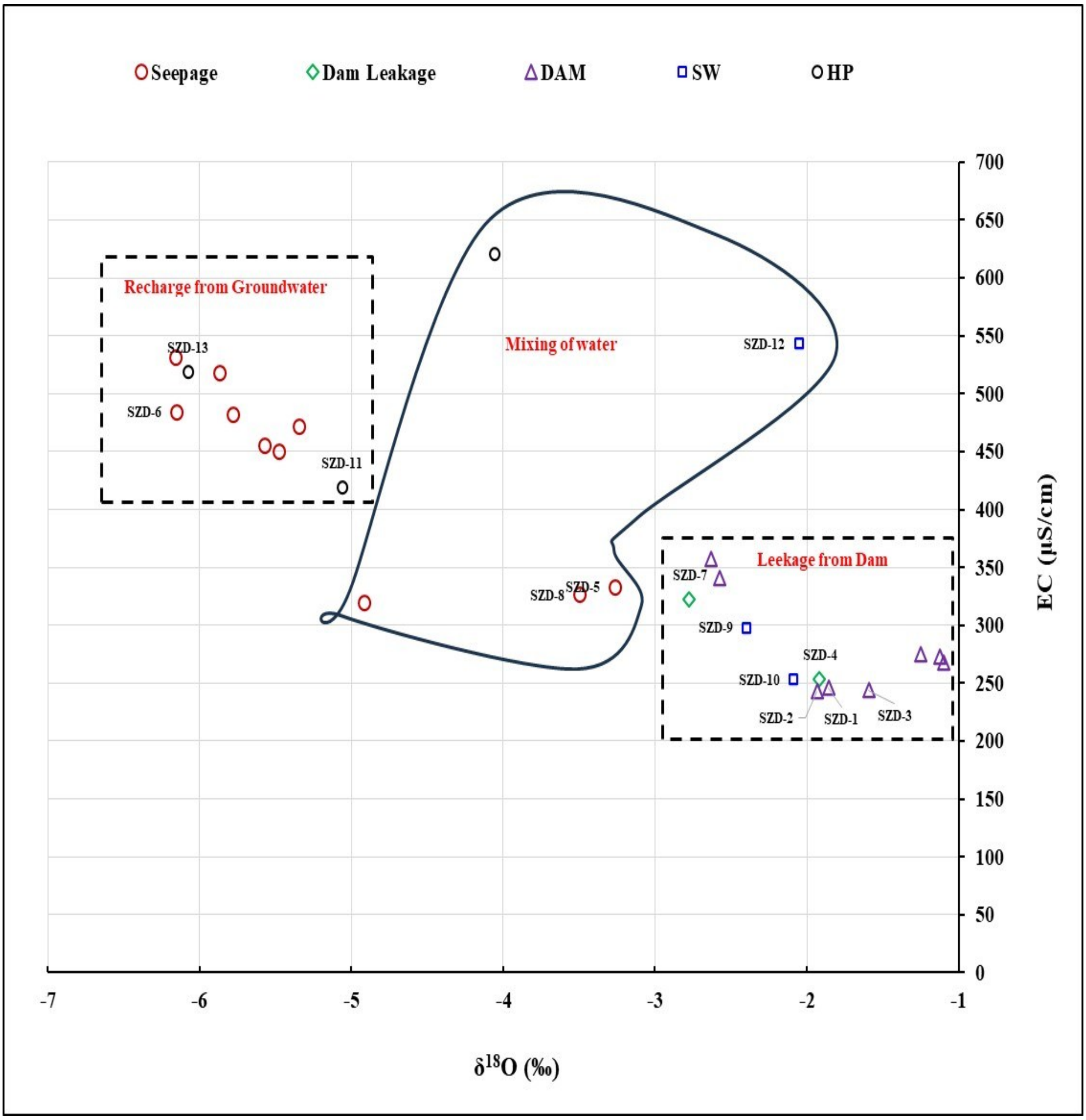

Figure 5. Plot between $\delta^{18}O$ and EC of water samples from various sources

In the present study, Temperature, water level and conductivity (TLC) profiling was done at a depth interval of 1 meter. From the observations, it has been found that the temperature from the depth of 6 m from the surface started decreasing and became constant at a depth range of 8-10m. On the other hand, the EC values started increasing after the depth of 6 m and recorded highest at the depth of about 10 m.

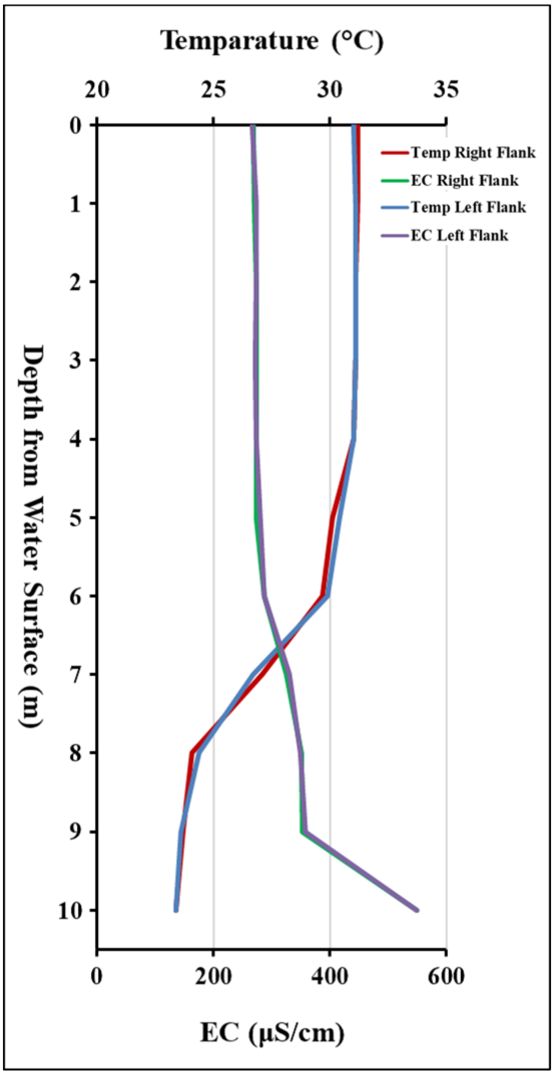

Figure 6. Depthwise temperature and and EC profiles

7.1 Tracer test Analysis

First experiment was conducted on June 11, 2024 but there were no signs of leakages in June 2024, the test has again carried out in the month of August, 2024. EC was measured in the seepage water and no change was observed indicating no connectivity.

The second experiment was conducted on August 17, 2024. Salt bags containing 20 kg of salts were immersed at a specific depth approximately at 318 to 319 MSL of the reservoir on left and right flanks, respectively. EC was measured in leakage water at different points (figs. 7) in left as well as right flanks at different points of the leakages and the measurements are shown in the figures 8 and 9.

Figure 7. Points for EC measurement on left and right flanks

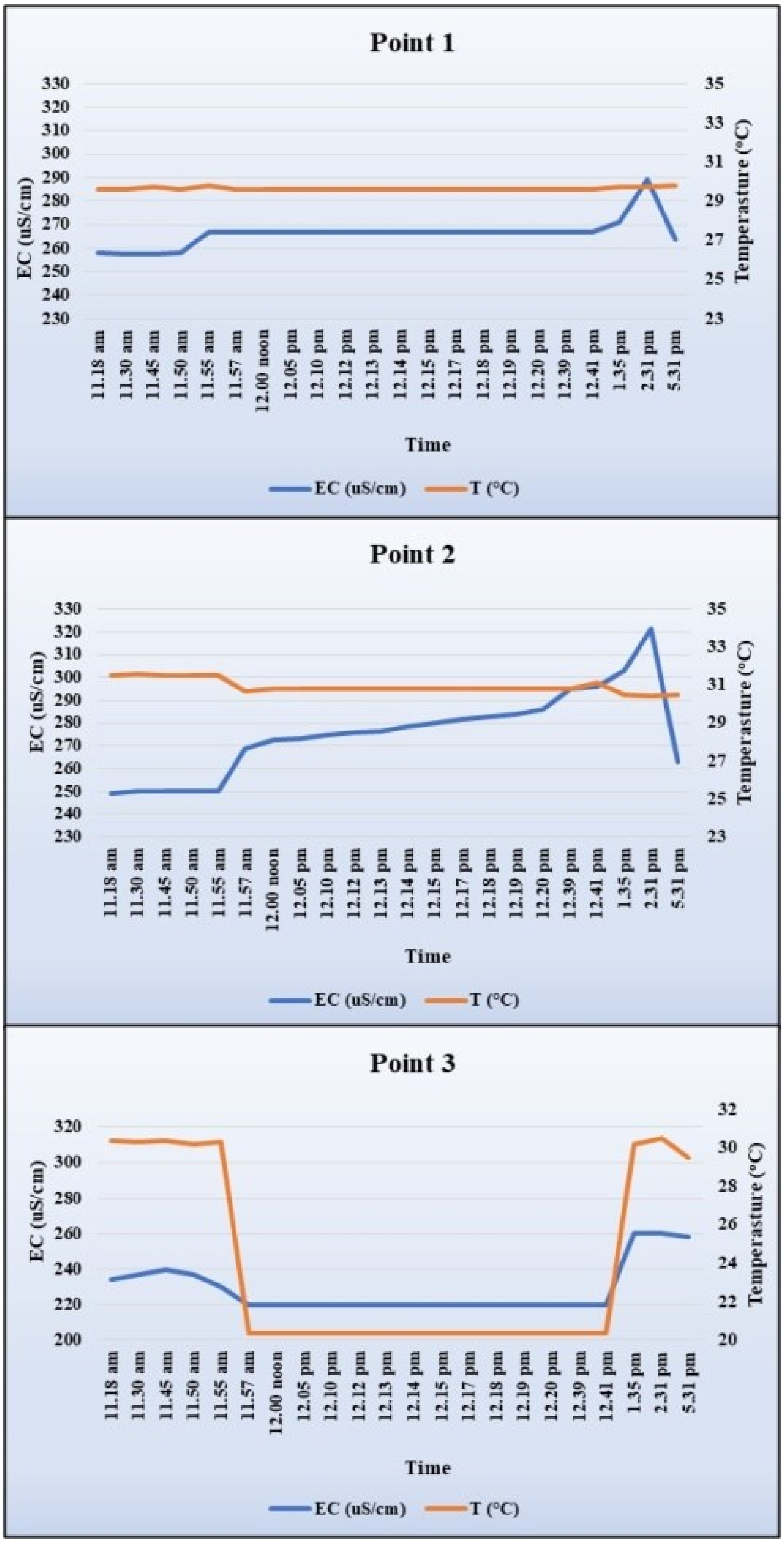

Figure 8. EC and temperature values of the water at the leakage points on left flank

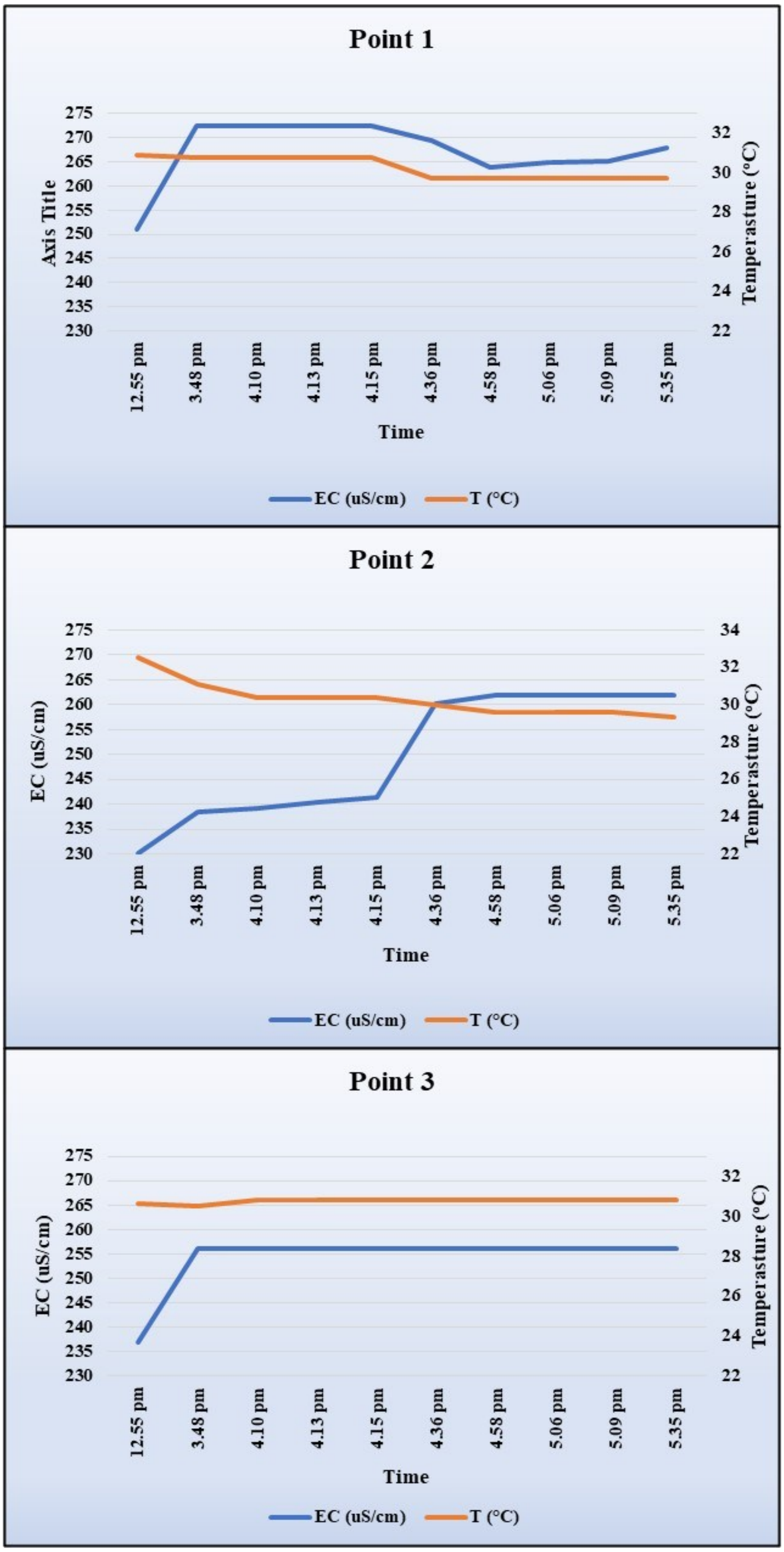

Figure 9. EC and temperature values of the water at the leakage points on right flank

There was a sudden rise in EC values of water at the leakage points in left flank after the immersion of salt bag and kept on rising from 250 µS/cm to 320 µS/cm in a time span for more than 2 hours (11.35

am to 1.30 pm) at point 2 as shown in fig. 8. The rise in values were very quick indicating highly porous and coonective hydraulically and is categorised in Class I.

On the other hand, the values on the right flank slowly changed as shown in fig. 9 from 240 µS/cm at 2.30 pm to 260 µS/cm at 5.30 pm. The fluctuation in values indicate less porous and less connective as compared to left flank and is categorised in Class 2.

On the basis of experiment the point source of leakage was found at point 24.94947°N, 78.522076°E at 318-319 msl depth in the reservoir and is the source point and the leakage point of same water is confirmed at the 24.949602°N, 78.522046°E and these are shown in fig, 10.

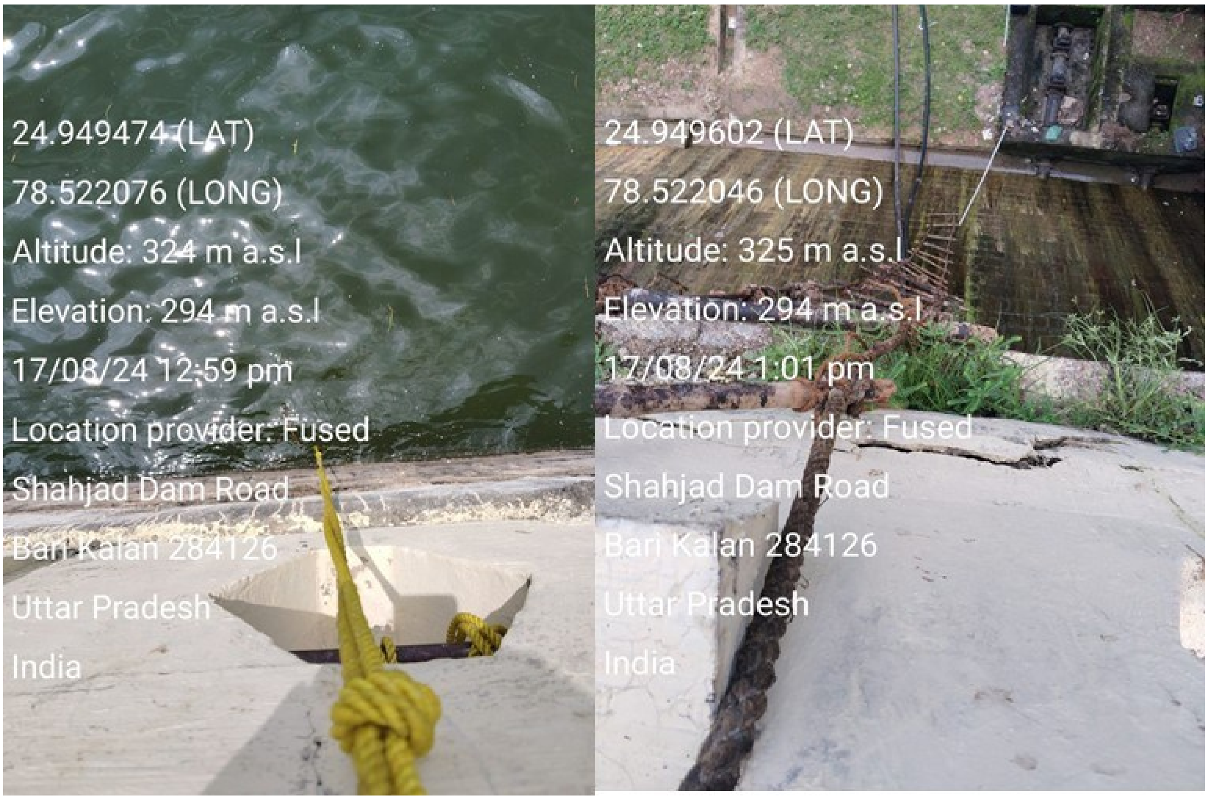

Figure 10. Source point and the leakage points on left flank

On the right flank, the response was there but not quick, so there may be small voids between 317-319 msl causing the leakage.

8 CONCLUSIONS

The temperature and conductivity determinations are combined with water sampling campaigns for determination of chemical composition and stable isotopes. The information collected allows formulating a flow hypothesis. From the model (Fig. 11), it is clear that the most of the leakage is from the points between 316-320 m

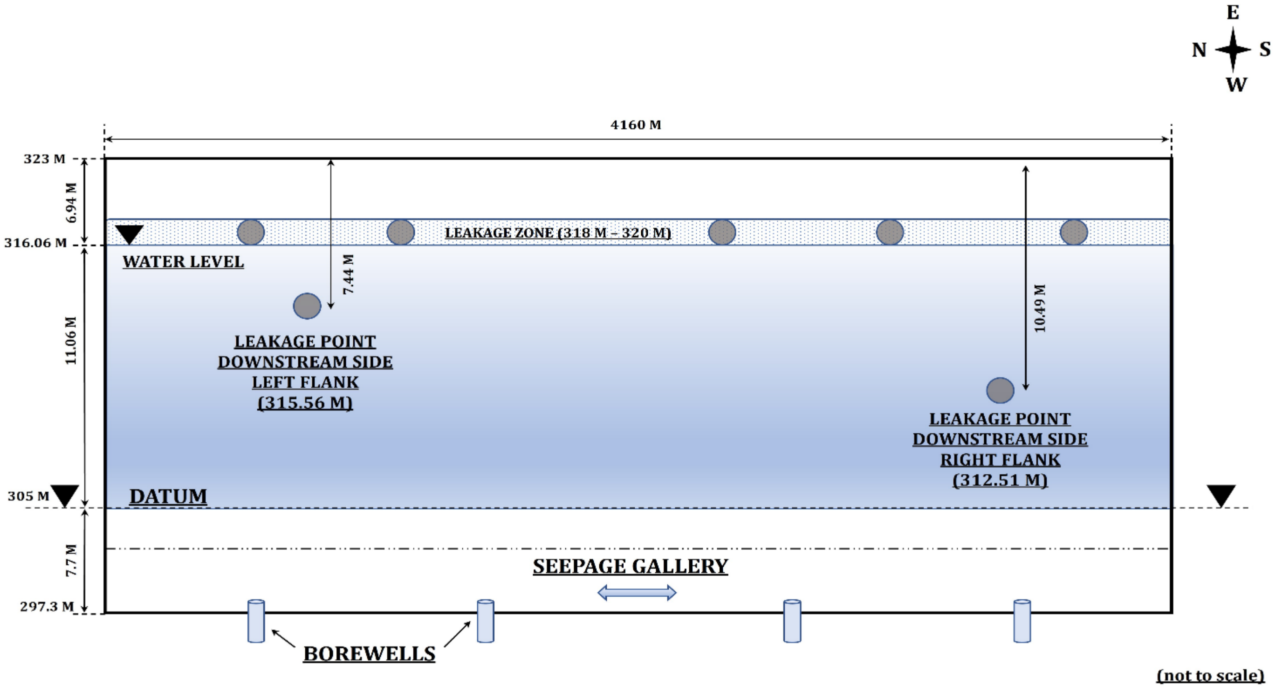

Figure 10. Conceptual model of the leakage points

There are strong evidences to suggest that the leakage water is the reservoir water while seepage water is mostly the groundwater. The study is very useful in illustrating environmental tracer methods for obtaining more precise and rapid information of the points of water leakages, for dam safety.

Acknowledgement

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Proceeding of International Conference on Dam Safety-2025
Latest Technologies for Rehabilitation and Dam Safety, 20-22 March 2025, Shimla

CHEMICAL GROUTING TO ARREST WATER LOSS THROUGH AGING DAMS USING SUNANDA MAKE SUNGEOGROUT

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8.1.1 ABSTRACT

Most of the dams in the country are more than 50 years old. The most common indicators of seepage in concrete and masonry dams are; wetness and seepage on the downstream face of the dam, and in inspection and foundation galleries. The main causes of seepage from ageing dam body is the continuously increasing porosity of the concrete and masonry along with hydrostatic pressure head exerted by the reservoir created by the dam.

Due to the seepage, the cementitious and other fine particulate matter gets washed away further increasing the seepage. Thus, seepage aggravates if left untreated properly. Grouting is the classical method of treating seepage. However, the grout has a tendency to get washed away in course of time if proper material and methodology is not adopted, requiring frequent grouting operation.

It is experienced that, leakages cannot be brought under total control and hence the grouting with cementitious materials is needed to be done periodically because of the continuous loss of cementitious material from the body of the dam. The loss of fines weakens the structure. In addition, the cement grout has limited penetration ability in the capillaries, pores in sub micron region compared to the chemical grouts.

Capillary and pores in mortar or concrete range from 100 Nm to few microns. The Gel pores are of size 0.5 to 10 Nm while larger voids (macro pores) are of several millimeters. The cement Particle size is 15000Nm, which obviously cannot penetrate through the capillaries, pores less than 15000 Nm in size. Hence subsequent to cementitious grout, chemical grout with particle size less than 5 Nm is to be used to get a complete impervious compact material.

8.1.2 INADEQUACY OF CEMENTITIOUS GROUTING

Most of the dams in the country are more than 50 years old. The most common indicators of seepage in concrete and masonry dams are; wetness and seepage on the downstream face of the dam, and in inspection and foundation galleries. The main causes of seepage from ageing dam body is the continuously increasing porosity of the concrete and masonry along with hydrostatic pressure head exerted by the reservoir created by the dam.

Due to the seepage, the cementitious and other fine particulate matter gets washed away further increasing the seepage. Thus, seepage aggravates if left untreated properly. Grouting is the classical method of treating seepage. However, the grout has a tendency to get washed away in course of time if proper material and methodology is not adopted, requiring frequent grouting operation.

To obtain a durable and high strength hardened cementitious grout it is necessary that, the grout is stable in terms of bleeding and sedimentation[R1, R2.] Sedimentation and bleeding causes incomplete filling of the voids, which creates paths for leach through the voids. Furthermore, the w/c-ratio of the grout is very high which results in the formation of a matrix which is full of over hydrated cementitious with very less binding capacity due to lack of CSH matrix and thus is susceptible for washout in subsequent leaching of water when the hydrostatic pressure increases due to raised reservoir level.

It is experienced that, leakages cannot be brought under total control and hence the grouting with cementitious materials is needed to be done periodically because of the continuous loss of cementitious material from the body of the dam. The loss of fines weakens the structure. In addition, the cement grout has limited penetration ability in the capillaries, pores in sub micron region compared to the chemical grouts.

8.1.3 Mismatch in sizes of pores and grout particles

Capillary and pores in mortar or concrete range from 100 Nm to few microns. The Gel pores are of size 0.5 to 10 Nm while larger voids (macro pores) are of several millimeters. The cement Particle size is 15000Nm, which obviously cannot penetrate through the capillaries, pores less than 15000 Nm in size. Hence subsequent to cementitious grout, chemical grout with particle size less than 5 Nm is to be used to get a complete impervious compact material.

To draw parallels such chemical grouts are used for soil stabilization which is akin to the internal matrix of aging dams. The properties of such grouts are established by us and many others which have proven enhanced properties mainly [R3.]

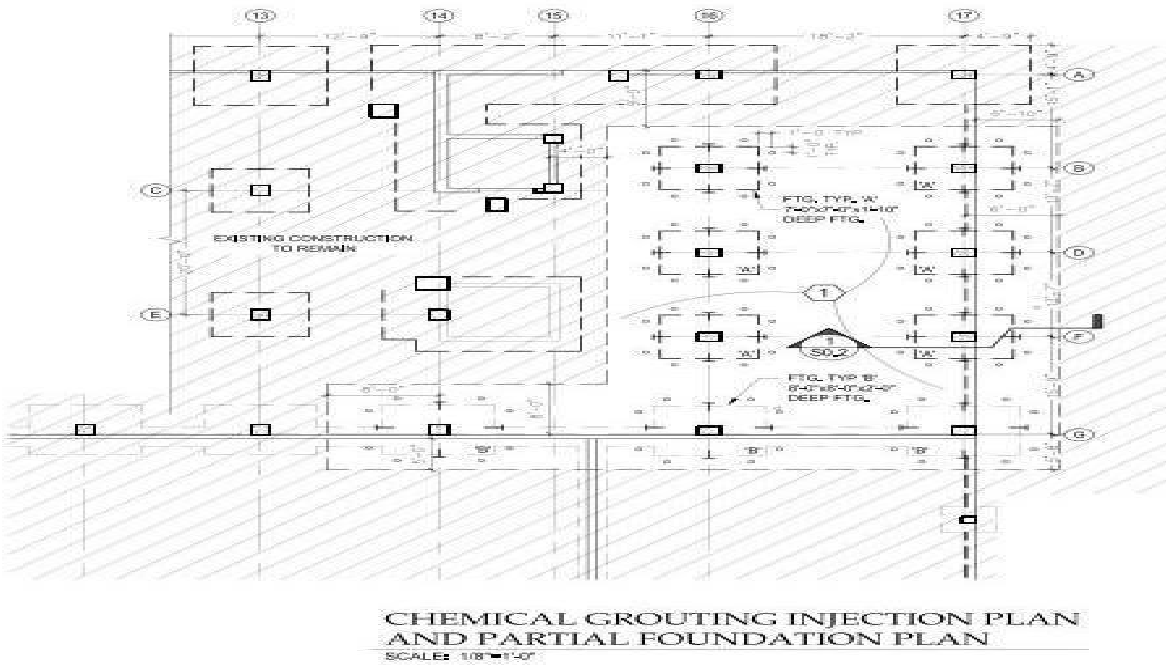
- Falling head permeability test
- Enhancement in CBR
- Bearing capacity of matrix

8.1.4 INDIGENOUS PRODUCT DEVELOPED

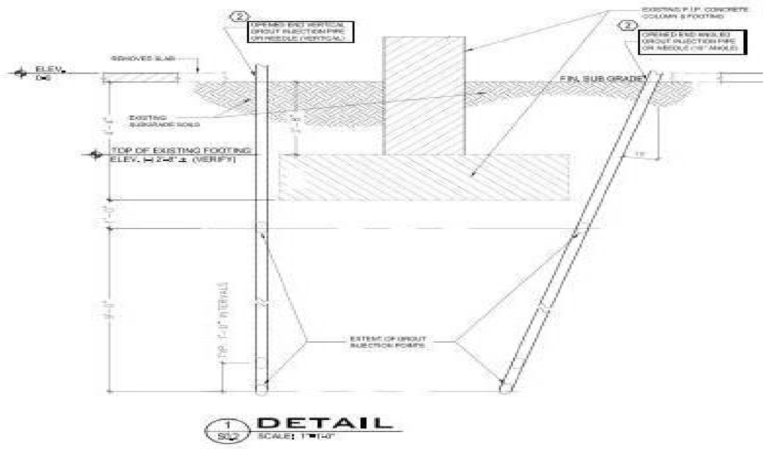
In a similar exercise of grouting done in Florida (USA) the grout [R4] was introduced through open ended pipes in soil matrix under the foundation in vertical as well as inclined position. The injection was done from the bottom of pipes and grouting was continued in stages upwards. In our case, we should do grouting till HFL for the embankment of the earthen/concrete dam.

GUIDING CASE STUDY

Increasing SBC where additional floors to be added to existing structure (Marriott Hotel) in Florida



Details of a vertical & inclined injection grouting



Sequence of Construction/Chemicals Grouting Plan as given by consulting engineers from USA[R4] for soil stabilization in summary is as under:

1. Saw cut and remove existing floor slab from this area for chemical grout injection preparation. Make sure not to disturb any underground piping, conduits etc. If any contractor shall check and verify this prior to start of grouting.

2. Insert open ended pipes or needles to inject chemical grout siroc starting at 9'-0 below the bottom of the existing footing. Inject minimum 40 liters of grout at each injection point. Raise pipes in increments of 1'-0. Injecting 40 liters at each point. Terminate injection at 1'-0 below footing bottom. Chemical grout to be used is 40 % Siroc # 132. Existing footing bottom 4'-8' below finished floor slab. Injection points shall be at locations as shown on each footing. At each location. The injection procedure shall be monitored by geotechnical engineer or independent testing lab retained by owner.

The same sequence of chemical grouting for strengthening and arresting leaks from masonry/concrete ageing dams was adopted as under.

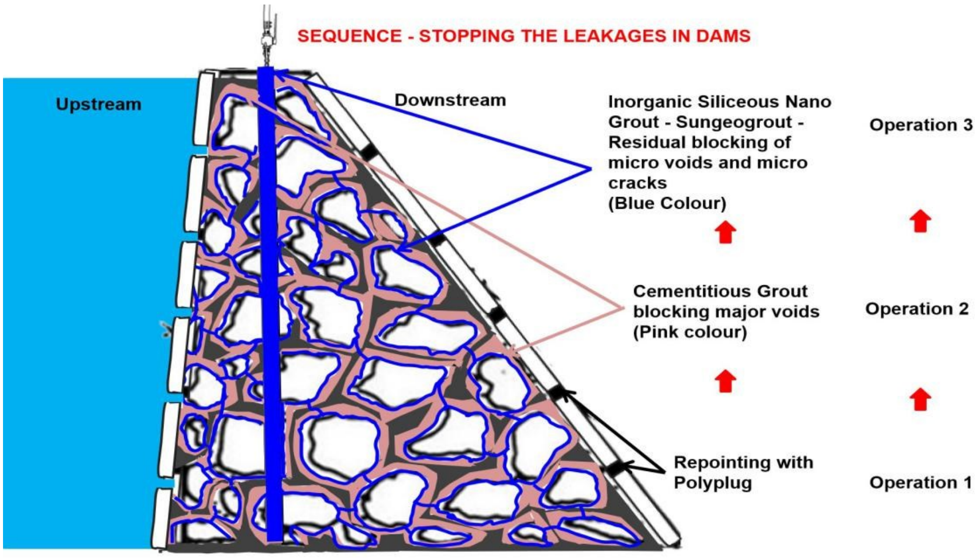
1. Drill the holes from the top of dam, complete cementitious grouting to achieve maximum possible impermeability for chemical grout injection preparation. Make sure that position of embedments parts, instruments, galleries and other openings are avoided. The contractor shall check and verify this prior to start of grouting

2. Insert open ended pipes or needles to inject chemical grout to a required depth of hole above a safe distance from existing gallery top & openings. Inject grout at each injection point. Raise pipes in increments of 3 M. Inject required quantity of chemical grout till refusal. Terminate injection at top of dam. Chemical grout to be used is "Sungeogrout". Injection points shall be decided on the basis of water intake test results. The injection procedure shall be monitored by expert engineer or independent testing lab retained by owner.

3. On the similar lines grouting of the solution with nano particles which is having particulate matter in the range of 1-100 nano meters[R5] is to be executed for the masonry dam sections as well. This will stabilize, consolidate, ensure inter particulate bonding & fill micro capillaries & pores and will make the dam impermeable. Sunanda Speciality Coatings Pvt. Ltd India (SSCPL) has developed a product named 'Sungeogrout' suitable for the problem.

The nano particles of this product have thorough penetration in the unfilled mass of the dam which was previously grouted with cementitious grout in order to take care of bigger voids & capillaries bigger than 15000 to 20000 Nm. Laboratory / site results have shown that cementitious grout reduced permeability up to an extent of 10 to 15 lugeons.

Hence subsequent nano particle chemical grout was introduced which reduced the hydraulic permeability substantially to 1.18 which is less by the order of 10 vis-a-vis cementitious grouting. Falling head permeability test[R6] conducted after this grout has shown permeability to almost zero Lugeons. The test is to be done in accordance with ASTM D 2434 as well as IS 2720 Part 17-1986.



8.1.5 SEQUENCE OF OPERATIONS RECOMMENDED:

i. Holes are drilled in the dam body. The size, number and the spacing of holes is as per the relevant IS Code and as per the site situation. Procedure for the same is as per Annexure I (Drilling for grouting of masonry/concrete dams).
ii. Conduct water intake tests as per Annexure II (for Water Intake tests for masonry/concrete dam)
iii. Cementitious grouting is done as per the IS Code. The procedure is given in Annexure III (Grouting of masonry/concrete dams)
iv. Again conduct water intake tests.
v. Carry out chemical grouting with nano particulate grout. SUNGEOGROUT is a two components chemical grout material. Mix Part A and Part B in the specified ratio in consultation with SUNANDA's Concrete Materials Consultancy Division.
vi. Grout must be continuously stirred during addition of the accelerator and fully mixed prior to grouting. Simple pumping equipment can be used to grout Sungeogrout of SSCPL.
vii. The gel time can be customized based on the site requirement by adjusting amount of accelerator added. Typical gel times achievable vary from 10 minutes to several hours. Site tests are recommended in consultation with "Concrete Materials Consultancy Division - CMCD" of Sunanda. Dye tests will give the travel time of the grout and the grout can be designed to suit the site requirement.
viii. Continue grouting till refusal.
ix. Conduct water intake test after grouting and setting of Sungeogrout. Minimum 48 hours are advisable.

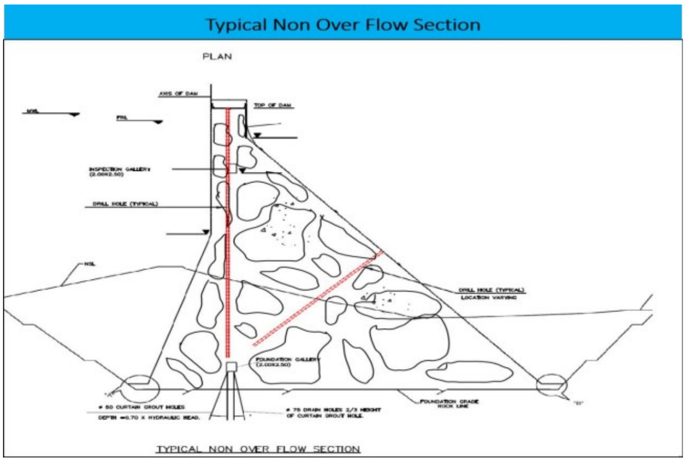


Figure 1 Drilling of Holes (Vertical from top or inclined from d/s face)

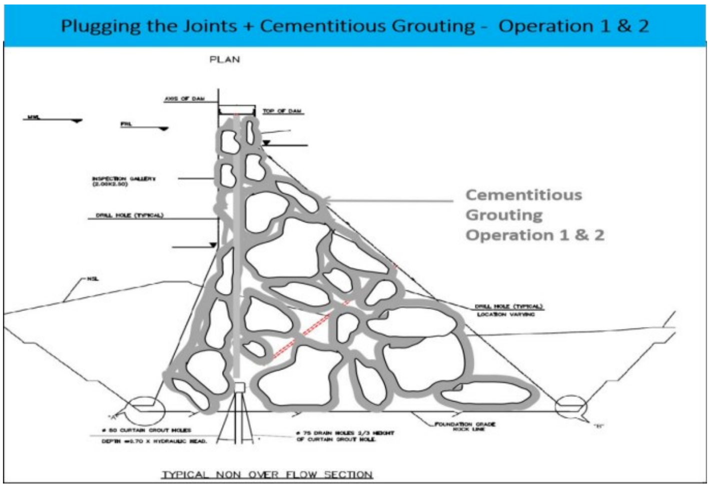


Figure 2 Cementitious grouting (from top or from d/s face)

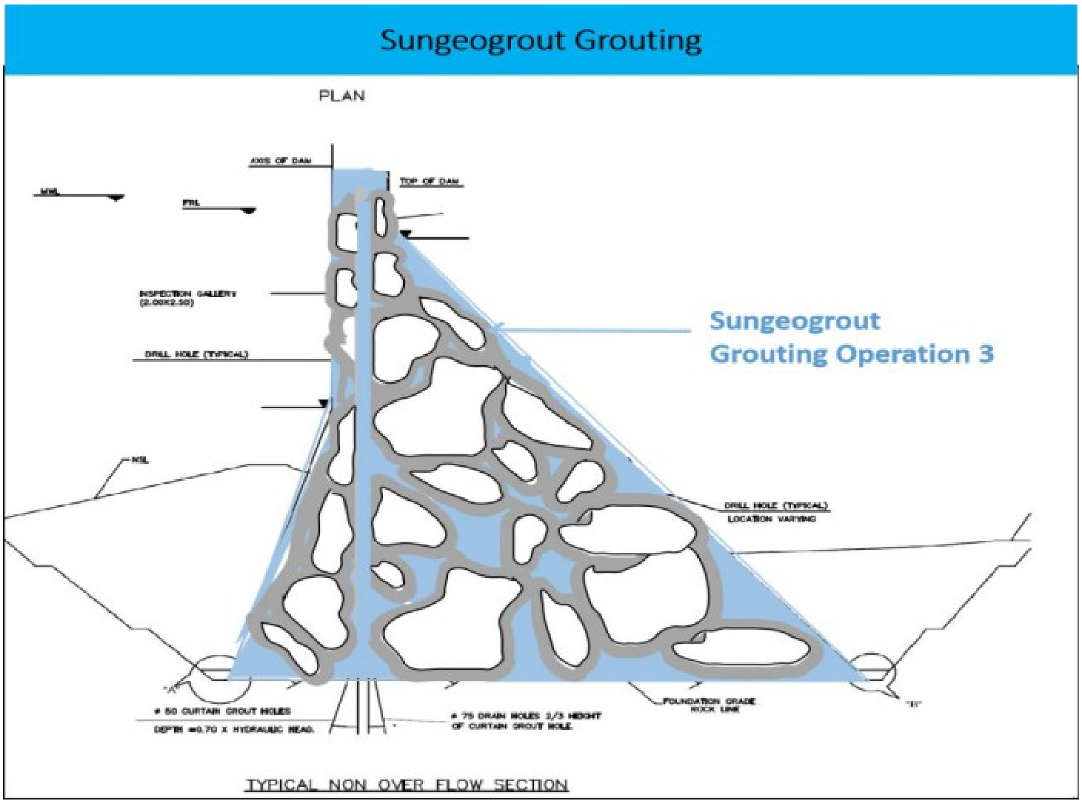


Figure 3 Nano grouting (from top or from d/s face)

8.1.6 CASE STUDY OF RANSAI DAM:

The water loss in Ransai Dam(Tal: Uran, Dist: Riagad, Maharashtra) before any treatment was more than 43.92 Lugeons on an average. Cementitious grouting brought down this to 10-15. However, as this is beyond permissible limit, chemical grouting (Sungeogrout) was resorted. After nano grouting with Sungeogrout the water loss was brought down to 0.515 Lugeons on the average. (Reference: RILEM paper) which is well within the permissible limits of 2.5 Lugeons.

The steps adopted were as below:

i. After the initial setting of cementitious grout material, the grouting with nano material was commenced in the same hole by redrilling the same hole to full depth, chemical grouting was started initially up to 3 M from bottom using packer system. (Refer point no 5 of Annexure II)

ii. The grouting was continued till its refusal.

iii. Pressure applied was 2 kg per Sq cm or 2 kg per Sq cm + hydrostatic pressure in the dam

iv. Similar procedure was continued for each subsequent stage of 3 m using packer system till grouting was completed up to top of OF/ NOF

v. Standards of Performance Testing: As per IS 11216 - 1985 - CODE OF PRACTICE FOR PERMEABILITY TEST FOR MASONRY DURING AND AFTER CONSTRUCTION STANDARDS OF IMPERMEABILITY: Masonry — Standard of impermeability aimed at shall be a water loss of not more than 2.5 and 5 lugeons in the upstream and downstream portions of the dam respectively.

vi. Results of Water intake test in Non Overflow Section

WATER LOSS IN LUGEONS		
Before Grouting	**Cement Grouting**	**Cement Grouting + SUNGEOGROUT**
82.404	15 to 20	1.18


Figure 4 Observation of dryness in inspection gallery in 2016 - Totally dry


Figure 5 Observation of dryness in inspection gallery in 2024 - Totally

8.1.7 CONCLUSIONS & RECOMMENDATIONS:

i. Chemical grouting is an established, effective technique for treatment of seepage in dams.

ii. Nano-particles of the grout improve the penetration capability of the grout.

iii. It is possible to explore various combinations of chemical grout reactants to enter into micron, nano and sub nano particle zones.

iv. Entering into the nano zone may enhance the performance of the grout in terms of unconfined compressive strength and overall economics. As a result chemical grouts can be injected into dam body containing voids that are too small to be penetrated by cementitious or other grouts containing suspended solid particles. This enhances the in-situ strength as well as overall integrity of the dam structure.

v. Chemical Grouts, once reacted, become a material analogous to sandstone and hence have no detrimental effect on the soil and ground water but have great binding factor.

vi. Chemical grouts have an adaptability over a wide range of applications in various types of dams.

vii. Nano grouts are the strongest and nontoxic in the existing range of chemical grouts.

8.1.8 REFERENCES

R1 IS 11293 (2018): Guidelines for the design of grout curtains (Part-2). Bureau of Indian Standards, New Delhi. 2018.

R2 IS 6066 (1994): Pressure grouting of rock foundations in river valley projects - Recommendations. Bureau of Indian Standards, New Delhi. 2023.

R3 IS 11216 (1985): Code of practice for permeability test for masonry (during and after construction). Bureau of Indian Standards, New Delhi. 2023.

R4 NKT ENGINEERS, INC - CASE STUDY - Increasing SBC where additional floors to be added to existing structure Marriott Hotel in Florida. FL 32709.

R5 Study of Particle Size in Natural and Technological Water Suspensions Using Photon Cross Correlation Spectroscopy with Nanophox - G. Oyungerel 1, G. Batdemberel 2, G. Munkhsaikhan

R6 1st International Institute of Concrete Technology(ICT), UK Conference on Cement and Concrete Technology - Concrete for the Modern Age Developments in Materials and processes.

9 ANNEXURE I

9.1 DRILLING FOR GROUTING IN MASONRY / CONCRETE DAM

1. TYPE OF DRILLING:

Drilling operation can be of two types.

i. Core or Diamond drilling.
ii. Percussion drilling.

A 46mm to 76mm dia. core or diamond type drilling shall be employed along the dam axis of dam. The spacing of holes initially shall be at 6m C/c. Lugeon tests in foundation bed (Water Intake test) will decide the need of further grouting the foundation at close interval or otherwise.

2. DRILLING PATTERN:

The grouting pattern is primarily decided to satisfy the design requirements to safe guard the risk of failure and to reduce quantity of seepage. This is achieved by grouting the masonry / concrete.

After observing the core recovery of bore hole in the dam body and results of water intake tests, the extents of grouting required can be estimated. Accordingly the size, depth and spacing for injection of grout hole is decided in such way that, the results are best at minimum cost. Thus the drilling pattern will vary according to grout intake.

i. **Location of Hole:** The hole shall be drilled in such a way that, there should be sufficient cover on upstream as well as downstream.
ii. **Spacing of Hole**: Split spacing method shall be adopted for grouting of dam body . In the first stage drilling and grouting shall be of holes is done at spacing of 12 to 15m c/c. The second stage consists of drilling of hole with spacing of 7.50m c/c. In the third stage hole shall be spaced at 3 to 3.50m c/c. Even after grouting of these holes the seepage is not reduced, the spacing of hole shall be drilled at 1.50 m c/c. All the holes drilled shall be along to dam axis.

3. METHODS OF GROUTING:

Grouting for dam is recommended from center towards abutments. There are two methods to carry out the grouting e.g. Full depth Grouting and Stage grouting.

3.1 FULL DEPTH GROUTING:

In the full depth grouting method each hole is drilled to full desired depth, washed, pressure tested and grouted in one operation. This method is usually limited to short holes e.g. 5m or less in depth or holes up to 10m that have only small cracks and joints with no risk of surface leakages. As full depth of grouting involves the risk of disturbance in the upper elevations it is not generally considered suitable for grouting deep holes.

-

3.2 STAGE GROUTING:

Stage Grouting is conducted to permit treatment of various zone individually by grouting successively increasing depth after grouting the upper zones. Grouting is done by drilling the hole to a predetermined depth and grouting this initial depth to an appropriate pressure. Grout is then washed from the hole prior to its final set (within 2 to 4 hrs.) and the hole is then deepened for next stage. Alternatively the grout is allowed to harden and re-drilling is carried out through hardened grout and hole is extended to the next stage. For the stage grouting, the connection at the top of the hole can be made directly to the header or by seating a packer at the top of the hole in the casing pipe. Alternatively, it is sometimes advantageous to install a packer immediately above the stage that is being grouted in order to isolate the upper portion of the hole. Higher pressure can then be permitted for grouting of the lower stage without causing upheaval in upper stages.

Grouting with double packer is suitable and the packers can be seated above and below such zones. Rotary diamond drilling method is preferred when double packer are used.

3.3 DOUBLE PACKER METHOD:

In this method hole is drilled to the final depth desired and cleaned with water. Two packers connected to the ends of a perforated drill rods of a length equivalent to the tests section should be fixed to in a drill hole. The bottom of the perforated rod is plugged before double packers grouting is proceeded with. Grouting will be done from bottom upwards or from top to downwards. However it will be convenient and economical to start the grouting from the bottom of the hole and then works upwards.

3.4 SIZE & DEPTH OF HOLE:

Depth of hole is governed primarily by the design requirements. When the purpose is to consolidate, the holes will be arranged in a regular pattern over the entire surface area required to be strengthened and the depth is determined by the extent of broken stones in the dam body as well as structural requirements regarding the deformability and the strength. The size of grout hole is generally less important than the cost of drilling holes. For grouting with cement, 38mm dia. holes are used.

10 ANNEXURE II

10.1 *WATER INTAKE TESTS FOR MASONRY / CONCRETE DAM*

1.0 PREAMBLE:

To ensure the quality of concrete/masonry constructed earlier and to ensure its imperviousness, water intake tests are carried out which help in ascertaining the necessity of grouting. Thus, results of water intake tests are vital, reliability of which ultimately depends upon the correctness of the procedure adopted.

2.0 TYPE OF DRILLLING:

Core / diamond drilling is preferred in order to have least disturbance to the already completed concrete/masonry. The drilled hole shall be vertical.

3.0 DRILLING PATTERN:

After observing the core recovery, extent of drilling pattern will be decided. Location of holes shall be selected in such a way that, there should be sufficient cover on both upstream and downstream. Initially location of hole shall be selected monolith wise, at least one hole for 30m length or two holesfor 50m length shall be drilled. avoiding position of embedments parts, instruments, galleries and other openings

4.0 WASHING OF HOLES:

Drilled hole shall be washed with air and water jet.

5.0 TESTING PROCEDURE:

After drilling of hole (46mm to 76mm diameter) of appropriate depth and washing the same, it shall be tested for water intake tests. The tests shall be carried out in stages of height 1.50m. The hole shall be filled with water & saturated for at least for 48 hrs. prior to conducting test. These holes shall be subjected to water loss tests to determine lugeon value notwithstanding the tests pressure specified for lugeon value. The actual tests pressure should not be so high as to cause disturbance to the concrete. Assuming linear variation of water loss with respect pressure applied, the water loss in lugeon may be interpreted.

6.0 PRESSURE TO BE APPLIED:

The minimum pressure applied shall not be less than 10m of hydrostatic head. The applied shall be equal to 1.75 times the hydrostatic head at that elevation. However, pressure shall not exceed 2 to 2.5 Kg/sqcm provided there is no storage of water in the reservoir. Pressure limits may be decided by analysis of the results of cyclic penetration tests. If there is impounding of water behind dam additional pressure shall be applied equivalent to standing water column.

7.0 STANDARDS OF PERMEABILITY:

Standards of permeability aimed at shall be a water not more than 2.50 Lugeons as mentioned in IS code.

8.0 Lugeons:

It is water loss in liters per minute per meter depth of drilled hole under a pressure of 10 atmospheres maintained for 10 minutes in a drilled hole of 46mm to 76mm dia.

9.0 TEST PROCEDURE:

After drilling of hole (46mm to 76mm diameter) of appropriate depth and washing the same, it shall be tested for water intake tests. The spacing of hole shall be 6m C/c along dam axis. The tests shall be carried out in stages of height 1.50m so that the entire hole is covered depending upon geological conditions. However, the test length should not be less than 5 times the diameter of hole. These holes shall be subjected to water loss tests to determine Lugeons value notwithstanding the tests pressure specified for Lugeons value. The actual tests pressure should not be so high as to cause disturbance to the mass. Assuming linear variation of water loss with respect to pressure applied, the water loss in Lugeon may be interpreted.

The tests shall be carried out from bottom to top of hole in stages of 1.50m with packers at top & bottom. The tests shall be conducted for pressure varying from zero to maximum hydrostatic head at that elevation. For every stage minimum three readings shall be taken. The intake shall be observed for at least for 10 minute till it give consisting reading in cyclic operation. The subsequent 1.50m pocket should be tested with double packer at top and bottom. Pressure applied and water loss in 10 minute shall be noted for each pressure and recorded in the table given below.

Pumping in tests will be conducted in an uncased and un-grouted section of the drilled holes. Packers are employed for conducting these tests depending upon the use of single or double packers respectively. When there is possibility of stucking of packer in the hole in such cases, it is recommended to grout the earlier stage , extend the bore hole and carry out the test. The tests are based on measuring the amount of water accepted by the 'test section' of the bore hole confined by the packer / packers while water is pumped into it.

Considering the condition of masonry / concrete, water percolation tests in a section of the drilled hole using a double packer is recommended. In this method entire hole is drilled to a final depth desired and cleaned with water until clear water returns. Two packers connected to the ends of a perforated drill rod of a length equivalent to the test section should be fixed in a drill hole. The bottom of the perforated rod shall be plugged before the double tests are proceeded with. The tests shall be done from bottom upwards or from top to downwards.

The tests are recommended to be performed in 1.50m to 3.0m test section so that entire hole is covered depending upon the conditions of masonry/ concrete. Water should be pumped into the section under pressure. The pressure should be maintained until the reading of water intake at intervals of 5 minute show a nearly constant reading of water intake for one particular pressure at the collar.

10.0 COMPUTATION OF EQUIVALENT PERMEABILITY FROM PERMEABILITY TEST DATA:

When the permeability tests are conducted in masonry/concrete the water intake is generally due to joints due to internal voids in masonry/concrete. Water loss may be expressed in Lugeons.

11 ANNEXURE III

11.1 GROUTING OF MASONRY / CONCRETE DAM

1.0 SPACING AND SEQUENCE OF GROUT HOLES:

For grouting, the holes are proposed along dam axis at 6m c/c. However, this spacing will be reduced to 3m c/c. Necessity of further grouting will be decided depending on grout intake and water intake tests. This spacing will be further extended at 3m upstream as well as 3m downstream from dam axis. However, in case of multiple line grout holes outer and inner rows shall be strictly followed. Spacing between primary holes is generally so selected that, the drilling could be carried out without interference from grouting due to inter connection from adjoining holes. In such cases interconnections will be prevented if sufficient cover of masonry/ concrete is available.

When it is desirable to test the efficacy of a grouting by comparing the grout absorption in primary and secondary holes a rectangular pattern shall be followed .

2.0 PREPARATION OF GROUT HOLE:

Before starting of actual injection of grout, drill hole shall be washed with water and air jet.

3.0 Grouting Procedure:

The most suitable pressure to be applied for grouting operation is difficult to determine in advance. In the interest of economy and efficiency, it is always desirable to use the safe pressure. Normally pressure to be applied shall not exceed more than 2 to 3.0Kg per sq.cm. Additional pressure shall be applied only if water is stored in the reservoir. The applied pressure should such that it should not cause disturbance to masonry/ concrete of dam body. It is always advisable to begin with very low initial pressure and builds up gradually. Pressure shall be raised until rate of intake falls than the initial grout intake. The pressure at when grout is injected frequently is varied with the depth of injection and the stage at which grout is injected.

4.0 INJECTING GROUT:

It may be necessary to start within a batch of thin grout and then thicken each succeeding batch by reducing the water cement ratio until the maximum practical thickness is determined. The grout may be injected into a portion of the length only from bottom of a hole. As the depth of injection is reduced the pressure may be reduced accordingly. This referred to as

Very careful and systematic control shall be exercised and recording made of grout pressure, rate of acceptance during the progress of work. Grouting is to be done from bottom to top in a stage first of 3m from bottom upwards.

After grouting operations are completed for the shift of the day, the remaining unused grout mix shall not be used. Also a mix not used up completely within one hour after mixing and clogging shall be thrown away.

5.0 METHOD OF APPLYING GROUT:

The pumping shall be continued until the hole being grouted consumes grout as given below.

Quantity of Grout	**Time in minutes**	**Pressure**
0.03 Cum	20	2.50 Kg per sq.cm or less.

The grouting shall however be stopped when the pressure gauge registers suddenly rise. After the hole has been grouted it shall be closed by means a valve to maintain the grout under pressure in the seams or crevices into which it has been forced. After the hole is grouted it should be examined if cement has settled properly. Holes where such settlement has taken place shall be cleared of all sort of sediment and grouted. Grouting shall be continued until at the specified pressure till the grout pipe refuses to take grout more than specified above. If for any reasons grouting operations must be stopped before a hole has been grouted to refusal. Clear water shall be kept running in to hole until grouting is again resumed.

6.0 GROUT MIX PROPORTIONS:

Lean grout mix 1:10 by weight is proposed to be used in the first instance. This lean mix will facilitate the grout to reach into very thin seams and will travel more distance. If the desired pressure is not built up with the lean mix even after grouting for three minutes then the mix may be thickened progressively. The maximum thick mixture proposed is 1:1 by weight.

7.0 MONITORING GROUTING:

A pressure gauge and water meter are installed in a grouting line so as to measure grouting pressure and grout consumption. An upheaval gauge is also placed near the hole.

8.0 EFFICIENCY OF GROUTING:

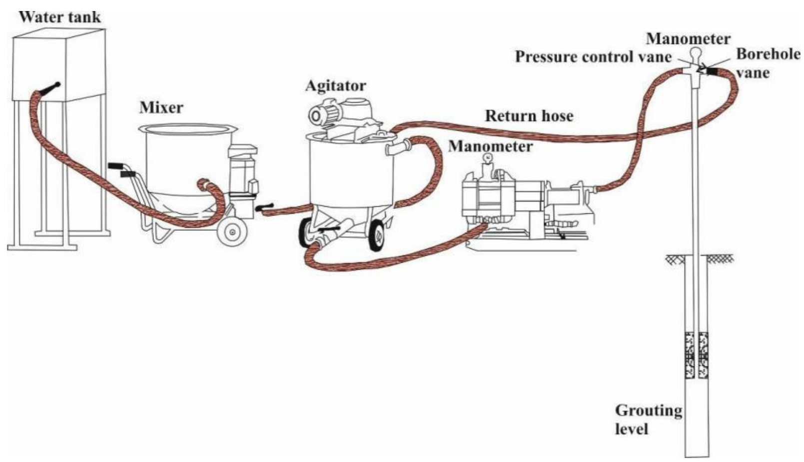
To determine the extent of flow of grout additional holes may be drilled at various locations within the area that has been grouted and tested for Lugeon tests. If these Lugeon tests satisfy the permissible limits values fixed by the Grouting experts. This indicates that the grouting operation is successful. Sometimes cores are also obtained from the grouted area. The effectiveness of the grouting operation also may be tested by attempting to inject water or grout into hole from which the cores were obtained. If these holes refuse to take grout, the test indicates that sub surface formations have been consolidated adequately by pressure injection.

9.0 MATERIAL :

9.1 Cement: Cement shall confirm to I.S. 269:1976 Ordinary Portland Cement shall be used for foundation grouting. The cement shall be weighed and used.

9.2 Water: Water shall not be salty and shall be clean reasonably clear and free from objectionable quantity of silt and traces of oils and injurious alkali salts. Water fit for drinking will generally be found suitable for mixing cement. Sea water shall never be used.

9.3 Admixture:- It shall be SUNPLEX shrinkage compensating type.The stabilizer of the admixture shall be capable of providing homogeneity, counteracts intrinsic shrinkage and settling characteristics while the plasticizer helps to achieve high workability.



Dr. Surendra Manjrekar has been bestowed (2018) with the highest honour of the world by American Concrete Institute (ACI) of 'Honorary Member'. Honorary Fellow of “Institute of Concrete Technology”, UK (2022). RILEM international France honored him at Vancouver (2023). Ex-visiting professor at University of Leeds, UK in the field of ‘Reduction of Carbon Footprint’. Only Indian working as a voting member of ACI committee 364 on ‘Repair’ from 2004. Special Signatory to MOU for technical co-operation between India, Singapore, Vietnam and Malaysia organized by ACI in Singapore(2019). Vice Chancellor appointee on “Internal Quality Assurance Cell” of Homi Bhabha State University, Mumbai. Keynote in countries like UK, USA, Malaysia, Oman, Dubai, Kuwait, Romania, Hong Kong, Singapore, Mexico, Egypt, Kuala Lumpur, Turkey etc. Published 200 plus papers and been guest editor of international journals. CMD of Sunanda Speciality Coatings Pvt. Ltd. Active in concurrent R & D work specially Repairs of Dams, Irrigation Canals, water related structures, Underwater Concreting, Reduction in Carbon Foot Print, Nano molecule Soil Stabilization etc.

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Latest Technologies for Rehabilitation and Dam Safety, 20-22 March 2025, Shimla

INNOVATIVE CORROSION PROTECTION SYSTEM TO GATES OF MIDDLE VAITARNA, BARVI & RANSAI DAMS

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ABSTRACT

Innovative corrosion protection system to gates of the dams at Middle Vaitarna (2019), Barvi (April 2021) & Ransai (April 2024) was carried out and as a representative case the Ransai Dam gates protection treatment is presented in this paper. Ransai Dam is built across Vindhane river by Maharashtra Industrial Development Corporation (MIDC) in Raigad district near Mumbai. The part of 1st stage masonry dam up to RL 30.48 M was completed in 1970 and the remaining work of providing automatic tilting gates of size 6.00m x 2.00m at FRL 37.2 M was completed in the year 1981. During routine inspection in 2023 it was noted that the corrosion protection coating to the gates & its supporting mechanism is in advanced stage.

Generally coal tar epoxy coatings are applied and provided for in various schedule of rates. This is a hazardous process from health point of view also besides having short service life, as the coal tar contents are carcinogenic and should be avoided for any structure related to drinking water. In addition, the quantum of epoxy content also is compromised. Reduction of epoxy and there by addition of coal tar reduces abrasion properties along with susceptibility to uv attack resulting into early decay and hence over all corrosion losses lead to substantial life cycle reduction.

As per the studies conducted by National Association of Corrosion Engineers (NACE) the yearly loss due to corrosion in major sectors in India is about 4.2% of the GDP,[R1] which is quite substantial. Therefore for a developing country like India proper selection, application, regular inspections, and maintenance of the coatings are all crucial for a successful life span without much necessary stress on economic resources.

Paper discusses the innovative corrosion protection system of total rust removing chemical treatment followed by

a) High build special epoxies modified with extended chemical chain and
b) Tailor made polyurethane coating with cyclic nature offered to the molecules.The coating thus becomes more resistant to decomposition as more energy is required to destroy cyclic arrangement of molecules rather than in case of normal straight aliphatic chain.

These novel developmental efforts enhanced manifold the performance properties of the system as observed in Vaitarna and Barvi Dam. As a result MIDC took up the work of corrosion prevention treatment of gates of one more dam viz. Ransai Dam in March 2024. Post treatment and post monsoon in 2024, the gates were checked for the performance in January 2025 and is reported to be very satisfactory. This paper deals with the novel and effective protective systems for the important member of a dam assembly i.e. automatic tilting gates.

Keywords: Corrosion, Protection, LCNR, APR

ABBREVIATIONS AND ACRONYMS:

MIDC : Maharashtra Industrial Development Corporation an Undertaking of a Maharashtra State Govt responsible for developing & maintaining infrastructure like water supply scheme for catering water demand of industries in the state

Govt :Government

FRL: Full Reservoir Level,

RL: Reduced Level

MCM : Million Cubic Meters

LCNR: Long Chain Nylon Reticulant

APR: Acyclic, Aliphatic Polyisocyanate Reticulant

DFT: Dry film thickness

INTRODUCTION:

This paper describes the innovative corrosion protection coating system to automatic tilting steel gates on the spillway of an important large Ransai dam.

Storage capacity of the reservoir was raised to RL 37.20.M for the 2nd & final stage of construction and 15 automatic tilting gates of dimension 6.00 m x 2.00 M were installed. These were designed for FRL 37.2 M for the storage capacity of 10 million cum. The releases through dam is treated in a water treatment plant near to dam & is further supplied 24 x 7 to National Armament Depot (NAD)at Karanja, Maharashtra State Power Generation Company (MAHAGENCO), Oil & Natural Gas Corporation (ONGC), Uran Municipal Council & enroute villages in Raigad district.

CORROSION OF STEEL:

Steel structures are susceptible to corrosion and in Ransai Dam gate leaves and its supporting structure was heavily corroded and needed protection against corrosion (see Fig. 1). This is mainly due to constant contact with water, abrasion, biochemical interactions etc. Gates being important structure in the system of water supply through dams, it is necessary to protect them thoroughly and make sure that protection lasts for longer life cycle.

As per the guidelines of BIS 14177 - 1994[R2] protection of gates is carried with zinc rich epoxy primer and solvent less coal tar epoxy system.

ORIGINAL CORROSION PROTECTION SYSTEM PROVIDED TO GATES BEFORE 2023 AND ITS LACKING AND ITS POOR PERFORMANCE:

Corrosion protection coating was provided to gates periodically with zinc rich epoxy primer & solvent less coal tar epoxy paint as per the guidelines in BIS 14177 - 1994[R2].

Visual inspection of the gates in later part of 2023 revealed that, the corrosion protection system provided to gates and its accessories is worn out & fully disintegrated due to excessive corrosion.

Fig 1. Excessive Corrosion in 2023

Guide lines for inspection & the type of painting of gates & allied structures are issued in BIS 14177[R2]. The CWC has issued guide lines for rehabilitation of large dams[R3]& guide lines for preparing operation & maintenance of dams [R4]. These guidelines specify different protection system for

1. Surfaces which come in contact with water
2. Surfaces which do not come in contact with water.

Coatings can be considered a first line of defense for steel structures when trying to combat corrosion. This necessitates use of quality coatings with properties like abrasion resistance, uv resistance as well as non carcinogenic properties. Choice of well performing coating system reduces the maintenance cycle of the structure resulting into long term economic benefits. The current protection of use of zinc rich primer and coal tar epoxy was seen to be inadequate to provide long term protection and is so for following reasons.

Zinc Rich Primer Coat: Primer coat of any coating system is a soul of the protection system. The structure like mild steel gates are subjected to very high magnitude of abrasion due to high velocity of water along with suspended solids. In such scenario following things happen in the sequence given.

1. Abrasion creates scratches in the coating material exposing the mother surface of steel.
2. Zinc by itself is a sacrificial material to protect the steel.
3. However, on abrasion and scratching two electrodes are formed viz. steel (cathode) and zinc (anode) and water as the electrolyte which completes the electrochemical cell and sacrifice of zinc begins.
4. Being in thickness measurable microns the sacrificial zinc layer exhausts rapidly resulting into initiating further major electro chemical deterioration of steel that leads to corrosion and heavy sectional loss.
5. Zinc rich epoxy was the remedy in earlier times when there were no other options of potent polymer coatings available.

Solvent less Coal Tar Epoxy: Coal tar epoxy is for ordinary protection. Coaltar replaces epoxy content partially (in the coating) in order to economize the cost. This replacement compromises the structural and chemical resistant properties of pure epoxy coatings. However, this is the compromise in the degree of protection from corrosion.

The gates & its entire supporting structure like hoists, cranes, inspection bridges, stop log gates, pylons etc. will be exposed to sunlight. The epoxy as well as coal tar epoxy coatings are very vulnerable to ultra violet (uv) radiation from sunlight & degenerate throughout the year.

Periodic repainting is the generally adopted procedure for regular maintenance. However, in repainting earlier degenerated coat will not be removed totally and it ends up into new top coat on existing weaker coat (due to abrasion and uv etc.)Repainting process simply hides the disintegrated layer and itself once again undergoes the attack of abrasion and uv as mentioned earlier

The World Health Organization International Agency For Research on Cancer (IARC) has published a report on IARC monographs on the evaluation of carcinogenic risk to humans[R5]. As per this report the use of coal tar epoxy coating is found carcinogenic under category 1 for human. The developed countries like Europe, America, Japan have banned use of coal tar epoxy paint.

In conclusion improper selection of material for corrosion protection system, poor maintenance of coatings has resulted into heavy corrosion to hydraulic structures and the partial and full replacement becomes inevitable. Thus considering past experience of ill performance of conventional coatings, use of more strong, stable, durable & high performing innovative coating system to important components like gates of Ransai dam,was imperative, considering the life cycle cost and operational cost due to stoppages and break downs.-Hence it was felt necessary to adopt the innovative system used at Vaitarna and Barvi Dams.

CORROSION PROTECTION SCHEME FOR RANSAI DAM GATES

New system of corrosion protection consist of following methodology**:**

1. **Surface Preparation**: The existing paint was removed to reach the original surface of gates with help of mechanical grinders & mechanical wire brush. It was ensured that surface is prepared to desired level.

2. Following factors were considered while selection of corrosion resistant coating to the dam gates & supporting mechanism which are in contact with water.
    a. Corrosion removal,
    b. Coating material complying

        i. High adhesion,
        ii. High corrosion resistant.
        iii. Chemical resistant
        iv. Abrasion resistant
        v. UV resistant
        vi. Durable based on test parameters and past track records of innovative systems.
        vii. Meets requirements to health safety and environment
        viii. Properties related to application conditions, equipment and personnel.
        ix. Availability and economics of coating materials.

Above will automatically reduce carbon foot print. It is imperative that the manufacturer shall have credentials like accreditation of ISO for R & D, manufacturing process, CE certification for quality of product.

SELECTION OF CORROSION PROTECTION COATING WITH ITS PROPERTIES AND PROCEDURE:

1. One coat of Rusticide -- loosens rust, removes rust & converts into stable compound of iron.
2. One coat of Sungard LCNR primer -- This molecule has extended chains and hence mechanical properties are improved. Increases weather resistance and anti aging properties
3. One coat of Sunepoxy HB -- Is a high build protective epoxy coating to take care of continuous abrasion.
4. Top 2 coats of Sungard APR -- This is a acyclic polyurethane coating resistant to ultra violet attack, abrasion and corrosion. Being acyclic additional degree of protection is offered to tendency of general disintegration. The total DFT of the system is in the range of 400 to 450 microns.

The work started in March 2024 and completed in April 2024. The application of material was done with the help of state of the art implements. Following is the pictorial presentation of the procedure.

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Figure 2. Excessive Corrosion in 2023

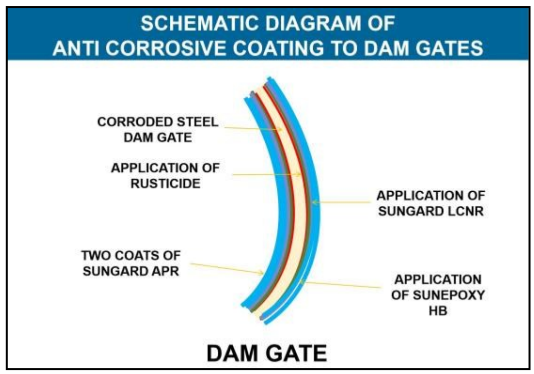

Figure 3. Application Procedure to Gates

Figure 4. After Completion of Corrosion Protection Work of All Gates

The performance of the treatment to the gates was monitored after monsoon of 2024 through visual observation, The performance of the coating system will be monitored using half cell potential & cathodic disbondment testing.

FUTURE SCOPE OF WORK

Additional corrosion protection to embedded parts of gates in RCC piers is also necessary. This can be achieved using bipolar corrosion inhibiting admixture while casting the concrete for piers. The molecules of this admixture will protect the steel from corrosion in concrete.

The life of gates can be further enhanced by installing galvanic sacrificial anodes at appropriate locations.

In conclusion a novel method of using modified epoxy and polyurethane polymers shows the promise to resist corrosion in the gates of the dam. In addition it highlights the importance of thorough corrosion removal by physical as well as chemical treatment before the polymeric treatment. This procedure can be extended to other connected steel structures of the gates/dams.

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Surendra Manjrekar has been bestowed (2018) with the highest honour of the world by American Concrete Institute (ACI) of 'Honorary Member'. Honorary Fellow of “Institute of Concrete Technology”, UK (2022). RILEM international France honored him at Vancouver (2023). Ex-visiting professor at University of Leeds, UK in the field of ‘Reduction of Carbon Footprint’. Only Indian working as a voting member of ACI committee 364 on ‘Repair’ from 2004. Special Signatory to MOU for technical co-operation between India, Singapore, Vietnam and Malaysia organized by ACI in Singapore(2019). Vice Chancellor appointee on “Internal Quality Assurance Cell” of Homi Bhabha State University, Mumbai. Keynote in countries like UK, USA, Malaysia, Oman, Dubai, Kuwait, Romania, Hong Kong, Singapore, Mexico, Egypt, Kuala Lumpur, Turkey etc. Published 200 plus papers and been guest editor of international journals. CMD of Sunanda Speciality Coatings Pvt. Ltd. Active in concurrent R & D work specially Repairs of Dams, Irrigation Canals, water related structures, Underwater Concreting, Reduction in Carbon Foot Print, Nano molecule Soil Stabilization etc.

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"LATEST TECHINIQUES IN IMPROVING WATERTIGHNESS IN DAMS – GEOMEMBRANE SEALING SYSTEM"

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ABSTRACT:

Dams are widely constructed to serve a specific purpose that may include irrigation, potable needs, power generation, safety of the environment etc. As the construction of dam is a tedious and time consuming process which is of great importance, the longevity to serve the purpose is of primary importance. Most of the present dams is constructed way back in 20th Century and served for nearly more than 60 years. The effects of aging leads to loss of water tightness which have a domino effect on reliability of the structure. The accustomed method of repairing like grouting, pointing doesn't address the issue at source leading to its recurrence. Hence an effective long term solution with high durability and performance is necessary. One such remedy is the PVC Geomembrane which proves its performance since 1959 addressing safety against opening of fissures and joints, settlements, differential displacements and complete water tightness on the upstream face of any types of dam. In India, the geomembrane has been used for rehabilitation of 3 important dams at the verge of perviousness in surface namely Kadamparai, Servalar and Upper Bhavani, a pressure tunnel called Bajoli Holi is also addressed by our geomembrane. The process and progress of installation of membrane and its results are discussed briefly in this paper.

1. INTRODUCTION:

In the view of new climatic change and growing population the need for energy and water is increasing drastically which in turn leads to prudent use of existing resources. The risk associated with aging of dam is generally loss of water tightness which can cause increase in the uplift pressures, erosion and embankment instability. In this fast paced techno era, solution to any issue is expected to be quick and long lasting. At early stage of leakage in dam, local repairs like mortar resin grouting seems like effective but in the longer run, the repair technique fails invariably leading to recurrence of the same issue all over again with more aggression. So, the rehabilitation of existing dams for a longer run is mandatory. Using the PVC geomembrane which are specifically engineered to address this issue in more rapid way saves all the concern. Moreover PVC Geomembrane are the most suited when the distribution of leakage point are confined to a large area unlike the localized ones with mortars and resins.

The PVC Geomembranes with thickness greater than 2.5mm can be engineered for service life of even 100 years when covered and for half a century when it is exposed which outcome the shortcomings of conventional repair methods. The membranes are light weight hence easy to transport, easy to install and can accommodate differentiate settlements too. The geomembrane liner also has a very low carbon footprint contributing lesser risk to the environment. It can adopt to any rigid faces of dam, it may be masonry, concrete or bituminous concrete. The reinstating to effective working of the dam is quicker than any other alternative which makes it cost effective too. Such one in kind of PVC geomembrane is laid in rehabilitation for 3 dams and a canal and tunnel in India which are elaborated as case studies here.

2. CASE STUDIES OF REHABILITATION IN INDIA

2.1. Kadamparai, India, 2005

Kadamparai Dam is a hydroelectric dam and first pumped storage plant situated in Coimbatore district, Tamil Nadu. It was built in 1983 and commissioned in 1987 is owned by the Tamil Nadu Generation and Distribution Corporation (TANGEDCO). The dam is a composite structure made of stone masonry and earthen embankments with 67 meters high and 478 meters long with reservoir capacity of 30.85 million cubic meter and electricity generation about 417.67 GWh. The masonry section has a central spillway, a scour vent tower, and an inspection gallery. The dam's water level can vary by 6 to 8 meters per day due to pumping operations.

In this significant dam, after first impoundment in 1984 the leakage is measured about 38,200 l/minute (636l/s). In view of addressing the issue, many local repairs have been carried out throughout the years including racking, packing the joints, vertical drilling at the crest and grouting in specific areas, underwater treatment by specific chemicals. All these works seem to be of any significance as there was continuous increase in the leakage amount and visible outburst in the drainage gallery which exceeded permissible limits. After in depth studies the maximum leakage area in the dam is concluded to be at the masonry section where extensive treatment is necessary. The leakage in foundation area is minimal so exposed masonry area was decided to be treated.

Figure1&2 Condition of the Kadamparai Dam before Geomembrane Installation

Figure 3&4 Conventional Racking and packing treatment methods carried out

In 2003, Carpi Tech SA, a waterproofing specialist contractor from Switzerland, was invited to study the conditions of Kadamparai dam and to suggest remedial measures for watertightness in which PVC Geomembrane is suggested as a permanent remedy. In 2004 a global tender was floated and Carpi Tech S.A. from Switzerland was successful in winning the bid and awarded the rehabilitation work.

CARPI took over the site by end of November 2004 and started surface preparatory works which includes hydroblasting the upstream face area which had lose mortars. The removed mortar joints were repacked with new mortar. It is important to mention that the geomembrane system has several steel anchors at designed intervals which needs a fairly stable subgrade. The base of each of the 12 vertical joints in the masonry has been drilled and then grouted to avoid water by-passing the perimeter seal of the geomembrane system. The places where anchorages has to be laid is treated with a strip of mortar for even surface.

Figure 5 Surface Preparation by Shotcreting with minimum reinforcement

Figure 6 Laying of Vertical anchorage lines minimum reinforcement

Figure 7 Placement of Geocomposite over Geotextile layer

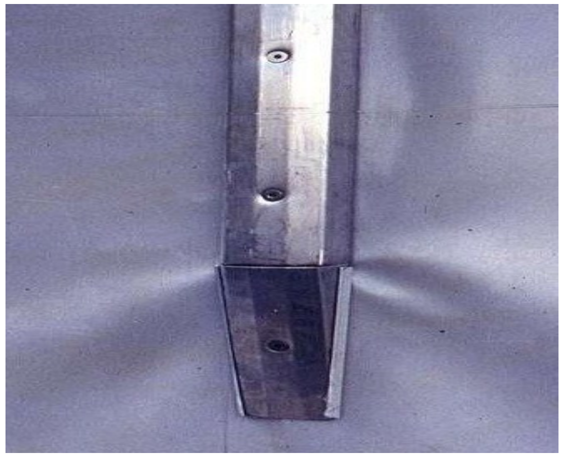


Figure 8 Tensioning profile anchorage

The vertical anchorage lines at designed spacing is laid throughout the section initially. The stainless-steel profiles which fasten the geocomposite to the face of the dam, are adjusted to follow the shape of the upstream face and fastened by chemical anchors placed in the stone masonry. As masonry surface has many undulations and sharp surface than other concrete surfaces, to protect the geomembrane from puncturing, a sacrificial layer of Geotextile, 2000 g/m^2 is laid initially. Then the waterproofing liner SIBELON CNT 3750, consisting of a 2.5mm thick thermoplastic SIBELON membrane heat bonded during fabrication to a non-woven needle punched 500g/m^2 of polypropylene geotextile is rolled. The rolls of geo-composite are overlapped and welded together by process of heat seaming. After welding all geo-composite panels together, the external tensioning profiles are placed over the geo-composite and connected to the lower profiles laid earlier. This anchorage system is patented by Carpi. The geomembrane is held at the crest consists of a batten strip, placed over the surface smooothened with mortar. The external tensioning profiles are covered by a PVC geomembrane strip SIBELON C3900 which is heat welded to the parent geomembrane below.

As far as drainage concerned, the gap between the masonry surface and geotextile acts as a vertical conduit for water from top. At bottom in the perimeter seal area, a high transmissivity non-compressible geonet was placed to collect the drained water and convey it to the discharge point in the gallery. Piezometers are installed behind the geomembrane to read presence of any unattended hydrostatic head in the drainage layer. For monitoring the leakage data, an optical fibre cable leakage detection system is laid below geocomposite in the perimeter seal area.

The waterproofing liner is confined at al peripheries by mechanical seal perimeter watertight seals at submersible areas. It is created, by placing chemical anchors, bedding mortar, geocomposite, gasket, stainless strip batten strip and anchoring with bolt to make it watertight. In this project to have a separate measurement of water drained from foundation and from the upper composite area double perimeter seal is adopted.

The complete geomembrane system is installed in 3 months' time laying an area of 17,300 m^2. Impounding after work began on April 12, 2005. After 10 years of limited operation (well below the full level), with an experienced leakage of up to 38,000 lpm, after rehabilitation is again at full supply level with rate of leakage reduced to 80 lpm. Now its approaching 20 years of successful operation with no further increase in leakage testifying the credibility and longevity of the system.

Fig 9 Kadamparai Dam after Rehabilitaion works *Fig 10. Dry condition of Inspection gallery*

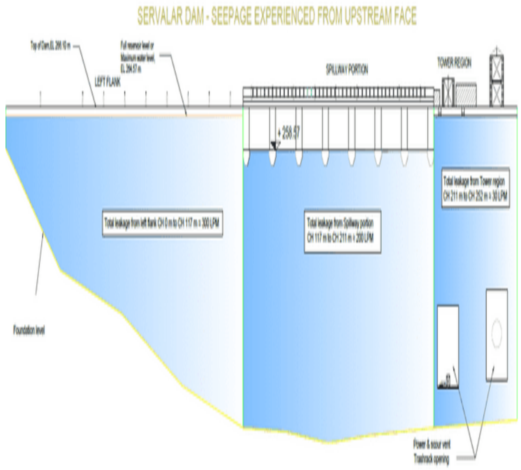
2.2 Servalar, India, 2018

The Servalar Dam is located on Servalar river, a tributary of Thamarabarani in Tirunelveli District, Tamil Nadu. The Servalar Hydro Electric Project (20 MW) was completed and commissioned during 1986 by the Tamil Nadu Generation and Distribution Corporation (TANGEDCO). It is a stone masonry dam of 57 meter high and 465 m length with a storage capacity of 35 Million CUM.

The major problem encountered in the dam is a heavy seepage and leaching through the left flank portion with a maximum seepage of 743 LPM in 2009 at FRL. and choked vertical shafts and foundation drains in tur resulting in saturation of dam body which were observed on the downstream side of the dam.

Fig 11. Flooded downstream surface *Fig 12. Choked Drainage Shaft*

Every 2 years attempts to control leakage consist of conventional repair method persisted which yield no good result. So, after a series of meeting and discussions, the Client decided to go in for a long term permanent solution. With already Kadamparai presenting a success case, the decision was taken to go for geomembrane waterproofing system and the contract was awarded to Carpi in 2016 and the site was handed in Feb 2017.This was the first rehabilitation project funded by DRIP in India. The scope of rehabilitation of dam includes desilting, cavity Filling, Racking/Packing and Pointing, drilling and grouting and finally geomembrane works.



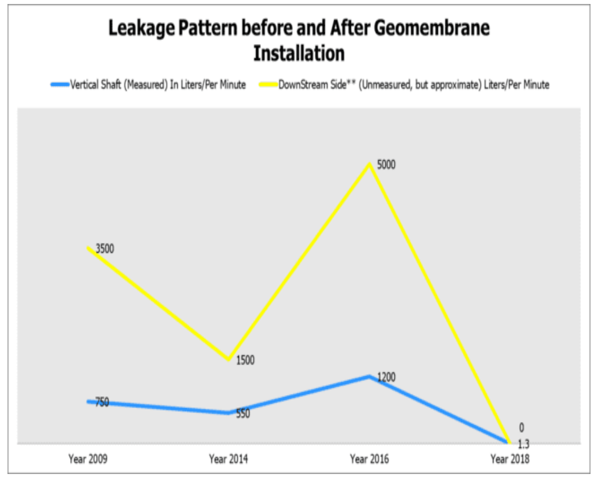


Fig 13 Seepage area in US face *Fig 14 Leakage pattern after Geomembrane Installation*

Acute water shortage in the district resulted in delay in opening up of the reservoir for beginning of work. So the total work has to be done at round the clock studded between seasons, as it is one of the main source of drinking water/irrigation source of water to the Tirunelveli district.

The preparatory works consists of hydrojetting followed by racking and packing to weak mortar joints. The geosynthetic works started with an antipuncture layer of 2000gsm of geotextile to enhance drainage capability too. The drainage system is similar to that of Kadamparai dam 4 compartments collecting water from upstream face and 2 compartments collecting water from foundation area discharging by 10 pipes- 6 for upper gallery, 4 for lower gallery.

The methodoly of laying of geocomposite is similar to that of Kadamparai dam consisting of primary vertical anchorage, geotextile laying, Geocomposite laying, Seaming of geocomposite, Secondary tensioning profile placement, Crest anchorage, Perimeter sealing.

In this project too SIBELON CNT 3750, consisting of 2.5mm thick PVC geomembrane heat bonded to 500g/m2 polypropylene geotextile was used.

The entire rehabilitation work was finished in two phases due to monsoon break and all the works were completed on October 2018. The leakage of 7000 l/min has been reduced to to 20l/min.

Fig 15 Servalar Dam After Rehabilitation installation *Fig 16. Dry downstream surface after*

2.3 Upper Bhavani, India, 2021

Upper Bhavani is a stone masonry dam owned by TANGEDCO located in the border between state of Tamil Nadu and Kerala deemed as first Source of water for a series of power houses downstream side (> 600 MW) at the time of construction in 1960’s.It is 80m high and 419m long dam with leakage rate of 8000 l/min reported in more than 8 shafts which is evident in the downstream side. Since 1990s various leakage arresting methods like grouting, Epoxy pointing and shotcreting is carried out. In the year At

2019 at FRL the leakage rate surpassed to 15000lpm leading to 30million unit power loss per year. So, the Client TANGEDCO decided to do large scale rehabilitation repair which was done and proven successful last time and was funded by World Bank, IIT Madras under DRIP project.

Fig 17,18,19 Upper Bhavani dam after installation

The rehabilitation system adopted in Upper Bhavani is the widely used technique mentioned in bulletin 135 of ICOLD 2010 similar to Kadamparai and Servalar. The geomembrane used here is SIBELON CNT 4400, consist of 3mm thick geomembrane heat bonded to a non-woven needle punch 500g/m2 polypropylene geotextile. All anchorage lines are smoothened by applying a layer of mortar over which tensioning profiles are laid at a spacing of 5.70m. An anti-puncturing geotextile 2000g/m^2 is covered as a sacrificial layer over which the membrane is laid and seamed. At crest 50x3 flat stainless steel batten strip tied to the dam and in submerged peripheries 80x8 mm flat batten strip with resin is anchor rods is done. The drainage and the optic fibre cable monitoring system is followed as done in Kadamparai dam.

The project is completed in three stages due to pandemic spread over the world. Some of the works in phase III has to be carried out in underwater conditions due to cofferdam breach and heavy rainfall. The leakage rate in shafts after rehabilitation reduced to less than 60lpm from 8200lpm.

2.4 Bajoli Holi Head Race Tunnel (HRT), 2023

Bajoli Holi HEP 180MW is under operation with M/s GMR Bajoli Holi Hydropower Private Ltd. Since commissioning of the project, water seepage in order of 12,000 lpm has been observed in the vicinity of the Middle Horizontal Pressure Shaft. The seepages were first observed during the initial filling of the HRT on 19 Dec. 2021. As reported, the seepages started during the process of HRT charging which was filled upto a length of 5 Km. Subsequently the seepages stopped when the WCS was emptied after the wet commissioning of the turbines. Surface monitoring revealed that the seepage occurred at two places. Presumptuous that the seepages are occurring from the HRT in the close vicinity of the Surge Shaft, extensive ordinary cement grouting at every 3m interval from seven points was carried. After the completion of grouting, the filling of Water Conductor System of the Project started in March 2022 for commissioning of first unit. The Project Authorities witnessed seepages at the same locations where it occurred earlier during the wet commissioning stage with no change in the quantum of discharge. So the membrane lining is opted to save the tunnel.

The maximum hydraulic head in the headrace tunnel is around 118 m. The mean flow velocity in the tunnel during normal operation is about 2.7 m/s. The maximum flow velocity during transient conditions is about 3.3 m/s.The headrace tunnel is affected by cracking of the concrete lining. An important seepage flow was currently observed in the right flank of the valley, in a sector comprised between El. 1800 m and El. 1825 m. The flow rate has been estimated of around 50 l/s possibly originated from a stretch of 200 m. So installation of geomembrane was initially planned for a length of 200m and later looking at the condition of the cracked concrete the lining was extended further to 266m.

For surface preparation, the optimal concrete surface for the installation should have surface Profile equal to or lower than #5 (CSP ≤ 5). Any kind structural repairs like large Cracks / Fissures / Cavities / Damaged Reinforcements /Water gushing through cavities/weak zones etc to be attended. Thus, cleaning with hydro-jetting to remove calcite deposits and sediments, filling of honeycombs with grout, grinding of protruding aggregates and sharp edges, levelling with plaster are carried out.

The Geo-membrane system provides a practically watertight liner capable to maintain imperviousness under the acting loads, and with elongation characteristics allowing bridging those discontinuities that may arise in the subgrade (e.g., cracks forming in the concrete liner on which the geomembrane is applied). A geomembrane liner stops leakage and prevents water entering the deteriorated concrete lining through cracks and joints, and affecting the durability of the reinforcement, and avoids that water inside the tunnel can escape into the surrounding ground with potentially dangerous effects on the stability of the slopes and consequently of the structure.

The entire Geomembrane Sealing system consisted of several anchorage lines designed to withstand the water head and flow velocity. The design of the anchorage was done after analyzing all the data provided to Carpi. The first steel profile 6x60 is placed along the line marking and drilled, anchored and then hammered in the hole and the profile temporary close without excessive torque in correspondence of the first, last and middle bolts to keep it in place. At all joints, patches of a sacrificial layer made of a non-woven polypropylene geotextile with mass 2000 g/m2 and second sacrifice layer of SIBELON CNT 4400 (3mm thick at joints) in correspondence of damages is laid with impact anchors. The main PVC Geocomposite CNT4400 is laid above sacrificial membrane and anchored to the bottom profile. The same methodology is carried for all over the tunnel. All the composites are heat seamed to the adjacent membrane. The geomembrane lining is terminated at the end of lining by using special watertight seal made of thick Stainless Steel (80x8mm) material. The geocomposite sheets are connected to the perimeter seal at the overt and invert section of Horse shoe shape HRT with high bonding resin is used with rubber gaskets and seals to prevent entry of water from the bottom. This same perimeter sealing is done as compartment sealing at every at every 60mtrs. Now the system is fully watertight.

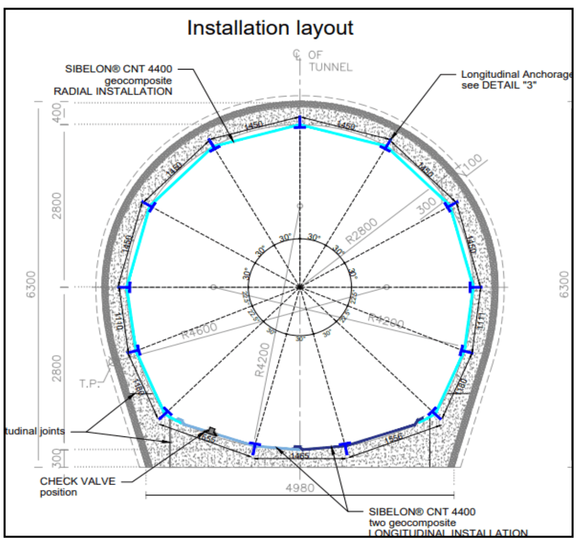


Fig 20,21 Bajoli Holi Method of Installation

Fig 22,,23 Before and After Installation

3 CONCLUSION:

The paper state that adoption of the latest technology with proper design can always be an effective solution which enhances the life of the structure and make it economical addressing the issue permanently. The system adopted has several advantages compared to any other conventional repair works which has been tried and tested several times in the past but with limited durability. The table below gives the summary of the major projects in India and their durability and performance even after decades after installation.

Name of the Structure	Year of Installation	Installed Area (m^2)	Leakage Before Geomembrane	Leakage After Geomembrane	Present Service Life
Kadamparai	2005	17,303	38,000 Lpm	< 80 Lpm	20 years
Servalar	2018	9854	5,000-6,000 Lpm	< 30 Lpm	7 Years
Upper Bhavani	2021	17,904	8,000-12,000 Lpm	< 80 Lpm	4 Years
Bajoli Holi	2023	4,851	12,000 lpm	< 40 lpm	2 Years

4 ABOUT THE AUTHOR:

Jagadeesan Subramanian is a management graduate with over 20 years' experience in Engineering Construction and Enterprise Resource Management. His association with Carpi dates back to 2004 and has been involved in all Carpi's project in India since beginning. He has been involved in the several first projects in India. Actively involved in the first geomembrane rehabilitated dam in India at Kadamparai Dam. The project was a remarkable success where the dam which was on the verge of decommissioning is brought back to life. Since then, he has been involved in several dam rehabilitation projects in India using polymeric geomembranes. He has successfully handled the 'Servalar Dam' and "Upper Bhavani Dam" Rehabilitation projects under DRIP (Dam Rehabilitation and Improvement Program) of World Bank and several other rehabilitation projects in India. His passion towards hydraulic structures and water consciousness drove him to push for the introduction of geomembrane in high pressure hydraulic tunnels which is considered as the most challenging and feared field where the water flows with high velocity and water head. He successfully concluded the deal and the successful commissioning of the Bajoli Holi Tunnel in Himachal with a complex geology. The recent years the push for energy transition paved way for introducing the First Geomembrane Faced Rock Fill dam in India. The dam is commissioned and is under filling. The technology is seen as a breakthrough in engineering construction field to enable reach Green energy in India at a much faster pace. He also has international experience for a short tenure in Venezuela and other countries. Presently he oversees the Carpi's Indian division.

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Geotechnical Investigation carried out to study the causes for substantial seepage of earthen embankment between LS 350m and 500m of Nanganjiyar Dam

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KEYWORDS
Earthen embankment, Geotechnical investigation, Seepage, Flow net, Permeability

ABSTRACT
Dam Safety Review Panel (DSRP) inspected the Nanganjiyar dam on 21.02.2023 and observed profuse seepage between LS 350m and LS 500m in the earthen embankment. The panel recommended for geotechnical investigation to identify the cause of seepage and to conduct a flow net analysis for the seeping earthen dam section from LS 350m to LS 500m in the left flank of earthen dam in Nanganjiyar Dam. The study was taken by Soil Mechanics & Research Division of Tamil Nadu Water Resources Department. Preliminary site investigation was carried out to identify the locations of test bore holes and trial pits. Accordingly, locations for five bore holes and three trial pits were identified and necessary field testing was carried out. Necessary disturbed and undisturbed soil samples from various bore holes and trial pits were collected for carrying out necessary laboratory testing. The soil samples were tested for its index and engineering properties. The parameters obtained from laboratory testing were used for flow net analysis. Graphical method proposed by Casagrande was adopted for flow net analysis. Flow net analysis was carried out at LS 375m and LS 450m. The quantum of seepage is assessed and found to be excess for the embankment height of 7m. Further, the seepage data measured in left V-notch at LS 850m collected since the inception of dam (2008) was analysed. It is observed that whenever the level of the reservoir exceeds +229.00m, substantial seepage is observed in the toe drain.

1. INTRODUCTION

Nanganjiyar Reservoir has been constructed across Nanganjiyar River in Oddanchattram Taluk of Dindigul District. The reservoir was first opened for irrigation on 06.04.2008. It is an earth cum masonry dam constructed for a length of 2680m. The surplus arrangement of this reservoir is an uncontrolled spillway from LS 1325m to 1755m with a crest level of +231.00m, the discharging capacity of the spillway is 3467.45 cumecs.

Dam Safety Review Panel (DSRP) inspected the dam on 21.02.2023 (DSRP – 27 /2023). On the date of inspection, the reservoir level was at 230.92m (FRL @ 231.00m). From LS 350m to LS 500m (Left flank of earthen dam) significant water seepage was seen in the toe drain. The observed seepage on the date of inspection in the left V – 1 (LS 850m, representing left flank) notch was 14.42 lps. The maximum seepage recorded on the left flank was 18.00 lps. Water sample collected from the toe drain was clear and no soil particles were seen.

Between LS 350m and LS 500m, downstream area of the toe drain and service road, numbers of irrigation wells were present. The water was overflowing from the well on to the road. The quantity of seepage water appeared to be more than permissible seepage in the earth Dam. Considering the above aspects, the panel recommended for geotechnical investigation to identify the cause of seepage and to conduct a flow net analysis for the seeping earthen dam section from LS 350m to LS 500m.

2. SCOPE OF WORK

Preliminary investigation of Nanganjiyar Reservoir was carried out by team of officials of SM&R Division on 04.05.2023. Seepage of reservoir water below the foundation of embankment was suspected as profuse water logging and submergence of open well was noticed on the downstream of embankment. The objective of this study is to conduct flow net analysis for the seeping earthen dam section of Nanganjiyar Reservoir (LS 350m to LS 500m, Left flank of earthen dam) by carrying out necessary field and laboratory testing.

3. GEOTECHNICAL INVESTIGATION

Detailed investigation for studying the probable causes for profuse seepage in Nanganjiyar Reservoir commenced on 20.07.2023. As the seepage was observed from LS 350m to 500m of the earthen embankment, two borehole locations were identified in this range. In addition, a borehole was also proposed at LS 630m as a reference point. These three boreholes were proposed to be drilled on the top of the embankment using Hand Auger and SPT was also carried out. Similarly rotary drilling was proposed in the downstream side of the earthen embankment in the transverse direction to the borehole in the embankment so as to ascertain the properties of the foundation soil.

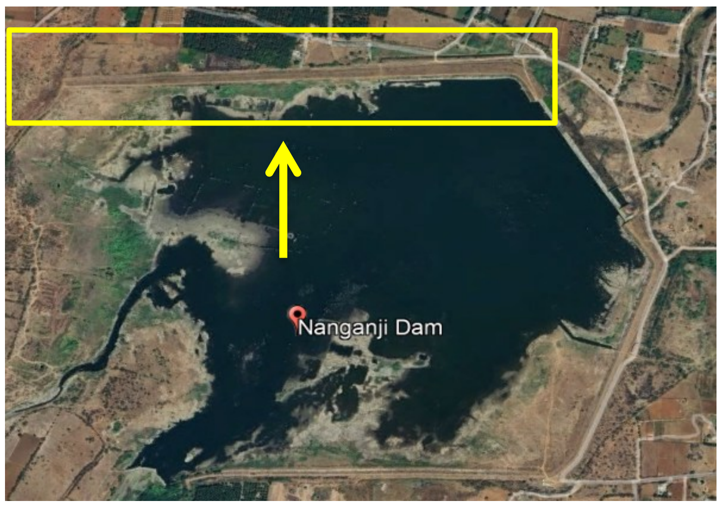


Figure 1. Aerial view of Boreholes in left flank of Nanganjiyar Reservoir

The Aerial view of Boreholes in left flank of Nanganjiyar Reservoir is shown in Figure 1 and locations of the boreholes are tabulated below as Table 1

Table 1 Location of Boreholes

Sl. No.	Borehole No.	Location
1.	BH I	LS 375m Top of the earthen embankment
2.	BH II	LS 450m Top of the earthen embankment
3.	BH III	LS 630m Top of the earthen embankment
4.	BH IV	LS 375m D/s side of the embankment
5.	BH V	LS 450m D/s side of the embankment

4. LABORATORY INVESTIGATION – SOIL

4.1 Bore Hole @ LS 375m

The bore hole was drilled on the left flank of earthen embankment from the top using Hand Auger. SPT and UDS collection were alternatively carried out for a penetration depth of 1.5m. A total of 9 disturbed samples and 2 undisturbed samples were collected. The 'N' Value recorded at a depth of 1.50m, 3.00m, 6.00m and 8.25m are 13, 18, 19 and >60 (for 15cm penetration) respectively. The classification of the soil is carried out as per IS 1498:1970 (reviewed 2021). Soil samples collected at various depths in this borehole along with the classification is listed in Table 2.

Table 2 Classification of soil in BH – 1 @ LS 375m

S. No.	Depth below TBL/ R.L +235.50m	Soil Classification as per IS 1498:1971
1.	0 m to 2.30 m	SM-SC (Silty sand – Clayey sand)
2.	2.3 m to 4.00 m	SP (Poorly Graded sand)
3.	4.0 m to 8.25 m	SC (Clayey sand)

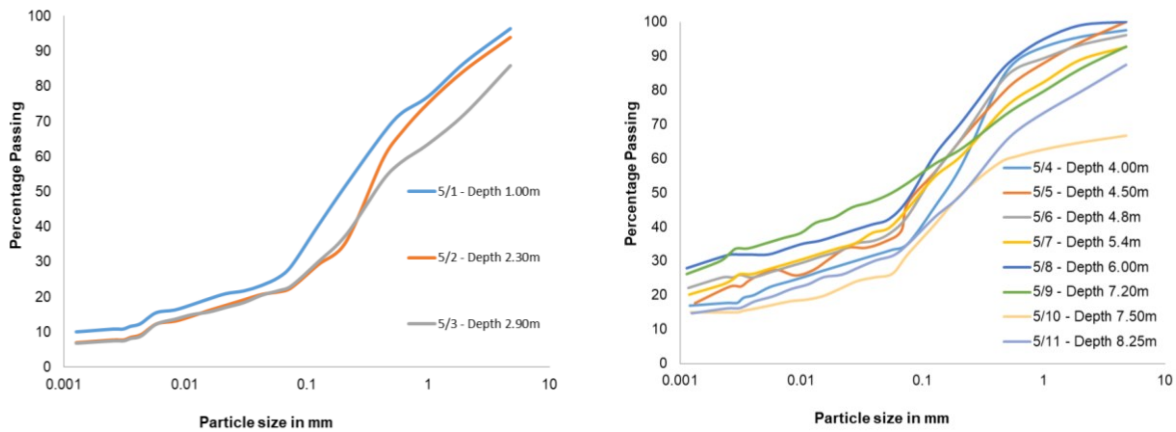


Figure 2. Grain Size Distribution Curves for Samples from BH - 1@ LS 375 m

From the grain size distribution curve shown in Figure 2, it is inferred that the sand content of the soil varies between 50 – 70%. This clearly indicates the embankment has less water retention capacity. As, the top of the embankment is not covered with bitumen top, there is possibility of seeping of water into the embankment during rainy seasons. The free swell index of the soil is less than 50% representing Low to medium degree of expansion. The natural moisture content of the soil varies from 10 % to 23% from a depth of 4.00 m to 8.25m and clearly indicates that the moisture content of the sample is increasing with respect to depth. The same is observed during boring operation also. Permeability test was carried out for the samples collected at a depth of 6.00m. The profile of the soil is similar from a depth of 4.00 m to 8.25m. The co-efficient of permeability of the soil is 2.405 x 10^{-4} cm/sec.

Undisturbed samples were collected at depths of 4.50 m and 7.50m. The shear strength of these samples was tested in the tri-axial test apparatus. The samples were tested for consolidated un-drained condition. The Mohr's envelopes of the sample are presented below as figure 3 & 4. From the test results, it is inferred that the embankment material exhibits good shear characteristics and is stable. The effective and total shear parameters obtained from Mohr's circle is tabulated in table 3.

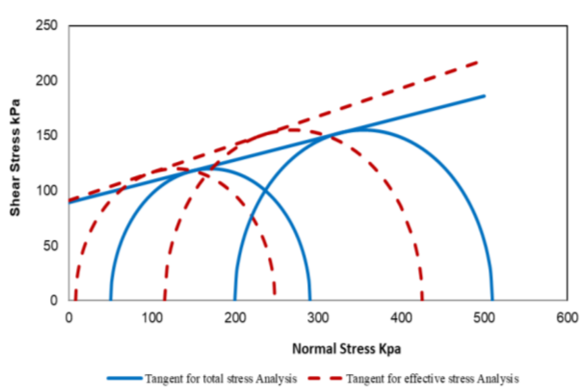


Figure 3. Mohr circle of sample No. 5 Triaxial CU (BH – I, LS 375m, Depth 4.5m)

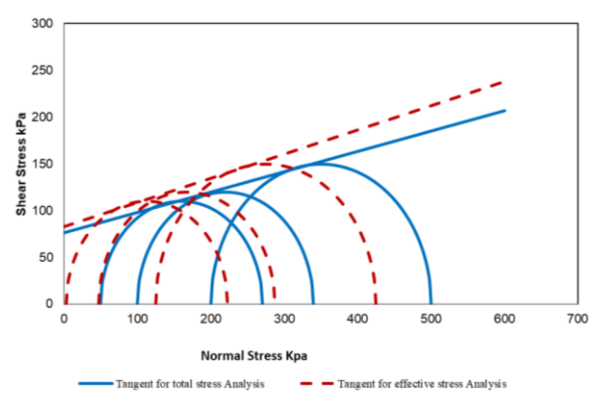


Figure 4. Mohr circle of sample No. 10 Triaxial CU (BH – I, LS 375m, Depth 7.50m

Table 3 Shear parameters of tested samples in CU Tri-axial shear test

Sample No.	Depth	C, kN/m²	Φ, Degree	C', kN/m²	Φ', Degree
5	4.5 m	89	11	91	14
10	7.5 m	76	12	83	15

4.2 Bore Hole @ LS 450 m

A total of 8 disturbed samples and 2 undisturbed samples were collected. The 'N' Value recorded at a depth of 1.50m, 3.00m, 4.50m and 8.52m are 13, 18, 19 and >50 (for 12cm penetration). Soil samples collected at various depths in this borehole along with the classification is listed in Table 4.

Table 4 Classification of soil in BH – 1 @ LS 375m

Sl. No.	Depth below TBL/ R.L +235.50m	Soil Classification as per IS 1498:1971
1.	0 m to 5.30 m	GC (Clayey Gravel)
2.	5.30 m to 7.00 m	SC (Clayey sand)
3.	7.0 m to 8.50 m	SM-SC (Silty Sand and Clayey Sand)

From the grain size distribution curve it is inferred that the gravel content of the soil varies from 0 - 55% and the sand content of the soil varies between 19 – 78%. This clearly indicates the embankment has less water retention capacity. The liquid limit of the soil sample varies between 27 – 43 %, indicating the degree of severity for expansion is ranging from Non-critical to Marginal. The natural moisture content of the soil varies from 5 % to 19% from a depth of 0 m to 8.00m. It is observed that the moisture content of the sample is increasing with respect to depth. Permeability test was carried out for the three samples collected at LS 450m the values are given in table 5

Table 5 Co-efficient of Permeability for samples at LS 450m

S. No.	Depth below TBL/ R.L +235.50	Soil Classification as per IS 1498:1971	Co-efficient of Permeability, 'k' cm/sec
1.	0 m to 5.30 m	GC	7.68×10^{-6}
2.	5.30 m to 7.00 m	SC	2.41×10^{-4}
3.	7.0 m to 8.50 m	SM-SC	1.35×10^{-2}

From the permeability values, it is inferred that the soil is pervious from 7.0m to 8.50m. Undisturbed sample was collected at a depth of 6.00m. The effective and total shear parameters obtained from Mohr's circle is tabulated in table 6. From the test results, it is inferred that the embankment material exhibits good shear characteristics and is stable.

Table 6 Shear parameters of tested samples in CU Tri-axial shear test

Sample No.	Depth	C, kN/m²	Φ, Degree	C', kN/m²	Φ', Degree
18	6.00m	32	25	33	35

4.3 Bore Hole @ LS 630 m

A total of 9 disturbed samples and 2 undisturbed samples were collected. The 'N' Value recorded at a depth of 1.50m, 3.00m, 4.50m and 9.00m are 13, 28, 11 and >50 (for 8 cm penetration) respectively. Soil samples collected at various depths in this borehole along with the classification is listed in Table 7.

Table 7 Classification of soil in BH – 3 @ LS 630m

Sl. No.	Depth below TBL/ R.L +235.50	Soil Classification as per IS 1498:1971
1.	0 m to 3.60 m	SW (Well Graded Sand)
2.	3.60 m to 5.8 m	SP (Poorly Graded Sand)
3.	5.80 m to 6.00 m	SM-SC (Silty Sand and Clayey Sand)
4.	6.00 to 9.00 m	SC (Clayey sand)

From the grain size distribution curve, it is inferred that the sand content of the soil varies between 59 – 72%. Grain size distribution curve resembles the curve of a sand dominated sample. This clearly indicates the embankment is made up of semi-pervious material. The liquid limit of the soil sample varies between 21– 43 % for samples between depth 5.8m to 9.0m , indicating the degree of severity for expansion is ranging from Non-critical to Marginal. The embankment is made up Non-plastic well and poorly graded sand up to a depth of 5.8m. As the top of the embankment is not covered with bitumen top, there is possibility of seeping of water into the embankment during rainy seasons. The free swell index of the soil is less than 50% representing **low to medium degree of expansion.** The natural moisture content of the soil varies from 3 % to 20% from a depth of 5.80 m to 8.00m. It is observed that the **moisture content of the sample is increasing with respect to depth**. The same is observed during boring operation also. The effective and total shear parameters obtained from Mohr's circle is tabulated in table 8. From the test results, it is inferred that the embankment material exhibits good shear characteristics and is stable.

Table 8 Shear parameters of tested samples in CU Triaxial shear test

Sample No.	Depth	C, kN/m^2	Φ, Degree	C', kN/m^2	Φ', Degree
31	6.00	22	33	39	37

4.4 Bore Hole @ LS 375 m at downstream side of the embankment

The borehole BH – 4 drilled @ LS 375 m at downstream portion near to the road, using rotary drilling was penetrated to a depth of 4m from the existing ground level. Hard rock was encountered at a depth of 2.5m itself. Boring operation was continued to retrieve rock samples for carrying out necessary compression strength. However, drilling could not be continued beyond 4m depth as the drill bit was stuck in the hard rock. Hence JCB was brought to retrieve the rod. Hence a trial pit was made around the rod itself. During this operation it was noticed that there is continuous flow of water. A photo of the same is shown below as figure 5.

Sample @ 2.00 m – 3.00m depth

Flow of water observed at BH – 4, d = 2.0m

Figure 5. Photos of Rotary drilling samples at LS 375 m

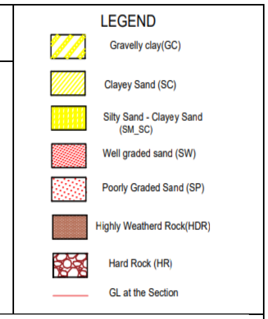
The unconfined compressive strength test on rock core sample collected at the depth of 3.50 m at BH IV drilled at LS 375 m is 41.51 N/mm^2.

4.5 Bore Hole @ LS 450 m at downstream side of the embankment

The borehole BH – 5 was drilled @ LS 450 m downstream portion near to the road, using rotary drilling was penetrated to a depth of 11.5 m from the existing ground level. Highly weathered rock was encountered at a depth of 2.0m. Hard rock was encountered at a depth of 10m. The picture of the rotary drilling is shown as figure 6. The individual and the consolidated bore log of all the boreholes is shown as figure 7

Rotary drilling at LS 450m	Sample @ 5m depth	Sample @7.0m depth	Sample at 10.5m depth (15cm)

Figure 6. Rotary Drilling at LS 450m



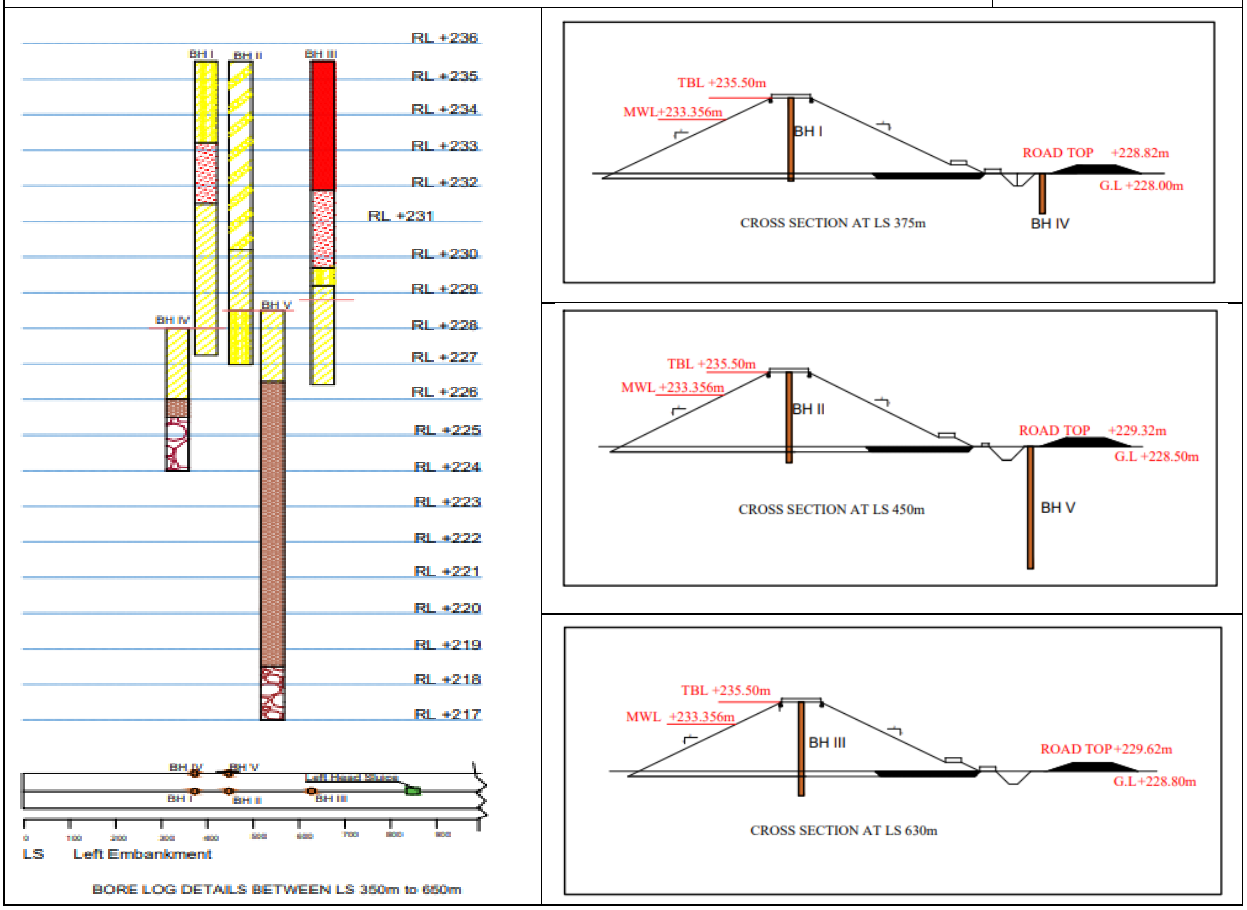


Figure 7. Consolidated borelog.

4.6 Dispersivity of soil – Laboratory test

The crumb test, double hydrometer test, pinhole test and chemical analysis are conducted on selected samples to ascertain the dispersive characteristics of the soil samples collected in the embankment. Based on the various tests consensus has been arrived as non-dispersive in nature.

5.0 ANALYSIS OF TOE DRAIN SEEPAGE RECORDS OF NANGANJIYAR DAM

Observation from the Seepage recorded in the Left V1 Notch @ LS 850m.

The cross section of the earthen embankment at LS 850m parallel to left V1 notch @ LS 850m is shown in Figure 8.

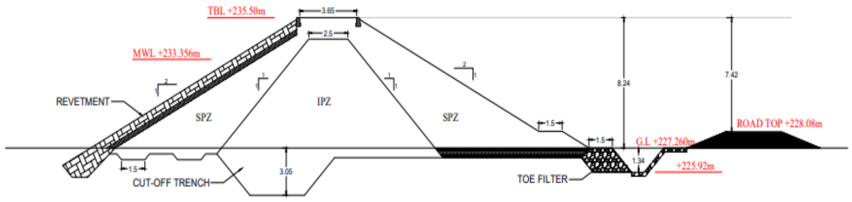


Figure 8 Cross section at LS 850m nearer to Left V Notch

5.1 General:

- While analysing the records of seepage from 2008, it is observed that Nanganjiyar reservoir receives rainfall from October to January.
- The high intensity of rainfall in the above months, results in substantial seepage in the Left V1 Notch.
- The reservoir FRL condition is +231.00 m, this level has been reached only during the years 2021, 2022 and 2023 right from its inception in the year 2008. However, level nearer to FRL condition has been reached during the years Jan 2008, Dec 2011 and Jan 2012 also
- The reservoir is in FRL condition @+ 231.00 m for 75 days during the year 2022 and is the maximum period in the past 15 years.

5.2 Seepage

Steady Seepage

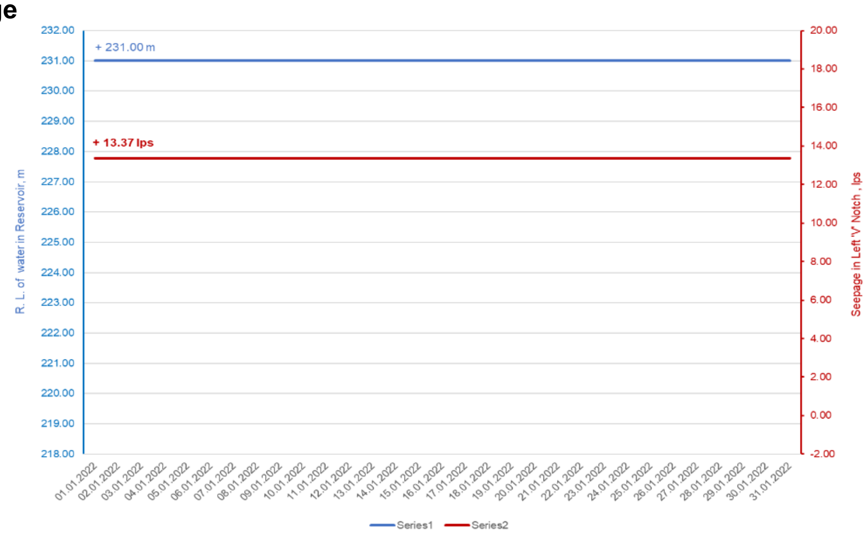


Figure 9. Seepage in the V Notch with respect to Reservoir Water Level for the period of Jan 22

- When the reservoir was at FRL during the month of January 2022, the steady seepage observed in the Left V1 notch is 13 lps, whereas the steady seepage observed in the Left V1 Notch for the month of December 2022 is 18lps.

Observed Seepage increase

- During the month of Jan 2021, when the reservoir level increased from +230.87 m to +231.00 m, a sudden increase of seepage from 0 lps to 13 lps (+13 lps) was observed in the toe drain.

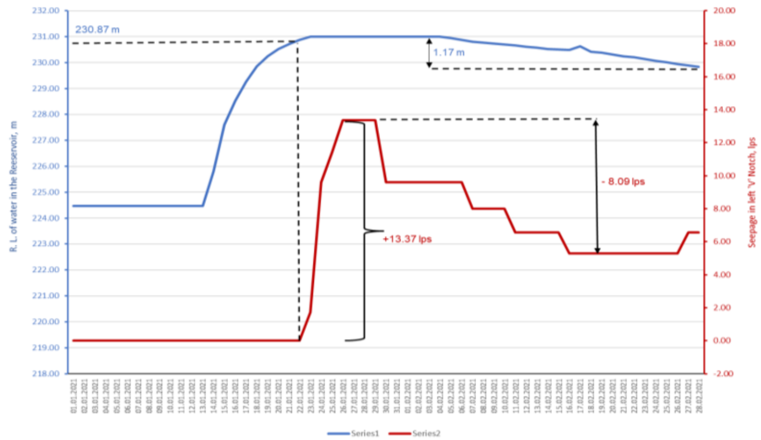


Figure 10. Seepage in the V Notch with respect to Reservoir Water Level for the period of

January – February 2021

- Similarly, a spike in the seepage was observed in November 2021, when the reservoir level increased from +229.90 m to + 231.00m, the seepage increased from 3.21 lps to 13.37 lps (+10.16 lps).
- A year after in November 2022, a similar spike was observed in the same month, when the reservoir level increased from +229.35 m to +231.00 m, the seepage increased from 5.28 lps to 17.9 lps (+12.6 lps)

Observed Seepage decrease

- In the month March 2012, when the level of reservoir receded from +228.84m to +227.50m, there is drastic reduction of seepage from 11.39 lps to 0.75lps (-10.64 lps)
- Similarly, a dip in the seepage was observed in Jan 2021. When the level of reservoir receded from +231.00m to +229.83m, the seepage reduced from 13.37lps to 5.28 lps(-8.09 lps)

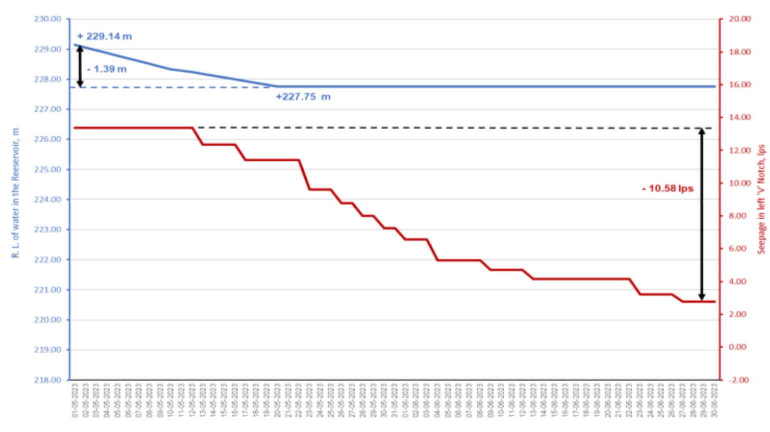


Figure 11 Seepage in the V Notch with respect to Reservoir Water Level for the period of
May – June 2023

- In June 2023, when the reservoir level was receding from 229.14m to 227.75m, a sudden drop from 13.37 lps to 2.79 lps (10.58 lps) was observed in the measured seepage.

Interpretation from seepage record

- Whenever the R.L of water in reservoir exceeds +227 m, there is commencement of seepage in the left V1 Notch @ LS 850 m.

- Similarly, when the reservoir level, recedes from the FRL (+231.00m) and decreases beyond +227 m, the seepage in the left V1 Notch reduces significantly.
- Hence, from the seepage data, it is evident that there is drastic increase and decrease in the seepage observed in the left V – Notch, when the reservoir level reaches +229.0 m and +231.0 m
- As per the seepage record, the steady seepage observed in the reservoir is 18 lps for FRL +231.00m. Whenever this level is maintained in the reservoir for a period of 1 month (30 days), then the loss of water due to seepage will be 4,66,56,000 litres.
-

6 Graphical Method of Flow Net Analysis

Flow net analysis carried out at LS 375m adopting Casagrande graphical method is given as Figure 16. In this analysis FRL condition of the reservoir is taken.

6.1 Flow Net at LS 375 m for FRL condition +231.00

The expression for calculation the discharge passing through the flow net is

$$q = kh \frac{N_f}{N_d}$$

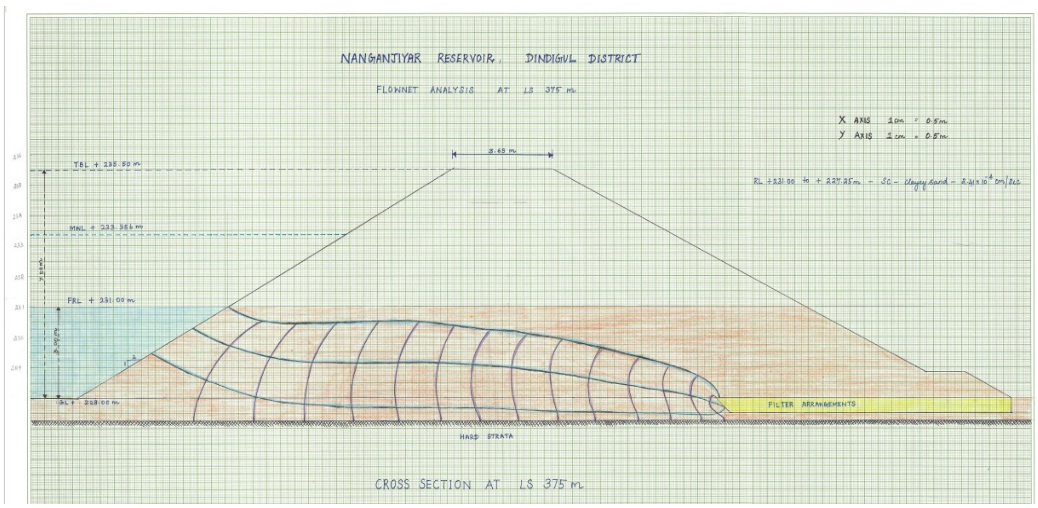


Figure 12. Flow net analysis @ LS 375 m

$$q = 104\ litres/day\ per\ 'm'\ length\ of\ the\ \ embankment$$

6.2 Flow Net at LS 450 m for FRL condition +231.00

Flow net analysis carried out at LS 450 m adopting Casagrande graphical method is given as Figure 13. In this analysis FRL condition of the reservoir is taken

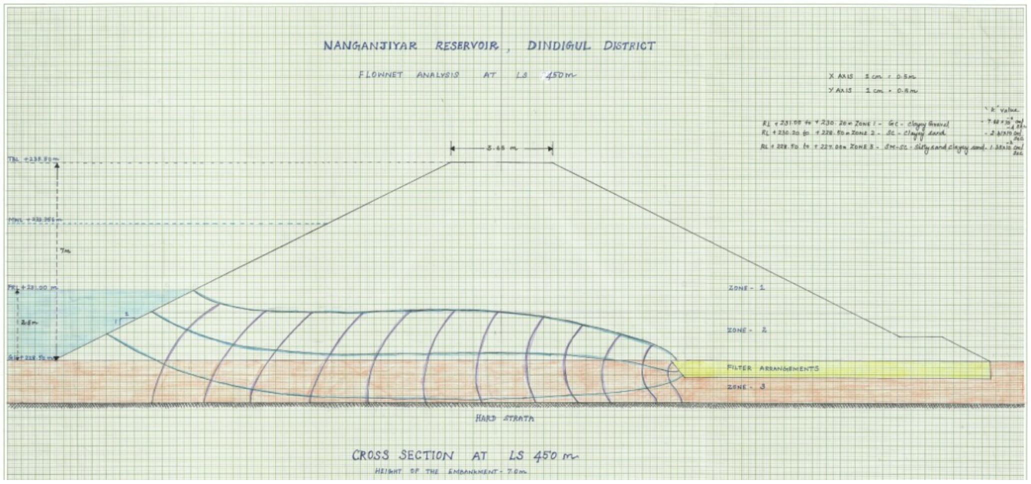


Figure 13. Flow net analysis @ LS 450 m

$$q = 2230\ litre/day\ per\ 'm'\ length\ of\ the\ embankment$$

7. INTERPRETATION FROM FLOW NET ANALYSIS AT LS 375M AND AT LS 450M

- In flow net analysis at LS 375m for FRL condition at +231.00m, the head causing flow is 3.0m from EGL, the flow completely takes place within the Clayey sand, SC of co-efficient of permeability in the range of 10^{-4} cm/sec itself. The Seepage calculated is 104 litres / day per ‘m’ length of the embankment.
- In flow net analysis at LS 450m for FRL condition at +231.00m, the head causing flow is 2.5m from EGL, the flow takes place dominantly in Clayey sand, SC (thickness 1.7m) of permeability in the range of 10^{-4} cm/sec and Silty sand – Clayey Sand, SM-SC (thickness 1.5m) of co-efficient of permeability in the range of 10^{-2} cm/sec. The discharge calculated is 2230 litres / day per ‘m’ length of the embankment.
- There is no permissible limits of seepage in the earthen embankments prescribed in IS codes. However, the book on “Handbook of Geotechnical Investigation and Design table” by Burt Look (Published by Taylor & Francis, London) discusses the topic on seepage loss through earth dams. The extract of the content is given below.

“*All dams leak to some extent. Often this is not observable. Design seeks to control that leakage to an acceptable level. Guidance on the acceptable seepage level is vague in the literature. The following is compiled from the references, but interpolating and extrapolating for other values. This is likely to be a very site and dam specific parameter*”

Table 9 Seepage Values vs. Dam Height

Dam Height (m)	*Seepage, litres / day / metre (Litres/minute/metre)*	
	O.K.	*Not O.K.*
< 5	*< 25 (0.02)*	*>50 (0.03)*
5 - 10	*< 50 (0.03)*	*>100 (0.07)*
10 - 20	*<100 (0.07)*	*>200 (0.14)*
20 - 40	*< 200 (0.14)*	*>400 (0.28)*
> 40	*< 400 (0.28)*	*>800 (0.56)*

The earthen embankment height of Nanganjiyar Reservoir ranges between 7 – 7.5m between LS 350m to LS 500m. The permissible seepage for a Dam Height of 5 - 10m from table 9 is less than 50 litres / day per m length of the embankment. While comparing the above, to the calculated seepage values at LS 375m and LS 450m in Nanganjiyar Reservoir it can be noted that the seepage values come under Not O.K as per Table 9. This implies that the seepage in the Nanganjiyar Reservoir is significant, which needs to be attended.

8. CONCLUSIONS

1. The laboratory investigation shows the embankment of Nanganjiyar Dam is made up of different layers of soil and there is no specific core material encountered.
2. In general, the cut off trench of the earthen embankment is to be made of clay soil having co-efficient of Permeability 'k' value in the order of 10^{-6} cm/sec. However, the soil below the embankment at LS 350 m and at LS 450m has a co-efficient of Permeability 'k' in the order of 10^{-4} cm/sec and 10^{-2}cm/sec, which indicates the pervious nature of the soil. The pervious to semi pervious nature of the soil in the foundation of the embankment is the cause for the profuse seepage between LS 350m to LS 500m.
3. From the flow-net analysis, it is learnt that the seepage through the Nanganjiyar Dam is not under the permissible limit and needs to be attended.
4. Hence, the profuse seepage between LS 350m to LS 500m in the earthen embankment of Nanganjiyar Reservoir needs to be attended by proper upstream treatment methods after necessary discussion with DSRP and SPMU, DRIP.

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Latest Technologies for Rehabilitation and Dam Safety, 20-22 March 2025, Shimla

Risk analysis of the Koyna dam using reduced order model and machine learning

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KEYWORDS
Reduced order model, Risk analysis, Fragility, Machine learning, Dam

ABSTRACT:
Several important dams in India, such as Bhakra Nangal, Hirakud, and Koyna, were constructed decades ago. Over time, their structural and mechanical properties deteriorate due to material aging, environmental changes, and natural disasters like earthquakes impacting their performance under dynamic conditions. Thus, their continual monitoring and safety assessment becomes crucial. A key challenge in such assessments is determining the changed distribution of material properties throughout the dam. We need to install sensors at various locations and then estimate such distributions for various parameters. Once we know these distributions, we can use fragility analysis to estimate the reliability of the dam. However, traditional methods are computationally inefficient for such an analysis with varying material properties. This study presents a novel long short-term memory (LSTM) integrated reduced order model designed to estimate fragility curves efficiently, incorporating the variability in material properties and seismic events. With the developed methodology, fragility curves can be estimated considerably faster compared to the full-scale models. Therefore, informed decisions can be made about the safety of the existing dams. The proposed method was verified for the Koyna Dam, Maharashtra, India, using models of varying complexities --- from 2D linear to 3D nonlinear. Numerical studies demonstrate that the proposed approach achieves high accuracy and substantial computational speed-up, making it a practical tool for efficient seismic risk analysis of large dam structures.

1. INTRODUCTION

Dams are vital in society, serving as critical infrastructure for water management, energy generation, and flood control. In India, several important dams, such as Bhakra Nangal, Hirakud, and Koyna, were constructed decades ago. These dams were constructed with the then-available knowledge and resources, which may sometimes lead to inappropriate design. For instance, in 1967, the Koyna dam sustained significant damage during an earthquake of magnitude 6.6. Horizontal cracks observed in the non-overflow monolith revealed unexpected seismic behavior. Moreover, the mechanical and structural properties of these dams deteriorate with time due to material aging, environmental changes, and natural disasters like earthquakes impacting their performance under dynamic conditions. Thus, their timely monitoring and safety or reliability assessment using state-of-the-art methods become crucial (Chopra and Zhang, 1991, Hartford and Baecher, 2009). This paper presents a novel strategy to estimate the reliability of existing dams under varying material properties and seismic loading.

Ensuring the safety of existing dam reservoir systems requires determining the changed distribution of material properties throughout the dam. We need to install sensors at various locations and then estimate such distributions for various parameters. Once we know these distributions, we can use fragility analysis to assess the reliability of the dam. The fragility analysis offers a method for safety evaluation and decision-making by employing a probabilistic framework to account for the various uncertainties influencing the performance of the dam (Lupoi et al*.,* 2006, Varsha and Manohar, 2024). Seismic fragility is defined as the probability of failure, conditioned on the intensity measure (IM) of the earthquake, with respect to a predefined limit state (Ellingwood et al., 2001, Hariri-Ardebili, et al., 2016). Different IMs (Hariri-Ardebili and Saouma, 2016), such as peak ground acceleration (PGA), spectral acceleration, and peak ground velocity, are widely used in the literature to estimate the fragility curves corresponding to various limit states. Traditional methods of fragility analysis are, however, computationally inefficient. Existing methods use small sample sizes (Wang et al. 2018) that introduce a large coefficient of variation (CoV) in the estimate of the fragility curve. To reduce the CoV, we need a large sample size; however, this requirement will make the estimation of the fragility curve computationally prohibitive for large and complex structures like dams.

Thus, an alternative approach is needed, and accordingly, a new approach is proposed in this work. The above-mentioned computational issue is addressed here by adopting reduced order modeling. In the last two decades, non-intrusive reduced order models (ROMs) have been widely used for the efficient analysis of large and complex structures (Xiao et al., 2017, Guo and Hesthaven, 2018, Halder et al. 2022, Sun and Choi, 2021 Bharti and Ghosh, 2024a, 2024c). Recently, Bharti and Ghosh, 2024b have developed proper orthogonal decomposition (POD)-based non-intrusive ROMs for the reliability estimation of stochastic systems under random excitations. The LSTM-integrated non-intrusive ROM developed by Bharti and Ghosh, 2024b efficiently simulates complex nonlinear systems under random external loading, providing significant computational savings over full-scale numerical methods. The current work extends the ROM proposed in (Bharti and Ghosh, 2024b) to incorporate material uncertainty, which, in combination with random loading, can lead to significant changes in system response and, consequently, in the fragility curve estimation. This paper introduces two-fold novelties: (i) development of a modified LSTM-integrated ROM and (ii) a more comprehensive and accurate fragility curve estimation that accounts for material uncertainty.

The performance of the LSTM-integrated ROM and its modified version are numerically tested for the Koyna dam reservoir model. The full-scale modeling of the dam-reservoir system, including hydrodynamic interactions, is performed using Abaqus (ABAQUS Inc., 2006). The training dataset is generated by running multiple HDM simulations. This process is fully automated through a MATLAB-Abaqus-Python interface. Numerical studies show that the LSTM-integrated ROMs accurately predict various quantities of interest, such as relative displacement, velocity, and stress distribution. Further, the ROMs are found to be efficient in estimating the fragility curves for different limit states and achieved a significant speed-up of about three orders of magnitude.

The remainder of the paper is organized as follows: The proposed ROM is presented in Section 2, followed by numerical modeling and studies in Section 3. Conclusions are presented in Section 4.

2. THE PROPOSED LSTM-INTEGRATED ROM

Let us consider a randomly excited time-invariant nonlinear dynamical system with parameter $\boldsymbol{p} \in \mathbb{R}^{N_p}$

$$\mathbf{M}(\boldsymbol{p})\, \ddot{\mathbf{u}}(\boldsymbol{p},t) + \mathbf{C}(\boldsymbol{p})\dot{\mathbf{u}}(\boldsymbol{p},t) + \mathbf{f}_{nl}\big(\mathbf{u}(\boldsymbol{p},t), \dot{\mathbf{u}}(\boldsymbol{p},t)\big) = \mathbf{f}(\boldsymbol{p},t) \quad (1)$$

with zero initial conditions, that is, $\boldsymbol{u}(\boldsymbol{p},0) = \mathbf{0}$, and $\dot{\boldsymbol{u}}(\boldsymbol{p},0) = \mathbf{0}$. Here $\mathbf{M}(\boldsymbol{p})$ and $\mathbf{C}(\boldsymbol{p}) \in \mathbb{R}^{N\times N}$ are mass and damping matrices, respectively; $\mathbf{f}_{nl}(\cdot)$ is the nonlinear restring force. $\mathbf{u}(\boldsymbol{p},t) \in \mathbb{R}^{N}$ denotes displacement and a dot over $\mathbf{u}(\boldsymbol{p},t)$ denotes derivative with respect to time. Eq. 1is referred to as high dimensional model (HDM). N and N_p denote the dimension of HDM and parametric space, respectively.

The first step in the construction of a non-intrusive ROM of Eq. 1 is to compute POD bases $\boldsymbol{\Phi} \in \mathbb{R}^{\mathrm{N}\times\mathrm{m}}$, where $m \ll N$. These bases are constructed by performing singular value decomposition (SVD) on the snapshot matrix. Readers are advised to go through (Bharti and Ghosh, 2024a) for detailed information on POD bases. After computing the bases, reduced space solution $\mathbf{u}_{\mathbf{r}}(\boldsymbol{p},t) \in \mathbb{R}^{N}$ are found as

$$\mathbf{u}_{\mathbf{r}}(\boldsymbol{p},t) = \boldsymbol{\Phi}^{T}\mathbf{u}(\boldsymbol{p},t) \quad (2)$$

where $\boldsymbol{\Phi}^{T}$denotes the transpose of $\boldsymbol{\Phi}$. The last step involves the construction of a surrogate model $\mathcal{S}$ between $\boldsymbol{p}$ and $\mathbf{u}_{\mathbf{r}}(\boldsymbol{p},t)$, that is, $\mathcal{S}\colon \boldsymbol{p} \to \mathbf{u}_{\mathbf{r}}(\boldsymbol{p},t)$. As mentioned in Section 1, (Bharti and Ghosh, 2024b) developed regression based surrogate model using LSTM network for randomly excited dynamical systems. The developed LSTM-integrated ROM (Bharti and Ghosh, 2024a) consider only excitation uncertainty, assuming the system parameters to be deterministic. This limitation is addressed here by proposing a modified ROM, which now incorporate the system parameters uncertainties along with random excitations.

Among various possible ways of handling this issue, the modification presented in Fig. 1 is adopted in this paper. This figure shows the core idea of creating an LSTM network between the combined parametric space (excitation and system) and the corresponding reduced space solution. For brevity, $\mathbf{u}_{\mathbf{r}}(\boldsymbol{p},t_1)$ is denoted as u_{t_1}in Fig. 1. In the proposed modified ROM, the system parameters, along with the random excitations, are given as input to the LSTM network. However, as the system is assumed to be time-invariant, the same values of the parameters are fed at each time instant. This modification gives the proposed LSTM network explicit information of the parameters, enabling it to capture the dynamics of the system in a more accurate way. After the construction of this ROM, the fragility curve of the dam can be estimated efficiently. We can simply evaluate the surrogate model for new set of parameter values $\boldsymbol{p}^*$ and to find $\mathbf{u_r}(\boldsymbol{p}^*, t)$. The corresponding HDM solution $\mathbf{u}(\boldsymbol{p}^*, t)$ is computed as

$$\mathbf{u}(\boldsymbol{p}^*, t) = \mathbf{\Phi}\mathbf{u_r}(\boldsymbol{p}^*, t) \quad (3)$$

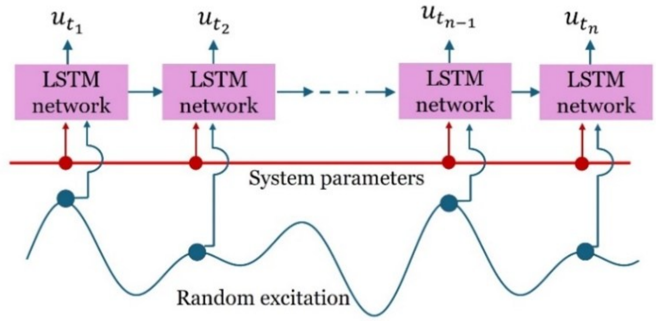


Figure 1: LSTM-integrated ROM

Once we know the HDM solutions, we can estimate the fragility curve of any desired quantity. This whole procedure bypasses the need to solve the HDM for a new set of parameter values and, thus, gains computational efficiency. We tested the accuracy and efficiency of the proposed ROM for the Koyna dam. The modeling details of the Koyna dan and numerical results are described next.

3. NUMERICAL STUDIES

3.1. 2D model

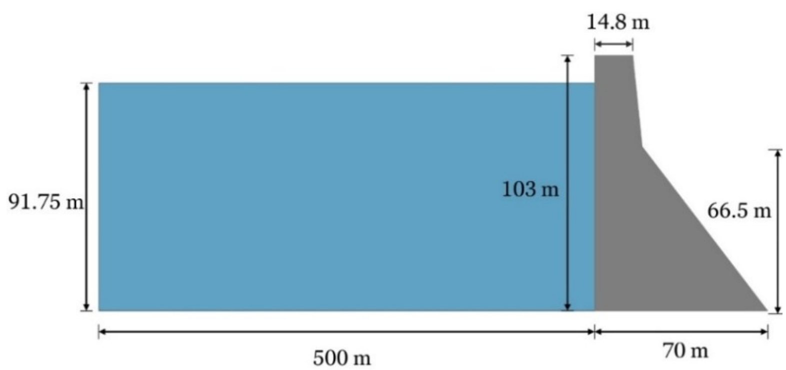


Figure 2. Schematic of the Koyna dam

The dam section, shown in Fig. 2, is modeled after the Koyna dam with the concrete properties as presented in Table 1. The dam is modeled as a 2D concrete gravity dam with a height of 103 m, a base width of 70 m, and a crest width of 14.1 m. A finite rectangular reservoir of 500 m in length with water up to a height of 91.75 m impounded by the dam is considered. Later, the analysis is extended to a 3D model. The full-scale analysis is performed using the Abaqus finite element software.

The dam section comprises 2356 nodes and 2250 elements, and the reservoir model has 6666 nodes and 6500 elements. The gravity load and the hydrostatic force, which accounts for the static pressure exerted by the water on the dam, are considered in addition to the hydrodynamic interaction between the dam and the reservoir. A continuum plasticity-based damage model, developed in Lee, J. and Fenves, G.L.,1998. is employed to model the inelastic behavior of concrete. To validate our Abaqus model, we plotted the propagation of the damage using the *DAMAGET* output variable in Fig. 3. It is seen in this figure the dam suffers significant damage at the kink and heel region. This behavior is in accordance with the observation of the crack formation and propagation in the Koyna dam due to the earthquake in 1967.

After validation, we created the LSTM-integrated ROM for this model for UQ and the fragility analysis. As mentioned earlier, seismic fragility is defined as the probability of the exceedance of the engineering demand parameter (EDP) beyond a chosen limit state (LS) for a given intensity measure (IM) of earthquakes. In this paper, maximum relative displacement $u_{\max}$ and PGA are chosen as the EDP and IM, respectively. In this paper, the probability of exceedance or failure probability yp_f is estimated using Monte Carlo simulation given as

$$p_f = \mathrm{P}(u_{\max} \geq LS|\mathrm{IM} = im) \approx \frac{N_{LS}}{N_S} \qquad (4)$$

where N_{LS} is the number of instances of $u_{\max}$ reaching or exceeding the LS, and N_S is the total number of samples.

Table 1. Material properties of the concrete gravity dam

Material property	Symbol	Unit	Model
Modulus of elasticity	E_c	MPa	Lognormal (23.9, 0.1)
Poisson's ratio	ϑ_c	-	0.15
Density	ρ_c	kg/m^3	2643
Dilation angle	ψ	-	Lognormal (3.5, 0.1)
Compressive initial yield stress	σ_{c_0}	MPa	13.0
Compressive ultimate stress	σ_{c_u}	MPa	24.1
Tensile failure stress	σ_{t_0}	MPa	2.9

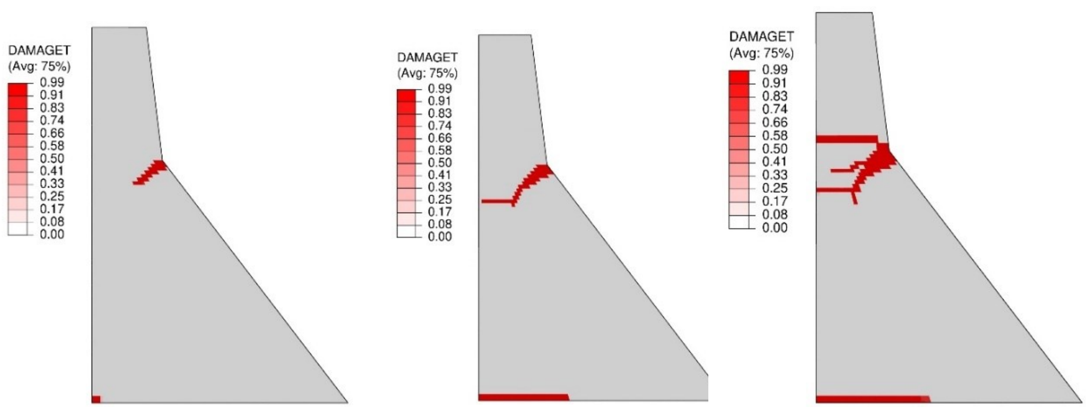
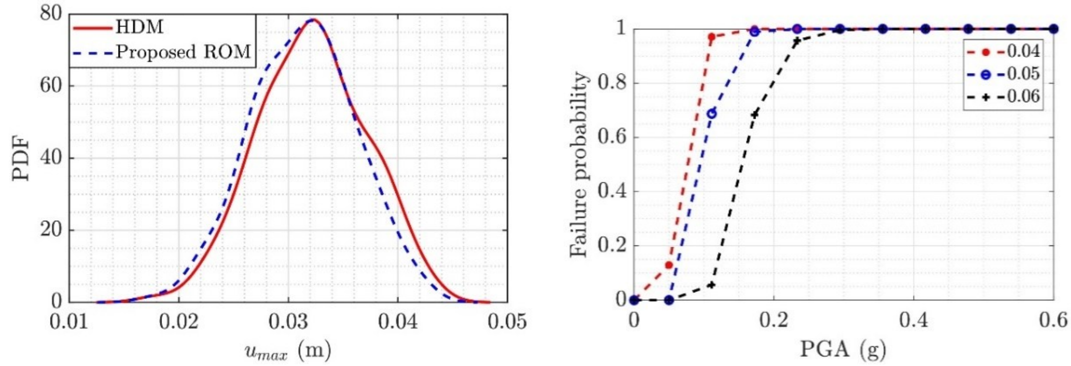


Figure 3: Propagation of the damage in the Koyna dam model

Next, we present the accuracy and efficiency of the proposed ROM. For this numerical example, Young's modulus and the dilatancy angle are considered to be uncertain with lognormal distribution, details of which are tabulated in Table 1. To construct the proposed LSTM-integrated non-intrusive ROM, first, a set of training samples consisting of random excitations, Young's modulus, and the dilatancy angle, is realized. The HDM --- modeled in Abaqus --- is then solved for each sample. Subsequently, the POD bases are computed, and the reduced solutions are found. Now, an LSTM network is trained between the input and output data. The number of input features is three, and the number of output features is equal to the number of POD bases. Each training point is a $3 \times N_t$ matrix, where N_t denotes the sequence length. The accuracy of the modified ROM is checked in predicting the PDF of maximum relative displacement $u_{\max}$. The ROM is trained using 1000 samples. It can be observed in Fig. 4 (a) that the ROM is accurate in capturing the target PDF. Further, Fig. 4 (b) shows fragility curves for three different limit states. A total of fifteen hundred thousand simulations are required to plot these fragility curves. Table 2 shows the total computational time for plotting these fragility curves. It can be inferred from this table that the fragility analysis using the HDM is practically impossible as it would take around 3.7 years to compute these values in a multi-core computer. However, the proposed ROM took only 31 hours, giving an overall speed-up of about 1047. These studies show the developed ROM is highly efficient in estimating the fragility curve of the dam with uncertainty in material properties. The applicability and efficiency of the LSTM-integrated ROM for the 3D dam model, which offers additional computational difficulties, are tested next.



(a) Probability density function

(b) Fragility curve

Figure 4: Nonlinear dam under non-stationary excitations with material uncertainty. (a) PDF of maximum relative displacement $u_{\max}$. (b) Fragility curves.

Table 2. Computational cost in obtaining the fragility curve.

Core	HDM (estimated)	Proposed ROM			Speed-up	
		Offline	Online	Total	Online	Overall
Multi (8 cores)	3.7 years	21.6 hr	9.36 hr	30.96 hr	3463	1047

3.2. 3D model

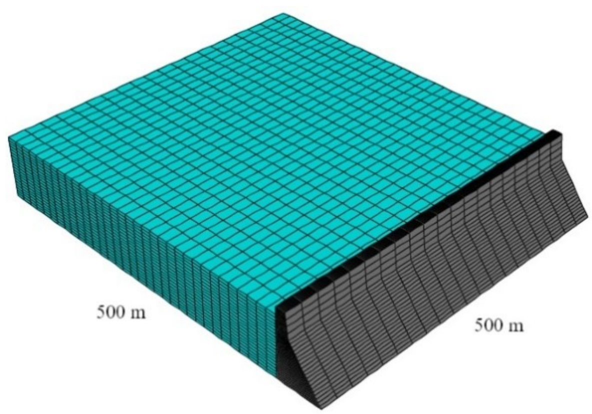
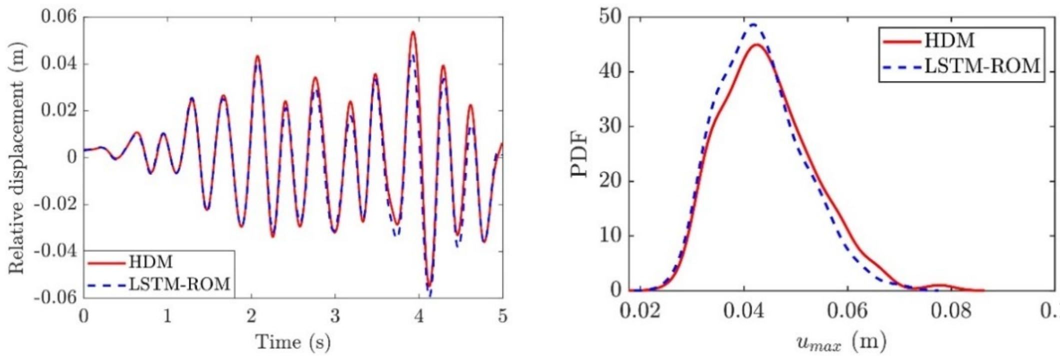


Figure 5: 3D schematic of the Koyna dam

The schematic of the 3D dam model, along with its finite element mesh, is shown in Fig. 5. This model was generated by extruding the 2D dam model, illustrated in Fig. 2, by 500 m along the direction perpendicular to the plane of the paper. The dam structure is discretized using eight-node C3D8R elements. Similarly, the reservoir part is modeled with eight-node, three-dimensional brick acoustic AC3D8 elements. The finite element meshing process resulted in a total of 29,446 elements and 32,676 nodes. The ROM is trained using 200 random non-stationary excitations. POD bases are constructed using a two-step SVD method. The accuracy of the trained ROM in capturing the relative displacement of the top node is demonstrated in Fig. 6 (a), where the ROM is observed to exhibit good accuracy. Furthermore, the ROM effectively captures the PDF of the maximum displacement $u_{\max}$, as illustrated in Fig. 6 (b). These results confirm that the ROM performs accurately for a large-scale system and can be reliably employed for the fragility analysis of the dam.

4. CONCLUDING REMARKS

This paper presents an efficient and accurate approach to the fragility analysis of a dam. To account for material uncertainties alongside variations in seismic ground motion, a modified version of the ROM is introduced. The LSTM-integrated ROM demonstrates accuracy across a spectrum of complexities,



(a) Displacement-time history (b) PDF of $u_{\max}$

Figure 6: Nonlinear 3D dam under non-stationary excitations. (a) displacement-time history (b) PDF of maximum relative displacement $u_{\max}$

from 2D dam models to large nonlinear 3D dam models. Numerical results validate that the modified ROM achieves high accuracy while delivering a computational speed-up of three orders of magnitude, making it highly suitable for fragility analysis.

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A few concepts of Geology applicable to dam safety.

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Abstract

Dam failures are normally associated with catastrophic consequences, like loss of life and property, followed by several related sufferings. Dam failures can occur through several possible reasons. Among them, more than 50% cases could be mainly due to geological reasons. It is attempted to present here a few prominent geological reasons, which can be a source of a dam failure and also a few geological observations towards dam safety. Types of geological studies to be carried out in different stages of investigations, prominent geological features, a few important foundation issues related to dam failures are discussed. The need of critical evaluation of geological issues at every suspected malfunctioning of dam and its appurtenant works is highlighted. Also, suggested that projects that pose serious safety risks and aged dams having recurring issues of their safety may be decommissioned in a phased manner, either complete removal of the dam or partial reduction or lowering of dam's height duly utilizing current state-of-the-art technology related to dam safety.

Introduction

As per National Register of Large Dams (2023), India has a total of 6281 specified dams (6138 completed and 143 under construction). It is the third highest number in the world after China (23,841) and the USA (9,263). Among them about 234 large dams are above 100 years old which need constant vigil. Dam failures are normally associated with catastrophic consequences, like loss of life and property, followed by several related sufferings. Dam failures can occur through several possible reasons. A few of prominent causes that can lead to a dam failure are such as, flood event, piping/seepage (through dam body or dam foundations), land slide, earthquake, foundation failure, hydromechanical equipment failure (malfunction of gates, etc.), structural failure, etc. Among them, it was reported that about 34% were caused by overtopping, 30% due to foundation defects, 28% from piping and seepage, and 8% from other modes of failures. (Costa, 1985) and (USBR, 1998). It can be implied that by adding 30% due to foundation defects, 28% from piping and seepage, if both added together, they amount to be more than 50% cases of failures which could be mainly have geological reasons. It is attempted to present here a few prominent geological reasons, which can be a source of a dam failure.

1. Geological studies during Investigation stage of a dam project

Geological studies basically help to get information of the terrain and its foundation characteristics. The studies get fine tuned in different stages of investigations, add more information at every stage, such as, reconnaissance stage, preliminary investigation stage and detailed investigation stage.

During the stage, it is also necessary to take up Geophysical traverses at specified locations which compliment geological mapping to indicate prominent characteristics of rock formations and overburden soils.

Subsequently, it is necessary to take up drilling activity, to drill a few boreholes to required depths at proposed locations of important project components and study core samples collected as core recovery. The study is necessary to know thickness of overburden soils present and more about the sub-surface geological information.

At the time of drilling, it is necessary to conduct in-situ permeability tests using packers in rock formations at different depth sections to assess permeability of the terrain.

Laboratory tests to study mainly soil properties like grain size, cohesion, angle of internal friction, plasticity, permeability etc., and rock properties like compressive strength and tensile strength etc., need to be conducted as per the need of the project to formulate its design and execution.

On having a detailed examination of all the tests carried out, one can understand that it is a difficult proposition in nature to find a flawless and uniform location to build a dam at the point where we require. In order to design a stable dam, engineers should have a thorough understanding of the existing site conditions. It involves an extensive geological and geotechnical site investigations. This will help to evaluate critical geotechnical issues and to incorporate defensive design measures to counteract adverse conditions. In fact, the structure needs to be suitably designed to suit the prevailing ground conditions. The data thus obtained to be used to prepare a baseline geological report.

(i) Physiography

Normally, physiography and topography of the terrain governs selection of place and type of dam to be constructed (apart from concerns of political, socio-economical etc.). Except in cases where only one specific type of dam is obvious the possible alternative types of dams and their layouts will have to be investigated and explored to take a final decision. Similarly, Geology and foundation conditions also influence the type of dam to be preferred.

(ii) Geological mapping

In order to design a dam, whether it is concrete or earth or rock fill dam, it is necessary to carry out geological mapping of the terrain and to conduct certain investigations at dam foundations. Investigations may be by means of test pits, trenches, auger holes or drill holes as per requirement of ground conditions. These are required to study nature of rock and soils present at locations of proposed dam and its appurtenant structures and actual depth of bed rock, need to collect samples systematically to determine sequence of soil strata, and to assess percolation and permeability data and also to evolve suitable methods of treatment against any unfavorable ground conditions noticed. Possibility of presence of pervious strata, settlement or shear failures may also be ascertained in the process. The number of drill holes required and their locations may be guided by of geology of the terrain.

If foundations are found predominantly having gravel, they need to be compacted well and they may be suitable for earth fill and rock fill dams. Silt and fine sand foundations are also considered suitable for earth fill dams, but such foundations may pose problems in respect of settlement, piping, excessive percolation etc. Rocky foundations too can pose problems, especially with the intervening soft layers, shear zones, fault zones, gouge material etc., which may require specific treatment and special design requirements. The foundation condition at the dam site has to be studied and the design of the dam to be made to suit ground conditions. The studies of foundation investigations and laboratory tests should always be supplemented by geological judgment to have a suitable design.

(iii) Investigation for borrow areas

Prior to design of a dam, it requires to take up borrow area study. Borrow areas need to be investigated to assess availability, suitability, nature, quality, quantity and other characteristics of construction material needed of the dam within economical reach of the proposed dam site. The most economical type of dam will often be one for which construction materials are found in sufficient quantity within reasonable distance from the site. Based on laboratory tests, their engineering properties are assessed and available quantities are estimated. The strength of concrete / masonry should exceed the maximum stress anticipated in the structure by a safe margin. The concrete / masonry / mortar should be satisfactory in regard to placing characteristics, weathering resistance, impermeability and resistance to alkali-aggregate reaction. Other conditions being satisfied, availability of sufficient quantity of fine and coarse aggregate and rock will be favourable factor for construction of concrete / masonry dam. On the other hand, if suitable soils of required quantity are available in the nearby borrow areas, earth fill dam may prove to be economical. It is therefore, necessary to take advantage of available local resources and design the structure suitably. It becomes the task of a Geologist to ensure enough material of acceptable quality is available in the vicinity of construction site. Starting the construction activity without ascertaining properties of construction materials being used may lead to time and cost overruns.

(iv) Karst topography

Limestone terrain can exhibit karstic nature, where there is a possibility of having sinkholes and solution channels. Geohydrological tests may indicate whether the cavernous limestone is competent enough to hold water under reservoir conditions. Tracer tests by using chemicals and fluorescence dyes may indicate presence of any interconnection of sub-surface solution channels.

(v) Palaeochannels

Presence of paleochannels in the terrain in near vicinity of the proposed dam site, especially, if they are extending across the dam, and if they are not properly identified, delineated, and addressed in advance, there is likelihood that they may turn out to be vulnerable in respect of depth of foundation excavation and subsequently competency of the reservoir also can be affected badly. Such features need to be studied during geological mapping of the area and if required to drill a few boreholes there, study core recovery and to conduct in-situ permeability tests in the boreholes drilled.

(vi) Reservoir rim stability

Detailed geological mapping of the proposed reservoir area, especially all along the proposed rim of the reservoir is necessary to identify presence of any landslides, unstable slopes or any other vulnerable features, which might affect competency of reservoir.

(vii) Geophysical studies

Geophysical studies may be applied in engineering geological investigations, to determine depth of bedrock underneath the overburden soil cover, to assess rock condition in the foundation medium, etc. Geophysical methods used may be broadly grouped as, magnetic, electrical, electromagnetic, gravity, seismic and radiometric. Different methods are used as per requirements of terrain conditions.

(viii) Seismicity

An earthquake is shaking of the surface of the Earth resulting from a sudden release of energy in the Earth's lithosphere that creates seismic waves. The seismicity or seismic activity of an area is the frequency, type, and size of earthquakes experienced by that area over a period of time. Any major structure needs to be designed to meet safety requirements against the established seismicity of the terrain.

(ix) Liquefaction

Liquefaction occurs in soils during shaking of terrain in tremors. In the process, water-saturated granular material, such as sand and other type of soils, temporarily loses its strength and transform from solid to liquid state. Soil liquefaction may cause rigid structures, like buildings, bridges and such heavy structures, to tilt or sink into the liquefied deposits, and eventually collapse upon themselves. Dams are to be built to avoid such terrain conditions and if needed, to design structure so as to meet such vulnerable conditions.

2. Geological studies during Construction stage of a dam project

On excavation of dam foundations, as a part of foundation preparation, adequacy of depth excavated and suitability of the foundation medium thus exposed need to be geologically assessed keeping in view of the structure to be built over it. Objective of geological investigations during construction stage is mainly to keep a record of geological features exposed during construction or excavation of the project requirements and to appraise construction and design engineers regarding any special or vulnerable geological feature revealed in the excavation which could not be inferred during earlier period of investigation stage. It may help to address the new feature, if required, in updating design or in subsequent construction.

(i) Foundation mapping

The final foundation of dam or any other important structure need to be geologically mapped. Scale of study may be on 1:100/200 or should depend on importance of the structure and severity of vulnerable feature identified, if any. Presence of vulnerable geological conditions, such as, bedding plane weaknesses, pockets of weathered seams, pockets of loose rock, shear zones, fault zones, solution cavities etc., need to be identified, delineated and to be removed from the foundation.

(ii) Dental treatment and Shasta formula

Proper foundation study is necessary for economical execution of dam construction and to minimize any kind of post construction problems. Despite many prior studies to finalise foundation grade, there may be always a possibility to encounter a surprise, for which the design is not addressed. Experience reveals that it may be a difficult task to find a dam foundation without a shear zone. It happens often, that the foundation excavation exposes a geological fault, seam, shattered or inferior rock condition, extend to such depth that it is impracticable to remove the same at the foundation grade. Hence, it is always necessary to evaluate foundation condition during construction, especially at the final foundation grade. These conditions may require special treatment in the form of selective removal of weak material and backfilling with concrete or to make certain acceptable adjustment. It is also a common practice to drive rock bolts across such shear seam. This procedure of reinforcing and stabilizing such weak zones is called 'dental treatment.'

In the process, a procedure adopted by United State Bureau of Reclamation (USBR) in connection with foundation problem encountered at Shasta and Friant dams and to determine depth of treatment of transverse seams, led to an empirical equation, which has been subsequently standardised in the name of Shasta formula, and the same is being followed by many in field to treat shear zones which are small in width compared to size of the dam blocks. As per the Shasta's formula, all weak material along the length of shear zone to be excavated to a depth 'D' and then the same to be filled with rich mix of concrete before laying concrete on adjacent foundations. Proper filling, compactness and consolidation may be ensured in the filling.

(i) $D = 0.00656bH + 1.526$ (for $H > 46$ m) and also
(ii) $D = 0.3b + 1.524$ (for $H < 46$ m)

Where, H = height of dam above general foundation level in meters;
b = width of weak zone in meters;
D = depth of excavation within the weak zone from general foundation level in meters.

The formula provides some guideline to indicate how much depth to be excavated. But final geological judgment may be exercised in the field during actual excavation operations.

(iii) Consolidation grouting

A few foundations may require to be treated by drilling and providing with consolidation grouting with cement slurry. Consolidation grouting is mainly pumping cement slurry by applying low pressure to seal off crevices, joints, seams faults etc., present in foundations. Consolidation grouting is the usual method of treatment to make foundations compact, monolithic and for general improvement of foundation condition. Consolidation grouting is normally provided upto a maximum depth of 10 m, spaced at 3 m c/c. Also, can be decided as per site conditions.

(iv) Curtain grouting

The high-pressure curtain grouting, normally carried out through foundation gallery or cut-off trench provided in dam body. It aims at providing a water tightness across the dam foundation and facilitating a maximum drop in the hydraulic gradient through the curtain to reduce the uplift. Cut-off trenches are provided to extend the impervious zone of embankment with the foundation.

(v) Contact Grouting

Contact Grouting is provided to fill up the shrinkage gap and voids, if any, between reinforced concrete slab and plain concrete slab or rock; or between plain concrete slab and fractured rock/shear zone.

3. Geological studies during post-construction stage of a dam project

After successful completion of project, there is a possibility in a few projects, development of a few problems, which may distract functioning of the project to its full capacity. Or else, after putting certain life span of the project, there could be possibility of developing certain issues which may be of great concern to the safety and stability of the project. In such cases, as a part of dam safety and enhancing effective life of the project, a team of dam safety engineers along with an engineering geologist may try to understand and assess the problem, and if it has a source to its geological or geotechnical background, he may try to assess the problem in a detailed way. Geological maps prepared during construction stage vis-a-vis design drawings are to be reviewed and if any lapses occurred in the process, they need to be brought to the notice to the concerned. Additional measures as a part of treatment and or strengthening the existing foundations may be worked out.

(i) Rock fatigue

It is indicated that there are over 1,115 large dams about 50 years old, 234 large dams 100 years old, and 64 large dams will be 150 years old in India by 2025. Apart from all other reasons, aging of a dam also can be a cause of dam deterioration due to rock fatigue that leads to gradual natural degeneration, which may eventually fail.

(ii) Foundation defects

Dam failure due to foundation defects may mainly lead to differential settlement, development of cracks in dam body and development of slope instability on flanks etc. Another important foundation defect is excessive seepage through dam foundation and when it is not properly filtered and soil particles due to internal erosion, are carried away through seepage water, which ultimately lead to piping action in foundation beneath the dam body.

(iii) Vulnerability of plunge pool and apron

A few dams / spillways have plunge pool and/or apron in their design to facilitate passage of water from reservoir towards downstream side. When water plunges on to the downstream side, there is a possibility that the rock formation present at immediate downstream of the dam body might get effected and if it is not properly protected and strengthened, rock formations can get further deteriorated and can effect safety of dam.

(iv) Acidic nature of water

Acidity of the reservoir waters can damage underwater parts of the dam and other appurtenances. Asphalt concrete having prominent bitumen, is widely used in construction of impervious facings and central cores in dams and dykes. Acidic water reacts with bitumen, mine tailings and mineral materials and may cause deterioration of asphalt concrete with time.

(v) Alkali aggregate reaction

Alkali aggregate reaction is a type of chemical reaction that occurs in concrete between the alkaline cement and aggregates present in concrete: Alkali–aggregate reaction is a reaction between the alkali hydroxides in the pore solution of concrete and certain aggregates; Alkali–silica reaction involves certain silica minerals in the aggregate such as opal, chert, flint, and volcanic ash that have a high concentration of silica. Alkali–carbonate reaction involves certain argillaceous dolomites with a specific texture. Alkali–aggregate reaction is an expansive-type reaction in which micro fractures are initiated and subsequently developed as prominent cracks damaging concrete.

(vi) Siltation of reservoir

Dams are normally designed for approximately 100 years of effective age. Their functional life gets decreased with progressive reservoir sedimentation concurrently reducing project benefits. The loss of storage capacity of large dams over time is part of the ageing process. Large storage structures, whether concrete, masonry, or earth, can become vulnerable with age.

4. Conclusions

(i) All dams of more than 100 years age, have to be reviewed annually for their safety point of view and to take needful action promptly.

(ii) It is necessary to have periodic review of dams that have reached 50 years of age and above, to assess their safety and take an informed decision about their repairs.

(iii) Apart from dams aged above 50 years, there are many other dams in the country having even less age, developed with serious problems of their safety. They also need to be placed under periodical review and to address the requirements without much delay in their deterioration.

(iv) Projects that pose serious safety risks and aged dams should be listed and decommissioned in a phased manner, either complete removal of the dam and its associated structures or partial reduction or lowering of dam's height duly utilizing current state-of-the-art technology related to dam safety.

(v) A poor understanding of geology of the terrain can be a major cause of dam failure. Hence, it is suggested to study geology of the terrain and proper maintenance of records of geological observations made at the dam site and to take a view point of geological background information at every stage of dam construction and its maintenance. It needs a critical evaluation of geological issues at every suspected malfunctioning of the dam and its appurtenant works to assess any geological bearing on the dam safety and dam failures.

Lessons Learnt from Dam Failure Incidents

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ABSTRACT:

Dams are critical infrastructure for water resource management, flood control, and energy generation. However, dam failures have occurred throughout history, resulting in significant loss of life, environmental damage, and economic losses. This paper examines the primary causes of dam failures, such as structural deficiencies, overtopping, seepage, poor maintenance, and natural disasters through case histories, including the Malpasset Dam (France), Teton Dam (USA), Vaiont Dam (Italy), Machchhu-2 Dam (India), and Teesta Dam (India). The study highlights the diverse factors contributing to these failures. The paper emphasizes the importance of comprehensive risk assessments, robust design and construction practices, regular maintenance, and early warning systems to prevent similar incidents in the future. Key lessons learned from past failures underscore the need for improved dam safety protocols, better monitoring technologies, and enhanced emergency preparedness. Ultimately, the paper provides insights into how the dam industry can evolve to minimize risks and enhance the safety and resilience of dams worldwide.

INTRODUCTION

Dams play a critical role in managing water resources, controlling floods, supplying water for irrigation, and generating hydroelectric power. These massive structures are essential for ensuring water security, energy production, and agricultural sustainability, benefiting millions of people worldwide. In many regions, dams help mitigate the impact of droughts and floods by regulating river flows, providing a stable water supply for both urban and rural areas. However, despite their engineering marvel and economic importance, dam failures have occurred throughout history, often leading to catastrophic consequences. When a dam collapses, the sudden and uncontrolled release of vast amounts of water can trigger widespread flooding, causing mass casualties, large-scale displacement of communities, destruction of infrastructure, and severe environmental damage. The impact of dam failures extends beyond immediate loss of life, as they can lead to long-term economic and ecological crises, affecting industries, livelihoods, and entire ecosystems**.** Understanding the root causes of dam failures and learning from past disasters is crucial for enhancing dam safety, improving construction techniques, and developing better monitoring and maintenance strategies. This paper examines multiple case studies of historical dam failures, analysing the specific factors that led to their collapse. By exploring these incidents, we aim to identify key lessons that can be applied to strengthen dam design, enhance operational practices, and establish more effective risk mitigation measures.

CAUSES OF DAM FAILURES

Dam failures can occur for a variety of reasons, often involving a combination of factors related to design flaws, construction deficiencies, poor maintenance, environmental conditions, and operational failures. The primary causes of dam failures can be broadly categorized into the following:

1. Structural Deficiencies and Design Flaws

One of the most common causes of dam failure is inadequate design or construction flaws. Structural weaknesses such as poor materials, inadequate foundations, or improper design for the dam's intended purpose can lead to the failure of a dam under stress. Issues with the spillway, inadequate drainage systems, or failure to account for extreme weather conditions can exacerbate these problems.

2. Overtopping and Inadequate Spillways

Overtopping occurs when water exceeds the height of the dam, leading to erosion of the dam face or embankment. This is typically caused by inadequate spillways or improper dam sizing, which fails to account for extreme flood events. Many dam failures are attributed to overtopping during heavy rainfall or snowmelt.

3. Seepage and Foundation Weakness

Seepage through a dam or its foundation can undermine the structure's integrity. If water seeps through cracks or weak points in the dam or its foundation, it can erode the material over time, leading to progressive failure. Internal erosion, often referred to as "piping," can also occur, leading to a collapse of the dam.

4. Inadequate Maintenance and Operational Failures

Dam safety relies heavily on routine inspections, maintenance, and proper operation. Lack of maintenance, including the failure to inspect critical components such as spillways, embankments, and monitoring systems, increases the risk of failure. Additionally, operational errors, such as failure to properly manage water levels or respond to emergency situations, can exacerbate the consequences of dam malfunctions.

5. Natural Disasters

Natural events such as earthquakes, landslides, and extreme weather can cause dam failures, especially if the structure was not designed to withstand such forces. Earthquakes can destabilize dams, while floods and landslides can overwhelm the dam’s ability to control water flow.

CASE HISTORIES OF DAM FAILURES

Several dam failures have highlighted the importance of understanding the underlying causes and taking preventive measures. The following case histories illustrate some of the key factors that contribute to dam failure and the lessons learned from these incidents.

1. Malpasset Dam Failure, France (1959)

The construction of Malpasset Dam in France began in April 1952 and was finished in 1954. On December 2, 1959, the Malpasset Dam in France failed catastrophically after a combination of poor construction materials and design flaws led to the collapse of the dam's structure.

Figure 2: Ruins of Malpasset Dam

Figure 3: Location of Malpasset Dam

Dam was built with a faulty grout curtain and inadequately compacted materials, which weakened its ability to withstand pressure from water stored behind it. The collapse of the dam resulted in 423 deaths and significant damage to surrounding towns.

Lessons Learned:

- The need for proper materials and construction techniques, particularly when it comes to ensuring dam stability and integrity.
- The importance of comprehensive design reviews and safety assessments before construction begins.

2. Teton Dam Failure, USA (1976)

The construction of Teton Dam in USA began in 1972 & was completed in November 1975. One of the most catastrophic dam failures in U.S. history, the failure of the Teton Dam in Idaho on June 5, 1976, resulted in the flooding of downstream areas, leading to 11 deaths and extensive property damage.

Figure 4: Teton Dam Failure

Figure 5: Teton Dam

The failure was caused by a combination of poor foundation design and inadequate geological investigations. The dam was built on a foundation of highly permeable material, which allowed seepage to occur. Over time, this seepage eroded the dam’s embankment, leading to its collapse

Lessons Learned:

- The importance of thorough geological investigations and foundation design cannot be overstated. The dam's design failed to account for the presence of weak and permeable soil layers.
- The necessity of monitoring and maintaining seepage control systems in embankment dams.
- The failure also highlighted the importance of early warning systems and emergency preparedness.

3. Vaiont Dam Failure, Italy (1963)

The Vaiont Dam was constructed between 1957 to 1960. The failure of the Vaiont Dam on October 9, 1963, in Italy is one of the most tragic events in dam history, with over 2,000 lives lost. The dam, which was located in a narrow canyon, suffered from a massive landslide that triggered a wave of water to overtop the dam, causing its collapse. The landslide was caused by unstable geology and poor understanding of the area’s seismic and geological conditions.

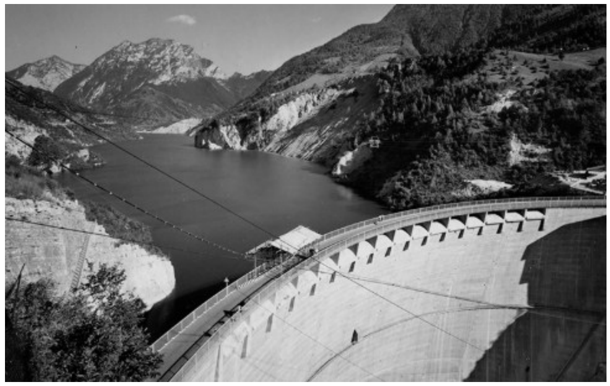

Figure 6: Vaiont Dam, Italy

Figure 7: Vaiont Dam Disaster

Lessons Learned:

- A comprehensive understanding of local geology and the potential for natural events such as landslides is essential for the safety of dams built in challenging terrain.
- Dams built in narrow valleys or geologically unstable areas must include detailed hazard assessments.

The importance of adequate risk management strategies to address unforeseen natural events, such as landslides.

4. Machchu-2 Dam Failure, India (1979)

Machchu-2 Dam was built in 1959, having a catchment area of 730 square kilometres. On August 10, 1979, the Machchhu-2 Dam in Gujarat, India, failed after intense monsoon rains caused the dam to overflow. Inadequate spillway capacity and poor maintenance were identified as key factors contributing to the failure.

Figure 8: Machchuu-2 Dam Failure

The dam was unable to handle the excessive water load, leading to the breach of the embankment and flooding of nearby villages, resulting in approximately 1,000 deaths.

Lessons Learned:

- The need for a reliable and adequately sized spillway system to manage flood events.
- Regular inspection and maintenance are critical to identifying vulnerabilities such as cracks or erosion in embankments.
- The importance of effective flood forecasting and early warning systems to help mitigate the impact of such disasters.
- The necessity for robust monitoring systems to detect early signs of potential failure.

5. Teesta Dam Failure, India (2019)

The construction of Teesta Dam was completed in 2013 located in Sikkim. In December 2019, the dam's upper reaches suffered structural damage, which led to the collapse of a portion of the dam's embankment. As a result, a massive amount of water was released downstream, leading to severe flooding in areas along the river. The disaster caused significant damage to infrastructure, property, and human lives affecting the state of Sikkim and the neighbouring regions.

Figure 9: Teesta Dam Failure

Lessons Learned:

- Ensuring dams are built with the highest safety standards.
- Frequent inspections to detect potential weaknesses.
- Establishing early warning systems and evacuation plans.
- Mitigating damage to ecosystems and habitats.
- Educating those near dams about risks and safety measures.
- Updating dam safety protocols regularly.
- Coordinating efforts between authorities for a swift response.

11.1.1 KEY LESSONS LEARNED FROM DAM FAILURES

From the analysis of historical dam failure incidents, several key lessons have emerged that are essential for improving dam safety and ensuring the protection of communities and the environment.

1. Comprehensive Risk Assessments

Before building a dam, a comprehensive risk assessment should be conducted. This includes understanding the geological conditions, hydrological data, and potential natural hazards (e.g., earthquakes, floods, landslides). Dam design must account for these factors, with contingency plans in place for unforeseen events.

11.1.1.1 2. Improved Design and Construction Practices

The importance of robust engineering design and high-quality construction materials cannot be overstated. Design flaws, particularly in the foundation and spillway systems, are often primary causes of dam failures. Dams must be built to withstand extreme conditions, including heavy rainfall, floods, and seismic events. Additionally, the use of modern technologies such as geotechnical monitoring equipment can help detect early signs of structural failure.

3. Regular Maintenance and Inspection

Dams require regular maintenance and inspections to ensure they remain safe over time. Routine checks of structural integrity, spillway functionality, and drainage systems are essential. Early identification of weaknesses or damage, such as cracks or seepage, can prevent catastrophic failures. Implementing a rigorous maintenance schedule is essential for long-term dam safety.

4. Monitoring and Early Warning Systems

Installing modern monitoring systems, including sensors to track water levels, seepage, and structural movement, can help identify potential problems before they become critical. Early warning systems can provide vital information that allows for timely evacuations and flood management efforts to mitigate the effects of dam failure.

5. Public Awareness and Emergency Preparedness

Communities living downstream of dams should be educated about the risks of dam failure and the appropriate actions to take in case of an emergency. Emergency response plans should be developed and regularly tested, ensuring that communities are prepared for a quick evacuation if necessary.

CONCLUSION

Dam failures, while rare, have had devastating consequences throughout history. The lessons learned from past incidents emphasize the need for rigorous design, construction, and maintenance standards, as well as comprehensive risk assessments and effective emergency management systems. By adopting these lessons, the safety of dams can be greatly improved, reducing the risk of failure and ensuring the protection of human lives and the environment. With the enactment of the Dam Safety Act, 2021, India has established a uniform regulatory framework for dam safety, ensuring stricter safety regulations and inspections nationwide.

To prevent dam-related incidents, the Act mandates:

- Regular Inspections & Risk Assessments to identify potential hazards.
- Dam Break Analysis for each specified dam to evaluate hazard potential and formulate effective Emergency Action Plans (EAPs).
- Proper Maintenance of Operation & Maintenance Manuals at all dam sites.
- Comprehensive Dam Safety Evaluations to be conducted every ten years for long-term risk assessment.

Additionally, the Act includes penal provisions for non-compliance. Failure to adhere to dam safety regulations can result in penalties, and government officials who neglect their safety responsibilities can also be held accountable.

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DAM SAFETY ASSURANCE UNDER CLIMATE CHANGE

By Rajib Chakraborty, Chief Advisor, Water Resources: Lea Associate South Asia

INTRODUCTION: Climate change refers to long-term shifts in temperatures and weather patterns. As a result of climate change, the Earth's average surface temperature is now approximately 1.2°C warmer than it was in the late 1800s (prior to the industrial revolution) and warmer than at any point in the last 100,000 years. The impacts of climate change include severe flooding, sudden cloudbursts, intense droughts, water scarcity, devastating fires, rising sea levels, melting polar ice and glaciers, catastrophic storms, and declining biodiversity. According to the IHE Delft Institute for Water Education, many of the world's 70,000+ dams face a heightened risk of failure due to the increasing frequency and intensity of extreme weather events driven by climate change.

RECENT EVENTS OF DAM FAILURE DUE TO CLIMATE CHANGE EVENTS.
Numbers Of Dam Failure Events Have Been Observed In The Year Of 2024 Through Out The Globe Due To Climate Changes. Few Of Those Are As Below:

- Alua dam, impounding a major reservoir on the Ngadda River, one of the tributaries of the Lake Chad in Nigeria collapsed, causing catastrophic flooding in Borno State and killing over 150 people, with at least 419,000 people displaced.
- At the end of August 2024, Arbaat dam in Sudan collapsed, causing catastrophic flooding in downstream communities with over 60 people reported dead, and 20 villages destroyed.
- Nakuru Dam burst caused due to heavy rains and flash floods have left more than 168 people dead, 50 injured, 21 missing, 150,365 people (30,073 families) displaced, and more than 190,937 affected across Kenya as of April 28,2024
- The coastal city of Derna of Libya suffered most of the catastrophic flooding from Storm Daniel

Figure 1: Failure of Arbaat Dam in Suda

after two catchment dams burst, unleashing floodwaters that swept away whole neighbourhoods, imperilling thousands of residents in a region .

- On 5 April 2024, the Orsk Dam collapsed due to flooding along the Ural River in Russia
- In India Malana dam of Himachal Pradesh, India collapsed in 1st August,2024.
- On the intervening night of October 3 and 4, 2023, a glacial lake outburst flood (GLOF) in North Sikkim, India exacerbated by large hydropower projects on the Teesta River, caused havoc to people, environment and infrastructure downstream in Sikkim and West Bengal. Many lives were lost. The dam of the 1200 MW Teesta III hydropower project was washed away, leading to cascading impacts.
- Afterwards on 20th August,2024 a severe landslide in Sikkim has destroyed the Teesta-V hydropower station of the state-owned NHPC Limited. This was under restoration after the *Figure 2: Damages of Teesta III H P Project for GLOF* damage of previous year
  under restoration after the damage of previous year

Figure 2: Damages of Teesta III H P Project for GLO

CLIMATE CHANGE IMPACT ON DAM SAFETY

Failure of dams, as well as protective dikes and levees which are key infrastructures, has significant economic and social consequences. Historically, most risk assessments assumed a stationary climate, where variability in climate phenomena, including the frequency and intensity of extreme events, remained constant. However, climate change has introduced variations in factors such as extreme temperatures and the frequency of heavy precipitation events, which are likely to impact the drivers of dam-related risks. Updating risk components (such as loads, system response, and consequences) to reflect new climate scenarios is essential for adaptation and decision-making within a more resilient framework.

A comprehensive approach is crucial to addressing the influence of climate change on dam safety management. In this context, dam risk models provide a valuable foundation for structuring such assessments. Climate change is expected to impact multiple factors affecting dams, from incoming flood patterns to defining downstream consequences. To analyze these impacts comprehensively, it is necessary to break them down into the various elements that constitute dam risk. Certain techniques can facilitate such analyses, ensuring a structured and thorough approach to risk assessment.

RISK MODEL COMPONENTS IN DAM SAFETY

Loads of the System: This term corresponds to the loads to which the dam will be subjected and focuses on the upstream components of the dam. In particular, incoming floods are envisaged as the main hydrological load, and the rest of the component defines how the dam–reservoir system responds when confronted by such hydrological events.

System Response (or Failure Probability): This involves identifying potential failure modes and defining the conditional probability of failure under given circumstances.

Consequences (Economic, Loss of Life, or Other): This component estimates the downstream impacts of significant failure modes, including dam break modeling, to understand potential economic losses, fatalities, and other effects.

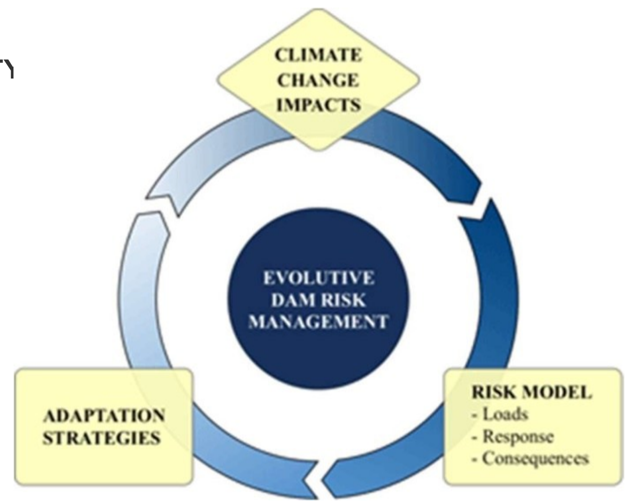

Figure 3: Dam Risk Manageme

In the risk modeling approach, the analysis is conducted comprehensively, evaluating total risk and climate change impacts together while considering their interdependencies. Climate change-driven risks are influenced not only by climatic factors but also by non-climatic drivers such as population growth, economic development, and water management adaptations. In some cases, these non-climatic factors play a substantial role in dam risk calculations and have been accounted for in the research.

FLOOD RISK MANAGEMENT

Flood risk management can be done efficiently through proper estimation of flood, controlling the reservoir level and improving the performances of gates.

FLOOD:

The primary component of dam safety affected by climate change is the hydrology of river basins, determined by incoming floods. While heavy precipitation plays a major role, other factors such as snow cover and snowmelt, vegetation, and soil moisture also influence flood behaviour. In cases where detailed catchment- level information is unavailable, site-specific analyses are necessary. Most studies rely on adapted global and regional climate models (GCMs and RCMs) combined with hydrological and land surface models to assess expected changes in flood patterns at the watershed level. However, climate change projections from GCMs often require downscaling and bias correction, as their spatial resolution is too coarse for regional or local hydrological modeling. While most downscaling techniques focus on reproducing average climate signals, some studies address extreme event projections. Despite abundant research on river changes over recent years, there remains limited global evidence on flood magnitude and frequency patterns. Nevertheless, physical reasoning suggests that variations in heavy rainfall and other factors in certain regions will likely lead to changes in local flood behaviour.

RESERVOIR WATER LEVEL

The distribution of water storage in a reservoir ,and thus the pool levels, determines the loads on a dam during flood events. A frequently full reservoir subjects the dam to higher hydrostatic loads. These distributions depend on factors such as inflows, demands, reservoir management rules, and water losses (e.g., evaporation and infiltration). These can be analyzed using historical pool level records or through simulations of water resource management systems. Under climate change, fluctuations in surface water availability are expected, driven by increased precipitation variability, evapotranspiration linked to global warming, and reduced snow and ice storage. Changes in agricultural land use, responsible for about 90% of global water consumption, further impact freshwater systems and irrigation needs. Additionally, demographic, socioeconomic, and technological changes, such as population growth and reservoir operation adaptations, influence water demand and allocation. These factors are likely to alter the balance between water availability and supply, impacting both the quantity and temporal distribution of stored water, which is critical for dam safety. Assessing these effects requires simulation-based analyses that incorporate uncertainties in climate and hydrological projections.

When assessing the effects of climate change on the distribution of the reservoir water levels, analyses must rely on the simulation of the system of water resource management. This allows the reproduction of the water balance in the reservoir under specific management rules and for future conditions. First, the inflows are assessed, preferably using long updated climatic series obtained from specific climate models as inputs to a hydrological model. This in turn models the basin behaviour and provides the inflow discharges at the reservoir. These results can then be coupled with the modelling of the system of water resources that computes the allocation and use of the water based on the reservoir's exploitation rules. The uncertainties inherent in climate and hydrological projections should be incorporated in the analysis.

Figure 4: Over topping of Dam reservoir

12 GATE PERFORMANCE;

Spillways and outlet works play a fundamental role in ensuring dam safety. They are responsible for providing the necessary discharge capacity when required during a flood event. Consequently, it is essential to evaluate any factors that might increase the likelihood of failure in their regulating gates. A key requirement for the proper functioning of gates is maintaining them in good condition. Severe deficiencies or deterioration could render the outlet works or spillways ineffective.

An increase in the sediment content of water can exacerbate abrasion and erosion processes on the gates, their mechanical components, or the spillways, thereby compromising their reliability. Additionally, if the water contains larger amounts of suspended material, such as trees, branches, or debris, this could result in blockages of one or more gates, reducing their discharge capacity.

Temperature changes can also influence the proper operation of gates. Extreme heat or cold, as well as significant fluctuations in temperature, may subject gate mechanisms to additional stresses or deformations. Over time, this could lead to malfunctions or blockages in the gate.

Observing these impacts on the reliability of gates can be accomplished by conducting a qualitative evaluation of the gate system's condition. This assessment is crucial to ensuring the long- term functionality and safety of spillways and outlet works under varying environmental and operational conditions.

Figure 5: Debris stuck in dam gate

13 FLOOD ROUTING STRATEGY:

Proper reservoir operation is critical for maintaining safety levels, which are determined by the specific characteristics of the dam– reservoir system. Some reservoirs rely primarily on their storage capacity to absorb inflowing water volumes, while others depend on their ability to release peak inflows effectively. Flood routing helps to reduce the loads placed on the dam, and its efficiency depends heavily on the condition of outlet works and the implementation of gate operation rules.

Climate change may require significant adjustments to flood routing strategies. For instance, increased sediment transport caused by soil erosion can lead to accelerated sedimentation within the reservoir, which could impair operational efficiency and decrease routing capacity. This sediment accumulation poses a safety risk to dam infrastructure by reducing its ability to handle large inflows. Additionally, changes in rainfall patterns can alter the characteristics of flood hydrographs, potentially reducing the time available for response and adaptation during extreme weather events.

To address these challenges, it is essential to first identify the factors that influence operational rules. Once these drivers are understood, a detailed analysis of their susceptibility to climate change impacts should be conducted. Operational criteria must then be re-evaluated to address the most significant issues. Considering the inherent uncertainties in climate and hydrological projections, these analyses should focus on the most critical aspects to avoid inefficiencies. A well-structured and targeted approach will help ensure that flood routing strategies remain effective and adaptive under changing environmental conditions.

To adapt, drivers affecting operational rules must first be identified, followed by analyses of their climate change impacts. Operational criteria are then re-evaluated to address the most relevant issues. Given the uncertainties involved, analyses should focus on key aspects to ensure efficiency.

GLACIAL LAKE OUTBURST FLOOD (GOLF) MANAGEMENT:

In the current climate change scenario, the occurrence of Glacial Lake Outburst Floods (GLOFs) is increasing at a rapid rate. Mountainous rivers at high altitudes are suddenly inundated with an unprecedented volume of water, leading to disasters that affect both the projects constructed across these rivers and their downstream areas. Effective mitigation measures, monitoring systems, and early warning mechanisms are crucial to manage and reduce the impacts of GLOFs.

The primary mitigation strategy to reduce GLOF risks involves lowering the water volume in glacial lakes, thereby decreasing the peak surge discharge. In addition to these preventative measures near the lake area, it is equally important to protect infrastructure from the destructive forces of GLOF surges. Implementing robust monitoring and early warning systems is an essential aspect of GLOF management, as these can provide critical lead time to minimize damage and ensure the safety of downstream communities.

Factors contributing to the hazards / risks of moraine-dammed glacial lakes include:

(a) large lake volume; (b) narrow and high moraine dam; (c) stagnant glacier ice within the dam; and (d) limited freeboard between the lake level and the crest of the moraine ridge. Potential outburst flood triggers include avalanche displacement waves from calving glaciers, hanging glaciers, rock falls, settlement and/or piping within the dam, melting ice-core and catastrophic glacial drainage into the lake from sub-glacial or englacial channels or supraglacial lakes.

Figure 6: Picture of a glacier

REDUCING THE VOLUME OF WATER IN THE LAKE :

The volume of water in the lake may be reduced by the following methods of considerations:

- controlled breaching,
- construction of an outlet control structure,
- pumping or siphoning out the water from the lake, and
- making a tunnel through the moraine barrier or under an ice dam.

Controlled Breaching*:* Controlled breaching is carried out by blasting, excavation, or even by dropping bombs from an aircraft to create a outflow channel. Bogatyr Lake in Alatau in Kazakhstan is an example of success of GOLF management.

Construction of an outlet control structure: Rigid structures made out of stone, concrete, or steel may be constructed for permanent and precise control of outflow. But the construction and repairs of the required mitigation works at high elevations, in difficult terrain conditions and in glacial lake areas far from road points and not easily accessed, will cause logistic difficulties. Therefore, preference should be given to construction materials available locally such as boulders and stones.

Pumping or siphoning out the water from the lake: Pumping and syphoning out of the water from the lake is an immediate mitigative solution but it requires electricity for the pumping which may not be available In the high altitude like Hindu-Kush Himalayan region.

Making a tunnel through the moraine dam: Tunnelling through moraines or debris barriers, although risky and difficult because of the type of material blocking the lake, has been carried out in several countries. Tunnelling can only be carried out through competent rock beneath or beside a moraine dam. The costs of such a method are very high. The construction of tunnels would pose difficulties in the Himalayas due to the high cost of transporting construction materials and equipment to high elevations.

PREVENTATIVE MEASURES AROUND THE LAKE AREA

Any existing and potential source of a larger snow and ice avalanche, slide, or rockfall around the lake area which has a direct impact on the lake and dam has to be studied in detail. Preventative measures against the instabilities of the moraine dam and the surrounding area, such as removing masses of loose rocks to ensure there will be no avalanches into the lake, will reduce to some extent the danger of GLOF.

PROTECTING INFRASTRUCTURE AGAINST THE DESTRUCTIVE FORCES OF THE SURGE

The sudden hydrostatic and dynamic forces generated by a rapid moving shock wave can be difficult to accommodate by conventionally designed river structures such as diversion weirs, intakes, ridges, settlements on the river banks, and so on. It will be necessary to build bridges with appropriate flow capacities and spans at elevations higher than those expected under GLOF events. Slopes with potential or old landslides and scree slopes on the banks of the river near settlements should be stabilised. It is essential that appropriate warning devices for GLOF events be developed in such areas.

MONITORING AND EARLY WARNING SYSTEMS

A programme of monitoring GLOFs throughout the country should be implemented using a multi-stage approach, multi-temporal data sets, and multi-disciplinary professionals. Focus should first be on the known potentially dangerous lakes and the river systems on which infrastructure is developed. Monitoring, mitigation, and early warning system programmes could involve several phases. The inventory of glaciers and glacial lakes using geographic information systems (GIS), and the remote sensing techniques and identification of potentially dangerous lakes shall be carried out properly.

In summary, the increasing frequency of GLOFs due to climate change necessitates a comprehensive approach that combines mitigation strategies, robust infrastructure protection, and advanced monitoring and early warning systems. Addressing both the causes and triggers of such events is essential for minimizing their impact on human life, infrastructure, and the environment.

National Disaster Management Authorities, Ministry of home affairs, Govt of India in association with **Swiss Agency for Development and Cooperation SDC** has prepared Guidelines for management of GOLFs. It has provided the mitigations measures to be adapted for management of GOLF, creation of awareness for short term, medium term and long term in the community, awareness drive for the specific target group including communities residing in on the downstream areas; vulnerable groups including women, children and senior citizens; urban planners and architects, geologists and civil engineers should be undertaken. Administrative integration among the government departments, public sector agencies, NGOs and civil bodies should be given special attention in order to integrate activities related to creation of awareness and preparedness. Education and capacity building at the university level, training of professionals involved in the assessment and management of GLOF risks, and capacity building within local, potentially affected communities is addressed in the guidelines.

OPTIMIZATION OF RESERVOIR OPERATION AND INTEGRATED RESERVOIR MANAGEMENT

For optimisation of reservoir systems, procedures based on coupling models with numerical search methods have been developed. Traditionally, the simulation-optimisation problem has been solved using mathematical techniques such as linear or non-linear programming. Application of these methods, however, puts severe restrictions on the formulation of optimisation problem with respect to description of water flow in the system and definition of control variables to be optimised and associated optimisation objectives. Recently, procedures that directly couples simulation models with heuristic optimisation procedures such as evolutionary algorithms have been proposed. These methods have proven to be effective for optimisation of reservoir systems.

There are three types of optimisations.

- **Short-term Optimization:** Aimed at providing an improved reservoir operation guidance during floods to assist in decision making when an inflow forecast is available from RTSF&ROS
- **Long term Optimization:** Developed for round the year water allocation for irrigation and water supply considering power development
- **Seasonal Optimization:** To minimize downstream flooding while considering the need of keeping the reservoirs full at the end of the rainy season

The main purposes of optimisation are: never exceeding maximum water level, compliance with the rule curve, avoid excessive spilling and mitigate downstream flooding. Based on a standard forecast and already scheduled releases the operator decides if an optimization is required. The optimization results can then be fine-tuned by the operator in an iterative process if it is judged that modifications will improve the operation. The optimization can also be bypassed and user defined scenarios can be used to decide on the releases.

Development of Comprehensive River Model: Flow simulation across the entire basin, with optimized releases for irrigation, is essential to ensure water supply and maintain hydropower demands in wet years. A combined optimization simulation exercise will prove invaluable for analyzing the flow situation throughout the basin at any point during the operation of the system.

If all the aforementioned measures—such as risk model components in dam safety, flood risk management, GLOF management, optimization of reservoir operation, and integrated reservoir management—are adapted with due care and precision, the risk to dam safety due to climate change can be mitigated considerably.

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MEASURES AND COMPLIANCE OF DAM SAFETY ASPECTS IN NHPC

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ABSTRACT:

Assessment of Dam Health is an important aspect for the safety of dams and to ensure continued accrual of benefits of sustainable development as well as protecting the downstream from any potential hazard and establishing public confidence. For safe operation, maintenance, repair & renovation of dams rest with dam owners. With aging of dams, the probable risk of dam failure incidents increases which needs to be addressed effectively.

Advancements in technology, survey & investigation methods, improved design, construction, operation & maintenance and rehabilitation with adoption of new materials, the incidents related to dam failures has reduced significantly. However, challenges such as flash floods (cloud burst & Glacial Lake Outburst Flood - GLOF), climate change and global warming presents new risks which needs to be addressed in a timely and positive manner on dam safety issues.

NHPC, for safe operation & maintenance and to assess the health of its dams, conduct dam safety inspections (through scheduled pre-monsoon and post-monsoon inspection, etc.) and comprehensive dam safety evaluations of dams by independent panel of experts in compliance to Dam Safety Act' 2021 and subsequent regulations.

1. INTRODUCTION

Hydroelectric projects such as dams are national assets constructed for overall economic development involving large investments and resources. Water and energy security are to be substantiated with the construction, operation and maintenance of safe dams for sustainable development. The health of the dams is an important aspect which needs to be examined for continued accrual of benefits as well as to protect the downstream reaches from any potential hazard and establishing public confidence.

2. INSTITUTIONAL FRAMEWORK IN INDIA

Government of India has enacted Dam Safety Act' 2021, which will help dam owners to adopt uniform safety procedures to ensure dam safety who own, control, operate, or maintain a specified dam according to the regulations made by the Authority under this Act.

3. HIGHLIGHTS OF DAM SAFETY ACT, 2021

Dam Safety Act provides for surveillance, inspection, operation and maintenance of specified dams (height >15m or 10-15m with certain requirements) for prevention of dam failure related disasters and to provide for institutional mechanism to ensure their safe functioning and for matters connected therewith or incidental thereto.

The Act envisages institutional setup at centre as National Committee on Dam Safety (NCDS) for evolving dam safety policies and regulations and National Dam Safety Authority (NDSA) as a regulatory body to implement policies so framed. Subsequent to DSA'2021, 19 regulations are also published by NDSA. All State governments has also constituted State Committee on Dam Safety (SCDS) and State Dam Safety Organisations(SDSO) to discharge functions under this Act.

However, in certain cases the NDSA will act as the SDSO viz. (i) Is owned by one state but situated in another state, (ii) Extends over multiple states and (iii) Is owned by a central public sector undertaking (CPSU).

3.1 **Obligations of dam owners:**

- Owners of specified dams are required to provide a dam safety unit in each dam.
- This unit will inspect the dams:
  - (i) Before and after the monsoon season, and
  - (ii) During and after every earthquake, flood, or any other calamity or sign of distress.
- Dam owners will be required to prepare an emergency action plan, and carry out risk assessment studies for each dam at specified regular intervals.
- Dam owners will also be required to prepare a comprehensive dam safety evaluation of each dam, at regular intervals, through an independent panel of experts.

The evaluation will be mandatory in certain cases such as major modification of the original structure, or an extreme hydrological or seismic event.

4. IMPLEMENTATION OF SAFETY INSPECTION OF DAMS AS PER GUIDELINES

In NHPC, Dam Safety Inspections are being carried out regularly at all 20 existing power stations and remedial measures undertaken. Further nine dams are under various stages of active construction. Preparation and Implementation of Emergency Action Plan (EAP), and other statutory documents such as Data book, Reservoir operation manuals, SOP (for sudden downstream releases), Instrumentation manuals, etc. for all the Dams. However, for better health and reliable operation of these valuable assets, implementation of continued repair & rehabilitation of civil structures is essential.

The revised guidelines “Guidelines for Safety Inspection of Dams” Jan-2018 by CWC and latest NDSA Regulations address all aspects of dam safety inspection programs. This guideline also consider the preparatory steps to be taken before planning the inspections, selection of inspection team, collection of required document and report preparation including suggestive measures.

4.1. Objectives of Dam Safety Inspections

- Ascertain that the dam system is performing as expected,
- Identify deficiencies or areas that need monitoring or immediate repair,
- Assess the health of the dam and record any changes that have occurred due to aging,
- Collect information to make informed decisions about needed remedial measures, and
- Find out, if the dam is being operated and maintained properly

4.2 Comprehensive dam safety evaluation

a) **Purpose:**

Before Dam safety Act, 2021, Comprehensive safety reviews were carried out by a group of ‘Independent panel of experts’ called ‘Dam Safety Review Panel’ (DSRP) to assess the integrity of a dam based on current acceptable safety criteria (e.g. Engineering standards, dam safety guidelines) or risk management criteria by an independent panel of experts. DSRP team consist of various domains e.g., Dam Safety Specialist, Design Expert, Construction Supervision Expert, Geologist, Hydrologist, Hydro – Mechanical Expert and Instrumentation Expert.

In compliance to the above, NHPC has carried out DSRP of dams of Bairasiul, Chamera-I & II, Rangit, Dhauliganga, Dulhasti, Teesta-V and Barrages of Tanakpur & Uri-I power stations and recommended remedial measures were implemented.

Now, after enactment of Dam safety Act, 2021, CDSE has been carried out as per latest guidelines/ regulations and SOP for CDSE issued by NDSA. CDSE reports of five dams (namely Chamera-I, Sewa-II, Teesta-V, TLD-III barrage & TLD-IV dam) has been submitted to NDSA and CDSE of two dams i.e. Nimoo Bazgo & Chutak barrage are underway. Replies to the technical comments received from NDSA on CDSE reports of Teesta basin have been submitted and work is in progress. CDSE of remaining dams are planned to be completed within 5 years of enactment of the Act.

4.3 Scheduled inspections

Scheduled inspections (pre & post-monsoon) are performed to gather information on the current condition of the dam and its appurtenant works, to establish needed repairs and repair schedules, and to assess the safety & operational adequacy of the dam, to evaluate previous repairs.

Scheduled inspections includes (i) Review of past inspection reports, monitoring data, (ii) Visual inspection (iii) Preparation of a report or inspection brief, with relevant documentation and photographs.

In NHPC, the scheduled pre & post monsoon dam safety inspections of Civil & HM equipment's of Dam are carried out for all 20 power stations by dam safety units (comprising of officials of power station) and a team of officials from corporate office related to Civil, Hydro-mechanical, Geology & Hydrology domain. The suggestive remedial measures are implemented. Two types of safety inspection reports are prepared (i) dam safety inspection as per prescribed format in Inspection Regulations, 2024 by filling online at DHARMA (Dam Health And Rehabilitation Monitoring Application) portal for submission to NDSA & State DSO and (ii) Safety inspection report covering all the project components viz, dam, water conductor system, power house and other important structures for internal use. Some critical issues requiring attention are escalated to the top management. Further, monthly monitoring of actionable points raised during Dam safety inspection are done through online module & regular interaction with the power stations for their successfully implementation.

The scheduled dam safety inspections have helped to timely identify the major issues related to the health of dam and other components of the power stations which have been effectively restored by suggesting corrective measures along with repair methodology in view of available limited working period along with quality assurance and monitoring system.

5. INSTRUMENTATION

Instrumentation plays a pivotal role providing an understanding of the foundation and structural behavior both during construction and operation of dam. In NHPC, instrumentation is planned and installed with various instruments such as survey targets, piezometers, pendulums, water levels, weather stations, SMAs, joint meters, stress/strain meter, etc. for project components. Regular collection of data, data processing and continuous monitoring program is being adopted to understand the performance. Remedial measures, if required are suggested and carried out.

During dam safety inspections, the status of installed instruments, registers/readout observations and damages, if any are observed. Some of the instruments such as Strong Motion Acceleographs (SMA) installed at all the dams in NHPC power stations are continuously monitored at the control room in the dam area and a centralized control room at corporate office. In order, to achieve the objective of dam safety, implementation of Real Time monitoring of dam safety, hydro-mechanical equipment's and automatic weather station, Early Warning System (EWS) for inflow flood forecasting, communication & PA system, etc. installed at project components of all power stations are continuously monitored at the control room in the dam area and at centralized control room, corporate office.

6. DAM SAFETY IN NHPC LTD

- Pre & post monsoon dam safety inspections are regularly being carried out for all 20 specified dams. For effective monitoring of actionable points raised during Dam safety inspection, development of online module for monthly progress has been successfully implemented.
- Submission of Pre & Post monsoon (Twice in a year) dam safety inspection report as per prescribed format published by NDSA in "Inspection, Instrumentation, Seismic data, Risk Assessment and

Evaluation of Specified Dam Regulations,2024" as Schedule-I to NDSA & State DSO by filling online at DHARMA (Dam Health And Rehabilitation Monitoring Application) portal.

- Submission of Instrumentation report as per prescribed format to by NDSA & State DSO.
- Comprehensive Dam Safety Evaluation (within 5years of Dam Safety Act,2021) by independent panel of experts for NHPC Dams are carried out as per latest guidelines of NDSA.
- Rapid Risk Screening of all specified dams have been submitted to NDSA.
- Prepare various generic statuary documents i.e. Emergency Action Plan (EAP), SOP (for EWS) & SOP (for sudden d/s release) as per guidelines issued by relevant authorities like CWC, NDMA etc. and NDSA regulations. Updation of EAP including Dam break analysis is being taken up at all power stations.
- Implementation of Emergency Action Plan in consultation with DDMA (District Disaster Management Authority).
- Preparation of Data book, Instrumentation manuals & other statutory documents such as SOP (for sudden d/s release) as per guidelines published by CWC/NDSA.
- Preparation of Initial filling plan, EAP, etc. before initial filling of reservoir for inspection by SDSO.

	
Erosion in Spillway Glacis	*Restoration of Glacis with steel liner*
	
Erosion of D/s slope before repair	*Restoration of D/s slope after repair*

	
Repair with PP Rope & wiremesh gabions	*Protection of bunds with Tetrapod Nosing*

7. BENEFITS OF DAM SAFETY MEASURES

Implementation of various dam safety inspections and procedures has resulted in

- Enhanced upkeep of dams resulting in reduction in the cost & frequency of repairs,
- Maintaining live capacity of reservoirs for designated use,
- Increased generation as forced shut down period is minimized

The innovative idea of fixing steel liner in existing spillway crest & upper part of glacis with the countersunk bolts at Dhauliganga dam and use of High performance cementitious mortar conforming to EN 1504-3(R4) with high bond strength was done for the first time in India. Based on the satisfactory performance, these are used in many other projects of NHPC Ltd.

Preparation and issue of "Guideline for Repair & Rehabilitation of Civil structures of hydroelectric power station" covering Technical Specifications & BOQ for different type of generic repair materials for uniformity of repair works within the Dams under NHPC. These standardized repair materials & methodologies proved handy for implementation of repair & rehabilitation works of all power stations. Further these were extensively used for 'Renovation & Modernization, Uprating and Life Extension (RMU&LE) of hydro plants' of Bairasiul & Loktak power stations as per CERC norms after 35 years for extending the useful life by 25 years.

Erosion condition depends on various characteristics of the river. Based on the operational experience, erosion conditions can be categorized into three major categories i.e. mild, moderate and severe using three major characteristics i.e., hydraulic head over the component, annual sediment load, and maximum size of sediment/boulder as detailed in Table below.

Erosion condition	**Hydraulic head over component (m)**	**Annual sediment load (MCM)**	**Max size of sediment / boulder**	**Material for construction / repair of the component**
Mild	0-15	0-30	No Boulder rolling, only silt/sand	Standard Concrete (M25 to M30)
Moderate	10-30	0-30	No Boulder rolling, only sand/silt/gravel	High Performance Concrete (M65 to M80) on the glacis/bucket/stilling basin, Cementitious mortar (R4) on the piers
Severe	10-60	1-40	Boulder rolling, along with sand/silt/gravel	Steel Liners on the upper glacis/crest, HPC at the lower glacis, bucket/stilling basin, Steel liner/Cementitious mortar (R4) on the piers

Further, case studies of some important repair works taken up for dams/barrages and other components of the power stations are as follows:

7.1 Bairasiul Power Station

Bairasiul power station (3x60MW) is the first commissioned power station of NHPC and completing its 35 years of operation in 2017. Renovation, modernization and life extension works was scheduled from April 2017 to March 2021. During dam safety visits, the critical issues viz. lower hoisting capacity of desilting/diversion gate which results in reduced live capacity of reservoir, Higher head loss in water conductor system, wastage of precious inflow water of Bhaledh trench weir, repair of intake/DT bridge, requirement of protection wall at u/s of power house, Heavy damage observed at outlet of DT, repair of Siul weir has been observed. The most of the repairs require complete shutdown of power station.

During 2014, as part of Renovation & Modernization works for life extension (RMU & LE), Visual inspection followed by Non-destructive & semi-destructive tests such as (i) Rebound Hammer Test (RHT) - for surface hardness, (ii) Ultra Sonic Pulse Velocity test (UPVT)- for quality of concrete, (iii) Cover Meter Test (CMT)- to determine the cover to reinforcement, (iv) Core test- for compressive strength, (v) Thickness test for Penstock liner and Gates were carried out on various components. Predominantly E&M equipment were proposed to be replaced/refurbished. No major deterioration /weakness in major civil structures were observed and in general they were in good health. With minor repair / renovation, these structures can further be used for a sufficiently long period and accordingly RMU & LE proposal as per CERC norms (after 35 years for extending the useful life by 25 years) was submitted and cleared by CEA for execution.

Further in Sep 2017, an unprecedented flood has been observed which have caused major damage at downstream slope of the dam which further endangered the stability of dam. All these issues have earmarked during visits and inspite of their criticality, the project has meticulously operated the spillway gates and mandatory repair undertaken before renovation and major repairs shifted to complete shutdown period of power Station under renovation &modernization.

During complete shutdown the hoisting capacity of DT has increased from 65t to 185t, Diversion tunnel, HPC on glacis, construction of additional trench weir, enhancing capacity of feeder tunnel, HRT repair and other repairs has been made for extension of useful life of the power station by 25 years.

7.2 Tanakpur Power Station

Tanakpur Power Station was commissioned in 1992 and has attained 33 years of operation. In June-2013, the barrage experienced an unprecedented flood of 5.35 Lakh cusec. This caused large deposition of RBM in central portion of the reservoir and substantial increase in flow concentration of the river channel along the both afflux bunds, thereby scouring and damaging them. Innovative solutions of activation of central channel was adopted for diverting a part of river flow through it. Polypropylene (PP) rope gabions below water, mechanically woven double twisted hexagonal wire mesh gabions and small nose spurs of tetrapods were used for restoration and to protect afflux bunds on both banks. The frequency of damages has significantly reduced even at high velocities and this has helped to standardize the procedure of repair now.

In spillway glacis, a wearing layer of 500mm is provided over the structural base concrete with Epoxy grouted L-shaped steel anchors along with epoxy bonding. The performance of the stilling basin provided with HPC has been found satisfactory over the years. The repair of the warp wall d/s of barrage gate no.1 could be done by converting counterfort retaining wall into gravity wall in place of dismantling damaged counterfort retaining wall and also due to meticulous planning and execution.

Further the reservoir operation guideline was also updated to improvise sediment flushing and to minimize damages. In 2021 & 2024, about 5 & 4.7 lakh cusec flood respectively was managed without any major damage.

8. CONCLUSIONS

Effective implementation of dam safety management program of regular maintenance and operation coupled with elaborative monitoring mechanism of instrumentation, surveillance and dam safety inspections has resulted in

- Enhanced health & safety of dams and other project components,
- Reduction in the cost & frequency of repairs due to use of advanced repair materials, innovative methods & standardization of the same. Regular monitoring to evaluate the performance for the durability of the repair works,
- Maintaining of live capacity of reservoirs through optimized reservoir operation procedures and increased generation and
- Safety inspection report covering all the project components including dam, water conductor system, power house have helped to timely identify the major issues related to the health of dam and other components of the power stations and restoration with optimization of shutdown time.

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HIGH-PERFORMANCE CONCRETE FOR THE REPAIR OF SLUICE SPILLWAY OF NATHPA DAM

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KEYWORDS
Abrasion-erosion, High-performance concrete, High-performance FRC, Hydraulic structures, Abraded depth.

ABSTRACT
Sluice spillways in concrete gravity dams are highly susceptible to erosion and structural damage, especially during the monsoon season when high water discharge occurs. The flow carries large-sized boulders and sand particles, accelerating surface wear. These solid materials, transported at high velocity, collide with the concrete structure, causing severe hydro-abrasion and erosion. Over time, this continuous impact leads to surface degradation, material loss, and structural weakening. If left unaddressed, such damage can compromise the dam's efficiency and safety, highlighting the need for high-strength, abrasion-resistant concrete in hydraulic structures. The Nathpa Dam of SJVN's Nathpa Jhakri Hydro Power Station (1500 MW), a gravity dam located on the River Satluj, experiences severe erosion in its sluice spillways due to high-volume boulders—some as large as 5 meters in diameter—along with sand particles transported during peak discharge. This necessitates frequent repairs after the peak discharge season. To mitigate this, SJVN collaborated with the Indian Institute of Technology Delhi to explore the application of two types of high-strength concrete: High-Performance Concrete (HPC) and High-Performance Fiber-Reinforced Concrete (HPFRC), both of M80 grade. These mixes were prepared using 53-grade Ordinary Portland Cement, undensified silica fume, fly ash, and locally available aggregates, with a water-to-cement ratio of 0.28. Additionally, HPFRC incorporated micro-steel fibers (13 mm length, 0.3 mm diameter) at a 1.5% volume fraction. The hydro-abrasion resistance of both concretes will be assessed by measuring wear depth and residual strength after the peak discharge season. This study aims to reduce the annual repair costs of Nathpa Dam's sluice spillways and promote the broader adoption of HPC and HPFRC for hydraulic structures.

1.0 INTRODUCTION

The Nathpa Jhakri Hydroelectric Station, with a 1,500 MW capacity, is the largest hydropower plant in India. This run-of-the-river project is located on the River Sutlej, a major tributary of the Indus Basin, in Shimla district, Himachal Pradesh, North India. The Sutlej River originates from the Tibetan Plateau at an elevation of approximately 4,570 meters above sea level (masl) and eventually joins the Chenab River to form the Panjnad River, which later merges with the Indus River. The project is owned by SJVN Ltd, a Navratna Central Public Sector Enterprise (CPSE). The Nathpa Jhakri Hydroelectric Station comprises a 62.5 m-high concrete gravity dam, a large underground desilting complex, and a hydropower plant with an installed capacity of 1,500 MW. Water is diverted through an intake structure consisting of four intakes with a total capacity of 486 m³/s into the desilting complex, which contains four chambers, each measuring 525 m in length, 16.31 m in width, and 27.5 m in depth. The water is then conveyed via a 27.4 km-long headrace tunnel to an underground power station, where six 250 MW Francis turbines generate approximately 6,778 GWh annually. After power generation, the water is discharged back into the Sutlej River through a 982 m-long tailrace tunnel. Figure 1. shows the layout of project.

The Sutlej River, originating from the Tibetan Plateau and flowing through the Himalayas, carries a high volume of boulders and sediment, particularly during the monsoon season and glacial melt periods. Due to its steep gradient and high-energy flow, the river erodes its banks and bed, transporting massive boulders—some exceeding 5 meters in diameter—along with significant loads of sand, silt, and gravel. This intense sediment transport poses major challenges for hydropower projects, causing abrasion and erosion of hydraulic structures, particularly spillways. The abrasion-erosion damage caused by large

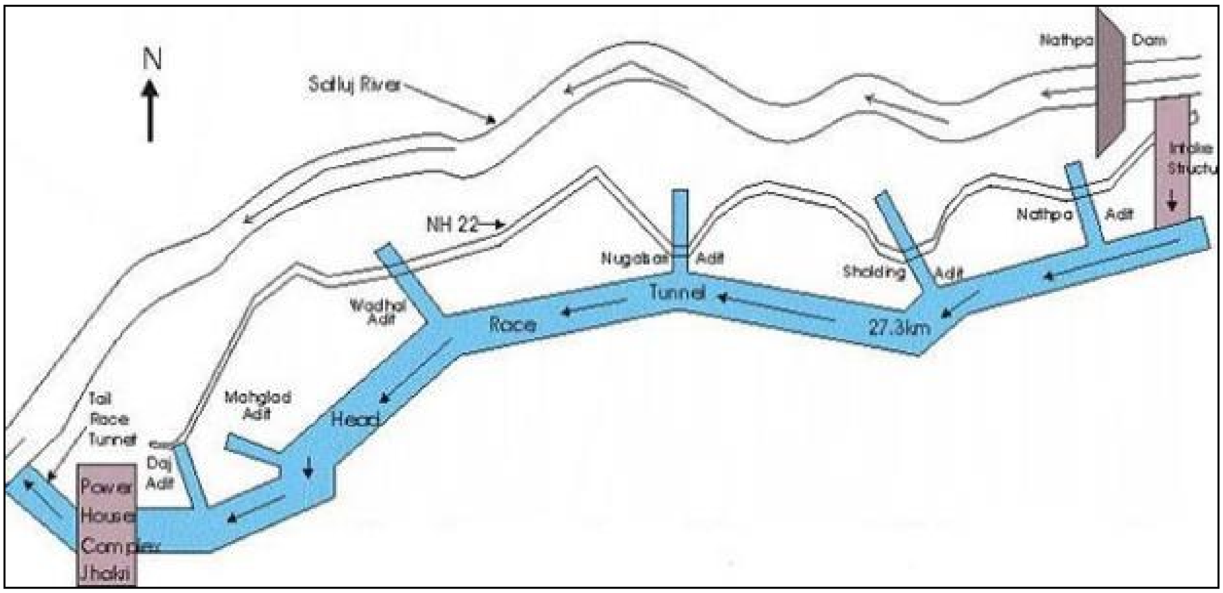


Figure1. Layout of Nathpa Jhakri Hydro Power Station

boulders and high-velocity water discharge has been a critical issue for Nathpa Dam (Figure 2). Such damage, particularly in the sluice spillway of Nathpa Dam, has led to significant structural wear, as illustrated in Figure-3. If left unaddressed, abrasion processes can severely reduce the service life and safety performance of concrete structures.

Figure 2. Downstream View of Nathpa Dam

Figure3. Abrasive damages occurred in Nathpa Dam sluice spillway.

Many previous studies demonstrated that abrasive resistance of concrete mostly depends on its compressive strength. For this reason, the high strength concretes with high resistance to abrasive action are sometimes used as for coating of hydraulic structures. High quantity of cement in the mixture of these concretes causes increase of hydration temperature and concrete shrinking. To reduce the heat of hydration, alternative supplementary cementitious binders such as fly ash, silica fume are incorporated in the cementitious mixes. The other significant factors influencing the abrasion behavior of concrete are aggregate type, fiber types, surface finish, and concrete curing. The various fibers such as steel, polypropylene and cellulose fibers inhibits the generation and development of micro-cracks in the concrete, and improves the concrete flexural strength to some extent. Moreover, the presence of fibers contributes in the early phase (2–6 h upon placing of concrete) by reducing both size and frequency of the cracks because they allow concrete to endure higher internal stresses. Also, addition of fibers to concrete improves the hydration of cement by reducing the separation of water from the fresh concrete. In a later period, in a somewhat more mature concrete, the fibers bind the potential cracks and reduce the risk of concrete destruction. In addition, some researchers reported that the abrasion resistance of concrete is closely related to toughness, and the abrasion resistance increases with increasing toughness. Therefore, researchers used crumb rubber as suitable alternate to the natural aggregates and findings showed the improved toughness of concrete. Considering the positive attributes of the fibers and different mineral admixtures as alternative binders in the cementitious mixes. This study aims to develop high-performance concrete (HPC) and high-performance FRC (HPFRC) of M80 grade each for the repair of sluice spillways. The developed HPC and HPFRC was then used to repair the damaged sluice spillways at Nathpa dam.

2.0 MATERIAL AND METHODS

Ordinary Portland Cement 53 (Ultratech) was used as per the specification of the Indian Standard code of practice [9]. Supplementary cementitious materials i.e., fly ash and undensified silica fume were used to prepare concrete mixes. The properties of these binders and aggregates are provided in the Table 1. The locally available aggregates were used to prepare the concrete mixes.

Table 1. Properties of binders and aggregates used in the study

Properties	**Cement**	**Fly ash**	**Silica fume**	**Aggregates**		
				Sand	**Coarse aggregate (20mm)**	**Coarse aggregate (10mm)**
Specific gravity	3.12	2.18	2.47	2.44	2.70	2.69
Water absorption	-	-	-	0.95	0.50	0.60

One other significant contributing to the development of HPC and HPFRC is the quality of coarse aggregates used therein. The test methods in accordance to IS: 2386 Part IV were adopted to assess the quality of coarse aggregates. These include aggregate crushing value, aggregate impact value and aggregate abrasion value. The values obtained from the test and permissible limits provided in the IS: 2386 Part IV are presented in Table 2. The aggregate chosen was amongst locally available and suitable as per codel requirement. However, the best sample passes in abrasion and impact value but failed marginally from crushing point of view. The impact and abrasion are significant in spillway concrete than crushing so this sample was used.

Table 2. Test results and permissible limits of the coarse aggregate

Parameter	**Coarse aggregates**	**Permissible limit for wearing surfaces**
Aggregate crushing value	23.62 %	Less than 30% For M65 grade of concrete or more, shall not exceed 22%
Aggregate impact value	20.15 %	Less than 30% For M65 grade of concrete or more, shall not exceed 22%
Aggregate abrasion value	27.70 %	Less than 30%

The investigation was carried out to achieve various concrete mixes for target strength of 80 MPa. Further, to improve the early age strength, silica fume in the range of 12.5 percent by weight of cement was incorporated in different concrete mixes. The dosage of silica fume is generally responsible for the high heat of hydration, leading to early shrinkage cracking. To counter act the high heat of hydration, the dosage of fly ash is preferred and added in the range of 25 to 30 percent by weight of cement. Further, to achieve higher strength and provide a bridging effect during failure, micro steel fibers of 0.2 mm (diameter) and 13 mm (length) were used in the present investigation, as shown in Figure 4. The micro-steel fibers were used in the optimized range of 1.5% by volume of concrete. The dosage of chemical admixture was kept in the range of 0.5 to 1.5% by volume of concrete to achieve the desired workability.

Figure 4. Steel fiber used in the present study.

The concrete mix proportions adopted in the present study and the nomenclature are tabulated in Table 4. 'M' represents the concrete mix and 'S' represents the steel fiber used in the concrete mix. The numerical number adjacent to M represents the target strength of the concrete mix.

Table 4. Mix proportions of prepared concrete mixes

Mix details	**Cement kg/m^3**	**Fly ash kg/m^3**	**Silica fume kg/m^3**	**Sand kg/m^3**	**20 mm Aggregate kg/m^3**	**10 mm Aggregate kg/m^3**	**Water kg/m^3**	**Admixture kg/m^3**	**Steel Fiber kg/m^3**
M80	545	164	68	615	526	526	178	8.3	-
M80S	500	125	62	615	526	526	140	5.3	118

3.0 DEVELOPMENT OF 'HPC' AND 'HPFRC' OF DESIRED STRENGTH

The desired mix was prepared in a rotating pan mixer of capacity 0.01 m^3. To obtain the mix, all the mix ingredients, viz. coarse aggregates, fine aggregates and cementitious binder were weighed in required proportions and mixed thoroughly for about two minutes to get a uniform mix in the dry state. After this, water was added to the dry mix in two equal proportions and a uniform mix was obtained by mixing for another three minutes. All the molds were greased with oil before pouring of concrete in the molds. Cubical specimens of size 150 mm were cast to determine the compressive strength of mixes. The specimens were placed in the temperature-controlled curing chamber for desired curing as shown in Figure 5.

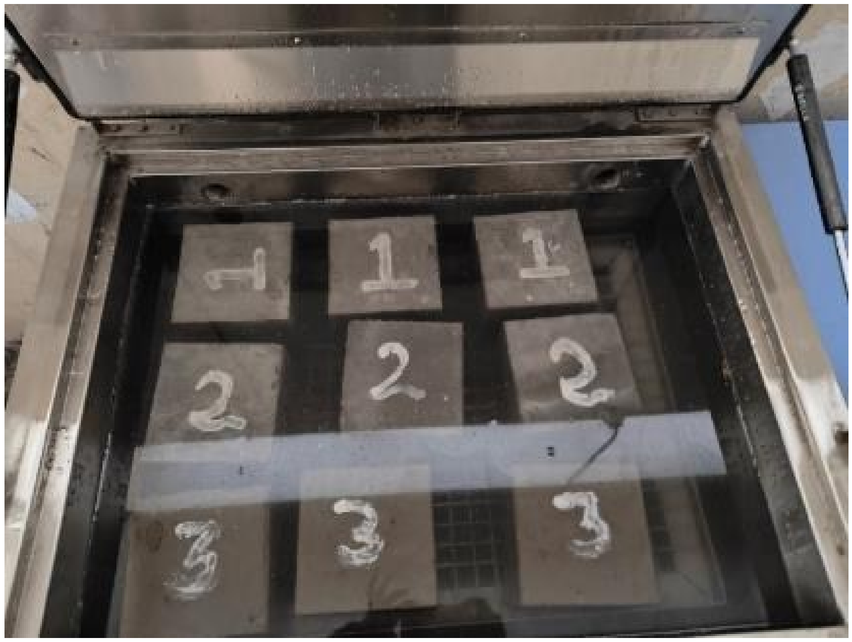


Figure 5. Specimens placed for curing in temperature-controlled chamber.

3.1 Compressive Strength

Compressive strength of the mixes was determined as per BIS 516-2004. The compressive strength of each mix was measured at 7 and 28 and days of casting. On the testing date, the cubical specimens were taken out of the curing tank and were kept in the laboratory environment for 30 mins. After that, the cube was placed under the platens of a compression testing machine of 2500 kN capacity. The position of cube was such that the load was applied perpendicular to the as-cast position. The compressive load was applied through the hydraulic jack at a constant rate of 140 kg/cm^2/min till failure as shown in Figure-6 . At each testing age, three specimens were tested and the average of three specimens was taken as the reference value. Compressive strength of the mix was calculated according to the following equation.

$$f_c = \frac{\mathrm{P}}{\mathrm{A}}$$

Where, f_c is compressive strength in MPa or N/mm^2, P is maximum load in N and A is cross sectional area of cube in mm^2.

Figure 6. Testing and failure of 150 mm cube specimen.

3.2 Results of Compressive Strength

The cured specimens at the age of 7 and 28 days were taken out from the temperature-controlled curing chamber. These specimens were surface dried and then subjected to compressive testing. The results of compressive test is shown in Figure-7. It can be inferred from the figure that the strength attained in the M80S at both testing age was higher in comparison to the M80 mixes. The addition of steel fibers in concrete provided bridging effect, thus reducing the crack propagation. However, both the concrete mixes achieved the target strength of M80 grade of concrete.

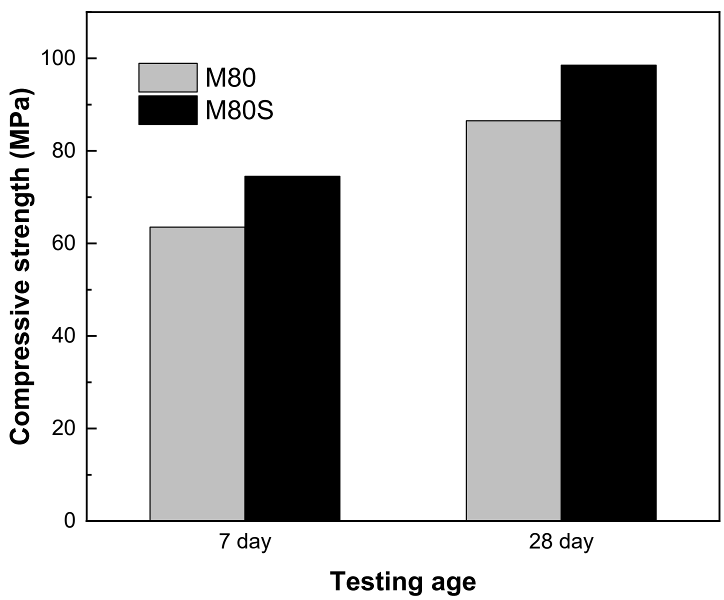


Figure 7. Testing results of the specimens subjected to compressive loading.

4.0 APPLICATION OF HPC AND HPFRC AS REPAIR MEASURE OF SLUICE SPILLWAYS IN DAMS

The developed HPC (M80) and HPFRC (M80S) was used to the repair the damaged sluice spillways of Nathpa Dam. Sluice spillways undergo high degree of abrasion erosion due to the high amount of water flow containing silt and boulders. The description and the complete procedure of the repair measure is elaborated below.

4.1 Description of the Dam

There are total of five sluice spillway and an overhead spillway in the dam to regulate the excess water flow as shown in Figure 8. The sluice spillway of the dam experiences huge amount of larger-sized boulders, even of 5-meter diameter and sand particles when high water discharge is released during monsoon season. These solid particles in the fast-flowing water extensively damage the sluice spillways as shown in Figure 8 Therefore, it necessitates to repair the sluice spillway regularly post-peak discharge season for smooth functioning.

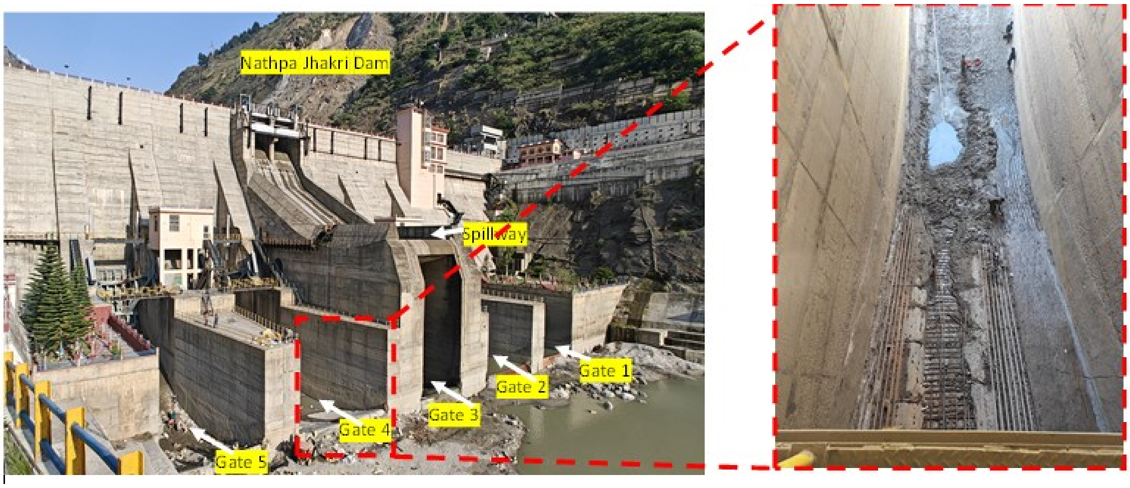


Figure 8. Location of sluice spillways and damaged spillway at Gate No. 4.

4.2 Application of HPC and HPFRC

The repair of damaged sluice spillways in Radial Gate No. 3, 4 and 5 was carried out using HPC and HPFRC as developed in the previous section. Initially the repair method for the sluice spillway of Radial Gate No. 3 is discussed. The abraded surface of all the sluice spillways were thoroughly cleaned prior to the repairing using high-pressure air gun. The new reinforcement in both longitudinal and transverse was placed as per the requirement of site. The new reinforcement was tied or welded with the existing/old reinforcement of the sluice spillway. The repair plan adopted for the gate 3 spillway is shown in the Figure 9.

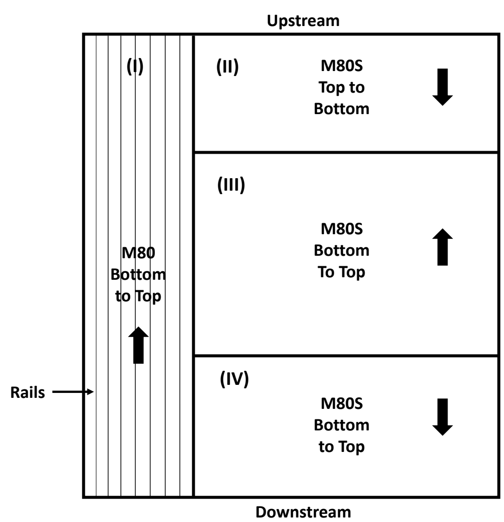


Figure 9. Direction of casting of concrete at sluice spillway at Radial Gate No. 3.

Before placing the new concrete, structural bonding agent was applied to the surface of old concrete. Then the Radial Gate No. 3 sluice spillways was repaired using the combination of HPC and HPFRC i.e. M80 and M80S. The M80S grade concrete was applied in three sections as shown in Figure 6 following top to bottom for section (II) and vice versa for section (III) and (IV). This was chosen as per the site requirement and inclined profile of the sluice spillway. M80 grade concrete was used in section (I) between the rails. The repaired sluice spillway at the Radial Gate No. 3 is shown in the Figure-10.

Figure 10. Repaired sluice spillway at Radial Gate No. 3.

The repair process adopted for the sluice spillway of Radial Gate No. 4 is discussed now in the following section. The surface was thoroughly cleaned and new longitudinal and transverse was placed as shown in the Figure11. In this spillway, major damage was observed in the flip bucket of nearly 1m depth due to higher impact of boulder. The longitudinal reinforcement was embedded and crossed inside the flip bucket for better connection and bond with the parent concrete. The flip bucket was filled with M80 grade of concrete till the level of old parent concrete. Moreover, cavities adjacent to the existing reinforced were repaired using the epoxy grouting for better bond. The overall repair was done using M80 grade of concrete in sluice spillway Radial Gate No. 4.

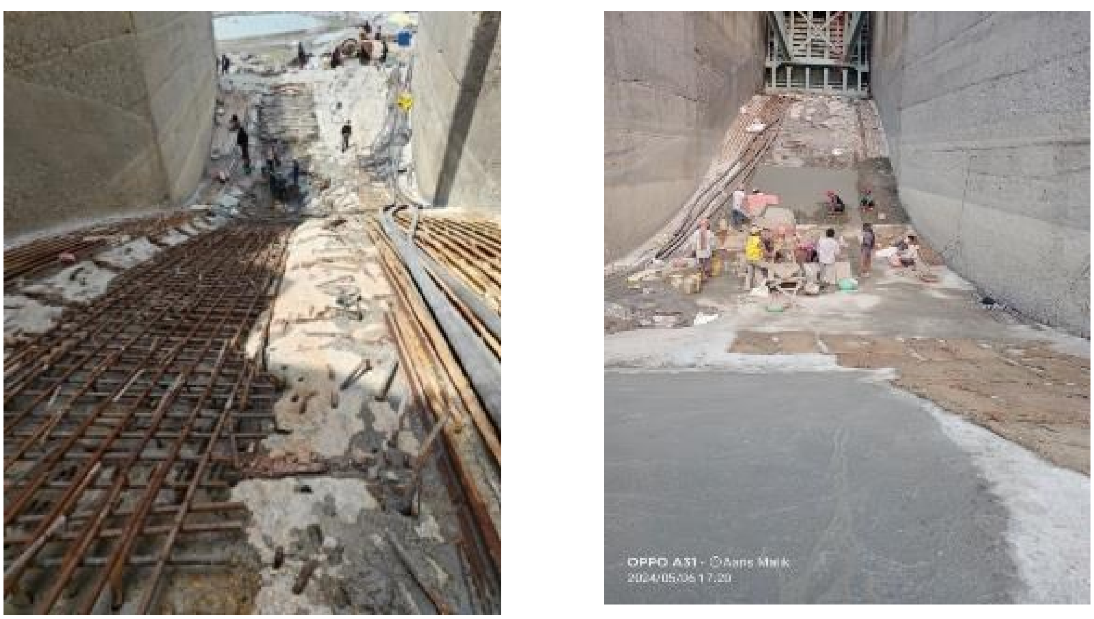
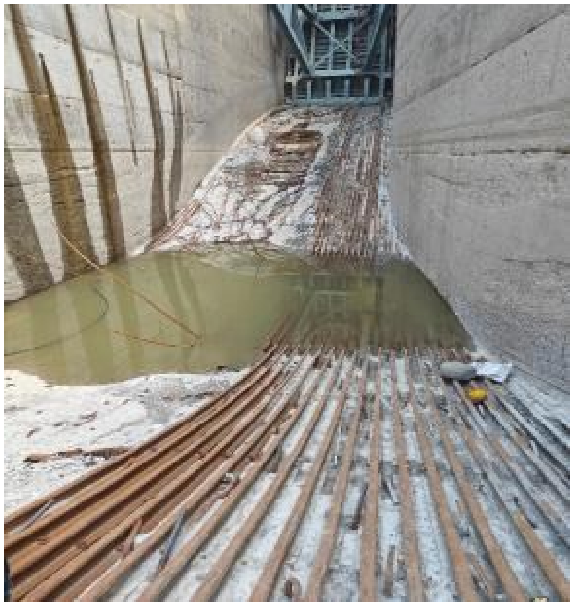


Figure 11. Surface preparation and laying of reinforcement of sluice spillway at Radial Gate No. 4.

The sluice spillway at Radial Gate No. 5 was repaired using the combination of M80 and M80S and rails were mounted over the top layer to reduce the abrasion and impact loss due to the fast-moving boulders. The condition of sluice spillway at Radial Gate No. 5 prior to the repair is shown in Figure 12. It was found that the existing rails on the right side of Radial Gate was intact. To further safeguard the existing rail connections/bolts, sacrificial steel rods were welded towards upstream side to protect the existing bolts and anchors. After this, surface under the existing rails were thoroughly cleaned using the hydro-demolition and bonding coat was applied on the existing concrete.



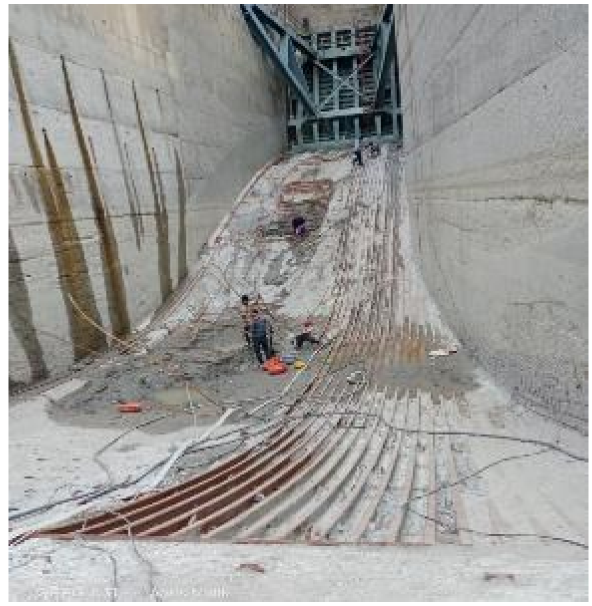
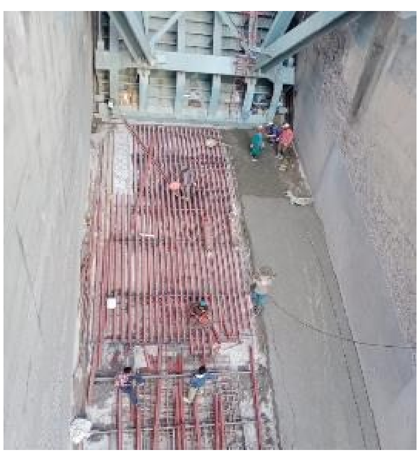


Figure 12. Sluice spillway at Radial Gate No. 5.

After this M80S concrete was placed in between the existing rails. The longitudinal and transverse reinforcement was placed as per the drawing and the new rails as visible on the left side as shown in Figure 13 were placed and these rails channels were welded with the existing anchors connected to the parent reinforcement. The M80 grade concrete was placed in this section of new rails till the flip bucket of sluice spillway of Radial Gate No. 5. The downstream portion adjacent to the flip bucket was completely repaired using M80S grade of concrete.



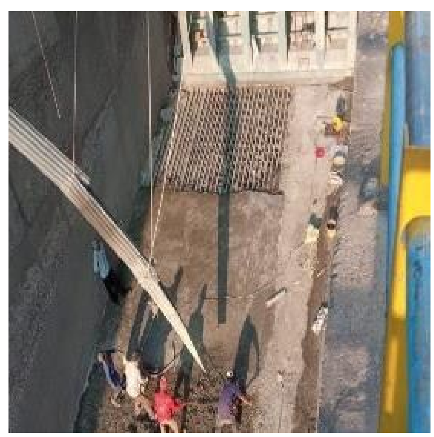


Figure 13. Placement of M80S grade concrete on the right side of the existing rails.

The contour profile of the repaired sluice spillways at Radial Gate No. 3, 4 and 5 was done at the end of completion work. The performance of the HPC and HPFRC for abrasion erosion attributes will be evaluated post peak season. The depth of erosion, residual strength and visual examination will be examined to study the abrasion erosion properties of the HPC and HPFRC. The concrete core will be extracted using core cutter from the certain sections to investigate durability properties in laboratory.

5.0 CONCLUSIONS

HPC and HPFRC were designed and implemented to mitigate the high abrasion erosion of the sluice spillway at Nathpa Dam. These materials were applied for repair works at different radial gates: HPFRC was used at Radial Gate No. 3, HPC at Radial Gate No. 4, and a combination of both at Radial Gate No. 5. This study represents the first application of HPC and HPFRC for repairing sluice spillways at Nathpa Dam. The preliminary finding of application of HPC on sluice spillway of Nathpa Dam shows significant improvement to abrasion & erosion resistance. However, the performance of the HPC and HPFRC for abrasion erosion attributes will be evaluated in coming months. The findings contribute to the development of an effective repair methodology using abrasion-resistant concrete. Additionally, the results will help in scaling up the use of HPC and HPFRC for other hydraulic concrete structures, improving their durability and long-term performance.

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Proceeding of International Conference on Dam Safety-2025
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Importance of Dam Safety in India and Dam Safety Act 2021

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Keywords Dam, Dam Failures, Dam Safety Act

ABSTRACT

Dams construction is an old art practiced by man since thousands of years. Multi-purpose water reservoirs and dams play a major role for water supply, irrigation, flood protection and hydropower generation in India. In order to ensure long-term operation and safety of the dams, adaptation planning, maintenance, and rehabilitation actions are needed.

Despite the notable benefits that dams bring to society, they involve immediate and potential risks and impacts, which makes dam safety extremely important. Dam Safety Act 2021 is enacted to provide for surveillance, inspection, operation and maintenance of the specified dam for prevention of dam failure related disasters and to provide for institutional mechanism to ensure their safe functioning. Dam Safety Act define dam and dam failure as under:

"dam" means any artificial barrier and its appurtenant structure constructed across rivers or tributaries thereof with a view to impound or divert water which also include barrage, weir and similar water impounding structures but does not include— (a) canal, aquaduct, navigation channel and similar water conveyance structures; (b) flood embankment, dike, guide bund and similar flow regulation structures

"dam failure" means any failure of the structure or operation of a dam which leads to uncontrolled flow of impounded water resulting in downstream flooding, affecting the life and property of the people and the environment including flora, fauna and riverine ecology.

Through its history, dams show great modernizations, but instances of dam failure, however, designate breaches in this evolution of knowledge that could have averted them. Human mistakes have undermined safety and leads to failures. Safety hazards also were exasperated by increasing population and land use in the downstream areas of dams and by failing to do necessary inspection and maintenance or upgrading works. More emphasis over dam safety measures is needed now in our existing dams and in their future development of dams if they are to continue delivering their benefit without causing harm to human communities.

1.0 INTRODUCTION

Dam can be defined as "Dam" means any artificial barrier and its appurtenant structure constructed across rivers or tributaries thereof with a view to impound or divert water which also include barrage, weir and similar water impounding structures but does not include— (a) canal, aquaduct, navigation channel and similar water conveyance structures; (b) flood embankment, dike, guide bund and similar flow regulation structures

India has 5334 large dams; ranks 3rd in the world in the number of dams, after China and USA. This number will be rising in the coming years as India constructs more dams to meet the rising demands by growing population as India overtakes China to be most populous country in the world.

Dam are being constructed after selecting the types of dam for a particular site and can be classified commonly as under: -

- Earth dam: Earthen dam utilizes natural materials with a minimum of processing. In India most of the dams are earthen dam.

- Gravity dam: A gravity dam is a dam constructed from concrete or stone masonry and designed to hold back water by primarily utilizing the weight of the material. Gravity dams provide some advantages over embankment dams.
- Composite dam: It is an earthen dam which is provided with a stone masonry or concrete overflow (spillway) section.

As per ICOLD, Earth dams predominate for some 67 % of all reported dams. This is of course the oldest type and there are traces or earth dams in the remains of the most ancient civilizations. Furthermore, this type of dam can accommodate a wide range of different foundations. The world's second highest dam is Nurek dam in Tajikistan (300 m high). Across world percentage of various types of dams are given as under:-

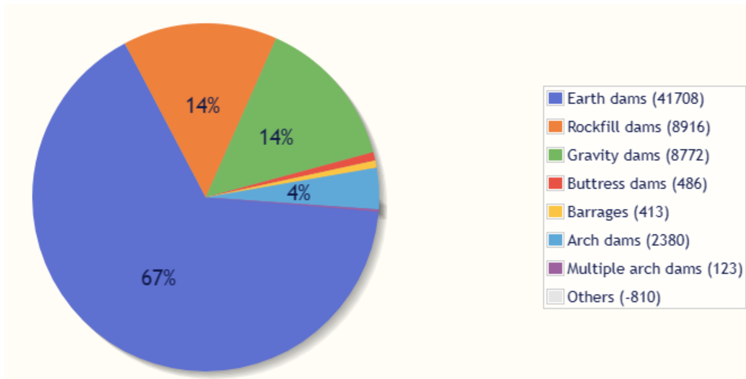

Figure 1. Distribution of various types of Dams across world (Source: ICOLD)

Dams and Multi-purpose water reservoirs play a major role for water supply, irrigation, flood protection and hydropower generation in India. Human mistakes have undermined safety and leads to failures. Safety hazards also were incensed by increasing population and land use in the downstream areas of dams and by failing to do necessary inspection and maintenance or upgrading works. More emphasis over dam safety measures is needed now in our existing dams and in their future development of dams if they are to continue delivering their benefit without causing harm to human communities.

2.0 IMPORTANCE OF DAMS

For almost 5000 years, dams have obliged the mankind to ensure an adequate supply of water by storing water in times of surplus and releasing it in times of scarcity, thus making a significant contribution to the efficient management of limited water resources that are unevenly distributed and subject to large seasonal variations. Hence, the construction of dams always been considered as a basic requirement to meet with the human needs for water and water related services.

Most of the dams are single-purpose dams, but now more multipurpose dams are being constructed. Using the most recent publication of the World Register of Dams, irrigation is by far the most common purpose of dams. Among the single purpose dams, 48 % are for irrigation, 17% for hydropower (production of electricity), 13% for water supply, 10% for flood control, 5% for recreation and less than 1% for navigation and fish farming. (ICLOD)

As it is evident from above date that most of the dam are being constructed for irrigation and this was also reinforced with the information that after independence, India also tread the same path. It is evident from the example of Bhakra Dam (Figure. 2), as after independence most of the irrigated area went to Pakistan and results in shortage of food grain. Even the wars of 1962 and 1965 with neighboring countries, failure of monsoon in 1965 and 1966 forced India to heavily import food grains. But the reliable irrigation provided by Bhakra Dam project led to massive gains in production of food grains in the Indian states of Punjab and Haryana as well as increase in productivity which was much higher than the country as a whole. It ushers the era of Green Revolution in the country at that time. It is a multipurpose project and providing power also, First Prime Minister of India rightly said them as “Temples of Modern India”.

Figure 2. Bhakra Dam

3.0 DAM FAILURES

"Dam failure" means any failure of the structure or operation of a dam which leads to uncontrolled flow of impounded water resulting in downstream flooding, affecting the life and property of the people and the environment including flora, fauna and riverine ecology.
As per ICOLD, Dam failures can be occurred due to following reasons: -

1. Overtopping of a dam is often a precursor of dam failure. Overtopping can be due to Inadequate spillway design, debris blockage of spillways, or settlement of the dam crest.
2. Foundation defects, including settlement and slope instability, are another cause of dam failure.
3. Piping, that is internal erosion caused by seepage, is the third main cause. Seepage often occurs around hydraulic structures, such as pipes and spillways; through animal burrows; around roots of woody vegetation; and through cracks in dams, dam appurtenances, and dam foundations.

The other causes of dam failures include structural failure of the materials used in dam construction and inadequate maintenance. Aspects such as climate change, ageing of the existing dams and high population growth may bring our dams at higher risks in future.

About 200 notable failures of large dams in the world (ICOLD 1973) has been reported. In India many of our dams have deficiencies in structure, operation and monitoring on account of ageing and deferred maintenance. The first failure in India was in Madhya Pradesh in 1917 when the Tigra Dam failed due to overtopping. The worst dam disaster was the failure of Machu dam (Gujarat) in 1979 in which about 15,000 people died. There are more than 40 reported failures cases so far in India.

Recently reported dam failure in India is "Chungthang dam" (also called Teesta III), Figure- 3, has been destroyed as a result of torrential rains and a glacial lake outburst flood (GLOF) upstream on the Teesta River in the state of Sikkim, in the early hours of October 4, 2023. More than 30 people are confirmed to have been killed by the resulting floods. The flood waters washed away several bridges and sections of the main National Highway-10, which cut off access for multiple villages.

Figure 3. Chungthang dam (also called Teesta III) in Sikkim, India

4.0 NEED OF DAM SAFETY

However, globally about 2.2% of dams build before 1950 have failed mainly due to flooding, inadequate spillway capacity, bad workmanship etc. In India, about 80% of the dams – i.e. about 3700 – are over 25 years old, out of which about 209 dams are more than 100 years old. Many dams have varied structural deficiencies and shortcomings in operation and monitoring facilities. Most of the States have been failing to provide sufficient budgets for maintenance and repair of the dam. Many States also lack the institutional and technical capacities for addressing dam safety issues. With the increasing number of dams becoming older and older, dam failures are more expected now. Ensuring Dam Safety is essential for safeguarding huge investments in infrastructure. It is also crucial for safeguarding human life, and properties of the people living downstream of the dams.

5.0 DAM SAFETY FRAMEWORK IN INDIA

Now a days, world-wide, dam safety is considered an inherent function in the planning, design, construction, maintenance and operation of dams. The safety of the dams in India is the principal concern of the State agencies and other organizations that own the dams and are involved in various aspects of their investigations, planning, design, construction operation and maintenance.

i. **National Committee on Dam Safety (NCDS)**
Constituted by Govt. of India in 1987. Chaired by Chairman, CWC and is represented by all the States having significant number of large dams and other dam owning organizations. Suggest ways to bring dam safety activities in line with the latest state-of-art consistent with the Indian conditions. Acts as a forum for exchange of views on techniques adopted for remedial measures to relieve distress in old dams.

ii. Central Dam Safety Organization (CDSO)
Central Dam Safety Organization was established in CWC, in 1979. The objective of Central DSO was to Assist in identifying causes of potential distress; Perform a coordinative and advisory role for the State Governments; Lay down guidelines, compile technical literature, organize trainings, etc.; and create awareness in the states about dam safety.

iii. State Dam Safety Organizations (SDSO)-
DSO/Cell established in 18 States and 5 dam owning organizations.

iv. **Routine Periodic Inspection: -**
Done by trained and experienced engineers from DSO At least twice a year: pre-monsoon and post-monsoon. Examination of general health of the dam and appurtenant works. Preparedness of dam and hydro mechanical structures for handling expected floods.

v. **Comprehensive Dam Safety Evaluation**
More comprehensive examination by multi-disciplinary team for holistic view has been carried out once in a 10 year. May order additional field and laboratory investigations as well as numerical simulations, if warrant.

Government Initiatives: -

i. **Dam Rehabilitation and Improvement Project (DRIP)**
Ministry of Water Resources, River Development & Ganga Rejuvenation through Central Water Commission, in 2012, launched the six-year Dam Rehabilitation and Improvement Project (DRIP) with World Bank with an objective to improve safety and operational performance of selected dams, along with institutional strengthening with system wide management approach. Presently 198 dam projects are being rehabilitated under the DRIP Project.

ii. **Dam Safety Bill**
Union Cabinet in June 2018 approved proposal for enactment of Dam Safety Bill, 2018. The Bill provides for the surveillance, inspection, operation, and maintenance of specified dams across the country. The Bill also provides for the institutional mechanism to ensure the safety of such dams.

iii. **Dam Health and Rehabilitation Monitoring Application (DHARMA)**
It is web-based software package to support the effective collection and management of Dam Safety data in respect of all large dams of India. The software is designed for users at Central, State and Dam level, with user permission rights governed by their respective licenses.

iv. **Seismic Hazard Mapping along with development of Seismic Hazard Assessment Information System (SHAISYS)**
It is also web based interactive application tool being developed in CWC under Dam Safety Organization (DSO) to estimate the seismic hazard at any point in Indian region. The SHAISYS shall be capable of estimating seismic hazard using the deterministic as well as probabilistic approach.

v. **Other Initiatives** – Other important activities include Design Flood Review, publication of important Guidelines as well as Manuals dealing with Dam Safety Management, preparation of O&M Manuals, Emergency Action Plans.

6.0 STATUS BEFORE DAM SAFETY ACT 2021:

Before Dam Safety Act 2021, legal framework does not have any provision for penalizing the owner in case of a dam failure causing a disaster in the upstream or downstream of the dam. Lack of systematic assessment and monitoring coupled with inadequate resources is the primary cause of poor maintenance of dams and appurtenant works. Real time inflow forecasting systems are not in place even in important reservoirs. Such systems can add to dam safety measures besides improving operational efficiencies.
The procedure requires that revision study of dam hydrology needs to be completed much in advance of any rehabilitation exercise but this not being the case has led to delays in DRIP implementation. Dam design drawings or drawings as constructed are not available with project authorities in many cases. Dam Safety Organizations (DSO) in states is short of adequate man power and need to be strengthened. Siltation of reservoir is a serious issue, though in most cases the extent of siltation continues to remain unknown. De-siltation of reservoir is difficult in many cases owing to environmental concerns related to sediment disposal.
Appropriate Interventions for Sediment Management is not available in most cases. In few cases river sluices are available in dams, but they have not been operated for long periods, and are no more functional.

7.0 DAM SAFETY ACT, 2021

Safety of dams is very important for safeguarding the huge investment in critical physical infrastructure and the benefits derived from the projects. Dams have substantial investments in terms of social and environmental costs.
Many of the dams have inter-state ramifications in case of actual failure or perceived threats of failures. Advisory role of CWC and NCDS without any legal backing. The provisions of the proposed Dam Safety Act were to empower the dam safety institutional set-ups in both the Centre and the States.

7.1 Dam Safety Legislation Background

Standing Committee constitution in 1982 to evolve unified dam safety procedure in India. Report has been submitted in 1986. It recommended legislation on dam safety. Draft "Dam Safety Bill' drafted in 1987 and circulated among NCDS members by the committee. Comments were received from 12-member states.
Based on these suggestions the Bill was comprehensively revised and modified draft prepared in 2002. This draft was circulated among concerned Principle Secretaries of the states. Earlier efforts were to encourage the state govt. to enact the legislation on similar lines. The state of Bihar enacted the legislation in 2006. Some states like Andhra Pradesh and West Bengal favored uniform central legislation. Draft Dam Safety Bill -2010 was introduced in Loksabha in 2010 under Article 252 of the Constitution. The Dam Safety Bill 2010 sought to enjoin responsibility on the Central Government, State Governments and owners of specified dams. It defined the duties and functions of these institutions in relation to perpetual surveillance, routine inspections, operation & maintenance and funds for maintenance and repairs.
The Bill addressed the issues of emergency action plan and disaster management, and also enlisted the requirements of comprehensive dam safety evaluation.
The Bill referred to Parliamentary Standing Committee (PSC), which gave its report in 2011, suggesting major modifications. The bill was withdrawn and Ministry decided to introduce a new Bill. Water is mainly a state subject but it finds mention in Entry 56 of the Union List I also which covers "Regulation and development of inter-State rivers and river valleys to the extent to which such regulation and development under the control of the Union is declared by Parliament by law to be expedient in the public interest".
The term of the 15th Lok Sabha came to an end, and the Dam Safety Bill, 2010 lapsed with the dissolution of 15th Lok Sabha. The Dam Safety Bill (DSB), 2018 was prepared for whole of India incorporating the recommendations of the Parliamentary Standing Committee on the Dam Safety Bill, 2010. With dissolution of the Sixteenth Lok Sabha, the Dam Safety Bill, 2018 also lapsed.
The DSB-2019 was finally passed by Lok Sabha in August 2019 and Rajya Sabha in 2021 as DSB-2021. The Dam Safety Act, 2021 has been notified through Gazette of India on Dec 14, 2021 and has come into force with effect from 30.12.2021.

7.2 Provisions of Dam Safety Act-2021:

The Act provides for surveillance, inspection, operation and maintenance of the specified dam for prevention of dam failure related disasters and to provide for institutional mechanism to ensure their safe functioning and for matters connected therewith or incidental thereto. It extends to whole of India for Dam constructed before or after the commencement of this Act.
The Act provides for constitution of the *National Committee on Dam Safety* with Chairman, Central Water Commission as Chairperson, to discharge functions for the purposes of maintaining standards of dam safety and prevention of dam failure related disasters, evolve dam safety policies and recommend necessary regulations as may be required; act as a forum for exchange of views on techniques to be adopted for remedial measures to relieve distress conditions in specified dams and appurtenant structures; analyze the causes of major dam incidents and dam failures and suggest changes in the planning, specifications, construction, operation and maintenance practices in order to avoid recurrence of such incidents and failures to name a few.
Act also has provision for establishment of the *National Dam Safety Authority*. The Authority shall be headed by an officer not below the rank of Additional Secretary to the Government of India or equivalent to be appointed by the Central Government who have knowledge of, and adequate qualification, experience and capacity in, dealing with problems relating to the dam engineering and dam safety management. Functions of the authority includes the purpose of maintaining standards of dam safety and prevention of dam failure related disasters, discharge such functions as related to implementation of the policies made by the

National Committee including making regulations on the recommendations of the National Committee; resolve any issue between the State Dam Safety Organizations of States or between a State Dam Safety Organization and any owner of a specified dam in that State; provide the state-of-the-art technical and managerial assistance to the State Dam Safety Organizations etc.
Further, *State Committee on Dam Safety* shall be established by State Government under the chairmanship of the Engineer-in-Chief or equivalent officer of the Department of the State responsible for Dam Safety as Chairperson. Few if the functions consist of maintaining standards of dam safety and prevention of dam failure related disasters, discharge such functions as may be necessary as per the guidelines, standards and other directions issued by the Authority; review the work done by the State Dam Safety Organization; establish priorities for investigations in case of specified dams under distress condition etc.
In addition to this, the State Government shall, for the purposes of this Act, establish in the Department dealing with dam safety, a separate organization, to be known as the *State Dam Safety Organization.* Provided that in States having more than thirty specified dams, the State Dam Safety Organization shall be headed by an officer not below the rank of Chief Engineer or equivalent, and in all other cases, the State Dam Safety Organization shall be headed by an officer not below the rank of Superintendent Engineer or equivalent. Every State Dam Safety Organization shall, keep perpetual surveillance; carry out inspections; and monitor the operation and maintenance. The State Dam Safety Organization shall classify each dam under their jurisdiction as per such vulnerability and hazard classification criteria as may be specified by the regulations.
Every owner of specified dam shall earmark sufficient and specific funds for maintenance and repairs of the dams; All new specified dams shall have a minimum number of set of instrumentations necessary to adequately monitor the performance and safety of dams. A well-designed hydro-meteorological network shall be established in the vicinity of every specified dam. A seismological station shall also be installed in case of dams higher than thirty meters or falling in the seismic zone III or above Dam Safety.
Every dam owner shall carry out risk assessment studies at such intervals as specified by the National Committee. Emergency Action Plan for disaster preparedness shall also be prepared for every specified dam and updated at regular intervals. Every owner, organization and authority shall render necessary cooperation to the authorities under the Disaster Management Act, 2005 to meet or mitigate any disaster or emergency arising out of the specified dams.

An obligation upon the National Dam Safety Authority to forward its Annual Report to the Parliament and the National Disaster Management Authority and the State Dam Safety Organization to forward their Annual Reports on safety status of dams to the concerned State Legislative and State Disaster Management Authority.

There are offences and penalties provisions under this Act. In case of offence, it shall be punishable with imprisonment for a term which may extend to one year or with fine, or with both, and if such obstruction or refusal to comply with directions results in loss of lives or imminent danger thereof, shall be punishable with imprisonment for a term which may extend to two years.

8.0 WAY FORWARD

India has invested substantially in infrastructure necessary to store surface runoff in reservoirs formed by large, medium, and small dams with associated appurtenances. It is very important to ensure that the existing dams continue to operate as designed, producing intended benefits at full potential to the society in the form of water supply, irrigation, flood control and hydropower.

So far, the dam safety directions and protocols issued by CWC are basically advisory kind of and have limitations during implementation. Enactment of the Dam Safety Bill-2021 resulting in notification of Dam Safety Act 2021 gave legal powers to dam safety agencies for implementing dam safety protocols and giving teeth to the implementing agencies for monitoring, implementing the dam safety. Safe Dams mean Safe nation.

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Jaswant Kapoor graduated in Civil Engineering from the National Institute of Technology, Hamirpur, India. He is currently, working as Head of Department, Civil Engineering Department, SJVN. He has experience of more than 27 years in the area of Hydro Power Development. He has been involved in investigation of Hydro Projects, detailed planning, design and preparation of feasibility studies/Detailed Project Report for a number of large and small hydro projects in India. He was involved in the construction of 1500 MW Nathpa Jhakri HEP, 412 MW Rampur HEP, 60 MW Naitwar Mori HEP in India and 900 MW Arun-3 HEP, 679 MW Lower Arun HEP and other projects in Nepal. He is also looking after India's biggest underground powerhouse project Etalin HEP 3097 MW in Arunachal Pradesh. He has written many papers for the National and International Conferences/Seminars/Workshops held in India and Abroad and attended some of these conferences in India and Abroad.

Abhishek, graduated in Civil Engineering from Panjab Engineering College, India. He also holds a Master Degree in Highway Engineering from the same institute. He has an experience of more than 19 years in hydro power sector while working in various departments such as Cost Engineering, Civil Design and project site of Arun-3 HEP, Nepal. He has been involved in investigation of Hydro Projects, detailed planning, design and preparation of feasibility studies/Detailed Project Report for a number of large and small hydro projects in India. He is currently working as Deputy General Manager in Civil Design Department looking after 3097 MW Etalin HEP in Arunachal Pradesh.

Priti Thakur, graduated in Civil Engineering from National Institute of Technology, Hamirpur, India. She has experience of 14 years in Hydro power sector. She has been involved in the Construction of Surface Power House of 412 MW Rampur Hydro Power Project (H.P.) as a Site engineer. As a Design Engineer, she has been involved in the detailed planning & design of various Civil components of Arun- 3 HEP (900 MW) & Naitwar Mori HEP (60 MW), preparation of Feasibility Reports/ Detailed Project Reports for various Survey and Investigation Projects in Chenab Basin of Himachal Pradesh and Dibang Basin of Arunachal Pradesh. Currently, she is working as Sr. Manager in Civil Design Department and involved in the planning and design of 05 Hydro Projects (allotted to SJVN Ltd.) in Dibang Basin of Arunachal Pradesh including 3097 MW Etalin Hydro Power Project.

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SAFETY INSPECTIONS OF NATHPA DAM

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KEYWORDS
Dam Safety, Post-Monsoon Inspection, Pre-Monsoon Inspection, Structural Integrity, Risk Mitigation.

ABSTRACT
Dam safety inspections are essential for maintaining the structural integrity and operational reliability of dams. Post-monsoon inspections assess the impact of extreme weather conditions, such as heavy rainfall, floods, and reservoir fluctuations, identifying potential vulnerabilities like erosion, slope instability, and structural distress. In contrast, pre-monsoon inspections focus on preventive maintenance by ensuring the functionality of spillways, gates, and other critical components to enhance the dam's resilience against upcoming monsoon stresses. Nathpa Dam, a key component of the Nathpa Jhakri Hydropower Station (NJHPS) in Himachal Pradesh, plays a crucial role in India's renewable energy sector, contributing 1,500 MW of power. Additionally, it also serves as the main dam for the downstream Rampur Hydropower Station (RHPS), a tailrace development of NJHPS, which generates 412 MW using the 383.88 cumecs of design discharge available at the NJHPS tailrace outfall. Both projects have been operating successfully in tandem since 2015. Given its exposure to extreme hydrological conditions, Nathpa Dam undergoes rigorous inspection and maintenance protocols. These assessments help identify potential risks such as sediment deposition, erosion of sluice spillways, landslides, and structural stress while implementing preventive measures to enhance long-term resilience. This paper presents a comprehensive approach to Nathpa Dam's inspection framework, focusing on post-monsoon and pre-monsoon assessments to ensure its sustainability and operational safety.

1.0 INTRODUCTION

The 1500 MW (6x250 MW), Nathpa Jhakrl Hydro Power Station a run-aff the river scheme is largest hydro power station of India and is the first project commissioned by SJVN Limited. The main features of the project are a 62.5m high concrete Dam, four desilting chambers each 525m long, 16.3m wide and 27.5m high, the head race tunnel (HRT) is 27.4km long and of 10.15 m diameter, surge shaft at the end of HRT is 21.6m in diameter and 301m deep. Three butterfly valves are provided on the pressure shaits, 4.9m dia with lengths varying from 571m to 622 m housed in an underground valve chamber.

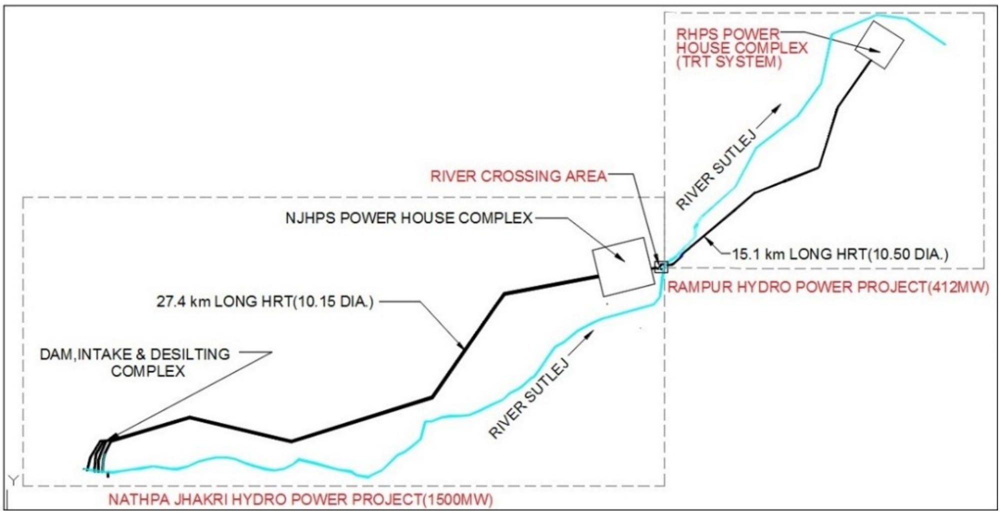

Figure 1. Layout of NJHPS (1500MW) and RHPS (412 MW) in Tandem Operation

79.52 m x 9.50 m x 22.34 m. The machine hail cavern is 222m x 20 m x 49 m high and transformer hall cavern is 195 m x17,5 m x 27 4m high. The rated head for the turbine is 428m and releases from the six turbines are led through a single tail race tunnel 10.15m dia and 982m long to the tail pond.Commissioned in the year 2003, this project is contributing more than 6800 Million Units of energy in a year to the country. Downstream Rampur Hydroelectric Project (412 MW), commissioned during 2015 is a tail race development of Nathpa Jhakri Hydro Power Station (1500 MW) commissioned during 2003 utilizing 383.88 cumecs of design discharge available at tail race outfall of NJHPS. Both the projects are successfully working in tandem. Rampur. The discharge of Nathpa Jhakri Hydro Power Station (NJHPS) outfall is diverted through the Intake structure already constructed for this purpose thus saving the cost of independent desilting and major diversion arrangement. The layout of both hydro power station is shown in Figure 1.

Nathpa Dam is the backbone of both the Nathpa Jhakri Hydropower Project (NJHPS) and the Rampur Hydropower Project (RHPS), playing a crucial role in their successful operation. As the primary water diversion structure on the Satluj River, it regulates and feeds water into NJHPS, which generates 1,500 MW of power through its underground powerhouse at Jhakri. Additionally, RHPS, a tailrace development of NJHPS, utilizing the 383.88 cumecs of design discharge of NJHPS to generate an additional 412 MW. By maintaining hydrological stability, Nathpa Dam enables both projects to function in tandem, maximizing energy production while ensuring efficient resource utilization without requiring a separate dam for RHPS. Therefore, the safety inspection of Nathpa dam is essential for maintaining its structural integrity and operational reliability to run both the projects successfully. Designed as Concrete Gravity of 62.5 m height, are total of five sluice spillway and an overhead spillway in the dam to regulate the excess water flow. The downstream amd upstream view of dam is shown in Figure 2. & Figure 3.

Figure 2. Downstream View of Nathpa Dam

Figure 3. Upstream View of Nathpa Dam

2.0 DAM SAFETY INSPECTION

Following three type of inspection are done for the safety inspection of Nathpa dam:

1) Pre-Monsoon Dam safety Inspection
2) Post- Monsson Dam safety Inspection
3) Post Monsoon Dam Safety Inspection by External agency.

2.1 Pre- Monsoon Dam safety Inspection

The Pre-Monsoon Safety Inspection is done in the month of March- April to assess its structural integrity and operational readiness before the onset of the monsoon season. Also progress of some measures suggested during post monsoon are also checked. A multidisciplinary team from Civil Design, Hydro-Mechanical Design, Geology and project site carries out a thorough inspection covering civil works, instrumentation, hydro-mechanical systems, and geological aspects to identify vulnerabilities and recommend preventive measures.

The civil works inspection focused on key structural elements, including drainage holes, galleries, under sluices, spillways, and the dam body. Drainage holes are inspected for its satisfactorily functioning, while in inspection galleries cracks in surrounding area any seepage is checked. Seepage is checked through V-notches installed in surface drain in inspection galleries and seepage discharge is recorded in LPS. Conditions of V-noches are also checked for its rusting and cleanliness.

Conditions of surface of sluice spillways and their roller bucket are also checked and any repair required is recommended. Also ongoing activities are also recorded for example repair of roller bucket etc. (Figure 4). Exposed rock slopes on the left side of the dam and right side of dam are also observed. Requirement of any shotcrete or any slope stability measures if required are recommended. The stilling basin is also observed from sediment accumulation point of view. Repair of downstream area downstream of stilling basin is also observed.

Figure 4. Steel liner at the end of roller bucket

Operation of all instruments such as stress meters, strain meters, temperature meters, and piezometers are also assessed and monitored regularly. Repair or replacement of instruments such as uplift pressure meters, joint meters, and plumb lines are required sometime and recommended the same. Functionality of the Strong Motion Accelerographs (SMA) and Micro Seismic Recorders, essential for monitoring seismic activity, are also checked and recommended for their restoration if found un-functional. From a hydro-mechanical perspective, the leakage of all Radial Gates is checked, along with the gate position indicators. The intake gates are also inspected. Lubrication and painting of all gates are ensured, and the installed control panels are examined for smooth operation. To ensure safe functioning, water level sensors are also checked. Geological observations are conducted to identify any rock detachment issues on both banks. The condition of all slope supports, such as shotcrete and rock bolts, is assessed, and cable anchors are inspected for any deformations.

After the inspection, preventive maintenance measures are recommended before the monsoon season. Key recommendations include replacing rusted V-notches, monitoring seepage levels, completing sluice and spillway repairs, restoring non-functional instrumentation, and shotcreteing of exposed loose rock areas. An example of repairing the downstream slope is shown in Figure 5.

Figure 5. Concreting Backfill in progress at base of Cladding.

2.2 Post- Monsoon Dam Safety Inspection

The Post-Monsoon Safety Inspection are carried out in the months of September- October to evaluate the impact of the monsoon season on the dam's structural and operational integrity. The inspection is generally focused on assessing civil structures, instrumentation, hydro-mechanical systems, and geological stability to identify damages caused by monsoon-induced stresses and ensure preventive maintenance.

All components and structural elements, such as drainage holes, inspection galleries, spillways, under sluices, and the dam body, highlighted in the pre-monsoon inspection, are also examined during the post-monsoon inspection. The drainage holes and inspection galleries are assessed, and seepage levels are recorded in LPS (liters per second). If a significant increase in seepage is detected after the monsoon, it is monitored regularly, and trends are documented. The glacis floor of the under sluices is also inspected. Nathpa Dam experiences severe erosion in its sluice spillways due to high-volume boulders, some as large as 5 meters in diameter, along with sand particles transported during peak discharge. After each post-monsoon inspection, the sluice spillways are consistently found to be damaged (Figure-6). To address this issue, SJVN collaborated with the Indian Institute of Technology (IIT) Delhi to explore the use of two high-strength concrete types: High-Performance Concrete (HPC) and High-Performance Fiber-Reinforced Concrete (HPFRC), both of M80 grade. The study is currently in its advanced stage, and its findings are expected to provide an effective solution for mitigating spillway erosion, thereby significantly reducing long-term structural degradation.

Figure 6. Damaged floor of under sluice

The leakage of in radial gated and any structural damage radial gates are assessed. Conditions of sill beams, seals, lintels, and wipers are also inspected. Sluice stoplogs also checked.
Rock detachment and slope instability on both the banks are assessed and any protective measures such as wire crates and walls are also suggested. To mitigate rockfall hazards, RCC barriers are recommended. The plunge. Cable anchors on both banks are checked.
The inspection team recommended, several post-monsoon repair and maintenance tasks which are necessary to prevent long-term deterioration. Key recommendations include repairing damaged sluices and radial gates, enhancing slope protection, restoring non-functional instrumentation, and reinforcing erosion-prone areas. Addressing these issues will ensure the dam's long-term resilience and operational safety.

2.3 Post Monsoon Dam Safety Inspection by External agency.

Carrying out dam safety inspections through an external agency is essential for ensuring an objective, comprehensive, and expert evaluation of the dam's structural integrity, operational performance, and safety compliance. External agencies or specialized consultants bring independent expertise, advanced monitoring techniques, and an unbiased assessment, which helps identify critical issues that may be overlooked in routine in-house inspections. Moreover, engaging an external agency enhances accountability and transparency in dam management. An independent evaluation reduces the risk of internal biases or conflicts of interest, which might lead to the underreporting of deficiencies.

SJVN engaged the Dam Safety Organisation (DSO), Nashik, Govt. of Maharashtra, to carry out the post-monsoon inspection of Nathpa Dam on a consultancy basis. Every year, DSO conducts an inspection of Nathpa Dam. The Water Resources Department of Maharashtra established the Dam Safety Organisation (DSO) in 1980 to monitor dam safety. Over the years, DSO has inspected numerous dams and identified several deficiencies reported by its field officers across the country.

The objectives of DSO's inspection are:

i) To carry out inspection of dam structure/components and instruments,
ii) To communicate important deficiencies of dam through inspection report to SJVNL, NJHPS Nathpa, Himachal Pradesh for compliance,
iii) To carry out analysis of instrument data
iv) To suggest remedial measures to field authorities for deficiencies

The Post-Monsoon Inspection for Nathpa Dam, conducted by DSO, evaluates the dam's structural integrity, instrumentation performance, and overall safety. The inspection is generally carried out in November every year. The DSO team also analyses the in-house pre-monsoon and post-monsoon inspection reports and suggests measures to mitigate risks highlighted in these reports. DSO inspections help in long-term safety planning by providing advanced monitoring solutions, such as non-destructive testing (NDT), remote sensing, and real-time data acquisition systems. These technologies enhance predictive maintenance, reducing the risk of sudden failures and ensuring the dam's long-term stability.

The above describes the routine inspections of Nathpa Dam that were carried out before the enactment of the Dam Safety Act, 2021, in December 2021, and have continued to be carried out to date. However, the dam safety requirements as per the Dam Safety Act, 2021, are also being followed in addition to the routine inspections mentioned above.

3.0 CONCLUSIONS

The safety inspections of Nathpa Dam play a critical role in ensuring its structural integrity, operational reliability, and long-term sustainability. The dam is a key component of the Nathpa Jhakri Hydropower Station (NJHPS) and the Rampur Hydropower Project (RHPS), making its continued functionality essential for India's renewable energy sector. Through a structured inspection framework, including pre-monsoon, post-monsoon, and external inspections by the Dam Safety Organisation (DSO), Nashik, potential risks such as sediment deposition, erosion, structural stress, and mechanical wear are regularly assessed and mitigated. The pre-monsoon inspection focuses on preventive maintenance, ensuring that key structural and hydro-mechanical components are functional before the onset of monsoons. The post-monsoon inspection evaluates the impact of heavy rainfall and floods, identifying any deterioration or damages that may have occurred. Additionally, the DSO-led external inspections provide independent assessments, enhancing accountability and transparency while ensuring compliance with safety standards and best practices.

Findings from these inspections have led to critical safety interventions, such as repairing sluice spillways, stabilizing slopes, restoring instrumentation, and improving erosion control measures. Collaborative research with IIT Delhi on high-strength concrete solutions demonstrates the commitment to continuous improvement and innovation in dam safety. Furthermore, advanced monitoring technologies like non-destructive testing (NDT), remote sensing, and real-time data acquisition are being integrated to improve predictive maintenance and reduce the risk of sudden failures.
The multi-tiered inspection process of Nathpa Dam ensures its long-term resilience and operational efficiency. By implementing recommendations from periodic assessments and adopting modern engineering solutions, Nathpa Dam continues to function safely and efficiently, supporting India's energy infrastructure while mitigating potential risks associated with extreme hydrological conditions.

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Jaswant Kapoor graduated in Civil Engineering from the National Institute of Technology, Hamirpur, India. He is currently, working as Head of Department, Civil Engineering Department, SJVN. He has experience of more than 27 years in the area of Hydro Power Development. He has been involved in investigation of Hydro Projects, detailed planning, design and preparation of feasibility studies/Detailed Project Report for a number of large and small hydro projects in India. He was involved in the construction of 1500 MW Nathpa Jhakri HEP, 412 MW Rampur HEP, 60 MW Naitwar Mori HEP in India and 900 MW Arun-3 HEP, 679 MW Lower Arun HEP and other projects in Nepal. He is also looking after India's biggest underground powerhouse project Etalin HEP 3097 MW in Arunachal Pradesh. He has written many papers for the National and International Conferences/Seminars/Workshops held in India and Abroad and attended some of these conferences in India and Abroad.

Revati Raman is a highly accomplished civil engineering professional with over 19 years of experience in the hydropower sector, specializing in the design, survey, and investigation of major hydroelectric projects. He holds a Bachelor's degree in Civil Engineering from the University of Pune, an M.Tech from the Indian Institute of Technology (IIT) Delhi, and an MBA in Operations Management from Indira Gandhi National Open University, Delhi. Throughout his career, he has played a pivotal role in the design and development of several high-capacity hydroelectric projects, including: Rampur Hydroelectric Project (412 MW), Naitwar-Mori Hydro Power Project (60 MW), Arun-3 Hydroelectric

Project (900 MW) in Nepal. His expertise extends beyond design, as he has been deeply involved in the Survey & Investigation (S&I) works of large-scale projects such as: Lower Arun HEP (669 MW) in Nepal – Contributed to the preparation of the Detailed Project Report (DPR) in a record time of just three months, securing approval from the Investment Board of Nepal (IBN) and the Central Electricity Authority (CEA), India, within one year, Arun-4 HEP (630 MW) in Nepal – Involved in preliminary investigations and authored six key chapters of the initial project documentation. Currently serving as Deputy General Manager in the Civil Design Department, Shimla, he oversees the design works for Dhaulasidh HEP (66 MW), ensuring the highest standards of engineering excellence. Beyond his technical contributions, he is an active researcher and thought leader, having authored several technical papers for national and international conferences. He has also represented SJVN and India at prestigious conferences held both domestically and internationally.

Sumeet Thakur graduated with a degree in Civil Engineering from the National Institute of Technology, Hamirpur, India. With 14 years of experience in the hydro power sector, he has held key roles in various significant projects. He served as a Site Engineer for the construction of the Head Race Tunnel of the 412 MW Rampur Hydro Power Project in Himachal Pradesh. His expertise extends to Procurement and Contract Management, where he has been involved in bid preparation, issuing tenders, bid evaluation, awarding contracts, and managing Extension of Time and Variations in Contracts. As a Design Engineer, he has contributed to the preparation of bid documents and the detailed planning and design of various civil components for the under-construction Arun-3 Hydro Electric Project (900 MW) in Nepal. Currently, he is working as Senior Manager in Civil Design Department, and is involved in the planning and detailed design of the Dhaulasidh Hydro Electric Project (66 MW), which is under construction in Hamirpur, Himachal Pradesh.

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Earthen Dam Failure Investigations: Correlating Geophysical Methods with Geotechnical Ground Truthing for Enhanced Reliability

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Renuka Consultants

1. Introduction

Earthen dams are crucial infrastructure elements for water storage, flood control, and irrigation. However, they are vulnerable to structural distress and failures caused by factors such as seepage, piping, differential settlement, and slope instability. Timely detection of such issues is essential to ensure dam safety and prevent catastrophic failures. To achieve this, geophysical methods offer an effective means of non-intrusive subsurface investigation, providing rapid and cost-efficient assessments of dam health.

Among the various geophysical techniques, **Multi-Channel Analysis of Surface Waves (MASW)** has gained significant attention for its ability to assess subsurface stiffness characteristics. MASW is a seismic method that analyzes surface wave dispersion to estimate the shear wave velocity (Vs) profile of the subsurface. Since shear wave velocity is directly related to soil stiffness and compaction, MASW can be used to detect zones of weak material, voids, or potential failure planes within an earthen dam. Compared to conventional seismic methods, MASW offers advantages such as better resolution in near-surface layers, robustness against noise, and the ability to function in difficult terrains.

In addition to MASW, other geophysical techniques such as **Electrical Resistivity Imaging (ERI), Ground Penetrating Radar (GPR), and Seismic Refraction Tomography (SRT)** are widely used for detecting seepage paths, internal erosion, and structural anomalies. However, the reliability of geophysical findings is significantly enhanced when validated with geotechnical ground truthing methods, including borehole drilling, Standard Penetration Tests (SPT), and soil sampling. This integration of intrusive and non-intrusive methods ensures accurate identification of distressed locations and provides a comprehensive assessment of dam stability.

This paper presents a case study of an earthen dam failure investigation, where geophysical methods, including MASW, were utilized to detect anomalies, and the results were validated through geotechnical ground truthing. The study highlights the advantages and limitations of MASW, the necessity of corroborating geophysical data with direct intrusive methods, and the significance of a multi-disciplinary approach in dam failure diagnostics.

The findings from this study reinforce the need for an integrated framework that combines geophysical and geotechnical investigations to enhance the reliability of failure assessments. By systematically utilizing MASW and complementary geophysical techniques, engineers can improve the accuracy of dam distress evaluations and develop effective remedial measures for sustaining dam safety.

2. Problem Statement: Sudden Failure of Earthen Bund in a Newly Constructed Reservoir

The sudden failure of an earthen bund in a newly constructed raw water reservoir raised critical concerns regarding design adequacy, construction methodology, and the geotechnical behavior of the embankment materials. The reservoir, which had just been completed and was undergoing its **first fill**, experienced an unexpected failure near the intake structure, leading to the rapid release of water and debris. This event required a comprehensive investigation to understand the underlying cause and develop insights for improving dam performance and safety.

Key Factors Contributing to the Criticality of the Failure:

- **New Construction**: The absence of long-term operational history ruled out aging-related distress, making material behavior and construction methodology primary considerations.
- **Fresh Water Load**: The failure occurred during the initial filling phase, suggesting a potential issue related to **hydrostatic pressure response**, soil strength, or compaction quality.
- **Sudden and Rapid Onset**: Unlike gradual seepage-related failures, this event was abrupt, indicating a potential geotechnical instability such as shear failure, liquefaction, or inadequate compaction.
- **Proximity to the Intake Structure**: Failures near intake structures can disrupt reservoir operations, increasing the complexity of repairs and potential long-term performance issues.

Geotechnical Characteristics of the Embankment

The embankment was designed using **engineered cohesionless soils** to achieve stability and optimal load-bearing performance. Key geotechnical properties included:

- **Shear Strength Parameters**: The friction angle (ϕ) ranged between **28° to 32°**, providing adequate resistance against shear forces under normal loading conditions.
- **Bulk Density**: The soil had a bulk density in the range of **17.5 to 18.5 kN/m^3**, ensuring appropriate compaction and stability.
- **Permeability**: The formations exhibited a permeability of **$\sim 1 \times 10^{-5}$ m/s**, allowing controlled water movement within the design specifications.
- **Seepage Control Measures**: The embankment was internally lined with a **geomembrane** to regulate seepage and enhance durability. Understanding the interaction between the soil and geomembrane was essential to assess long-term performance.

Assessment of Failure Causes

The **sudden nature of the failure without prior distress signals** was a major concern for the authorities. A detailed assessment of site conditions and construction records suggested the following key aspects:

- **Location of Failure**: The breach occurred in the bund sections **adjacent to the intake well**, which had been constructed after the embankment was completed. This section also housed the **sluice valve chamber**, which controlled water flow within the reservoir system.
- **Construction Sequence Suspicions**: Authorities suspected that the **construction sequence** of the intake well and associated structures might have influenced the embankment's stability. Any disturbance to the existing soil conditions during the intake well construction could have led to localized stress concentrations or differential settlement.
- **Potential Piping Failure**: Given the embankment's permeability characteristics and the presence of a water-retaining structure, **piping failure** was considered a plausible failure mechanism. Piping can occur when water flow through a soil mass initiates erosion along paths of least resistance, gradually removing soil particles and creating voids. This process, if unchecked, can lead to a sudden structural collapse.
- **Hydraulic Interactions**: The embankment's geomembrane lining was intended to prevent seepage, but interactions between the lining, soil, and intake structure foundation needed further evaluation. Any gaps, unsealed joints, or local stresses at the interface could have created conditions for internal erosion.

Approach for Investigation

To determine the precise cause of failure, a structured investigation approach was implemented:

1. **Assessment of Mechanical Properties of Soil**:
   - Both invasive and non-invasive methods were used to evaluate **soil stiffness, compaction efficiency, and shear strength**.
   - **Multi-Channel Analysis of Surface Waves (MASW)** was conducted to determine **shear wave velocity (Vs)**, a critical indicator of subsurface integrity.
2. **Triangulation of Data**:
   - **Invasive Geotechnical Testing**: Standard Penetration Test (SPT) and laboratory analysis for soil classification, density, and mechanical properties.
   - **Non-Invasive Geophysical Investigations**: MASW to evaluate subsurface stiffness variations and detect potential anomalies.
   - **Statistical Methods**: Data analytics and regression models to ensure consistency and accuracy in interpretations.

By integrating these techniques, this study aims to enhance the understanding of embankment behavior, validate geophysical interpretations through geotechnical ground truthing, and provide recommendations for optimizing reservoir embankment performance in future projects. In this paper one of the breach sections is considered for observations and analysis.

Figure 1. Location and details of Earthen Dam Failure

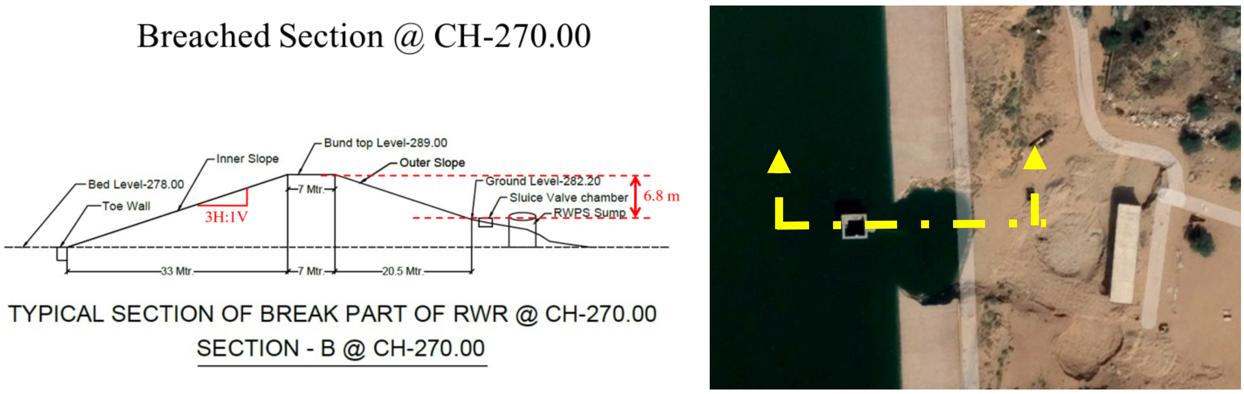

Figure 2. Breach Cross-section

3. Results and Analysis

Geophysical and Geotechnical Data Analysis for Failure Assessment

The failure analysis of the embankment required a comprehensive assessment of both **geophysical (MASW) and geotechnical (SPT) data** to understand subsurface conditions and potential failure mechanisms. The MASW data provided insights into the **shear wave velocity (Vs) variations**, which correlate with soil stiffness, while the SPT data validated the in-situ soil strength characteristics. A comparative evaluation of these datasets at two key locations, **T1 (near the intake structure) and T5 (further away from the failure zone),** helped establish correlations between material properties, embankment behavior, and potential instability factors.

MASW Data and Interpretation

The **Multi-Channel Analysis of Surface Waves (MASW) 2D plots** revealed distinct variations in **shear wave velocity (Vs) values** across different sections of the embankment. At **T1**, the Vs values in the upper **0-7m depth** predominantly ranged between **200-400 m/s**, with a clearly identifiable weak zone between **3-6m depth** characterized by significantly lower Vs values. This indicated a possible region of **looser or water-susceptible soil**, potentially associated with inadequate compaction or seepage pathways. Additionally, localized transitions from low to high Vs values within this range suggested differential material conditions, which could lead to variations in settlement and stress distribution.

Conversely, at **T5**, the MASW results indicated a relatively more uniform distribution of Vs values, ranging from **250-500 m/s**, with a **gradual increase in stiffness with depth**. Unlike T1, there were no pronounced low-velocity zones, suggesting **better-compacted soil** with relatively stable subsurface conditions. The absence of sudden changes in Vs values at T5 pointed to a more **homogeneous soil profile**, reducing the likelihood of differential settlements or seepage-induced instability.

SPT Data and Interpretation

The Standard Penetration Test (SPT) values provided direct **geotechnical validation** of soil density and strength, further complementing the MASW observations. At **T1**, the SPT values in the upper **2-3m depth** were relatively low (~10-12), indicating **loose to medium-dense material**, followed by a sharp increase in values beyond 3m, reaching up to 35 at 5m depth. This trend strongly correlated with the MASW findings, where the **3-6m depth zone with lower Vs values** aligned with the **weaker SPT zones**, reinforcing the presence of **a relatively weaker, less compacted soil layer in this section**.

In contrast, the **T5 location exhibited consistently higher SPT values (~20-45) throughout the profile**, reflecting **a denser soil formation compared to T1**. The **gradual increase in SPT values with depth at T5** further supported the MASW interpretation of a **steadily compacting subsurface**, reinforcing the assumption that this section was structurally more stable. The close match between the SPT and Vs values at T5 validated the reliability of the geophysical interpretation, demonstrating a **strong correlation between higher Vs values and denser soil conditions.**

Comparison and Failure Implications

A direct comparison of MASW and SPT data at **T1 and T5** highlighted the **significant differences in subsurface conditions, which likely influenced the embankment failure.** The lower **SPT values (10-12) in the upper 2-3m at T1** matched the **low Vs values (200-300 m/s)**, indicating a **looser soil layer susceptible to seepage or stress-induced failure.** The presence of a **localized weak zone in the 3-6m range at T1**, as identified in the MASW plot, further supported the hypothesis that **this area might have been prone to piping or settlement-induced instability.** Given that the intake well and sluice chamber were constructed post-embankment completion, potential **disturbances in the soil structure around this region** could have contributed to a localized failure mechanism.

On the other hand, **T5 exhibited a more gradual and stable increase in both SPT and Vs values**, indicating a **well-compacted embankment section** with a **low risk of seepage failure.** The absence of significant weak zones at T5 suggested that this section was less affected by post-construction activities or differential settlement, reinforcing its stability.

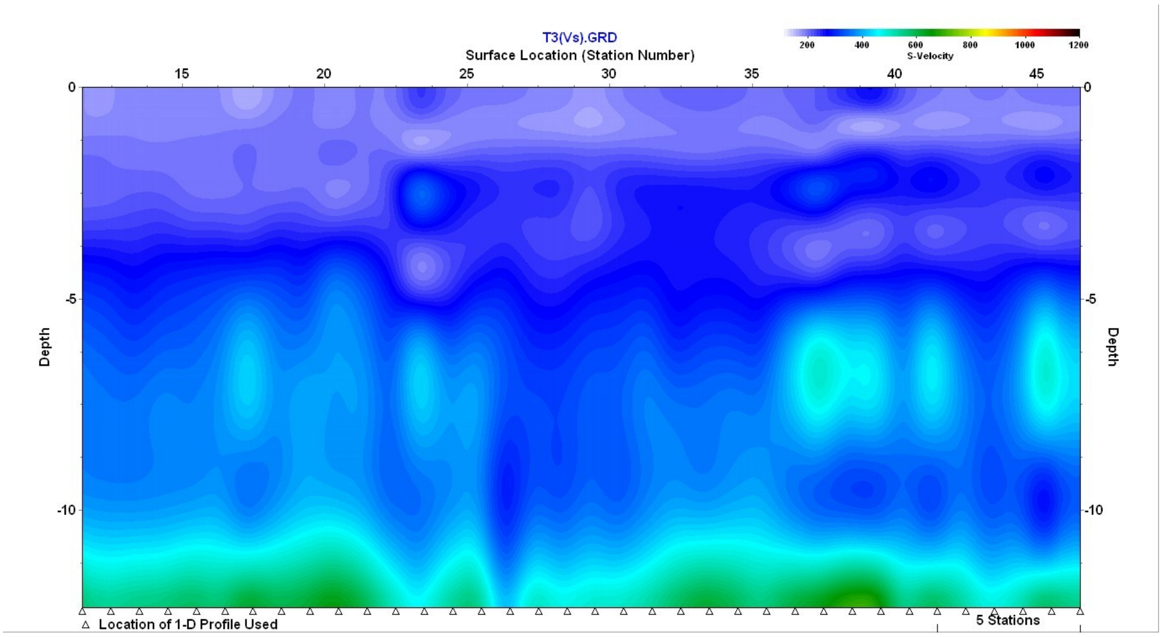

Figure 3 2-D Analysis at T3 Location

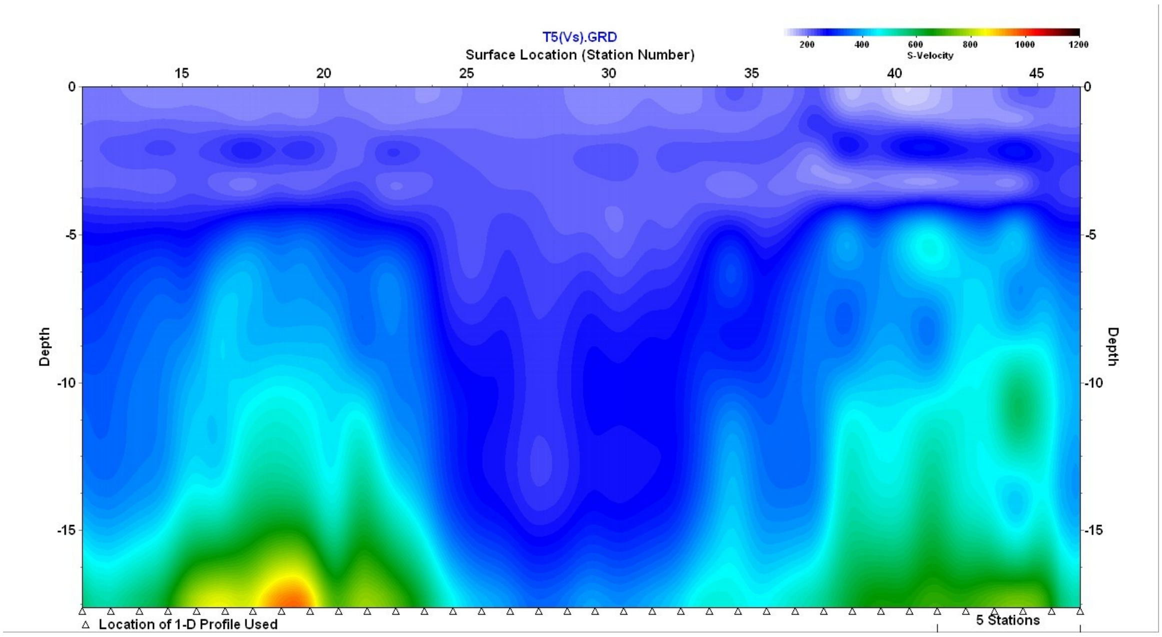

Figure 4 2-D Analysis at T5 Location

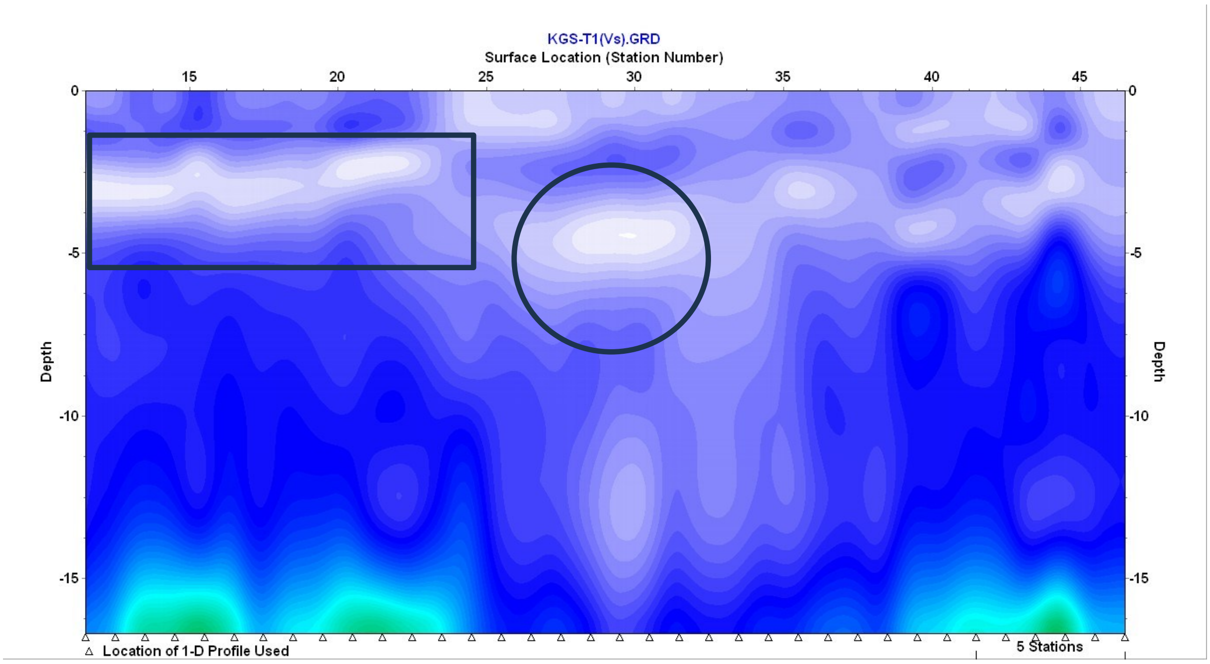

Figure 5 2-D Analysis at T5 Location

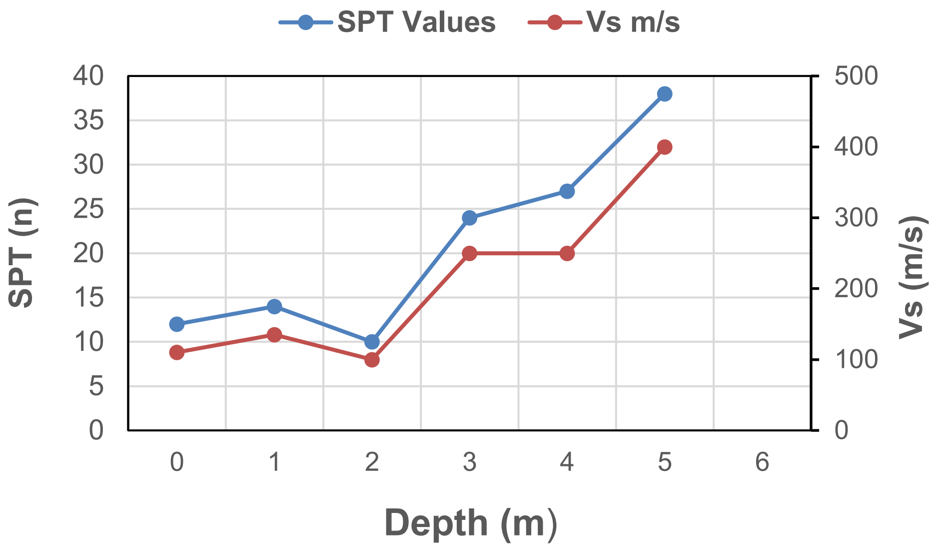

Figure 6 SPT comparison with Vs at T1 location

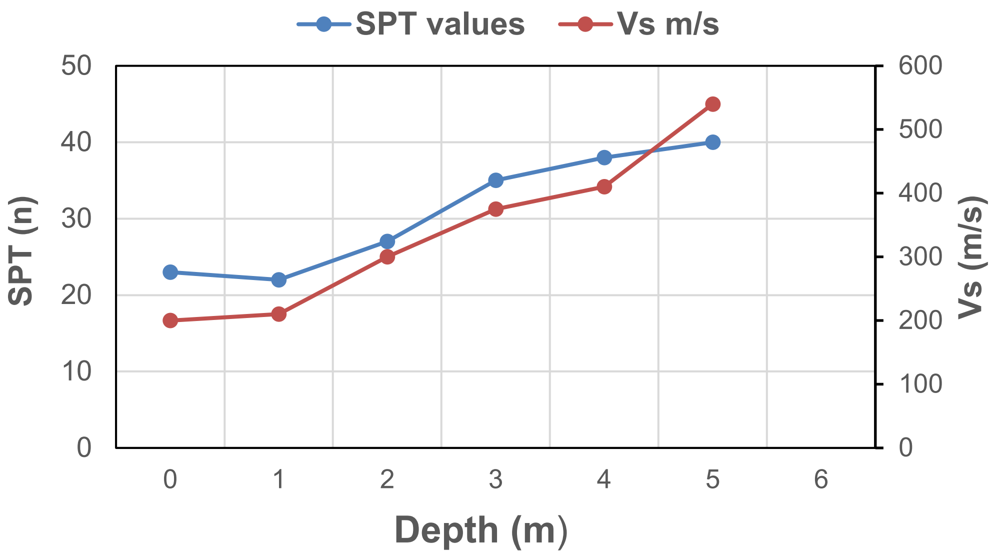

Figure 7 SPT comparison with Vs at T3 location

4. Key Comparisons and Failure Analysis

Parameter	T1 (Near Intake Structure)	T5 (Further Away)
SPT in Upper 2m	Lower (~10-12) – Loose soil	Higher (~20-25) – Denser soil
Vs in Upper 2m	Lower (~100-200 m/s)	Higher (~150-250 m/s)
SPT Increase Rate	Rapid after 3m (loose to dense transition)	Gradual increase with depth
Vs Increase Rate	Steep rise after 3m depth	Steady rise with depth
Potential Weak Zones	3-6m depth, as seen in MASW	No major weak zones
Failure Susceptibility	Higher due to weaker upper layers and possible seepage effects	More stable section with denser formation

Statistical Analysis at T1 Location

Parameter	count	mean	std	min	25%	50%	75%	max
Depth (m)	9	5.99	4.91	0.60	2.20	4.80	8.70	15.00
Vp (m/s)	9	1292.78	201.17	1150.00	1200.00	1235.00	1300.00	1800.00
Vs (m/s)	9	508.44	68.25	450.00	472.00	487.00	511.00	678.00
Poisson's Ratio	9	0.41	0.00	0.40	0.40	0.41	0.41	0.42
Density (gt)	9	19.91	0.24	19.50	19.80	20.00	20.10	20.14
Dynamic Shear Modulus (MPa)	9	534.03	163.00	402.52	449.66	483.53	535.02	943.74
Dynamic Young's Modulus (MPa)	9	1504.69	464.93	1134.79	1266.67	1355.24	1507.28	2675.17

Statistical Analysis at T3 Location								
Parameter	**count**	**mean**	**std**	**min**	**25%**	**50%**	**75%**	**max**
Depth (m)	9	5.99	4.91	0.60	2.20	4.80	8.70	15.00
Vp (m/s)	9	1199.22	105.60	1075.00	1140.00	1180.00	1235.00	1426.00
Vs (m/s)	9	476.89	43.90	426.00	454.00	468.00	489.00	577.00
Poisson's Ratio	9	0.41	0.00	0.40	0.40	0.41	0.41	0.41
Density (gt)	9	19.91	0.24	19.50	19.80	20.00	20.10	20.14
Dynamic Shear Modulus (MPa)	9	465.68	94.44	360.73	416.01	446.53	489.94	683.51
Dynamic Young's Modulus (MPa)	9	1309.43	264.28	1015.00	1169.63	1254.54	1378.73	1916.70

Statistical Analysis at T3 Location								
Parameter	**count**	**mean**	**std**	**min**	**25%**	**50%**	**75%**	**max**
Depth (m)	9	5.99	4.91	0.60	2.20	4.80	8.70	15.00
Vp (m/s)	9	1233.33	105.45	1085.00	1180.00	1230.00	1270.00	1450.00
Vs (m/s)	9	489.78	40.84	432.00	472.00	479.00	512.00	568.00
Poisson's Ratio	9	0.41	0.00	0.40	0.40	0.41	0.41	0.41
Density (gt)	9	19.91	0.24	19.50	19.80	20.00	20.10	20.14
Dynamic Shear Modulus (MPa)	9	490.55	87.91	370.97	449.66	463.09	537.12	662.35
Dynamic Young's Modulus (MPa)	9	1379.74	247.59	1043.01	1263.32	1303.22	1507.11	1866.99

Statistical Analysis Summary

The statistical analysis highlights the **variability in geophysical parameters** across T1, T3, and T5, allowing us to assess soil stiffness, strength distribution, and potential weak zones.

- **Mean and Standard Deviation Analysis:**
    - **Vs (Shear Wave Velocity)**: The **higher mean values at T5 compared to T1 and T3** suggest **a more stable embankment section** at T5. T1 exhibits **greater variability in Vs**, potentially indicating **heterogeneous soil conditions or weak zones**.
    - **Dynamic Shear Modulus & Young's Modulus**: Lower mean values at **T1 suggest relatively weaker soil layers**, which is consistent with observations from failure zones. **T5 has the highest mean values, reinforcing its stability.**
- **Box Plots Interpretation:**
    - **T1 shows a wider range of Vs values**, indicating non-uniformity in soil stiffness, which could be a factor in embankment failure.
    - **T3 has slightly improved stiffness but still shows variability**, suggesting transition zones.
    - **T5 exhibits tighter clustering of values**, indicating **consistent compaction and more homogeneous soil conditions** compared to T1 and T3.

Comprehensive Interpretation & Triangulation with MASW and SPT Data

a. Comparison with MASW Data

- MASW plots revealed localized weak zones at T1, especially between 3-6m depth, where lower Vs values were recorded.

- The statistical analysis validates this observation, as the higher standard deviation in Vs and lower mean values at T1 confirm the presence of material inconsistencies.
- T5 showed a smooth Vs increase in MASW data, which aligns with the statistical data confirming uniform soil conditions.

b. Correlation with SPT Data

- At T1, SPT values were significantly lower in the upper 3m, aligning with the lower Vs values in MASW and higher variability in the statistical dataset. This confirms that T1 had less compacted soil, making it more susceptible to seepage failure.
- T3 SPT values showed a moderate increase with depth, supporting the transitioning Vs values observed in MASW and moderate variability in statistical results.
- T5 had the highest SPT values, and the statistical data confirm lower variability and higher mean Vs, reinforcing its stability compared to the other two locations.

c. Triangulation of Data & Failure Hypothesis

- T1 exhibits the highest variation in Vs and geophysical parameters, supporting the hypothesis that post-construction disturbances near the intake structure weakened the embankment.
- T3 represents a transition zone, where gradual improvements in soil stiffness suggest moderate compaction but still some degree of variation.
- T5 is the most stable section, as evidenced by higher and more consistent Vs, higher SPT values, and strong statistical clustering of geophysical parameters.

5. Conclusion

The integrated approach of non-invasive and invasive assessment greatly improves the confidence in embankment stability evaluations compared to relying solely on geophysical investigations. By calibrating geophysical findings with geotechnical evaluations, the reliability of the analysis is enhanced, ensuring more informed and data-driven decision-making in dam safety assessments.

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Latest Technologies for Rehabilitation and Dam Safety, 20-22 March 2025, Shimla

Feasibility study of suitable Grouting technique for Seepage Remediation in Sikaser Dam of Chhattisgarh

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Extended Abstract

The Sikaser Dam is located near Sikaser village, District Gariyaband in Chhattisgarh. The dam is constructed on Pairi river in the year 1977. The dam can be classified as homogeneous earthen dam which consists of two parts, namely the main earthen dam which is 876.31m in length and, and the saddle dam which is 666.60m in length. According to Inspection report on Sikaser Dam by the state dam safety review panel (State DSRP) dated 09.02.2022, the seepage was observed on the downstream slope of the dam at the chainage 28.75 of main dam. The seepage location was approximately 3-4 m above the ground level. It was also observed that the seepage on the downstream occurs continuously and finally stops until the water in the reservoir reaches a particular level. The seepage then recedes with the lowering of the water table. It was concluded that seepage began due to internal erosion through the core of dam. Since seepage only starts when water reaches a particular level and recedes with lowering of reservoir level, piping has been occurring through the dam body. Reconstruction/replacement of core section of the dam was suggested as one of the treatment methods in the report, and it was advised to observe and record the downstream seepage discharges.

In the present study, the feasibility of the two grouting techniques namely, the permeation grouting and jet grouting has been assessed by means of laboratory studies. First, the two site visits were performed in July (pre-monsoon) and September (during monsoon) of year 2024. The objective of the first visit was to examine the seepage location and collect soil samples. It was observed that a substantial cavity had formed at the seepage point location. In the second visit, the seepage condition was re-examined after monsoon season and samples were collected from five different locations. Given the monsoon conditions in the month of September, continuous water seepage was evident through the cavity.

Next, the collected samples were brought to Geotechnical laboratory, Civil Engineering Department of Indian Institute of Technology Roorkee. Both bulk density and dry density were evaluated by core cutter method. Tests were conducted for grain size analysis, Atterberg limits, specific gravity, falling head permeability and unconfined compressive strength (q_u), as per the relevant IS codes.

Based on the grain size analysis and Atterberg limits, the soils from the five locations were classified as CL-ML, SM-SC, CL, SC and CL. The percentage of fines ranged from 44 % to 61 %. The liquid limit and plastic limit were found to be in the range of 21 %-27 % and 13 %-17 %. UCS test was performed on 3 soil samples of location. The q_u values were obtained as 95. 2 kPa, 118.17 kPa, & 116.5 kPa. The average value of q_u and undrained cohesion c_u can be taken as approximately 110 kPa and 55 kPa, respectively, indicating medium to stiff clays.

As soil is found to be silts and clays, permeation grouting with conventional particulate grouts such as ordinary portland cement is not possible (Mitchell, 1981). Recently, researchers have claimed that the chemical grouts such as acrylic resin and colloidal silica can be injected in silty sands with permeabilities as low as 10^{-6} m/s (Fraccia et al. 2022, Liu et al. 2023). The permeability for soil collected from location near cavity of Sikaser Dam was obtained as approximately 1.26 x 10^{-6} m/s. However, effectiveness of acryclic resin and colloidal silica has not been proven for low plastic silts and clays. As at three locations, the soil comprise of low plastic clays, permeation grouting with afore-mentioned advanced grouts is still not possible.

Thus, in the present scenario, a cut-off remediation wall constructed by either jet grouting (JG) or deep soil mixing (DSM) technique seem to be viable option. Unlike permeation technique which depends on the permeability of in-situ soil, JG and deep soil mixing (DSM) techniques construct soil-cement columns. DSM involves cutting and mixing tools for cutting the soil while simultaneously injecting and mixing cement and water with the in-situ soil, the JG technique applies high-pressure jets of grout slurry with/without water and/or air in to in-situ ground. Therefore, in the present scenario, ordinary portland cement with water to binder ratio of 1.5 to 2 with 3% bentonite seems to be a viable solution for construction of cut-off wall with overlapping columns.

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Integrating Dam Safety Aspects in Pumped Storage Projects for Sustainable Hydropower Development: A GIS-Based Approach

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ABSTRACT

Post-Industrial Revolution, there has been an alarming rise in the global average temperature, leading to a severe climatic threat to all living beings on the planet, primarily due to emissions of greenhouse gases from industries, power plants, vehicles, and other sources. With coal still playing a dominant role in the energy mix, efforts are being made to transition to renewable and cleaner energy sources like solar, wind, and small hydropower.

However, the intermittent nature of solar and wind energy poses challenges for grid stability, as energy production does not always align with demand. The integration of Pumped Storage Projects is pivotal in addressing this issue, providing a reliable solution to store excess energy and release it during peak demand periods.

Pumped Storage Projects are analogous to large water batteries, consisting of two reservoirs at a significant elevation difference. These systems can generate power when demand is high, and during periods of low demand, water is pumped back to the upper reservoir using the excess energy available. This cyclical process not only aids in stabilizing the grid but also enhances the overall efficiency of the energy system by maximizing the use of renewable resources.

One of the major challenges in these projects is locating reservoirs with sufficient capacity over a small distance, along with a desirable elevation difference, minimal resettlement and rehabilitation (R&R) issues, minimal submergence area, no interstate disputes, and proximity to the grid. Identifying suitable sites is crucial, as it can significantly impact the feasibility and success of these projects.

Geographical Information System (GIS) is a computer-based tool used to collect, analyze, and visualize geographical and spatial data. The tool analyzes various criteria, including existing reservoirs, elevation differences, and environmental considerations, to determine the most feasible locations for pumped storage hydropower (PSH) development within a particular region. One such location satisfying these conditions was chosen for a detailed case study.

The traditional method of identifying such sites is time-consuming and costly. This paper presents a GIS-based methodology for identifying potential sites for PSPs, emphasizing the importance of dam safety in accordance with the Indian Dam Safety Act 2021. The integration of GIS tools allows for efficient site selection, while adherence to safety regulations ensures the long-term reliability and stability of hydropower infrastructure. The study focuses on the Chhattisgarh region of India, which presents unique geographical and geological characteristics conducive to the development of PSPs.

LIST OF ABBREVIATIONS

DEM: Digital elevation model

PSP: Pump Storage Project

Q: Discharge (cumecs)

WCS: Water Conductor System

1. INTRODUCTION

The global energy landscape is undergoing a significant transformation as countries strive to reduce greenhouse gas emissions and transition to renewable energy sources. While solar and wind energy are among the most cost-effective renewable energy sources, their intermittent nature poses challenges for grid stability. Pumped Storage Projects (PSPs) offer a viable solution by storing excess energy during periods of low demand and releasing it during peak demand, thereby balancing the grid and enhancing the efficiency of renewable energy systems.

The success of a PSP largely depends on the selection of an appropriate site, which must meet several criteria, including sufficient elevation difference, adequate water storage capacity, proximity to the grid, and minimal environmental and social impacts. Traditional methods of site selection are often time-consuming and costly. This paper proposes a GIS-based approach to streamline the process of identifying potential PSP sites, with a particular focus on the Indian state of Chhattisgarh.

In addition to site selection, dam safety is a critical consideration in the design and operation of PSPs. The Indian Dam Safety Act 2021, which came into effect in December 2021, provides a robust legal framework for ensuring the safety of dams across the country. The Act mandates regular inspections, real-time monitoring, and the implementation of emergency action plans to prevent dam failures. This paper explores the implications of the Act for PSPs, particularly in terms of structural integrity, spillway capacity, and the management of fluctuating water levels.

2. STUDY AREA

2.1 LOCATION

India has a very wide range of geography and has high mountains, fertile plains, scorching deserts, hills and plateaus. The Indian peninsula has a huge potential of pumped storage power due to the availability of steep slopes in a very small distance which is a desirable criterion for the pumped storage power. The area chosen for study lies in Chhattisgarh state of India.

The DEM coordinates selected for the study are 21.999861°N to 23.000139°N latitude and 81.999861°E to 83.000139°E longitude and they fall within Chhattisgarh state. Chhattisgarh, a central Indian state, features a diverse landscape with plateaus, hills, rivers, and dense forests. It is part of the Deccan Plateau, with the Maikal Hills in the northwest and the Bastar Plateau in the south.

The Chhattisgarh Plain, shaped by the Mahanadi River and its tributaries like the Shivnath, Hasdeo, Arpa, Indravati, and Jonk, dominates the central region. The Surguja and Jashpur hills in the north extend from the Vindhya and Satpura ranges, while the forested slopes of the Eastern Ghats are in the east. The state is rich in minerals such as coal, iron ore, and bauxite, playing a vital role in India's mining and industry. Chhattisgarh is also home to tropical dry deciduous forests and wildlife reserves like Indravati and Udanti-Sitanadi, contributing to its ecological diversity.

2.2 GEOLOGICAL AND GEOTECHNICAL CONDITION

The geological formations in the study area primarily consists of basalt, granite, shale, and sandstone, present varying degrees of suitability for the construction of dams and WCS for a PSP. Basalt, a dense and fine-grained volcanic rock, is highly favorable due to its durability, low porosity, and resistance to weathering, making it ideal for dam foundations and reservoirs, as it minimizes the risk of seepage. Similarly, granite, known for its strength and low permeability, provides an excellent foundation material for hydropower infrastructure, ensuring stability and long-term performance.

In contrast, shale, a fine-grained sedimentary rock, can be problematic in areas with high water flow, as its susceptibility to weathering and erosion may lead to reduced structural integrity and increased seepage risks, complicating the design and construction of dams and water conduits. Sandstone, with its higher porosity, may also require additional reinforcement to prevent water seepage, particularly in the construction of water conductor systems.

The geological suitability for Pumped Storage Projects (PSP) in the study area is influenced by the predominant rock types, including basalt, granite, shale, and sandstone. Basalt, with its dense, fine-grained structure, offers excellent durability, low porosity, and high resistance to weathering, making it ideal for the construction of stable dam foundations and reservoirs with minimal seepage risks. Granite, known for its strength and low permeability, is equally suitable for ensuring long-term stability in hydropower infrastructure.

However, shale and sandstone present challenges due to their susceptibility to erosion and higher porosity, which could lead to seepage issues and undermine the structural integrity of dams and water conductor systems. Therefore, basalt and granite are highly favorable for PSP construction, while shale and sandstone require careful engineering to mitigate water retention and stability concerns.

3 METHODOLOGY

A GIS analysis has been done in the area using DEM and Arc GIS Pro to find out potential sites in the area and to find out alternative layouts for the water conductor system. The analysis consists of five-part processes, including the following steps:

- Defining pumped storage site selection criteria.
- Generation of drainage lines in the DEM to analyse the drainage pattern in the area.
- Reclassification of the area into multi-colour bands to identify possible area suitable for a pumped storage project.
- Identifying possible areas for the project.
- Generation of the contours

A. Defining pumped storage site selection criteria:

A Pumped Storage Project is chosen based on the significant height difference over a short distance, as the energy produced is dependent on both head and discharge.

The energy output can be increased by either raising the head or the discharge. Since the head is influenced by the topography and increasing the head is more cost-effective, alternatives with a greater head should be prioritized. A predefined constant energy output can be maintained with a reduced discharge if the net head is increased.

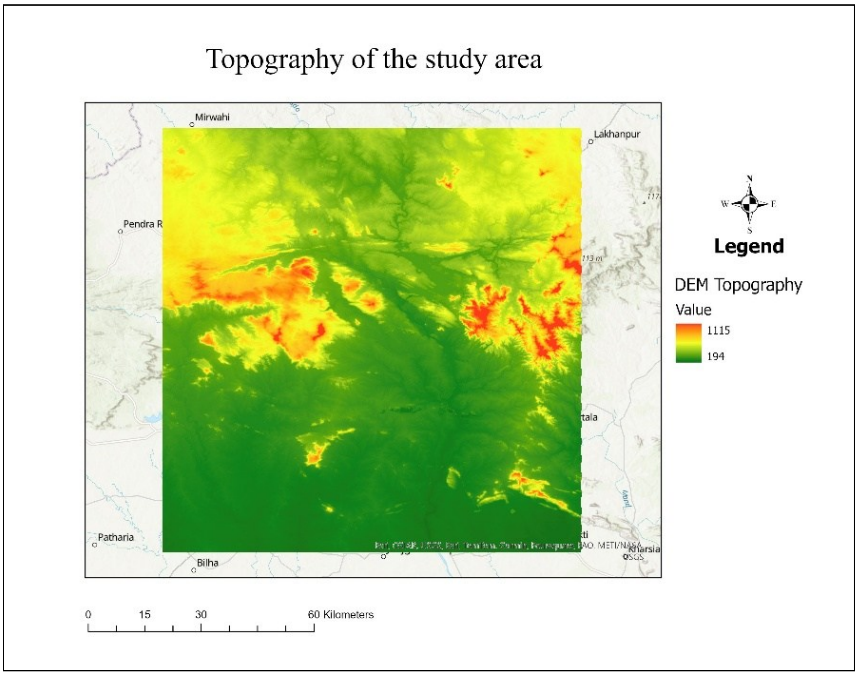

Figure 1: Elevation range in the study area

A low WCS length-to-head ratio is advantageous; as the head increases, the conveyance length also increases. This is beneficial when the waterway length is short at a constant head. An alternative is more economical when the conveyance length-to-head ratio decreases. A high length-to-head ratio also raises the likelihood of needing a surge tank, which further escalates the project's cost as well as the suitability of topography for surge shaft chamber.

B. Generation of drainage lines in the DEM to analyse the drainage pattern in the area:

The drainage lines are required to understand the drainage pattern in the region and to also identify the possible reservoirs that are feasible along the streams. The streams are generated using various operations on Arc GIS. The following steps/operations are undertaken to generate the streams on the DEM:

- Project the DEM raster in the desired coordinates (UTMN in this case)
- Fill sink
- Flow direction
- Flow accumulation
- Raster Calculation with flow accumulation greater than the required number
- Raster to Polyline

C. Reclassification of the area into multi-colour bands to identify possible area suitable for a pumped storage project:

The DEM is reclassified by editing the symbology to convert the DEM into multiple bands where each band represents an interval of elevations on the DEM. The points where there is significant change in the spectrum of colours in a very small distance are our potential sites where we can see the possibility of making reservoirs in the area. In other words, we can say that these points are also an indication of areas with significant slopes in smaller lengths.

We must reclassify the DEM as per our desired head for example if we desire to look for areas where we can find a head difference of say 400 m, then we may preferably use the bands in an interval of 50 m class interval in order to easily identify the potential sites by visual examination.

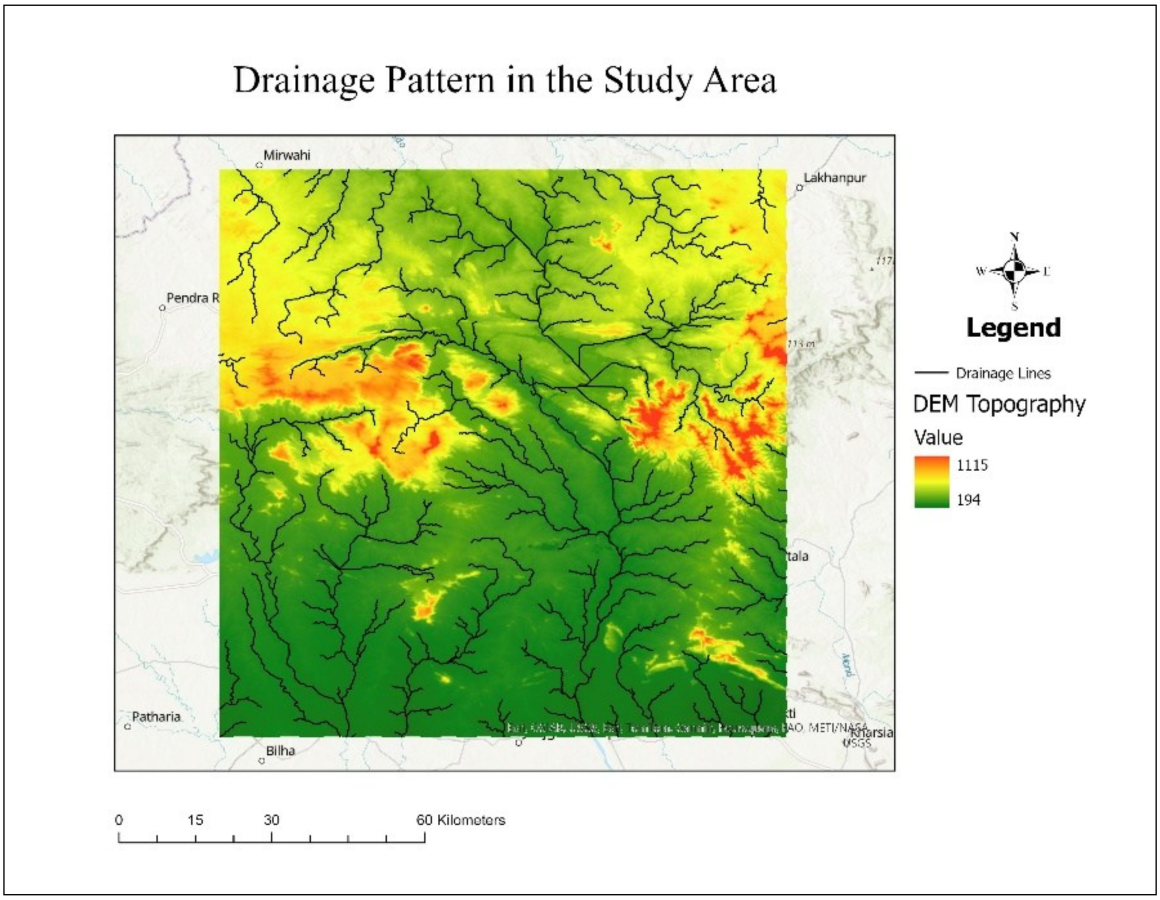

Figure 2: Drainage lines in the study area

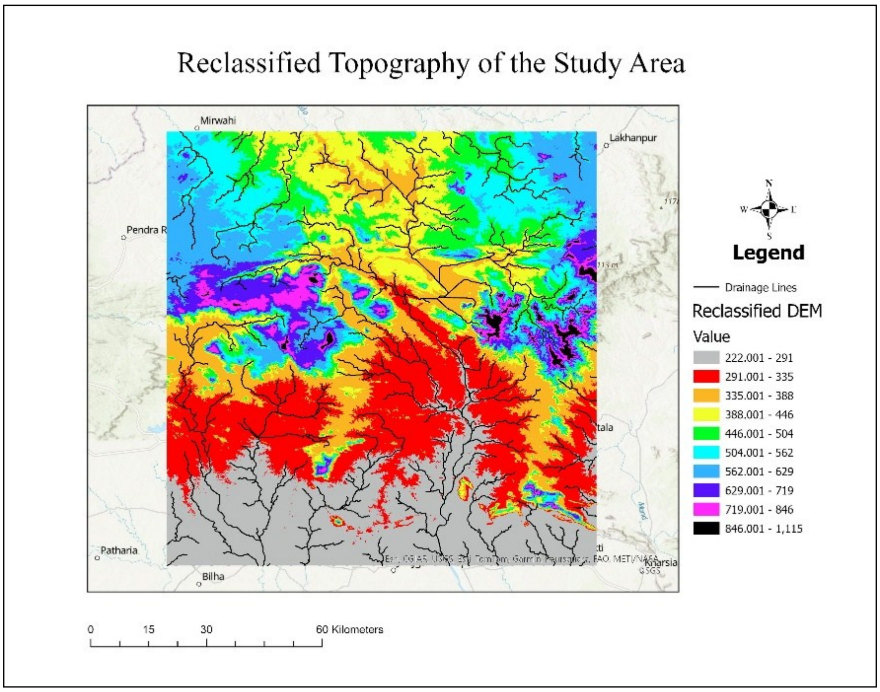

Figure 3: Colour coded elevation ranges the study area

D. Identifying possible areas for the project:

The next step is to select a potential point for the upper reservoir and consider it as a centre of a circle and start making concentric circles of radius say 1000 m, 2000 m, 3000 m, 4000 m and 5000 m.

While shorter length of WCS is generally for Pumped Storage Projects, the radius in this has been kept up to an upper limit of 5000 m as this is an illustration.

These circles represent the area inside which we are looking to find the second reservoir which will serve as a lower reservoir. From visual examination of these circular areas, the potential site for the lower reservoir is selected based on the required characteristics like elevation difference, storage etc.

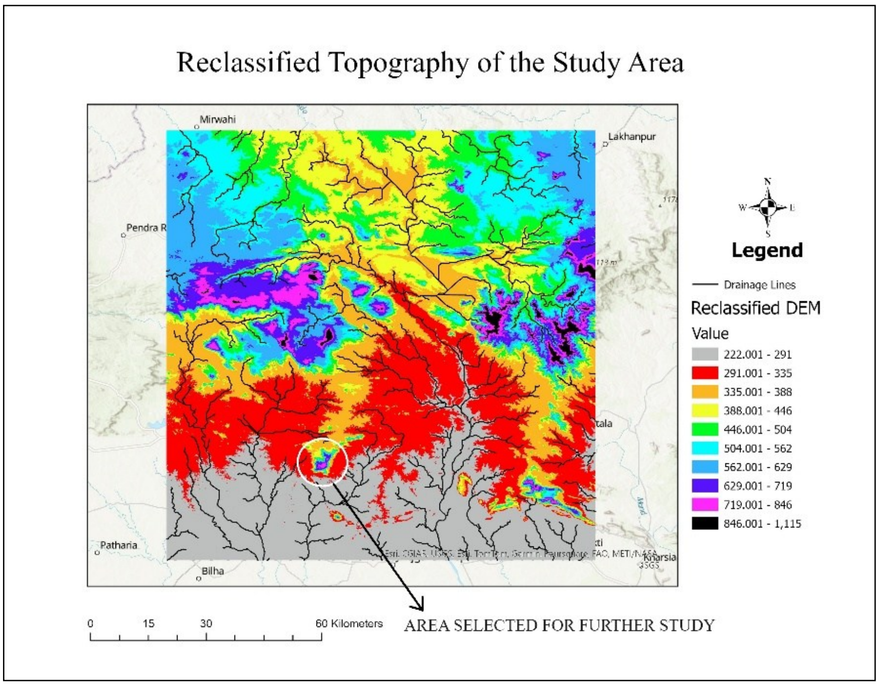

Figure 4: Area chosen for further study for reservoir location

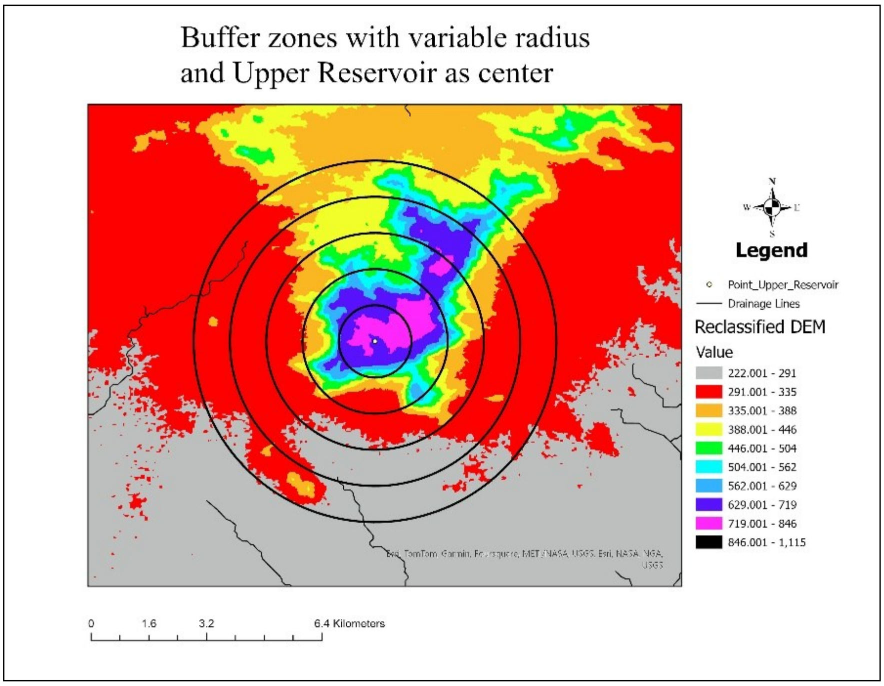

Figure 5: Possible Upper and Lower reservoirs in the DEM with concentric circles

E. Generation of the contours:

The contours are generated once the potential area for identification has been selected at a desired contour interval to find out the possible reservoirs on the streams by inserting dams on the DEM and check the water levels and the areas of submergence due to the dam.

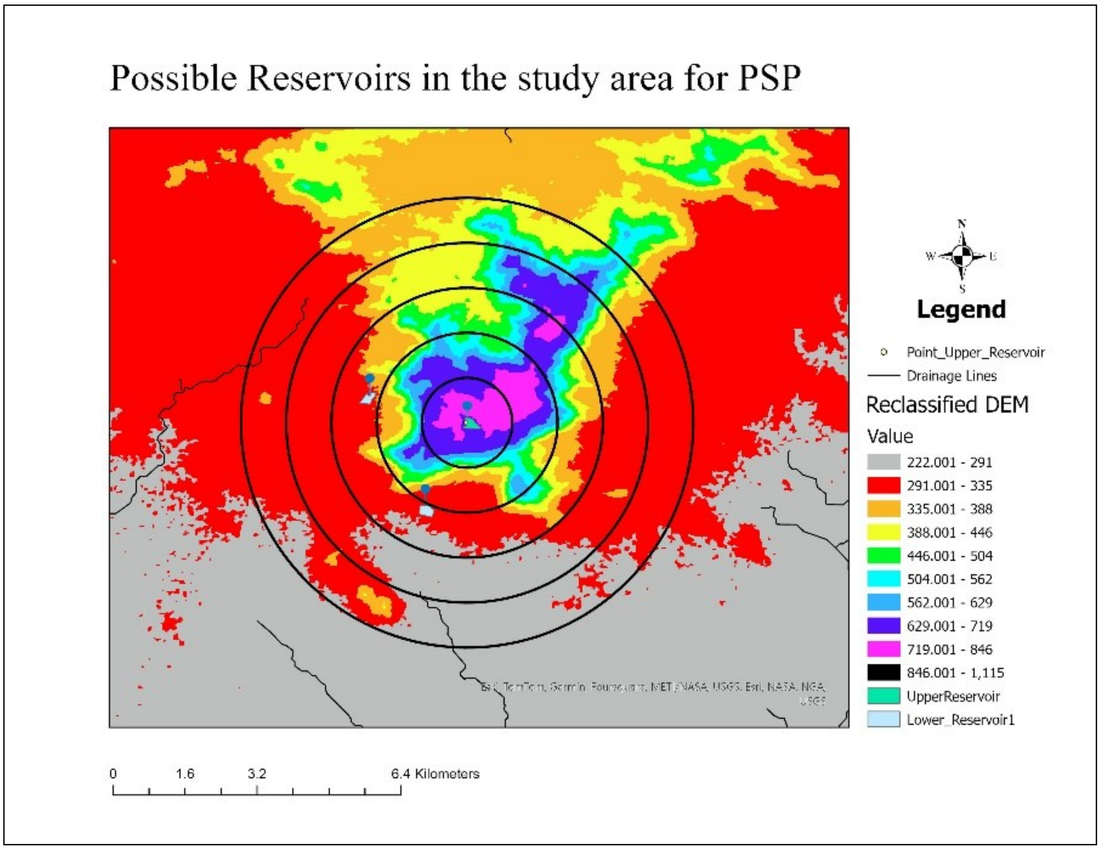

Figure 6: Possible reservoirs located

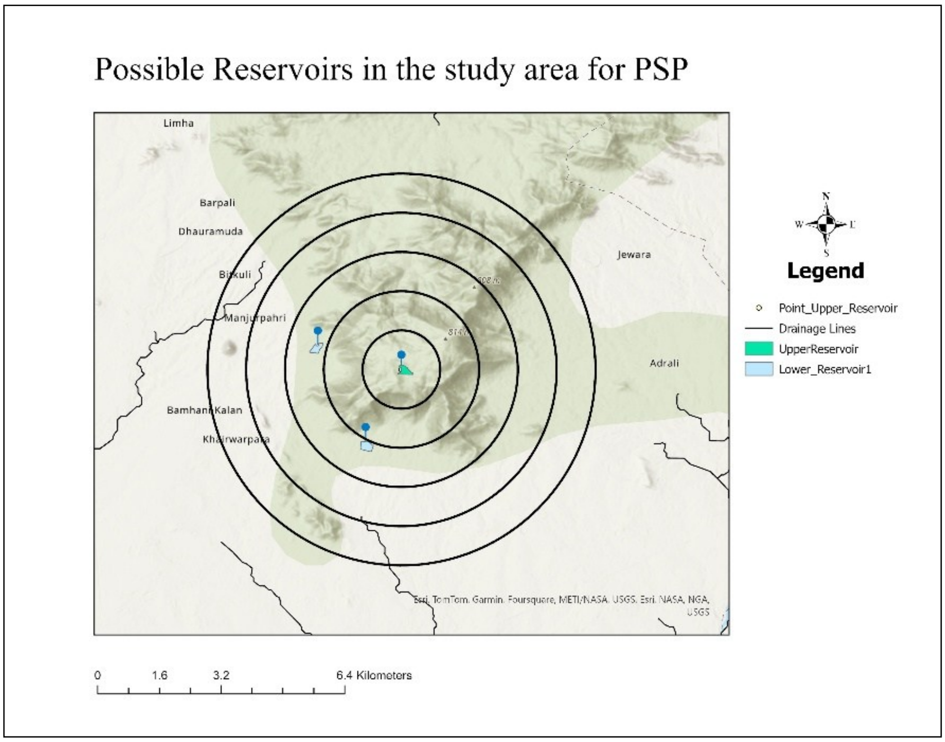

Figure 7: Possible reservoirs located other view

The volume of the reservoirs can also be calculated to further check the capacity as well as the water requirements for power generation.

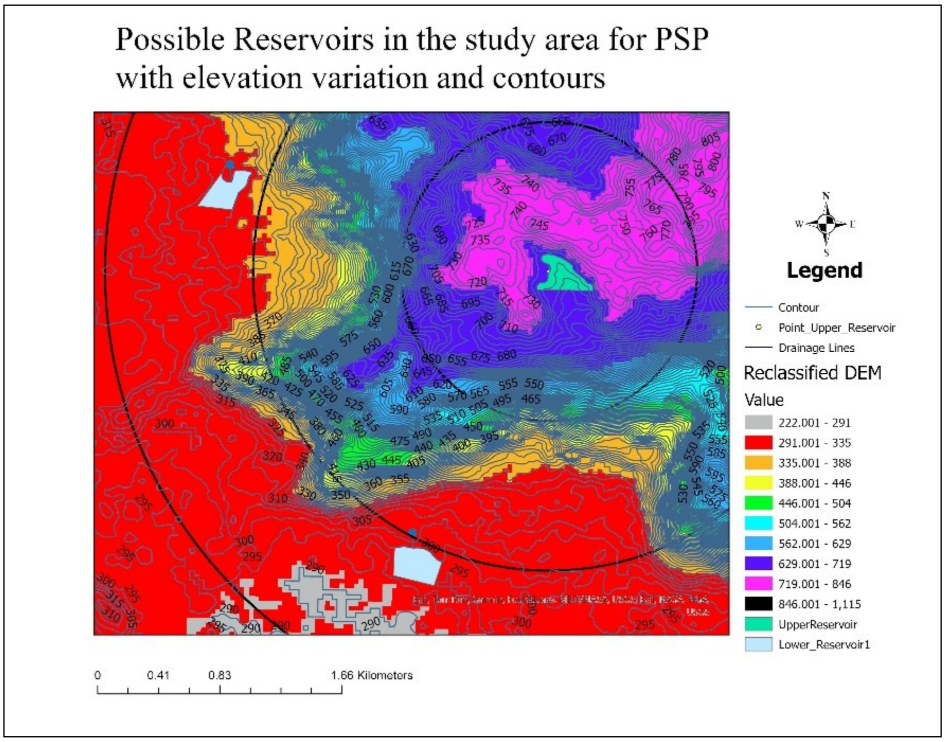

Figure 8: Possible reservoirs located with contours

4 DISCUSSION

In the method above the upper reservoir has been selected as the centre of the circle and concentric circles of radius 1000 m to up to 5000m have been drawn with a difference of 1000 m. In the study area the possible areas with desired elevation difference and optimum length were considered and the options have been presented for further study and investigation to figure out the main choice for the alignment of the WCS.

In addition to that the length of the dam, sufficient storage etc. should also be taken into consideration while choosing the alternatives.

5 DAM SAFETY ASPECTS IN PUMPED STORAGE PROJECTS

Dam safety in pumped storage projects (PSPs) is a critical aspect that must be rigorously evaluated to ensure the long-term stability and reliability of the infrastructure. The fluctuating water levels inherent in PSP operations, which involve water being pumped between upper and lower reservoirs, impose dynamic stresses on the dam structure and associated water conductor systems.

Key safety considerations include:

- **Structural Integrity:** Regular assessments of the dam's physical condition to prevent failures.
- **Spillway Capacity:** Ensuring that spillways can handle extreme inflows to prevent overtopping.
- **Seepage Management:** Monitoring and managing seepage to maintain dam stability.

Furthermore, the operation of PSPs demands careful monitoring of hydrological conditions, particularly with respect to rapid changes in water levels, which could affect the foundation of the dam or cause erosion.

The presence of geotechnical hazards, such as weak foundation materials or seismic risks, also warrants close attention. Regular safety inspections, real-time monitoring, and the implementation of contingency plans for emergency scenarios are essential to mitigating risks. Overall, maintaining dam safety in PSPs requires an integrated approach that combines robust design, continuous monitoring, and adaptive management to address the unique operational challenges posed by these systems.

Dam safety is a critical consideration in the design, construction, and operation of pumped storage projects, which are subject to various risk factors.

5.1 DAM SAFETY ASPECTS IN OFF STREAM PSP

In the case of an off-stream pumped storage project, the primary safety concerns involve the construction of an artificial reservoir, often located away from existing water sources, and the integrity of the water conductor system.

Key safety aspects include the stability of the embankments, the design of spillways to handle extreme inflow conditions, and the management of seepage to prevent embankment failure or erosion. Additionally, robust monitoring systems for structural deformations, reservoir water levels, and real-time hydrological data are essential to mitigate any unforeseen risks.

5.2 DAM SAFETY ASPECTS WITH EXISTING RESERVOIR IN PSP

For a pumped storage project with an existing reservoir, safety considerations extend to the condition of the pre-existing dam structure. These include assessing the structural integrity of the dam, the adequacy of the spillway capacity, and the potential impact of fluctuating water levels on the stability of the reservoir's embankments.

The operation of the pumped storage system must be managed to avoid excessive pressure or stress on the existing dam infrastructure, ensuring that rapid water level fluctuations do not compromise the safety of the reservoir.

Finally, when evaluating existing reservoirs, several key factors must be examined including the condition of the dam's foundation, the presence of cracks or seepage in the dam body, and the performance of spillway and sluice systems.

The structural health of the dam, particularly in the context of aging infrastructure, should be rigorously monitored, and periodic safety inspections must assess the adequacy of seismic stability, the functionality of emergency action plans, and the resilience of the dam under potential extreme hydrological events.

Continuous evaluation of these aspects is crucial to ensuring the long-term safety and reliability of both off-stream and existing pumped storage projects.

6 CONCLUSION

The methodology discussed in this paper can be used to identify potential sites for Pumped Storage Projects (PSPs), complementing existing methods and reducing the time and effort involved in identifying these locations. Along with identifying sites, it can also be applied to determine alternative layouts for the water conductor systems (WCS) of PSPs.

However, when identifying possible sites for PSPs several other factors must also be considered beyond just selecting the reservoir location. These include accessibility to the sites, distance from major roads, proximity to the electrical grid, area submergence, environmental factors such as forest presence, interstate considerations, and issues related to resettlement and rehabilitation (R&R). Once these factors are satisfied, the site would be most suitable for a PSP.

Additionally, dam safety is a critical aspect that must be rigorously evaluated during the site identification process. This includes assessing the structural integrity of the dam, the spillway capacity to manage extreme inflows, the potential for seepage, and the dynamic stresses imposed by fluctuating water levels.

The design, construction, and operation of PSPs must address these safety aspects to ensure long-term stability and reliability. For both off-stream and existing reservoir PSPs, it is vital to include robust monitoring systems for hydrological conditions and structural integrity, and to ensure the dam's foundation is secure, particularly in the context of aging infrastructure. Incorporating these dam safety considerations into the site selection and design process will significantly mitigate risks and enhance the safety of the entire pumped storage system.

All these criteria once fulfilled and satisfied will be the best suited site for the PSP.

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8 KEYWORDS

1. Geographical Information System (GIS)

Geographical Information System (GIS) is a computer-based tool used to collect, analyze, and visualize geographical and spatial data. It helps in understanding relationships, patterns, and trends by overlaying multiple layers of data on maps. For example, GIS can be used to map flood-prone areas or plan efficient routes for infrastructure projects.

2. Pumped Storage Power

Pumped storage power is a type of hydropower technology used to store energy. It works like a rechargeable battery: during periods of low electricity demand, water is pumped from a lower reservoir to an upper reservoir. When electricity demand is high, the stored water is released to flow back down through turbines, generating power. It's a key technology for balancing the grid and integrating renewable energy sources.

3. Hydropower

Hydropower, or hydroelectric power, is energy generated by harnessing the movement of water, typically using a dam and turbines. It is a renewable energy source that relies on the water cycle, including rain and river flow. Hydropower is widely used for electricity generation and is known for its reliability and low greenhouse gas emissions.

4. Digital Elevation Model (DEM)

A Digital Elevation Model (DEM) is a 3D representation of the Earth's surface, showing elevation information. It is commonly used in mapping, hydrology, and infrastructure planning. DEMs help identify slopes, valleys, and water flow paths, making them essential for flood modeling, terrain analysis, and designing water systems.

5. Water Conductor System

A water conductor system refers to the network of structures that transport water from a source to a hydropower plant or other utility. This system often includes components like canals, tunnels, pipelines, and penstocks. In hydropower projects, it ensures the efficient transfer of water to the turbines for power generation.

6. Renewable Energy

Renewable energy is energy derived from natural sources that are replenished over time, such as sunlight, wind, water, and biomass. Unlike fossil fuels, renewable energy produces minimal environmental impact and helps combat climate change. Popular examples include solar power, wind power, and hydropower.

7. Grid Balancing

Grid balancing is the process of ensuring a steady supply of electricity to meet demand. Since renewable energy sources like wind and solar are variable, grid balancing ensures there is enough power at all times by adjusting supply and demand or using storage solutions like batteries and pumped storage systems. This maintains the stability and reliability of the electricity grid.

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Re-energizing Power Plants through Renovation, modernization, and life extension of hydroelectric projects

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ABSTRACT:

With increasing awareness of global warming, there has been a strong need to scale up energy with new hydropower projects & optimize the current generation capacity by rehabilitating existing facilities. Renovation & Modernization (R&M)/Upgradation works is a continuous and cost-effective option for optimization of energy resources through improved efficiency, better availability and capacity augmentation by resorting to modernization with state of art technology. This extends the life of the power station for about 25 years, involving Residual Life Assessment (RLA) studies and retrofitting new uprated machine in the existing space, repairing/replacing worn-out or damaged components, upgrading generation capability/availability, etc. One of the earliest projects of NHPC Limited, namely, Loktak Power station, Manipur (105 MW), commissioned in 1983 have undergone/ is undergoing R&M works, after seeking necessary approvals as per CERC norms. This paper describes various aspects of R&M and life extension works undertaken for various civil components along with hydro-mechanical and electromechanical works.

10. INTRODUCTION

Renovation, Modernization & Life Extension (RM & LE) of old power stations aims for selective replacement of components & equipment which can lead to increase in efficiency, peak power and energy availability apart from giving a new lease on life to the power plant. Normally, civil assets are subjected to less wear and tear in comparison to electro-mechanical components. In design of hydro-electric projects, the life of the civil works is considered more than 100 years, while the normative operating life of electro-mechanical generating equipments is considered as 35-40 years. Over the period of operation, due to wear & tear, ageing of plant infrastructure and technological obsolescence, RM & LE becomes necessary. The various aspects of R&M and life extension works undertaken for various civil components along with hydro-mechanical and electro-mechanical works in Loktak Power station, Manipur are discussed further.

10.1 Project History

Loktak Power Station (3x35 MW), situated at a distance of 35 Km from Imphal, Manipur is an integral part of Loktak Lake Multipurpose Project of Government of Manipur. The Power Station is in commercial operation since 1st June, 1983 & is providing valuable power to the North Eastern region of India.

10.2 Brief of Loktak Power Station

Loktak Power Station envisages diversion of 58.8 cumec of water from natural reservoir (Loktak Lake) to Leimatak valley with a view to utilize 42 cumec for generation of power and supply 16.8 cumec for lift irrigation. It comprises of 10.7m high & 58.8m length barrage at Ithai, 2.33 km long power channel, 6.89 km long 3.81m dia horse shoe HRT and a surface power house of 105 MW installed capacity. The annual design energy from the power station is 448 MU.

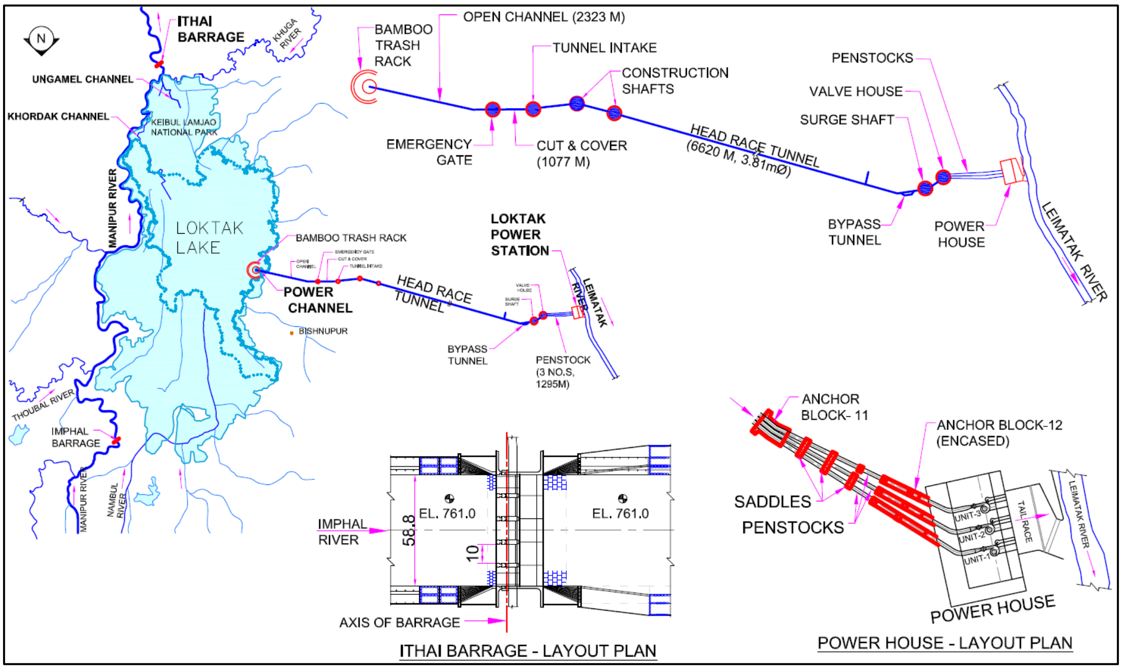


Figure-1: Project Layout Plan of Loktak Power Station.

11. RENOVATION AND MODERNIZATION PROPOSAL

Loktak power station completed its useful life in June 2018 based on criteria of 35 years of useful life as per Central Electricity Regulation Commission (CERC) Tariff Regulation 2014. Accordingly, residual life assessment studies have been carried out and R&M proposal has been planned to extend the life of power station to 25 years with improved reliability and availability. The planning of R&M works has been done in-house by NHPC with its own available technical expertise and experience.

The DPR of R&M proposal for life extension have been examined & vetted by CEA on 06.04.2018 for Rs. 236.07 Cr at Sep-2017 price level. The complete R&M works is divided in 12 packages comprising of 5 Civil, 1 HM, 4 E&M, 1 miscellaneous civil works and 1 infra-structure works packages.

12. ASSESSING HEALTH OF EXITING STRUCTURE AND TESTS CONDUCTED

In order to assess the health of Civil & Hydro-mechanical components, an in-house inspection team was constituted including officers from Civil & Hydro-mechanical design to carry out detailed visual inspection of all the components of power station.

As per the visual inspection, no major deterioration /weakness in major civil structures were observed and in general they are in good health. However Non- Destructive & Semi Destructive tests were carried out in Ithai Barrage, Bye pass area, Surge shaft, Valve House, Penstock, Power House, consisting of

- Rebound Hammer Test (RHT) for surface hardness
- Ultra-Sonic Pulse Velocity test (UPVT) for quality of concrete
- Cover Meter Test (CMT) to determine the cover to reinforcement
- Core test for compressive strength
- Thickness test for Penstock liner and Gates

Results of tests conducted revealed that the surface hardness, quality and condition of concrete are satisfactory and with minor repair/ modification works, these structures can further be used for a sufficiently long period in future.

13. DESCRIPTION OF STRUCTURE, OBSERVATIONS DURING INSPECTION AND R&M PROPOSAL FOR VARIOUS CIVIL COMPONENTS

13.1 Barrage Complex

13.1.1 Ithai Barrage

13.1 Ithai Barrage has been constructed on Imphal River with 10.70m height (above river bed level), 58.8m width (5 spans of 10m each) having a discharging capacity of 566 cumec.

13.2 Observation during inspection

- Damages observed in the top surface of concrete in spillway glacis/ stilling basin due to
- Deep scouring in downstream of stilling basin and damage in left wing wall
- Pitting observed on the surface of piers & abutments.

13.3 R&M Proposal

The Renovation & Modernization proposal for civil works at Barrage area are:

- Erosion damages on the surface of piers, abutments, gate grooves & bridge to be repaired with cementitious mortar conforming to EN1504-3-R4.
- Glacis /stilling basin damages to be repaired with HPC (M60/A20 grade).
- Underwater inspection to be carried out & repair of components of stoplog arrangements, apron, piers, gate grooves & abutments with epoxy mortar.
- To prevent frequent under-scouring & damages of the downstream left-wing wall, it is proposed to replace it with new structure comprising of toe wall & RCC panels. Further, cast-in-situ concrete blocks (2.5x2.5x1.25m) in a stretch of about 15m throughout the width of the river along with boulder wire crates proposed to be provided along the entire length of toe as additional safety measures.
- To facilitate centralized operation of barrage gates, it is proposed to construct a prefabricated control room.

Figure-3-: Ongoing repair work in stilling basin

Figure-4: Ongoing work in downstream of barrage

13.1.2 Bamboo Trash Rack

The Loktak Lake is observed to be infested with phumdies/ acquatic weeds. To restrict the entry of phumdies in to the power channel, 5 rows of bamboo trash rack of varying length (200-1000m) have been provided. Damaged bamboo trash racks were replaced/ repaired

Figure-5: View of the Bamboo trash rack at Power channel

13.1.3 Bye Pass and Penstock Area

The water conductor system is 10.3km long comprising of 2.33km long open channel, 1.07km long cut and cover portion, 6.89km long Head Race tunnel (3.81m dia), 3 no.s penstocks of 1.3km each 2.286m dia,) leading to a surface power house at Leimatek.

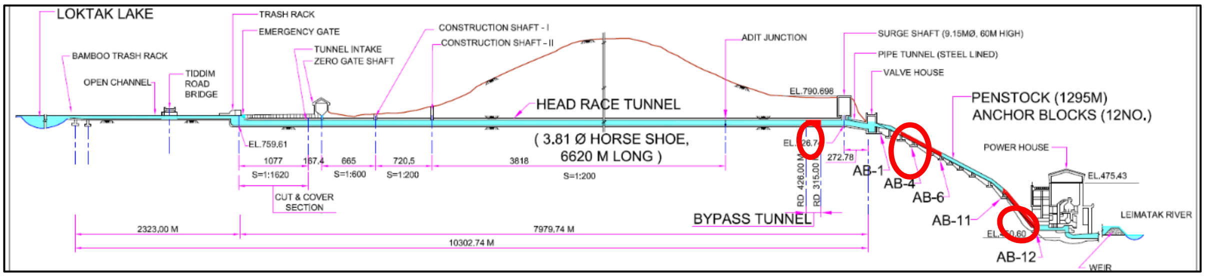


Figure-6: Longitudinal section of water conducting system

The 3 penstocks take-off from the valve house to the power house, supported on 12 anchor blocks (AB) and 68 cement concrete saddles. AB-1 forms part of valve house and AB-12 forms part of Power House. Major stability problems were faced in the erection of penstock lines in between AB- 4 to 6 and 11 to 12 due to steep slopes and geological conditions. Further, in 1985, due to reactivation of old slide, upheaval as observed in the downstream area of AB-11.

Also, a section of the HRT (±35m) had got damaged due to heavy landslides and hill movements in the low cover zone during July, 1983 monsoon. To circumvent the damaged portion, a bye pass tunnel from RD 315m to 426m (from surge shaft) with a lateral deviation of 25m & 2.9m dia. circular shape was constructed.

Since then, the bypass tunnel and penstock area had remained vulnerable from slope stability point of view. Various slope protection measures like installation of anchorage cum drainage shafts in byepass area, 12m deep shafts below saddles supports of penstock between AB-4 to 5 and 11 to 12, encasing of about 50% of penstock length between AB11-12 in concrete were provided. 2 pile groups of 500mm dia. RCC cast-in-situ bored piles were also provided on the right side of penstock lines to reduce movements between AB-11&12.

13.4 *Observation during inspection*

- Existing bored piles were found severely damaged and the junction of pile & pile cap were crushed at many locations exposing the reinforcement.
- Encased portion of penstock-1 was covered with slided overburden material
- Tilting/dislodging of rockers and cracks/subsidence over slope
- Inclinometers installed in this area were showing movement and later got damaged.
- Minor deterioration in the existing anchorage cum drainage shafts in bye-pass area.
- The drainage system was also found to be damaged/ non-functional.

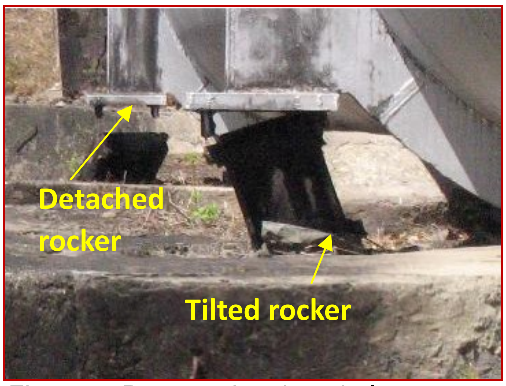
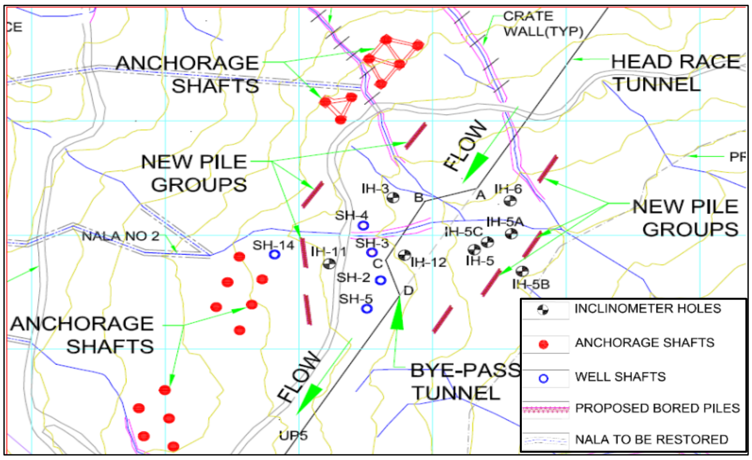


Figure-7: Damaged rockers below penstock

Figure-8: Damaged existing piles

13.5 R&M Proposal

- To improve the stability of the hill slopes, it is proposed to provide 500mm dia. RCC cast-in-situ vertical bored piles, upto 30m deep (7 no. groups with 26 piles in each group) in the bye pass tunnel & penstock area. These piles shall be interconnected at the top with caps and tie beams arrangement to form as a rigid structure.
- Repair of existing bored piles with cementitious mortar conforming to EN1504-3-R4.
- Repair of anchorage cum drainage/ well shafts with cement concrete/ mortar along with restoration of existing shafts by re-drilling of choked drainage holes and provide submersible pumps for regular dewatering of the well shafts.
- Restoration of the drainage network and other slope protection measures like rock trenches, contour drains, nallahs, culverts, etc., in the entire area (wherever required).
- Extensive monitoring mechanism comprising of inclinometer for gauging sub-surface movement and survey target points/ topographical observation stations for ascertaining the surface movement to be established for keeping constant vigil of the area & also to check the efficiency of the provided rehabilitation measures.



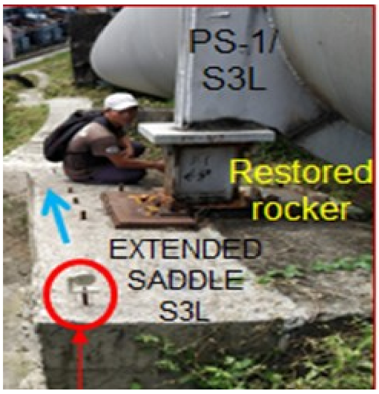


Figure-9: Slope stabilization measures with in Bye-pass area

Figure-10: Extended saddle for supporting rockers

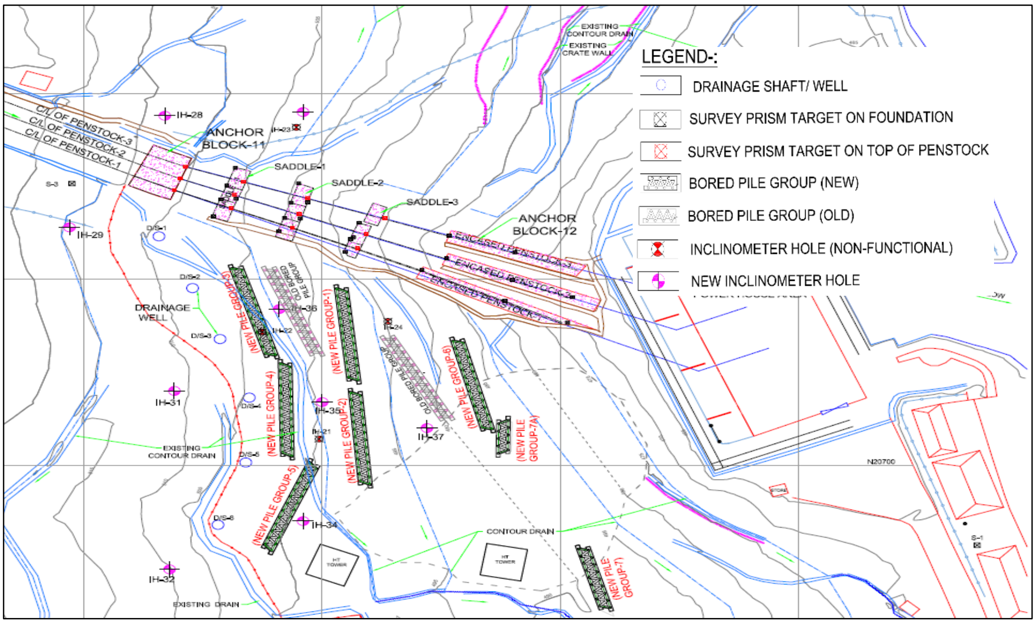


Figure-11: Layout plan of penstock AB-11 to 12 showing the proposed pile groups

13.1.4 Power House

A surface Power House (61m (L) X 15.5m (B) X 34m (H)) with three units of 35MW is located on the right bank of river Leimatak.

13.6 Observation during inspection

- Floors & walls were found deteriorated due to continuous wear & tear.
- Minor seepage observed inside the Power House Building.
- Minor erosion on piers of Draft Tube/ PRV and counterfort wall were observed.
- Concreted boulder wire crate walls found damaged in the tail pool area at some locations.
- Pressure relief holes provided in the upstream slope of power house were found choked.

13.7 *R&M Proposal*

- Repair of surface of draft tube, PRV piers, counterfort wall, edges of slab & beams with cementitious mortar conforming to EN1504-3-R4.
- Face lift of the power house for providing better aesthetics & working conditions
- Architectural works such as replacement of false ceiling, repair/ replacement of floors.
- Repair/ restoration of concreted boulder crate walls in the tail pool area.
- Re-drilling of choked pressure relief holes.

14. R&M WORKS FOR OTHER COMPONENTS

14.1 Hydromechanical components

Hydro-mechanical issues like difficulty in lowering of barrage gates, distortion of skin plate, dislodgement of wheels, etc. have been experienced for which repair & renovation works are required at various locations. Repair & restoration of embedded parts of barrage gates, strengthening of hoist support structure, trash rack panels, treatment/filling of deep scratches, restoration of misaligned saddle supports of penstocks, etc. are proposed. Further, supply, erection, testing & commissioning of barrage gates, barrage stoplog gantry with automatic engaging/ disengaging lifting beam is to be carried out. Dedicated rope drum hoist for each of the draft tube and pressure relief valve gates is also proposed to be introduced for prompt isolation of the powerhouse to guard against possibility of flooding exigency.

14.2 Electro-mechanical components

In order to extend the life of and sustain generation and machine availability, major electro-mechanical equipment of power station like turbine, generator, governors, GSU transformer, excitation system, auxiliary system like DC system, LT switchgear, auxiliary transformers, cooling water system, etc. are proposed to be replaced/ refurbished as per the requirement.

15. CONCLUSION

Renovation and Modernization works of Loktak Power Station is under progress and is expected to be completed soon and will yield considerable improvements in output, efficiency, reliability and availability.

Hydropower has and continues to make an essential contribution to increasing energy needs, boosting prosperity and meeting climate targets. In the present scenario of severe resource constraint, renovation and modernization (R&M) of power plants of existing old hydroelectric power plants is considered very important to utilize current capacity effectively and efficiently and to bridge the gap between the demand and supply of power as R&M schemes are cost effective and have lesser gestation period quicker than the setting up new power plants

14 REFERENCES

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• CERC Tariff Regulations 15(1), 2014

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Innovative Technologies in Dam Safety and Rehabilitation of Tanakpur Barrage-a case study

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ABSTRACT:

The Tanakpur Power Station, commissioned in 1992 on the Sharda River in the Himalayan foothills, was designed to handle a flood 7.0 lakh cusec. The project includes a 475m-long barrage, afflux bunds, power channel, forebay, powerhouse, and tailrace channel. Over the years, frequent floods led to sedimentation, erosion, and reservoir capacity loss despite scheduled flushing. In June 2013, an unprecedented flood of 5.35 lakh cusec caused heavy sediment deposition in the reservoir's central portion and increased flow along the afflux bunds, leading to severe scouring and structural damage. To tackle the issue & minimize repair frequency, innovative measures were implemented, including central channel activation, polypropylene rope gabions, hexagonal wire mesh gabions, tetrapods, updated reservoir operation guidelines, and spurs.

Structural repairs with epoxy (ASTM C-881), cementitious mortar (EN 1504-3 R4), HPC (M65A20), and large cement concrete blocks significantly reduced damage frequency. This was evident when Power Station successfully mitigated ±5.0 lakh cusec floods in 2021 and 2024. As water and energy demands grow, use of modern technologies in survey, design, construction, and O&M practices are crucial. Standardizing advanced materials and repair techniques has not only enhanced Tanakpur's resilience but also benefited other power stations.

1. INTRODUCTION

The Tanakpur Power Station in district Champawat, Uttarakhand, is a 94.2 MW run-of-river project generating 460 MU annually. It features a 475.3m-long barrage on the Sharda River (FPL 246.70m) with 13 weir bays (EL 238.10m) and 9 under sluice bays (EL 237.50m, 18.30m wide). A 78.45m head regulator on the right diverts 566 cumecs through a 6.4 km power channel to a surface powerhouse with three 31.4 MW Kaplan units (21m design head). The 1100m tailrace channel discharges back into the Sharda River upstream of Banbassa Barrage.

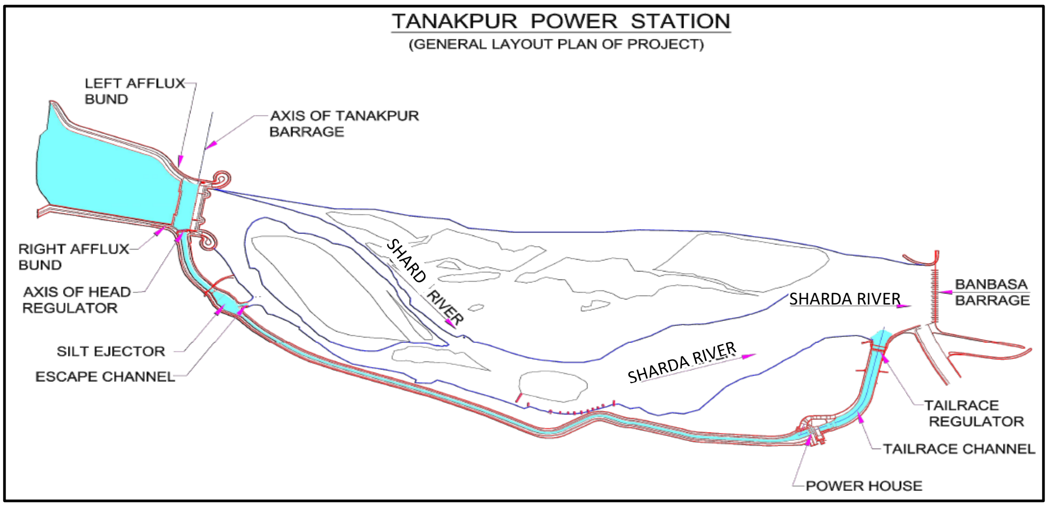

Figure 1. General Layout of Tanakpur Power Station

1. REHABILITATION OF RESERVOIR

1.1 *Activation of Central Channel*

Over the years, siltation and river aggradation at Tanakpur Barrage led to a central island formation in the reservoir. Severe floods in 2013 (5.35 lakh cusec) and 2014 (>3 lakh cusec) worsened sediment deposition and flow concentration toward the afflux bunds, raising safety concerns. As flushing was ineffective, a 700m-long, 50-80m wide central channel was excavated in 2014-15 to divert flow and stabilize the bunds. Further widening of central island (80m-160m) between RD 900-2100m was completed in 2015-16, and flushing increased from 4 to 6 times annually. These measures had significantly improved reservoir flow conditions and increasing its capacity.

1.2 *Updation of reservoir and silt flushing guidelines*

To enhance flushing efficiency while ensuring structural safety, the Reservoir Operation and Silt Flushing Guidelines were revised and issued in April 2016.During power generation, river discharge up to 24,000 cusec (680 m^3/s) is routed through the head regulator, while excess flow is managed via barrage gates.

14.1 Reservoir flushing during monsoon shall be carried out as per following guidelines to avoid further deposition of sediment:

Table-1 Flushing operation regime

Month	Minimum River inflow	Minimum no. of reservoir flushing
June	35000 cusec (991 m^3/s)	One time
July	90000 cusec (2549 m^3/s)	Two times
August	90000 cusec (2549 m^3/s)	Two times
Septembe r	90000 cusec (2549 m^3/s	One time

If the requisite minimum discharge does not reach during the month, the flushing shall be carried out within last two days of the month. Drawdown flushing upto 125,000 cusec (3540 m3/s) shall preferably be carried out at minimum possible elevation in free flow condition. However, if the condition of the afflux bund does not permit free flow condition even below 125,000 cusec (3540 m3/s), flushing should be converted to sluicing at lower discharge keeping the reservoir level at EL 245.2m or above. The minimum period between two successive flushing has been worked out as 10 days depending upon long term data. When silt concentration at d/s of Head regulator gates in power channel is more than 5000 ppm or river discharge is more than 150,000 cusec (4248 m3/s), power generation shall be stopped & Head regulator gates shall be lowered and flushing be carried out. The flushing should continue till silt concentration in the reservoir U/S of barrage is equal to silt concentration in the D/S of barrage but for a minimum period of 12 hrs.

Barrage Gate no. 1 shall be operated regularly at least twice daily during monsoon and every 30 days in the lean season to flush silt excluder tunnels. If no spillover occurs in the lean season, it shall be opened up to 1m monthly or as directed by the Engineer-In-Charge. Figure-2 shows the reservoir capacity and the maximum flood experienced over the years.

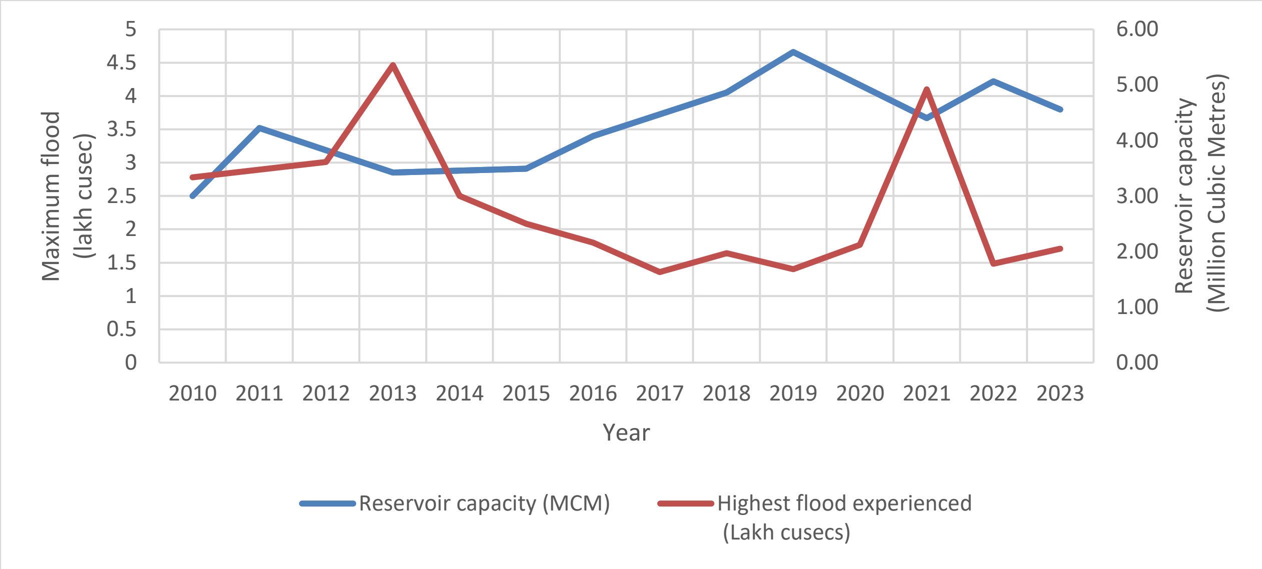

Figure 2. Plot between maximum flood and reservoir capacity over the years.

2. REPAIR & RESTORATION OF SPILLWAY GLACIS & STILLING BASIN

Hydraulic structures in Himalayan rivers require frequent repairs due to cavitation, abrasion, and impact. The spillway glacis, stilling basin, and end sills, made of concrete and granite stone, experience face erosion from high floods (max 5.35 lakh cusec/15,150 m^3/s) and annual flushing.

2.1 Repair of Spillway

Repair of the spillway glacis, as given below, depends on the extent of the damages:

- If the depth of erosion in the joints of granite stones is less than 30mm, the repair is carried out with epoxy mortar, conforming to ASTM C881 (Type-IV).
- For more than 30mm depth of erosion, repair below 30mm depth is carried out with cementitious mortar conforming to EN1504-3 (R4) and then top 30mm depth is carried out with epoxy mortar.
- Cavities upto 100mm depth can be filled with 2-3 layers of cementitious mortar.
- Cavities more than 100mm depth is filled with High Performance Concrete bonded with epoxy complying with ASTM C881 (Type-II grade 2).

2.2 *Repair of Stilling Basin*

Repairs with HPC involve a 500mm wearing layer over the base concrete, anchored with 25mm Fe500 steel bars (1200mm long) fixed with epoxy grout at 1m spacing. Anchors penetrate 500mm into old concrete, bent 90° downstream, with a 50-200mm clearance. The epoxy grout has a 10 MPa bond strength (ASTM C882). HPC repairs have proven effective, extending maintenance intervals to 7-10 years.

Figure 3. Repair done in spillway glacis

Figure 4. Ongoing repair work in stilling basin area

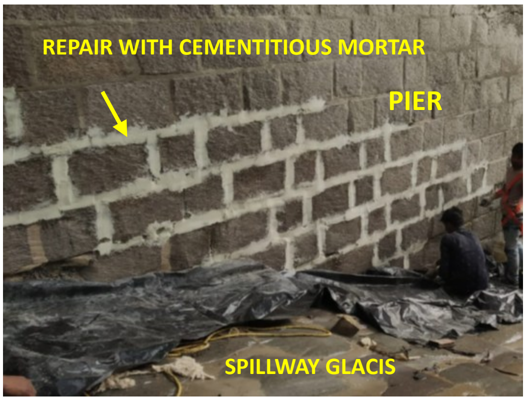

Figure 5. Repair work in Spillway pier

Figure 6. Repair of Launching apron with CC blocks

2.3 *Repair of Spillway Piers*

Spillway piers are subjected to erosions mainly in lower reaches which varies from few to several ‘centimeters’ causing reduction in the concrete cover or exposure of reinforcement at places. Cementitious mortars conforming to EN1504-3(R4) are preferred to repair such damages because the physical & mechanical properties of these mortars are similar to the parent concrete. In such repair, bonding of cementitious mortar with the old concrete is very much important. As per EN: 1504-3(R4), minimum adhesive bond strength of 2 MPa has been specified. Due to this property, the repaired layer becomes integral part of the structure and the failure would not take place at the joint. Layer thickness from 12.5mm to 50mm with cavities upto 100mm has been repaired and performance is found satisfactory.

14.2 *2.4 Repair of Launching Apron*

The launching apron beyond the stilling basin’s end sills, originally made of concrete blocks over boulders, suffered damage due to scouring and cavity formation. Repair work using PP rope gabions and tetrapods proved ineffective during high floods. Accordingly large M15 concrete blocks (2.5m × 2.5m × 1.5m) were installed being tied together, which effectively reducing damage. Repair intervals have now extended to over five years.

14.3 *2.5 Restoration of warp wall at right guide bund*

Three blocks of the warp wall (RD 61.9m–103m) downstream of the right abutment were constructed as part of the guide bunds, with a concrete floor up to RD 61.9m and a wire crate-cement block floor beyond it. Since 2006, the warp wall showed settlement and distress, worsening in 2014 when blocks suffered 1-2m settlement and cavity formation due to frequent gate operations. Repairs were challenging due to proximity to Barrage Gate No. 1, which operates continuously for silt flushing. The damaged warp wall was repurposed as a gravity retaining wall instead of being dismantled. The repair process involved excavation, cavity filling with M20A40 concrete for structural reinforcement, and backfilling with muck/RBM to ensure stability.

14.4 ***2.6 River training in downstream reaches using spurs***

In the downstream reaches of the Tanakpur Barrage, suitable arrangement of deflecting spurs is provided in series at appropriate intervals to train the river along its flow path and arrest any erosion of the banks during floods.

3. REHABILITATION/STRENGHTENING OF AFFLUX BUNDS

Guide cum afflux bunds at Tanakpur Barrage, extending ~2200m on the left and ~2188m on the right, are compacted riverbed material with a 2.5H:1V slope. Tied to high ground in Nepal (left) and Sharda Ghat (right), these bunds have experienced scouring due to siltation-induced flow channel shifts, with velocities reaching 6m/s. Damage assessments led to strengthening with PP rope gabions, hexagonal wire mesh gabions, and tetrapods, significantly reducing damage frequency. This standardized repair approach aligns with CWC's 2018 "Manual for Rehabilitation of Large Dams."

Figure 3. View of damages in Left afflux bund after monsoon 2013 flood

14.5 ***3.3.1 Mechanically woven hexagonal wire mesh gabion***

Earlier the repairs of the embankment were carried out with conventional hand-woven boulder wire mesh gabion, assembled at the site. These gabions easily got damaged once some knots or wire were broken. Now a days, factory-made mechanically woven hexagonal wire mesh gabion conforming to IS:16014-2012 is being preferred.

14.6 ***3.3.2 Polypropylene (PP) rope gabion***

PP rope gabions are flexible, permeable, and corrosion-resistant stone-filled structures made of polypropylene ropes. Their high tensile strength and adaptability allow them to withstand significant deformation without damage. They effectively restrict scour depth at bund toes and can be easily placed or relocated during emergencies. With an underwater lifespan of 8-10 years, they have been successfully used at Tanakpur Power Station since 2014-15.

3.3.3 *Tetrapods*

To overcome the high erosive hydraulic forces along the afflux bunds, small nose spurs of tetrapods at an interval of ±100 m were also provided. The tetrapods have been found very effective in protection of the toe and diversion of the flow.

3.3.4 *Toe wall along with cladding*

Several vulnerable stretches were identified in the left and right afflux bund where heavy scouring/damages were observed during the floods. These stretches were required to be rehabilitated with robust arrangement which shall be effective in withstanding high velocities during floods. Accordingly, protection measures with concrete toe wall were provided. Further, as per site requirements a launching apron in front of the toe wall using boulder wire crates was also provided. Concrete cladding is then provided along the riverside slope of the bunds laid over boulder wire crate for slope protection. In the subsequent monsoons, no further damages were observed in these areas and performance of the restoration works has been found satisfactory.

4. CONCLUSION

Study of river behavior before and after the construction of hydraulic structures and design of appropriate river training measures is essentially needed for the safety of the structure. Following conclusions are drawn from above discussions:

1. Reservoir operation and silt flushing guidelines need to be updated from time to time based on actual observation.
2. Vigilant reservoir management & river training works be carefully undertaken with application of appropriate measures in identified vulnerable reaches.
3. The repair/ restoration of the afflux bunds evolved with PP rope gabion/ mechanically woven wire mesh gabion along with tetrapod nosing has greatly reduced the frequency of repair.
4. Use of appropriate materials with standardized performance characteristics laid down with systematic repair guidelines/methodology along with technical specifications can optimize the cost and frequency of repair and enhance safety aspects of hydro structures.
5. Spillway glacis, piers, stilling basin were repaired with epoxies conforming to ASTM C-881, cementitious mortar conforming to EN 1504-3 (R4) & HPC (M65A20) have greatly reduced the repair frequency.

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DESIGN FLOOD REVIEW AND MANAGING REVISED FLOODS

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ABSTRACT

Large Dams not only provides water for Irrigation and Drinking but also plays very important role in mitigation of floods. As per National Register of Large Dams 2023, there are 6281 Large Dams (6138 operational and 143 under construction) in India i.e. third highest after China & USA. In the last few years, there is considerable change in increase in extreme rainfall events i.e. excess rain in short span of time due to cloudbursts & Cyclones etc. Many of these dams are very old and Designs of these dams was done as per prevailing practices/norms at that time and may not have considered large changes in extreme rainfall events due to climate change. Further, due to regular availability of water for irrigation and drinking, there is considerable increase in habitation in Reservoir area as well as in Downstream area which has compounded the Risk in event of extreme rainfall event. In view of above, Review of Design flood of such dams needs to be carried out for hydrological safety of Dam as well as safety of population living in Reservoir and D/S area of the Dam. If the Revised Design Flood is significantly higher than the original design flood, it is crucial to assess the adequacy of the spillway capacity. If the revised design flood hydrograph cannot be routed through the spillway without exceeding the freeboard limits, several alternatives can be considered to make the dam hydrologically safe such as construction of additional spillways, installation of automatic flood control gates, Provision of Breaching Sections or Fuse Plugs, Raising the freeboard above the FRL/MWL by adding parapets, Lowering the conservation storage level to increase flood storage capacity, enhancing flood moderation and the establishment of early warning systems(EWS) etc. The solution adopted will vary depending on the type of Dam and space available in the Project area.

For strengthening of dam safety in India, Central Water Commission (CWC) supported by the World Bank has taken a significant initiative through the Dam Rehabilitation and Improvement Project (DRIP) for Assessment and Rehabilitation of Dams. Design flood values for over 200 reservoirs has been checked and action is being taken wherever required. This paper describes the need of revised design flood estimates, and mitigation measures to safely pass the Revised Design Flood. Case studies of two significant dams - Hirakud and Chandpatha - are provided to illustrate how DRIP has implemented both structural and non-structural measures to enhance dam safety and resilience against higher flood risks. The findings emphasise the need of updating hydrological models to ensure that dams can continue to operate safely in the face of evolving flood risks, especially in light of climate change.

Keywords: *Design Flood, Dam Safety, Inflow Design Flood (IDF), Dam Rehabilitation, Climate Change Impact, Spillway Capacity, Hydrological Safety, Dam Safety Guidelines, Flood Mitigation Measures.*

1. Introduction

The design flood (DF) is the flood used to design a dam and its appurtenant structures such as spillways & outlet works, for determining surcharge storage and the required Dam height. Originally, design floods for dams were based on the hydrological practices and available data of that time, which often relied on limited flood records and rainfall events. With the passage of time, new data sources, including more comprehensive flood records and improved hydrological models, have enabled the refinement of these estimates.

As part of the Dam Rehabilitation and Improvement Project (DRIP), the revised design floods (IDFs) have been calculated for numerous dams, reflecting updated methodologies and better data. These revisions also consider the influence of climate change on rainfall patterns, which can alter the frequency, intensity, and duration of extreme weather events. In the DRIP portfolio, it was observed that, in 32 reservoirs (42 dams, 26 dam projects), the revised IDF was either equal to or lower than the original design flood. However, in 175 reservoirs (approximately 85% of the total 207 reservoirs, 198 dam projects), the revised design floods exceeded the original estimates.

Previously in India, dams were not designed for extreme flood events such as the Standard Project Flood (SPF) or Probable Maximum Flood (PMF). Instead, design floods for these dams were determined using empirical formulae, based on the limited data available at the time of their construction. In the DRIP portfolio, 25 dams followed this empirical approach for determining the design flood.

The failure of Machhu-II dam in 1979, is a classic example of overtopping of the dam due to inadequate spillway capacity of 5663.37 Cumecs, compounded by gate failure resulted into death of more than 2000 persons. Machhu Dam II was ultimately rebuilt with a spillway capacity of 26692.29 Cumecs and was completed in 1989. It is still in operation today. This failure serves as a stark reminder of the potential consequences of underestimating flood risks.

As per National Register of Large Dams 2023, there are 6281 Large Dams (6138 operational and 143 under construction) in India i.e. third highest after China & USA. Given the evolving hydrological landscape and the increasing risks posed by climate change, there is an urgent need to reassess and update design flood estimates for existing dams to ensure their continued safety and resilience.

To standardized procedures in India for selecting inflow design floods (IDFs) for existing dams, the "Guidelines for Selecting and Accommodating Inflow Design Flood for Dams" published by the Central Water Commission (CWC) in 2021 provides a structured approach for determining IDFs for existing dams. This tiered framework is designed to enhance the safety of dams and help ensure that they are adequately prepared to handle future hydrological challenges.

High safety standards for large dams are thus imperative to prevent failure that would cause extensive environmental and property damage, economic hardships, and, in the worst case, loss of life. As per the findings of the International Commission on Large Dams, approximately one third of the failures of dam are attributed to the direct result of flood exceeding the capacity of the dam spillway.

2. Inflow Design Floods

Design Flood (DF) is an important consideration in the safety and design of dams. The estimation of design flood values is integral for ensuring that dams can handle extreme hydrological events without failure. Hydrology is a dynamic science. Original design flood used for planning and construction of an existing dam, used the prevailing hydrological practices of that time, and design storm input of the past. Further, data available at the time of construction, which were often limited and conservative. As new data becomes available and the impacts of climate change become more pronounced, it has become necessary to revise and review the design flood values for both existing and new dams.

The Indian Standard IS 11223:1985 provides guidelines for selecting Inflow Design Floods (IDF) based on the gross capacity of the reservoir and the static head of the dam. The standard categorizes dams into three main classes - small, intermediate, and large and recommends corresponding design floods.

Guidelines as per IS11223, 1985, for different Inflow Design Floods (IDF)

Gross Capacity (MCM)	Static Head (m)	Dam Classification	Recommended IDF
0.5 to 10	7.5 to 12	Small	100-Year Flood
10 to 60	12 to 30	Intermediate	SPF
>60	>30	Large	PMF

While these guidelines are well-defined, they represent a simplified approach to selecting design floods. The SPF is derived from the Standard Project Storm (SPS) and is intended to represent the most severe combination of meteorological and hydrological factors that can reasonably occur. This flood may correspond to a return period of 1,000 years or more. On the other hand, the PMF is based on the Probable Maximum Storm (PMS), which considers the physical upper limit of precipitation and may correspond to a flood with a return period of 10,000 years or higher.

Although SPF and PMF are not probabilistic estimates of the design flood, they provide a useful framework for flood risk management. However, the guidelines in IS 11223 involve substantial jumps in flood probabilities, from the 1-in-100-year return period flood to the 1-in-1,000-year and 1-in-10,000-year return periods, depending on the dam's classification. These significant increases in flood magnitude are based on the hydraulic head (related to dam height) and gross storage capacity of the dam. Notably, the criteria for hydraulic head and gross storage need to be met independently, rather than concurrently, in determining the appropriate design flood for a dam.

While these standards have served as a foundation for the design and safety of dams, the rapidly changing hydrological landscape and the growing influence of climate change have necessitated the revision of these design flood estimates. The Dam Rehabilitation and Improvement Project (DRIP) has led to the recalibration of design flood values for existing dams, taking into account newer data and more refined methodologies. As climate change continues to affect rainfall patterns, the potential for extreme flood events has increased, emphasizing the need for a more comprehensive approach to dam safety.

This paper reviews the existing guidelines and the revised approach to design floods, exploring the implications of updated hydrological data and methodologies. It also addresses the challenges faced by existing dams in accommodating the revised design floods and provides recommendations for improving the resilience and safety of these critical infrastructures.

3. Brief Description of DRIP

The Dam Rehabilitation and Improvement Project (DRIP) is a significant initiative aimed at strengthening dam safety in India, supported by the World Bank. The project was initially implemented in four states: Kerala, Madhya Pradesh (MP), Orissa, and Tamil Nadu (TN) and in the Central Water Commission (CWC). At a later stage, three additional states and organizations - namely Karnataka, Damodar Valley Corporation Ltd., and Uttaranchal Jal Vidyut Nigam Ltd. joined the project, with provisions made for unallocated resources in the project estimate.

The primary goal of DRIP is to rehabilitate existing dams, bringing them up to current safety standards and ensuring that they meet the latest technical and safety requirements. In addition to physical rehabilitation, the project also focuses on institutional strengthening for dam safety management at the state and central levels. The participating entities include the Water Resources Departments and State Electricity Boards of the respective states, who serve as the key implementation agencies for the project.

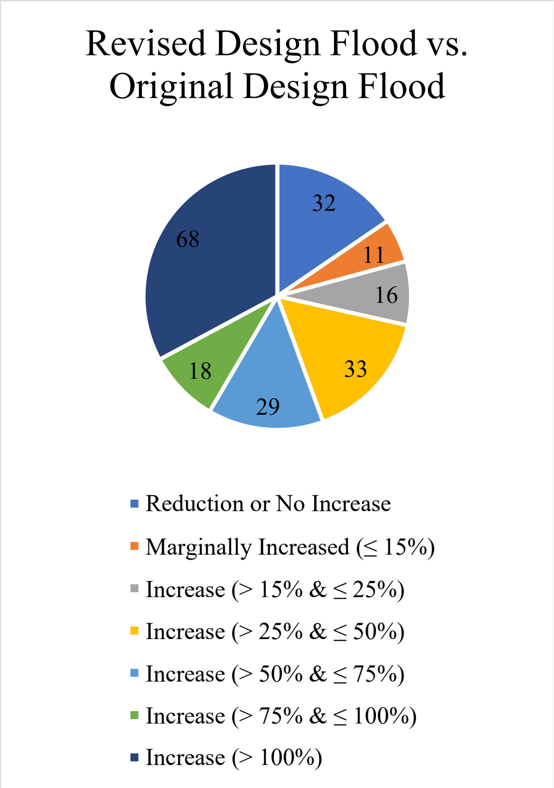
The overall coordination of DRIP is managed by the CWC, with assistance from an Engineering and Management Consulting Firm. DRIP became effective on April 18, 2012, and is set to be implemented over a six-year period.

4. Design Flood Review

The design flood review forms a crucial part of the preparatory phase of dam rehabilitation. Initially, the design flood for 237 reservoirs (253 dams or 228 dam projects) was reviewed and updated. Following this review, 198 dam projects (223 dams) were selected for rehabilitation. As per the current classification in IS: 11223, the category of Inflow Design Flood (IDF) was revised for 48 of the selected dam projects.

The re-estimation of IDF values was carried out using the latest hydrometeorological methodologies. To assist the states in this process, the Central Water Commission (CWC) developed Atlases of Probable Maximum Precipitation (PMP) for the country, employing updated standards. To calculate the design flood, storm depth was derived from either the PMP Atlases prepared by CWC, the Indian Meteorological Department (IMD), or available historical rainfall data. Additionally, the unit hydrograph parameters were obtained from the subzone Flood Estimation reports of CWC or by applying other state-of-the-art methods.

As a result of these updates, the design flood values for the selected IDFs increased in 172 dam projects (181 dams). Out of the 198 selected dam projects (223 dams, 204 reservoirs), the revised IDF included Probable Maximum Flood (PMF) for 74 dam projects (87 dams, 76 reservoirs), Standard Project Flood (SPF) for 109 dam projects (117 dams, 114 reservoirs), and the 100-year return period flood for 15 dam projects (19 dams, 17 reservoirs).



It can also be seen that for 115 reservoirs (56%), the revised design flood increased by more than 50% and for 68 reservoirs (33%), it increased by more than 100% of the original design flood. This shows that periodical review of design flood for existing dams is of paramount importance to mitigate associated risks and ensure hydrological safety.

The rehabilitation measures implemented as part of the DRIP can be broadly classified into five major categories:

- Hydrological Safety: Enhancing the dam's ability to safely accommodate increased design floods.
- Operational Safety: Ensuring that the dam can be operated safely under all conditions.
- Structural Safety: Strengthening the dam structure to withstand various stresses.
- Seepage Control: Reducing seepage to acceptable levels to prevent potential structural damage.

- Basic Dam Safety Facilities: Improving the essential safety infrastructure, including monitoring and warning systems.

These comprehensive measures are aimed at ensuring that dams are able to meet updated safety standards and remain resilient in the face of evolving hydrological challenges.

5. Design Flood Risk Mitigation Measures

Once the revised design flood is determined to be significantly higher than the original design flood, it is crucial to assess the adequacy of the spillway capacity. If the revised design flood hydrograph cannot be routed through the spillway without exceeding the freeboard limits, several alternatives can be considered to make the dam hydrologically safe such as construction of additional spillways, installation of automatic flood control gates, Provision of Breaching Sections or Fuse Plugs, Raising the freeboard above the FRL/MWL by adding parapets, Lowering the conservation storage level to increase flood storage capacity, enhancing flood moderation and the establishment of early warning systems (EWS), etc. The solution adopted will vary depending on the type of Dam and space available in the Project area. The solution adopted will vary depending on the type of Dam and space available in the Project area, and the following alternatives will be explored during the implementation of DRIP:

- Augmenting Spillway Capacity

Adding more spillway bays of the same type as the existing ones to increase the overall spillway capacity.

- Provision of Breaching Sections or Fuse Plugs

If suitable sites are available, breaching sections or fuse plugs can be added. These should ideally be located on a saddle rather than the main dam section to reduce potential damage. The alignment of the surplus channel must be carefully evaluated to assess the likely impact on the surrounding valley in case of a breach during a design flood event.

- Increasing Freeboard Above Full Reservoir Level (FRL) or maximum Water Level (MWL).

Raising the freeboard above the FRL/MWL by adding parapets or strengthening existing sections will increase the available flood cushion and allow the dam to accommodate higher water levels.

- Establishment of Early Warning System (EWS)

Installing an early warning system that utilizes real-time data on rainfall, stage, and discharge from upstream stations. This system will aid in evacuating downstream populations in anticipation of severe flood inflows.

- Increasing Flood Storage

Lowering the conservation storage level to increase flood storage capacity, enhancing flood moderation. While this measure may slightly reduce the benefits of the dam (e.g., power generation or irrigation), it requires minimal investment.

6. Case Studies on Design Flood Mitigation Measures

I. Provision of Construction of Additional Spillway at Hirakud Dam

Hirakud Dam

The Hirakud Dam in Odisha is being rehabilitated under the Dam Rehabilitation and Improvement Project (DRIP). The dam is of high hazard potential, with two important towns, Sambalpur and Burla, situated downstream, along with several critical industries and facilities. A failure of this dam would have catastrophic consequences in the region.

- Existing Spillway Capacity: 42,450 m³/sec
- Revised Design Flood: 69,632 m³/sec (as per the recommendations of the Central Water Commission, CWC)

Given the significant increase in the revised design flood, the current spillway capacity is inadequate to handle the new flood levels. To address this, structural and non-structural measures have been proposed.

- Proposed Mitigation Measure: Construction of an additional Spillway.
- World Bank Agreement: The World Bank has agreed in principle to the first proposal of adding a spillway on the left bank of the main dam (in the saddle on the left of Gandhi Hillock).
- Tendering Process: The Government of Odisha has invited tenders for the construction of this additional spillway.
- This intervention aims to enhance the dam's ability to manage the revised design flood and prevent potential disaster scenarios.

II. Installation of New Automatic Tilting Gate at Chandpatha Dam

Automatic Godbole Gates at Chandpatha dam, MPWRD

Chandpatha Dam, located in Madhya Pradesh and owned by the Madhya Pradesh Water Resources Department (MPWRD), is over 100 years old. Originally, the dam used an ungated weir for floodwater release. However, with the revision of the design flood, the existing infrastructure needed to be upgraded.

- Original Flood Capacity: 424.8 cumec
- Revised Flood Capacity: 1,226 cumec

To accommodate the increased design flood, an innovative solution was implemented under DRIP:

- Mitigation Measure: Installation of 17 Godbole-type automatic tilting gates, each measuring 4.5m (width) x 1.5m (height). These gates operate in dual modes: automatic and manual (via hydraulic cylinder).
- Functionality: These gates automatically adjust to changes in water levels on the upstream side of the dam, ensuring that the outflow from the spillway adjusts to match changes in inflow. The automatic operation eliminates the need for manual intervention, constant monitoring, or electrical power.
- Override Feature: The gates can be manually overridden using a hydraulic hoisting system if needed, allowing for human intervention when necessary.

This solution enhances the dam's ability to safely handle increased flood inflows without requiring continuous human oversight, contributing to both operational and hydrological safety.

These case studies demonstrate the effective implementation of both structural and non-structural measures under the Dam Rehabilitation and Improvement Project (DRIP) to accommodate revised design flood estimates. They underscore efforts to enhance the safety, functionality, and resilience of dams across India, ensuring that critical infrastructure can withstand extreme flood events and continue to serve their intended purposes.

7. CONCLUSION

The Dam Rehabilitation and Improvement Project (DRIP) has been a pivotal initiative in enhancing the safety and operational effectiveness of India’s dams, in response to an evolving hydrological landscape shaped by both natural factors and human influence, including climate change. The paper has not only addressed the historical underestimation of flood risks in many existing dams but also incorporated updated methodologies and advanced hydrological models to reassess and recalibrate design flood (DF) values for dams across India. This ensures that dams are better equipped to handle the extreme flood events of today and those anticipated in the future.

Key outcomes of DRIP have had substantial impact on dam safety and resilience, particularly in light of the growing threat posed by climate change. The paper highlights several key points:

1. Enhanced Spillway Capacity: Many dams, initially designed to handle flood events far less extreme than those now forecasted, have undergone significant upgrades. As demonstrated in the Hirakud Dam case, where the spillway capacity was substantially increased to accommodate the revised Inflow Design Flood (IDF), these interventions ensure that dams can safely manage the heightened flood risks associated with climate change and changing weather patterns. The addition of new spillways or the augmentation of existing spillway bays, such as in Hirakud, has significantly improved the capacity of these dams to cope with future extreme flood events.

2. Advanced Flood Control Systems: The installation of automatic flood control gates, as exemplified by the Chandpatha Dam in Madhya Pradesh, marks a crucial step forward in flood risk management. These gates, operating both automatically and manually, enable the dam to dynamically adjust to fluctuating floodwaters without requiring constant human intervention.

3. Non-Structural Measures: The introduction of Early Warning Systems (EWS) and Emergency Action Plans (EAPs) has bolstered flood preparedness and response, allowing for timely evacuations and disaster management.

4. Climate Change Considerations: Revised estimates reflect the impact of climate change, accounting for changing rainfall patterns and flood frequencies, ensuring dam safety in the long term.

5. Institutional Strengthening: DRIP has helped build the capacity of stakeholders through the development of guidelines and training, enhancing dam safety management at both state and national levels.

6. Comprehensive Risk Management: The combination of structural upgrades and non-structural measures offers a balanced, holistic approach to managing flood risks, ensuring that dams are more resilient to extreme events.

7. Future Resilience and Sustainability: As we look to the future, it is clear that continued investment in infrastructure, technology, and governance will be essential for ensuring the long-term resilience and sustainability of India's dams. The lessons learned from DRIP should be used as a model for future dam safety and rehabilitation projects, ensuring that they are capable of responding to evolving flood risks and climate uncertainties.

In conclusion, the Dam Rehabilitation and Improvement Project (DRIP) has made significant strides in improving the safety, functionality, and resilience of India's dam infrastructure. By updating design flood estimates, enhancing spillway capacity, integrating advanced flood control systems, and strengthening institutional frameworks, DRIP has ensured that India's dams are better prepared to withstand the challenges posed by extreme weather events. These improvements reflect a forward-thinking approach to infrastructure management, emphasizing the importance of continuously adapting to changing hydrological conditions and climate change. The successful implementation of these measures under DRIP sets a critical foundation for the continued safety and sustainability of India's dams and water resource management systems.

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Comoe Dam Lateritic Foundation Treatment

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Abstract

Comoé dam in Burkina Faso is an embankment dam of 24m maximum height above the riverbed and around 1170 m crest length. The dam foundations on the abutments are lateritic ground made of a superficial hard duricrust covering a lateritic carapace, soft to firm layer of 6 to 10 m thickness with canaliculus and cavities. The bedrock is made of highly weathered dolerite at the top, sound and watertight at depth. Since the first impoundment of the dam in 1991, uncontrolled seepage appeared at the downstream toe of the embankment on both abutments, particularly on the left bank. Seepage flow gradually increased with time. In 2010, the situation became critical with development of subsidence downstream in the left abutment. Remedial works launched in 2013 included a positive sheet piles cutoff of 40m maximum depth, tubes-à-manchettes grouting and compaction grouting. Because of the political crisis and economic difficulties, works stopped before completion of compaction grouting and tubes-à-manchettes grouting on the left bank. Although remedial works have not been completed, already a significant reduction in seepage flow and pressures has been observed.

INTRODUCTION

Comoé dam, also known as the Moussodougou dam, is located close to Moussodougou village in the Western part of Burkina Faso upstream of Banfora city, the third largest city of Burkina Faso (fig. 1). The dam, built during the period 1989 to 1991, was commissioned and impounded in July 1991. Comoé dam creates the main reservoir regulating the Comoé river for the development of sugar cane irrigation up to 4 000 ha and water supply to the city of Banfora. The reservoir has a catchment area of 500 Km². Due to its location upstream of Banfora city, Comoé dam is classified as a high hazards dam.

During the first filling of the reservoir, seepage started just at the downstream toe of the dam on both abutments. The seepage flow increased very rapidly during the early years (1991 to 1995). The increase in the monitored seepage flow slowed down but without stabilization. As only a part of the seepage was resurging, the hidden (underground) flow and some emerging seepage on the right abutment and further downstream reach of the river, escaped being measured so that its progress with time could not be assessed.

Figure 1. Dam Location (red star)

Statistical modelling of the seepage flow collected in the left bank up to 2010 is represented in figure 2.

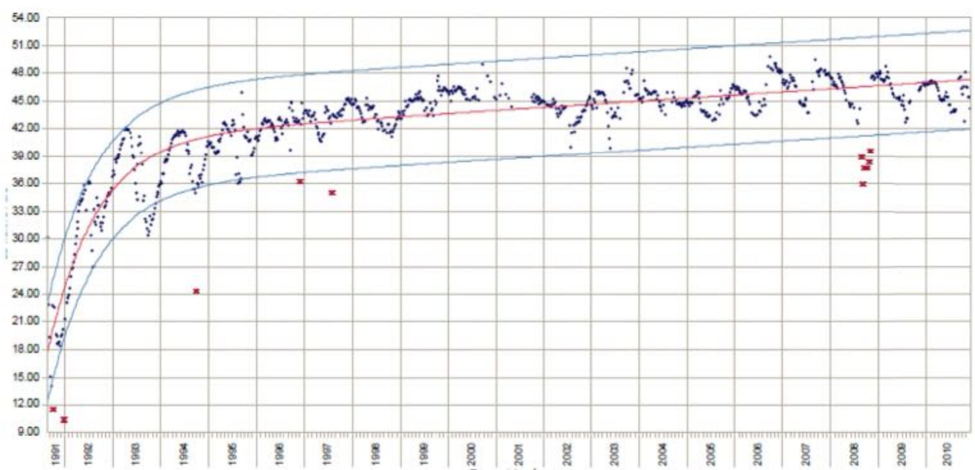


Figure 2. Statistical modelling of the seepage flow on the left bank (correlation coefficient = 0.98), X=years and Y=flow (l/s)

Since 2005, worsening signs of the situation appeared as subsidence spots developed in the ground downstream of the dam on the left abutment. Concurrently, one longitudinal depression of 10 to 30cm depth, locally 50 cm, in line with the emerging seepage zone, was observed on the upstream face of the dam at el. 451, 3m below the normal water level. Except for the upstream, most of depressions' magnitude increased rapidly during wet years particularly under high reservoir level. The situation was therefore considered critical, jeopardizing the dam's safety. It was then decided to limit the maximum reservoir level to 1m below the normal storage level and to carry out necessary investigations to obtain a realistic view of the main flow paths and an objective assessment of the gravity of the situation.

MAIN FEATURES OF THE DAM

Comoé dam is a homogeneous embankment with a vertical chimney filter-drain, having the following main features:

- The dam has 1 170 m crest length, a maximum height of 31 m above the foundation (24m above natural ground) and total volume of 800 000 m^3. The dam crest width is 6 m.
- The main spillway, located in the left abutment, is made of a free overflow concrete gravity structure with an ogee crest and a total length of 115m.
- A service spillway, combined with the outlet work located in the center right of the valley equipped with vertical lift gates with a capacity of 70 m3/s for the 10 years' return period floods management.
- An outlet work in the form of a tower is equipped with an intake for irrigation with an 800mm pipe and reservoir drawdown intake and culvert, ending with a water energy dissipation structure and basin downstream.
- The reservoir capacity at normal water operation level (454 asl) is 38 Mm^3.

The dam body is made of fine clayey lateritic soils in the upstream shell (upstream of the filter) and gravelly lateritic soils downstream. Both materials were obtained from borrow areas close to the dam site.

Figure 3 provides a schematic typical cross section of the dam and the foundation. The top layer of the foundation is made of "duricrust", very hard cemented material with variable thickness generally from 1 to 2m. It overlies the lateritic carapace made of soft to firm clayey soil of 6 to 10 m thickness with canaliculus and vugs, some of them reaching several centimeters' diameter and are of large extent. The bedrock is made of dolerite, sound and watertight at depth, and weathered to highly weathered clay materials on top. It is often not easy to differentiate between lateritic carapace and highly weathered dolerite - not unusual for tropical residual soils' profile.

At the valley bottom, where the ground is permanently saturated, duricrust is missing and the lateritic carapace is thin and free of canaliculus.

The upstream shell is anchored, along the centerline, in the weathered bedrock at the valley bottom and in the carapace at the abutments, where it is in contact with a horizon containing canaliculus which cross the foundation from upstream to downstream. During the works, the foundation treatment by tubes-à-manchettes grouting was limited to the central part of the dam subject to the highest hydraulic gradient. During the excavation of the cut off trench on the left bank localized concentration of vugs and canaliculus were discovered and a very localized attempt of grouting was undertaken.

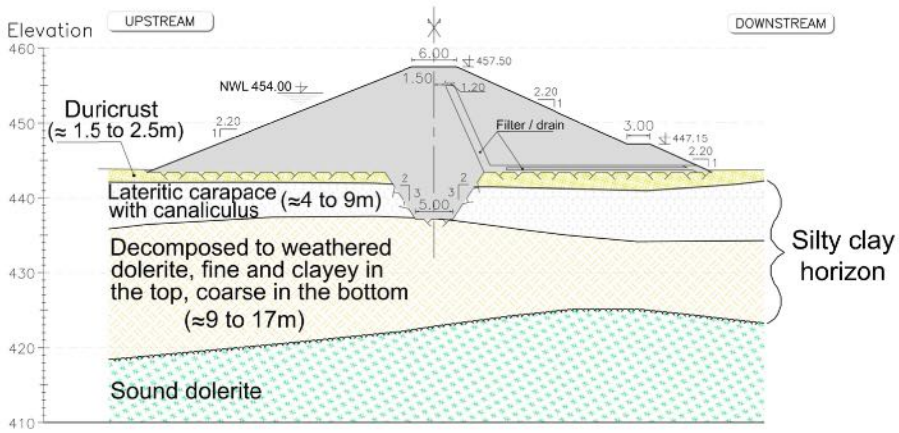


Figure 3. Typical cross section of the dam and foundation

INVESTIGATIONS CARRIED OUT

It was established since the first inspection carried out by the authors that the dam body was sound. The seepage was more likely affecting the foundation in the abutments, particularly the lateritic carapace and, to a lesser extent, the highly-weathered bedrock. This assessment was based on the site visual inspection and monitoring data analysis. The latter was related to pore pressure cells installed in the embankment, relief wells at the downstream toe, standpipe piezometers and seepage flow measurements.

Furthermore, the owner required maintaining the reservoir operation to remain almost unchanged during repair works, as the dam is vital for irrigation and domestic water supply of Banfora city. Emptying the reservoir to carry out, for example, a positive cutoff at the upstream toe, as envisaged at the beginning, was rejected.

Other solutions had to be considered. They had to focus mainly on (1) the treatment of the main flow paths from the dam crest to be in continuity of the cutoff trench and (2) the consolidation of the loose zones in the foundation aiming at preventing any subsidence in the dam itself.

Whatever the final solution, a good knowledge of the conditions in the foundation was necessary. For this purpose, the following investigations program was conducted:

- Boreholes with partial or total recovery and drilling parameters monitoring carried out from the crest and the downstream toe;
- CPTU soundings from the crest and downstream;
- Geophysical survey either by electrical method (performed by Fugro using self-potential method) or magnetic field measurement (Willowstick patented method);
- Laboratory testing;

- Injection of tracers and temperature measurements at different depths in the reservoir and downstream.

In order to minimize disturbance of the dam and prevent any adverse incident while the reservoir remained in normal operation, boring was carried out using steel casing protection of Ø128mm, enabling the passage of a double or triple tube core barrel for undisturbed sampling. Using air as the only drilling medium was specified in order to prevent any hydro-fracturing particularly in the embankment (ICOLD bulletin 158, §7.3). This requirement was respected for core drilling but could not be adhered to when using a tri-blade (a tool developed for destructive drilling), as it clogged. Only water under a gravity head prevented clogging. The embankment became wetter with depth and the foundation was waterlogged as the reservoir was never emptied.

Investigations were carried out in several stages with adjustments as these progressed. At their completion, most boreholes were equipped with standpipe piezometers. Almost two years were necessary to complete these investigations as all of them required mobilization of foreign companies. They were completed in August 2013. Corresponding main findings may be summarized as follows:

1. The embankment of the dam showed nothing unusual except at contact with the foundation occasionally on the left bank. CPTU soundings, conducted from the crest of the dam, showed a tip resistance systematically higher in the dam body (around 5 MPa), compared to the foundation (1 to 2 MPa), except in the valley center where no difference was noticed between embankment and foundation.
2. The foundation is very heterogeneous and comprises mainly of four residual horizons described as follows from top to bottom:
   a. Silty clay, fine red to yellowish becoming sandy downward. A process of laterization is underway at the top of this clayey material
   b. Highly weathered clayey dolerite sandier at the base. This horizon is not continuous.
   c. Grayish dolerite sandy-gritty, fractured and permeable. This horizon is also not continuous.
   d. Dark, sound and watertight dolerite.
3. Both geophysical surveys did not lead to very accurate location of the main seepage paths in the foundation. However, they provide their approximate position, along with the presence of bypassing flow far in the left abutment. All significant seepage paths are located in the silty clay horizon forming the top part of the foundation. Willowstick method allowed deeper investigation and demonstrated the absence of any significant seepage at the valley bottom.
4. On both abutments, some boreholes penetrated the foundation so quickly that drilling advancement was considered as a tool drop. These very loose horizons reached almost 11m height in the left bank. Additional investigations showed that these singularities were of very small extension laterally, not exceeding a few

meters along the dam axis. Their extension in upstream-downstream direction could not be determined.

5. The main geotechnical parameters of remolded silty clay, representing the horizon where the most significant seepage paths were located, are as follows:
   a. Natural water content 35% and dry density 1.4 same range as during the design. There is almost no variation of these parameters with depth.
   b. Dmax = 2mm, 0.08mm content between 70 and 80%.
   c. Plasticity Index and Liquid Limit average respectively 18.5% (16 to 24%) and 53% (45to 64%), corresponding to silty clay moderately plastic.
   d. Undrained cohesion was evaluated between 20 to 60 KPa.
6. Permeability of the silty clay, according to CPTU and constant head tests was estimated to 10^{-7} to 10^{-5} m/s depending on the in situ structure of the materials. It could be higher where this cohesive material has been hydro-fractured, or within the lateritic carapace, rich in canaliculus, vugs and cavities, far in the left abutment. Localized water losses and communications between boreholes were observed. The presence of thin horizons of silty sand to sandy gravel have been identified in the silty clay and may participate in conveying water and fine particles.
7. Subsidence area observed on the upstream slope of the dam around el. 451 is located close to the vertical of the main seepage path highlighted by Willowstick method on the left bank. This might be a sign of erosion that extends to the upstream face of the dam.

REMEDIAL WORKS DESIGN

Remedial works focused on the following targets:

1. Stoppage of the erosion causing the downstream subsidence;
2. Elimination of any risk of subsidence affecting the embankment dam, where it may be the most critical;
3. As a corollary, a significant reduction of leakage rates.

As it was not permitted to empty the reservoir or even to limit water storage elevation less than 453 asl, i.e. 1m below normal water storage (representing 15% of the reservoir volume), remedial works could only be located on the crest and downstream.

Under these conditions, the first solution considered was a positive diaphragm wall located along the dam axis, keyed in the sound bedrock. It was to be carried out from the crest of the dam and should have reached a maximum depth of 40 to 45m. This solution is indeed expensive, but very effective in providing the necessary water tightness. However, the presence in the foundation of very soft and loose material or cavities along with free water passages highlighted by communications between boreholes and water losses, led the designer to discard this solution as it presented the risk of a sudden drawdown in the wall trench, during either excavation or concreting. This situation was not acceptable as the trench might have collapsed causing unpredictable damage, including large flow release as the reservoir remained operational.

Grouting, using Tubes à Manchettes (TAM) is considered unsuitable for silty clay material where it is likely to generate useless and potentially dangerous hydraulic fracturing mainly sub-vertical in upstream-downstream direction. Furthermore, at the exit of the manchette and the sleeve grout, it is not possible to monitor the grouting pressure applied at the ground contact. However, as the silty clay is cohesive and contains void like passages, TAM grouting can be relevant for foundation treatment.

The solution adopted finally was as follows:

- A positive cutoff made of sheet piles (SP) located along the dam axis and reaching a maximum depth of 40m;
- Grouting using TAM immediately upstream of the SP to plug any defect in the SP keys and plot and to complete foundation treatment outside SP limits.

The main potential issue of this solution was the difficulty to drive the SP to the required depth, due to the eventual presence of coarse elements in the foundation. Furthermore, it was not possible to carry out any full-scale trial before corresponding call for bids, as neither sheet piles nor driving equipment were available in Burkina Faso or even in neighboring countries.

The reassessment of investigations' findings, boreholes and CPTU, confirmed the absence of any elements likely to stop SP driving. This was discussed with SP manufacturers who confirmed the low risk of driving problems and recommended using high inertia profiles, i.e. AZ-36-700N made by Arcelor Mittal. The surface in the foundation to be covered by SP wall was fixed in connection with the main seepage paths identified during investigations.

Where the required depth could not be reached and outside the limits of the SP wall, TAM grouting remains the only available alternative. This grouting is therefore extended vertically to the sound bedrock in continuation of the SP wall and laterally outside its limits.

Regarding the risk of subsidence of the dam, due to the development of erosion cavities and loose soils under the duricrust, leading to pseudokarstic behavior of the foundations, the compaction grouting was considered the most suitable. This treatment was limited to the clayey silt horizon considered as the weakest in the foundation. It would have been interesting to perform some in situ testing before launching the corresponding tender, but this was not possible. The design was therefore set on the most realistic assumptions regarding mortar consumption and holes' spacing. Call for bids documents offered, however, flexibility in the design adaptation during the course of actual works.

Figure 4 represents the position of each component of the remedial works.

The SP cutoff placed on the dam axis along with the TAM grouting holes, located upstream, were far enough from the chimney filter to have no impact on its integrity. Compaction grouting holes may cross the filter but under systematic casing protection. Each hole was filled under very low pressure during the casing removal.

The durability of the SP is considered satisfactory as they are totally buried, having no contact with air. For its part, compaction grouting is performed after completion of SP driving and TAM grouting, so as to minimize mortar leaching.

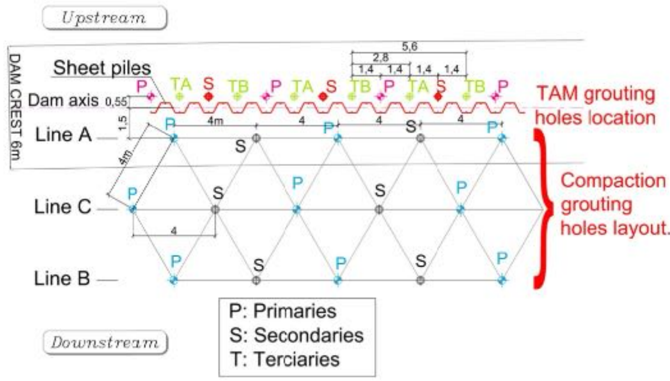


Figure 4. Remedial Works Definition

WORKS CARRIED OUT

Remedial works consisted of three main activities conducted in the following order:

- Driving SP through the embankment and the foundation to a maximum depth of 40m;
- TAM grouting to seal SP joints and, as much as possible, improve the water tightness of the foundation at depth and laterally beyond SP wall limits;
- Densifying the foundation by compaction grouting targeting the very loose silty clay horizons in the dam foundation.

No grouting was carried out in the body of the dam, except sleeve grout of TAM and filling under minimum possible pressure the upper part of the holes used for compaction grouting.

SP were paired by crimping in the factory (Photo 1) to provide high buckling resistance. Installation was conducted using heavy impact hammers. Vibratory hammers were abandoned because they could not drive SP through the embankment, confirming its good compactness.

Photo 1. Paired sheet piles delivered on the jobsite (left), driving on the dam crest (right)

To prevent deflection of the SP, these were set on a frame of almost 3m height, anchored on the crest. Each pair of sheet was installed inside the frame, plumbed and secured before setting the next pair. Driving started when a whole panel was set, respecting an increment smaller than 2m within the panel. Interlocking joints were filled with a special sealing product aiming at providing the required water tightness.

The driving stop criterion of the piles was 10 blows of the hammer for a maximum penetration of 2cm, i.e. 500 blows per meter. Actual refusal systematically extended to 800 blows per meter, where the design depth was not reached. The number of blows was accurately monitored and used for characterization of the foundation.

During SP driving, particle velocity was monitored to check the incidence of vibration on the dam and appurtenant structures. No problem was recorded, except that a few localized sinkholes appeared due to the shearing of the embankment material during several SP drives. These zones were excavated and filled with compacted lateritic clayey material.

The total surface of SP wall installed is almost 12,000 m², 7,650 of which in the left abutment reaching a maximum depth of 36m and 4,350 m² in the right abutment, reaching 34m maximum depth. Figure 5 provides the as-build configuration of the SP wall.

The TAM grouting holes (Ø 101 mm) were located 0.55 m upstream of the dam axis on the crest as shown in figure 4. They are very close to the SP wall in order to grout corresponding key joints where needed. They included primary, secondary and tertiary holes. Primaries were spaced 5.60m where SP are installed and 8m elsewhere. Thus, in the former case for the tertiaries, the final spacing is 1.40m corresponding to the distance between each pair of sheet piles. Out of SP walls, the final spacing for the tertiaries is of 2m. TAM are 50mm diameter, manchettes opening occurs under almost 0.1 MPa. No grouting was carried out in the dam embankment.

Two types of grout were specified, (1) sleeve filling and (2) foundation injection. Table 1 provides the main requirements and Table 2 corresponding proportioning.

Type of grout	Bleeding at 2 hours	Viscosity Marsh (s)	UCS at 28 days
Injection	<3%	<40s	>10 MPa
Sleeve filling	<5%	<40s	0.5 to 1 MPa

Table 1. Requirements for sleeve and injection grout used with TAM

Type of grout	W/C	Bentonite/Cement (%)	Admixture/cement (%)
Injection	0.75	0.7	0.5 to 0.9
Sleeve filling	3	10	0.5

Table 2. Sleeve and injection grouting proportioning

TAM grouting was conducted according to GIN method (Lombardi et al 1993) in order to reduce the risk of grout wastage due to hydro-fracturing. The GIN number was low, limited to 100MPa.liter/m, with a Vmax of 300l/m and Pmax as fixed in Table 3.

Depth (m)	0 to 4	4 to 7	7 to 10	10 to 13	13 to 16	16 to 20	20 to 25	25 to 45
Pmax at the top (MPa)	0	0.4	0.6	0.8	1.0	1.2	1.5	2.0

Table 3. Pmax specified at the top of the hole

All grouting parameters were set after testing on two trial sections, one of them in a very loose zone of the foundation. They led also to the conclusion that quaternaries were not necessary.

The main observations were as follows:

- GIN method, even considered unsuitable for this kind of soil, turned out to be well adapted to the silty clay grouting.
- There was no reduction in average grout consumption from primaries to tertiaries. However, at the tertiaries, almost no hydro-fracturing was observed and no grouting was stopped at Vmax.
- On the left bank, average grout consumption (sleeve and injection) far in the abutment was almost half (22 kg of cement/m) than that at the upstream of the resurgences zone (40 to 60 kg/m).
- Sleeve grout consumption was much higher near the SP wall due to the disturbance of the ground during SP driving.
- With regard to the very low hydro-fracturing occurrence in the left bank grouted first, the GIN number was increased in the right abutment TAM grouting to 300 MPa.l/m and Vmax to 1000 l/m. These parameters turned out to be well adapted to this foundation. A retreatment of the other abutment using these parameters is scheduled.

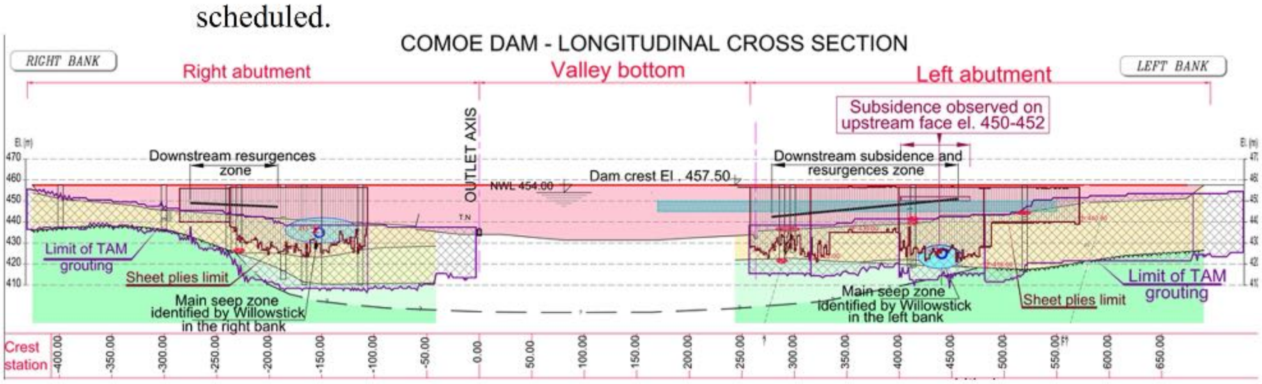


Figure 5. Longitudinal section along the dam axis, presenting the as build SP and TAM grouting on both abutments

Compaction grouting was carried out where SP and TAM works were completed. It was conducted according to ASCE *Compaction Grouting, Consensus Guide (2010).*

Requirements related to the mortar components were fixed for sand (1) a dmax of 20mm, (2) 10% to 20% of minus 200 and (3) fines plasticity index smaller than 10%. The slump required was 50mm at the injection pump delivery and UCS at 28 days greater than 10 MPa.

The final formulation of the mortar mix fulfilling these requirements is given in Table 4.

Sand (kg)	OPC Cement (kg)	Water (kg)	Admixture (HRWR) (kg)
1582	300	205	3

Table 4. Compaction grouting mortar proportioning

Sand produced from natural alluvium was scalped to 12.5mm. It contained 90% minus 5mm and 16% of non-plastic minus 200. The 28 days UCS exceeded 13 MPa. Due to the high ambient temperature at the site, the slump at the batching plant was fixed at 80mm so as to respect the 60 to 50mm during injection.

A regular piston concrete pump was used for the injection. It is not designed to control grouting pressure accurately. Grouting pressure was fixed to a maximum of 6 MPa in the beginning and reduced to 4 MPa, measured by a manometer at the exit of the pump. Unlike TAM there was no possibility to get automatic recording and monitoring of injection parameters.

A few months after their beginning, compaction grouting works stopped due to political instability in the country particularly in 2015. Out of a total drilling estimated to 15 000 m, only 3 600m were achieved, and only in the left abutment. Corresponding results are considered very promising and may be summarized as follows:

- Compaction grouting highlighted the presence of very loose areas, so they may be considered as voids. It is called cavernous soil. Indeed, grout takes exceeding 5m^3/m at low pressures are numerous. Near the above mentioned main tool drop area, 60 m^3 of mortar was injected. Coring conducted near big takes crossed almost intact mortar sometimes for a few meters. There was no permeation of mortar away from the injection point.
- Due to the large takes observed at each 0.5 to 1m grouting stage, jamming of casing tubes occurred very often. As it had a significant negative impact on progress of work, it was decided to reduce the casing diameter from 100 mm to 70mm, to install hydraulic jacks to pull out the tube and to move it slightly every 15 to 30 minutes. Depth of grouting, reaching a maximum of 30m, had minor impact on casings' jamming.
- Compaction grouting, which seems to be effective and additional TAM grouting in the left side should resume in 2017, now that the political situation is stable, enabling necessary funding arrangement.

DAM SURVEILLANCE – WORKS EFFICIENCY

Dam surveillance through regular visual inspections and monitoring data gathering and interpretation was set since completion of the dam. It helped in detecting early signs of seepage problems and led to carrying out remedial works described in this paper.

During remedial works, significant disturbance to the dam and the foundation occurred. The dam surveillance team permanently monitored seepage flow at several strategic points and hydrostatic pressure along the dam axis and downstream. The team also inspected the dam and adjoining areas, during working hours, looking for any eventual suspect new element such as cracking, subsidence, rebound or grout / mortar resurgence.

Furthermore, outlet gates were checked to ensure they could be operated if necessary.

During remedial works no damage or any suspect events occurred either on the dam or the foundation. Seepage flow never showed any cement particles content. The following harmless observations were reported:

- During compaction grouting clean water came out momentarily from some piezometers, confirming the densification action of these works;
- Resurgence of mortar from insufficiently plugged holes during grouting of an adjacent hole.

Although works are not completed, a significant decrease in measured seepage flow and a reduction of the hydrostatic pressures downstream of the dam was observed. Large wet and seepage resurging zones became dry except on the top of the left abutment, when the reservoir is above el. 450 ASL.

Restrictions on operating reservoir level were lifted, and monitoring data has shown good stabilization of the situation so far.

REFERENCES

ASCE (American Society of Civil Engineers). (2010) *Compaction Grouting, Consensus Guide,* Standard ASCE/G-I53-10. ASCE, Reston, VA.

ICOLD (International Commission on Large Dams), *Dam Surveillance Guide*, Bulletin 158, 2012, §7.3.

Lombardi, G. and Deere, D. (1993). "Grouting Design and Using the GIN Principle." *Water Power &Dam Construction, issue June 1993, pp. 15-22.*







NON-DESTRUCTIVE TESTS ON DAM PIER

CONDITION ASSESSMENT OF DAM PIERS

- Problem statement : Condition assessment was required for changing dam gates.
- Parameter required to evaluate the condition :
  - Reinforcement details
  - Homogeneity of concrete
  - Strength of Concrete
  - Chemical condition of concrete
  - Permeability of Concrete

REINFORCEMENT DETAILS THROUGH GPR METHOD

How to locate the reinforcement in structure?

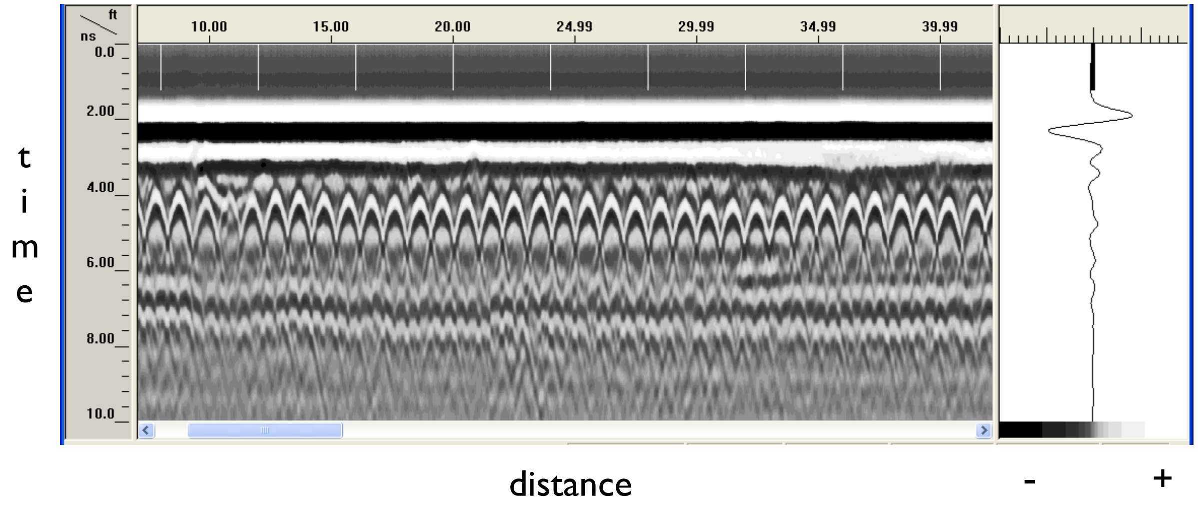
- GPR stands for **G**round **P**enetrating **R**ADAR
- RADAR stands for **RA**dio **D**etection **A**nd **R**anging
- High-frequency electro-magnetic (EM) reflection
- Active technique: emits energy and records the response
- Major uses include civil-engineering and geotechnical applications Like Utility scanning and structure scanning.

HOW DOES GPR WORK?

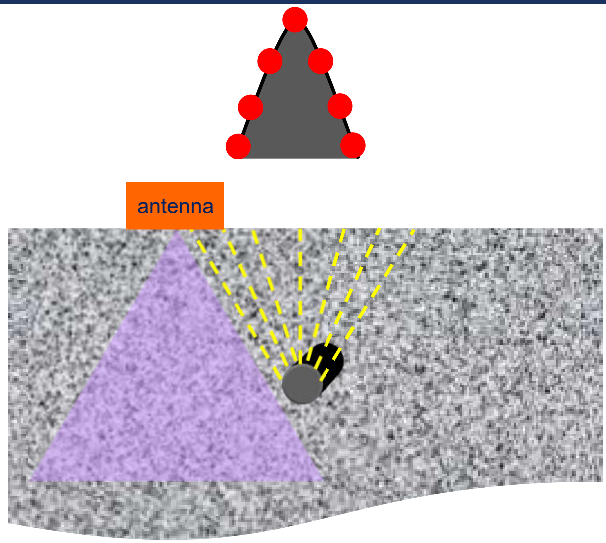
We record: Two way travel time

Amplitude

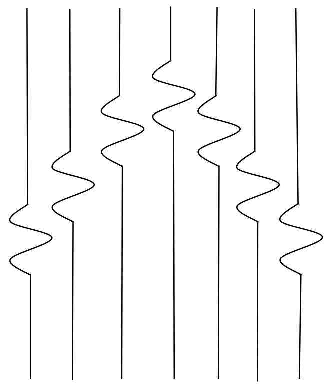
Phase (pos/neg)



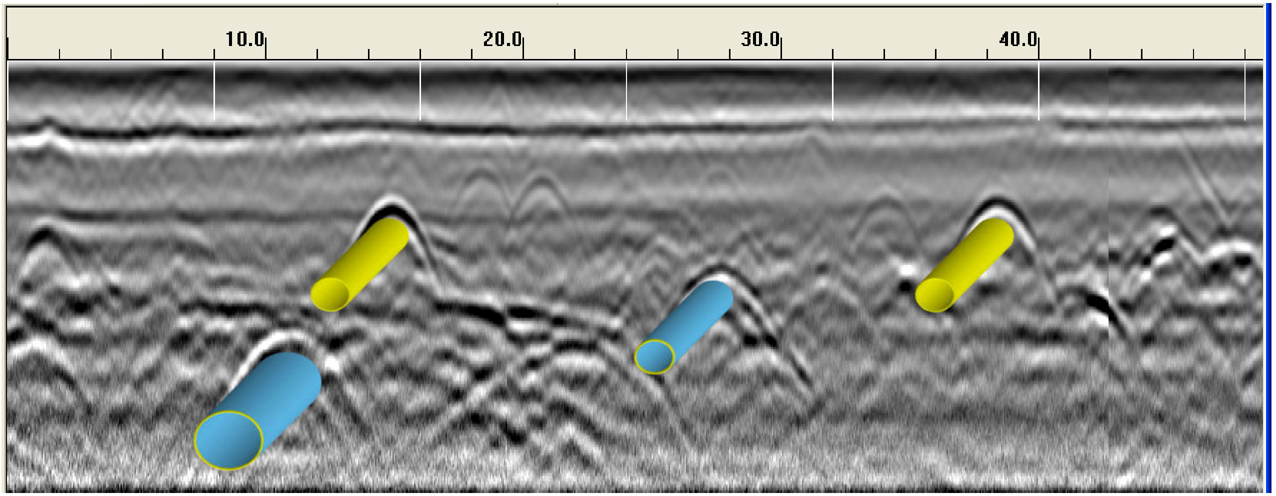
THE HYPERBOLA SHAPE



The increasing then decreasing two way travel time of the reflections from the object produces the hyperbola shape

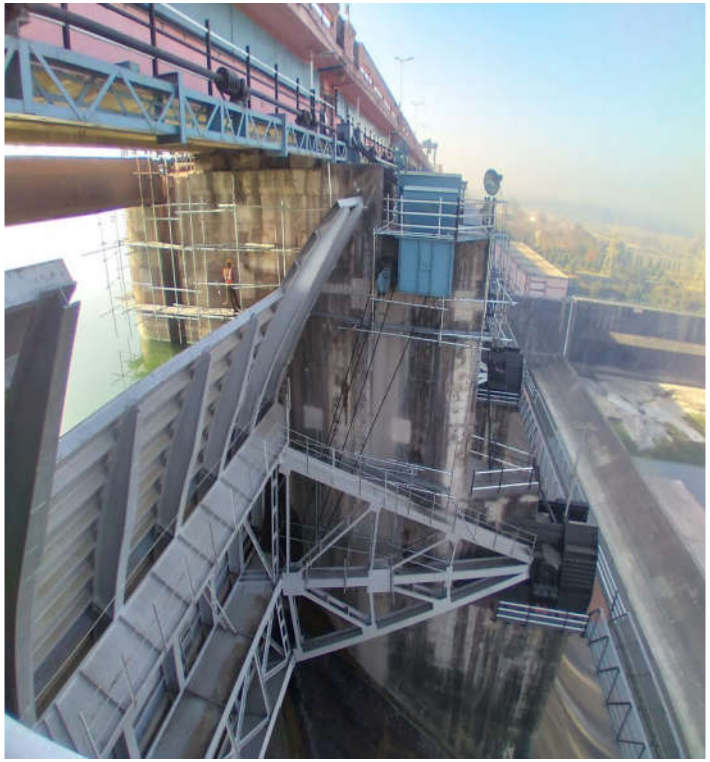
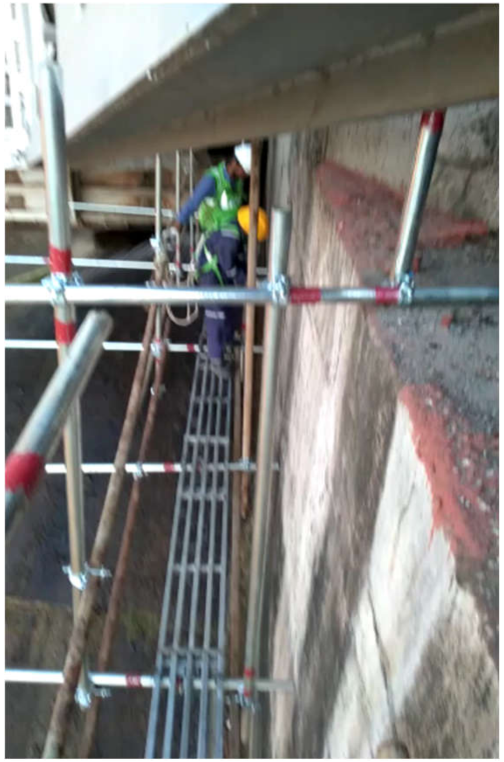
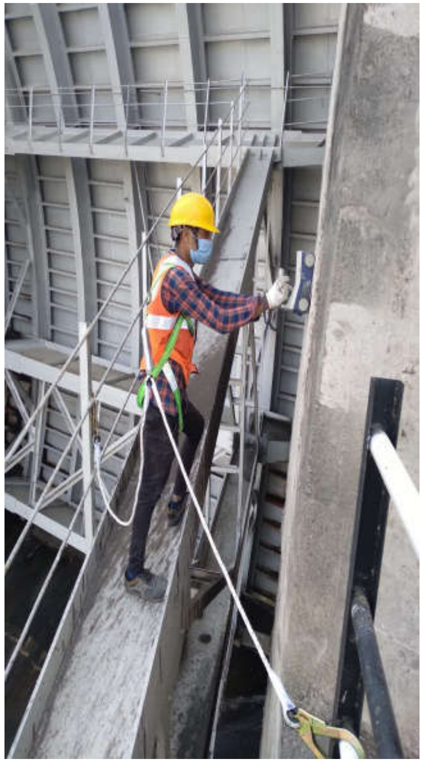


UTILITY DETECTION

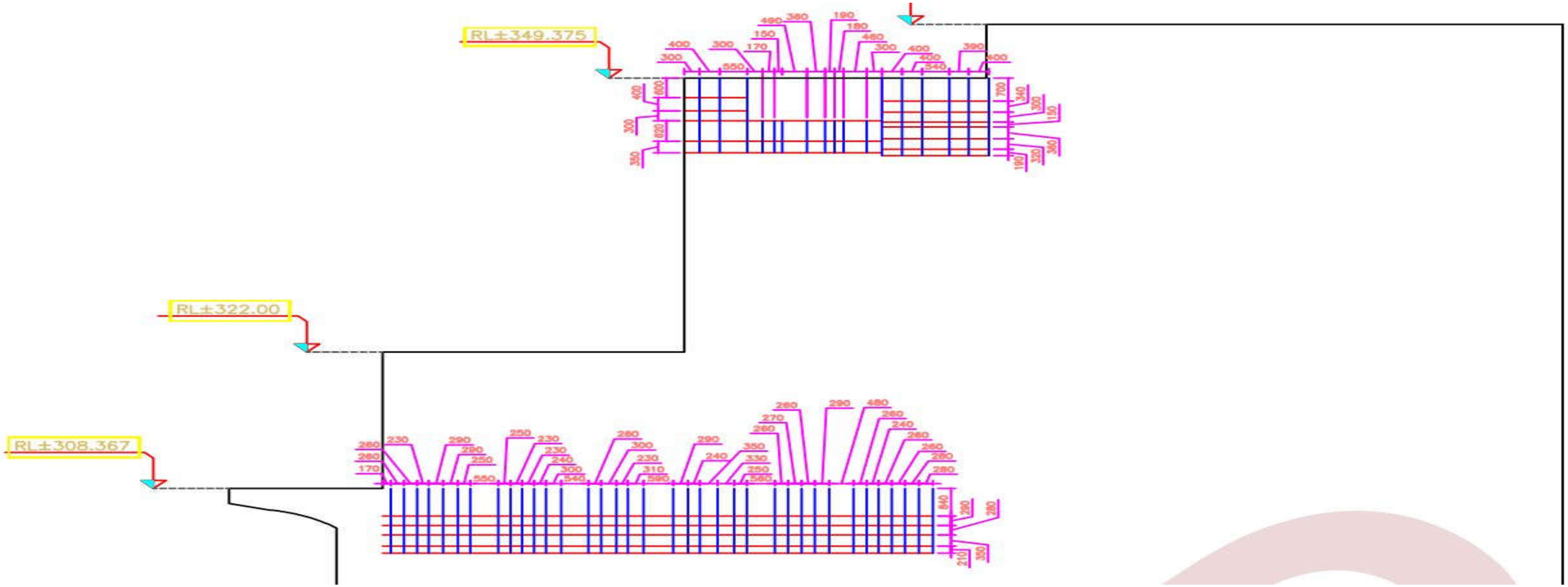


Underground Pipes

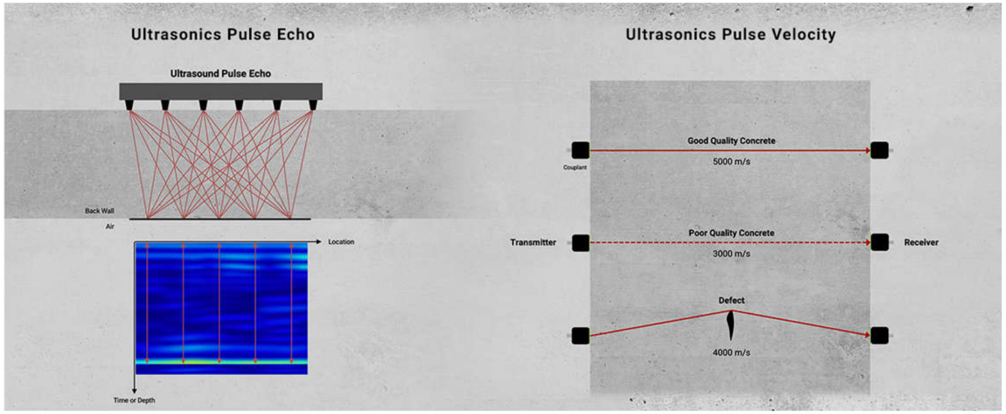
GPR SURVEY AT DAM SITE

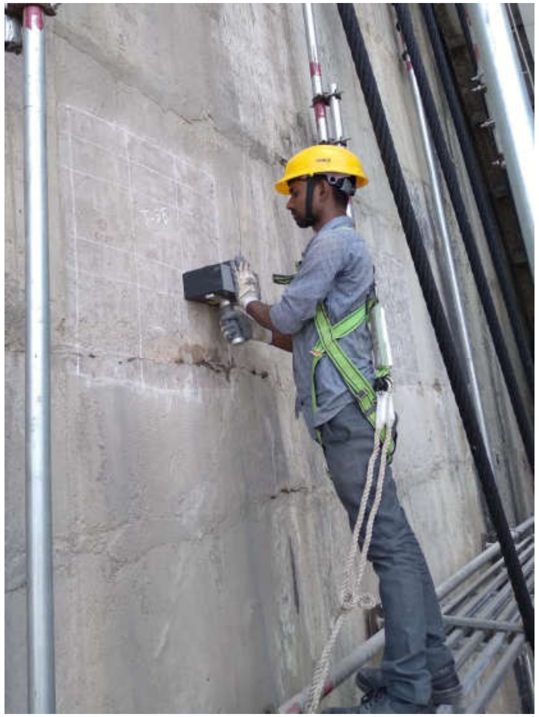
SAMPLE OF STEEL DETAILS

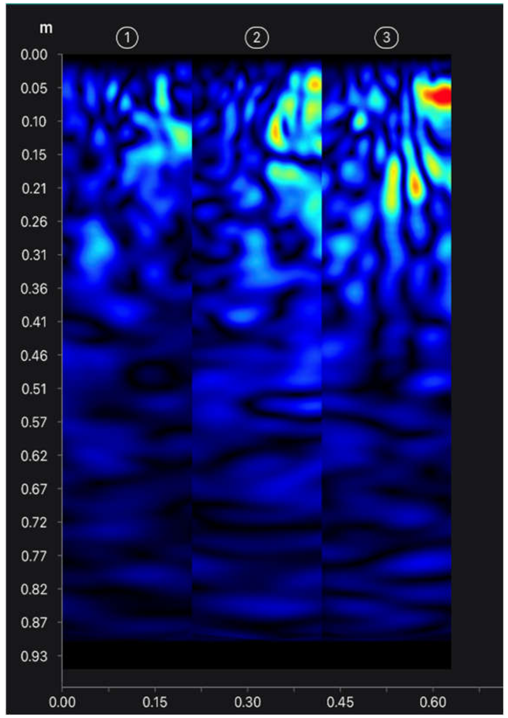


HOMOGENEITY OF CONCRETE THROUGH ULTRASONIC TOMOGRAPHY



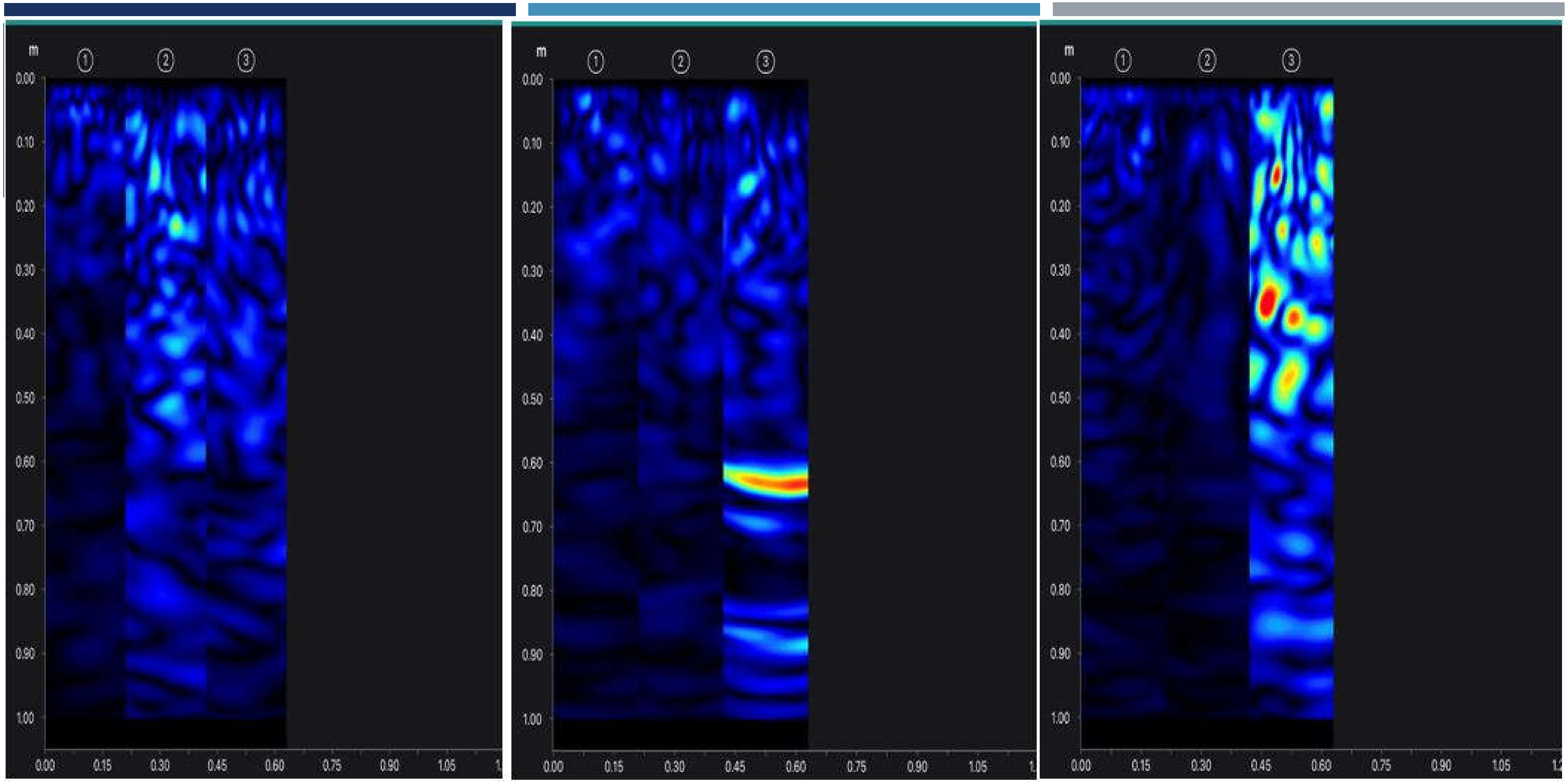
The 250 array is based on ultrasonic multichannel pulse echo technology using 8 channels. One channel transmits and echoes are received by other 7 channels. Each channel transmits in turn.





LOCATION	D/S- FRONT FACE/L-1@3.61 Ht.
GRID	3 x 3

RANGE OF DISTRESS			LEGENDS
			No or Very Minor Voids/Honeycombing
			Minor Voids/Honeycombing
			Severe Honeycombing


m
1 2 3
0.00 0.10 0.20 0.30 0.40 0.50 0.60 0.70 0.80 0.90 1.00
0.00 0.15 0.30 0.45 0.60 0.75 0.90 1.05
m
1 2 3
0.00 0.10 0.20 0.30 0.40 0.50 0.60 0.70 0.80 0.90 1.00
0.00 0.15 0.30 0.45 0.60 0.75 0.90 1.05
m
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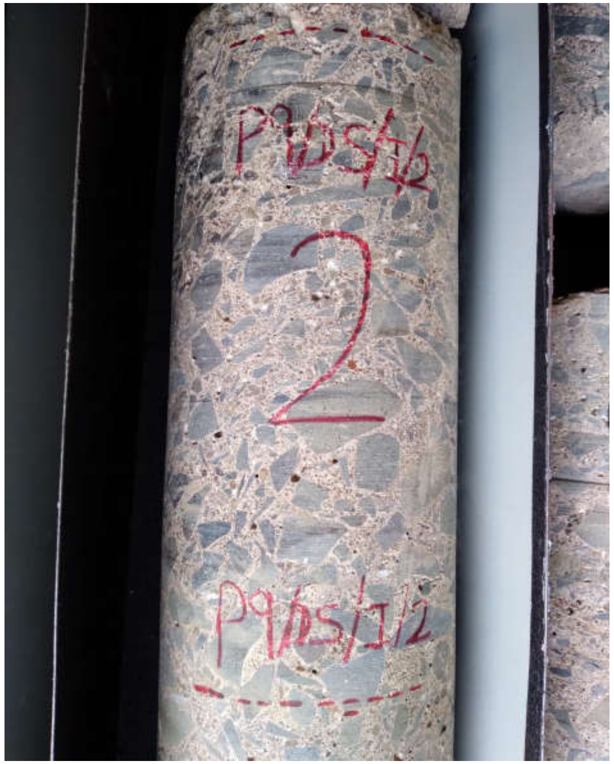
0.00 0.10 0.20 0.30 0.40 0.50 0.60 0.70 0.80 0.90 1.00
0.00 0.15 0.30 0.45 0.60 0.75 0.90 1.05

STRENGTH OF CONCRETE

DRILLING CHALLENGES

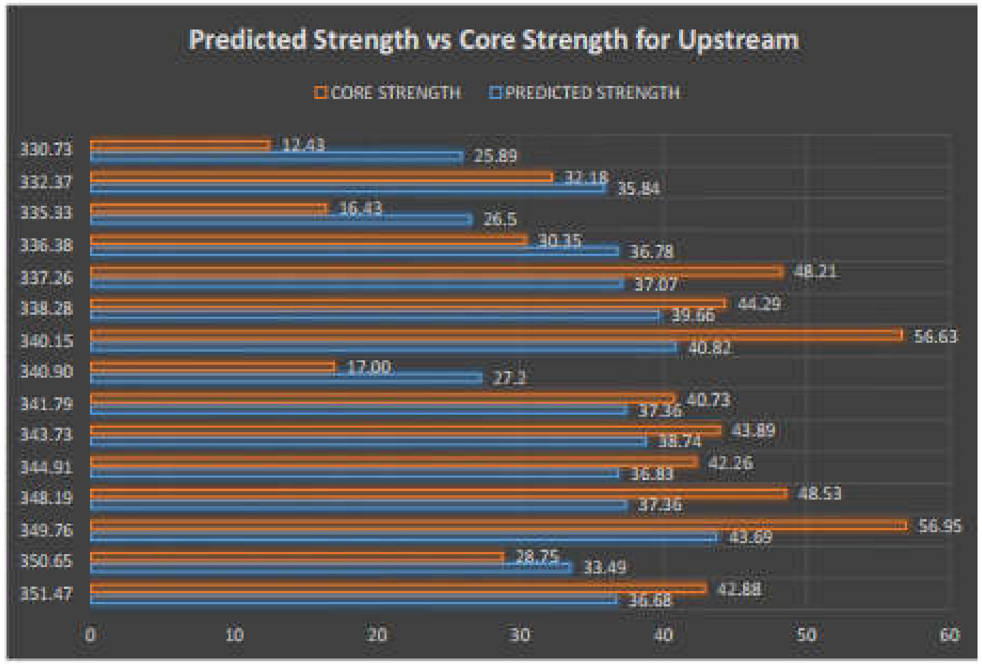
CONCRETE TESTING



RESULTS AND REGRESSION ANALYSIS

Below table shows the predicted strength vs core strength for the upstream:

Sr. No.	Location	UPV Test Result	Height	Compressive Strength on basis of UPV, MPa	Core Compression, MPa
1	P9/US/I/1	4.36	351.47	36.68	42.88
2	P9/US/I/2	4.01	350.65	33.49	28.75
3	P9/US/I/3	5.02	349.76	43.69	56.95
4	P9/US/II/1	4.43	348.19	37.36	48.53
5	P9/US/II/2	4.36	344.91	36.83	42.26
6	P9/US/II/3	4.57	343.73	38.74	43.89
7	P9/US/III/1	4.43	341.79	37.36	40.73
8	P9/US/III/2	3.21	340.90	27.2	17.00
9	P9/US/III/3	4.77	340.15	40.82	56.63
10	P9/US/IV/1	4.66	338.28	39.66	44.29
11	P9/US/IV/2	4.40	337.26	37.07	48.21
12	P9/US/IV/3	4.37	336.38	36.78	30.35
13	P9/US/V/1	3.11	335.33	26.5	16.43
14	P9/US/V/2	4.27	332.37	35.84	32.18
15	P9/US/V/3	3.02	330.73	25.89	12.43



PERMEABILITY TEST

The pumping in test is applicable for strata in both cases of above and below water table. The test is especially performed in formations of limited thickness. The tests give permeability of the material in the immediate vicinity of the bottom of the drill hole. It may thus be used for determining the permeability of different layers in stratified foundations and thus check the effectiveness of grouting in such formulations.

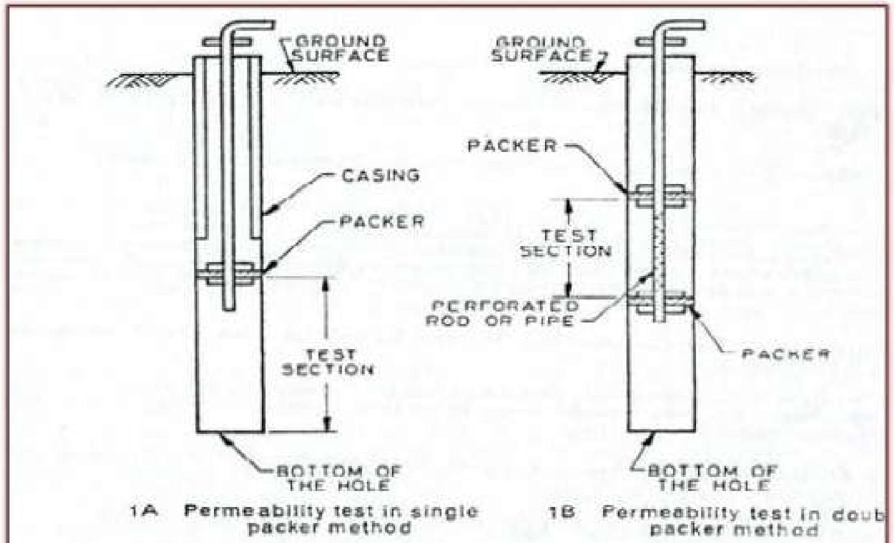


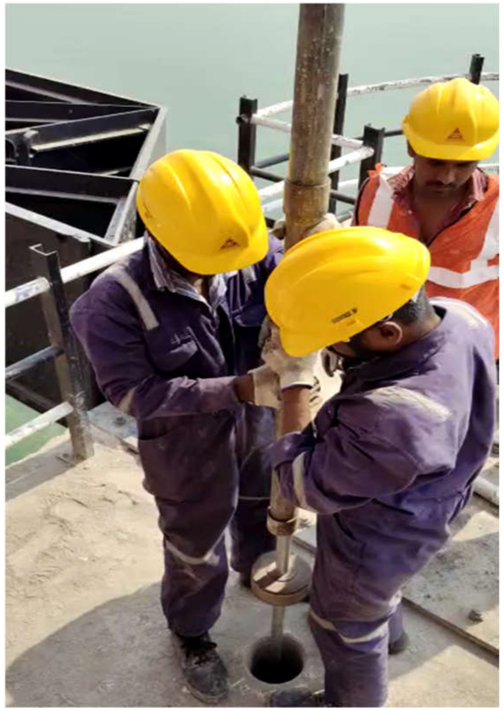
FIG. 1 PERMEABILITY TEST BOTH IN SINGLE AND DOUBLE PACKER METHOD

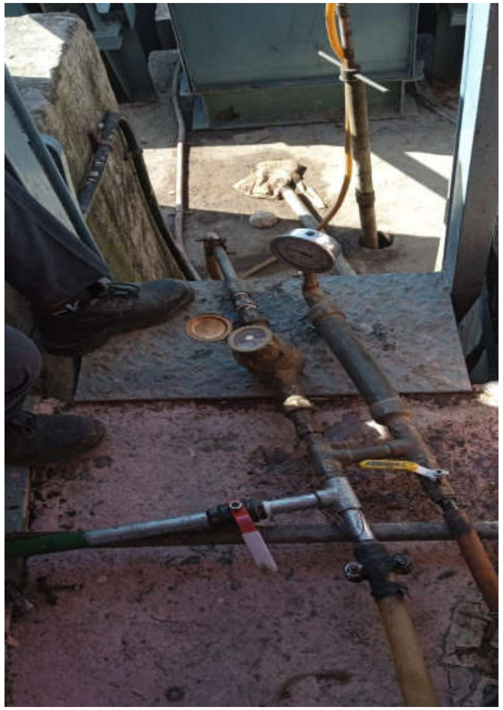
PACKET PERMEABILITY TEST

CHALLENGES

- DIAMETER OF BOREHOLE
- LIFT JOINTS
- HIGH PRESSURE

PACKET PERMEABILITY TEST





PACKET PERMEABILITY TEST

Sr. No.	Stream	Test Section (m)	Lugeon Value	Group	Classification	Remarks
1	U/S	1.50 – 4.50	0.736	D	Washout	
2	U/S	4.50 – 7.50	0.00	C	Dilation	
3	U/S	7.50 – 10.50	15.337	B	Turbulent Flow	There is possibility of leakage
4	D/S	0.0 - 3.0	0.00	C	Dilation	
5	D/S	3.0 - 6.0	0.00	C	Dilation	

As per the permeability test result, Lugeon value is falling between 0 to 0.736 and the medium can be considered as medium.

CONCLUSION

We got precise as mentioned below :

- Location and cover of steel reinforcements
- Strength of concert
- Chemical properties like carbonation , sulphur , chloride.
- Homogeneity of concrete
- Permeability of concrete and after grouting we are assured for leakages between shift joints.

THANK YOU

THEME D: Dam safety assurance under climate change

Increasing Dam Safety against floods: Rehabilitation of a dam with Fusegates in USA, case study.

Evangelos (Angelos) Rabias
Business Development Manager, Hydroplus

Hasan Kocahan
Area Manager, Hydroplus

1 . Basic Dam Details

Cedar Cliff dam is located on the East Fork of the Tuckasegee River near Cullowhee, North Carolina and is used for hydropower generation. Constructed in 1952, the dam is owned by Duke Energy and is operated in concert with two upstream hydroelectric projects for optimized generation and to satisfy low flow and recreational flow requirements downstream of Cedar Cliff Dam. Cedar Cliff reservoir has a surface area of 49 hectares (121 acres) and a total reservoir volume of 7.65 million cubic meters (6,200 ac-ft) at a normal maximum pool elevation of El.710.18 m (El.2,330 ft) mean sea level (msl). The drainage area of the reservoir is 209 square kilometers (80.7 square miles).

The dam is a rock-fill embankment with a sloping compacted earth core, a length of 179.83 m (590 ft) and a maximum height of 52.73 m (173 ft). The embankment has a nominal downstream slope of 1.3:1 (H:V) and a variable upstream slope of 1.3:1 (H:V) that flattens toward 2.2:1 (H:V) for the lower two-thirds of the structure height. The dam crest is 7.62 m (25 ft) wide and includes a concrete parapet wall along its entire length that varies in height from 0.45 m (1.5 ft) to 1.13 m (3.7 ft). The nominal crest elevation along the parapet wall is El.714.30 m (El.2,343.5 ft) msl. An overall view of the dam and its two spillways are shown in Figure 1.

The existing spillways at the dam include a principal spillway (Figure 2) with one Tainter gate at the right abutment (when looking downstream) and an auxiliary spillway (Figure 3) with two erodible fuse plugs at the left abutment. The principal spillway Tainter gate is 7.62 m (25 ft) wide by 7.62 m (25ft) high with a sill elevation of El.702.56 m (El.2,305 ft) msl and is supported by concrete sidewalls doweled into rock. The principal spillway is a rock cut channel that conveys water to the bypassed East Fork of the Tuckasegee River located near the downstream toe of the dam.

Figure 1 Satellite view of Cedar Cliff dam

The auxiliary spillway channel is also a rock cut channel with the inner wall adjacent to the left abutment of the dam. The outer wall of the channel is steep (near vertical) with variable rock cut heights ranging from 15.24 m (50 ft) to 50.29 m (165 ft). Water conveyed through the auxiliary spillway enters the bypassed East Fork of the Tuckasegee River on the opposite side from the principal spillway discharge.

The two fuse plugs, at the auxiliary spillway control section, are separated by a concrete splitter wall and have a total length of 60.96 m (200 ft). The left fuse plug, when looking downstream, is 34.14 m (112 ft) long with a crest elevation ranging from El.711.10 m (El.2,333 ft) msl to El.711.40 m (El.2,334 ft) msl. The invert elevation at the base of the left fuse plug is approximately at El.705.61 m (El.2,315 ft) msl. The right fuse plug is 26.82 m (88 ft) long with a crest elevation ranging from El.710.49 m (El.2,331 ft) msl to El.711.10 m (El.2,333 ft) msl. The fuse plugs are constructed of crushed stone and sand with a sloping core of compacted impervious fill.

Activation of the fuse plugs and conveyance of water through the auxiliary spillway channel for flood operation purposes is attributed to overtopping of the erodible fuse plugs.

Figure 2 Main Spillway

Figure 3 Auxiliary spillway with fuse plug

2. Overview of the project

The spillway capacity for Cedar Cliff needed to be increased to safely pass the new regulatory-required IDF of full PMF (5,437 cms). The construction of a Fusegate System in an enlarged auxiliary spillway channel was selected to increase spillway discharge capacity to meet the new requirement.

The reservoir surface area, and thus flood storage capacity is small with respect to flood volumes, therefore, an increase in spillway capacity was determined to be the best option to safely pass the PMF. The auxiliary spillway control section invert elevation was lowered from El.705.61 m (El.2,315 ft) msl to El.702.56 m (El.2,305 ft) msl and the width of the control section was increased to 76.2 m (250 ft). In addition, approximately 75 percent of the length of the spillway channel was modified with a nominal depth increase of approximately 3.05 m (10 ft) and a nominal width increase from 28.96 m (95 ft) to 44.20 m (145 ft). The existing fuse plug control section was replaced with the Fusegate System as shown in Figure 4. The Fusegate System is a passive system and serves as a semi-labyrinth spillway for a significant portion of the Cedar Cliff PMF.

The new auxiliary spillway sill is a broad crested weir with its crest coinciding with the top of rock surface at the approach channel. The Fusegates were constructed as cast-in-place concrete on this new weir. Toe abutment blocks were anchored at the downstream edge of the flat sill to prevent Fusegates from sliding under the upstream hydrostatic pressure.

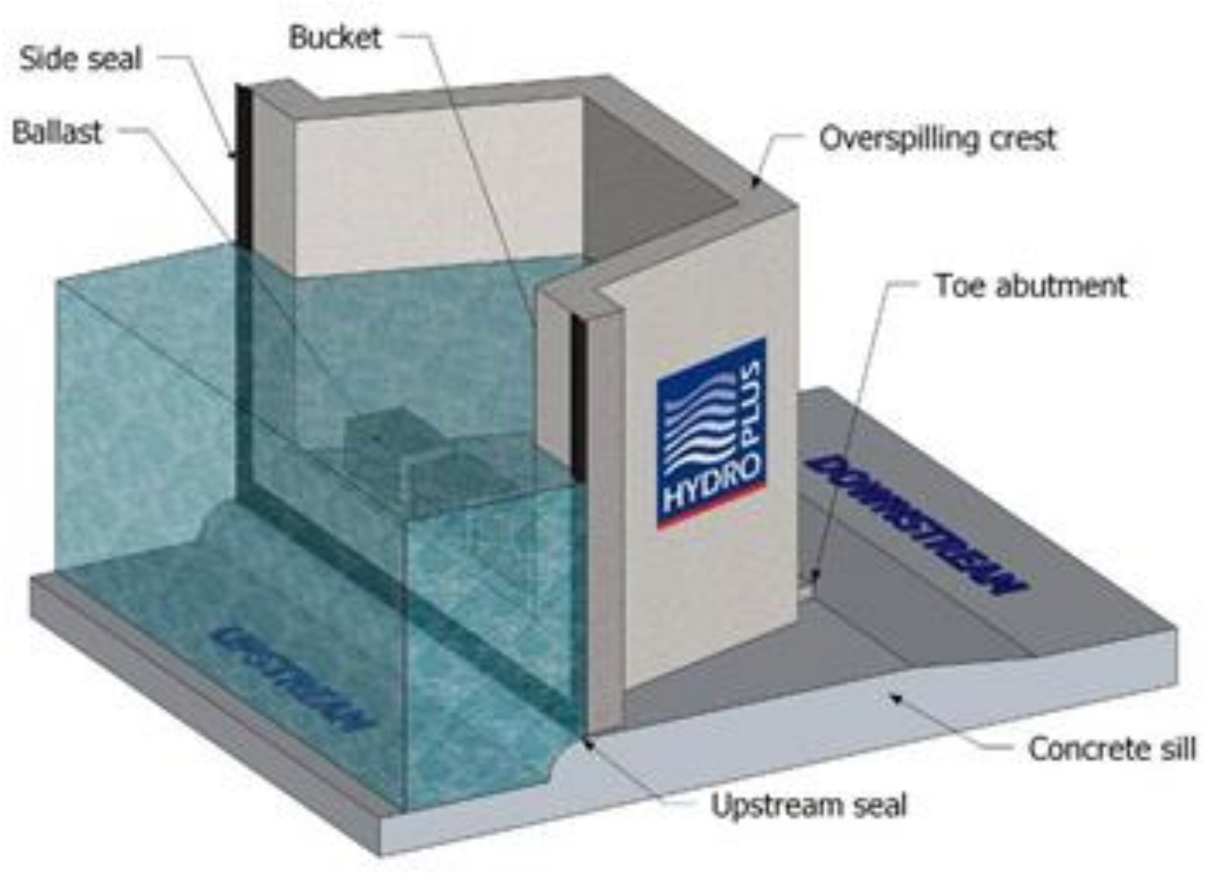


Figure 4 The 3D view of a semi-labyrinth type Fusegate

3. Design features of Fusegates

The proposed Fusegate System includes 6 semi-labyrinth Fusegates, each 7.62 m (25 ft) high and 12.70 m (41.67 ft) wide. The Fusegates are founded on the new broad-crested weir having an upstream to downstream length of 10.52 m (34.5 ft). Each Fusegate includes one stainless steel toe abutment block embedded into the control sill to enable the Fusegates to pivot or rotate during a tipping sequence.

The Fusegate design at Cedar Cliff dam involves the use of six remote protected inlet wells located in a structure called the protected inlet well tower. The water supply to each Fusegate's base chamber is through a feed system composed of an inlet well and pipeline, which is called an interconnecting pipe. When the reservoir reaches the activation elevation for a specific Fusegate, water is admitted through the corresponding inlet well inside the inlet well tower, flows through its interconnecting pipe extending through the spillway sill and discharges under the corresponding Fusegate. This discharge fills the pressure chamber and induces overturning of that Fusegate. The configuration of the Fusegate System and its appurtenant structures are shown in Figures 7 and 8.

The potential head losses due to the geometry of the reservoir and the variable tailwater levels in the channel prior to and following activation of one or more Fusegates favored the selection of this configuration for Cedar Cliff dam to improve the predictability of Fusegate activation. This solution also prevents any impact and accumulation of floating debris on the inlet wells.

The inlet well tower at Cedar Cliff dam is located approximately 60.96 m (200 ft) upstream of the Fusegate control section. The interconnecting pipes were installed in an excavated trench (approximately 6.40 m wide by 55.47 m long) and were embedded inside the concrete sill once they reached the spillway control section.

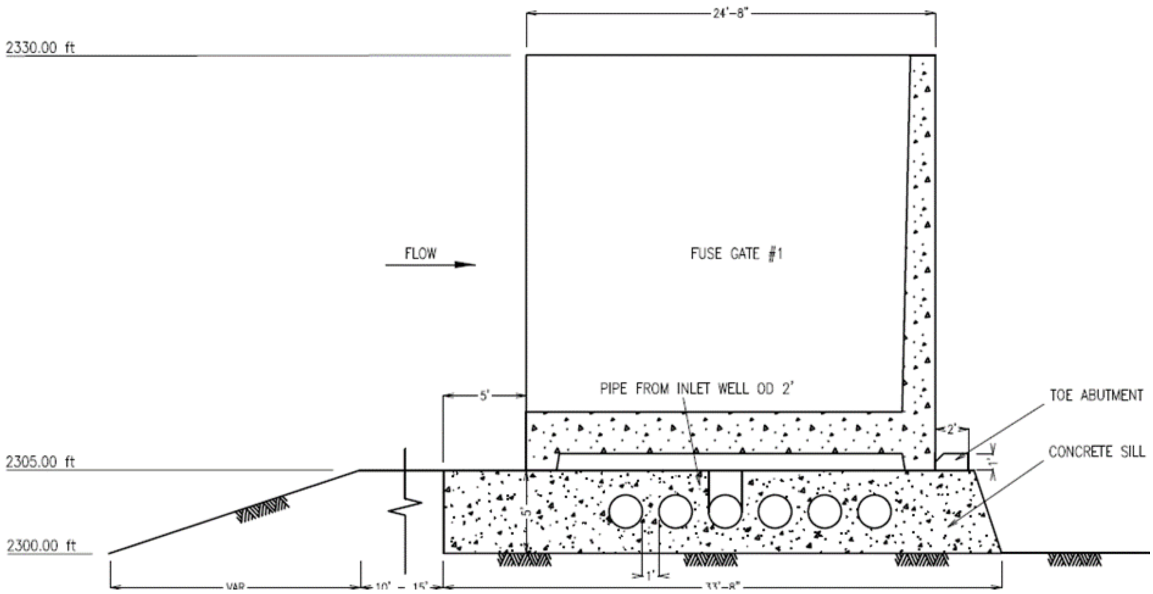


Figure 5 Cross-section of the spillway sill and a Fusegate

The six Fusegates serve as a passive semi-labyrinth spillway up to Cedar Cliff reservoir inflows exceeding approximately 1,727 cms (61,000 cfs) or 32 percent of the PMF. The flood return period for this event is approximately 60,000 years. For perspective, the flood of record for Cedar Cliff Dam is approximately a 90-year flood event. Floods exceeding 32 percent of the PMF will produce a reservoir elevation above approximately El.713.38 m (El.2,340.5 ft) msl which will result in the initiation of the first Fusegate activation. The remaining five Fusegates will activate with increasing reservoir inflow at reservoir elevations El.713.69, El.713.99, El.714.30, El.714.60, and El.714.91 m (El.2,341.5, El.2,342.5, El.2,343.5, El.2,344.5, and El.2,345.5 ft) msl, respectively. If a Fusegate tips, spillway discharge in combination with the steep channel slope will cause it to be displaced downstream of the spillway control section, and it will not cause an obstruction or increased tailwater on the Fusegate system.

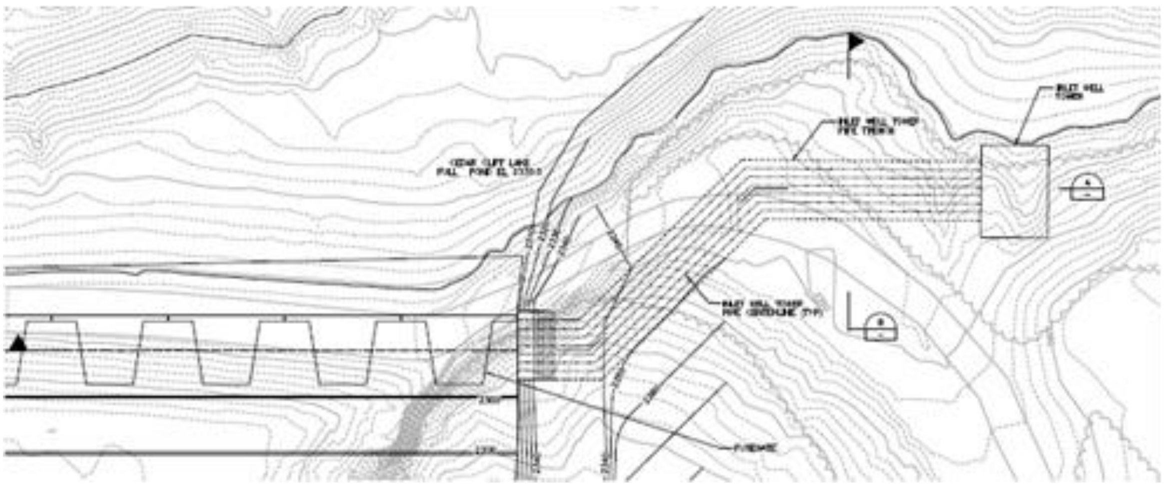


Figure 6 Protected wells and interconnection pipes layout plan

4. Design features of physical model

Activation of the Fusegate System is dependent upon the interaction of upstream water pressure, downstream water pressure (tailwater), and induced uplift pressure in the Fusegate chamber. Predictability of activation is dependent upon the confidence in these variables. The level of confidence in tailwater on the Fusegates in this physical setting and for the project flows was low. The extensive CFD modeling completed by HDR during the initial design stage predicted a reflected wave in the vicinity of the downstream face of the Fusegates; however, the uncertainty of the location of the wave associated with CFD model limitations left predictions of tailwater and thus Fusegate activation points with a wider range of flows than was considered acceptable. In addition, it was unknown to what extent sub-atmospheric pressures would develop due to air entrainment and how this would affect tailwater.

Significant air entrainment in flow over the Fusegate can reduce air pressure in the cavity between the nappe and the Fusegate resulting in a water level that is higher than what occurs for a well-ventilated system. To avoid this condition and reduce uncertainties of tailwater levels, a physical model was needed to determine if the system would be self-ventilating or if ventilation would be necessary. Alden Research Laboratory, Inc. (Alden) was engaged to evaluate whether the Fusegates would require ventilation or if they would be self-ventilating in this application through a physical model.

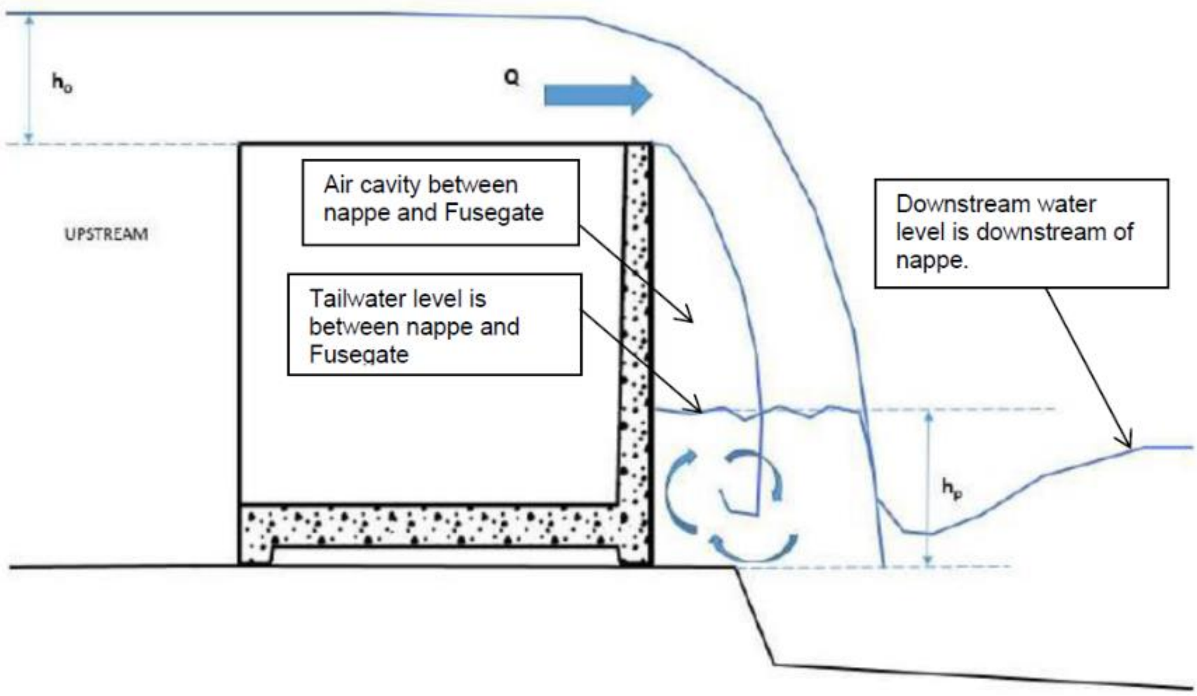


Figure 7 Water flowing over a Fusegate

The Cedar Cliff spillway Fusegate modeling evolved during the course of the project. The initial scope of work defined as Phase 1 was to:

- Determine whether the Fusegate System, in this physical setting and under project flow rates would be self-ventilating or if it would require ventilation for predictability of under nappe pressures.
- Suggest design modifications (or alternate designs) to ensure that air drawn into the void behind the nappe is sufficient to ensure near-atmospheric pressures.
- Determine the stage discharge rating curve for the spillway.
- Determine the tailwater levels at the Fusegate to be incorporated in the ballast weight calculations for the Fusegates.
- Measure water levels and water velocities in the spillway channel for select tests defined by HDR that could be used to validate their CFD model.

Two reduced scale physical models were constructed to determine the required size of a ventilated system for the proposed Fusegates. Recent literature shows that scale models used for determining air entrainment require a model scale of 1:10 or larger, some literature suggests a model scale no smaller than 1:15. Hence, Alden proposed a hybrid modeling approach using a large model (1:8) of a single Fusegate to understand the air entrainment characteristics of a single Fusegate (phase 1). The model was scaled to minimize scale and boundary effects on air entrainment and was constructed in a straight rectangular flume. A second model was constructed at a smaller scale of 1:19 and included all six Fusegates and the downstream auxiliary spillway channel (phase 2). The 1:8 scale model of a single gate was intended to be used to calibrate the results from the 1:19 scale model. Both models were designed with the pumping capacity necessary to satisfy the Phase 1 scope of work.

The 1:8 scale model (phase 1) showed that for the Cedar Cliff Fusegate geometry a single Fusegate will self-ventilate over the range of flows tested. Testing also showed that the air pressure in the cavity between the Fusegate and nappe decreases with increasing flow (Figure 7). With a ventilation system in place, the magnitude of the pressure decrease was reduced, however, the sub atmospheric pressure was not a significant factor in the tailwater level. Testing showed that the water level downstream of the nappe was important in the tailwater level at the Fusegate (Figure 8).

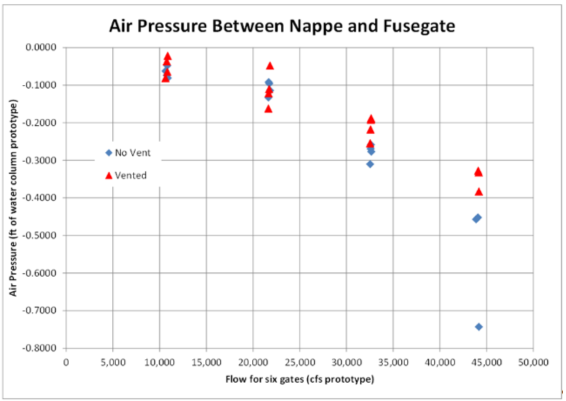


Figure 8 Sill tap locations during Phase 1 testing

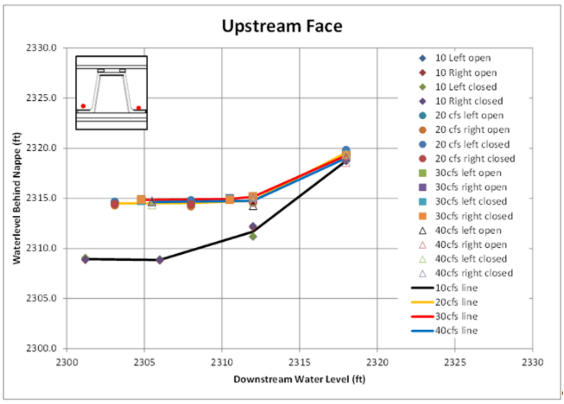


Figure 9 1:8 scale model looking downstream at Fusegate

The 1:19 scale model (phase 2) was expected to predict less air entrainment than a 1:8 scale model due to scale effects. Based on the results of the 1:8 scale model it was established that ventilation was not necessary to ventilate the air cavity under the modeled range of flows. Rather, the tailwater levels immediately downstream of the nappe were the principal influence on tailwater levels at the Fusegate. Tailwater levels downstream of the nappe were observed to have significant spatial variability with strong vertical velocity components and were influenced by the presence of a complex hydraulic jump (see figures 10 and 11). A comparison between the tailwater levels measured in the physical model and those computed in a CFD model showed that there was significant difference between the values. It was concluded that the CFD model could not be used for computing tailwater levels at the Fusegates.

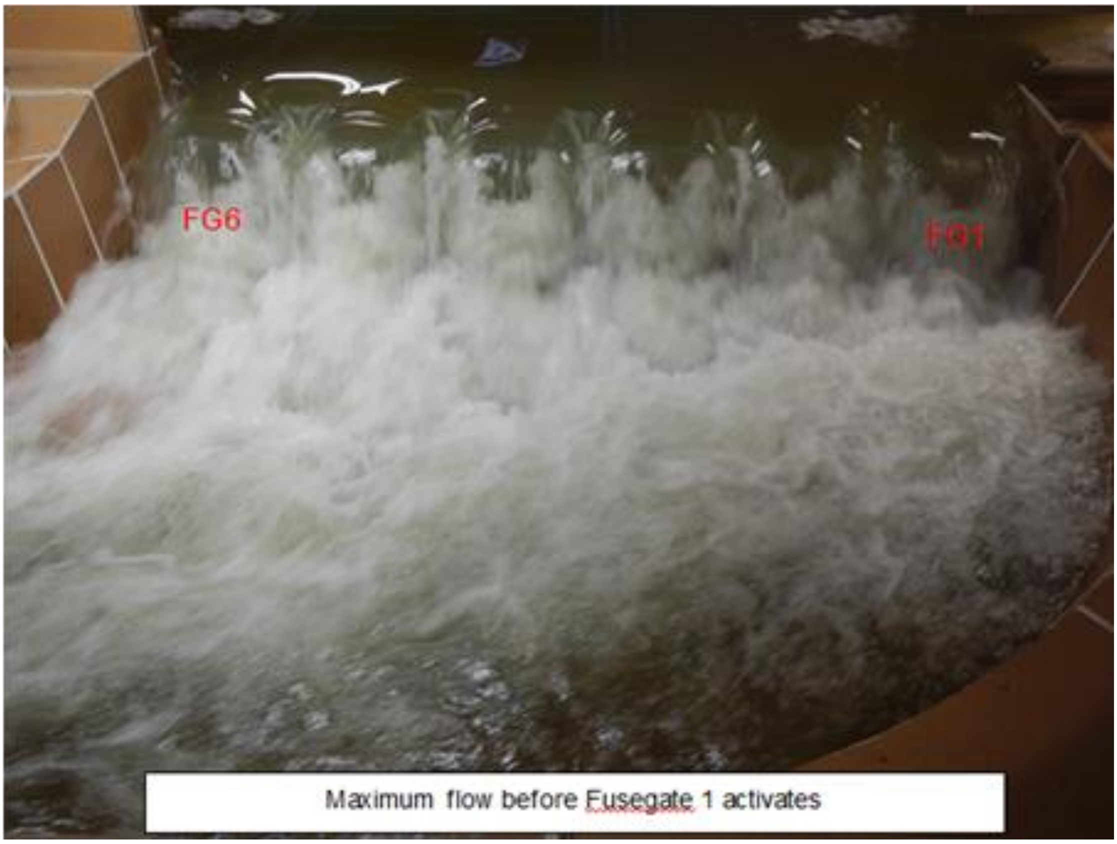


Figure 10 1,254 cms, nominal pool elevation of 713.38 m

Figure 11 Sill taps locations during Phase 1 testing

A stage-discharge rating curve was developed from the physical model tests. The variable spillway geometry results in a discontinuous rating curve (Figure 12). CFD predictions of the rating curved agreed with scale model tests within about 5 percent.

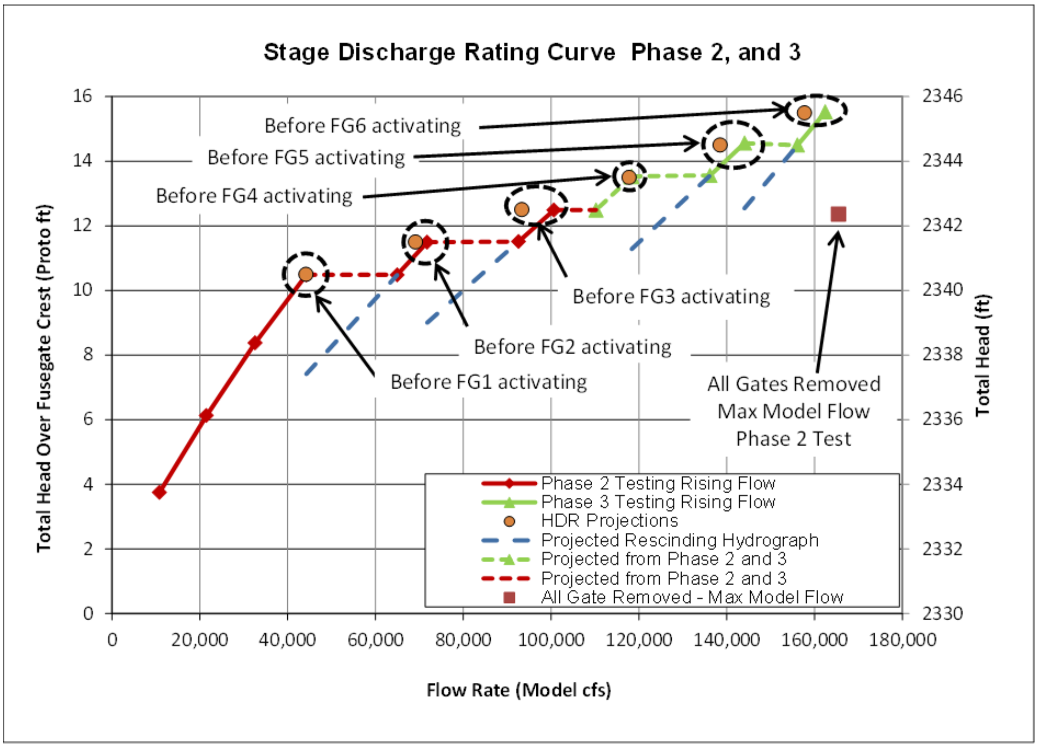


Figure 12 Composite stage discharge curve with extrapolated information

All of the model results shown in this paper are site specific and should not be assumed to apply to any other sites. The self-ventilation observed in the Cedar Cliff model are not a transferable finding and the tailwater levels on the Fusegate System are dependent on site-specific geometry and flow conditions which may not lend themselves to prediction using CFD.

5. Fusegates construction phase

The construction of Fusegates by Hydroplus started early December 2023 and was completed at the beginning of May 2024.

The Fusegates were constructed in cast-in-place reinforced concrete. Ballast blocks are a second stage concrete placement that was poured inside the bucket of the Fusegate. The Fusegate embeds, fixing plates, and watertightness seals were furnished by Hydroplus and installed as per the Construction drawings and specifications.

Fusegate construction was performed in the following order:

- Inspection of the spillway sill and abutment walls for tolerances
- Layout of Fusegate locations on the sill
- Construction of the Fusegates
- Jacking of the Fusegates to determine their weight and center of gravity.
- Placement of the ballast concrete
- Installation of the watertightness system
- Final as built survey and taking over of project

The survey crew established the Fusegate layout on the sill concrete based on the IFC (issued for construction) drawings. Local control points were established for layout and elevation verification during the construction process. Fusegate layout was established using the existing project control points.

Figure 13 Overview of the spillway sill during the initial survey

Fusegate construction was performed in two different stages. Stage 1 included the base and stage 2 the walls. Only one construction joint was used between the base and the walls.

A bond breaker was installed on the concrete sill where it comes in contact with the Fusegate's base. This prevents bonding of the Fusegate base to the spillway sill.

Perimeter forms were installed for the base slab of the Fusegate. These forms were handset and anchored as per the formwork design. Wood block outs were installed to form voids at the Fusegate lifting points. Once the forms were installed, TOC elevation lines were established onto the formwork by transferring elevations from our local benchmark with an optical or laser level.

Rebar reinforcement was installed and tied in place as per the construction drawings, with slight adjustments as necessary to work around embed studs.

6" Waterstop was installed in place per the construction drawings.

Concrete placements were performed using two pump trucks. One truck was staged above the left abutment and pumped into the second truck staged upstream of the sill. This eliminated the need for the concrete trucks to maneuver to the sill area.

After each Fusegate base was constructed, it was jacked to remove the false formwork at the bottom of the base, which was used to form the chamber of the Fusegate.

After each Fusegate was constructed (base and walls), it was jacked again in order to determine the actual weight of the Fusegate and verify the design quantity of the ballast or make adjustments as necessary.

The ballast was placed as a single lift for each Fusegate. Reinforcing was installed and tied in place as per the construction drawings. Once the rebar was installed, the concrete temperature sensors were placed as per the thermal plan. A vertical wall form was installed to contain the concrete in the Fusegate bucket. The thermal plan for the cooling of the ballast concrete included cooling pipes routed through the forms, inlet and discharge, and connected to PVC headers. This header had a connection port for each of the cooling lines required for the specific ballast and a bleed port. An HDPE pipe was connected to the headers and routed to the lake. One submersible pump was connected to the supply piping with water from the lake. Once the concrete was set and the thermal control duration was ended, the pipes were grouted.

Installation of the watertightness system was performed after the completion of the ballast for each Fusegate. The watertightness system consists of steel plates and EPDM seals installed between each Fusegate, vertically and between the Fusegates and the spillway sill horizontally. It ensures watertightness of the Fusegate system.

Figure 14 Fusegates downstream view after completion (during the as built survey)

Once the watertightness system installation was completed, the survey crew conducted the final as built survey to verify the construction was performed as per the IFC drawings and the taking over certificate of the project could be issued.

6. CONCLUSION

The dam needed to be rehabilitated in order to be able to safely pass the revised full PMF downstream. In short:

- Emergency spillway needed to be upgraded
- Safety had to be increased since fuseplugs are not considered as a safe solution
- Spillway discharge capacity had to be increased
- Cost saving solution had to be chosen

The solution chosen, the Fusegates System, were able to:

- Upgrade the emergency spillway with a robust and reliable system
- Increase the safety, since they are a passive solution which requires no human intervention or external power source to operate
- Increase the discharge capacity of the spillway; with the labyrinth shape more water is able to pass over the same spillway section
- Be the best cost saving solution from all the available ones

Figure 15 Fusegates downstream view in operation

7. REFERENCES

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Alden, 2017. Physical Model Testing of Cedar Cliff, *Fusegate Ventilation. Report No. 1170HPCC*. Alden Research Laboratory, Inc. Holden, MA 01520.

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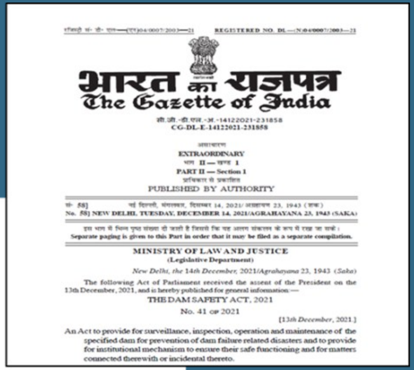

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भारत का राजपत्र
The Gazette of India

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असाधारण
EXTRAORDINARY
भाग II — खण्ड 1
PART II — Section 1
प्राधिकार से प्रकाशित
PUBLISHED BY AUTHORITY

No. 58] NEW DELHI, TUESDAY, DECEMBER 14, 2021/AGRAHAYANA 23, 1943 (SAKA)

Separate paging is given to this Part in order that it may be filed as a separate compilation.

MINISTRY OF LAW AND JUSTICE
(Legislative Department)

New Delhi, the 14th December, 2021/*Agrahayana* 23, 1943 (*Saka*)

The following Act of Parliament received the assent of the President on the 13th December, 2021, and is hereby published for general information:—

THE DAM SAFETY ACT, 2021
No. 41 OF 2021

[13*th December*, 2021.]

An Act to provide for surveillance, inspection, operation and maintenance of the specified dam for prevention of dam failure related disasters and to provide for institutional mechanism to ensure their safe functioning and for matters connected therewith or incidental thereto.

DRIP

Challenges in Implementation of Dam Safety Act in India

Simla

20th March2025

Vijai Saran

Former Chief Engineer,
Central Water Commission

History of Dam Building in India

- India has a long history of water management systems for efficient water storage and distribution, including traditional step wells, tanks, as well as construction of dams and canals.

- The Chola dynasty is renowned for its contributions to dam construction. The Kallanai Dam built on Kaveri River in Tamil Nadu during 1st Century AD is considered one of the oldest functional dams in the world.

- The Kallanai dam is a massive structure constructed with uneven stones to a length of 329 m and a width of 20 m across the mainstream of the river.

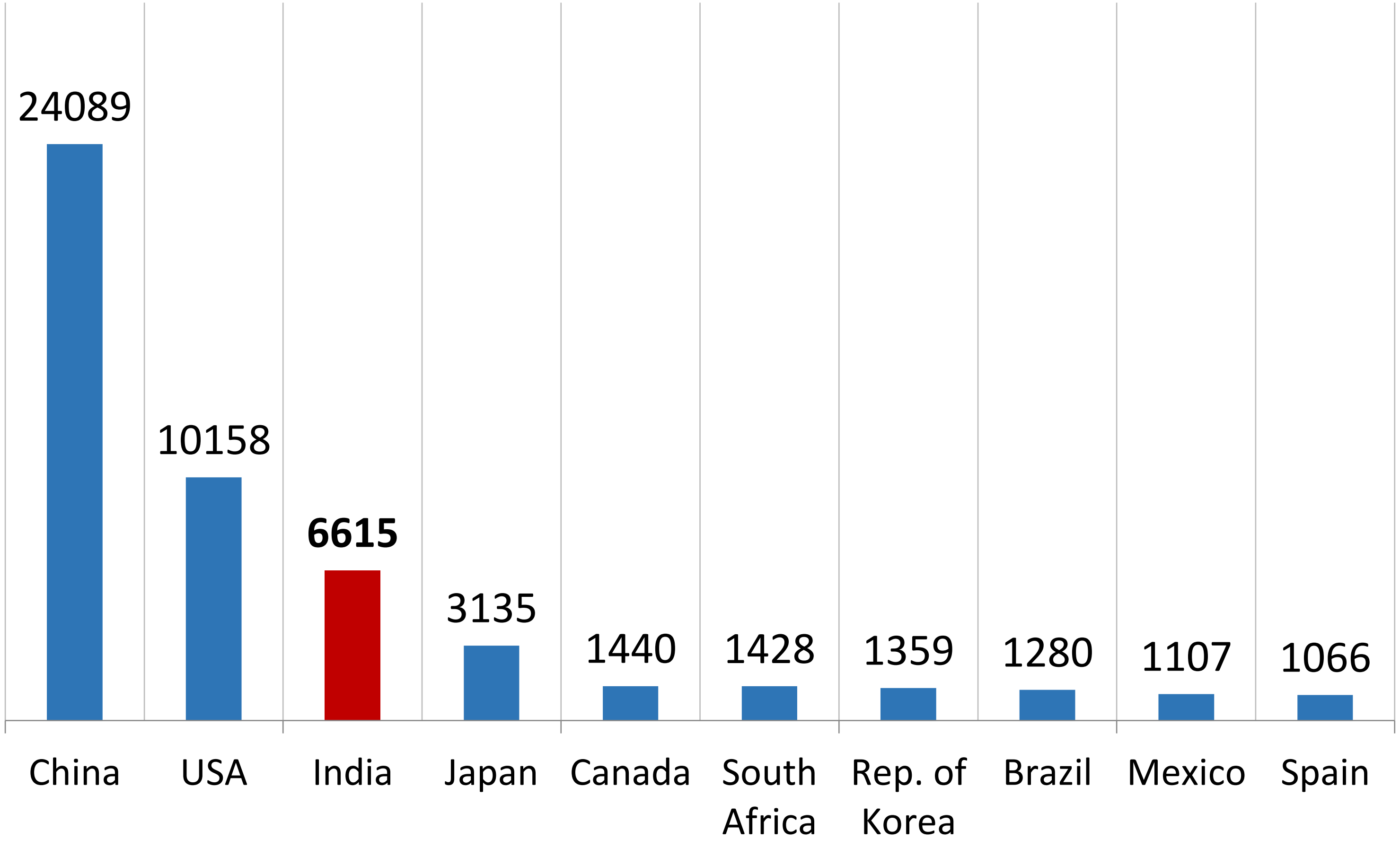
Grand Anicut (Kallanai Dam), 1st Century AD

Building Dams – Current Status

Post-Independence, India embarked on a massive dam-building program to support agriculture, develop hydropower, to manage floods and water supply for growing urban populationS.

India stands 3rd in terms of Large Dams worldwide having **6547 operational large dams** and **68 under-construction large dams**

World wide top 10 dam owning countries



Source: ICOLD & DHARMA

Operational Large Dams in India

In India, by and large, dams are owned, constructed and maintained by the State Governments, CPSUs and few Private Agencies.

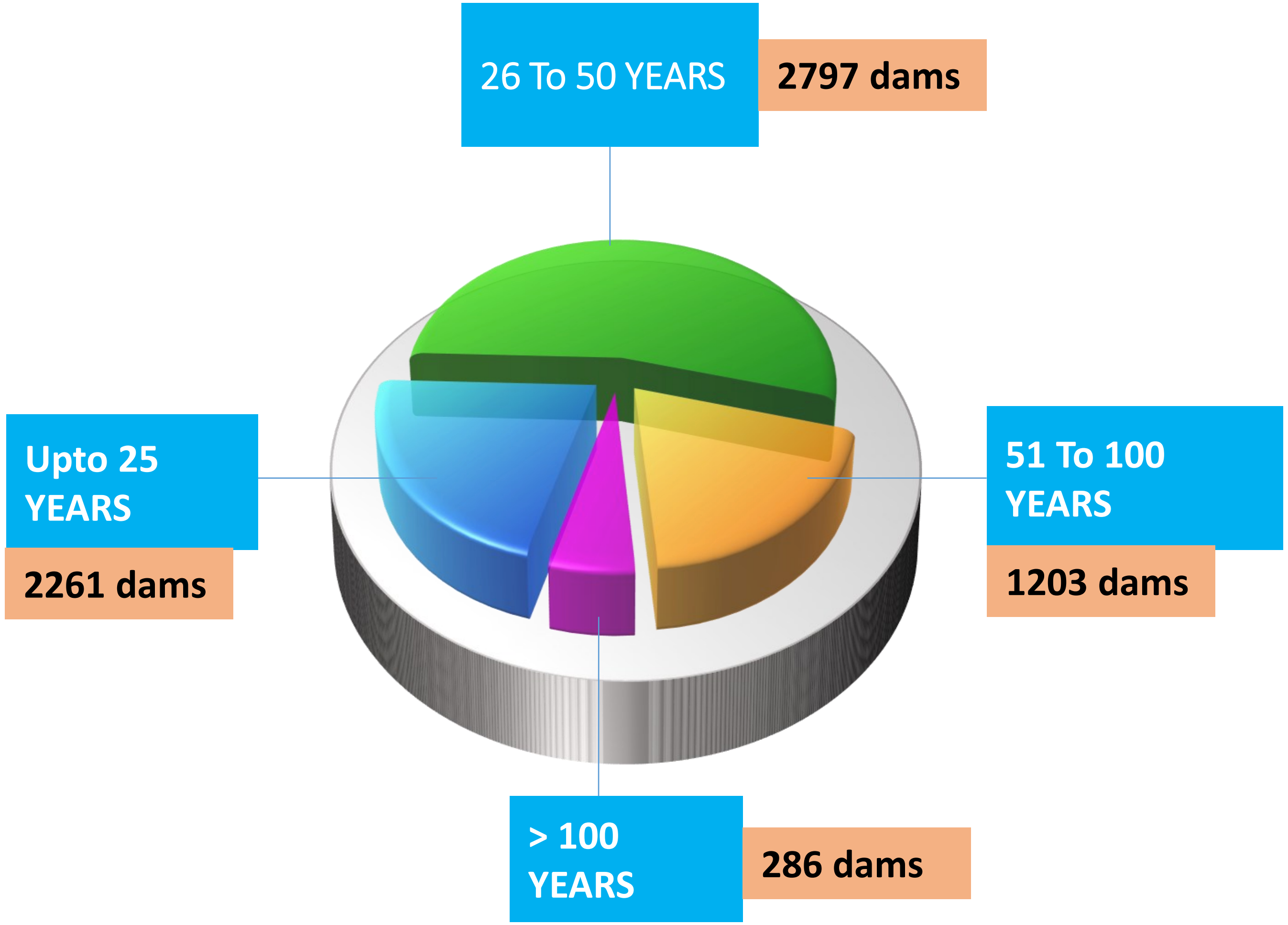
States/UTs	No. of dams
Maharashtra	2679
Madhya Pradesh	1369
Gujarat	525
Chhattisgarh	322
Rajasthan	315
Karnataka	231
Odisha	221
Telangana	174
Andhra Pradesh	170

States/UTs	No. of dams
Uttar Pradesh	142
Tamil Nadu	123
Jharkhand	73
Kerala	65
Uttarakhand	38
West Bengal	36
Himachal Pradesh	30
Bihar	28
Other States/UTs	74

TOTAL	6615

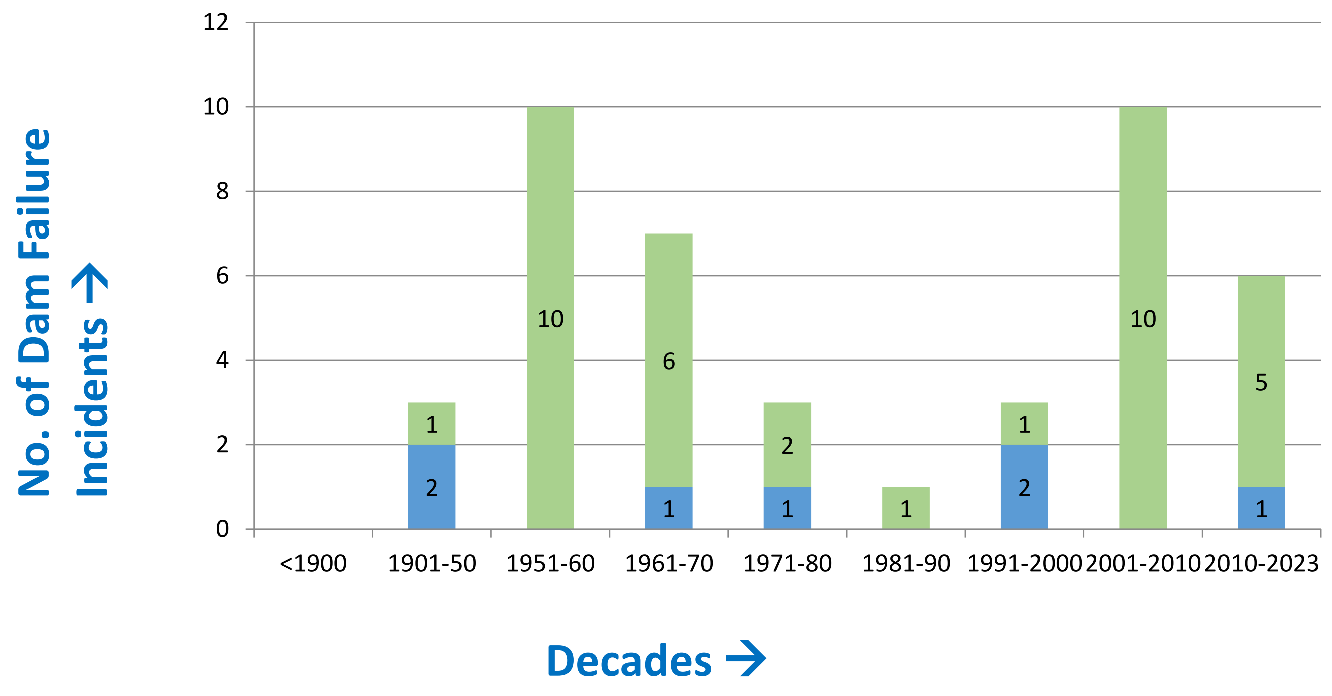
Source: DHARMA

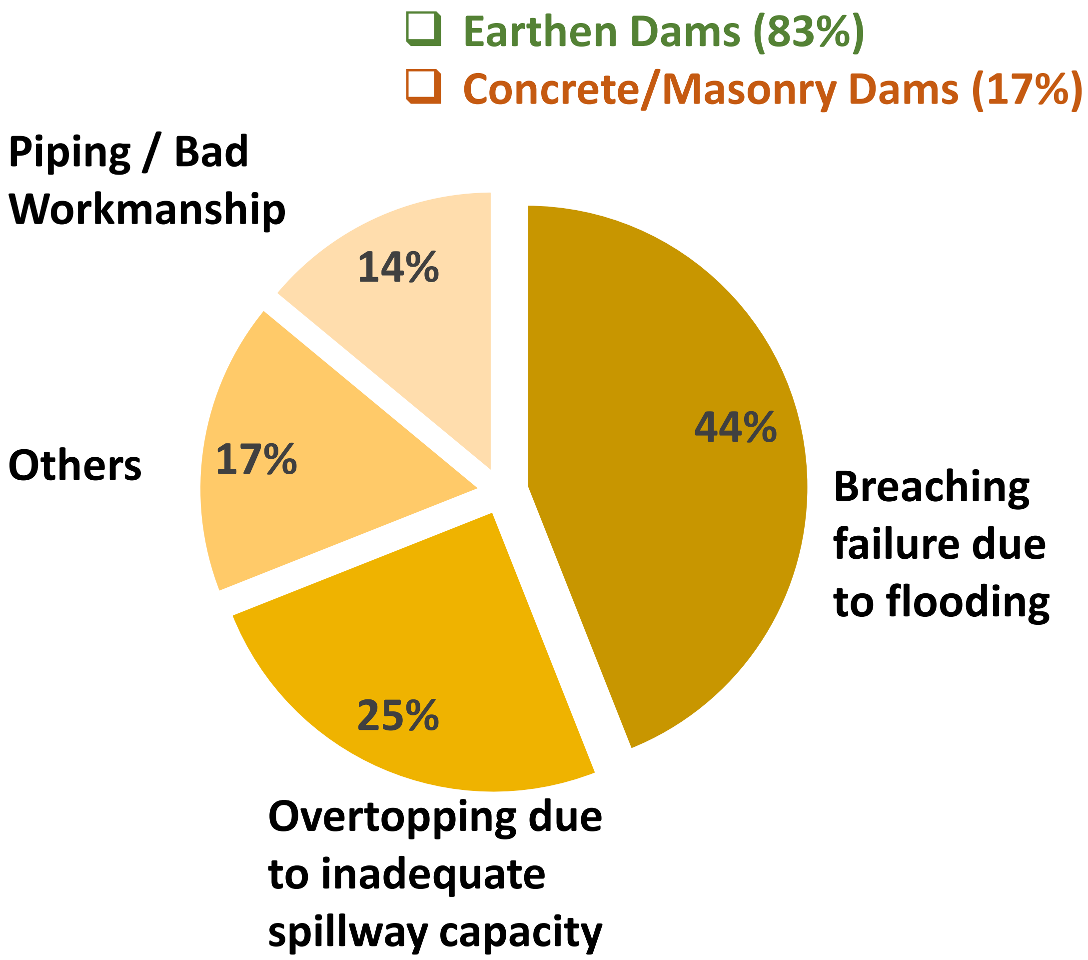
Age of Large Dams in India



Source: DHARMA

Dam Failures in India





- The first dam failure was recorded in Madhya Pradesh in 1917 when the Tigra dam failed due to overtopping.

- The worst dam disaster was the Machu dam (Gujarat) failure in 1979, in which many people were reported dead.

✓ 43 No. of reported dam failures
✓ 26 dams were of height less than 25 m
✓ 36 were Embankment Dams

DAM SAFETY MECHANISM PRIOR TO DSA-2021

Prior to enactment of the Dam Safety Act (2021), various committees, organizations, institutions of Central & State Govts. were working to ensure the safety of the dams across the country. Dam Rehabilitation Programs were also launched with the similar objectives.

1. Dam Safety Organizations(DSO), CWC

- established in 1979 in CWC
- working towards development of uniform dam safety protocols by laying down the standard guidelines/ manuals.
- To assist the State Govts. and perform a coordinating and advisory role for States.
- To create dam safety awareness among the Stakeholders.

2. National Committee on Dam Safety (NCDS)

- setup in 1987
- to evolve uniform simplified dam safety procedures
- forum for exchange of views on issues pertaining to dam safety amongst various stakeholders

3. State Dam Safety Organizations (SDSO)

- SDSO in18 States and 4 other dam owning agencies viz. NHPC, BBMB, DVC, KSEB were working before the enactment of the Dam Safety Act, 2021

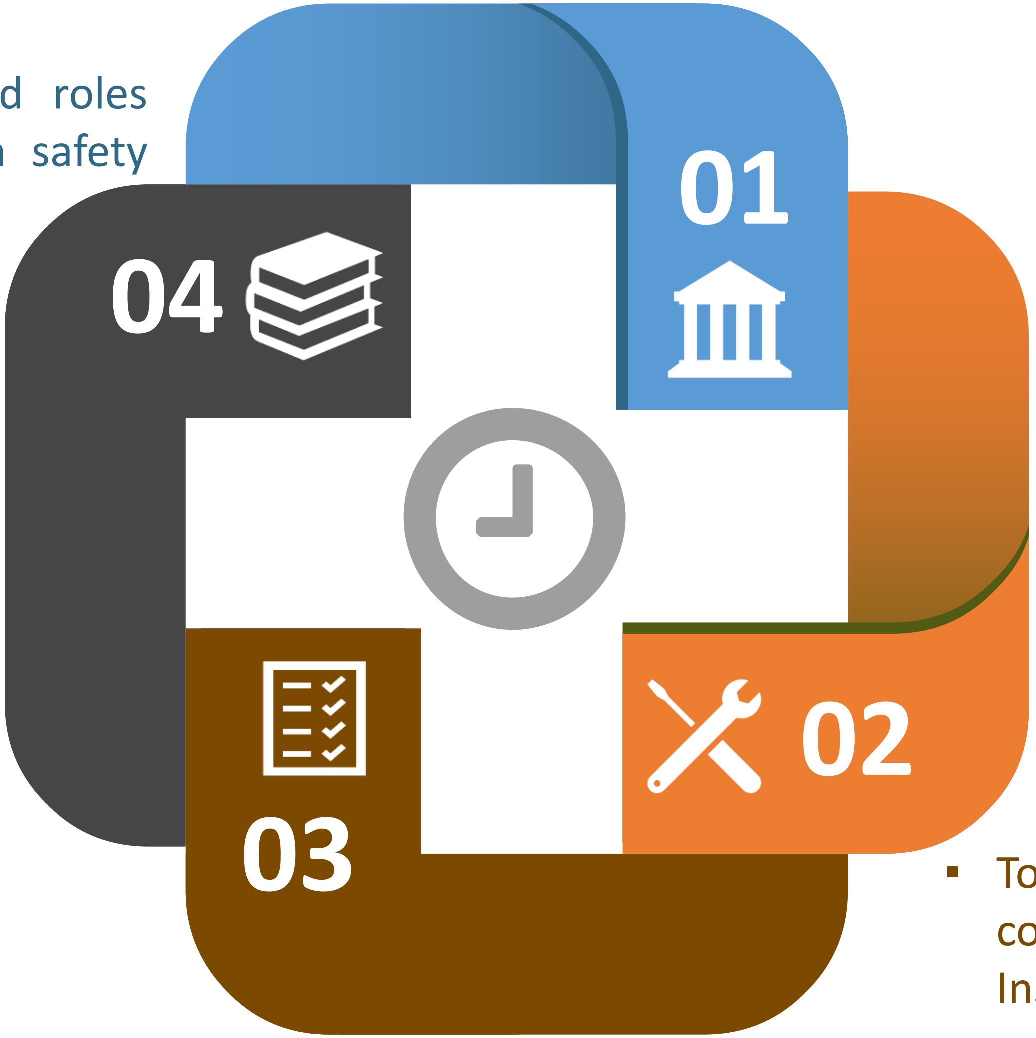
4. Dam Rehabilitation Programs

- Dam Safety Assurance and Rehabilitation Project (DSARP) (1991-99)
- Dam Rehabilitation and Improvement Project (DRIP) Ph-I (2012-21)
- DRIP Ph-II & III (2021-31)

NEED FOR DAM SAFETY LEGISLATION

- To create institutional setup across the country
- To formulate well defined roles and responsibility of dam safety stakeholders

- To address planning inadequacies, structural defects, deficiencies arising on account of ageing of dams.
- To ensure sufficient budget allocation for routine maintenance and repair of the dams



- To standardize dam safety protocols for implementation across the county using latest State of the Art Technologies and best global dam safety practices

- To ensure proper documentation comprising of O&M Manuals, EAPs, Inspection logbooks etc. for each specified dam

JOURNEY OF DAM SAFETY LEGISLATION

efforts of last 40 years resulted into the enactment of the Dam Safety Act, 2021 for the safe future of Indian dams

1982

Standing Committee was constituted to review the existing practices and to evolve unified procedure for safety of dams

1986

Standing Committee report recommended for creation of institutional arrangement as well as need for Dam Safety Legislation

2002

Comprehensive Draft Dam Safety Bill was prepared in 2002 and circulated to the State Governments

2010

Dam Safety Bill (DSB) was introduced in the Parliament in 2010 and was referred to Standing Committee

2021

Incorporating the recommendations of Standing Committee (2010-11), DSB 2019 was tabled in Parliament in 2019 and passed in 2021

Dam Safety Act, Dec. 2021

To evolve uniform dam safety procedures across the country

Applies on dams with height more than 15 m, or height between 10 m to 15 m with certain design and structural conditions

To provide surveillance, inspection, operation and maintenance of the dam for prevention of dam failure related disasters

To provide institutional framework at Central level and State level

DAM SAFETY ACT, 2021

Institutional Setup

National Committee on Dam Safety (NCDS)

To evolve dam safety policies and recommend necessary regulations and maintain standards of dam safety

National Dam Safety Authority (NDSA)

To implement the policy, guidelines and standards evolved by the NCDS

DSA, 2021

State Committee on Dam Safety (SCDS)

To supervise the work of SDSO and review the progress on measures recommended in relation to dam safety.

State Dam Safety Organization (SDSO)

To keep perpetual surveillance, carry out inspections and monitor the O&M of all specified dams

Important Provisions of the Act

uniform dam safety procedures to be followed by the dam stakeholders

Section 35, Section 38

carry out risk assessment, comprehensive dam safety evaluation

Section 27, Section 36

Prepare initial reservoir filling plan, emergency action plan

Section 32, 33,34

minimum number instrumentations and record of readings, establish hydro-meteorological & seismological stations

Section 31

carryout pre-monsoon and post-monsoon inspections in accordance with the guidelines and check-lists

Section 16, 17

keep perpetual surveillance, carry out inspections and monitor the O&M and classify vulnerability and hazard classification criteria of dams

Section 18, 28

maintain logbook/database, ensure that a well-documented operation and maintenance manual exists.

Section 21

earmark sufficient and specific funds for maintenance and repairs of the specified dam and to implement the recommendations of the SDSO

Section 22

compile all technical documentations and equip with the state-of-the-art information technology tools to store, retrieve, and distribute the data

Provisions of Dam Safety Act vis-à-vis Training needs

Item	Section	Duties and Functions	Expertise/Training Needs
Surveillance and inspection	16	Every SDSO shall a)keep perpetual surveillance; b)carry out inspections; c)monitor the Operation & Maintenance of all specified dams falling under their jurisdiction to ensure continued safety of such specified dams and take such measures as may be necessary to address safety concerns that are noticed with a view to achieve satisfactory level of dam safety assurance as per such guidelines, standards and other directions on dam safety as may be specified by the regulations.	• Dam Safety Inspection • Preparation of O&M Manual
Vulnerability and hazard classification of dams	17	classify each dam under their jurisdiction as per such vulnerability and hazard classification criteria	• Vulnerability and Hazard classification criteria of Dams

Provisions of Act vis-à-vis Training needs (contd.)

Item	Section	Duties and Functions	Expertise/Training Needs
Initial filling of reservoirs	27	Before initial filling of any reservoir of a specified dam, the agency responsible for its design shall draw the filling criteria and prepare an initial filling plan, with adequate time for monitoring and evaluating the performance of the dam	• Preparation of Initial filling plan of reservoir
Operation and maintenance (O&M)	28	Every owner of specified dam shall ensure that a well-documented operation and maintenance manual is kept at each of the specified dams and are followed at all times.	• Preparation of O&M Manual for dam; Rule Curves • Reservoir Routing studies; Gate operation schedule
Inspection	31	• undertake every year, through their dam safety unit, a pre-monsoon and post-monsoon inspections in respect of each such dam • carry out all inspections in accordance with the guidelines and check-lists as may be specified by the regulations; • forward the inspection report by to the SDSO	• Inspection methodology • Safety Inspection of Dams • Filling of checklist of Dam Safety inspection form • Preparation of Inspection Report • Best Practices for Safety Inspection of Dams

Provisions of Act vis-à-vis Training needs (contd.)

Item	Section	Duties and functions	Expertise/Training Needs
Instrumentation in specified dams	32	• All specified dam shall have a minimum number of such instrumentations, and installed in such manner as may be specified by the regulations for monitoring the performance of such dam. • a record of readings of the instrumentations shall be maintained and analyzed	• Dam Instrumentation - Types, Number, Location, Specifications, Installation & Commissioning • Analysis of dam instrumentation results for assessment of dam's health
Establishment of Hydrometeorological Stations	33	• All specified dam shall have Hydrometeorological Stations in its vicinity • shall collect, compile, process and store data	• Establishment of Hydro-meteorological stations – Type, Location, Specifications, Installation & Commissioning • Gauge & Discharge measurement • Collection, Compilation, Processing and Storage of Data
Installation of seismological station	34	• To install seismological station at each specified dams having height more than 30 meters or falling under seismic zone • shall collect, compile, process and store data	• Seismic Instruments – Type, Number, Location, Specifications, Installation & Commissioning • Collection, Compilation, Processing and Storage of Data

Provisions of Act vis-à-vis Training needs (contd.)

Item	Section	Duties and functions	Expertise/Training Needs
Risk Assessment Studies	35	• To carry out risk assessment studies at time interval specified by the regulation • the first risk assessment studies shall be carried out within five years from the date of commencement the Act	• Carrying out Risk Assesment studies to prioritise repair and budget • Hazard and Vulnerability Classification of Dam
Emergency Action Plan (EAP)	36	• To prepare EAP before allowing the initial filling of the reservoir and thereafter update at regular intervals • To prepare EAP within five years from the date of commencement of the Act for existing specified dams and thereafter update at regular intervals	• Dam Break Studies and Flood Inundation Mapping • Modelling Tools (HECRAS, MIKE etc) • Flood forecasting, Reservoir Routing Studies
Comprehensive Dam Safety Evaluation (CDSE)	38	• make or cause to be made CDSE of each specified dam through an independent panel of experts • first CDSE for each existing specified dam shall be conducted within five years from the date of commencement of the Act	• Review and analysis of available data on design, construction, operation, maintenance and performance of structure • Assessment of Hydraulic & Hydrologic condition, Design Flood Review • Assessment of Seismic Safety • Suggestion of relevant remedial measures

DSA, 2021 – Major Activities

Year	Pre-Monsoon Inspection	Post-Monsoon Inspection
2022	3,919	5,024
2023	6414	5881
2024	6366	6428

Pre-Dam Safety Act Period
1200 (Annual)

Pre & Post-monsoon Inspection of Dams & Follow-up

During pre-monsoon inspection of 2024, 3 dams were under Cat I and 180 dams were under Cat II. During post-monsoon inspection of 2024, 3 dams are reported as Cat I and 187 dams under Cat II.

NDSA takes up the matter with the concerned States having Category-I (rehabilitation of both dams proposed under DRIP) and Category-II dams (89 dams proposed under DRIP) for rehabilitation

Category-I: Deficiencies in dams which may lead to failure (If left unattended)
Category-II: Major deficiencies in dams requiring prompt remedial measures
Category-III: None or minor remedial measures in dams which are rectifiable

Dam Safety Rehabilitation Programs

- Externally aided Dam Rehabilitation Programs.
- Helped the dam-owning agencies by providing the latest state-of-the-art dam rehabilitation techniques, institutional strengthening and funds for rehabilitations of selected dams.

Dam Safety Assurance & Rehabilitation Project (1991-99)

Dam Rehabilitation & Improvement Project (2012-2021)

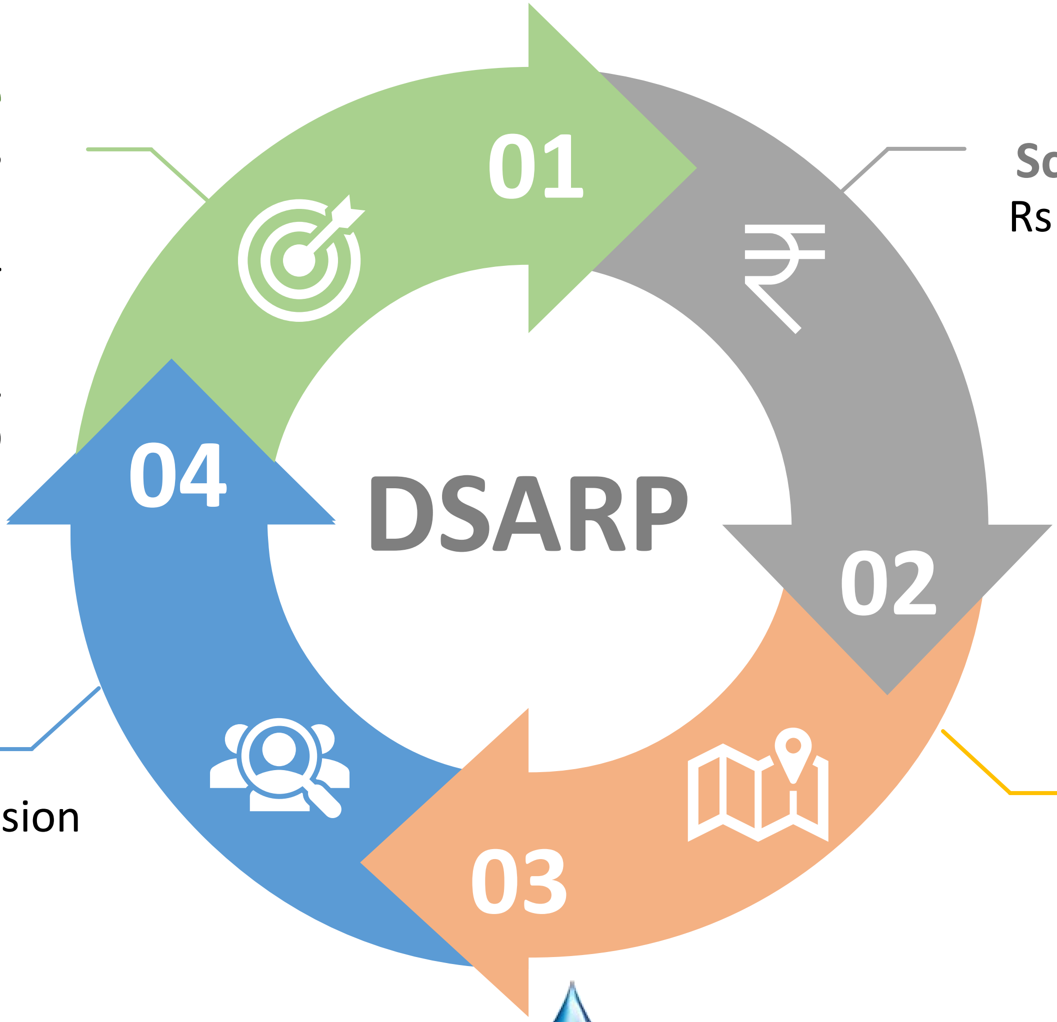
Dam Rehabilitation & Improvement Project Phase-II & III (2021- 2031)

Dam Safety Rehabilitation Programs

DAM SAFETY ASSURANCE AND REHABILITATION PROJECT (1991-1999)

Objective

To improve the safety of dams through physical rehabilitation works, install basic dam safety-related facilities and Institutional strengthening & capacity building of the SDSO



Scale

Rs 422.95 Cr Project Cost; 33 dams

Coverage

Assisted by World Bank, implemented in 4 States - Madhya Pradesh, Odisha, Rajasthan and Tamil Nadu

Supervision

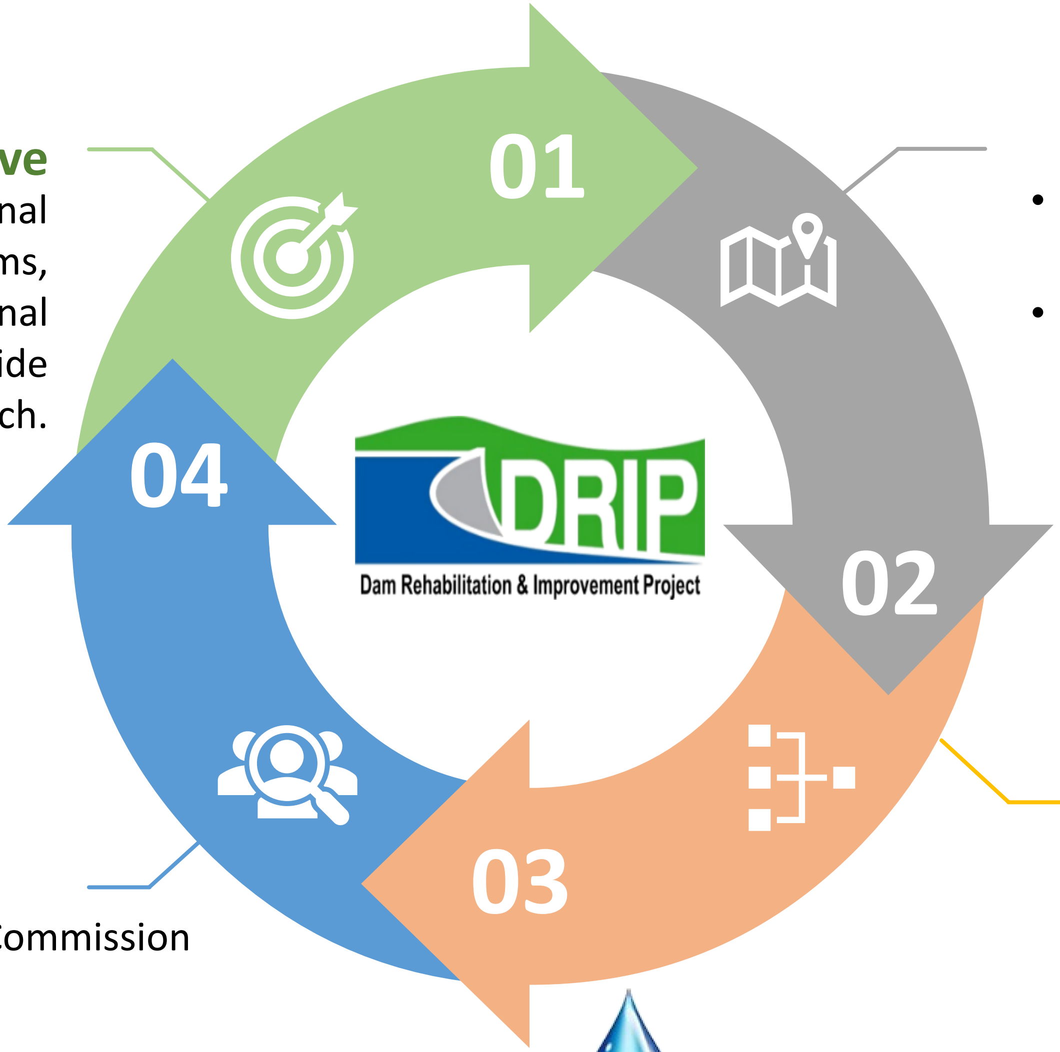
Central Water Commission

Dam Safety Rehabilitation Programs

DAM REHABILITATION AND IMPROVEMENT PROJECT (2012-2021)

Objective

To improve safety and operational performance of selected dams, along with institutional strengthening with system wide management approach.



Scale & Coverage

- Rs. 2567 Cr Project Assisted by World Bank.
- **Rehabilitation of 223 dams located in 7 States** - Kerala, Tamil Nadu, Karnataka, MP, Odisha, Jharkhand & Uttarakhand

Components

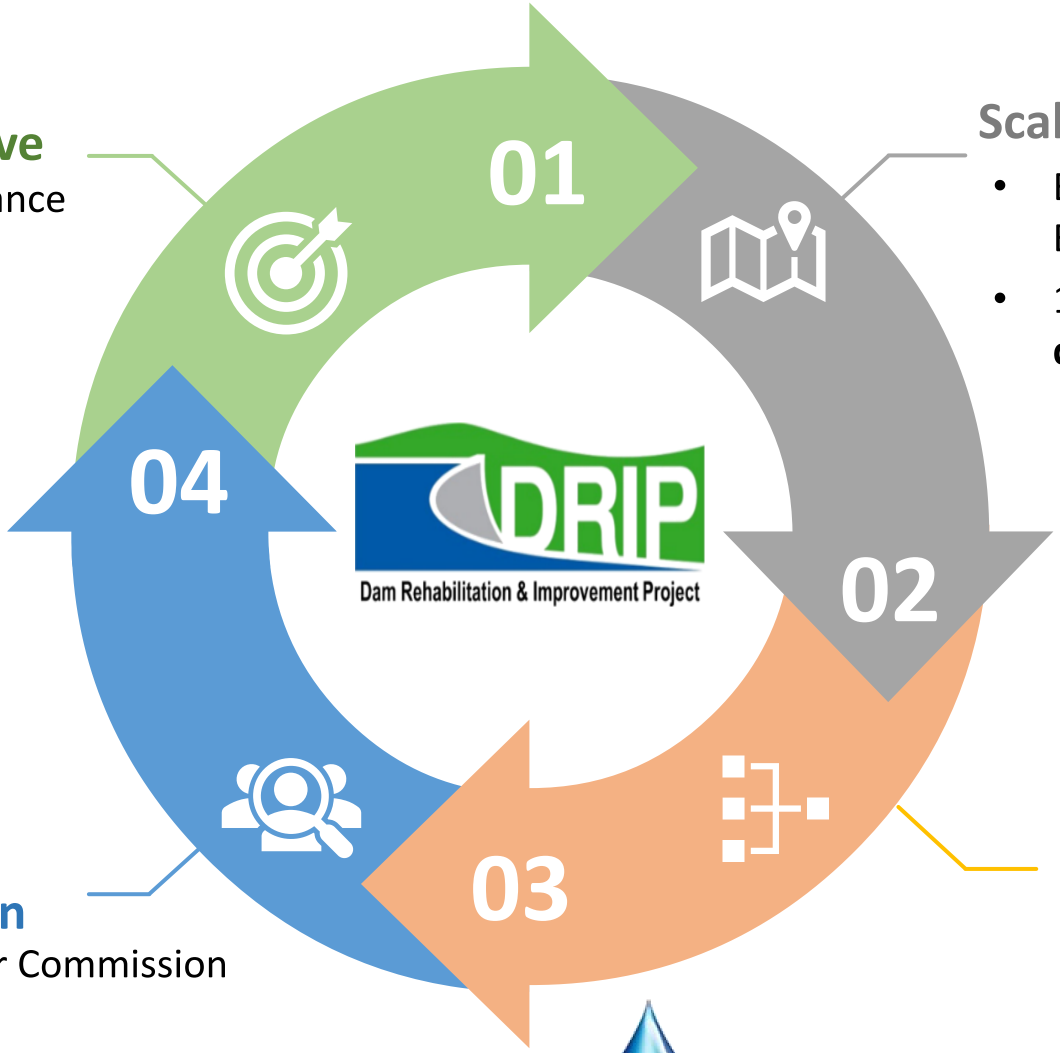
- Rehabilitation of Dams And Associated Appurtenances
- Dam Safety Institutional Strengthening
- Project Management

Supervision

Central Water Commission

Dam Safety Rehabilitation Programs

DAM REHABILITATION AND IMPROVEMENT PROJECT PHASE II & III (2021-2031)



Objective

To improve the safety and performance of dams and associated appurtenances in a sustainable manner.

Scale and Coverage

- Budget of Rs. 10211 Cr with World Bank assistance of Rs 7000 Cr.
- 19 States, 3 Central Agencies, **736 dams**

Components

- Rehabilitation of Dams and Associated Appurtenances
- Dam Safety Institutional Strengthening
- Incidental Revenue Generation for sustainable operation and maintenance of dams
- Project Management

Supervision

Central Water Commission

Dam Safety Rehabilitation Programs

DAM REHABILITATION AND IMPROVEMENT PROJECT PHASE II & III (2021-2031)

Present Status:

- Loan signed in August 2021 and scheme declared effective in October 2021 by World Bank.
- AIIB loan agreement(US$250 Million) signed in May 2022; effective on 29th Dec 2022
- 14 States on board. (Gujarat, Kerala, MP, Maharashtra, Manipur, Meghalaya, Raj, Odisha, Tamil Nadu, Chhattisgarh, Karnataka, Uttarakhand, UP, WB).
- 4 States (Punjab, Andhra Pradesh, Telangana, Jharkhand) yet to fulfil world bank readiness conditions to join.
- 3 agencies (BBMB, DVC and Goa) to join shortly.
- Under the Scheme, tenders amounting approx. Rs 2909.61 Cr published, and contract(s) amounting Rs 2429.38 Cr awarded.
- The expenditure incurred so far is approx. Rs 1564.65 Cr.

Aligning DRIP II & III with Dam Safety Act

- Inspections, Maintenance, Repair, Rehabilitation & Retrofitting works to improve the safety performance of dams
- Hydrological Safety particularly in the context of climate change
- Managing Sedimentation of reservoirs
- Instrumentation in existing dams
- Seismic Safety
- Uniform protocols and documentation:
    - Risk Assessment
    - Comprehensive Dam Safety Evaluation,
    - Preparation of Rule Curves, Operation and Maintenance (O&M) Manuals
    - Dam Break Analysis, Emergency Action Plan (EAP)
- Institutional Capacity Building
- Manpower and Budget Availability

DRIP - Key Achievements

Institutional Strengthening

- Institutional setup was strengthened for a sustainable dam safety culture.
- Central Project Management Unit: At National level
- State Project Management Unit: At State level
- Project Authorities: At the dam level

Uniform dam safety procedures

- 13 Guidelines/Manuals were published to ensure uniform/standardised dam safety practices/ protocols/guidelines/manuals across the country
- Prepared 217 EAPs and 221 O&M Manuals

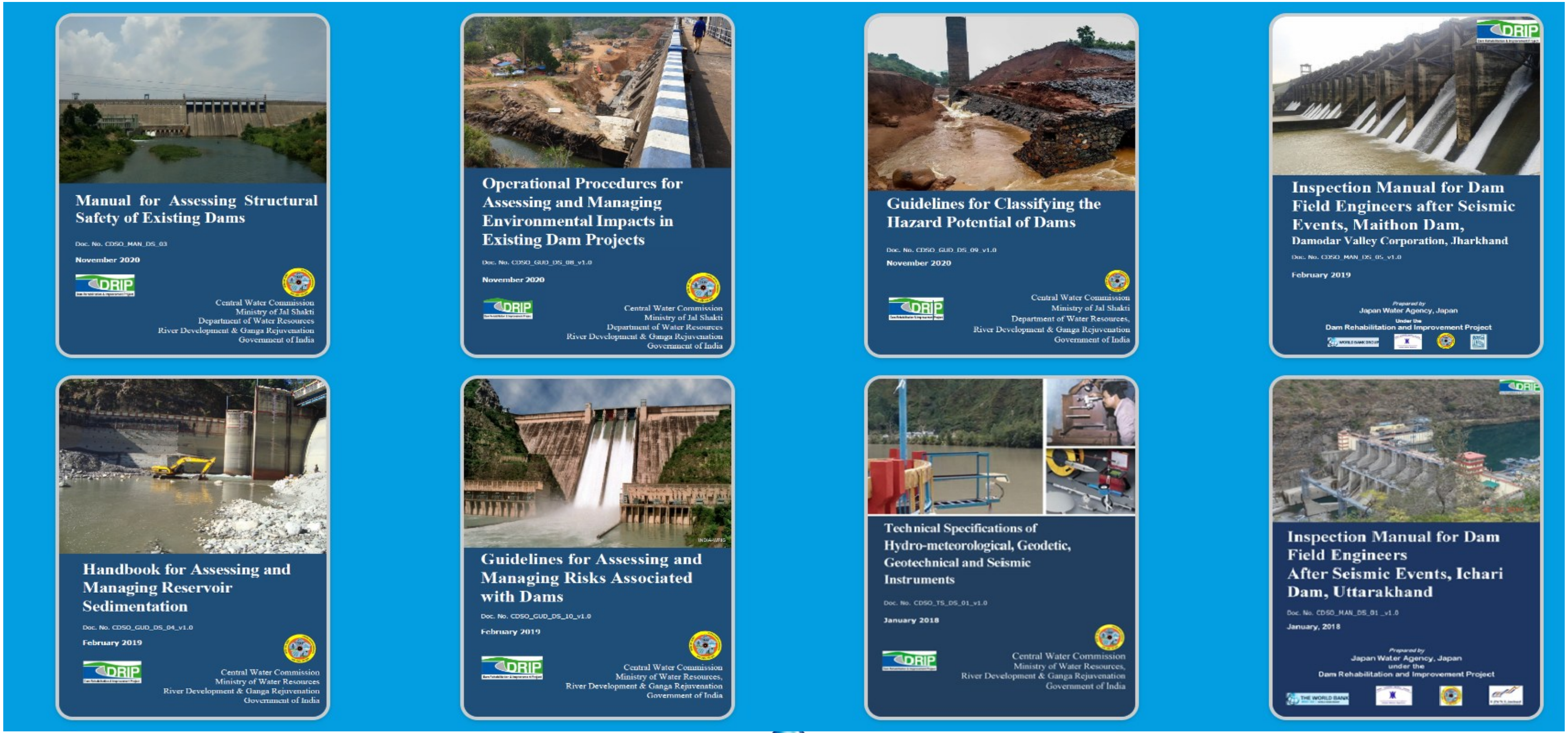
Capacity Building

- 186 National and international trainings organized benefitting about 5500 officials
- 3 National Dam Safety Conferences and 2 International Dam Safety Conferences organized.
- M. Tech courses in Dam Safety started at IIT Roorkee and IISc Bangalore

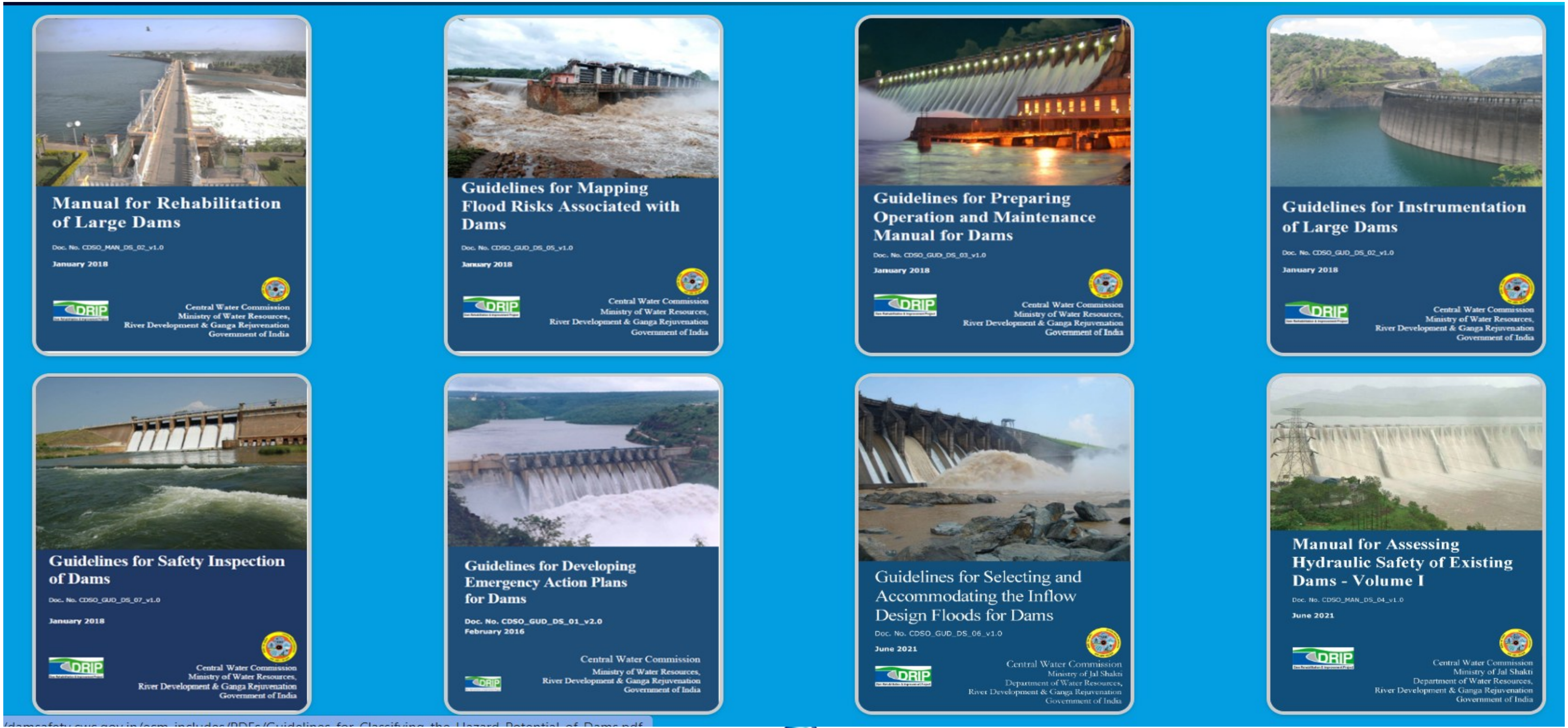
Dam Health And Rehabilitation Monitoring Application (DHARMA)

- A web-based asset management tool to support effective collection and management of asset and health data of dams
- Multilevel access at Central, State and Project Levels

Publications under DRIP



Publications under DRIP



Available at damsafety.cwc.gov.in

Structural Issues and State of Art Solutions

Seepage Control

Seepage Control at Malaprabha dam, Karnataka

Initial observed seepage (in gallery) = 850 L/min

- ✓ Dam body & foundation grouting
- ✓ U/s face racking pointing with UV resistant high strength mortar including under water.
- ✓ Seepage after treatment (gallery) = 90 L/min

Seepage Control at Servalar dam, Tamilnadu

Initial observed seepage (gallery) = 742.65 L/min

- ✓ Left flank and overflow section u/s face- Geomembrane
- ✓ Right flank – Racking & pointing plus joint treatment
- ✓ Seepage after treatment (gallery) = 66.34 L/min

Structural Issues and State of Art Solutions

Tackling Hydraulic and Operational safety

Hydraulic Safety at Maneri Dam

Repair of spillway glacis and energy dissipater with M90 concrete (Maneri Dam, UJVNL)

Operational Safety at KRS Dam, Karnataka

Ageing of H-M equipments, gate leakage, surface deterioration, poor water quality

- ✓ Replacement of 136 sluice gates

Structural Issues and State of Art Solutions

Tackling Stability:

Pechiperai Dam, Tamil Nadu

- Seismic Zone - III
- The existing dam was found unsafe under seismic loading
  - ✓ Backing Concrete

Dam Safety Act, 2021- Paradigm shift

- Enactment of Act has made its provisions legally enforceable. Earlier Centre had advisory role in Dam Safety.
- Institutional Strengthening by constitution of bodies at Central (NCDS & NDSA) & State (SCDS & SDSO in all states) level.
- Notification of Dam Safety Regulations by NDSA are mandatory.To be implemented at Pan India level.
- Regular Pre/Post-Monsoon inspections of almost 100% specified dams, i.e. from about 1200 inspections annually before the Act to about 12000 inspections annually after the Act.
- Categorization of dams based on pre and post-monsoon inspections and accordingly planning the prioritized remedial measures for dams at risk.
- Dedicated funds for maintenance and repair of dams.
- Introduced M. Tech courses on dam safety at IIT, Roorkee and IISc., Bengaluru for capacity building
- Establishment of Centre of Excellence for Dams at IIT-Roorkee and IISc, Bengaluru
- Establishment of National Centre for Earthquake Safety of Dams at MNIT, Jaipur
- NDSA has empaneled pool of Dam Safety Experts for different domains
- Capacity building of dam owners through regular training programs at NWA, Pune, CWC etc.

Challenges in implementation of Act

1) Trained manpower and budget requirement for Repair and Maintenance of dams:
2) Institutional:
   - Manning all the Institutions constituted under the Act, both at Central and State level, on regular basis rather than on additional charge basis.
   - Continuous capacity building of all stakeholders
3) Uniform protocols and documentation:
   - Comprehensive Dam Safety Evaluation, Risk Assessment, Emergency Action Plan (EAP), and Operation and Maintenance (O&M) Manual of all the Specified dams (> 6000 dams) to be prepared within 5 years of enactment of the Act.
   - This is a challenge as well as opportunity. Engineering Institutes and private sector to assist state Govt.

Conclusion

- Dam Safety is important to India for protecting the precious assets created and providing safety to the population settled downstream.
- The enactment of DSA 2021 has made Dam Safety mandatory on the part of all stakeholders.
- Capacity building of all stakeholders is necessary for implementation of provisions of the Act.
- Strengthening of Institutional Mechanism is essential for implementation of the Act.
- Availability of adequate funds to undertake proper operation and maintenance and also carry out rehabilitation to ensure dam safety.
- Collaboration with the States in creating the Eco-system in the country on Dam Safety aspects. Dam Rehabilitation Programs have created a favorable dam safety culture in the participating states. It is required to be scaled up.
- International collaborations with expert agencies specially in advanced areas of dam safety such as Risk Assessment and Comprehensive Dam Safety Review etc.

THANK YOU

Presentation for Case Study on "Optimizing Seismic Monitoring Systems for enhancing Safety at Tehri Dam - Uttarakhand"

Presented by

Vinod Taamar – Managing Director

Pinnacle Geosystems , India

Introduction

- The Project is for monitoring the Structural Health of the Tehri and Koteshwar Dam.
- The Project is for monitoring the Structural Health of the Tehri and Koteshwar Dam.
- At Tehri Dam the instrumentation is installed at the Top of Inspection Gallery , AGBR tunnel and Power House and for Koteshwar Dam the instruments are installed at top of the dam, Centre and at foundation of the dam.
- Large concrete dams are infrastructures of vital importance for populations, contributing decisively to the management of freshwater resources, particularly for water supply, energy production, flood control and soil irrigation, especially nowadays due to the effects of climate change. However, these dams are usually high potential risk structures, designed to operate for several decades, and often located in seismic zones. It is essential to conduct a continuous and reliable assessment of dam behaviour, which can be affected by progressive deterioration or by damage caused due to extreme events (such as earthquakes or overtopping), in order to control structural health over time and ensure optimal operating conditions

Key Features of Seismic Monitoring for the Dams.

- Seismic monitoring of the Dams bodies are Critical aspect of Dam Safety and Risk Management ensuring the structural integrity and operational reliability of these infrastructures.
- This process involves the prefeasibility analysis , Site Survey, deployment of advanced instrument such as Strong Motion Accelerograph (SMA) and Broad Band Seismograph strategically positioned on and around the Dam Structure essential to ensure the safety , stability and functionality of these critical structures allowing for the early detection of Stress or Damage by understanding ,recording and analyzing the dynamic behavior of structures response in case of Earthquake.
- These instruments continuously record Seismic Activity , Capturing the Data on Ground Movements and structural response to both minor tremors and significant Seismic Events.
- The Collected Data is further analyzed to access the dam ability to withstand Seismic Forces , identify potential vulnerabilities and guide the implementation of necessary reinforcement or mitigation measures.
- This proactive approach enables the dam authority to assesses the dam condition , implement timely maintenance or reinforcement measures and optimize Emergency response plans accordingly.
- In addition of the above , Seismic Monitoring helps in understanding the dynamic behavior of the Dam and surrounding geological conditions , contributing to improved design standards and risk mitigation strategies.

Why Seismic Monitoring required for Dams

- *The Process of Seismic Monitoring of Dams involves approach to collect , analyze and interpret seismic data to ensure the dams' structural integrity and safety including selection and Installation of Instruments.*
- *The important point is to defined the location at Dams , Foundation , Abutment and surrounding areas and design and establishing the communication network to ensure accurate data collection and timely acquisition , Filtering unwanted noise , Identifying Seismic events.*
- *Finally the Processing and analyze ,setting up the Threshold Setting and Automated Alarm or Trigger System of decision making for the Alertness.*
- *Also Providing the adequate power source with proper battery back up and load balancing.*
- *These instruments are required periodic maintenance and the calibration of the sensor is mandatory to make the instruments trouble free operational.*
- *Further comes the Data Integration , plotting the graph for different parameters , reporting and visualizing to provide a clear understanding of the Dam to the concerned authority.*

Key Points to be remember while planning to Install the Seismic Instruments on the Dam site.

- *Choose a correct location of Strong Motion Accelerograph and Broad Band Seismograph.*
- *Reliable Power back up for the instruments.*
- *Routine Maintenance.*
- *Reliable Communication and Data Availability at Central station.*
- *Large Data Storage Capacity for monitoring and Analyzing the Data.*
- *Timely and corrective action by the Dam authority.*

Installation and Maintenance of the instruments and data.

- *The instruments were installed at Tehri and Koteshwar Dam in the year of 2013.*
- *At the time of installation every instrument had the Broad Band Telephone line connected with Static IP address.*
- *The online data will streamed to IIT Roorkee as IIT Roorkee was the consultant for THDC.*
- *Also we are providing the support system to IIT Roorkee to maintain the whole network.*
- *Initially Our team of Engineers used to visit every month to the dam site to make sure that all the instruments are working properly.*
- *For over 12 years all the instruments are working fine and the data is coming to IIT Roorkee and they are doing the analysis.*
- *Now we have also the dedicated team who can do the data analysis and prepare the report as per the requirement.*

Thank You!

Please ask if you have any queries.

Comparative Study of Statistical Analysis & Dam Safety Compliances at Koldam HPS

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ABSTRACT

Dam Safety has been a domain of universal interest with multi-disciplinary expertise and recent revised and stringent regulations with multi-level monitoring has further highlighted its importance. This paper summarizes comparative statistical analysis of dam failure data across the globe. The failure phenomenon recorded in the historical data were compared with the parameters prevailing at Kodam e.g. most of the dam failures had occurred in the first two years after construction. Koldam is on the safer side as it is approaching about 10 years of age. Several such comparisons were performed with the statistical data regarding type of dam, age of dam, height and storage capacity of dam, Spillway capacity and modernization in dam design philosophy etc.

In recent past, the frequency of dam failure related incidents due to inadequate spillway capacity and mal-functioning of hydro-mechanical equipment has been reported from across the world. Few such unwarranted failures have occurred in India also which led of huge loss of man, material and money. At Koldam, due care has been taken in designing spillway capacity after detailed hydraulic and hydrological studies. Scheduled maintenance and testing of HM equipment is ensured to avoid any such untoward event. Effect of GLOF is also analyzed and it was observed that due to sufficient distance of dam from nearest glaciers and in between dam projects, the spillway is safe from GLOF considerations.

Koldam Hydro Power Station is NTPC's flagship installation in the hydro sector situated on river Satluj in district Bilaspur, Himachal Pradesh, India. The dam safety compliances being followed rigorously at Koldam HPS are also elaborated in the paper. It concludes safe dam condition from the analysis of previous dam failure data.

Koldam HPS (2x 400MW) is managing its operation and maintenance activities along with the compliances of Dam Safety Act 2021 with optimal man megawatt ratio. In fulfilment of provisions of Dam Safety Act 2021, Dam safety unit at Kodam HPS is implementing the dam safety related assignment in true sense. A muti-disciplinary dam safety committee with diversified experienced minds has been set up for holistic examination and resolution of dam safety related issues at site level. Detailed Pre and Post monsoon inspections are being carried out with a systematic approach in a time bound manner and as per CWC guidelines. Apart from Pre and Post monsoon inspections, inspection of dam and appurtenant structures is being carried out on monthly basis in Monsson season and bi-monthly in lean period. An SOP has been framed and implemented for regulating the frequency of magnitude and intensity of various inspections. Inspection of all components of dam complex and reservoir rim is carried out after every critical eventuality e.g. high floods and earthquakes etc.

Proactive measures for dam safety such as information sharing with people downstream of dam, keeping them aware about EAP/DMP, mock drill for safe evacuation, marking of danger water levels, displays at entry points along the river etc. are also summarized in brief. Important dam safety components alike Automatic Weather Station (AWS), Early Warning System (EWS), river inflow and flood forecasting system, hydro-meteorological stations etc. are fully functional. Reservoir siltation study, bathymetry, river depth survey, analysis of instrumentation data etc. are being undertaken regularly. Based on the Statistical analysis of dam failure data as well as compliance patterns of Dam Safety Act 2021, it can be safely concluded that Koldam is truly safe in all the aspects.

INTRODUCTION

A famous proverb says, “to err is human” Meaning it’s basic human nature to meet errors. But in reference to dam safety even a small error can lead to a massive devastation, so there is no scope of error while dealing with dam safety. Hence the responsibility and importance both of such a critical topic is very pertinent in today’s scenario.

Dams lay the foundations of a developed nation. As per the World Register of Large Dams published by ICOLD (International Commission On Large Dams) there are about 62,000 large dams in the world. India is at 3rd seniority position after China and USA. As per National Register of Large Dams (NRLD) published by Central Water Commission (CWC), a nodal agency in India for all Hydro Projects in the country, there are about 6000 large dams in India. Out of which about 70% are older than 25 years of age, 20% are between 50 to 100 years and 5% are more than 100 years old.

Hence the importance of dam safety increases exponentially while addressing the issues in the Indian context. As per ICOLD definition of large dams, dams having height of more than 15 meters or pondage more than 3 MCM are considered as large dams. Similar structures are addressed as Specified Dams in Dam Safety Act 2021 with minor modifications in the definition. Even in the small state of Himachal Pradesh there are 23 Specified Dams.

Sl No	Country	No. of Dams
1	China	23841
2	United States of America	9263
3	**India**	**6138**
4	Japan	3130
5	Brazil	1365
6	Korea (Republic)	1338
7	South Africa	1266
8	Canada	1156
9	Mexico	1079
10	Spain	1064

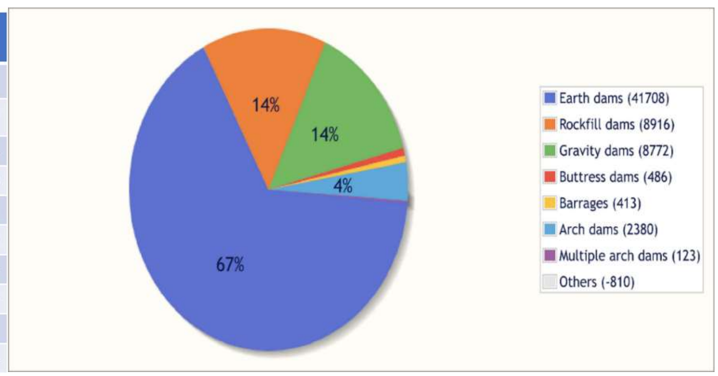


Figure 1: Dam Count and Distribution of Type of Dams

Figure 1 depicts the distribution of types of dams across the world. Most of the dams in the world are earthen dams (67%). Rockfill and gravity dams share an equal proportion of population of total dams. The Koldam Hydro Power project is basically a run-of-the river scheme with storage facility for the initial 30 years. Koldam HPS envisages utilization of a head of about 140 m by constructing a 167m high (from the deepest foundation) rock/ gravel fill dam with impervious central clay core and a dam toe powerhouse with installation of 4 units of 200 MW each. The Gross capacity at FRL is 576 MCM and the dead storage capacity is 486 MCM.

As far as dam failures across the globe are concerned, the single largest dam failure mechanism is reported as overtopping. As per Association of State Dam Safety officials (ASDSO) data, the second largest mechanism triggering dam failures is marked as unknown, embarking that a lot of further research is required to ascertain the reasons of dam failures. This indicates the quantum of uncertainty in this field and need of strengthening the understanding and practice of dam safety. Thus, the role of such critical avenues to discuss dam safety in details is manifold and manifested.

After the recent dam failure events of Libya and Sikkim (India), the common public and media are raising many questions about the dam safety, responsibility of state, role of dam owners, preparedness to handle disasters caused by dam failures and many more issue.

DAM FAILURE STATISTICS

Dam failure data published by ICOLD and other sources were analyzed and inferred in relation to the prevailing parameters at Koldam. A holistic analysis of different dam failure mechanisms was done and results were compared with the site conditions prevailing at Koldam.

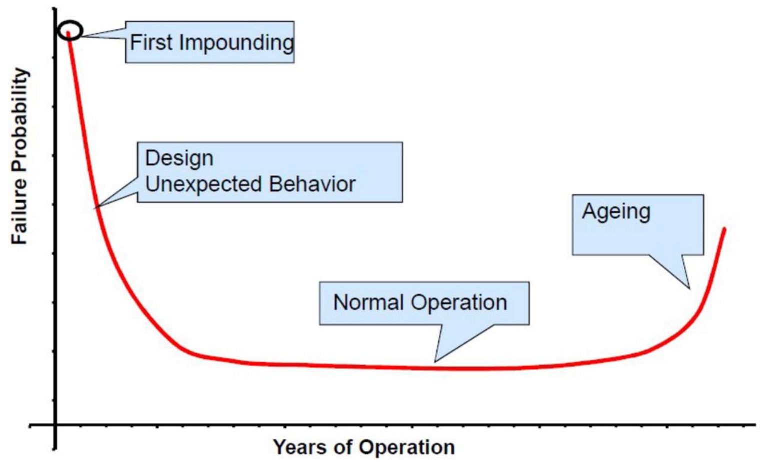


Figure 2: Probability of failure along the time (R. Melbinger, 2018)

Figure 2 shows the probability of failure of a dam v/s its year of operation. It is observed that the highest probability of failure of dam is during the first impounding followed by the initial period when failures may occur due to unexpected design behavior of structure. Therefore, the relationship between the age of the dam and the risk of an incident depending upon the stage of the dam needs to be evaluated. At Koldam smooth impounding was done in 2014-15 and it is running in 10th year of normal operation period, so the failure probability shall be lowest.

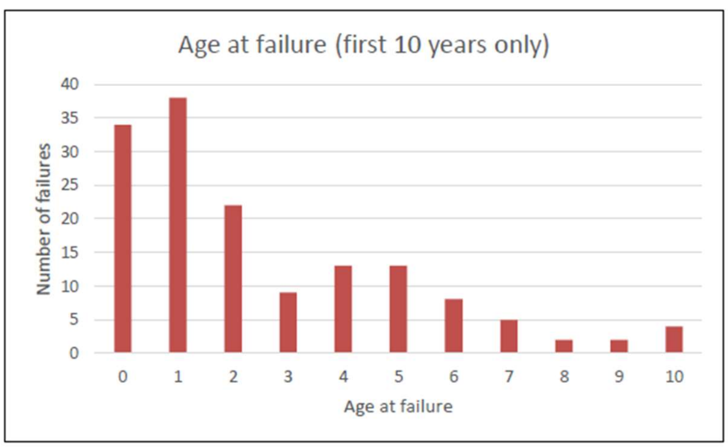
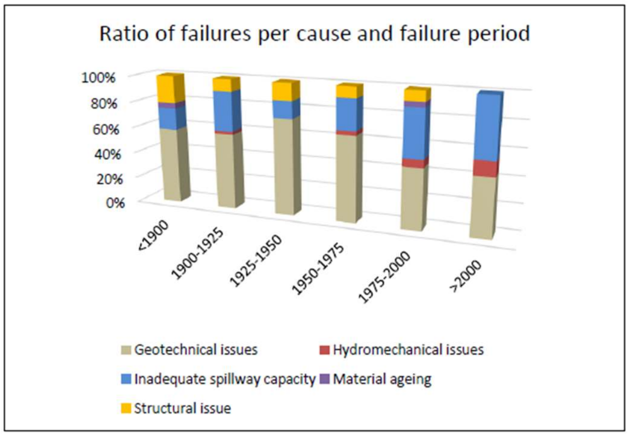


Figure 3: Age at Failure and Number of failures

Figure 3 depicts the relation between the number of dam failures and the age of dam. As per statistical data published by ICOLD, maximum dams fail within the first 10 years of age and about 50% of dam failures had occurred within 2 years of age. The construction of dams and impounding of reservoirs are two very crucial phases in dam safety. The above data of ICOLD demonstrate that a dam would be in a relatively safer zone if it had sustained for the first ten years.

The Koldam Project is about to cross the age of 10 years from impounding and as inferred from data analysis, there is very less probability of failure. Further, as per ICOLD data regarding dam failures and height of dam, only 0.13% of dams have failed with height more than 100m. The main reason being that dams of such height are designed and constructed with best practices in the field and therefore the chances of their failure are the least.



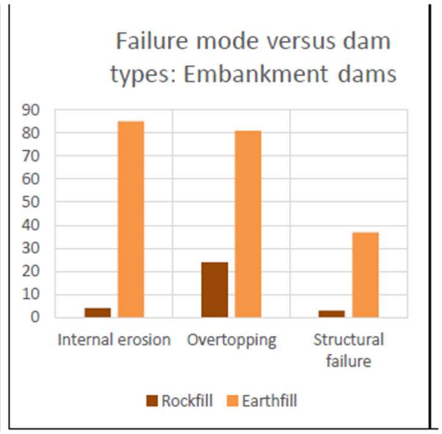


Fig: 4: Failure per cause and failure period, failure modes of embankment dams

Figure 4 shows the failure causes of dams in different construction periods and failure modes of embankment dams. It has been observed from the dam failure pattern data that in earlier times say up to 1975 or maximum up to year 2000, most of the dam failures are either due to geotechnical issues or due to structural issues. Due to improved understanding and analysis of geotechnical and structural issues, most of the dam failures after 1975 or 2000 have happened due to inadequate spillway capacity which has grown up to 50% since 2000. This effect is probably due to the climate change scenario across the globe and here comes the role of accuracy in predicting hydro meteorological events.

Further, from the above data it is evident that the failure rate of rock fill dams is about 10 to 20% of failure rates of earthfill dams. If the failure by dam type is considered, about 1.39% of rockfill dams have failed due to various reasons.

Koldam being a rockfill dam with design capacity of spillway as 16500 cumec corresponding to PMF (Probable Maximum Flood) will lead to lowest possible chances of failure. Being very far from the glaciers, the dam is not affected by GLOF and GLOF has an almost negligible contribution towards spillway PMF.

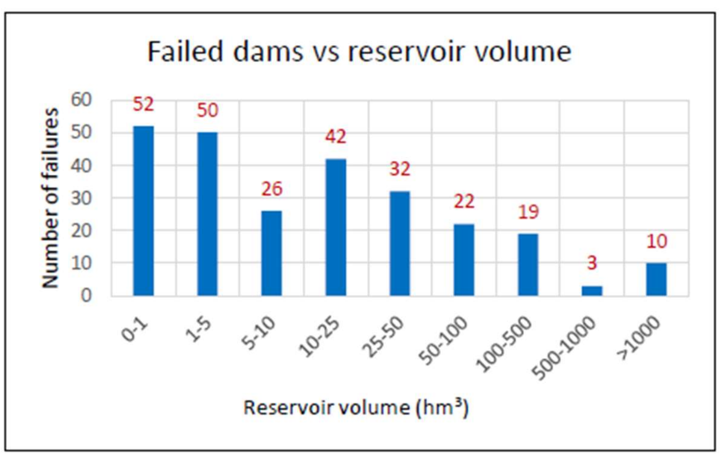


Fig 5: failed dams Vs reservoir volume

Figure 5 demonstrates the dams failures occurred in dams of different reservoir volumes. As per ICOLD data only 0.87% of dams failed under the category of storage volume more than 1000 hm. The total reservoir capacity of Koldam is 576 MCM, which is quite higher than 1000 hm (10MCM). With this historical data consideration also, the failure probability of Koldam is low.

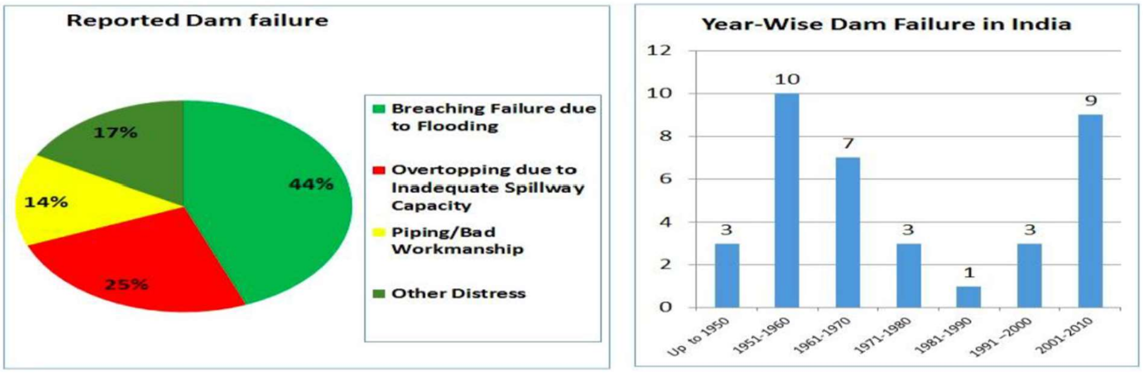


Fig 6: Dam failure data in India

Figure 6 shows the dam failure related data in India. Breaching failure due to flooding is the biggest reasons of dam failure in India, and secondly 25% failure are attributable to inadequate spillway capacity. For all dam types most of the failures have occurred due to floods and during floods, so the dam authorities should be extra careful during the floods.

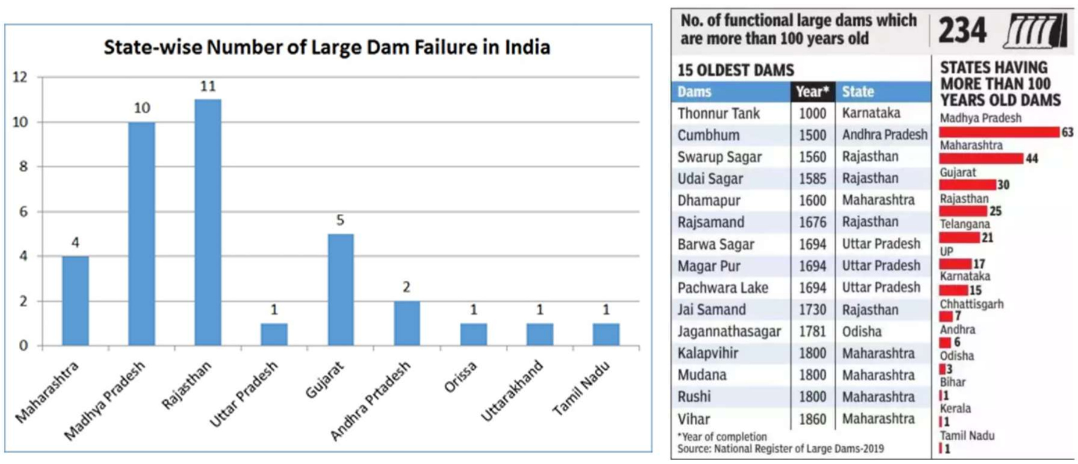


Fig 7: State wise old dams and Dam failure data in India

Figure 7 shows the state wise distribution of old dams and dam failure related data in India. highest dam failures are observed in Rajasthan followed by Madhya Pradesh.

At Koldam spillway capacity is adequate with PMF of 16500 cumec, sufficient cushion is kept for absorbing the flood by keeping the reservoir level at the lowest possible level, proper maintenance of hydro mechanical equipment is practiced. Hence, as inferred from Indian historical data also, the failure probability of Koldam is lowest.

NEED OF DAM SAFETY IN INDIAN CONTEXT

India has the third largest number of dams across the globe and a legislation to regulate dams is an essential and an important step towards ensuring dam safety in India. The Dam Safety Act 2021, which is the outcome of over 30 years of deliberations is a positive step in this direction.

Though 92% of the dams are located on inter-state rivers, most of them are built, owned and operated by the respective state governments. Hence, any law that seeks to provide a robust mechanism for dam safety must be based on the principle of cooperative federalism.

To cater the concerns related to dam safety in a comprehensive and uniform manner, Government of India has enacted Dam Safety Act 2021. It's a powerful tool to handle the issues related to dam safety. This act provides a legal framework for ensuring dam safety and it has a provision of governing bodies at central, state and local level. e.g. NDSA: National Dam Safety Authority at central level, State Dam Safety Organization (SDSO) at state level, Dam Safety Unit (DSU) at local/site level. The provision of

Dam Safety Unit (DSU) at project level has been made to implement Dam Safety Act at Ground Zero or grass root level.

The dam safety unit compiles all technical documentation of the project and conducts various inspections of the dams namely pre and post monsoon inspections which include visual inspections of various components of the project as well as reservoir rim inspections. Apart from this, special inspections are also done in case of any event like sudden flood, earthquake etc. Dam safety unit also looks after various dam instrumentation data, it's analysis and trends. It briefs the management about the healthiness of the dam and actions to be taken for ensuring dam safety.

DAM SAFETY PRACTICES AT KOLDAM

To maintain the dam in structurally and functionally safe condition without any loss in generation, Koldam HPS has undertaken many standard dam safety practices as per provisions of dam safety act along with some new initiatives in this direction. Key dam safety practices being observed at Koldam Hydro Power Station are apprehended below:

The Dam Safety Unit is fully functional with cross functional team members from various disciplines like Mechanical, Electrical, C&I, Geology, Engineering, Quality, Disaster Management, Operation etc. The Dam Safety Unit takes care of holistic compliance of Dam Safety Act including pre and post monsoon inspections. Various technical documentation like DPR, inspection reports, Drawings, SOPs, Guidelines, Permissions, Clearances, Quality observations, as built drawings, relevant codes etc. have been compiled and proper online documentation is indexed for ready retrieval of the same.

Reservoir operation manual and the Rule curve are well documented and being followed rigorously. SOPs (standard operating procedure) and LMIs (Local Management Instructions) are formulated for various Dam inspections and instrumentation monitoring and analysis.

Proper scheduled of planned maintenance of all hydro mechanical equipment throughout the year is standard practice and trial of spillway gates and pumps is conducted before every monsoon. This practice ensures no malfunction of any hydro mechanical equipment or spillway gate happens during the monsoon season or other requirement.

Reservoir Rim Inspection: Entire reservoir rim of about 40 km length is inspected by a cross-functional team, in pre monsoon and post monsoon seasons. Different deformation and landslides along the reservoir rim are recorded. The pre and post monsoon condition of individual slide is compared. A comparison of movement and condition of slides in previous years is also done to analyze the relative position of the slide. The reason for slide movement and stabilization measures are suggested for active slides. For problems of bigger magnitude and critical in nature expert opinion is also taken. Figure 7 shows a comparison of the same landslide in pre and post monsoon seasons.

Figure 7: Comparison of same slide in pre and post monsoon reservoir rim inspections

Pre and post monsoon inspections are performed by dam safety committee as per CWC guidelines.

The analysis of reports is done to ensure the timely implementation of steps required for safety of dams. The reports are shared with SDSO and NDSA. Pre and post monsoon analysis of dam and spillway instrumentation is also done to summarize the quantum of equipment working properly, any need of repairs, variation in results of equipment and any alarming situation. Pre and post monsoon inspections carried out till date illustrate that all components of Koldam are behaving properly, all dam safety compliance are being followed and there are no major issues in any component.

Analysis of Instrumentation Data: Various instruments like piezometers, inclinometers, load cells, measurement beams, seepage measurement weirs, thermometers, SMA etc. are installed in different parts of dam including clay core and spillway.

A typical dam section indicating Piezometers provided in dam is as in Fig 8.

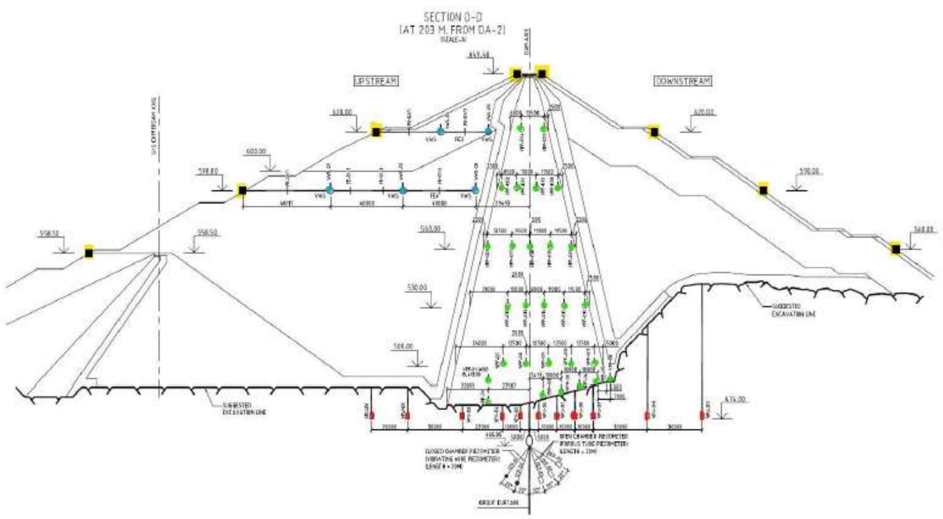


Figure 8: Dam Section indicating typical details of Piezometers installed

Two types of analysis of instrumentation data are carried out for different purposes. Monthly analysis of instrumentation data is carried out to analyze short term variations, data of current month is compared with previous month. In case of any abnormal variation, the historical data is compared. Pre and post monsoon instrumentation data analysis is carried out to see the variations in the instrument readings and its interpretation in pre and post monsoon seasons. Instrumentation data is recorded and stored in an automatic mode. Monthly dam safety inspection and analysis of instrumentation data reveals that the parameters of dam including settlement, seepage, temperature, water pressure etc. are within the permissible limits. The trend comparisons with previous years' data indicate that there is no unusual variation in the parameters in the long term. This is a further certification of the safer condition of the dam and its appurtenant structures.

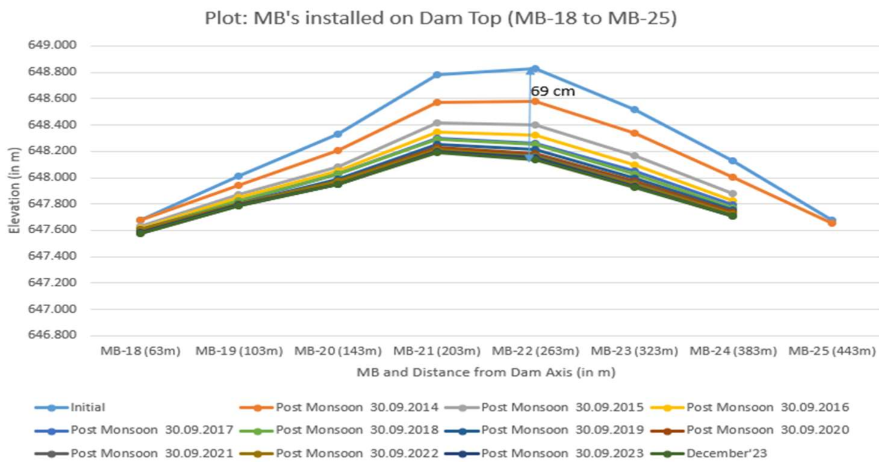


Figure 9: Yearly variation of Dam settlement in Post and post monsoon season

As shown in figure 9 the maximum settlement of dam is 69 cm which is well within the design value i.e. 1% of dam height.

Emergency Action Plan (EAP) and Disaster Management Plan (DMP) are well documented and implemented and these documents are being updated as per schedule and latest guidelines. Mock drills to handle various natural disasters are carried out in association with NDMA, SDMA, local administration and public. Villagers are being educated about dam safety and disaster management through newspapers, pamphlets, village level meetings, awareness sessions, Nukkad Nataks etc.

Hydro-meteorological stations and real time Inflow measurement: Hydro-meteorological stations including AWS (Automatic Weather Station) are installed at Koldam. Inflow is being measured at 40 km upstream of dam. The inflow information of the upstream project is also taken into account. This gives a lead time of about 3.5 hours in case of any excessive flow and to take corrective actions. ***Automatic Weather Station (AWS), Early Warning System (EWS), river inflow and flood forecasting system*** etc. are fully functional at Koldam. Well planned and modern flood forecasting system and Early Warning system (EWS) keeps the dam and downstream population (man, material and machines) safe in case of any possible disaster.

Figure 10 depicts the layout of downstream Early Warning system (EWS) consisting of real time PA system with electrical hooters. Early warning for routine plant operation discharge and spillway operation both are being disseminated to common public by various means like hooter systems, PA system, announcement vehicle, SMS etc.

Figure 10: Layout of Downstream EWS along with PA & Electric hooters

The inflow & flood forecasting system has been developed at Koldam which is capable of giving 10 Days Inflow forecast based on the weather conditions over catchment area. The predicted inflows are being compared with the measured inflows and found very reliable. The continuous display of measured inflow, predicted inflow and its comparison is shown at control room and other critical locations.

Siltation study by satellite data and drone survey is being carried out to estimate the silt deposition on the riverbanks near villages. Figure 11 shows the satellite image showing the siltation along two different zones in the reservoir. The siltation volume calculated from the satellite data was compared to the corresponding value obtained from the drone survey and found in good agreement.

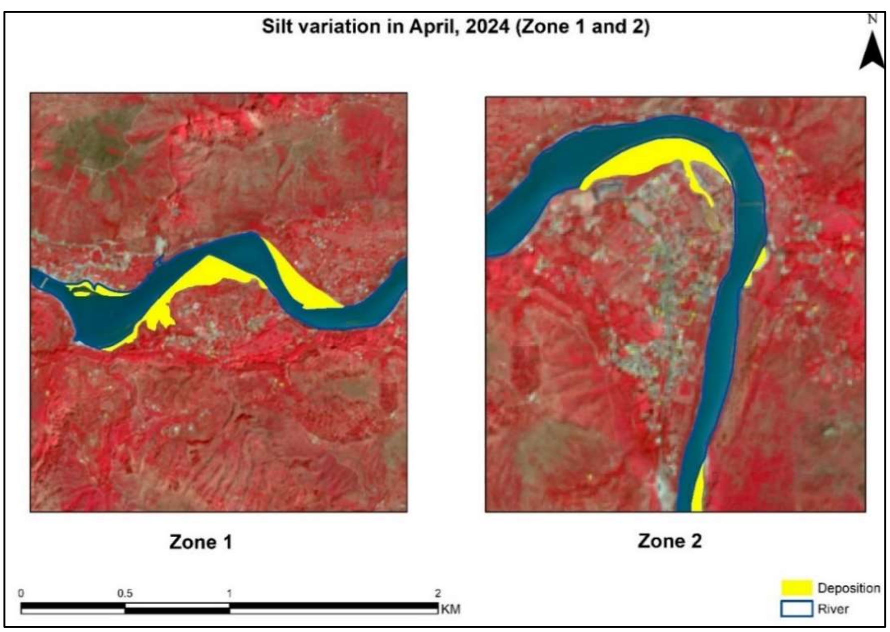


Figure 11: Siltation analysis along riverbanks using satellite image data

Bathymetry survey of reservoir is being conducted on an annual basis to estimate the siltation in the reservoir and to understand the effective reservoir capacity available for generation as well as to handle any flood. It also helps in predicting balance reservoir life before it becomes a run-off-the river scheme. As shown in figure 12, the area capacity curves are revised based on the inputs of bathymetry studies. It indicates the decrease in capacity of the reservoir with time. Regular study in this regard shall help in assessing the balance life of the reservoir.

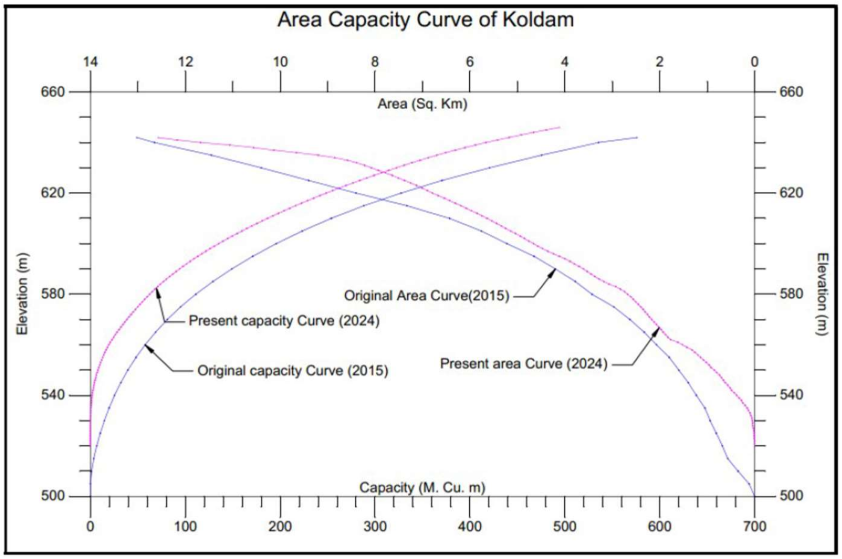


Figure 12: Revised area capacity curves based on bathymetry study

Quality control during the operation and maintenance of dams can significantly reduce the chances of dam failures. Koldam is a rockfill dam with a central clay core. A high degree of quality control was exercised during construction period for clay properties and field compaction of dam layers. Similarly, the critical tests of aggregates and boulders like Alkali reactivity test and slack durability test reduce the probability of deterioration of constituent materials and in turn of the dam and spillway structures. The tests carried out during various maintenance activities for quality control assured that the test parameters are within the limits.

Two basic concepts of knowledge sharing and data sharing are being implemented at Koldam for educating more and more people about dam safety. For Knowledge sharing Dam Safety Unit attends various training programs, conferences, mutual discussions and deliberations about the practical learnings of dam safety among the various partners and dam owners.

As an effective medium of data sharing, the generation parameters of Koldam like reservoir level, inflow, out flow, generation MW, spillway discharge etc. are being updated online on an hourly basis for data sharing among the dam owners and SDSO, NDSA.

FUTHER SCOPE:

A comprehensive dam safety evaluation is to be carried out as per Dam Safety Act 2021. This analysis is planned through a panel of expert or institute of repute in the near future at Koldam. Similarly, reservoir rim stability analysis is planned using remote sensing techniques in 2025.

CONCLUSIONS:

A comprehensive analysis of historical dam failure data was summarized in this paper along with the dam safety measures being implemented at Koldam. Different historical data related to dam failures, like type of dam, age, construction period, height, storage etc. are analyzed and compared with reference to parameters at Koldam is provided in table 1.

Sl. no.	Criteria of failure	Values	Koldam parameter	Status
1.	Age of dam	50% within 2 years	10 years	Safe
2.	Height of dam	≥100 m, 0.13%	167 m	Safe
3.	Tenure	Safe in normal operation	Normal operation	safe
4.	Dam type	Rock fill 1.39 %	Rockfill	Safe
5.	Storage	≥ 1000 $hm^{2,}$ 0.87%	576 MCM	Safe
6	Spillway capacity	Inadequate	PMF (16500 cumec)	safe

Table 1: Summary of historical dam failure parameters and Status of Koldam

The comprehensive comparison of statistical data of dam failures presented in paper and summarized in table 1 above indicates that dam is behaving with no dam safety issue till date conforming to the summary

In view of the details presented in the paper about the dam safety initiatives taken at Koldam, it would be safe to conclude that the Koldam HPS is maintaining all safety requirements as per Dam Safety Act 2021. The various inspections and analysis of instrumentation data including post Monsson inspection carried out in November 2024 reveal that the condition of Dam and spillway is good and it's safe in all aspects of monitoring. Further, the third-party inspections, investigations, reviews and visits by CWC, SDSO, NDSA etc. also verify that Koldam HPS is fully complying the provisions of Dam Safety Act 2021 and is setting up benchmarks for hydro dam based hydro power stations.

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- **नवरत्न का दर्जा:** कार्यनीतिक महत्व को मान्यता देते हुए 30 अगस्त, 2024 को प्रदान किया गया।
- **भविष्य के लिए विजन:** एनएचपीसी संधारणीय ऊर्जा और वर्ष 2047 तक विकसित भारत के विजन के प्रति समर्पित है।

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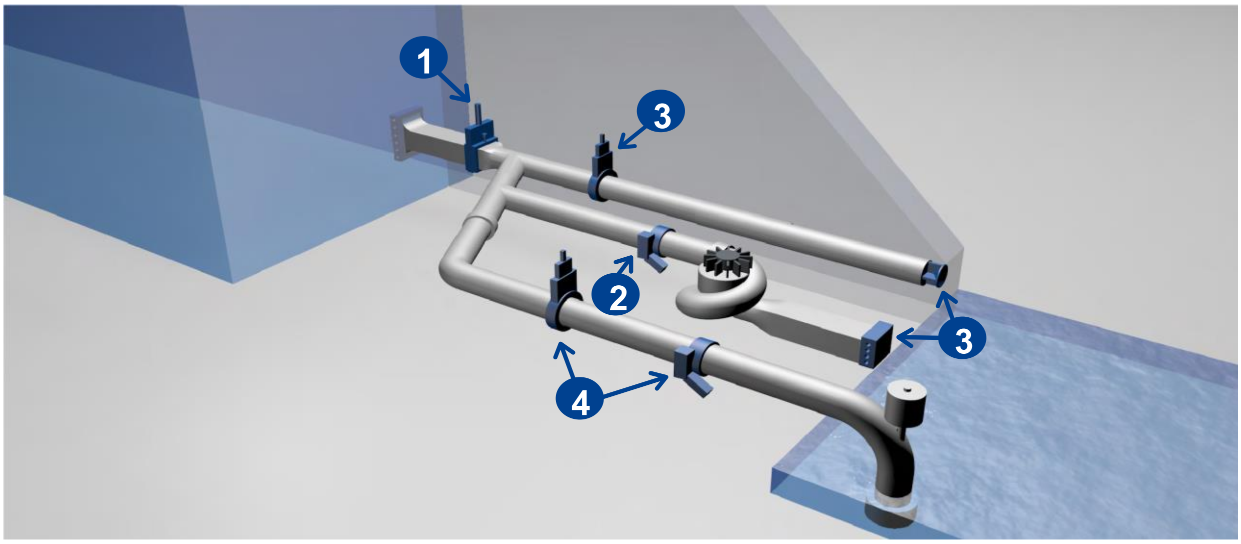

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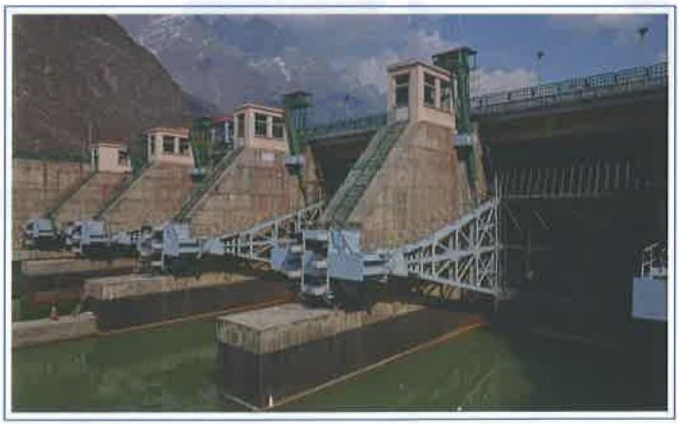
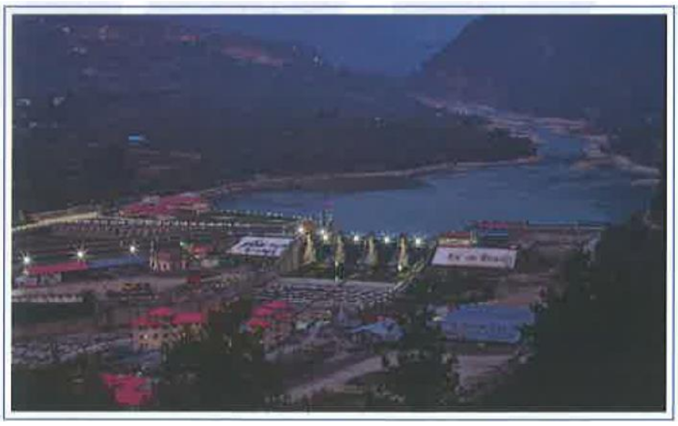
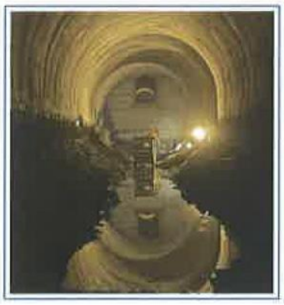
MACHINE HALL OF KARCHAM POWER HOUSE

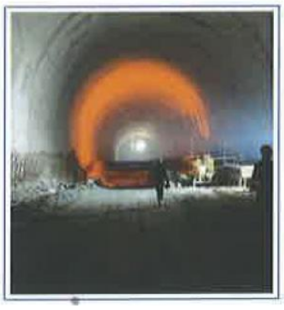
BASPA II BARRAGE (300 MW)

SURGE SHAFT OF KARCHAM WANGTOO HEP









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North Bengal Branch office : Room No: F0509, CITY CENTRE, Ambuja realty Development Ltd. Siliguri, Matigara.

GSTN: 19AEVPM7533H1ZT **UDYAM- WB-10-0039636**

MMA is MSME Registered MICRO Organization.

M MKHOPAFDHYAY &ASSOCIATES is an Expert Solution Provider for infrastructure Construction Problem solving through Judicious use of Latest Construction Chemical Technology & equipment.

Area of Expertise: -

- On site Training for CIVIL Engrs. In construction, WET shotcreting, Specialised Grouting , Anchoring.
- Marketing of CC& its application in Civil Engg. construction& problem solving, Concreting, Shotcreting, Injections, Water Proofing and Repair ,Geo technical problem handling.
- Inspection of any Problems during constructions or in existing structures for both UG and super structures , and also for structures in aggressive environment.
- Can arrange under water inspection by Divers and provides solutions for UW repair. Directly involved in Hirakud DAM Under water crack Repair (1995-98) .
- Conduct assessment of Structural Distress by NDT & DT, analyse the Data w.r.to IS and International standards &recommend remedial measures for Structural Repairs, Protection, strengthening.
- Analysing structural problem for Earthquake resistant and offers remedial solutions forRepair, structural strengthening by Retrofitting.
- Geo Technical Problem solving through Grouting. Inspection of Problem and right selection of Grouting materials ,its execution .
- Provides on site solutions for Wateringress Controls in UG Structures.
- Repair of Highway Bridges even in very short shutdown.
- Structural Grouting by using latest Equipment. Soil Grouting , anchoring for slope stabilisations.
- Slope protection, stabilization by nailing, shotcreting to retain earth.
- Heavy duty Industrial Flooring using FRC , Epoxy & PU based materials . Hygienic Flooring system for Operation Theatres, Electronic industry. Workshops, clean room system.

Soil stabilization & water ingress control Job under execution by M M A from July 2022 by Injecting 2Comp. PU Resin

CORE EXTRACTED FROM SOIL UP TO 6.5Meter BELOW AT ITD BOWBAZAR SHAFT TO CHECK THE EFFICACY OF CEMENT -POLYMER GROUTING , 2-COMP PUResin , ACRYLATE Gel GROUTING AS PER APPROVED METHOD STATEMENT

At present MMA possess the following equipments :-

we are possessing following Equipment, tools & plants for our job at present.

1.0 Three Nos Chemical Grouting pump Imported from GRACO USA along with its complete accessories.

2.0 Cement Grouting Screw Pump – Two nos. - one 400litre capacity and another 200 litre capacity attached with Colloidal mixer, VFD for controlling mixing and pumping rate. And 10HP motors ,

3.0 One shotcreting Equipment for DRY and wet spraying .

4.0 One concrete mixer 1M3 capacity . Electrical panel.

5.0 4nos Drilling machine along with maximum 1.5meter Drilling capacity.

6.0 TAM Packers, Nozzles for both PRE & POST Grouting into sub soil and behind Tunnel .

7.0 Complete Laboratory equipment for Cement and Chemical Grout field testing.

8.0 All safety Items like Reflectors, Safety shoe, Safety googles, face shield. Safety Helmet, Rain coat to work in watery condition, face mask, gloves, barrier cream for skin protection, foldable ladder ,platform etc as needed for safe execution.

9.0 One vehicle for local movement.

However, we may buy any equipment for our new job as per future need.

We have Two nos Twin PISTON Chemical Grouting Pump For 2K PU resin and 2K Acrylate Injection Grouting with maximum grouting pressure up to 150Bar.

MMA Cement grout Pump 400litre capacity Screw Type pump, electrically operated . Attached with Accelerator Pumping arrangement in same frame& Fitted with VFD, RPM control for mixing and agitation.

L&T Singoli Batwari Hydro Power Project at Uttarakhand – Drainage Gallery water ingress control by PU Injection

Water ingress @700LPM

Soil stabilization & Geotechnical.

SOIL STABILIZATION OF ITD BOWBAZAR SHAFT AGAINST BOILING OUT OF SUBSOIL WATER
By using Cement Polymer Low viscous slurry and Two Component PU resin & Acrylate

What we have done so far

Cconsultancy job :

1) L&T Power for water ingress Problem solving& Structural Repair, Geotechnical problem solving in Uttarakhand Hydro Power Projects in Rudraprayag before its commissioning .
2) AfconsInfrastructure , UG Projects Problem solving at Nasik Samruddhi Tunnel .
3) MEGHA Infra. Projects for Pollavaram Dam, for water ingress controld & Repair .
4) B L Harbert ,USA, for American Embassy Building Project New Delhi , for their PE certification and review of Design mix and drawings. It is ongoing.
5) TATA Project Ltd. Shree Jagganath Mandir Parikrama Projects PURI,Odisha for soil stabilisation sloping sand stabilization and Building foundation strengthening.
6) Megha Engg. & Infrastructure ,Lambadug HEP , Himachal, Water leakage from constructed Tunnel and surge shaft. Soil loss prevention.
7) High Rise Building Inspection at Kolkata for AOA SAMPOORNA TRITYA –mainly to provide solutions for Repair and water proofing.
8) J KUMAR Infrprojects Ltd. SURAT METRO Project., TBM Launching shaft water ingress control. Consultancy job.

Consultancy & execution job :

1) Consultancy &Execution of Kolkata Metro Railway UG 2-ITD -ITD CEM JV for TBM Tunnel UG 2 -soil stabilization and water ingress Control for Below retrieval shaft Base slab at Bow-bazaar. Consultancy and Contract job execution did successfully. Value @INR 2.5 Cr. contract for Chemical Grouting PU, Acrylate Injection & Cement polymer Grouting.

2) Consultancy &Execution of Kolkata Metro Railway UG 2-ITD -ITD CEM JV for TBM Tunnel UG 2 -soil stabilization and water ingress Control for Cross Passages (CP1 & CP2) to excavate safely and smoothly. Value @INR 4.1 Cr. contract for Chemical Grouting PU, Acrylate Injection & Cement polymer Grouting.

We select any grout material only after testing in the field & laboratory

WE ALSO PREPARE DESIGN MIX OF CEMENT GROUT AS PER SOIL CONDITION

DESIGN MIX OF CEMENT GROUT FOR SOIL STABILISATION AT KMRCL BOWBAZAR SHAFT Laboratory testing for designing Low viscous , chemical resistant Cement, SBR Polymer + HRWRA Grout Mix testing with the Tunnel soil & Dirty alkaline water To check its suitability for injection and efficacy at project site laboratory

MMA always do prior testing of Grouting materials at their laboratory and at site Laboratory in ambient Temperature to check the suitability of propose InjectionGrouting materials.

Laboratory testing of Grout materials by MMA before any selection: PU Resin, Acrylate, Colloidal Silica for soil stabilization other than cement grout. Using Tunnel soil & water To check its suitability for injection and efficacy at project site laboratory

After Acrylate Grouting , the Stabilized soil mass Vertically standing and no collapse

MMA ALWAYS USE BRANDED PRODUCT

MMA Organization chart for Consultancy & Execution of UG problem solving job through grouting :-

Present Organisations:Structure

Engr. **MilonMukhopadhyay**	Head of Organisation and Proprietor of MMA Expert Consultant cum Executor for UG or Underwater project problem solving in Infrastructure project. Repair of structure and distress analysis.
Ms Kakoli Bhattacharyya + Two Executive	Finance & Tax Control
Alf Mikael Johnsson , Lima, **(Visiting Expert)** swededn. Employed by MMA .	Swedish Expert for **Drilling, Grouting & Shotcreting** Over 40years Global project experience. Having Multiple Entry Valid VISA . At present posted in KMRC. ITD Job. Visiting Expert for our Execution job.
Mr Pradip Narayan Mukherjee	HR,. Industrial Relations, safety and Training . Ex HCC >40years Experience in Construction
Bryan Porter , Expert from IRELAND. CIVIL Engr. Trained in SIKA and BASF .Employed by MM	Having >23yeras Global UG project experience in TBM tunnel and UG project. Expert in Ug project problem solving. Joining KMRC job on 10^{th} Dec,22. Having Multiple entry VISA . Visiting **NATM Expert** for our execution job .
Engr. Santanu Chakraborty. Retired Executive Engr. Kolkata Municipal Corporation + **Nabarun Das** , Civil Engr., **Souvik Das** Electrical Engr. [with the experience of Bowbazar Shaft-grouting] + Operator, Fitter, skilled & semiskilled Workmen @ 25nos	Team from MMA for Execution of any job for ITD-ITD CEM JV for Ug2 TBM Tunnel cross passage water ingress control and soil stabilisation of soil below under unfinished Base slab at ITD Bow bazar shaft which was affected by Sub soil water blowing out in last may ,2022. This team is highly trained and skilled for both Cement grouting and Chemical Grouting.

MMA Foreign Expert along with MM And Engr. At KMRCL Bowbazar Shaft

CLIENTS

KMRCL

TATA PROJECT

BL HARBERT

AFCONS

AECOM

J. Kumar

ITD-ITD Cementation JV

Megha Eng. & Infr. Ltd.

L & T


Empowering the Nation
through Power Generation
Our Shared Vision
50000 MW by 2040
25000 MW by 2030
Pioneering sustainable energy, with clean and reliable energy for homes, industries and economies
Join us in creating a sustainable and inclusive future. Together, let's embrace nature's power, drive innovation, and build a brighter, greener tomorrow.
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SJVN LIMITED
(A Joint Venture of Govt. of India & Govt. of Himachal Pradesh)
'A NAVRATNA CPSE'
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