ISSN 2278 - 4683 (Print)
ISSN 2278 - 4691 (Online)

INCOLD

INCOLD JOURNAL

A Half Yearly Technical Journal of Indian Committee on Large Dams

Vol. 7, No. 2 ● July 2018

Indira Sagar Dam

INDIAN COMMITTEE ON LARGE DAMS
CBIP Building, Malcha Marg, Chanakyapuri, New Delhi - 110 021
Ph. : 91-11-2611 5984/2611 6567, Fax : 91-11-2611 6347
E-mail : sunil@cbip.org; cbip@cbip.org; Web: http//www.cbip.org

Incold Journal

INCOLD

Volume 7, No. 2 *July 2018*

CONTENTS

ABOUT JOURNAL

INCOLD Journal is a half yearly journal for fully-reviewed qualitative articles on aspects of the planning, design, construction and maintenance of reservoirs, dams and barrages, foundation and scientific aspects of the design, analysis and modelling of dams and associated structures.

In addition to the information on the research work on the relevant subjects, the journal shall provide information on the related technical events in India and abroad such as conferences/training programmes/exhibitions etc. Information related to ICOLD activities shall also be highlighted.

All communications to be addressed to :

Secretary General

Indian Committee on Large Dams, CBIP Building, Malcha Marg, Chanakyapuri, New Delhi – 110 021

Editorial

Every year between 1 to 2 % of worldwide reservoir volume is lost due to sedimentation. This loss is not even compensated by the actual new build of dams, in many cases making reservoirs not sustainable. By 2050 about 25% of all reservoirs will be inoperable due to sedimentation. At the same time, due to climate change and a growing world population, there is an increased need for additional water storage not only for renewable hydro power generation but also for drinking water supply, irrigation and flood protection. Once the effective storage volume is affected, sedimentation impairs the usage of the reservoir with economic and social consequences. Further, sediment accumulation can also lead to safety problems at the dam structure and blocking of outlet structures. Besides the sediment accumulation, organic matter also deposits within the storage basin. Microbial degradation of this organic matter leads to an increase of greenhouse gas emissions in form of carbon dioxide and methane to the atmosphere. Whilst "too much" sedimentation is causing problems within a reservoir, the sediment deficit downstream of a dam also causes problems due to increased erosion and substrate deficits. This alone surely makes sediment the most important long term problem of the entire dam business.

In addition, contaminated sediments count for extra complexity where industrial use at upstream rivers induces severe issues in sediment quality. In many cases all this actually makes the dam and hydro business not sustainable which is in contrast to the image of this sector. On the other hand there is a lot of knowledge, technology and processes available to overcome most of these problems, once these are applied.

There are practical guidelines on how to assess the sediment situation within a reservoir, identify a proper set of counter measures and establish a business case for sediment management. The latter is important to address the financial impact on "doing nothing" versus implementing a solution for reservoir recovery and sustainable sediment use. Every dam owner in a sediment affected reservoir is advised to early deal with a sustainable sediment solution for his own sake. Usually a study is recommended to analyse risks and to identify a suitable solution. Today a variety of technical equipment and procedures for a sound sediment solution are available.

We thank all the authors for their contributions. I also take this opportunity to thank all the members of the Editorial Board for helping us in our endeavour and providing us with their valuable suggestions in bringing out this journal.

We request all the water and dam professionals' readers to contribute technical papers/articles news etc. which would be of interest for publishing in the subsequent issues of the journal.

We also request for the comments /suggestions of the readers so as to improve the utility of the journal.

V.K. Kanjlia
Secretary General
Indian National Committee on Large Dams

Sediment Management of Projects on Himalayan Rivers - A Case Study of Bhakra Dam and Beas Sutlej Link Project

Devendra Kumar Sharma, S.K. Sharma and Sanjeev Suri
Bhakra Beas Management Board, Chandigarh

ABSTRACT

Sediment particles originating from erosion process in the catchments are propagated along the river flow. When river water is stored in a large reservoir, sediments settle down, thereby reducing the capacity of the reservoir. Reduction in storage capacity of large reservoirs beyond a limit hampers the purpose for which these are designed. Assessment of sediments deposition, their removal/exclusion and management becomes crucial for smooth operation of large storage reservoirs.

In case of river diversion schemes, silted water passes through turbines and damages the under water parts of the turbine, blades, nozzle and other equipment. It is, therefore, imperative to provide suitable silt management arrangement in the run-off the river schemes for the smooth functioning of the water conductor system as well as power house equipment.

This paper deals with the sedimentation management of a large storage reservoir as also a run-off the river scheme with small diurnal storage. Case studies of Gobind Sagar Reservoir of Bhakra Dam, a 225.55 m high concrete gravity dam having gross storage capacity of 9867.84 million cum located on river Sutlej and Beas Sutlej Link Project, a 990 MW capacity run-off the river project with small diurnal storage located on river Beas have been discussed. Both these projects are located in the lower foothills of western Himalayas. Various silt management and exclusion techniques used in both these projects, as described in the present paper, have proved successful.

1. INTRODUCTION

Bhakra dam a 225.55 m high, 518.16 m long concrete gravity dam is located on the river Sutlej in the foot hills of western Himalayas/lower Shiwaliks in Himachal Pradesh, a northern state of India. The Gobind Sagar reservoir, created by construction of Bhakra Dam having gross storage capacity of 9867.84 MCM (million cum), live storage of 7436.03 MCM and dead storage capacity of 2431.83 MCM[1], is one of the largest reservoirs in India. Since the sedimentation acts as a retrograde factor, its management, future exclusion from the reservoir and measurement of sediments deposited is an issue of regular monitoring.

The Beas Sutlej Link Project located in Mandi district of Himachal Pradesh, India is a run off the river scheme for inter-linking of river Beas with river Sutlej. Both these are the major rivers in the Indus basin. The Beas River originates from Beas 'Kund' in the Himalayas near Rohtang Pass. It flows in an almost North-South direction upto Larji, where it takes nearly right angle turn towards West and flows in that direction upto Pandoh Dam. At Pandoh Dam, the Beas Sutlej Link Project diverts 254.82 cumecs (annual 4,714 million m^3) of Beas water into the Sutlej river into the Bhakra reservoir, through a system of two tunnels, an open channel and a balancing reservoir. It utilizes a fall of about 320 m (1050 ft) available at its tail end for generation of 990 MW of power at Slapper where this water discharges into river Sutlej, upstream of Bhakra Dam[2]. The sediment management of the above two cases is discussed below.

2. SEDIMENT MANAGEMENT OF LARGE STORAGE RESERVOIRS – A CASE STUDY OF GOVIND SAGAR RESERVOIR

2.1 Gobind Sagar Reservoir

The Gobind Sagar Reservoir, also known as Bhakra Reservoir, has a catchment area of 56880 sq km (21960 sq miles), rainfall of about 109 cm (43 inches) and mean annual run off of 17178.33 million cubic meter (13.93 million acre feet). The average gradient of the river is about 1.89 m/km (10 feet /mile) in the reservoir area. One of the oldest built dams in India, the Bhakra Dam, is 225.55 m (740 ft) high concrete gravity dam on the river Sutlej, was commissioned in the year 1963. In addition to controlling the severe and devastating floods successfully, this Dam has not only provided Irrigation and Power benefits but also has brought prosperity in the Northern India. Having a gross storage capacity of 9867.84 million cubic meter (MCM), the Bhakra Dam has a designed dead storage of 2431.81 MCM and live storage of 7436.03 MCM. Water-spread area of Bhakra reservoir extends over 168.35 sq km at full reservoir level (El.515.11 m) and it touches the tail race of Kol Dam Project Power House (800 MW), a point about 12.87 km above Slapper, near village Kasol[1].

Precipitation in catchment area of the reservoir is in the form of rain as well as snowfall. Snow-melt contribution into Bhakra reservoir is about 59%. A study of precipitation distribution shows that maximum contribution to annual rainfall (42 to 60%) is received during the monsoon season (June to August), whereas a nominal 5-10% is received in the post-monsoon season. Consequently, the reservoir attains its maximum water level either during the monsoons or just after. The water level of the reservoir gradually reduces due to water use and reaches lower levels in the months of March/April.

River Sutlej transports a heavy amount of sediment, which is detrimental to life of the reservoir. The sediment contribution is mainly due to the dry desert portion of the catchment in Tibet area and Spiti area in Himachal Pradesh. Deforestation, over-grazing in the pasture lands, construction activities, farming at elevated terraces, cloud bursts etc. are other causes of concern. Landslides/ slips in the higher reaches of the catchment, steep topographic gradient, poor structural characteristics of soil, disintegrated rock mass with clay in Spiti Valley and minning of limestone deposits in the catchment area at other locations add to the sediment intake in the reservoir.

The catchment of river Sutlej at Bhakra Dam is about 56880 sq km (21960 sq miles), out of which 37050 sq km (14305 sq miles) lies in Tibet and only 19830 sq km (7655 sq miles) lies in India. For the purpose of silt studies, the catchment area falling in India has been sub-divided in to 57 sub-catchments. Topographically and climatically these catchments have been divided into five zones for monitoring purpose.

2.2 Capacity Surveys

After arrival of sediment-laden water flow into a reservoir, the coarser particles settle first in the upper reach of the reservoir due to sudden decrease in the flow velocity and widening of the cross-section area. Subsequently, the finer sediment material deposits further along the reservoir bed. Sediment deposition into reservoir reduces its storage capacity, damage to hydro-equipment and upstream aggradation. Assessment of reservoir sedimentation is part of the basic information needed for operation of any reservoir. A regular monitoring of the sedimentation process helps in ensuring suitable management measures that are taken well in advance so that reservoir operation schedules can be planned for optimum utilization. Two most common conventional techniques for quantifying sedimentation in a reservoir are - direct measurement of sediment deposition by hydro-graphic surveys and indirect measurement using the Inflow-Outflow sediment records of a reservoir.

To study and compare actual silt deposits vis-à-vis the project assumptions, capacity surveys of Bhakra Dam reservoir have been carried out annually from 1963 to1977 and thereafter biannually. The last survey was carried out from October 2013 to March 2016. The survey consists of sounding along pre-determined cross-sections approximately 610 m (2000 ft) apart. Executed with an echo-sounder, the results are superimposed on the previous observations for working out the quantity of silt deposited at each cross-section of the reservoir during that period. For the purpose of observation, the reservoir has been sub-divided into 273 cross-sections. In order to facilitate the survey operations in a systematic manner, each x-section has permanent survey-posts of suitable heights on both the sides of the reservoir[3].

2.3 Suspended Sediment Measurements

Whereas capacity surveys reveal the live and dead storage capacity lost in the reservoir, it does not indicate the area wise silt contribution from the catchment to pin point the areas, which contribute to the maximum sediment load. Consequently, measurement of suspended sediment load at various sites along the river and main tributaries is being carried out to know the sediment contribution from the catchment area between any two-observation sites. This helps in carrying out priority wise soil conservation measures in the sub-catchments. It involves the measurement of suspended sediment load in the main river including its major tributaries along with suspended sediment load being discharged downstream of the dam. By deducting the total suspended sediment flowing out of the reservoir from the total sediment brought in, the sediment retained in the reservoir is also being worked out. It is found that average trap efficiency of the reservoir for the last 50 years is 99.4%. Bed load carried by the river and its tributaries has not been measured and has been taken as 15% of the total suspended sediment load. The suspended sediment load in known volume of water is observed and from this, the total sediment load in the total inflow is derived[3].

The above measurements are carried out at four sites on the main river and also on five tributaries in the case of Bhakra Dam. The suspended silt observations are carried out throughout the year on the main river and during the flood season (June to Sept.) on its major tributaries.

2.4 Sediment Load, Rate of Sedimentation and Sediment Yield

Approximately 43% of the total sediment contribution is from Spiti and Sutlej catchments above Khab (Namgia), located downstream of the place where the river Sutlej enters India. It has further been observed that the average sediment load from the suspended sediment load measurements varies by about 7.05% when compared with the echo-sounding results.

Heavy erosion has been experienced in the higher reaches from Rampur to Kasol warranting soil conservation measures in the affected areas.

The average annual rate of sedimentation has been worked out as 38.87 MCM (31511 acre feet) for the year from 1965 to 2016 against a designed figure of 33.61 MCM (27250 acre feet).[3] The rate of sedimentation has shown a higher trend after the year 1990. Rate of sedimentation has increased noticeably after 2005, probably due to increased construction activities in the catchment area among other influencing factors.

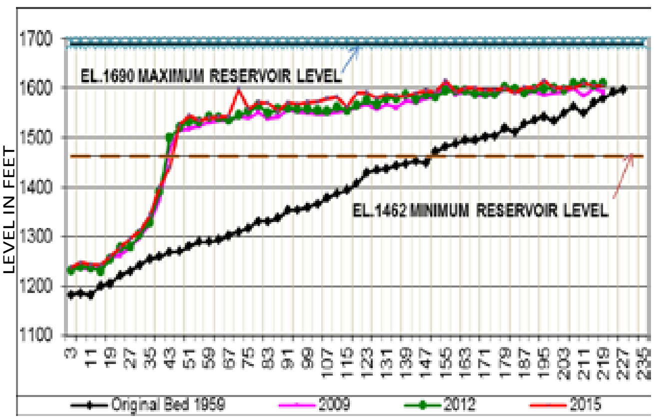
The average sediment yield per annum per thousand hectares of catchment area works out to be 6842.09 cubic meter (1.44 acre feet per sq mile of catchment area) for the period from 1959 to 2016.

2.5 Trap Efficiency and Delta/Hump Formation

A delta/ hump of sediments, positioned as per the reservoir bed profile, is still far away from the dam axis; and this deposited sediment is not yet finding its way through the dam outlets. Sediment observation taken on downstream of the dam shows that only negligible percentage of silt is flowing out. The average trap efficiency so far is 99.4%. Due to high trap efficiency, sediments have deposited over the years in the reservoir and are not discharging downstream of the dam.

Length of reservoir is 96.56 km (60 miles) and area of reservoir is 168.35 sq km (65 sq miles). Satluj river from Kasol to Bilaspur i.e. from RD (reduced distance) -273 to 159 follows a narrow and circuitous course. It fans out near Bilaspur to a width of about 914m (3000 feet). It again narrows down from RD-141 to 83 after which it opens into a wide expanse leading to a width of 6.44 km (4 miles) at full reservoir level just upstream of the dam[3]. River reaches, where the width of the reservoir increases, act as sediment trap. With the decrease in velocity, heavy sediment settles down in these reaches. Due to heavy inflows of water, this settled sediment in the upper reaches again gets eroded from the upper reaches and settles further at the still pond conditions in the reservoir, thus forming a delta at that point.

This manner of silt deposition in the reservoir over the years, has created a delta/hump from 41,000 ft to 91,000 ft, upstream of dam axis as shown in Fig. 1. The movement of crest of the hump towards the dam, though very slow, has taken place from 59,000 ft to 51,000 ft during 1998-2000. This was mainly due to low level operation of the reservoir in this period. The movement from 49,000 ft to 47,000 ft, as observed during 2002-05, is also attributable to a similar low level operation. A discrete balance between the desirable movement of the hump and the higher utilization of water in low level operations has been maintained during the reservoir regulation.



Distance in 1000 feet from the Dam Axis

Fig.1 : L-Section of Bhakra Reservoir Along River Sutlej

2.6 Loss of Storage Capacity

The percentage loss of live and dead storage capacity in 55 years (from 1961 to up to the end of the year 2016) has been worked out as 16.05% and 42.14% respectively. Against gross storage capacity of 9867.84 MCM (8000000 acre feet) of the reservoir, total sediments deposited up to the year 2016 are 2218.16 MCM (1798297 acre feet). Thus, overall loss of gross storage capacity is 22.47%[3].

3. SEDIMENT MANAGEMENT OF RUN-OFF THE RIVER PROJECT - A CASE STUDY OF BEAS SUTLEJ LINK PROJECT

3.1 Beas Sutlej Link Project

The Beas Sutlej Link Project comprises an earth-cum-rockfill type diversion dam at Pandoh having storage of 18.55 MCM, having a height of 76.20 m (250 ft), spillway with discharging capacity of 9,939 cumecs (3,51,000 cusecs) at maximum reservoir level of 2941 feet, Pandoh Baggi Tunnel intake structure which is on the left side of the spillway and a high crested forebay type intake with 10 bays each of 6.22 m (20.4 ft) clear span on the river side and high peripheral walls on the other three sides. Water conductor system consists of 13.11 km (8.14 miles) long Pandoh Baggi Tunnel having 7.60 m (25 ft) diameter and discharge carrying capacity of 254.82 cumecs (9000 cusecs); a gated control structure at Baggi Control Works; 11.80 km (7.33 miles) long Sundernagar Hydel Channel taking off from Baggi Control Works having a capacity of 254.82 cumecs (9000 cusecs) in the head reach upto silt ejector having flushing capacity of 28.32 cumecs (1000 cusecs) and 240.72 cumecs (8500 cusecs) in the tail reach, terminating into a Balancing Reservoir (BR) at Sundernagar with a live storage capacity of 3.7 MCM (3000 acre feet). The Balancing Reservoir takes care of the variations of supply from Sundernagar Hydel Channel and the actual requirement of water at Dehar Power Plant. Sundernagar Sutlej Tunnel (SST), 12.35 km (7.67 miles) long with finished diameter of 8.53 m (28 ft) having discharging capacity of 403.52 cumecs (14250 cusecs) takes off from the Balancing Reservoir and terminates in a Surge Shaft from where steel penstocks for the Dehar Power Plant (DPP) fan out. Generation capacity of the 6 units in Dehar Power Plant is 990 MW[2]. Fig. 2 shows the general layout of Beas Sutlej Link Project.

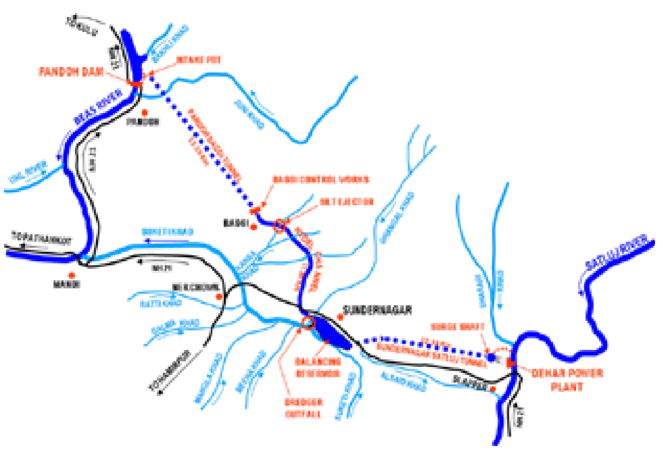


Fig. 2 : General Layout Plan of Beas Sutlej Link Project

3.2 Design and Actual Sediment Load

At the design stage, it was envisaged that an average sediment load of 407.63 ham (3312 acre feet) would reach Pandoh Dam annually. After flushing through the spillway, a sediment load of 215.38 ham (1750 acre feet) was expected to enter the Water Conductor System. The silt ejector of capacity 28.32 cumecs (1000 cusecs) provided in the head reach of Sundernagar Hydel Channel was designed to exclude 52.68 ham (428 acre feet) of sediment. The balance sediment load of 162.46 ham (1320 acre feet) was to enter into Balancing Reservoir. Further 80 ham (650 acre feet) of medium silt and a part of fine silt was assumed to settle down in balancing reservoir and 82.46 ham (670 acre feet) of balance fine silt to pass through the Sundernagar Sutlej tunnel. Sediments settled in the balancing reservoir were to be removed through dredging[2].

Presently, actual sediment load reaching Pandoh dam is more than what had been envisaged in the design criteria. The average annual sediment load reaching Pandoh Dam annually is of the order of 583.62 ham (4730.89 acre feet) out of which 424.34 ham (3439.71 acre feet) is flushed through the spillway and 4.45 ham (36.05 acre feet) is excluded through the silt ejector. The average annual sediment entering the balancing reservoir is of the order of 158.18 ham (1282.17 acre feet) and average annual sediment dredged out by a dredger from the Balancing Reservoir is 102.45 ham (830.48 acre feet). The average annual silt entering Sundernagar Sutlej Tunnel is of the order of 59.86 ham (485.22 acre feet)[4]. It passes through the machines of Dehar Power House and is less than the design assumptions. Average annual sediment load for the last 20 years entering into various components of water conductor system is given in table below:

Sl No.	Description	Design Sediment Inflow (acre feet)	Actual Sediment Load (acre feet)
1.	Sediments Inflow into Reservoir at Pandoh	3312	4731
2.	Flushing of Sediments through Spillway	1450	3440
3.	Sediments inflow into Pandoh Baggi Tunnel	1750	1318
4.	Exclusion of Sediments in Silt Ejector	428	36
5.	Sediments inflow into Balancing Reservoir	1320	1282
6.	Dredging of Sediments from Balancing Reservoir	650	830
7.	Sediments flowing through Sundernagar Sutlej Tunnel	670	485

3.3 Sediment Management Techniques

In view of the increased sediment inflow at Pandoh Dam compared to the sediment load envisaged at the design stage, it has become necessary that sediments are properly managed so that these do not adversely affect the water conductor system and the under water parts of the generating plant. Every possible effort is made to use water force to flush out sediments during reservoir flushing operation and also creating conditions that favour minimum amount of sediments entering into the water conductor system. Once sediments enter into the water conductor system, emphasis is laid on removing sediments efficiently and economically so that no more than the permissible amount of sediments pass through the turbines of the generating units. The various techniques adopted to achieve the above objectives are as follows.

3.3.1 *Still Pond in Front of Intake Structure at Pandoh Dam*

Almost still pondage has been created by providing a 99 m (325 ft) long divide wall in front of Pandoh Baggi Tunnel intake structure between bay No. 2 and 3 of the spillway of Pandoh dam. The top level of divide wall has been kept at El. 883.93 m (El. 2900 ft) in a length of 38 m (125 ft) and El. 886.98 m (El. 2910 ft) in a length of 61 m (200 ft.[2]. Spillway gates in bay No. 1 and 2, next to the intake structure, are kept in closed position while discharging lower discharges. This helps in creating stilling pool in order to allow water with minimum sediments into the Pandoh Baggi Tunnel (Fig 3)[5].

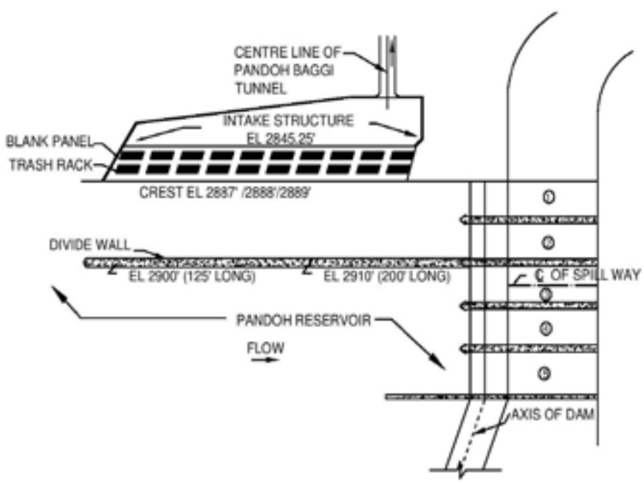


Fig. 3 : Layout Plan of Intake Structure and Spillway

3.3.2 *Raising of Crest of Intake Structure*

The intake structure in front of Pandoh Baggi Tunnel comprises of 10 bays with the varying crest level from El. 879.96m (El. 2887ft), 880.27 m (El. 2888ft) and 880.58 m (El. 2889ft).[2] During flood/rainy season when the sediment load is more, the crest of intake structure is raised further by putting in two blank panels in each bay (Fig. 4). This is to ensure that water is drawn into the Pandoh Baggi Tunnel from the top water layer having lesser sediment load.[5]

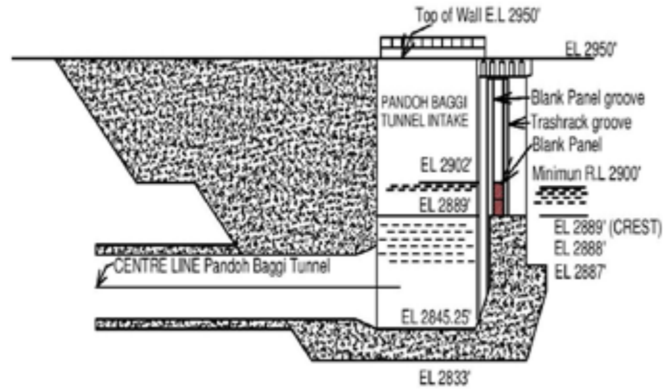


Fig. 4 : Pandoh Baggi Tunnel Intake Structure L-Section

3.3.3 *Flushing at Pandoh Reservoir*

Sediment load into the river is predominant during monsoon period. General trend of discharge v/s suspended sediment load (best fit curve) is shown in Figure 5. The quantum of sediment entering the carrier system is measured and recorded regularly at the balancing reservoir. Average annual trends of sediment inflows and flushing/exclusion/dredging values of last about 20 years are given above. Approximately 81.46% of annual silt load enters the balancing reservoir during three months of monsoon season from June to August. Remaining 14.93% enters in the months of April, May and September. Balance 3.61% of the sediment load inflows into the reservoir during the remaining six months in a year[4]. During the monsoon period considerable amount of sediments get deposited in the approach channel and the reservoir.

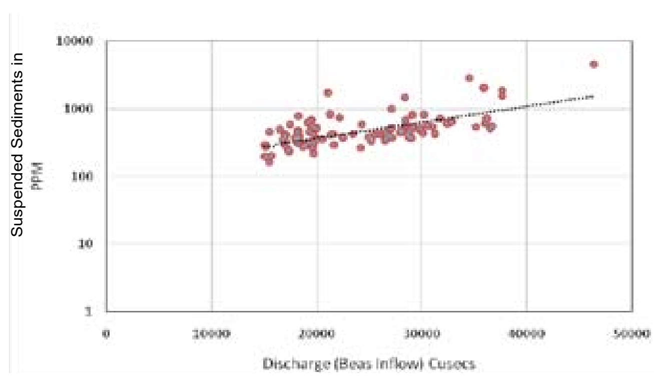


Fig. 5 : Discharge v/s Sediment Load in PPM Curve

The flushing of reservoir sediments is carried out at higher discharges generally more than 1133 cumecs (40000 cusecs), by closing the Pandoh Baggi Tunnel with blank panels in the intake structure. During the flushing operation all the five spillway gates are kept open to create free flow conditions. Flushing of sediment pocket in front of Pandoh Baggi Tunnel intake is carried out by opening of spillway gates no. 1 and 2, thereby clearing approach channel of the deposited sediments. On an average, 424.34 ham (3439.71 acre feet) of sediments are removed by flushing annually which is substantial and is more than the designed value of 407.63 ham (3312 acre feet).

3.3.4 *Silt Ejector in Sundernagar Hydel Channel*

A silt ejector of capacity 28.32 cumecs (1000 cusecs) has been provided in the head reach of Sundernagar Hydel Channel. Section of the hydel channel has been increased at the location of silt ejector. This is to achieve low velocities at the entrance of silt ejecting tunnels. It consists of two bays with carrying capacity of 14.16 cumecs (500 cusecs) each. The bed level of silt ejecting tunnels have been kept lower than the bed level of Sundernagar Hydel Channel at this location.[2] The silt ejector is designed to clear 52.68 ham (428 acre feet) of sediments. Keeping in view the difficulties faced by the local residents due to discharge of 1000 cusecs of water through the silt ejector, the silt ejector is being run with the discharge upto 14.16 cumecs (500 cusecs) only. The average silt clearance from the silt ejector is only 4.45 ham (36.05 acre feet).[4]

3.3.5 *Dredging in Balancing Reservoir*

A balancing reservoir has been provided at the tail end of Sundernagar Hydel Channel. As per design assumptions an average sediment load of 163 ham (1320 acre feet) would enter in the balancing reservoir. The gross storage capacity of the balancing reservoir is 369.23 ham (3000 acre feet). The capacity of reservoir below minimum pond level El.833.32 m (El.2734 ft) is about 110.77 ham (900 acre feet). About 80 ham (650 acre feet) is estimated to settle down annually in the balancing reservoir,[2] which if allowed for some years would encroach upon the limited live storage capacity of this balancing reservoir, thus, affecting the peak generation capacity of 990 MW Dehar Power Plant.

In order to dredge out the sediments, three Cutter Suction Dredgers have been provided in the balancing reservoir for the purpose of removing the sediment accumulation. In view of the very low velocities, of the order of 0.15 m/s (0.5 ft / sec), in the reservoir, most of the coarse as well as medium and part of the fine sediment load settles down in the balancing reservoir. The balancing reservoir has been divided into three zones. Coarse and medium sediments from first two zones are dredged out during monsoon season and discharged into Suketi rivulet, from where it flows with the natural river discharge. The dredged material is carried through 500 mm (20 inch) diameter M.S. pipe line with flanged ends for discharging at the prescribed location near the confluence of Gangal and Suketi streams. During non-monsoon season, very fine silt from third zone is dredged out and discharged through M.S. pipe at the mouth of Sundernagar Sutlej Tunnel intake to pass through Dehar power plant with a maximum limit of 500 ppm.

4. CONCLUSION

In large storage reservoir, trap efficiency has been found more than the designed value thereby reducing its useful life. Flushing of sediments in large storage reservoirs is extremely difficult. The movement of sediment hump/ delta is experienced during the low level operations of the reservoir. It is likely that as the hump moves nearer to the dam, some sediment will pass downstream with the density currents, through the low level openings of the dam, thus, increasing life of the reservoir. Pattern of sediments deposited in the reservoir is, however, greatly influenced by the reservoir operation and thereby water levels.

In run-off the river schemes having small storage, flushing of sediments annually during high discharge period through spillway combined with dredging operation has been found effective in the sediment management system. This helps in minimizing sediment entry into water conductor system for smooth operation and maintenance and minimizing damage to generating units in power station.

Operation of reservoir at lower levels in large storage reservoir, and flushing of reservoir combined with silt exclusion by mechanical means, in run-of- the river project, have been found useful in sediment management.

Acknowledgement

Authors thankfully acknowledge the useful technical support extended by Er. C.P. Singh, Additional Superintending Engineer/Designs, BBMB Nangal Township (India).

REFERENCES

1. Design Features of Bhakra Dam Vol.II, Bhakra Beas Design Publications, 1970.
2. Design Features of Beas Sutlej Link Project Vol. I, Bhakra Beas Design Publications, July,1984.
3. Sedimentation Survey report of Bhakra Reservoir, Reservoir Management Division, BBMB, Nangal, 2013-16,
4. Annual Observation Reports of BSL Project, Bhakra Beas Design Publications,1996-2016.
5. Operational and Maintenance Manual of BSL Project Vol II Part I, Bhakra Beas Design Publications, 1980.

Reservoir Sedimentation and Sustainable Development : Sediment Management Practice in Reservoirs of Two Power Stations Located on Himalayan Rivers of India

Balraj Joshi, Narendra Kumar, Rajeev Baboota and Manjusha Mishra
NHPC Limited, Faridabad

ABSTRACT

NHPC Limited, a Govt. of India Enterprise, is operating 19 hydropower stations with an installed capacity of 5121 MW along with two 1520 MW hydropower stations through joint venture. Most of the power stations of NHPC are located in high sediment prone Himalayas having small and medium size of reservoirs. Himalayan rivers carry much more sediment load than the capacities available in the reservoirs of power stations located in this region. Efficient sediment management techniques are needed in monsoon season to protect the economic and useful life of the reservoirs and turbines. NHPC ltd. has been successful in maintaining gross/live capacity of reservoirs of these power stations satisfactorily and is able to meet generation targets as per statutory requirement by practicing customized sediment management techniques for different projects having unique design features. This paper aims at sharing the experience of sediment management, being practiced in two reservoirs of NHPC's power stations, namely Chamera-I and Teesta-V power stations. Chamera Stage-I has relatively large reservoir capacity (391 MCM initially). The sediment management of this project is mainly done by following reservoir operation rules (at lower operating levels during monsoon season) and sluicing through low level sluice outlets. This method appears to be quite effective, resulting only 13.8% loss in the live capacity in 22 years meaning a, loss of around only 0.5% in live capacity per year, which is reducing with each passing year, as sedimentation profile reaches a stable regime. On the other hand, the reservoir of Teesta-V power station, which is a small reservoir having initial gross capacity of 13.5 MCM, has been maintained through sluicing combined with flushing. After 8 years of commissioning, the live capacity is maintained at 5-6 MCM as compared to initial capacity of 6.28 MCM. Loss in the live capacity is computed as only 3.8% in last 7 years meaning, 0.5% in live capacity per year (excluding 1st year).

Keywords : *Reservoir, Reservoir Capacity, Reservoir Operation, Sedimentation, Siltation, Silting, Power Station, Chamera-I, Teesta-V*

1. INTRODUCTION

NHPC Limited, a Govt. of India Enterprise, is operating 19 hydro electric power stations with an installed capacity of 5121 MW. In addition to this NHPC has completed two 1520 MW hydropower stations through joint venture. One of the major advantages of hydro projects out of a numerous others is the capability to provide the peaking power. To ensure that the run-of-the river projects maintain their useful life, some diurnal storage (live capacity) requires to be provided in the reservoir. Sediment spells doom for this diurnal storage as it occupies the precious little space meant for peaking purposes. Most of the power stations of NHPC are located in high sediment prone Himalayas having small and medium size of reservoirs. Himalayan rivers carry huge sediment during monsoon season. More than 80% of average annual sediment comes during the monsoon season. Efficient sediment management system is needed in monsoon season to protect the economic and useful life of the reservoirs. More often than not, these techniques are required to be customized for different projects depending on their reservoir size, valley shapes as well as their unique design features. The main objective of reservoir design and operation is to define suitable sediment control techniques to minimize the impacts of reservoir sedimentation on the life of reservoir as also on the life of water conductor system and turbines.

This paper aims at sharing the experience of sediment management being practiced in two reservoirs of NHPC's power stations located in different river basins in Himalayan region.

2. STUDY AREA

The study area comprises two power stations namely Chamera-I and Teesta-V power stations located in different river basins and states of India as shown in Fig 1. These two power stations are in cascade with other power stations as shown in Fig. 2 and Fig.4.

Fig. 1 : Location of NHPC's Power Stations

Chamera-I power station is located downstream of Chamera-II power station on Ravi river, which is further downstream of Chamera-III power station. The Chamera –I power station (Installed capacity : 540 MW), is a run of the river scheme with large Pondage on Ravi river in Himachal Pradesh State of India. It consists of a 140 m high (above deepest foundation level) concrete arch gravity dam across Ravi river, an underground powerhouse and a tailrace tunnel joining the river. The dam has been provided with 4 sluices (5.5 m high and 4.0 m wide each) and 8 bays of main spillway (10 m wide and 12.8 m high). The crest of spillway is at EL 730 m and of sluice is at EL 670m. The spillway has been designed to pass a flood of 24417 cumec at maximum reservoir level of EL 765 m General layout plan of the Chamera –I power station is placed as Fig. 3.

Teesta-V power station is located upstream of Teesta Low Dam-III power station and Teesta Low Dam-IV power station on Teesta river. Another power station, namely Rangit-III power station is also there on Rangit river in the Teesta basin. The Teesta-V power station (Installed capacity : 510 MW), is a run of the river scheme with a concrete gravity dam across river Teesta located about 2 km downstream of Dikchu confluence and an underground powerhouse near village Sirwani in Sikkim State of India. In the dam body a spillway has been provided for the release of floods as well as for the flushing of

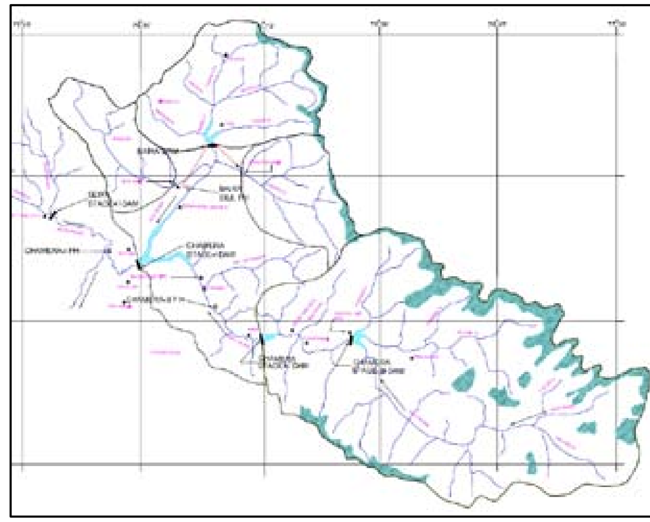


Fig. 2 : Catchment plan showing Power Stations in Cascade with Chamera-I Power Station

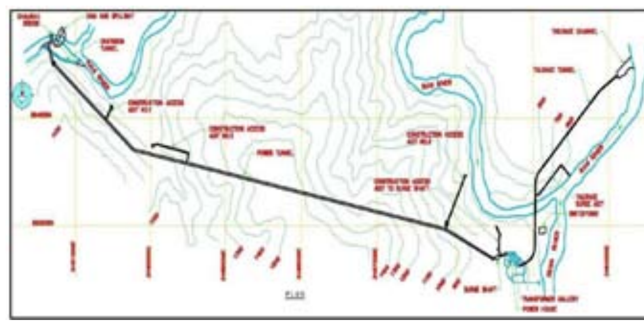


Fig. 3 : General Layout plan of Chamera-I Power Station

sediments deposited in the reservoir. It consists of five sluice bays, each 9 m wide x 12 m high, equipped with radial gates for regulating release of water. The spillway has been designed to pass a flood of 9500 cumec at maximum reservoir level of EL 580.72 m. The crest of spillway and gate seat elevation is at EL 540 m and EL 538.54 m respectively. General layout plan of the Teesta-V power station is placed as Fig. 5.

Principal parameters related to catchment of the two power stations are listed in Table 1.

Table 1 : Principal Parameters

Power Station	Catchment Area (sq km)	Average Annual Sediment (MCM)	Sediment Rate Ham/ Sq km/year	Average annual Rainfall (mm)	Average annual runoff (MCM)
Chamera I	4725	6	0.127	1100	6780
Teesta V	4307	10	0.232	2500	8842

3. RESERVOIR OPERATION AND SEDIMENT MANAGEMENT

Elaborate reservoir operation manuals have been developed for these power stations by NHPC. Two basic principals have been adopted for sediment management

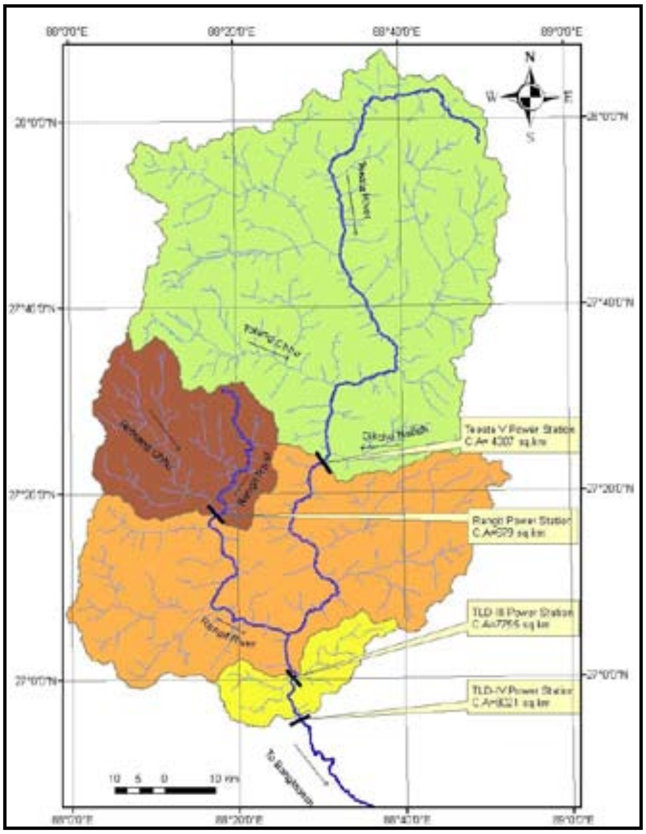


Fig. 4 : Catchment plan showing Power Stations in Teesta basin

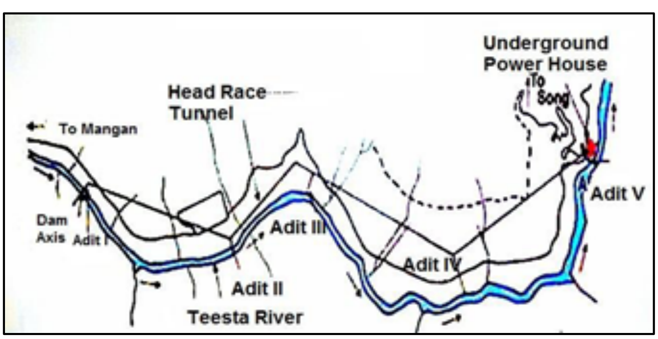


Fig. 5 : General Layout plan of Teesta-V Power Station

as explained below:

(a) Drawdown flushing during Monsoon Season (Flushing): In small size of the reservoirs preferably once in a month during monsoon period. This internationally accepted practice is being followed in all NHPC's run-of-river projects of small size. It re-mobilises the deposited sediment which passes through spillway. During drawdown flushing, the reservoir area takes the form of a pseudo- river.

(b) Maintaining reservoir level at lower level during the Monsoon (Sluicing): To pass a large quantum of the incoming sediment through the low level spillway and reduce trap efficiency (due to reduction in capacity inflow ratio) by keeping reservoir at lower level.

3.1 Chamera-I Power Station

The maximum water level (MWL), Full Reservoir Level (FRL) and MDDL (minimum draw down level) of the Chamera-I reservoir are at EL 763.2 m, EL 760 m and EL 749 m respectively. The reservoir of Chamera-I power station has two limbs, first limb is about 15 km in length in Ravi river and second limb is about 12 km in length along Siul river. Four low level sluices (invert level : EL 670 m) have been provided for de-silting of the reservoir in flood season, which are opened as and when inflow discharge becomes higher than machine design discharge. Under sluices have the capacity to pass up to 2600 cumec beyond which the main spillways (crest level EL 730 m) are operated.

During non-flood season, the reservoir level is to be maintained between EL 757 m to EL 760 m as per power generation requirement. Considering the size of the reservoir, inflow and quantity of sediment in the reservoir drawdown flushing is not practical at may take many days to complete the process of depletion, flushing and repletion of reservoir. Therefore during flood season, for passing the flood and sediment, the reservoir level is brought down from FRL and only low level sluicing has been adopted, as per the Table 2 mentioned below:

Table 2 : Rule Curve for Chamera-I Reservoir

Sl. No.	Period	Reservoir levels
1	1st June to 20th June	EL +/- 757 m
2	21st June to 31st Aug	EL +/- 753 m
3	1st Sep to 15th Sept	EL +/- 754 m
4	16th Sept to 15th Oct	EL 754 m to 757 m
5	1st Oct to 15th Oct	EL 757 m to 760 m

3.2 Teesta-V Power Station

The maximum water level (MWL), Full Reservoir Level (FRL) and MDDL (minimum draw down level) of the reservoir are EL 580.72 m, EL 579 m and EL 568 m respectively. The reservoir is about 5.1 km in length. 5 spillways (Each 12.0 m high, 9.0 m wide) at crest level EL 540 m have been provided to safely pass flood and sediment. 3 number desilting basins of size 19.7 m x 24.5 m x 250 m have been provided in the water conductor system to control the entry of sediment into turbines and under water parts.

During non-flood season, the reservoir of Teesta-V power station fluctuate between FRL (EL 579 m) and MDDL (EL 568 m) as per the requirement of power generation. Here considering the size of the reservoir, inflow and quantity of sediment drawdown flushing along with drawdown sluicing is practiced. During flood season for passing the flood safely, the reservoir level is brought down from

FRL (EL 579 m) to 0.5 m above MDDL i.e. EL 568.5 m. 4 flushings, 1 in each monsoon month, are carried out from June to September. During a month, flushing is normally carried once with specified discharge. The minimum period between two successive flushing has been worked out as ten days. However, in case the Teesta-V reservoir receives higher discharge immediately after flushing, then this flood is also utilized for flushing operation because, floods have the tendency to carry majority of the annual sediment inflow and by maintaining the continuity of flow (least obstruction to flow, as in flushing operation) it is possible to pass the bulk of the inflow sediment during sediment flushing.

During flushing operation, following observations are made to assess the effectiveness of flushing operations:

- Hourly reservoir levels at all the upstream locations.
- Hourly silt concentration at upstream and downstream of Dam.
- Hourly discharge passing through the spillway.
- Gate opening of all the five spillway gates on hourly basis.
- Reservoir Cross-sections at the end of monsoon at specified locations.

For some years, based on observed data, total sediment flushed out through different outlets (such as silt flushing outlet (SFT), machines, spillway etc) has been estimated as shown in Table 3.

As shown in Fig 4, two power stations namely TLD-III and TLD-IV came up on Teesta river in year 2013 and 2017 respectively, downstream of Teesta-V. Rangit Power station was commissioned in year 2000. After commissioning of TLD-III, the flushing schedule at Teesta-V was reviewed, as for power stations lying in cascade in a river basin, it was initially considered imperative to carry out flushing in tandem at these projects, within the limitations of river inflow, reservoir capacities and loss in power generation. As the reservoir capacity of Rangit-III is small as compared to the reservoir capacities of other three projects in the basin, the flushing operation at Rangit-III would have almost negligible effect in downstream projects in terms of discharge and sediment being added in the downstream due to reservoir depletion. The catchment areas and sediment load of Teesta-V and Rangit power station vis-à-vis catchment areas and sediment load at TLD-III and TLD-IV and large distance of about 66 km between Teesta-V and TLD-III power stations which results in about 6 hours travel time etc have been considered. The sediment observation were carried out during various flushing operation at just near the Teesta-V dam and at just upstream of TLD-III reservoir tail end (reach length of 66 km) and it was found that only 10% sediment concentration reaches upto TLD-III reservoir and most of the sediment settles down along the river. Therefore, it was decided that the reservoir flushing of Teesta-V and Rangit shall be independent of TLD-III and TLD-IV.

Table 3 : Total Sediment Flushed Out Through Different Outlets

Year	Inflow of Sediment into reservoir (M.Tons)	Sediment flushed out during Flushings (M.Tons)	Sediment passed through SFT (M.Tons)	Sediment passed through machines (M.Tons)	Sediment Passed through Spillway during Spillage (M.Tons)	Total Flushed out sediment (M.Tons)
2012	27.3	12.0	3.2	6.7	4.7	26.5
2013	29.8	11.0	3.6	8.9	6.80	30.3
2014	30.0	11.8	3.3	8.3	7.99	31.3
2015	22.8	8.0	1.7	8.9	6.0	24.6
% Removal						
2012		44%	12%	24%	17%	97%
2013		37%	12%	30%	23%	101%
2014		39%	11%	28%	27%	104%
2015		35%	7%	39%	26%	108%
Avg.		39%	10%	30%	23%	102%

4. COMPARISON OF ORIGINAL AND PRESENT RESERVOIR CAPACITIES

The cross sections of the reservoir for the power stations are taken at fixed locations every year after the monsoon to assess morphological and reservoir capacity changes, which are permanently marked along the reservoir length. Based on these hydrographic surveys after commissioning, reservoir capacities have been estimated and the summary of original and present reservoir capacity is given in Table 4. Comparison plots of longitudinal profile showing change in river bed with different years are shown in Fig 6 to Fig 7.

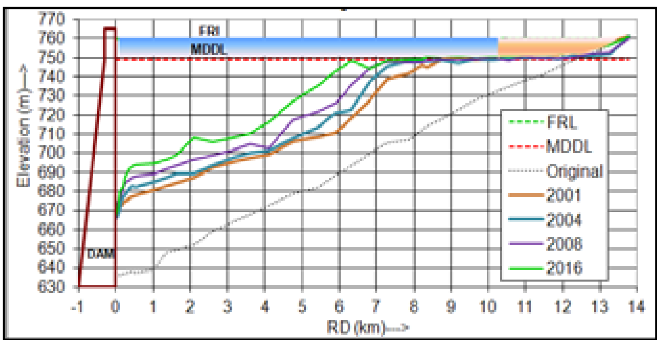


Fig. 6 : L-Section of Chamera-I Reservoir for different years

In Chamera-I Power Station, sediment got deposited in initial few years, however subsequently stable regime has been attained and the live storage is maintained at around 85-90 MCM. There is only 13.8% reduction in live capacity since last 22 years. Effect of sluicing can be measured from the post sedimented profile plots of year 1994 and 2016. As can be seen in Fig 6 that in the very first year itself, the river bed near the dam has risen by about 32 m but remained below EL 670 m, due to presence of sluices at that level. Other effect of under sluices is the steeper slope in the initial 1 km reach in comparison to slope between 1 km to 6.5 km. The slope is very flat from 6.5 km to tail end of reservoir (top set bed), leading to delta formation, which is gradually moving towards dam. Steeper angle of repose in the vicinity of under sluice can be visualized as the sedimentation process establishes itself over the years.

In Teesta-V Power Station, effect of sluicing and flushing can be measured from the post sedimented profile plots of DPR stage (commissioned in 2008) and year 2009. Within first year itself after commissioning, there was rise in river bed level up to crest level near dam site due to sedimentation. Sediment got deposited in first year itself and reduction in gross and live capacity was 21% and 10.3% respectively. Subsequently due to application of sediment management technique (flushing & sluicing), there is only 3.8% reduction in live capacity since last 7 years @ 0.5% per year. Stable regime has been attained in the shape of pseudo –river but at slightly higher elevation. Sediment bed remains below spillway crest level i.e EL 540 m, indicating the effectiveness of low level spillway provided in the power station, drawdown sluicing and reservoir flushing.

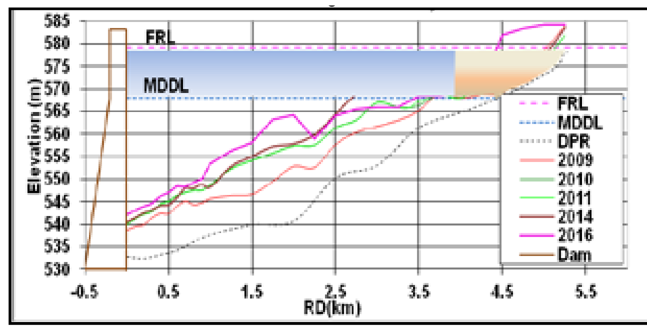


Fig. 7 : L-Section of Teesta-V Reservoir for different years

5. CONCLUSION

Chamera Stage-I has relatively large reservoir capacity (391 MCM initially). The sediment management of this project is mainly done by following reservoir operation rules and sluicing through low level sluice outlets. The low level sluice outlet helps in achieving high transport rate of sediment deposited in the vicinity of dam to the downstream channel and sluices helps in maintaining the relatively sediment free atmosphere in the vicinity of intake. The level of reservoir is kept at near to lower operating levels during monsoon season so that sediment may be routed to the downstream. It is observed that

Table 4 : Comparison of Original and Present Capacities

Power station	Capacity at FRL (MCM)		Live Storage (MCM)			Year of Commissioning/ No. of Years since Commissioning
	Original	Present	Original	Present	% Reduction	
Chamera-I (Large Pondage)	391.3	194.8	99.0	87.0	(-) 13.8	1994/22
Teesta-V (Small Pondage)	13.5	9.6	6.3	5.4	(-) 14.3	2008/ 8

the maximum sediment has been deposited towards tail end of the reservoir and a delta has been formed, which is slowly approaching towards dam axis. No sediment related problem has been reported in underwater parts from Chamera-I power station although the power station was commissioned about 22 years ago. The above method of sediment management in this project appears to be quite effective, resulting only 13.8% loss in the live capacity in 22 years meaning a, loss of around 0.5% in live capacity per year.

On the other hand, the reservoir of Teesta-V power station, which is small reservoir having initial gross capacity of 13.5 MCM, has been maintained in a satisfactory state through sluicing combined with flushing. Sediment got deposited within first year and reduction in gross and live capacity was 21% and 10.3% respectively. After 8 years of commissioning, the live capacity, is maintained at 5-6 MCM as compared to initial live capacity of 6.28 MCM, due to combination of reservoir flushing and sluicing. This method of sediment management appears to quite effective, resulting only 3.8% loss in the live capacity in 7 years meaning a, loss of 0.5% in live capacity per year (excluding 1st year), despite having high sediment inflow. Desilting basin provided in the project also played effective role in reducing impact of sediment in turbines and other underwater parts and it is estimated that about 10% of sediment passes through desilting basin.

Himalayan rivers carry much more sediment load than the capacities available in the reservoirs of power stations located in this region. NHPC ltd. is maintaining 19 power stations in Himalayan region and has been successful in maintaining gross/live capacity of reservoirs of these power stations satisfactorily by way of drawdown flushing and sluicing. Due to these effective sediment management practices, NHPC is able to meet generation targets as per statutory requirement. Hence, combinations of providing low level spillways/drawdown sluicing/reservoir flushing and/or maintaining the reservoir at lower levels during monsoon are the optimum and practical techniques to manage the reservoir sedimentation. These techniques are however required to be customized for different projects depending on their reservoir size, valley shapes as well as their unique design features.

REFERENCES

- ICOLD Bulletin 115, Dealing with Reservoir Sedimentation, 1999
- Atkinson, E., 1996. The feasibility of flushing sediment from reservoirs. Report OD 137, HR Wallingford, Wallingford, UK.
- Borland, W.M. and Miller, C.R., 1960. Distribution of sediment in large reservoirs. Paper No. 3019, ASCE, Transactions, Vol. 125, p. 166-180
- Morris, Gregory L. and Fan, Jiahua. (1998). Reservoir Sedimentation Handbook, McGraw-Hill Book Co., New York. Electronic Version, Ver 1.04, December 2010.
- Report No. OD 46, Wallingford, UK White, R., 2001. Evacuation of sediments from reservoirs. Thomas Telford, London.

Sedimentation Impact on Reservoirs and its Modeling Management Studies

S. Suneel
NSP-AMRSLBC Project, Government of Telangana, India

ABSTRACT

Sedimentation in reservoirs is becoming more problematic as water storage and supply become increasingly endangered with the aging of dams. This is causing severe consequences for water management, flood control, and production of energy. The worldwide loss in reservoir storage capacity due to sedimentation is about to be between 0.5% and 1.0% per annum. The gradual process of sedimentation proceeds with different speeds and that depend on a large number of factors, such as hydrology of the catchments and the characteristics of the river basin. On average sediment will eventually fill a reservoir within 50–200 years. This paper addresses about 3 general strategies of reservoir sedimentation managements and they are (1) Reduction of incoming Sediment yield, (2) Minimization of Sediment deposition and (3) Removal of Sediment from reservoirs. The main management methods associated with minimizing sediment deposition are construction of sediment bypass structures, sediment pass-through (or sluicing) and venting of a sediment-laden density current. The main management methods associated with removing sediment from reservoirs nearing critical storage loss are flushing and dredging. Drawdown flushing has been studied extensively and has been found to work optimally on narrow, gorge-shaped reservoirs where the water can be fully drawn down. Dredging is the most often used sedimentation management technique is also a highly expensive and time-consuming practice, although efficacious when complimented by other methods.

The methods available for sediment estimation are largely empirical, with sediment rating curves being the most widely used. In this study, Artificial Neural Network (ANN) technique based sediment transport system is taken as an example for discussion for Penganga River (a sub-basin of Godavari River) system and a comparison has been made between the results obtained using ANNs and sediment rating curves. The sediment load estimations in the river obtained by ANNs have been found to be significantly superior to the corresponding classical sediment rating curves.

1. INTRODUCTION

The sediment outflow from a catchment is induced by processes of detachment, transportation and deposition of soil materials by rainfall and runoff. The assessment of the volume of sediments being transported by a river is required in a wide spectrum of problems such as the design of reservoirs and dams; hydroelectric power generation and water supply; transport of sediment and pollutants in rivers, lakes and estuaries; determination of the effects of watershed management; and environmental impact assessment. Bureau of Indian Standard code IS: 12182-1987 “Guidelines for determination of effects of sedimentation in planning and performance of reservoir” is an important document which discusses some of the aspects of sedimentation that have to be considered while planning reservoirs. Some of the important points from the code are as follows (NPTEL):

- Performance Assessment (Simulation) Studies with varying rate of sedimentation.
- Effects of sedimentation at dam face.

The steps to be followed for performance assessment studies with varying rates of sedimentation are as follows:

(a) Estimation of annual sediment yields into the reservoir or the average annual sediment yield and of trap efficiency expected.

(b) Distribution of sediment within reservoir to obtain a sediment elevation and capacity curve at any appropriate time.

(c) Simulation studies with varying rates of sedimentation.

(d) Assessment of effect of sedimentation.

Main Impacts of Reservoirs Sedimentation

Storage Loss

It is usually the main impact for dams devoted to water storage as their benefit is quite proportional to the storage. This impact is lower for dams devoted to hydropower: their benefit may possibly be reduced by under 20% when the reservoir is 80% filled (including a large part in the designed dead storage).

Turbines Abrasion

Sediment coarser than 0,1 mm may greatly accelerate the erosion of turbines parts; even smaller grain sizes may cause damages if containing quartz. It may be the main siltation problem for high head hydropower. Also sediment concentration and total head are important factors.

Downstream Impacts

River reaches downstream of dams suffer large environmental impacts due to flow changes, reduction of sediment load, altered nutrient dynamics, temperature changes, and the presence of the migration barrier imposed by the structure and the upstream impoundment. Clear water released from the reservoir will cause downstream erosion and possibly bank failures. Sediment trapping by dams can even affect coastal morphology. It sometimes becomes a major factor contributing to the rapid shoreline recession and subsidence. One way of reducing this impact may be to build run-of-river hydroelectric projects which would allow passage of 100% of the fines and an important portion of the bed load.

Concepts of Reservoir Life

With reasonable levels of maintenance, the structural life of dams is virtually unlimited, and most reservoirs are designed and operated on the concept of a finite life which will ultimately be terminated by sediment accumulation rather than structural obsolescence. Design life is the planning period used for designing the reservoir project. Planning and economic studies are typically based on a period not exceeding 50 years, whereas engineering studies often incorporate a 100-year sediment storage pool in the design.The target of a very long reservoir life should be a key point of a right design and management of siltation problems.

Keeping this in view, in the present study sediment transport modeling has been presented for Penganga sub-basin system of the Godavari river system nearly 10% drainage area by means of rainfall, runoff and sediment transportation.

2. REVIEW OF LITERATURE

The sedimentation process of a catchment involves sediment detachment, sediment deposition, and sediment transport. Sediments travel from upland sources through the streams and may eventually reach to large water bodies. Erosion on land surfaces has been extensively studied by the several researchers in laboratories as well as in natural watersheds (Mohanrajuet al. 2011; Agarwalet al. 2006; Sarkaret al. 2004 and 2010 ; etc.). A physically based modeling approach to soil erosion processes has also been attempted by many researchers (ASCE Task Committee 2000 a,b; .Lohaniet al. 2007; Raiet al. 2008; .Agarwalet. al. 2006; Thomas et. Al. 2006; Kalinet al. 2003, 2004; etc.). Most of these studies considered either grid based discretization or conceptualization of watershed configuration for simulation of hydrological responses.

Based on the sedimentation rate of 239 reservoirs in India, the computed average annual percentage lossin gross storage due to siltation is 0.42% and based on the sedimentation rates of 86 reservoirsthe average annual percentage loss in dead and live storage is 0.494% and 0.04% respectively. The observed annual percentage loss in gross storage (minimum, maximum and average) is given in Table 1. The annual percentage loss in gross storage has been worked out as the average basedon the data of 239 reservoirs i.e. total annual loss in gross storage of 239 reservoirs X 100/ total gross storage of 239 reservoirs.

It is important to note that storage reservoirs built across rivers and streams loose their capacity on account of deposition of sediment. This deposition which takes place progressively in time reduces the active capacity of the reservoir to provide the outputs of water through passage of time. Accumulation of sediment at or near the dam may interfere with the future functioning of water intakes and hence affects decisions regarding location and height of various outlets. It may also result in greater inflow into canals / water conveyance systems drawing water from the reservoir. Problems of rise in flood levels in the

Table 1 : Observed Annual Percentage Loss in Gross Storage

Sl. No	Description	Minimum	Maximum	Average	Remarks
1	Annual percentage loss of gross storage	0.03	3.38	0.42	Based on average data of 239 reservoirs
2	Annual percentage loss of dead storage	0.007	5.32	0.494	Based on average data of 239 reservoirs
3	Annual percentage loss of live storage	0.003	3.32	0.04	Based on average data of 86 reservoirs

head reaches and unsightly deposition of sediment from recreation point may also crop up in course of time.

2.1 Reservoir Sedimentation Case Studies

2.1.1 *Nagarjunasagar Dam, Nalgonda Dist, Telangana, India*

Nagarjunasagar is one of the major multi-purpose river valley project on river Krishna connecting Nalgonda district of Telangana state. The purpose for which Nagarjunasagar is constructed is toprovide irrigation facilities to about 9 lakh hectares of parched lands and to develop seasonal power of 960 Megawatts per year. The project comprises of a dam with two main canals taking off, one on either side, viz., the Left Main Canal and Right Main Canal. Sedimentation studies were conducted by project authorities from 1964-71, 1974-76, 1978, 2001 and 2009. Some ground surveys were conducted in 1959-64 before construction of the dam by the silt survey division extending the surveys upto 16 kms upstream of Srisailam Dam taking the cross-sections approximately at 1.6 km intervals.

Fig. 1 : Nagarjunasagar Dam, Nalgonda Dist. Telangana State India

The following are the rate of sedimentation and revised capacity of reservoir based on the sedimentation surveys conducted:

2.1.2 *Musi Dam, Nalgonda Dist, Telangana, India*

Fig. 2 : Musi Dam, Nalgonda Dist. Telangana State India

The Musi Dam is located on the Musi river near Solipet village in Suryapet taluka, Nalgonda district, Telangana state. The dam was constructed in the period between 1954 to 1963. The project has a designed gross reservoir capacity of 136.940 Mm^3, with live capacity of 130.310 Mm^3. The purpose of the dam is to provide irrigation and flood control.Sedimentation assessment of Musi dam was done in June, 2016 through satellite remote sensing by MERI (entrusted by Central Water Commission, India) and the study revealed the live capacity of reservoir is reduced by 10.489 Mm^3 witnessing a loss of 8.05% in a period of 49 years from original survey in year 1963. This amounts to 0.16% loss per annum in live storage since 1963.

3. SEDIMENTATION MANAGEMENT

Reservoir sediment management strategies that both prolong reservoir life and benefit downstream reaches by mitigating the sediment starvation that results from sediment trapping. Some of the terms for sediment management have been used in different ways. Sediment management in reservoirs is largely classified into the four approaches: (1) To reduce sediment inflow to reservoirs, (2) To minimize sediment inflow so as not to accumulate in reservoirs and (3) To remove sediment accumulated in reservoirs & (4) Compensate for sediment accumulation in reservoir.

Table 2 : Revised Capacity of Reservoir Based on the Sedimentation Surveys Conducted

year	Original capacity 1967	1974-76	1978	2001	2009
Gross Storage Capacity (TMC)	408.237	393.837	359.216	328.783	312.045
Loss of Storage (TMC)	-	14.4	48.775	79.212	
Observed rate of sedimentation	-	2.706 ha.m/ 100 sq.km/ Year.	5.834 ha.m/ 100 sq.km/ Year	3.07 ha.m/ 100 sq.km/ Year	3.01 ham/100 sq.km/year.

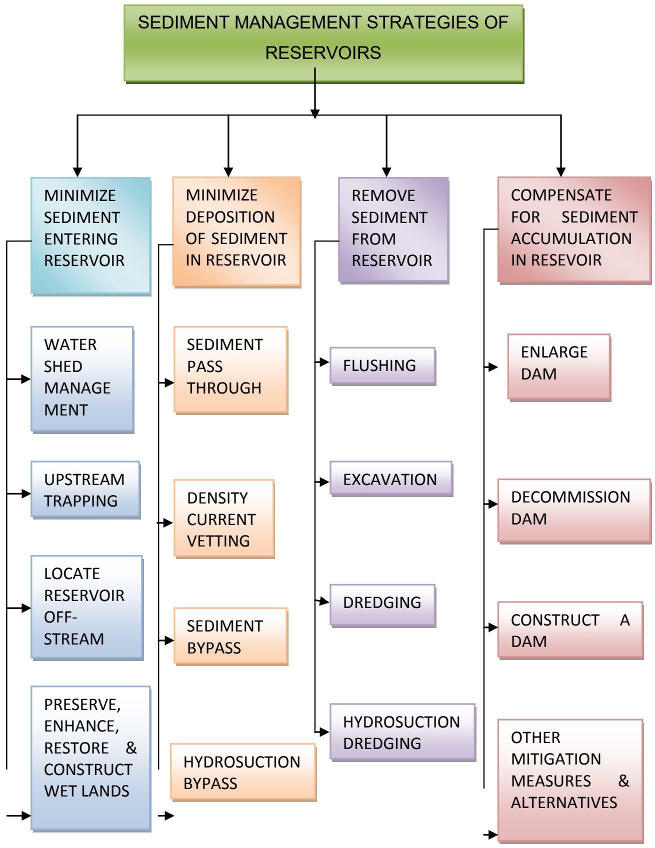


Fig. 3 : Main groups of Sedimentation management strategies of reservoir

3.1 Minimize Sediment Entering Reservoir

(a) Watershed Management

It is an adaptive, comprehensive, integrated multi-resource management planning process that seeks to balance healthy ecological, economic, and cultural/social conditions within a watershed. Watershed management serves to integrate planning for land and water. It efforts focus on upstream soil conservation, reservoir-level sediment removal, and downstream damage control from water pollution.

(b) Upstream Trapping

Trapping of sediment by dams is not inevitable, at least not by all dams. Some dams can be designed to pass sediment, either through the dam or around the reservoir, using a range of proven techniques, each applicable to a range of conditions. It may be that dam developers and operators are not aware of the range of potential management approaches, nor that they have been demonstrated to be effective. Thus, collectively we are missing opportunities to sustain reservoir functions into the future, and to minimize downstream impacts of sediment starvation. Sediment trapping in reservoirs improves the estimates of river sediment export, allows the useful life of reservoirs to be determined, and provides insights into sediment transport and dynamics of watersheds.

Sediments can be trapped on the upstream of dams by

- Construction of structures on upstream of Dam ex: check dam, debris dams etc.
- By providing vegetation filters.
- By providing sediment detention basins.
- By providingReservoir off-stream for sediments removal.
- Construction wet lands.

(c) Sediment By-Pass

Sediment bypass structures route high-sediment flows, generally resulting from floods, around the reservoir using canals, pressurized pipelines, or tunnels. Construction of canals is an expensive practice with its viability depending upon local topography, reservoir size and shape, and hydraulics of the river system. Pressurized pipelines for bypassing sediment are also rarely useddue to the specific conditions necessary for its successful implementation and the high cost of construction. Bypass tunnels are more common than canals or pressurized pipelines, but also suffer from high investment and management costs. A majority of these structures are operated in Switzerland and Japan where slopes are high (1% - 4%) due to mountainous topography and reservoirs are small. The effectiveness of this management method is seen in the Nunobiki Dam in Japan whose tunnel has allowed the reservoir to maintain a constant storage volume since 1908. Abrasion at the inlet due to high sediment concentrations is the main challenge with bypass tunnels. A design suggestion to combat the degradation of the inlet involves utilizing high strength concrete (30 N/mm^2 or more) and allowing for deep abrasion protection (10 – 35 mm). From an ecological standpoint, sediment bypassing boasts a lesser impact on the downstream environment when compared to sluicing, flushing. During flood events, sediment from the upper reaches of the river, which would naturally flow downstream without the existence of a reservoir, remains in suspension at approximately the same concentration. The suspended sediment concentration is neither higher from scouring of the reservoir bottom nor lower from releasing the clear water at the top of the reservoir.

Fig. 4 : Sediment bypass tunnel operational since 1922 (pfaffensprung, switzerland. amsteg power plant)

The main overall advantage of sediment bypass structures, however, is that they do not interfere with regular reservoir operation, as no drawdown of the water level is needed. However, the method is difficult to apply, requires careful planning, and cannot be utilized in arid climates where the need for water is high. In addition, regions with flood control concerns will find that sediment bypassing undermines the original intent of the reservoir.

(d) Density Current Venting

Density current venting, a seldom-used technique, involves the discharge of turbid sediment-laden water from a low-level outlet (like a sluice gate) while the surface waters remain clear or unchanged. Turbidity currents develop when water with a high sediment load enters a reservoir and immediately plunges to the bottom, travelling through the original channel until settling near the dam is called a "muddy pool". Management of these currents can drastically reduce sediment build-up at the base of a dam. However, density currents form only under certain conditions and can be difficult to detect. Under optimal conditions, approximately 50% of the total sediment can be vented, though the average is closer to 20%. One innovative way of managing density currents was developed in Japan at the Katagiri Dam where a curtain wall permitted only the sediment laden density current at the bottom of the reservoir to spill over the top of the dam. An illustration of the standard density current venting method is shown in below (Kathleen M. Healy, et. al)

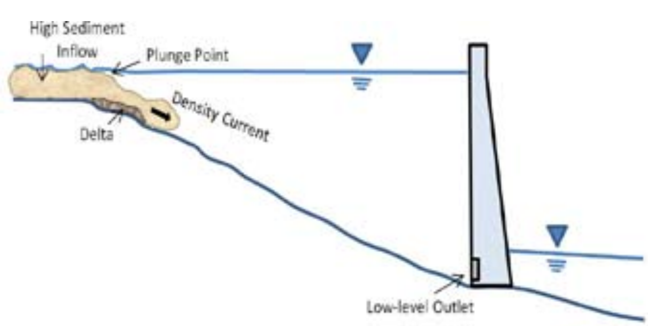


Fig. 5 : Density current along longitudinal reservoir profile

(e) Sediments Pass Through

Sediment pass-through (or) sluicing is another way of abating sediment deposition in reservoirs. In this method, the reservoir level is drawn down during the flood season and allowed to flow through the sluice gates to maintain the incoming sediment in suspension. When particles enter the low-velocity area of a reservoir, they settle and form a delta consisting first of the heavier coarse sediments, then further on a more shallow layer of fine sediment. This phenomenon can be seen in the illustration in Figure 6. A 1996 study of the North Fork Feather River in Northern California determined that using sluicing as a sediment-management technique viably maintained the equilibrium of sediment between the reservoir and downstream environment over an extended operating period. The study also showed that sluicing would not result in adverse impacts on the fish habitat downstream as long as it followed the suggested operating rules. One restriction of sluicing is its dependence upon the existing structure of the dam, as sluice gates must be positioned appropriately along the bottom for the sediment to flow through efficiently.

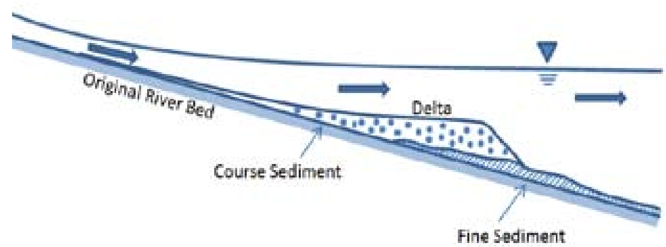


Fig. 6 : Longitudinal reservoir profile with delta formation

As per the Paul and Dhillon, 1998 sluicing techniques recommended that the sluice gates be at a height of 1.5 to 2.5 meters with an area determined from design curves presented in their paper. They also advised that only the width of the gate should be changed to increase effectiveness. In addition suggested that the sluicing technique is most effective when (1) Water depths are low and discharge is high (2) Sluice gates are wide and located near the bottom of the dam (3) The original stream bed is steep and the reservoir has a short, straight bottom and (4) The reservoir is in an advanced stage of siltation and the deposits consist of fine grained, recently settled material.

(f) Hydro-Sunction By pass

Hydrosuction sediment-removal systems (HSRS) remove deposited or incoming sediments from reservoirs using the energy represented by the difference between water levels upstream and downstream from a dam.

3.2 Sediment Removal from Reservoir

Removing sediment is quickly becoming an issue of great concern as reservoirs which were constructed decades ago without any consideration of sediment management have slowly built up enormous deposits of silt and sand and now are reaching levels which impede recreation and impact water supply. The main management methods associated with removing sediment from submerged reservoirs are flushing and dredging. Flushing can be further characterized as either drawdown or pressurized.

3.2.1 *Flushing*

Drawdown flushing is highly similar to sluicing however, it is not executed during flood season. Rather, it is done when the river is at low-flow conditions so that drawing down the water level takes less effort and does not affect the water supply. The operationally favourable conditions for drawdown flushing generally occur before the flood season or at the end of the dry season. A depiction of drawdown flushing is shown in Figure 7.

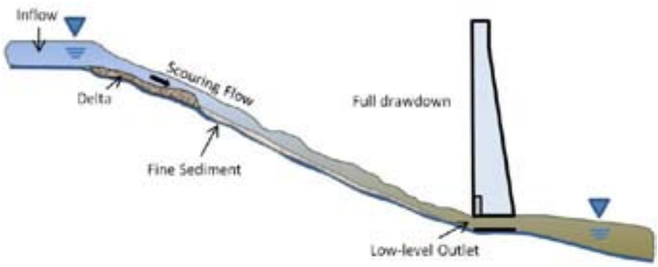


Fig. 7 : Longitudinal reservoir profile with sediment flushing

Drawdown flushing is generally most effective in narrow, gorge-shaped reservoirs and water must be allowed to be fully drawn down, making it near impossible to implement in hydropower dams. This technique is also impossible to implement in reservoirs without low-level outlets, such as sluice gates.

3.2.2 *Pressurized Flushing*

Pressurized flushing removes only a fraction of the amount of sediment when compared to drawdown flushing. This method is rarely used and its main purpose is to clear the area immediately surrounding the bottom outlets. As the sediment is flushed out the area around the outlet forms a funnel-shaped crater known as a flushing cone. The impact on the ecosystem downstream of the reservoir is an important consideration with either drawdown or pressurized flushing as a sediment-management method. A study of the Valgrosina reservoir in northern Italy where free-flow flushing was utilized to maintain reservoir capacity found that fish densities decreased up to 73 percent while biomass decreased up to 66 percent after a flushing event. To avoid spikes in suspended solid concentration and to manage scouring effects, they recommended that a yearly flushing occur in that particular reservoir. An earlier study of regulated rivers in California suggested that flushing could help clean fine sediment deposits from downstream gravel to encourage fish spawning. However, it also observed that flushing could be counter productive as potential high flow velocities might end up carrying some of this ecologically important bed load away with the fine sediment.

3.2.3 *Dredging*

Dredging is a technique generally used in large reservoirs. In the long term it is often a costly solution to the problem of sediment accumulation. Dredging techniques are classified in terms of such things as the type of equipment used and the season of the year in which dredging takes place. Dry excavation is typically used for sediment removal in a reservoir that is normally empty for a portion of the year or that could be easily emptied.

Fig. 8 : Illustration of Hydraulic dredging

Fig. 9 : Illustration of Mechanical dredging

After dewatering the reservoir, sediment is excavated with an appropriate excavating device. This method is restricted to large reservoirs and tropical reservoirs that are annually emptied during a dry season. Dredging systems are generally classified as either hydraulic or mechanical, depending on the type of machinery used. In hydraulic dredging, the sediments are thoroughly mixed with water to form slurry. They are then transported to a point of placement as a sediment-water slurry, dried, and either disposed or used. Whereas in Mechanical dredging sediment from the bottom surface of a water body, typically through use of a bucket. Hydraulic dredging is most effective when working with finely grained material. Coarser things, such as gravel, can be handled but will put a higher demand on the pump, causing it to wear more quickly.

4. ESTIMATION OF SEDIMENT TRANSPORT

4.1 Sediment Rating Curves

A sediment rating curve is a relation between the sediment concentration and river discharge. Sediment rating curves may be plotted showing average sediment concentration or load as a function of discharge averaged over daily, monthly, or other time periods. Rating curves are developed on the premise that a stable relationship between concentration and discharge (Ferugson 1986).

Mathematically, a rating curve may be constructed by log-transforming all data and using a linear least square regression to determine the line of best fit. The log-log relationship between load and discharge is of the form:

$$S = aQ^b \qquad ...(1)$$

And the log-transformed form will plot as a straight line on log-log paper:

$$\log S = \log a + b \log (Q) \qquad ...(2)$$

Where, S = sediment concentration (or load), Q = discharge, a & b are regression constants.

5. ARTIFICIAL NEURAL NETWORKS

An ANN is a computing system made up of a highly interconnected set of simple information processing elements, analogous to a neuron, called units. The main principle of neural computing is the decomposition of the input-output relationship into series of linearly separable steps using hidden layers.

The first step is the data transformation or scaling. The second step is the network architecture definition, where the number of hidden layers, the number of neurons in each layer, and the connectivity between the neurons are set. In the third step, a learning algorithm is used to train the network to respond correctly to a given set of inputs. Lastly, comes the validation step in which the performance of the trained ANN model is tested through some selected statistical criteria. The theory of ANN has not been described here and can be found in many books such as Haykin (1994).

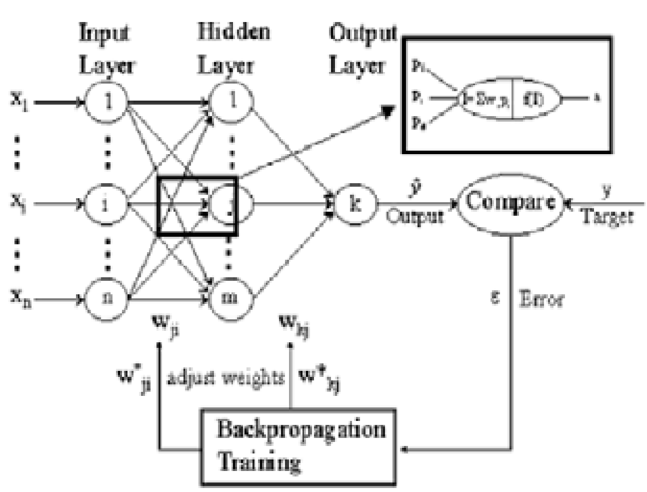


Fig.10 : Artificial Neural Network Model

6. STUDY AREA, DATA AVAILABILITY AND SELECTION OF INPUT/OUTPUT VARIABLES

In the present study the sediment transport modeling has been carried out for Penganga sub-basin system, and influences the Godavari river system to the maximum possible extent (with 10% drainage area) by means of rainfall, runoff and sediment transportation. The hydrological data for the study has been collected at Penganga bridge site on Pengangariver (Fig. 11). The daily data of sediment concentration and discharge were available at the Tekra site for four years (2000 - 2004) constituting a total of 1461 patterns. Out of this, 730 patterns were used for training, 365 patterns for testing and 366 patterns for validation.

The first step in developing any model is to identify the input and output variables. The output from the models is the sediment concentration at time step t; C_t. It has been shown by many authors that the current sediment concentration can be mapped better by considering, in addition to the current value of discharge, the sediment and discharge at the previous times. Therefore, in addition to Q_t, i.e., discharge at time step t, other variables such as Q_{t-1}, Q_{t-2}, and C_{t-1}, C_{t-2}, were also considered in the input.

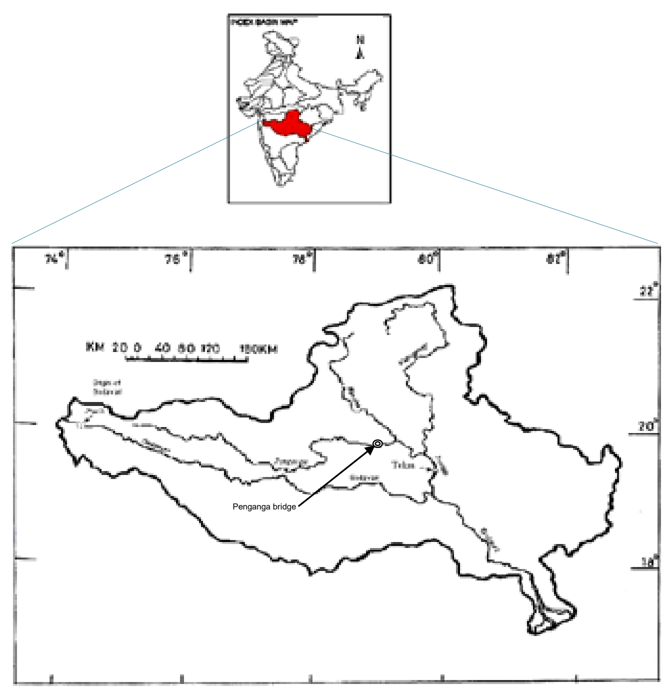


Fig. 11 : Penganga river system and hydrological study location (Penganga bridge site)

7. RATING CURVE ANALYSIS

Based on the sediment rating curve technique given by equation (1), the sediment rating equation between sediment load and discharge for Penganga River at Penganga bridge site for the training period is

$$C = 1.671E\text{-}03\ Q^{(0.921)} \quad \ldots(3)$$

Where, C = Sediment concentration in the River Penganga river at Penganga

bridge site in gm/l at time t

Q = Discharge in the River Pranhia at Tekra in Cumec at time t

8. DESIGN AND TRAINING OF ANN MODELS

Various combinations of input data considered for training of ANN in the present study are given in Table 3. However, the input-output variables of ANN-1 have been used for the conventional sediment rating curve analysis.

Table 3 : Various ANN Runoff-Sediment Models

ANN Model	Architecture	Output Variable	Input Variables
ANN-1	[1 – 2 – 1]	C_t	Q_t
ANN-2	[3 – 3 – 1]	C_t	Q_t, Q_{t-1}, C_{t-1}
ANN-3	[5 – 4 – 1]	C_t	Q_t, Q_{t-1}, Q_{t-2}, C_{t-1}, C_{t-2}
ANN-4	[7 – 6 – 1]	C_t	Q_t, Q_{t-1}, Q_{t-2}, Q_{t-3}, C_{t-1}, C_{t-2}, C_{t-3}
ANN-5	[9 – 7 – 1]	C_t	Q_t, Q_{t-1}, Q_{t-2}, Q_{t-3}, Q_{t-4}, C_{t-1}, C_{t-2}, C_{t-3}, C_{t-4}
ANN-6	[1 – 2 – 5 – 1]	C_t	Q_t
ANN-7	[3 – 3 – 5 – 1]	C_t	Q_t, Q_{t-1}, C_{t-1}
ANN-8	[5 – 4 – 5 – 1]	C_t	Q_t, Q_{t-1}, Q_{t-2}, C_{t-1}, C_{t-2}
ANN-9	[7 – 6 – 5 – 1]	C_t	Q_t, Q_{t-1}, Q_{t-2}, Q_{t-3}, C_{t-1}, C_{t-2}, C_{t-3}
ANN-10	[9 – 7 – 5 – 1]	C_t	Q_t, Q_{t-1}, Q_{t-2}, Q_{t-3}, Q_{t-4}, C_{t-1}, C_{t-2}, C_{t-3}, C_{t-4}

Where, C=Sediment Concentration at Tekra, Q=Discharge at Tekra, t represents the time step

A back-propagation ANN with the generalized delta rule as the training algorithm has been employed in this study. The ANN package Neural Power (2003) downloaded from the Internet has been used for the ANN model development. The structure for all simulation models is three layer BPANN which utilizes a non-linear sigmoid activation function uniformly between the layers. Nodes in the input layer are equal to number of input variables, nodes in hidden layer are varied from the default value by the NP package for various number of input nodes above to approximately double of input nodes (Zhu et al., 1994) and the nodes in the output layer is one as the models provide single output. According to Hsu et al. (1995), three-layer feed forward ANNs can be used to model real-world functional relationships that may be of unknown or poorly defined form and complexity. Therefore, only three-layer networks were tried in this study.

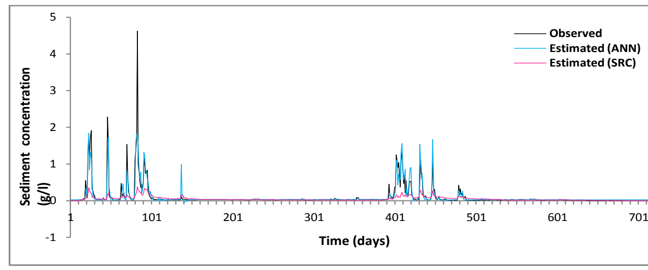
The modeling of ANN initiated with the normalization (re-scaling) of all inputs and output with the maximum value of respective variable reducing the data in the range 0 to 1 to avoid any saturation effect that may be caused by the use of sigmoid function. All interconnecting links between nodes of successive layers were assigned random values called weights. A constant value of 0.15 and 0.8 respectively has been considered for learning rate α and momentum term β selected after hit and trials. The quick propagation (QP) learning algorithm has been adopted for the training of all the ANN models. QP is a heuristic modification of the standard back propagation and is very fast. The network weights were updated after presenting each pattern from the learning data set, rather than once per iteration. The criteria selected to avoid over training was generalization of ANN through cross-validation (Haykin, 1994). For this purpose, the data were divided into training, testing and validation sets. Training data (730 patterns) were used for estimation of weights of the ANN model and testing data (365 patterns) for evaluation of the performance of ANN model during training. Training was stopped when the error for the testing dataset started increasing. In this way, the training and testing datasets have been used to assess the performance of various candidate model structures, and thereby choose the best one. The particular ANN model with the best performing parameter values was chosen and the generalized performance of the resulting network has been measured on the validation data set (366 patterns) to which it has never before been exposed. The performance of all the ANN models have been tested through three statistical criterion, viz, root mean square error (RMSE), correlation coefficient (R) and Coefficient of Determination (DC).

9. RESULTS AND DISCUSSION

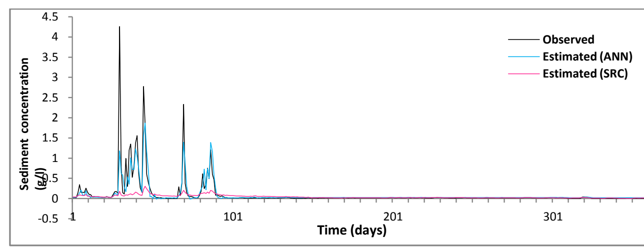
It is observed from Table 4 that the RMSE values are generally low (less than 0.2) for all the ANN models except ANN1. There is a candidate model in RMSE criteria, ANN5. In ANN5 model, the RMSE value is 0.079 during training. Now, it is observed from the Table 4 that the RMSE value is highest for the rating curve model except ANN1. The RMSE value is 0.271 during training which is more than the most ANN models and more than 2 times the best ANN model. It can be seen from Table 4 that the correlation coefficient (R) values are fairly high (more than 0.80, i.e. 80%) for almost all the ANN models, during all the three phases, i.e., training, testing as well as validation. It is also observed that there is not much decrease in the R values during validation as compared to the training phase. The performance of ANN4 model isthe best in R statistic. The R values for ANN4 are 0.890, 0.788 and 0.924 during training, testing and validation respectively. The increase in R values during validation indicates good generalization capability of both the ANN models. The performance of the rating curve model is very poor. The R values for rating curve model are 0.734, 0.637 and 0.785 during training, testing and validation respectively. These values are very low compared to the best performing ANN model, i.e., ANN4.

In the determination coefficient (DC) statistic, all the ANN models except ANN1 perform well. The DC values are fairly high (more than 0.75) for most ANN models during all the three phases. In DC statistic also, ANN4 model performs the best. The DC values for ANN4 are 0.790, 0.617, 0.853 during training, testing and validation respectively. The increase in DC values during validation indicate good generalization capability of both the ANN model. The performance of rating curve model in DC statistic has gone down drastically with DC values as low as 0.204, 0.178 and 0.269 during training, testing and validation respectively. These values are much lower than the best ANN models.Therefore, ANN4 is the best performing model in all the three statistical and hydrological criteria. The performance of the rating curve model is average in the R criteria but significantly poor in other criteria. It is because the estimated sediment series (from sediment rating curve model) follows a general trend as that of the observed sediment series which gives high R values, but there is a significant difference in the numeric values of observed and estimated sediment load due to which the RMSE and DC values are very poor.

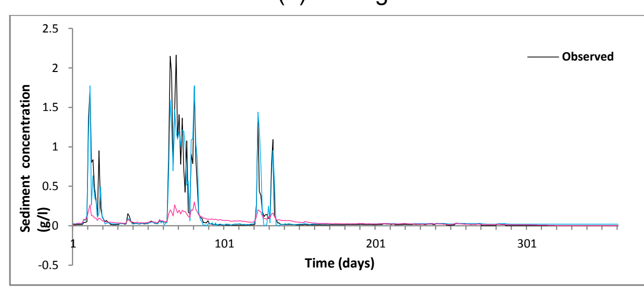
Figure12 present the comparative plots of observed and estimated sediment concentration i.e, ANN4 for training (a), testing (b) and validation (c). It is observed from Fig 13 that there is very little mismatch between the observed and estimated sediment concentration series for ANN4 model during all the three phases. It is seen from the graph that the ANN estimates very closely follow the observed curve, whereas the conventional approach has significant mismatch with the observed curve. The hysteresis in the sediment concentration obtained by ANNs and sediment rating relation are compared with the corresponding observed one combined for all the three phases. It can be seen that the hysteresis estimated by ANNs is almost the same as the observed one whereas; the SRC curve is not able to simulate hysteresis effect at all.



(a) Training



(b) Testing



(v) Validation

Fig. 12 : Comparative performance of observed sediment concentration with estimated sediment concentration using ANN9 and rating curve of sediment concentration at Penganga Bridge Site

Table 4 : Comparative performance of various runoff-sediment ANN models and conventional technique or Penganga Bridge site (Penganga river)

ANN model	No. of hidden layers	Training			Testing		Validation	
		RMSE	R	DC	R	DC	R	DC
ANN1	One	16.884	0.999	0.999	0.671	1.0232E-6	0.850	2.0127E-6
ANN2	One	0.120	0.919	0.845	0.784	0.589	0.911	0.827
ANN3	One	0.131	0.903	0.814	0.784	0.594	0.920	0.844
ANN4	One	**0.139**	**0.890**	**0.790**	**0.788**	**0.617**	**0.924**	**0.853**
ANN5	One	0.079	0.996	0.934	0.789	0.616	0.843	0.709
SRC	-	0.271	0.734	0.204	0.637	0.178	0.785	0.269

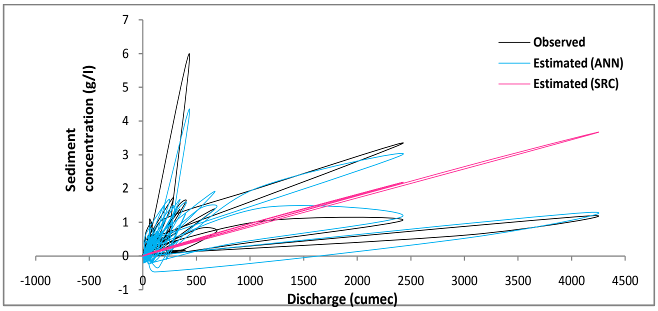


Fig. 13 : Comparison of the Observed Hysteresis with Estimated Hysteresis at Penganga Bridge -ANN5 (PENGANGA RIVER)

CONCLUSIONS

In the presented study a comprehensive review of sedimentation impacts on reservoirs and their remedial measures have been discussed and the status of the same was narrated briefly in respect Nagarjunasagar Dam and Musi dam located in Nalgonda district of Telangana State, India. The study also made an attempt to suggest simulation and modeling studies and their necessity to estimate, forecast and predict the sediment transportation in the river system. For the same an ANN technique utilized for modeling the sediment-discharge process in Penganga river at Penganga bridge site as a case study. The main objective of the study is to illustrate the capability of the ANN technique for modeling the sediment transportation in rivers. To achieve the objectives, a case study has been done utilizing the data of Penganga bridge gauging site of Penganga river (a sub-basin of Godavari River) in India for the analysis. At this juncture the study strongly recommends that the precise and less time consuming sedimentation estimation studies like ANN methodologies are required for the better development and management of the reservoirs in a river system.

REFERENCES

Agarwal, A., Mishra, S. K., and Ram, S., and Singh, J. K. 2006. Simulation of runoff and sediment yield using artificial neural networks. Biosystems Engineering, 94(4): 597-613.

ASCE Task Committee.on Application of Artificial Neural Networks in hydrology (2000 a). Artificial neural networks in hydrology. I: Preliminary concepts. Journal of Hydrologic Engineering, ASCE,5(2): 115-123.

ASCE,Task committee on Application of Artificial Neural Networks in Hydrology(2000 b).Artificial neural networks in hydrology. II: Hydrologic applications. Journal of Hydrologic Engineering, ASCE, 5(2): 124-137.

Hsu, K., Gupta, H.V., &Sorooshian, S. 1995, “Artificial Neural Network Modelling of the Rainfall-Runoff process”, Water Resources Research, vol. 31(10), pp.2517-2530.

Kalin, L., Govidaraju, R. S., and Hantush, M. M. 2004. Development and application of a methodology for sediment source identification. I: Modified unit sedimentograph approach.” Journal of Hydrologic Engineering, ASCE,9(3):184-193.

Haykin, S. 1994, “Neural Networks - a Comprehensive Foundation. Macmillan”, New York.

Kathleen M. Healy, St. Louis, MO, Amanda L. Cox, Daniel M. Hanes, Lisa G. Chambers, “state of the practice of sediment management in reservoirs: minimizing sedimentation and removing deposits”.

Lohani, A.K., Goel, N. K., and Bhatia, K. K. S. 2007. Deriving stage-discharge-sediment concentration relationships using fuzzy logic. Hydrological Sciences Journal, 52(4): 793-807.

NPTEL Version 2 CE IIT, Kharagpur, “Hydraulic Structures for Flow Diversion and Storage”.

Raghuvanshi, N. S., Singh, R. and Reddy, L. S. 2006. Runoff and sediment yield modeling using artificial neural networks: Upper Siwane River, India. Journal of Hydrologic Engineering, ASCE, 11(1): 71-79.

Rai, R.K., and Mathur, B. S. 2008. Event - based sediment yield modeling using artificial neural network. Water Resources Management, 22(4): 423-441.

Raju, M. M., Srivastava, R. K., Bisht, D.C.S., Sharma, H. C., and Kumar, A. (2011) Development of artificial neural-network-based models for the simulation of spring discharge, Advances in Artificial Intelligence, Volume 2011, Article ID 686258.

Sarkar, A., Kumar, R., Singh, R. D., Thakur, G., and Jain, S. K. (2004).Sediment-discharge modeling in a river using artificial neural networks.Proc. International Conference ICON – HERP, Oct 26-28, 2004, IITRoorkee, India.

Sarkar, A., Raju, M. M., and Anilkumar (2010).Sediment runoff modeling using artificial neural networks, Jr. ofIndian Wat.Resour.Society, IWRS ,IITRoorkee, India. January Vol. 30, No. 1 (39-45).

Thomas, T., Jaiswal, R. K. and Singh, S. 2006.Hydrological modeling of streamflows using artificial neural networks. Proc. 2nd International Conference on Hydrology and Watershed Management, Centre for Water Resources, Jawaharlal Nehru Technological University, Hyderabad – 500 072, Andhra Pradesh, India.

Zhu, M., Fujita, M., and Hashimoto, N. 1994, “Application of Neural Networks to Runoff Prediction, “Stochastic and Statistical Method in Hydrology and Environmental Engineering, vol. 3, K.W. Hipel et al., eds., Kluwer, , The Netherlands, pp.205-216.

Challenges in Design and Construction of Punatsangchhu-I Dam with Deep Foundation and Adverse Geology

R.K. Gupta and Vivek Tripathi
Central Water Commission, India

Kanupreiya
Wapcos Ltd., India

ABSTRACT

Construction of concrete dams with deep foundations poses several challenges during planning, design and construction stages. Planning of access to the dam foundation level through excavation is a major challenge. During deep cutting, abutments slopes may get destabilize and may require extensive stabilization measures. The problem aggravates if the adverse geological features are encountered in abutments or at dam foundation level. Effective measures for seepage control through foundation of coffer dams and main dam as well as through abutments are required. Well defined analysis using analytical tools are sometimes not possible due to heterogeneous and complex rock mass conditions. In Punatsangchhu-I Hydroelectric Project, which is in advanced stage of construction in Bhutan, deepest dam foundation level is 75 m below river bed. Adverse geology in terms of presence of shear zones and fractured rock mass in right abutment of dam resulted in slope stability problems. Stabilization measures in terms of grouting, micropiling and cable anchoring have been adopted to make the slope stable. Dam foundation improvement through grouting and installing 2 m diameter concrete piles has been carried out. For seepage control, plastic concrete diaphragm wall below upstream coffer dam and secant piles below main dam blocks resting on relatively weaker rock mass have been provided. In general, the challenges and difficulties of dam construction in deep and adverse foundations are many, but the same can be minimized through detailed investigations, meticulous planning and design and by using latest technologies during construction.

Key-words *: Abutment, Concrete Dam, Cofferdam, Cutoff Wall, Diaphragm Wall, Foundation, Geology, Geotechnical Investigation, Grouting, Pile, Punatsangchhu-I, Seepage, Slope Protection, Slope Stability.*

1. INTRODUCTION

Foundation of a dam is a critical structural component and requires special attention throughout the life time of the project. In recent years, several dams with difficult foundation problems have been built successfully.

Some geological settings require more than usual attention in a dam engineering, which cause problems and necessitate special care in planning and design as well as during construction. All dam sites with deep foundations and heterogeneous geological conditions are suspected to be problematic and require special attention. A Dam foundation of the order of 40 m to 50 m below river bed is quite common. However, deep foundations of the order of 75 m below river bed coupled with adverse geological features pose challenges in selection of dam type, planning of river diversion works, slope stabilization and foundation improvement works. In under construction Punatsangchhu-I Hydroelectric Project, the maximum depth of overburden or river borne material at dam axis is 75 m with heterogeneous adverse geology at right abutment. This necessitated special attention during planning and design of dam and appurtenant works and posed several challenges during construction.

Punatsangchhu-I Hydroelectric Project is located on river Punatsangchhu in Wangdue district of Bhutan. The project is a run-of-the-river type and envisages construction of 136 m high concrete dam. The water is diverted through a 10 m diameter, about 9 km long Head Race Tunnel and is taken to a underground Power House housing six turbines of 200 MW capacity each. The project is in advanced stage of construction.

2. GEOLOGICAL SETUP AT DAM SITE

2.1 Regional Geology

Regionally the project area is located within a part of the Tethyan Belt of Bhutan Himalayas, which lies in the north of Main Central Thrust (MCT) and the rocks of Sure Formation of Thimphu Group of Precambrian age are exposed. The rocks of Thimphu Group in general are characterized by coarse grained quartzo-feldspathic, biotite-muscovite gneiss with bands of mica schist, quartzite and concordant veins of pegmatite, leucogranite and migmatites with minor bands of limestone/dolomites and metabasics. In addition, colluvial deposits derived from the country rock and river terraces formed on the right bank of Punatsangchhu River are also seen in the area. On the basis of study of aerial photographs three lineaments have been picked up in the area, which trends in (i) N-S (ii) NW-SE and (iii) NE-SW directions. The Punatsangchhu River probably flows along one of such sympathetic N-S trending lineament in the project area.

2.2 Geology of the Project Area

The bedrock exposed in the project area are represented by garnet bearing, foliated and jointed quartz-biotite gneiss and quartzo-feldspathic gneiss with bands of muscovite-biotite schist and veins/bands of pegmatite & quartz. These are medium to high grade, crystalline, medium to coarse grained metamorphosed rocks with well-developed foliation and gneissosity. The general foliation of the strata trend in N10^0-60^0E to S10^0-60^0W and dips at 10^0-40^0/N100^0-150^0. At places, rock exhibits tight 'S' shape folds and broad warps as evidenced from the swing in foliation. The rocks had undergone polyphase deformations leading to development of numerous tectonic dislocations, mega shears, and discontinuities of different orders. In Dam complex, the rocks comprising quartz-biotite gneiss with small schistose bands, thin quartz veins and very small pegmatite bands are exposed. Eight sets of joint are recorded, amongst these five sets of joints are prominent and others are occasional or random joints.

2.3 Geology at Dam Site

The river valley at dam site is characterized by steep rocky cliffs on the left bank and gentle abutments on the right bank, exposing medium to coarse grained, moderately jointed and blocky quartz rich biotite gneiss as a bed rock. It was inferred that the left banktopography is controlled by valley ward dipping steep andcontinuous striking master joints, whereas that of right bank is controlled by gentle to moderate dipping foliation joints.The exploratory drill holes was done to evaluate the rock mass condition and to establish the fresh bed rock level, revealed that the presence of thickcolluvium/hill wash material consisting of rocky boulders, cobbles and gravels set in overburden soil (Silty and sandy matrix) with intermediate zones of RBM in the river bed and 20 - 25 m wide zone of RBM below El 1125 m, comprising of sandy layers and well-polished pebbles of gneiss, quartzite and leucogranite, located towards left edge of the present course ofthe river. This thick RBM zone represents a buried channeldeveloped due to deposition of riverine material along the old course of the river. Due to presence of thick colluviumunderlain by deep buried channel, the deepest level of the bedrock was found at El 1075 m, i. e about 75 m deeper from river bed. The overburden soil is lightyellowish to light brownish coloured, granular from very fineto medium grained. It is the matrix of silt, sand and fine gravels but the major part of the soil is silty. The average percentage of rocky boulders in soil matrix ranged from 30 % to 40%.

Profuse fracturing and alteration along the joints inthe drill cores on the right abutment strongly suggested the presence of a shear zone running slightly askew to the River channel in this area. Considering the fair rock quality designation (RQD), joints disposition and rock mass condition of the bed rock observed in the drill core, it was inferred that afair quality of rockmass with 'Q' value ranging from 4 to 8 would be available at the deepest foundation level.

3. SELECTION OF DAM TYPE

Generally, the foundation of a dam site is determining factor in the selection of the type of dam apart from other factors. For a site with thick overburden, an embankment dam is a preferred choice. However, at the dam site of Punatsangchhu-I Project, the valley is not wide enough to accommodate separate spillway requirements for passing high PMF of 15,800 m^3/s. Options of providing multiple spillways i.e. surface and tunnel spillways were also explored for an embankment dam. As, left bank has geologically sound set up, diversion tunnels and power intakes were planned at left bank. Planning of separate tunnel spillways in left bank became almost impossible. Moreover, use of diversion tunnels as spillways was not found to be sufficient due to capacity constraints. Planning of spillway tunnels through right bank was also not possible due to presence of thick overburden. Accordingly, it was decided to have a concrete gravity dam with sluice spillways in its body.

The top of the dam is at El 1205 m and the deepest foundation level is at El 1075 m. The river bed level at dam axis is El 1150 m. Hence, the maximum dam height is 130 m. Six bays of sluice spillway have been provided. The crest of sluices is at El 1166 m.

4. PLANNING OF RIVER DIVERSION WORKS

River diversion is one of the first steps to unwater the dam foundation area between the cofferdams to proceed with excavation and treatment of the foundation and to construct the lower part of the dam. It involves the construction of coffer dams as well as other works such as diversion tunnels, diversion channels or other conduits.

4.1 Deciding Diversion Discharge

For deep dam foundation of Punatsangchhu-I Project, one dry season was not found sufficient to excavate the dam pit. Considering the time required to excavate the dam pit, high risk of loss of lives & damage to equipment in case of overtopping of coffer dam and damage to the not fully excavated site & time required to clean it, it was decided to have a diversion discharge which is maximum of observed discharge in the river at dam site and a discharge corresponding to a return period of 1 in 25 years annually. Accordingly, a diversion discharge of 2430 m^3/s was considered appropriate for the dam site of Punatsangchhu-I Project. This discharge was considered sufficient for providing safe working conditions in dam pit.

4.2 Planning of River Diversion Works in Deep Foundations

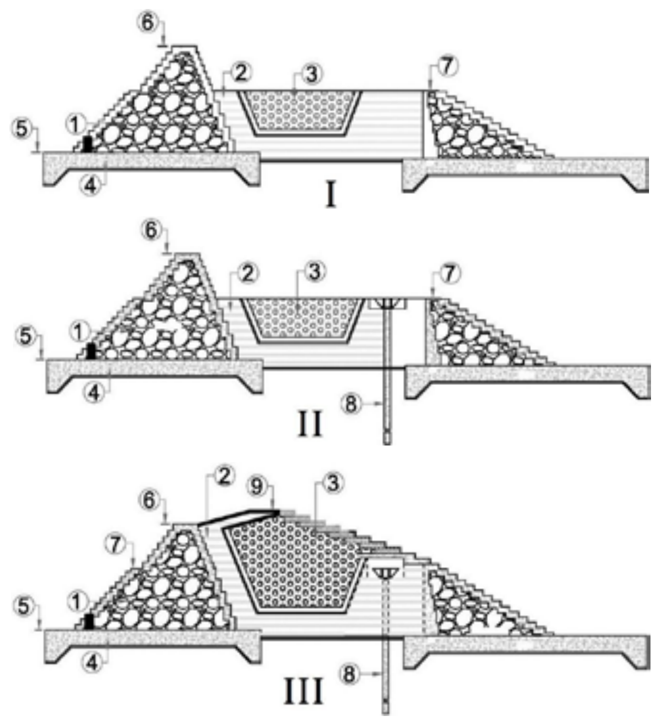
For passing a diversion discharge of 2430 m^3/s, two tunnels of 11 m diameter each with a top level of upstream coffer dam at El 1187 m was found sufficient. At the location of upstream coffer dam, the river bed level is at El 1163 m. Hence, a upstream coffer dam with maximum height of 24 m was planned. The maximum height and top elevation of downstream coffer dam is 12 m and El 1158 m.

Generally, both coffer dams are located little away from the point from where excavation in river bed starts to reach dam foundation. For a dam foundation which is 75 m below river bed, if excavation in river bed is carried out in 1: 1 slope, then a length of about 75 to 100 m is sufficient for slope cutting in 1:1 with berms. However, for movement of construction equipment, which generally require a minimum slope of 1: 10, this 100 m length was not found sufficient for providing haul roads with proper gradient to reach dam pit. As it was decided to approach dam pit from downstream side i.e. from top of downstream coffer dam, a length of about 700 m was required between downstream coffer dam and toe of main dam. Keeping coffer dams much away increases the length of diversion tunnels. Accordingly, it was decided to keep downstream coffer dam 450 m away from the dam axis. Instead of keeping straight, a meandering haul road in river bed was planned from top of downstream coffer dam tomain dam pit.

For planning the type of upstream coffer dam, seepage through its foundation was a serious concern considering the depth of dam pit. Several examples of leakage through abutments and foundation of dams have been witnessed world over, but with the advent of modern technology and innovative approach, such problems are being tackled successfully. The piping and internal erosion within thick overburden in river valley was a major geotechnical issue for upstream coffer dam of Punatsangchhu-I Project. The plastic concrete cut off wall as diaphragm was planned as best option to encounter the water leakage through the dam body up to acceptable level.

To facilitate the work of construction of diaphragm wall, a composite type coffer dam was designed, which has colcrete as well as rockfill with clay materials. The upstream coffer dam was constructed in three stages. In first stage, upstream colcrete portion upto El 1184.4 m, downstream colcrete portion upto El 1176 m and clay portion in between was constructed. This was planned to provide a working platform for construction of plastic concrete diaphragm wall through clay, which was going to take more than one dry season. In second stage, diaphragm wall was constructed. In third stage, remaining portion of coffer dam was completed.

Construction of 1.2 m thick, 132 m long and maximum 93 m deep plastic concrete diaphragm, which is one of its own kinds and deepest diaphragm wall in the geologically complex Himalayan terrain was carried out. Fig. 1 shows the details of upstream coffer dam.



1 Colcrete
2 Clay
3 Rockfill
4 Raft
5 EL. 1164.40 m
6 EL. 1184.40 m
7 EL. 1176.00 m
8 Plastic concrete diaphragm/ cutoff wall
9 EL. 1187.00 m

Fig. 1 : Details of upstream cofferdam constructed in three stages

The downstream coffer dam was made of rockfill material. To reduce the seepage through its foundation, jet grouting and permeation grouting were carried out.

5. ABUTMENT SLOPE STABILIZATION AND FOUNDATION IMPROVEMENT

Prior to start of excavation in dam foundation area, geological investigations were carried out through bore holes and drifts. Considering the high depth of overburden in river channel section, bore holes were drilled up to sufficient depth in sound rock. The left abutment was also thoroughly investigated and was found to be steep but stable with sound rock conditions. The right abutment is gently sloped and has thick overburden at various locations. Bore holes which were drilled up to about 20 m in rock revealed that the rock condition was fragile at few locations. It is pertinent to note that in right abutment bore holes in rock were not drilled up to the deepest level of dam foundation. Hence, deep seated shear seams and fractured rock mass below the top sound rock could not be delineated.

5.1 Slope Movement and Further Investigations

When excavation in dam pit reached the level of El 1190 m at right abutment, movement of the order of 3 m to 5 m occurred at a particular day. Accordingly, further excavation works were stopped. Slide area was demarcated and to find the cause of movement, geo-technical investigations were carried out by drilling a total length of about 4000 m in 37 number of holes. Deep geological investigations revealed presence of deep seated two major shear zones along with fractured rock mass of varying thickness at various locations.

While benching down in dam pit, stabilization measures in terms of shotcrete with wire mesh and rock bolting were being carried out. But, these were not sufficient to stabilize the deep seated shear seams and fractured zones and rock slope slipped when the shear zone got exposed little above El 1190 m.

5.2. Slope Stabilization Measures

Stability analysis of slope indicated that the slope has factor of safety marginally less than one. However, numerical model indicated that if the abutment is considered fully saturated, the factor of safety declines. To stabilize the slope, following stabilization measures were taken:

(i) Grouting of rock mass in slide zone

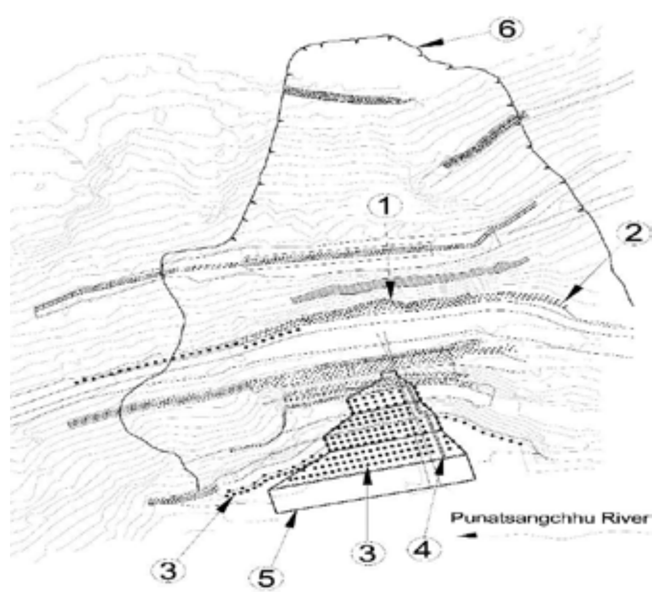
(ii) Micropiling

(iii) Cable anchoring

Grouting of rock mass efficiently treat discontinuities of minor to medium size where more or less open jointing allows penetration of grout of sufficient mechanical resistance. Accordingly, grouting with cement was carried out to consolidate the rock mass in slide zone specially the regions with shear seams and fractured rock mass. Grouting improves the shear strength of rock mass and thus increases stability of slope against sliding. At right abutment, this was done at different available benches i.e. at El 1205 m, El 1216 m, El 1265 m and above. The entire slide zone was covered.

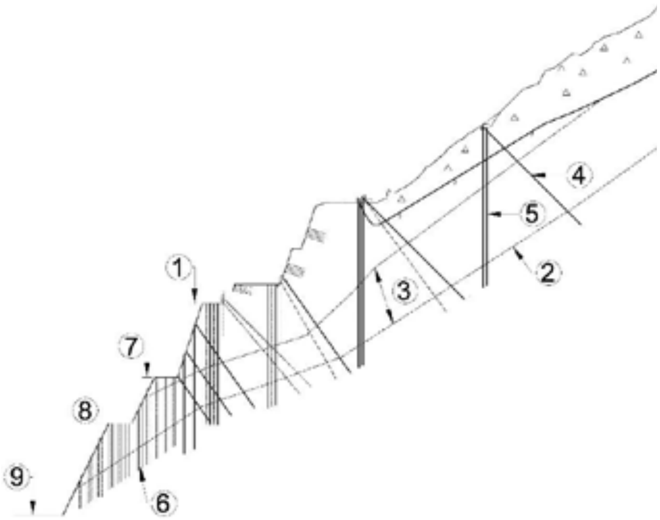
Micropiles were also installed at different benches covering the almost entire slide zone. As Micropiles have capacity for axial force as well as for bending, it increases slope stability by providing resistance to slip. 325 mm diameter micropile having steel circular casing filled with concrete were installed. Reinforcement bars were also provided inside the steel casing. These piles were taken to sound rock with a socket length of 10 m. The length of piles varied from 50 m to 80 m depending upon the depth at which sound rock was encountered. Two to three rows of closely spaced (2 m centre to centre) micropiles were provided in benches at different levels.

Cable anchoring was also carried to along with micropiles at different benches in the slide zones. Cable anchors of capacity 100 T were installed and were pre-tensioned. These cable anchors were also taken into sound rock by 10 m designed as socket length. Cable anchors are also one of the most effective way of improving the shear resistance of slope and thereby increasing its stability. Fig. 2 shows the stabilization measures in right abutment of dam. Fig. 3 shows the cross-section of right abutment at dam axis exhibiting various stabilization measures.



1 Cable anchors
2 Micropiles
3 2.0 m diameter piles
4 2.0 m diameter secant piles
5 Dam foundation plan of right most three blocks
6 Slide zone

Fig. 2 : Plan of right abutment showing stabilization measures



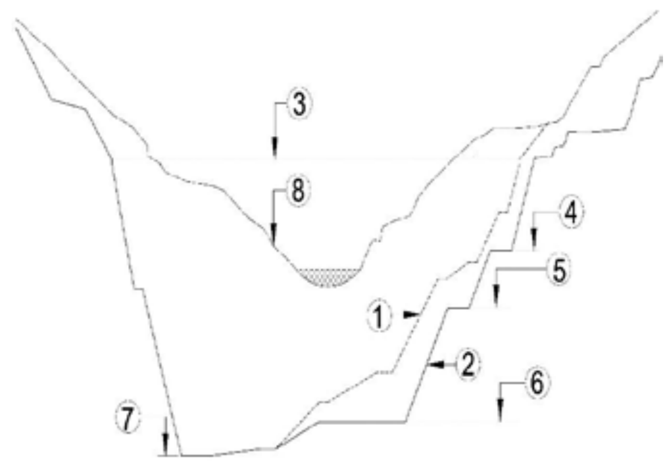
1 Dam top
2 Sound rock line
3 Weak rock zone
4 Cable anchors
5 Micropiles
6 2.0 m dia. piles
7 EL. 1165 m
8 EL. 1140 m
9 EL. 1090 m

Fig. 3 : Section along dam axis showing stabilization measures in right abutment

After the dam construction, it has been planned to fill back the excavated dam pit by rock muck up to El 1160 m i.e. 6m below the crest of sluices. This will almost restore the original river bed profile and will increase the right abutment slope stability.

5.3. Change in Abutment Cutting Profile and Foundation Improvement

Due to slip of slope, the joints in rock mass at excavated surface below El 1205 m were opened. Accordingly, it was decided to excavate the rock further at right abutment. As the rock mass was weak and consist of shear seams, it would have been more prudent to excavate right bank from El 1205 m to El 1090 m in a straight cut and place the right most dam block at El 1090 m. However, excavation of 115 m slope without any bench in between was proving to be completely unstable. Accordingly, two benches at El 1165 m and at El 1140 m were provided before reaching the level of El 1090 m. At these two benches, two right most dam blocks rest.Further excavation below El 1205 m upto El 1165 m was carried out in three stages and cable anchors were provided after each stage to keep the slope stable. Fig. 4 shows the old and new cutting profiles at right abutment.



1 Old cutting profile
2 New cutting profile
3 Dam top
4 EL. 1165 m
5 EL. 1140 m
6 EL. 1090 m
7 EL. 1075 m
8 NSL

Fig. 4 : Abutment excavation profiles for dam construction

As the two right most dam blocks rests on relatively weaker rock mass, foundation improvement was carried out by installing 2 m diameter concrete piles. These piles were provided at El 1165 m &El 1140 m.Piles were also provided on slopes between El 1165 m & El 1140 m and between El1140 m & El 1090 m. These piles were taken in to sound rock by 10 m designed as socket length. Reinforced concrete was used in these piles. A total number of 190 piles were installed. The rock at these two benches and at slopes was also thoroughly cement grouted.

5.4 Seepage Control

The seepage through the foundation of two right most dam blocks at El 1165 m & El 1140 m was a matter of concern. Accordingly, a row of 2 m diameter pile was provided as secant pile continuously from El. 1165 m to El. 1090 m. The location of secant piles has been kept upstream of foundation gallery through which curtain grouting will also be carried out. The secant piles acts as a barrier for water seepage.

Further, to control the seepage from reservoir to right abutment, grout curtain at El 1205 m from dam axis in upstream direction (along longitudinal direction) will be created.

CONCLUSIONS

Dams can be built successfully even at sites with difficult foundation conditions. Deep foundations and heterogeneous and adverse rock mass at foundation level as well as in abutments pose challenges in planning, design and construction of river diversion works, main dam foundation, stabilization measures for slope stability and seepage control measures. Thorough geological investigations are necessary to minimize surprises.

Foundation for dams usually requires some treatment to satisfy the requirements of stability, deformation and water tightness. By applying appropriate corrective measures, known as ground improvement works, foundations can be treated. Today, various measures are available to increase foundation shearing strength, stability of abutments, foundation stiffness and to reduce foundation seepage. Ground improvement works depend significantly on the type and size of dam. Various forms of treatment improve the response of the foundation to the loads and hydraulic conditions imposed by the dam and the reservoir, as well as seismic events and are essential elements in the safety of a dam.

Deep foundation and adverse geology at right abutment as encountered at dam site in Punatsangchhu-I Project posed several challenges in planning, design and construction of river diversion works, slope stabilization measures, foundation improvement and seepage control, which have been tackled using latest technologies.

REFERENCES

1. ICOLD Bulletin 129. Dam Foundations, 2005.
2. Gajbhiye, P.K. Construction of deep plastic cut-off wall in upstream coffer dam of Punatsangchhu-I Hydropower Project, Bhutan. Recent advances in rock engineering, 2016.

Soft and Erodible Foundation Rock in Pare Concrete Gavity Dam, Arunachal Pradesh, India

Ranendra Sarma and V.K. Singh
North Eastern Electric Power Corporation Limited

ABSTRACT

Pare Hydro Electric Project (110 MW), Arunachal Pradesh, India owned by North Eastern Electric Power Corporation, a Government of India Undertaking is in an advanced stage of completion with likely commissioning in Dec 2017. Soft, immature, poorly cemented sand stone belonging to Kimin Formation of Upper Siwalik Group of Pliocene-Pleistocene Age as dam foundation has posed a major challenge in design and construction of 63 m high concrete gravity dam. The soft rock in dam foundation with presence of a few wide fissures carries the risk of erodibility in the long run after impoundment of reservoir. Therefore, adoption of good seepage management measures in dam foundation to minimize the risk of foundation erodibility has become a necessity. Groutability test carried out prior to construction indicated a very poor permeability of bedrock in general with occasional grout intake in certain areas. Therefore, conventional curtain grout as seepage arresting measure beneath the dam foundation cannot be depended upon and this needs to be supplemented with other seepage management measures like positive cut-off wall, drainage curtain, etc. Based on seepage analysis, two cut-off walls, one at upstream edge and the other at downstream edge of dam are designed and constructed. Plastic concrete with low compressive strength is used in the walls. The main intention of plastic concrete is to provide desired compressive strength, low permeability and elastic modulus close to properties of adjacent soft foundation strata. Robust drainage system is provided to capture water from as many erratic bearing joints of the foundation as possible so as to reduce foundation uplift force and seepage force beneath the foundation. Additional drainage gallery in downstream connected with three rows of dewatering curtain is provided as part of drainage system. Contact/consolidation grouting with microfine cement is carried out before pouring of dam concrete in order to improve mechanical properties of foundation rock. Grout intake is not significant and it matches with earlier groutability test result. Curtain grout with deep holes shall be tried despite low grout intake estimation becuase of poor groutibility of rock. Based on Lugeon value of rock after completion of primary holes, decision on secondary and tertiary holes shall be taken. Post construction monitoring of Pare Dam is very crucial to assess performance of the structure vis-a-vis seepage and erodibility threat. Dam instrument outputs, observation of ground water in the vicinity, behaviour of already mapped and monitored seepage points in the adjoining areas shall help in evaluating performance of the dam.Successful construction and subsequent functioning of Pare dam shall be an important milestone in the path of dam construction in challenging rock condition.

Keywords *: Concrete Dam, Drainage, Seepage, Foundation Treatment, Grout Curtain, Cutoff*

1. INTRODUCTION

Pare Hydro Electric Project is a run-of-the river scheme situated in Papumpare district of Arunachal Pradesh, India. The project is being developed with creation of a pondage by constructing a 63 m high concrete gravity dam across Pare River and the pondage so created shall be diverted through a 2.81 km long, 7.50 m diameter tunnel to a surface power house of 2 x 55 MW installed capacity. The concrete dam is 134.0 m long with ten blocks; middle three blocks fitted with radial gates shall act as spillway. The project owned by North Eastern Electric Power Corporation (NEEPCO), a Government of India Undertaking envisages utilisation of water of both river Pare and tailrace discharge of Ranganadi Hydro Electric Project. With likely commissioning in December 2017, the project shall produce 506 Million Units of energy per year. The Project is being developed with financial assistance of kfW, Germany. Panel of Experts (POE) with international experts, Design Consultant appointed by owner, Design Consultant appointed by the financier from time to time are actively engaged in the project.

Soft and friable dam foundation rock having high erodibility potential is a major challenge in the project. Poor groutability of rock compounds the problem.

2. DAM GEOLOGY

The rock in the dam site belongs to Kimin Formation of Upper Siwalik Group of Pliocene-Pleistocene Age and represented by medium to coarse grained grey to buff coloured, micaceous soft and friable sandstone (sand rock). The sandstone displays a typical salt-pepper texture and gritty at places with pebble layers along the bedding plane[1]. The rock, in general, is soft, poorly cemented and immature in nature. Relatively stronger and slightly better lithified bands are also observed at places. Rock is characterised with a very low deformation modulus value (0.3-1.0 GPa). No major geological feature is available in dam and other project structures. In order to have geo-hydraulogical inputs, four piezometers are installed on both banks and river bed blocks (5.0-7.0 m from dam foundation level). Variation of water table is observed with higher water table (4.0-8.0 m) on right bank with respect to that of left bank. Two seepage points on left bank are monitored. Variation of yield is 0-1500 ml/min and 1800-2500 ml/min in dry season and wet season respectively.

3. GROUTABILITY TEST

In the Detailed Project Report a conventional, single row grout curtain was proposed to keep the seepage through the foundation within permissible limits. Keeping in view the nature of bedrock likely to be encountered in the foundation of the dam, a groutability test program with seventeen number holes inthree meter long sections with hole depths varying from 33.0 m to 43.0 m.was implemented near the dam site as shown in Fig. 1 to establish the efficacy of the grouting in such media. The test indicated very poor permeability of bedrock in general with occasional grout intake in certain areas. Grout intake was very low in majority of grout sections, as grout mixture did not travel considerably beyond the periphery of the holes[2]. The test confirmed very low dependability of conventional cement grout curtain as seepage arresting measure in the project. At the same time,erratic permeability and groutability in certain foundation areas confirmed co-existance of majority tight sections with a few wide fissures. Isolated seepage points noticed in otherwise dry dam excavation areas also indicated the same. The soft rock in dam foundation with presence of a few wide fissures carries the risk of erodibility in the long run after impoundment of reservoir[3]. Therefore, it became imperative to go for good seepage management measures in dam foundation to minimize the risk of foundation erodibility.

4. SEEPAGE MANAGEMENT

4.1 Cut-Off Wall

Detailed seepage analysis was carried out as part of cut-off wall design. Wall depth was optimized duly considering incremental benefit in the form of reduction of exit gradient with increase in wall depth. Accordingly, two walls, one at upstream edge of dam with 10.0 m depth below three overflow blocks which gradually reduces to 5.0 m at bank blocks and the other at downstream edge of dam with 5.0 m depth are provided as hydraulic barrier to the seepage underneath the dam. Plastic concrete with characteristic compressive strength of 8 N/mm^2 is provided in the walls. The main intention of plastic concrete is to provide desired compressive strength, low permeability and elastic modulus close to properties of adjacent soft foundation strata.

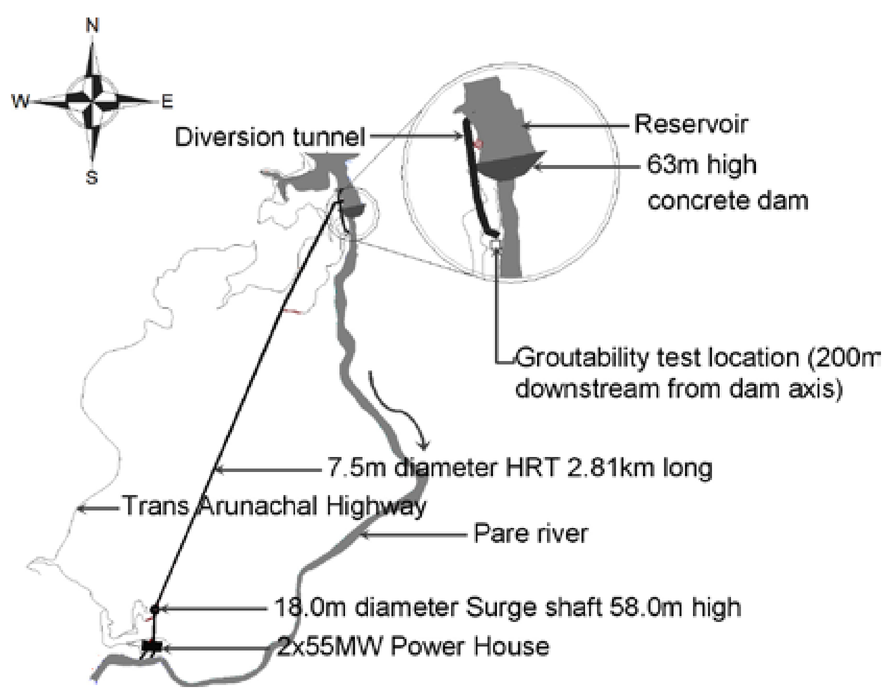


Fig. 1 : Project layout showing groutability test location

4.2 Foundation Drainage

Robust drainage system is provided to capture water from as many erratic bearing joints of the foundation as possible so as to reduce foundation uplift force and seepage force beneath the foundation. In addition to the upstream drainage gallery, a downstream drainage gallery of 2.0 m x 2.5 m is provided in three riverbed blocks. A 100 mm diameter steel pipe connects the downstrean gallery with three drainage curtains of 15.0 m depth spaced 3.0 m apart. Each curtain consists of pea gravel filled 75 mm diameter pipes at 3.0 m spacing. Upstream gallery is similarly connected with a single curtain of 15.0 m depth having 75 mm diameter pipes at 3.0 m spacing. Fig. 2 shows the basic elements of drainage system in downstream side of dam.

5. CONSOLIDATION GROUTING

In order to improve mechanical properties of foundation rock, consolidation grouting with 8.0 m depthin two stages of 4.0 m eachis carried out before pouring of dam concrete. The grid for grouting with primary, secondary and tertiary holesis made with a spacing of 1.5 m in both lateral direction and flow direction. Microfine cement is used to get better groutability. Grout consumption patternis matching with groutability test results with a consumption range of 330 kg/hole to 25 kg/hole. Primary holes are initially grouted followed by grouting in secondary holes. Because of low grout consumption in primary and secondary holes, tertiary hole grouting is carried out only in a few holes. Maximum pressure is 2.5 kg/cm^2 in first stage (4.0-8.0 m) with incremental increase of pressure. Similarly, maximum pressure is 1.5 kg/cm^2 in second stage (0-4.0 m) with incremental increase in pressure. The ratio of grouting material to water is maintained at 1:10.

6. CURTAIN GROUTING

Cut-off walls already placed shall be supplemented with curtain grouting. Isolated zones of high permeability shall be captured and managed with a systematic curtain grouting procedure formulated by the POE. First twenty 40.0 m deep alternating primary holes with a spacing of 12.0 m over the whole cross section of the dam shall be drilled in five meter steps.Lugeon tests shall be performed in each step with suggested pressure increments. Drilling shall be followed by microfine cement grouting from bottom of the hole.Gruting preesure shall be based on critical preesure leading to hydro fracture of rock to be determined during Lugeon test. This shall be followed by drilling and grouting works in second twenty primary holes. Based on grout consumption of primary holes, decision on secondary and tertiary holes alongwith their depths shall be taken. With threshold Lugeon value of 1.0 for grouting, significant grout intake is not ancipated.

7. INSTRUMENT SCHEME TO MONITOR PERFORMANCE

Initial instrument schme consisted of Uplift Measing Device, Pore Pressure Cell,Stress Meter, Strain Meter (group of five), No Stress Strain Meter, Joint Meter, Temperature Gauge, Direct Plumb Line, Inverted Pendulum, Stand Pipe Piezometer, Surface Settlement

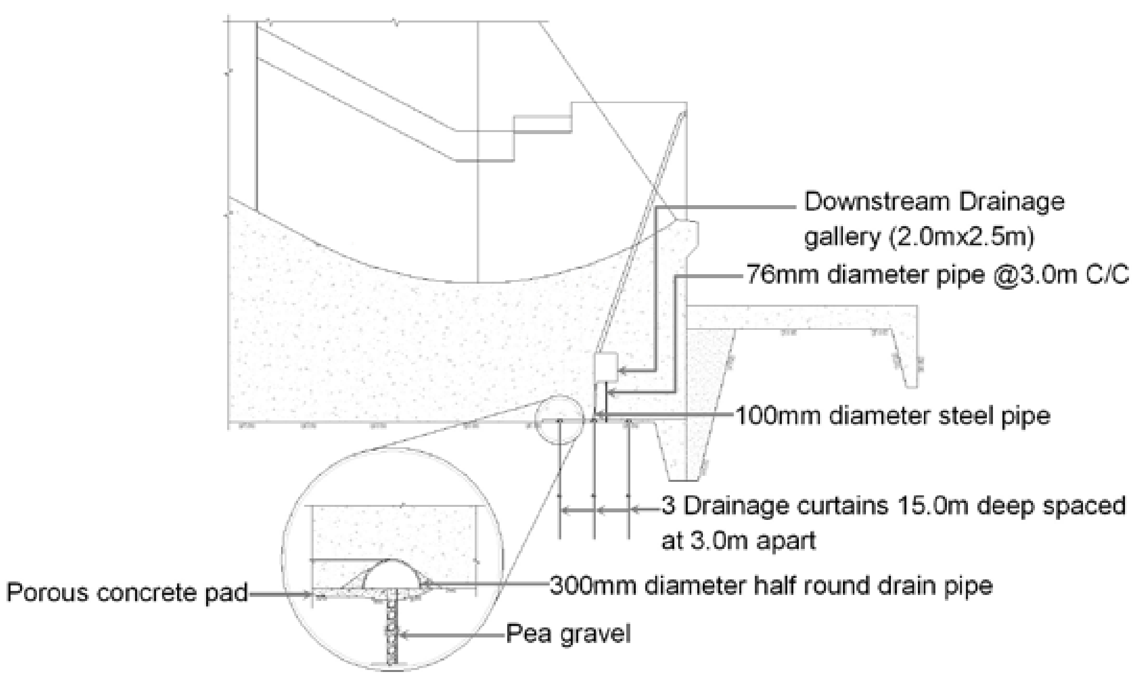


Fig. 2 : Drainage system in downstream side of dam

Point, Strong Motion Accelerograph and V-Notch Weirs. Provision for additional instruments like Settlement Measuring Bolts, 3D Joint Meter at all joints, Piezometer and Pore Pressure Cells at higher levels of non-overflow blocks, Stand pipes at two levels just downstream of dam in each abutment and MPBX into rock foundation is made duly considering poor foundation condition with uncertain seepage potentiality. Scheme is further strengthened with four V-Notch Weirs to differentiate between upstream & downsteam and left & right bank seepage, six Vertical Extensometers, 4 Horizontal Expensometers and five Piezometers at 3.0 m downstream of dam axis.Readings of already installed instruments don't show any noticeable behaviour of dam and its foundation.

8. CONCLUDING REMARKS

Performance of the Pare dam during operation of the power plant shall need a very close monitoring backed by proper engineering analysis. Output of dam instruments installed is meant to form the basis for performance observation. Close observation of movement of water table in the vicinity of dam shall indicate efficacy of various remedial measures. Already mapped and monitored seepage points in the adjoining areas shall to a large extent be guiding points in assessing water regime after reservoir impoundment.

A number of dam based Hydro Electric Power Projects are planned in North Eastern part of India, some of which shall encounter very weak foundation rock. Successful construction and subsequent functioning of Pare dam shall be an important milestone in the path of dam construction in challenging rock condition in this part of India.

REFERENCES

1. Geological Survey of India. 2016. Geological Report on Dam Foundation for Pare Hydro Electric Project, April 2016, pp.5
2. SNC-Lavalin. 2012. Report on Test Grouting for Pare Hydro Electric Project, April 2012, pp.6
3. Sarma, R. Sustainability of hydro structures in erodible rock in two Hydro Electric Projects, Arunachal Pradesh, India. In: EUROCK 2015 & 64th Geomechanics Colloquium. Schubert (ed.) © ÖGG

Delineation and Treatment of Mega Shear Zone in the Main Dam Foundation: A Case Study of Punatsangchhu HEP-II, Bhutan

Ayodhaya Prasad Thapliyal, Yoginder K. Sharma and R.N. Khazanchi
Punatsangchhu Hydroelectric Project Authority II, Bhutan

Krishna Kumar Sati
Jaypee Group, Bhutan

Santosh K. Sati
Wapcos, Bhutan

ABSTRACT

Pre-assessment of important geological features and their characteristics, while constructing any hydropower project in the Himalayan terrain, is always a challenging task. Most of the projects encounter one or more unforeseen adversely oriented geological features, either during excavation of dam abutments and foundation or tunneling and back slope stripping of power house and pot head yard. These features may be in the form of fault zone, shear or crushed zone with flowing condition, buried channel, heavy ingress of water, squeezing zone etc. which makes the task, of construction Engineer and Geologist as most difficult, leading to the over runs in respect of time and cost aspects. This paper brings out various aspects in respect of investigations and treatments of thick shear zone (maxi. thickness 17.0 m) encountered during excavation of dam foundation for execution of Punatsangchhu-II HEP (PHEP-II), under construction on Punatsangchhu river along Wangdue-Tsirang National Highway in Wangdue Phodrang Dzonkhang of Bhutan. The shear zone was suspected during excavation of left abutment while drilling exploratory holes for demarcating abutment rock slope profile. Subsequently, in order to establish its nature and thickness, four additional exploratory drill holes were carried out. The shear zone was unique in the sense of its dimension, frequent changes in thickness, behaviour and attitude with the depth. These characteristics of zone could only be established after excavating a 5.0 m deep trench along its strike and dip. A detailed geotechnical assessment and geological mapping was carried out to demarcate shear zone thickness, continuity and infilling material. The shear zone was traversing from heal to toe of the dam, cutting across the dam foundation of block nos. 4, 5, 6, and 7. The treatment of the shear zone was designed by Central Water Commission (CWC) in the form of excavation of 125.0 mm long, 5.0-35.0 m wide and 13.0-15.0 m deep trench (from u/s CH 22.0 m to d/s CH 103.0 m), consolidation grouting, providing rock anchors on the side walls and finally the trench filled with PCC and RCC. This paper exclusively deals with the detailed investigations that have been carried out at dam site and their inference and the methodologies adopted for delineating the shear zone in the dam foundation and its treatment. Besides, the present studies also briefly describe the evaluation of dam foundation and foundation grade rock mass characteristics.

Keywords *: Mega Shear Zone, Dam, Excavation, Geotechnical, Exploratory Drilling, foundation.*

1. INTRODUCTION

The PHEP-II dam with a crest length of about 224.0 m, a maximum height above the river bed level of 91.0 m, and a gross storage capacity of about 7.0 MCM is under construction on the Punatsangchhu river, about 115 km east of Thimphu city, capital of Bhutan. Punatsangchhu-II hydroelectric project (120 MW), is under construction on river Punatsangchhu undertaken by the under initiatives Punatsangchhu hydro power authority (PHPA), in the parts of Bhutan Himalaya, is the of the first three projects of the 10,000 MW initiatives taken by the Royal Government of Bhutan (RGoB) and the Government of India (GoI) in May, 2008 for the mutual benefits of both the countries. The Punatsangchhu-II Hydroelectric Project (1020 MW) is a run-of-the-river scheme and it is spread over a distance of about 15 km along Wangdue - Tsirang National Highway in Wangduephodrang Dzongkhag of Bhutan and it is approximately 115 km east of Thimphu, the Capital of kingdom of Bhutan. The project area in lies in the Eastern Himalayas between Latitude 26^0 70' and 28^0 40' N and Longitude 88^0 70' and 92^0 20' E covering a geographical area of 38,394 km^2 (Fig.1). The catchment area (up to dam site) 6390 km^2 extends between Latitude 27^0 15' and 28^0 30' N and Longitude 89^0 15' and 90^0 30' E. The nearest airport Paro is located at 125 km distance.

Fig. 1 : Location Map of the Project

1.2 Profile of the Project

The project envisages construction of 91.0 m high & 224.9 m long concrete gravity dam, a 12 m dia and 888 m long diversion tunnel, four nos. intakes, four nos. of desilting chambers (each of 420 m long x 19 m wide x 24.7 m high), a 11 m dia and 8580 m long head race tunnel, 31 m dia and 137 m deep open to sky surge shaft, 125 m long x 12.4 m wide x 23 m high butterfly valve chamber, 3 nos 5.5 m dia and 190 m/each deep vertical pressure shafts, 909.27 m long and 5.5/3.86 m dia horizontal pressure shafts including six nos. unit penstocks, 240.7 m long x 23.9 m wide x 51 m high machine hall cavern, 216.1 m long x 14.7 m wide x 26.5 m high transformer hall cavern, 314 m long x 18.8 m wide x 58.7 m high downstream surge gallery and 11 m dia and 3000 m long tail race tunnel on right of the Punatsangchhu river. A 1500 m long Highway tunnel was also constructed to reroute the existing NH for execution of the dam right bank work. The general layout plan of project components is shown in Fig 2.

2. REGIONAL GEOLOGICAL FRAMEWORK

Regionally the project area is located within a part of the Tethyan Belt of Bhutan Himalayas, which lies in the north of main central thrust (MCT) comprising of Precambrian coarse grained quartzo-feldspathic, biotite-

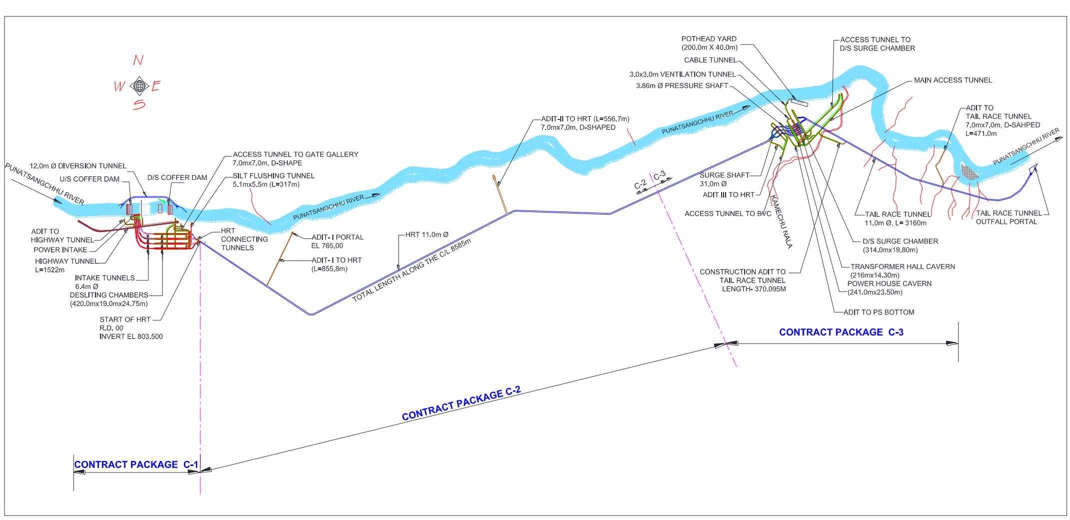


Fig. 2 : Layout plan of the project

muscovite gneiss, with bands of mica schist, quartzite and concordant veins of pegmatite, leucogranite and migmatites with minor bands of limestone/dolomites and metabasics belongs to Sure Formation of Thimphu Group. In addition, colluvial deposits derived from the country rock and river terraces formed on the banks of Punatsangchhu are also seen in the area. On the basis of study of Aerial Photographs three lineaments have been picked up in the area, which trends in (i) N-S (ii) NW-SE and (iii) NE-SW. The Punatsangchhu River probably flows along one of such sympathetic N-S trending lineament in the project area. A few NE-SW and NW-SE trending lineaments appear to be faults as indicated by the shifting of main river course at places.[1]

3. EVALUATION OF THE DAM FOUNDATION

3.1 Geology at Dam Site

The dam site is located between Latitude N 27^0 18.11' 11'' and Longitude E 89^0 57' 13.8''. The Punatsangchhu river in the project area has a WNW-ESE trend and is flanked by high hill ranges on its either bank, forming typical hill and valley topography. It has a distinctly straightened course up to its confluence with Kamechhu nala. Thereafter it exhibits distinct meandering, flowing in NNW-SSE direction. The river in the project area is highly influenced and controlled by the prominent joints sets.

Geologically, the area at the dam site exposes variety of gneissic rocks such as quartzo-feldspathic biotite gneiss, banded gneiss, augen gneiss, and thin seams or bands of biotite schist. These rocks are intruded by leucogranite and pegmatite. The right bank is comparatively steeper and occupied by well exposed rock outcrop (Fig. 3a & b) whereas the left bank is largely occupied by overburden consisting of talus and thick slide debris with some rock outcrop exposed at higher reaches (Fig. 4a & b). On the left bank foliation trend varies from N-S/40^0E to N70^0E-S70^0 W/15^0SE whereas on the right bank it varies from N80^0 W-S80^0E-320/S100 W to N50^0E-S50^0 W/30^0-35^0 S40^0 E. This swing in foliation is due to warping in the rock. However the general trend of the foliation is N60^0-70^0 E to S60^0-70^0 W/30^0 SE[1]. The rocks are moderately to highly jointed, traversed by four to five prominent joint sets and shear zones up to 50 cm thick. In general the foliation is dipping into hill side; indicates an anti-formal structure. The river section is largely occupied by river born material (RBM), which comprises rounded pebbles, cobbles and boulders of gneissic rock, quartzite, leucogranite, etc. along with fine to medium sandy silty material. The RBM is overlain by talus//slide debris on both the banks.

Fig. 3 a & b : Right abutment slopes before excavation (a; Left) and after excavation (b; Right)

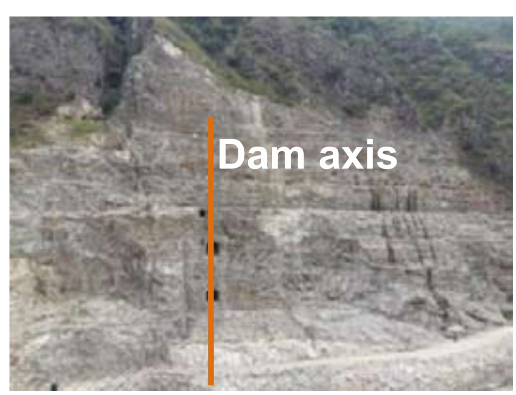


Fig. 4 a & b : Left abutment slopes before excavation (a; Left) and after excavation (b; Right)

3.2 Dam Layout Planning

The regional and local engineering geology have played a major role in the planning, design, construction and preference of the dam in the Punatsangchhu river basin. The geological investigations, which started with the reconnaissance studies and continued during and after DPR, included surface geological studies, detailed geotechnical assessment of the project components and sub-surface explorations. These geological investigations were done for providing the inputs for locating and designing various structures of the project and accordingly location of the different appurtenant structures of the project were finalized and executed. In case of layout planning of dam site the pre-feasibility report of the project was prepared by Nor consult International AS (NC) and dam site was proposed at about 100 m upstream of present axis, near the confluence of a right bank nala at the down slope of Uma village. Subsequently, considering the longer dam length and discrepancy in the river bed level, three alternative sites were selected besides NC site for techno-economic consideration and finally Dam axis alternative-III was found most suitable site for locating dam axis. After carrying out detailed geotechnical investigations[1] the dam axis was kept in N 39^0 E - S 39^0 W. The layout plan of dam site is depicted in Fig. 5.

3.3 Exploratory Drilling & Drifting

At the dam site a total of 23 boreholes with total depth of 1110.0 m were drilled to assess the condition of the dam foundations. In order to decide the stripping limit and for the direct observation of the rock mass conditions of both abutments of the dam, one exploratory drift each on the left and right abutment was excavated during the detailed investigation of the project. The right bank drift is located at EL 792.0 m and the drift on left bank is located at EL 842.0 m near dam axis. In the DPR stage a total of 11 nos. of holes were drilled at the proposed dam location for ascertaining the rock-overburden contact and depth of fresh rock for deciding the foundation level of various blocks. On the basis of DPR stage drill core examination the deepest foundation was proposed at EL 760.0 m. Subsequently, during construction stage, keeping in view the change in anticipated rock slope profile, 12 nos. of drill holes were drilled at EL 825 m to reconfirm the rock overburden contact and foundation grade rock at the dam block nos. 1, 2, 3, 4 & 5. As the fresh rock was encountered at higher level as compared to DPR stage investigations, the of block nos. 4, 5, 6, and 7 have been founded at EL 765.0 m, instead of at EL 760.0 m. The plan of exploratory drills holes and geological cross section along dam axis (DPR stage) are depicted in Fig. 6 and 7 respectively. The drill holes in the river bed area were proved to have a maximum depth of fluvial fill material/overburden of 52.50 m (DH-9: located in block no. 5, 35 m d/s of dam axis). The maximum depth of the quaternary alluvium/overburden in the river valley under the dam axis is 45.60 m (DH-14: located in block no. 3, at dam axis).

3.4 Geological Assessment

The exploration, at the dam site, revealed the presence of approximately 15 to 30 m overburden with the maximum thickness towards the right bank underlain by fresh and competent bed rock comprises Quartzo-Feldspathic-biotite gneiss, biotite gneiss, leucogranite and veins of pegmatite. The bed rock was traversed by wide to moderately spaced joints and fractured at places. A shear/weak zone running from the upstream to downstream across the dam axis had been anticipated from drill core logs at the left bank. On the basis of information obtained

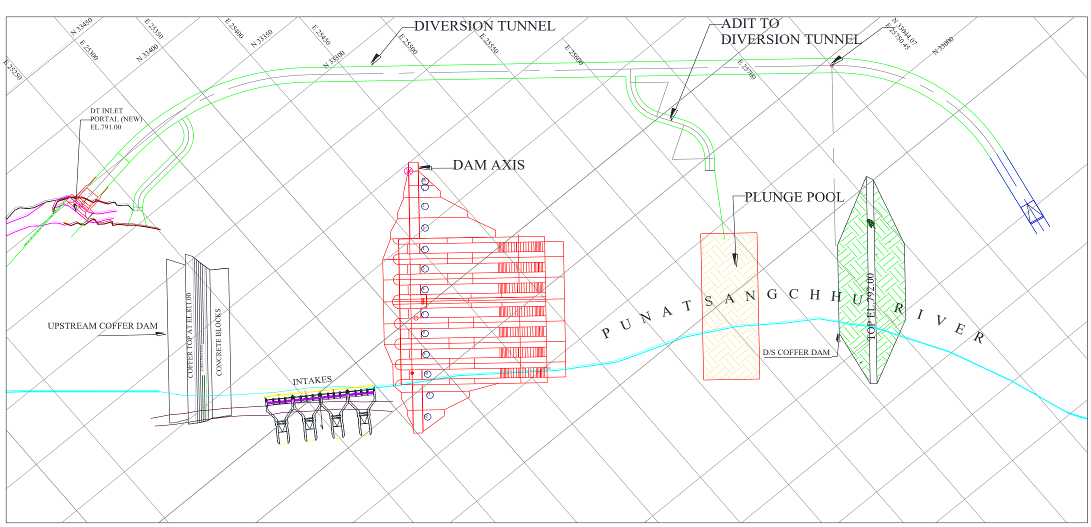


Fig. 5 : General layout plan of the dam site

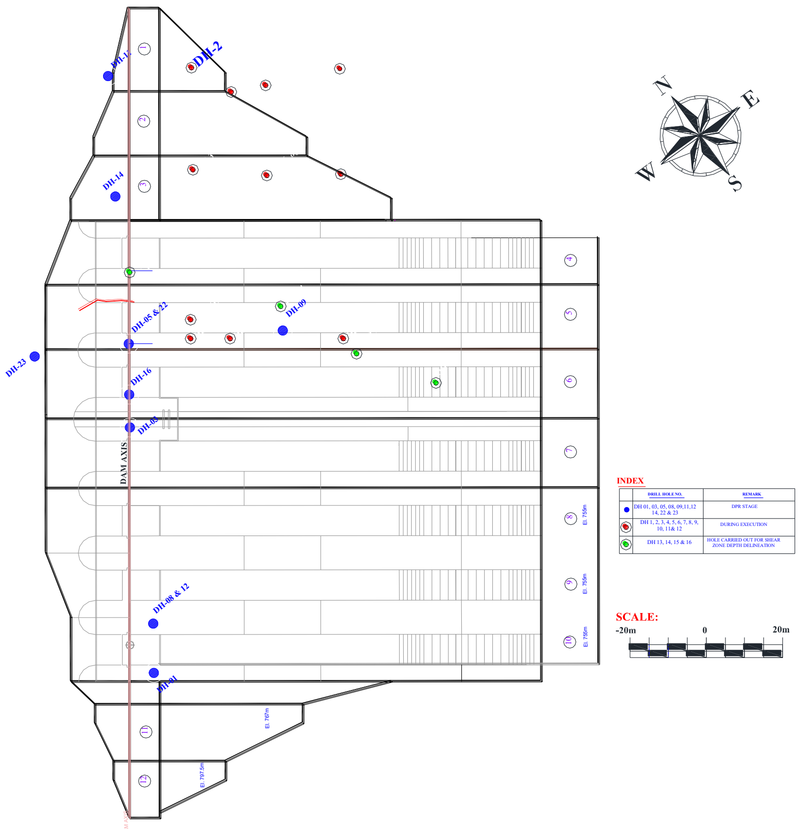


Fig. 6 : Plan of the exploratory holes in dam area

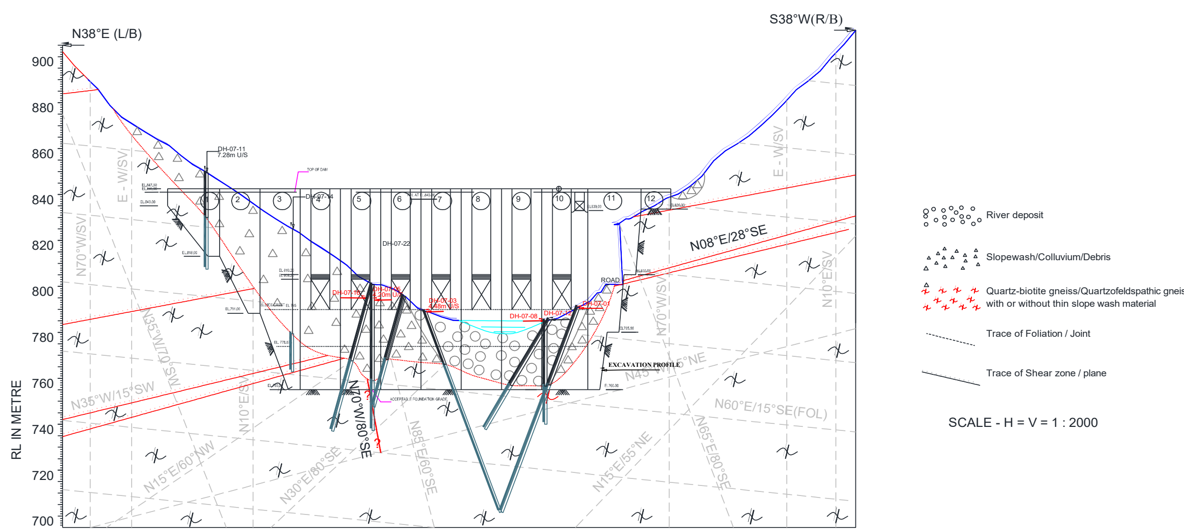


Fig. 7 : Geological cross section along dam axis (DPR Stage)

from core logs and surface features, geological section were prepared and main dam pit foundation (dam block nos. 8, 9, 10) lowered up EL 755.0 m (deepest foundation level).

3.5 Dam Foundation Excavation

The excavations for the foundation were started from top of the dam on right abutment. The excavation on the left abutment was started from EL 960.0 m, whereas, on the left bank it started from EL 860.0 m. The stripping had been mainly guided by site geological conditions and design requirement. On the right bank, at the abutment location, National Highway was rerouted as part of it was falling in the submergence and 1.5 long first highway tunnel was constructed[2].

During excavation and support of the cut slopes (on the left abutment), it was observed that isolated overburden patches comprising of slope-wash materials along with sand lenses and RBM in patches were also present intermittently sand witched within the negative profiles/ natural rock undercuts. These slope-wash materials were selectively removed. In view of the presence of these adverse features exercising control on the left bank excavation, steepness and height of the slope and presence of stress relived joints provision of 30 m deep cable anchors had been made in the design and provided at EL 848 m, 846.0 m, 825 m & 823 m, and 796 m. In the river section, 12-30 m thick overburden comprising river deposit, in general, was removed to expose the fresh and sound bed rock. Three empirical rock mass classification systems, namely the rock mass rating and Q (Barton, ZT Bieniawski, (2008)[3], Geological Strength Index (GSI) (V. Marinos, P. Marinos and E. Hoek 2005)[4], and Q-system (NGI 2012)[5] methods have been used to summarize the geological and geotechnical data, and to provide tools for the designer during construction.

General level of the exposed bedrock in the overflow dam section varied from EL 765.0 m in dam block no. 4 to EL 755.0 m in main dam pit (block nos. 8, 9, 10). The rocks exposed at the foundation grade comprised predominantly quartzo-feldspathic-biotite gneiss, biotite gneiss, and leucogranite (Fig. 8). These were intruded by veins of leucogranite, quartz and pegmatite. The foundation rocks were traversed by a number of shear seams and most of them were of short continuity and restricted to a single dam block. However, one mega shear was encountered during progressive excavation of left bank of dam shear encountered askew to the river dipping N70^0 to 90^0 at an inclination of 55^0-60^0 in block nos. 5, 6 and 7, initially with affected width of 3.5 m to 5.6 m with clay gauge >20.0 cm and crushed and highly fractured rock mass.

The rock mass qualities (based on field check only) of gneisses of the dam site (on the left bank) were assessed using RMR and results show, gneisses unit on the left bank at dam block no. 1 in general are classified as fair to good rock mass (RMR = 55 - 75). According to the Geological Strength Index the gneisses are classified as very blocky to blocky (GSI = 50 –70) and at few locations blocky to disturbed/ fractured (GSI = 30–45). The results show, in general rock mass on the both the right and left bank are classified as fair rock mass to good (RMR = 50-70)[6].

4. MEGA SHEAR ZONE

4.1 Background

During the construction stage exploratory drilling, on the left abutment, weak rock mass zone with sandy horizon encountered in the DH-10 and 12. From the drill core logging it had been interpreted that sand and crushed rock pocket exists from EL 763.4 m to El 759.40 m, however that time direction and inclination could not be confirmed. Encountering of weak feature during dam excavation was noticed during excavation/stripping of the left bank, as actual rock profile at EL 825.0 m was found shifted by 14.5 m towards hill side from the DPR stage investigations.

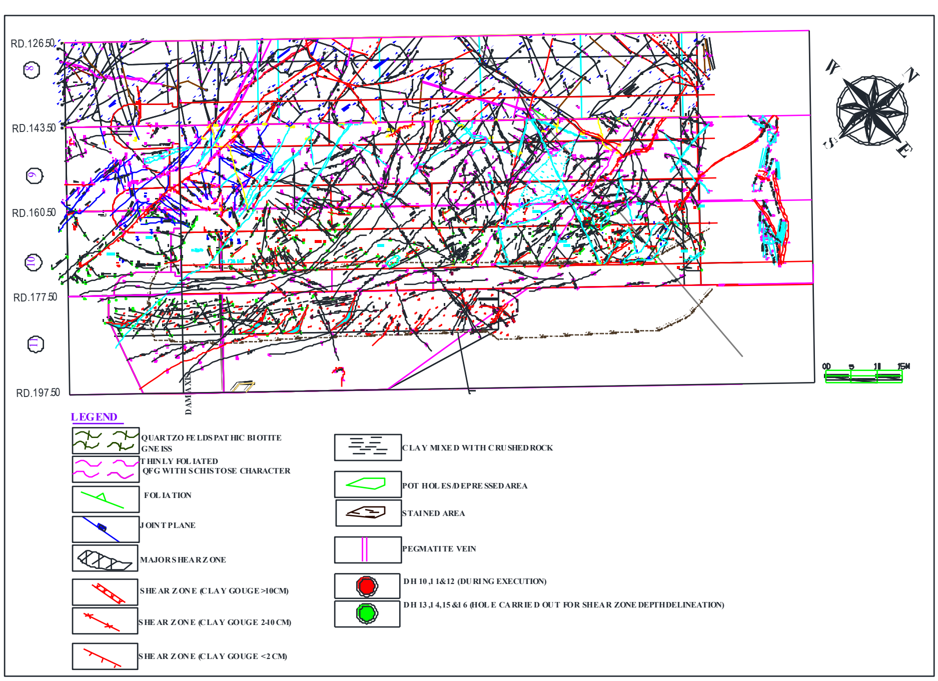


Fig. 8 : Geological plan of Dam Foundation (block nos. 8, 9, 10 and 11)

In view of this discrepancy 12 nos. of holes were drilled to ascertain the sound foundation grade rock. Based on drill holes the block nos. 1 & 2 which were earlier proposed to be rested on anticipated bench at EL 725.0 & 798.0 were shifted at EL 790.0 m & 785.0 m respectively. Besides, 4.0 m thick sand pocket was found resting over highly fractured and crushed rock mass near proposed joint of block nos. 5 & 6 which later on probed during drilling of hole nos. 10 & 12 at RD 87.0 m & CH 16.0 m & 27.0 m respectively below proposed block joint of 5 & 6. Considerable water loss was observed during drilling and sand was first appeared at EL 763.0 m during drilling hole no. 10 and continued up to EL 758.5 m. The rock strata encountered immediately below sand deposit was highly fractured and crushed up to the EL 749.90 m. The same sand deposit was further reconfirmed through hole no. 12 drilled at RD 87.0 m & CH 24.0 m downstream where it is appeared from EL 760.27 m to EL 758.77 m, resting over highly fractured and crushed rock mass up to EL 755.77 m. In view of uneven bed rock profile four drill holes were carried out to decipher the sound foundation grade rock below the EL 822.0 m and on the basis of interpretation of borehole core logs dam block no. 1 was lowered up to EL 795.0 m (instead of EL 825.0 m) and block no. 2 rested at EL 785.0 m (instead of 798.0 m).

Subsequently, during concurrent excavation on left bank of dam shear zone (55^0-60^0 / N070^0 to 090^0, affected zone varies from 3.5 m to 5.6 m, Clay gauge >20 cm, with crushed/fractured rock mass) encountered askew to the river flowing direction passing from u/s to d/s (heal to toe of the dam). In order to confirm the depth of shear four drill holes were carried out and shear zone encountered in all the drill holes at different depth with variable thickness.

4.2 Investigations, Delineation and Interpretation

The investigations of shear zone took place in four stages. The preliminary stage began with geotechnical observations/assessment of the foundation grade rock mass and identifying the weak zones with the progressive excavation for achieving the foundation grade rock for different blocks at different elevations as per design drawing. In the second stage detailed geological mapping of was carried out to delineated the shear zone disposition in the different dam blocks from where it was passing through. After geological mapping, in the fourth stage sub-surface investigation was done and four boreholes with total depth of 187.0 m were drilled on the dam site (dam foundation blocks 4, 5, 6 and block joint of 5 & 6). Geotechnical cross sections were constructed based on the exploratory boreholes. Borehole cores were critically examined/interpreted to decipher the depth of shear zone at different dam blocks and rock mass characteristics of shear zone material. Subsequently, shear zone trench excavations was geologically assessed frequently as per design need and after the shear trench was made geological cross sections were prepared at certain locations and shear zone flour was geologically mapped.

With the progressive excavation, when the excavation and stripping work on the right abutment for dam block nos. 8, 9 & 10, had been completed (EL ~755.00 m) and on left abutment excavation had reached up to EL ~795.00 m in block no. 1 (dam top EL ~846.00 m) and in block no. 6 & part of block nos. 5 & 7 excavation (EL ~765.50 m) was in progress one major shear/weak zone was encountered. Shear zone with varying thickness of clay gouge material and affected zone (approximately 3.5 m to 6.5 m) was encountered in the main dam pit area in the block no. 8 (at EL ~755.0 m: near toe of dam body) and in the inclined foundation of the dam block no. 7 (below EL 765.0 m) and below dam block nos. 5 & 6. The geological mapping, on 1:100 scale, of different dam blocks was carried out, with progressive excavation, after achieving the foundation grade rock and shear zone was delineated and mapped. Subsequently, exploratory drilling was also carried to ascertain the thickness, depth and behaviour of the shear zone. Drill cores were critically examined and interpretations were made. Once the shear zone extension and depth of the shear zone was established based on the attitude of the shear zone, at different locations/elevations projections were made for design and treatment purposes. However, with the frequent change in attitude of shear zone, from time to time, cross section were developed @ 3.0 m interval to finalize the excavation (for removal of shear material) depth in the shear zone trench. Finally, a 125.0 m long, 5.0-35.0 m wide and 13.0-15.0 m deep shear trench was made in order to treat the shear. The field observation suggests that this shear is a discrete fracture between blocks of rock containing several parallel or anatomizing (i.e. branching and reconnecting) shears particularly at the toe of dam body near dam block nos. 7 & 8 and may have formed in brittle ductile regime.

4.2.1 *Exploratory Drilling*

After the geological mapping four nos. of exploratory holes (DH-13 to DH-16) were drilled (Fig. 6) at different locations to confirm actual depth/thickness and sub-surface behavior of shear zone. The disposition/depth of the shear/weak zone in the dam block nos. 5, 6 and 7 was delineated on the basis of drilling. Drill core log abstracts were prepared for all the four holes and (Fig. 9) and projected sections were made (Fig. 10).

The encountered rock mass, in the drill holes, comprises predominantly quartzo-feldspathic-biotite-gneiss (QFBG), biotite gneiss, leucogranite with bands of biotite schist of variable thickness. At places the rock mass is intruded by thin veins/patches of pegmatite. Shear material mainly comprises highly pulverized rock mass, granular rock flour, fractured rock mass, clay gouge with slush, and broken rock fragments. The drill core logging revealed that the shear zone depth varies from 3.1 m to 11.22 m and affected/fractured rock mass zone below the shear zone varies from 6.0 m to 20.0 m in depth.

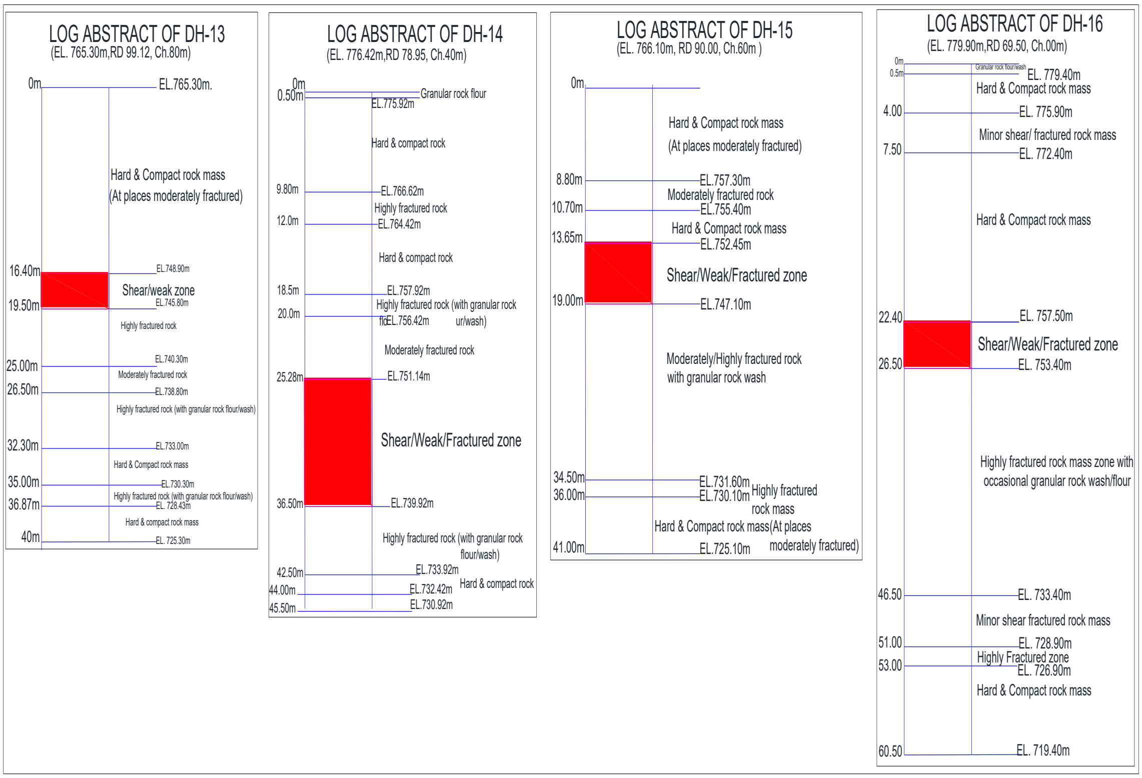


Fig. 9 : Drill core log abstracts for DH-13, 14, 15 and 16.

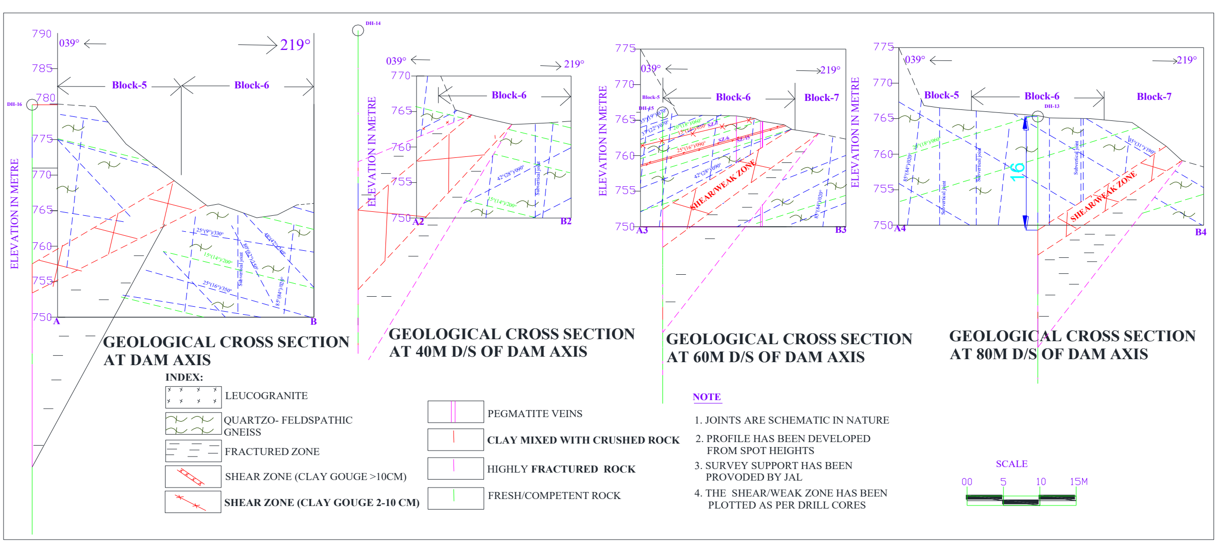


Fig. 10 : Geological cross-sections depicting shear zone projection at dam axis, at CH 40.0 m, 60.0 m and 80.0 m d/s of dam axis

4.2.2 *Water Permeability Test*

The permeability test was carried out in DH-13 with an aim to assess the groutability of shear and associated rock mass. The test was conducted at every 3 m interval in full hole depth of 33.0 m The WPT results reveal that in the shear zone reach the water intake is less as once the clay comes in contact with water, get swell and thereby decreasing water intake as test proceeds. In the fractured rock zone more water intake is observed. This indicates that the fractured zone is more groutable than the sheared mass .The test results are summarized in Table 1.

Table 1 : Results of WPT conducted at DH-13

Sl. No.	Reach below Collar Elevation of EL 765.25 m	Rock Type	Permeability in Lugeon (litre/metre/ minute)
1.	0 - 12 m	Fresh Rock	3.46 to 62.04
2.	12 - 15 m	Fractured rock	13.36 to 26.86
3.	15 - 21 m	Shear zone with clay gouge & sludge	2.09 to 4.95
4.	21 - 30 m	Fractured rock	2.34 to 24.03
5.	30 - 33 m	Fresh and slightly fractured rock	8.50 to 23.08

4.3 Characteristics of the Mega Shear Zone

Detailed geological mapping, on 1: 100 scale, carried out to delineate actual thickness, orientation, and behaviour of the mega shear zone. The shear zone is traversing from dam heal (block no. 5), dam center (block no.6) to dam toe (block no. 7) having curvilinear nature, however, it is encroaching in block no. 4 by ±1.0 m near RD 73.0 m in a limited area; between CH ±18.69 m and CH ±20.0 m at EL 765.0 m and in block no. 8 d/s portion it is intersecting between CH 97.0 m and CH.102 m at El 755.0 m. The general trend of shear is 35^0-42^0/N 070 to 080^0; however, there is variation in the shear's attitude due to warping of foliation and its curvilinear nature. In the block no. 5 the thickness of shear zone (including fracture zones and affected zones) at the dam axis is 13.97m whereas, in the u/s part of the dam (near CH ±20.0 m) it is 14.19 m thick. Sheared material comprises moist clay gouge (0.5 m to 1.0 m thick), rock flour, crushed/fractured rock, fragmented rock pieces and intermittently hard patches of parent rock (no strain zones). The affected zone varies from 4.0 m to 30.0 m.

The foundation surface in the mapped area is undulatory due to intersection of different joint sets and formation of wedges. The geological mapping revealed that the mega shear zone is bounded by two shear zones (SZ-1, forming left side boundary, having clay gouge thickness 5 cm -20 cm and SZ-2, forming right side boundary, having clay gouge thickness 8 cm - 100 cm) passing through block nos. 5, 6 and 7 from upstream to downstream direction, dipping into left abutment side, and having variable thickness. Infilling material of this weak zone comprises crushed rock mixed with minor clay and at places small lenses/chunk of QFG. The width of this zone varies from ±7m, at u/s side near dam axis, to ±2 m at d/s side. The attitude of its two boundaries i.e. SZ-1 (20^0-55^0/N070^0-100^0) and SZ-2 (20^0-55^0/N0500-1100 also vary from u/s direction to d/s direction at different places. Besides, major shear there are 13 nos. shear seams were recorded during foundation grade mapping.

A detail geological cross sections and geological plan (Fig. 10 & 11) were prepared based on encounter geology and exploratory drill holes and it revealed that the shear zone was expected to be passed below the block no. 5 and the other shear seams SZ-3 and SZ-5 will likely to be encountered in the foundation of block no. 7 and block no. 4 respectively. A total of 20 structural discontinuities, 15 shear seams and 5 joint sets have been recorded during the mapping. Except mega shear zone (Fig. 12a & 12b), in general in the mapped area most of the shear seams are less than 100 cm thick and have rock flour and/or gouge up to 10-15 cm thick as infillings.

4.4 Field Observations and Geotechnical Assessment

Shear zone is non-homogeneous in nature and forms curviplanar zone composed of rocks that area more highly strained/deformed than the rock adjacent to the shear zone. This is a discrete brittle-ductile deformation zone (?) zone containing observable mylonitic fabrics, largely obliterates older fabrics and shear zone is characterized by a prominent stretching lineations. Prima facie, the field observation and the stretching lineations (expressed by elongate clasts, quartz rodding, and biotite streaks) indicate that the northeastern block of the shear zone is up thrown relative to the southwestern block. The magnitude of stretching increased from the northwest margin toward the southeast of the zone. The shear zone transects foliated granitic gneisses. The shear zone, which sharply truncates and is distinctly younger than the regional foliation strikes S 10°-20° E. Within the shear zone, the rocks are penetratively deformed, but domains of relatively non-deformed (no strain zone) rocks as much as tens of meters across remain. Narrow zones of no strain are present within the main shear zone and deformation was inhomogeneous within the shear zone. Shear zone typically exhibit substantial heterogeneity, consisting of randomly occurring units of more or less undeformed, unaltered stiff rock fragments which are surrounded by a mainly soft, weak matrix. The matrix typically consists of highly sheared fine-grained clayey, silty gouge and intensely fractured rocks. The matrix often appears to flow around the blocks in an anastomosing pattern. The ratio of weak gouge matrix to rock blocks of different sizes, shapes and strengths is extremely variable. The distinction between fragments and matrix is basically a matter of the scale of observation and engineering interest. The field studies and delineation of shear zone suggests that it was concealed under considerable thicknesses of overburden, and it is inferred indirectly from field evidence.

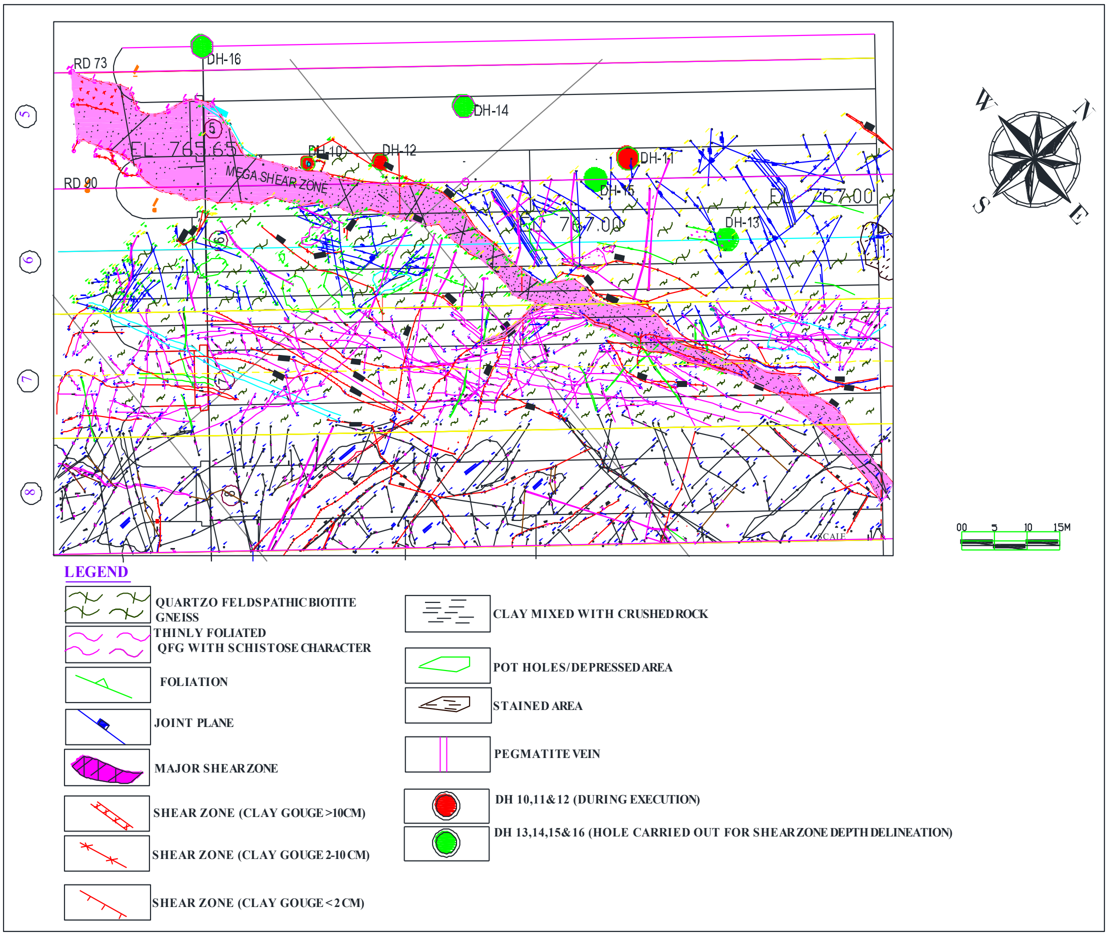


Fig. 11 : Geological plan of shear zone (EL 565.0 m) and adjacent blocks

Fig.12a : View of shear zone before final excavation and treatment

Fig. 12b : Close-up view of shear zone before final excavation and treatment

In order to assess foundation grade rock mass quality the strata adjacent to the shear was also examined to calculate rock mass rating (RMR), Geological Strength Index (GSI) and USC by Schmidt Hammer. The rock mass qualities and classifications of foundation rock at dam block no. 5 is assigned using the RMR and the GSI classification systems. As far as field observations are concerned rock mass either side of shear zone in the block no. 5 is fresh, hard and compact. However, to assess the competency of foundation grade rock a total of 8 nos. of core from four different locations (C1, C2, C3 & C4) 2 samples from each location for dry and wet test were taken for laboratory. In general rock is fresh (W0), compact and hard (as per field estimates strong to very strong; R3 to R5 Grade), dry and of fair to good category. The UCS value (as measured in the field by blow of geological hammer) of rock would be between 80 Mpa to 175 Mpa. The rock mass qualities (based on field check only) of gneisses at dam block no. 5 were assessed using RMR and results show, over all gneisses are classified as fair to good rock mass (RMR = 50 - 80). According to the Geological Strength Index (GSI) the gneisses are classified as blocky to massive (GSI = 55 – 86). Geotechnical assessment of foundation rock at all the four locations from where core samples have taken is summarized in Table 2.

Table 2 : Details of core samples locations and rock mass characteristics

Core sample location	**Rock Characteristics/ Category of rock mass**	**Hardness** **RMR** **GSI**	**Photograph of rock at or near the cores samples location**
C_1 **(Hill side of shear near dam axis)** RD: 70.76m Ch: 00.38m EL: 765.71m (Block no. 5)	Fresh & very hard, massive to blocky, whitish grayish medium to coarse grained, biotite bearing quartzo-feldspathic gneiss with more leucogranite part, well defined compositional banding, 3 sets of joints, joint opening 1-3cm. Sp 5-20cm. Staining along few joint planes. Dry rock mass. Predominantly Good category rock.	Field Estimates rock strength (Grade: R_5) UCS would be (150 to 175Mpa). GSI: 60-64 RMR: 70-75	
C_2 **(Hill side of shear near dam axis)** RD: 73.65m Ch: 10.40m EL: 765.42m (Block no. 5)	Fresh & very hard, biotite bearing quartzo-feldspathic gneiss, medium to coarse grained, gray to whitish grayish in colour, well defined compositional banding, blocky rock mass. 3 sets of joints, joint opening 1-5cm. Sp 2-40cm. Dry rock mass. Mostly Good category rock.	Field Estimates rock strength (Grade: R_5) UCS would be (120 to 175Mpa). GSI: 65-69 RMR: 68-70	
C_3 **(Valley side of shear near block joint of 5 & 6 and u/s Ch ± 18.0)** RD: 90.29m Ch: 18.00m EL: 765.28m (Block no. 5)	Fresh & strong. Predominately biotite rich quartzo-feldspathic gneiss, coarse grained, dark gray in colour, warped foliation, dry rock and moderately blocky rock mass, prominent gneissosity, and schistosity is equally well developed. Joint 3+1 joint opening 1-5mm. Sp 2-12cm. Mostly Dry rock mass. Fair category rock.	Field Estimates rock strength (Grade: R_3 to R_4) UCS would be (80 to 100Mpa). GSI: 55-59 RMR: 50-58	
C_4 **(Valley side of shear u/s Ch ± 12.0)** RD: 93.31m Ch: 07.77m EL: 765.40m (Block no. 5)	Coarse grained Biotite rich quartzo-feldspathic gneiss with numerous thin leucogranitic veins, dark gray in colour, crudely foliated, mostly massive rock. Dry rock mass. Good category rock.	Field Estimates rock strength (Grade: R_4 to R_5) UCS would be (100 to 120Mpa). GSI: 80-86 RMR: 75-80	

4.5 Shear Trench Floor

Detailed geological mapping/discontinuity surveying (taking cross section frequently) was performed to provide the basic parameters for classification of the rock masses and inputs to designers. The shear trench (approximately L-126.0 m; width 5.0-35.0 m, depth 13-15.0; below EL 765.0 m) was excavated to treat the shear zone in the block nos. 4, 5, 6 and 7. In the initial stage sitting bench was provided with the progressive excavation of shear trench, since there had been frequent variation in the attitude of shear zone (Fig. 13), hence, ultimately provision of sitting bench and cut off key at the u/s portion of shear zone in the design was excluded and on the basis of geological cross section it was decided to remove sheared/loose/fractured material up to EL 752.0 m i.e. 13.0 m depth. Shear trench floor was geologically mapped (Fig. 14) and supplied to designers for final treatment.

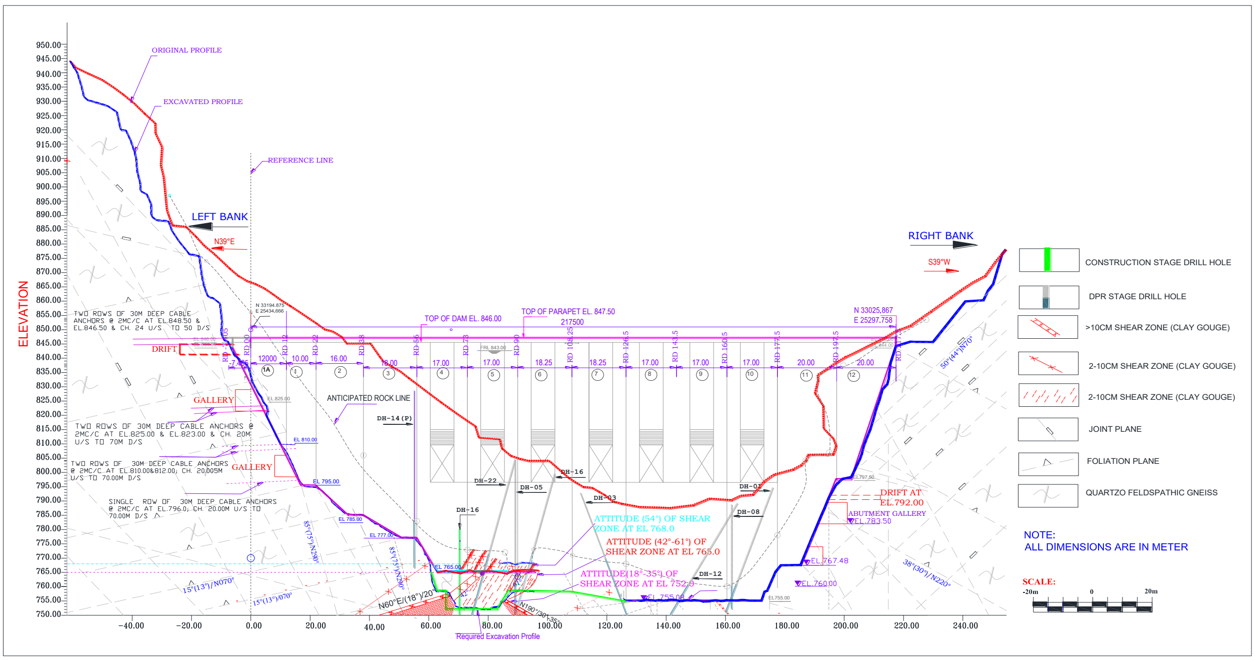


Fig. 13 : Geological section at dam axis showing orientation of shear zone

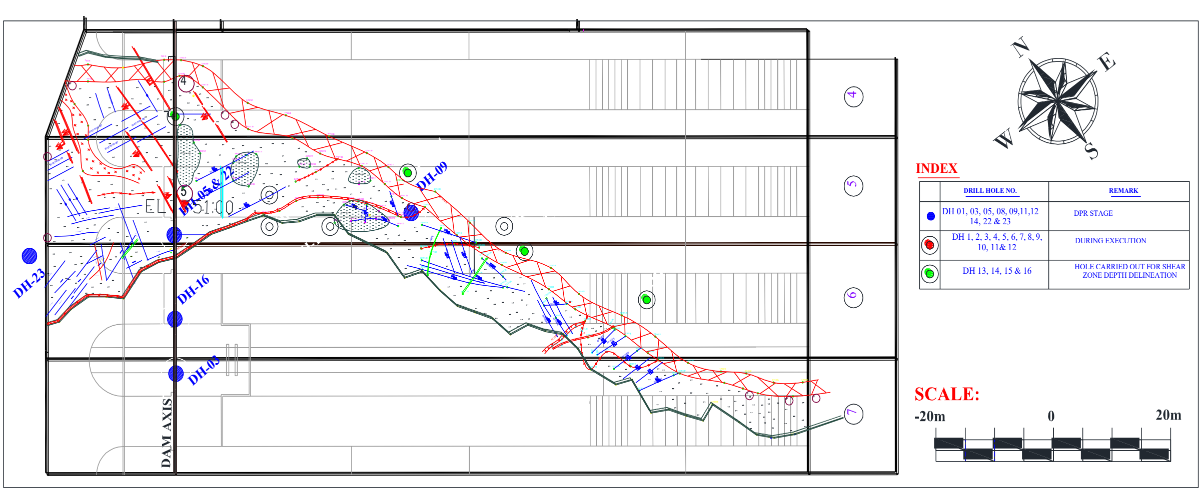


Fig.14 : Geological map of the shear trench floor

4.6 Treatment of Shear Zone

On the basis of detail geological observations, treatment of shear zone was provided and is being executed in following steps. An account of treatment is summarized in Table 3.

(i) The excavation was carried to remove sheared and affected zone selectively by making of a shear trench (L-126 m length, Width- varies from 5.0 to 35 m, depth-13.0-15.0 m depth, Fig. 15).

(ii) Consolidation grouting at 3.0 m c/c spacing was done through 10.0 m deep holes all along and across shear zone.

(iii) 32 mm dia and 6.5 m long and fully grouted rock anchors @ 1.5 m c/c staggered were provided on the walls of shear trench (Fig. 15).

(iv) The shear zone trench was filled with M20 and M25 grade with three layers of 32 mm and 20 mm dia steel reinforcement at bottom and top at EL 758.50 and EL 765.0 m respectively (Fig. 16). The reinforcement was covered with M25 grade concrete.

(v) Instrument such as Multipoint Borehole Extensometer (MPBX), pore pressure meter etc are being provided to check the behaviour of treatment and foundation.

Table 3 : Account of treatment in shear zone

Sl. No.	Particular's	Quantity
1.	Excavation	53000 M^3
2.	Cement grout	161.75 TON
3.	32 mm dia rock anchors	12209 RM
4.	Concrete	56000 M^3
5.	36 and 20 mm dia Steel reinforcement	2250 MT

Fig. 15 : Shear zone trench excavation and rock anchor support on trench walls

Fig. 16 : Treatment of shear zone at bottom (Left) and treatment at top (Right)

CONCLUSIONS

In the development of any hydroelectric project, particularly located in tectonically active Himalayan region, the geological prediction, interpretation and treatment of major shear zone play a decisive role in dam construction for ensuring its stability. Dam stability must be the primary concern. During excavation of dam foundations, the encounter of shear zone or any other adverse geological feature cannot be ruled out, even if the detailed geological exploration were carried out during investigation and pre-construction stages. The present study describes an engineering-geological assessment of the dam foundation grade rock mass and focuses on the delineation and treatment of a mega shear zone that affected the stability of dam. Surface geological studies and subsurface explorations were carried out to evaluate the characteristics of the foundation rocks and geotechnical behaviour of the shear zone. Exploratory drilling and interpretation of drill cores provided vital information regarding geotechnical behaviour and characteristics of shear zone/material which helped the designers in designing the shear zone trench dimensions and treatment sequences which includes consolidation grouting, provision of anchors on the shear trench walls, concrete plugging and RCC raft to arrest dilations and instabilities and thus improving/strengthening of the dam foundation for overall stability of the dam. Prediction of shear zone nature and orientation ahead of the progressive shear trench excavation has been a challenging task for geologists. As behaviour of any adverse geological feature is very uncertain; therefore, its accurate assessment is very crucial for time and cost implications involved in its treatment. The gradual revealing of this mega shear zone and its tackling has contributed towards time and cost overrun of the Project. Precision in geotechnical assessment, validation of projections/predictions and comparison of results i.e. interpretation of exploratory drill core data versus actual behaviour of shear zone, is the key to its successful tackling . Accurate assessment of these parameters has greatly helped in anticipating and understanding the problems and accordingly evolving the cost effective corrective measures timely. The most important outcome of this study has been towards establishing the weak feature in the overflow section of dam foundation which was suspected earlier, based on the poor to very poor core recovery of highly fractured rock mass along with clay gouge from the drill cores during sub-surface investigations.

Acknowledgments

The authors express their sincere thanks to the management of PHPA for giving permission to publish this paper. We are grateful to Central Water Commission for providing design drawings. The views expressed in this paper are of the authors and need not reflect the views of the organizations mentioned.

REFERENCES

1. Detailed Project Report of Punatsangchhu - II Hydroelectric Project, August, 2009
2. Santosh K. Sati, R.N. Khazanchi and K.K. Sati intrepid approaches in the successful construction of first highway tunnel in Bhutan” INDOROCK 2013: 4th Indian Rock Conference, 2013,
3. N. Barton, ZT Bieniawski, (2008). RMR and Q-Setting Records, Tunnels & Tunnelling, 2008.
4. The geological strength index: applications and limitations. Bull Eng Geol Environ, 2005, 64: 55–65 DOI 10.1007/s 10064-004-0270-5. V. Marinos, P. Marinos and E. Hoek.
5. The Q-system handbook in 2012 as a summary of NGI’s Best Practice.
6. Unpublished geotechnical notes by the Resident / Sr. Geologists on Punatsangchhu dam excavation, 2014 to 2016.

Strengthening of Overflow Section of Koyna Dam – A Case Study

Rajendra V. Panse

Secretary (Project Coordination), Water Resources Department, Government of Maharashtra, India

Ravindra N. Thakare

Chief Engineer, Hydro Projects & Quality Control, Pune, India

ABSTRACT

After the event of December 1967 earthquake at Koyna with, the epicentre near the dam, some of the non-overflow blocks showed considerable distress in the form of major cracks at the level of a kink both in the downstream and upstream faces. It was necessary to undertake immediate repairs to the dam. These were accomplished through immediate use of prestressing cables to stitch across the cracks developed on NOF faces.

Subsequently strengthening of NOF by concrete backing and buttressing was carried out in 1972 as a permanent measure. After a 6.3 magnitude earthquake in 1993 in the southern part of the state at Killari, the Government of Maharashtra set up a committee for deciding modifications to be carried out in design standards and ascertaining the safety of major and medium completed dams in earthquake prone regions of Maharashtra State. Accordingly, design standards were set and stability for 27 dams was received and it was decided to strengthen 11 dams, which included Koyna Dam spillway. Having an astounding generation of 3300 million units of power every year and catering to the irrigation need of about 50,000 Ha of land, Koyna catchment has a very high precipitation with a reservoir filling success rate of more than 80% over the last 40 years. However, known to be a silent seismic area of more than 200 years, the spurt of earthquakes started from 1963 and so far 1,11,098 shocks of various magnitudes upto a magnitude of 6.5 on Richter scale have been recorded. Therefore, in view of the importance of this project, the energy and irrigation needs of the State and in view of the continued seismic disturbances in the region, protecting this dam against any severe earthquake has become mandatory. At the same time storage augmentation to the tune of 184 Mm^3 was accomplished by providing 1.52 m high movable flaps over the existing radial gates. All these factors earnestly warranted the strengthening of Koyna spillway. It was decided to strengthen Koyna Dam spillway for revised seismic parameters and raised FRL.

The task involved designing the section through a series of analyses including dynamic analysis, and judging the efficacy of hydraulic profile through competent model experiments. It also required meticulous planning of construction activities as the entire strengthening work was to be completed within a span of 2 years, the net available time duration being 400 days.

This paper takes a brief overview of the impact of 1967 earthquake, strengthening measures adopted for the NOF portion thereafter and the necessity of strengthening of spillway portion. It also discusses the design of strengthening section for dam spillway and the construction planning, which is based on the thermal studies and blasting experiments conducted by CW&PRS and physical model studies conducted by Maharashtra Engineering Research Institute. The Construction Methodologies adopted and problems faced during construction and remedial measures adopted are also elaborately discussed in this paper.

1. PREAMBLE

Koyna Hydro-Electric Project is known as life line of Maharashtra. It is a major hydroelectric project in India and is the backbone of energy supply of the National grid with a total installed capacity of 1960 MW. This includes generation of (4 x 70 MW = 280 MW) in First Stage, (4 x 80 MW = 320 MW) in Second Stage, (4 x 80 MW = 320 MW) in Third Stage, (4 x 250 MW = 1000 MW) in Fourth Stage and (2 x 20 MW = 40 MW) in dam-foot powerhouse. Work of (2 x 40 MW= 80 MW) from Dam Foot Pumped Storage Scheme at left bank of the dam is in progress and additional generation of 400 MW is planned in near future (2 x 200 MW = 400 MW) from Stage-V and (2 x 200 MW = 400 MW) from Stage-VI. Stage-V and VI are also planned as Pumped Storage Schemes.

In the Indian Sub-Continent Koyna Project has several "firsts" to its credit, in which under water lake-piercing technique adopted with enormous success for its Stage-IV and IV(B) construction, is worth mentioning.

After independence, the increasing power demand from the industrial sector gave boost to many projects in India. Koyna was one of those selected projects which even today is the largest hydroelectric project in the State that accounts for about two thirds of the State's hydel installed capacity. Though initiation of planning of this project dates back to 1910 during British regime, the detailed investigation work was completed during 1947 to 1950. Several construction techniques were thought of, tried and tested for their efficacy and the best suitable for Indian conditions was finally adopted. Koyna Dam is a principal component of this mega-project. It is a 103 m high rubble concrete dam constructed in two stages across the river Koyna, a tributary of river Krishna that is a major river in Indian peninsula. It was for the first time in India that such a monumental structure was built in rubble concrete instead of conventional concrete or uncoarsed rubble (U.C.R.) masonry. With optimum cement content of the order of 177 kg/m^3, densities as high as 2.65 t/m^3 and 90 days' strengths to the tune of 2100 t/m^2 were achieved through scrupulous quality control. The dam construction was started in 1957 and impoundment was started from 1961. The earthquake parameters for evolution of design were decided on the basis of general trend and the seismological data then available. The total length of the dam is 807.72 m. It comprises of 53 monoliths in total. Out of these, six shallow monoliths are constructed in UCR masonry and the rest 47 in rubble concrete. The central spillway comprises of seven monoliths from monolith 18 to 24. The length of the spillway is 89.0 m. divided into six equal spans. Radial gates of size 12.41 m x 7.46 m are provided. These gates are attached with 1.5 m high flaps to raise the Full Reservoir Level (FRL) by 1.5 m. The Energy Dessipation Arrangement (EDA) is of sloping apron type with appropriate appurtenant structures. The original storage capacity of the reservoir is 2797 Mm^3 with 2503 Mm^3 of live storage which is now increased by 184 Mm^3 due to raising of F.R.L. by 1.52 m.

2. SEISMIC ACTIVITY IN THE AREA

The Koyna dam is founded on the trap formation of volcanic basalt of Deccan plateau of the peninsular India. The trap formations are almost horizontal with irregular interfaces. Deccan plateau was not known for any major historic seismic event in the past. Thus there is no record of major seismic activity in the region. Seismological stations in this region were established during the construction stage of the dam. In early sixties small tremors were experienced and recorded. By that time the construction of the dam had progressed considerably and partial impoundment up to Koyna Reduced Level (KRL) 632.6 m was created. Since 1962 quite a few small shocks were recorded in the reservoir area. The frequency of this seismic activity increased considerably from 1963 onwards.

2.1 Koyna Earthquake of 1967

It was not even dreamed in 1961-62 that the Koyna dam area would soon turn out to be seismically active on a large scale. But on 11th December 1967, a severe earthquake shock having a magnitude of 6.5 on Richter's scale was felt in this area releasing enormous amount of energy. The epicenter of this shock was just about 3 km away on the downstream of the Koyna dam. The occurrence being so close to the dam, it resulted in noticeable distress to the body of the dam in the form of horizontal cracks at quite a few locations. Horizontal cracks on the u/s and d/s faces of some deep non-overflow (NOF) monoliths were noticed. In addition, the appurtenant structures like components of the spillway bridge and the elevator tower showed distress in the form of spalling and cracking of concrete etc. However, the spillway portion of the dam remained fairly intact. Most of the damage occurred in NOF section and the place where an abrupt and severe change of slope was provided on the d/s face at KRL 627.9 m. These abrupt changes in the slope were made while accommodating the Stage II construction of the project over Stage I construction already completed. This also increased inertia mass at top due to wider top widths resulting in high stress concentrations at these locations under severe dynamic conditions.

This was the first time in the history that an epicentral earthquake having such a high magnitude was experienced by a large gravity dam. Naturally it attracted attention of experts on the global scale. All these experts tried to correlate several entities for the cause of the disaster including reservoir-induced seismicity. Based on their opinions and findings several papers were also published in that decade. Government of India (GOI) in consultation with the UNESCO appointed a committee of International Experts in 1968 to study the extent, causes and consequences of the damage and suggest remedial measures for strengthening of the dam.

3. NECESSITY OF STRENGTHENING

The experts committee suggested several temporary as well as permanent measures. The temporary measures included offloading the reservoir.

- Grouting of the cracks by epoxy resins.
- Guniting the u/s surface.
- Anchoring the top portion of dam to the bottom portion with the help of pre-stressed cables.

These works were done immediately before the monsoon of 1968

As the NOF portion had indicated distress at various locations, various analyses and studies by Finite Element Method (FEM) technique and also testing of physical models on shake table in Japan, permanent strengthening measures were suggested by the experts committee in the form of buttresses to the shallow NOF monoliths. This included provision of shear keys in between the original face and the buttress back. The design was complex and needed elaborate construction planning and arrangements. For the deeper monoliths strengthening was suggested by full concrete backing with provision of adequate shear keys upto KRL 600.456 m above which buttresses covering about half the monolith width were provided upto KRL 653.796 m. The design criteria adopted for the Rehabilitation measures were,

- No tension in Pseudostatic analysis with $\alpha h = 0.2$
- Tension upto 300 t/m^2 under dynamic conditions with seismic coefficient of 0.5 g.

The overflow (OF) portion was devoid of any noticeable distress and hence it was not felt necessary to strengthen it along with the NOF portion in those days.

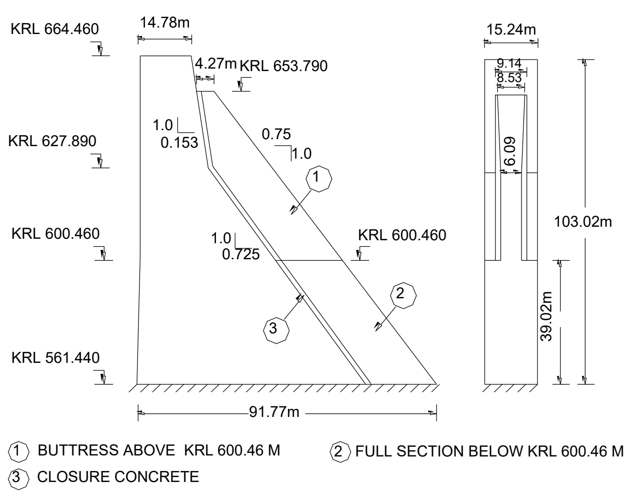


Fig. 1 : Strengthened NOF Monolith of Koyna Dam

3.1 Killari Earthquake of 1993

After major earthquake of Koyna of 1967 the Deccan plateau remained tectonically active with occurrence of earthquakes of small to mild intensities in and around Koyna-Warana basins till the year 1993. Again in the year 1993, Deccan plateau gave a big surprise to the engineering community in the form of Killari earthquake. On September 30, 1993, the Marathwada region of Maharashtra State was hit by an earthquake of 6.3 magnitude on Richter's scale at Killari. The result of this tectonic activity was so pathetic that the Government of Maharashtra had to re-evaluate the standards and commonly adopted norms for 'seismic design' of existing dams. The Government in due course constituted a committee under the chairmanship of Shri. V. R. Deuskar, Rtd. Secretary, Irrigation Department, comprising of eminent personnel from the Indian Engineering Community. The scope of this committee was to

- Review safety of all the 27 medium and major dams located in the earthquake-affected regions after deciding on the modifications to be carried out in design standards.
- The committee was also asked to check the necessity of strengthening of these dams and to suggest appropriate strengthening measures to be adopted based on the revised seismic parameters. These dams included Koyna and Kolkewadi dams under the Koyna Hydroelectric Project. The committee submitted its report to the GOM in 1997, which was accepted in early 1999.

3.2 Recommendations of the Deuskar Committee

The committee studied the Koyna dam in detail with revised earthquake parameters and confirmed that the NOF section of Koyna dam that was already strengthened with concrete backing after 1967 Koyna earthquake, is safe even for the proposed raising of FRL by 1.52 m i.e. upto KRL 659.43 m. Similarly, the expert committee studied the efficacy of the OF section for revised seismic parameters and recommended the necessity of its strengthening even without the proposed raising. The committee recommended to,

- Design the Over Flow section considering the raising of FRL by 1.52 m for 0.332 g PGA and for the accelerogram generated corresponding to the Maximum Credible Earthquake (MCE) of 6.8 magnitude measured on Richter's scale by dynamic analysis.
- Conduct comprehensive physical hydraulic model studies for the proposed profile of the OF section and energy dissipation arrangements.

4. EVOLUTION OF THE STRENGTHENING SECTION

The design philosophy adopted for spillway strengthening was to decide the preliminary section using Pseudostatic analysis with some margin for the stresses and then carryout detailed 2-D FEM analysis on the section thus arrived to find its efficacy under dynamic conditions and finally check the strengthened section from hydraulic point of view.

4.1 Pseudostatic Analysis

The pseudostatic analysis carried out for the seismic parameters recommended by the experts committee clearly indicated tension on the U/S face with a maximum value of 149 t/m^2 for the critical load combination 'G' (dam at F.R.L. + Earthquake + Extreme uplift). According to the provision of Indian Standard 6512-1984, the maximum tension allowed for 'G' condition is 0.04 fck i.e. 84 t/m^2. It was evident that the dam needed strengthening of the u/s face. However, considering the depletion restraints imposed by probability of disruption of power generation, it was impracticable to carry out any strengthening measures on u/s face. It was, therefore, decided to flatten the d/s slope so that the heel stresses were reduced to the desired extent. Various trials with different slopes indicated that flattening of the d/s slope up to 1(V):1.1(H) appreciably reduced the tension. For the load combination 'G', maximum tension observed was 61.3 t/m^2, which is less than the permissible value of 84 t/m^2. This section was adopted for further dynamic analysis studies.

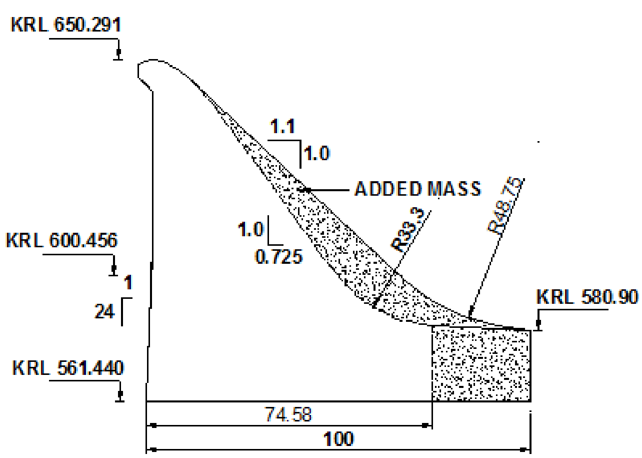


Fig. 2 : Strengthened Over Flow Section of Koyna Dam
* Levels and dimensions are in meter

The details of the input parameters and the results of pseudostatic analysis are furnished as following.

Details of input parameters:

(i) Concrete unit weight ……………….. 2.65 T/m^3

(ii) Top Bund Level …………………….. 664.464 m

(iii) Maximum Water Level……………… 661.337 m

(iv) Full Reservoir Level…………………. 659.434 m

(v) Foundation Level……………………. 561.440 m

(vi) Horizontal Seismic Coeff. (αh) at top.. 0.407

(vii) Vertical Seismic Coeff. (αv) at top….. 0.271

Stresses at Foundation Level (KRL 561.440)
For Load Combination G (FRL+EUL+EQ)

Stresses @ Heel (T/m^2)	Stresses @ Toe(T/m^2)
-61.3 (Tension)	192.05 (Compression)

4.2 Dynamic Analysis

The dynamic analysis was performed using a software developed by Prof. Dr. Saini from I.I.T., Roorkee and another software EAGD-84 (developed by the University of Berkeley) and It was observed that the value of E_{dy} of concrete (dynamic modulus of elasticity) used in this analysis has more impact on the results than any other parameter. The value of E_{dy} used was 5.8 x 10^6 t/m^2. The tensile stresses obtained in the FEM analysis with the new section were above 450 t/m^2 at a few locations, which were very high as compared to the permissible value of 300 t/m^2. As such the Board of Consultants for Koyna Project recommended use of realistic values of Edy of concrete in the analysis. An experiment was conducted on the 16 cores extracted from the dam body. Out of these 8 cores were from over flow portion of Koyna dam. These cores were subjected to sinusoidal loads with different frequencies and deformation time histories were obtained. Based on these the value of Edy was determined by method of superposition. Experiment was done in the IEOT laboratory of ONGC, Panvel with the help of CWPRS, Pune which gave the Edy value of 4.59 x 106 t/m^2 as against earlier assumed value of 5.8 x 106 t/ m^2 obtained from mathematical model studies.

4.3 Final Analysis for Strengthening Section

Based on the Edy value of 4.59 x 10^6 t/m^2, the dynamic analysis was carried out for the three different accelerograms viz. Koyna EQ (actual recorded on 11th Dec 1967), ARC (A.R.Chandrashekharan) and CW&PRS (Central Water & Power Research Station) later two of which were synthetic accelerograms, developed for similar sites. Softwares mentioned above were used for these analyses. Results of these analyses and sections showing the stress contours are furnished below.

Case I : Maximum Tensile Stresses using ARC Accelerogram with Peak Ground Acceleration (PGA) 0.332 g

Face	Node No.	Stress (T/m^2)
Upstream	139	1037
Downstream	130	934

Case II : Maximum Tensile Stresses using Warna Specific Accelerogram with PGA 0.46 g

Face	Node No.	Stress (T/m^2)
Upstream	383	986
Downstream	130	765

Case III : Maximum Tensile Stresses using Koyna Spec. Accelerogram with PGA 0.6416 g

Face	Node No.	Stress (T/m^2)
Upstream	383	962
Downstream	130	815

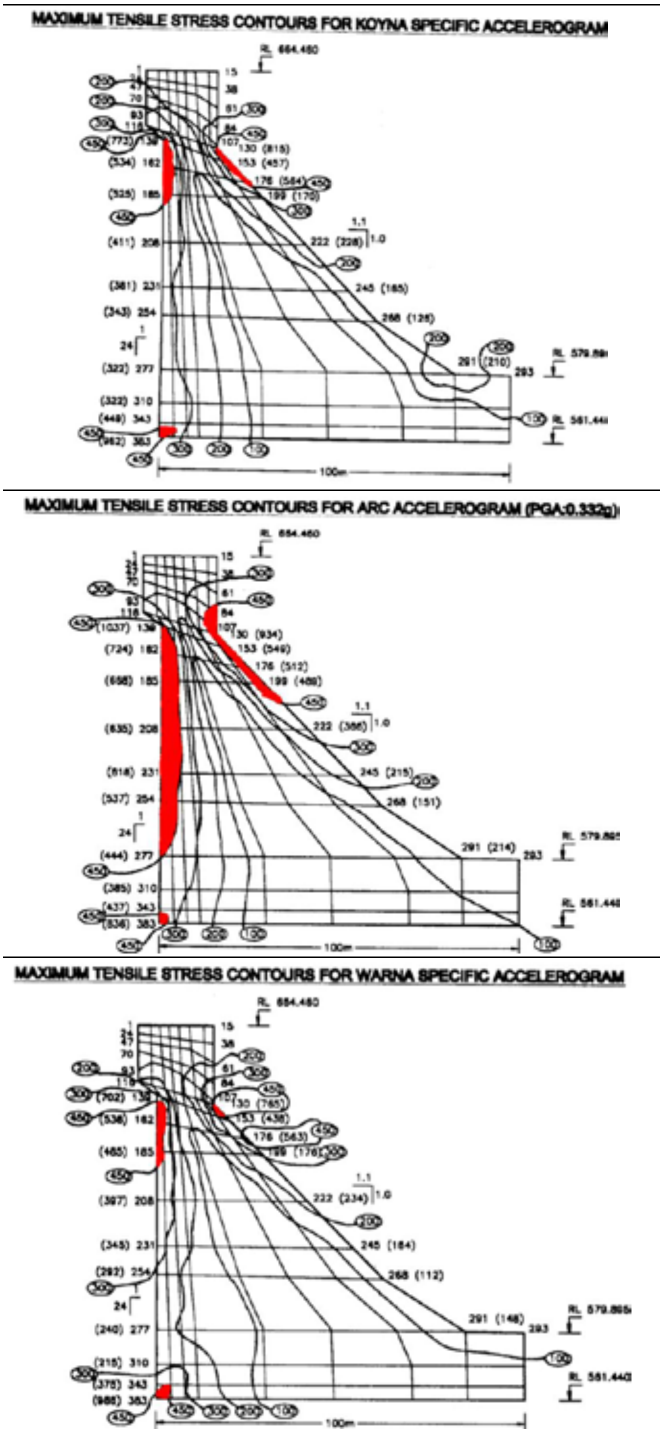


Fig. 3 : * Values in bracket indicates Tensile Stresses in T/sq.m.

It could be observed that the values of the tensile stresses were for Maximum Considered Earthquake (MCE) condition and at singular points where change in geometry & stiffness of the section were predominant. In rest of the section the tensile stresses were reasonably within the permissible limits (corresponding to the apparent seismic tensile strength of 540 t/m^2 for the rubble concrete of OF portion of Koyna dam). After deliberations with the Board of Consultants for Koyna Project, the strengthening section having slope of 1(V):1.1(H), which envisaged providing full concrete backing, was adopted for the strengthening of overflow portion of Koyna dam to cope with the raising of FRL and revised seismic parameters.

4.4 Hydrological Aspects

In the process of design of the strengthening section the hydrological aspect was also of major consideration. This aspect was important as the flood routing studies were to be conducted for the raised FRL of 659.43 m. These studies were carried out for the PMF corresponding to 68.38 cm rainfall in 24 hours. The catchment simulation study indicated probable maximum flood (PMF) of 17095 m^3/s. With unrestricted releases, the MWL computed was KRL 661.20 m. Based on this MWL and the corresponding spillway discharge of 5738 m^3/s, the preliminary design of the EDA was evolved. Model studies on 1:50 ground scale model for evolution of final layout & design of EDA were carried out by the Maharashtra Engineering Research Institute, Nashik.

Fig. 4 : 1:50 GS Model Study at MERI, Nashik

5. CONSTRUCTION PLANNING AND METHODOLOGY

The scheme of strengthening included providing full concrete backing of rubble concrete from the level of original dam foundation (KRL 561.44 m) to the upper tangent point of ogee (KRL 645.0 m). This concrete was to be essentially of the same grade and strength as that of the original dam. To overcome the interface problems between the old and the new concrete, it was decided to first cast the strengthening concrete with a gap of 1.2 m, allow it to cool and shrink for at least 30 days and then join it to the dam body by closure concrete. This was to be done when the reservoir level is at the predetermined level known as Bond RL, to account for the locked up stresses in the body of the dam, since the dam could not be emptied during the strengthening work for the reasons mentioned earlier.

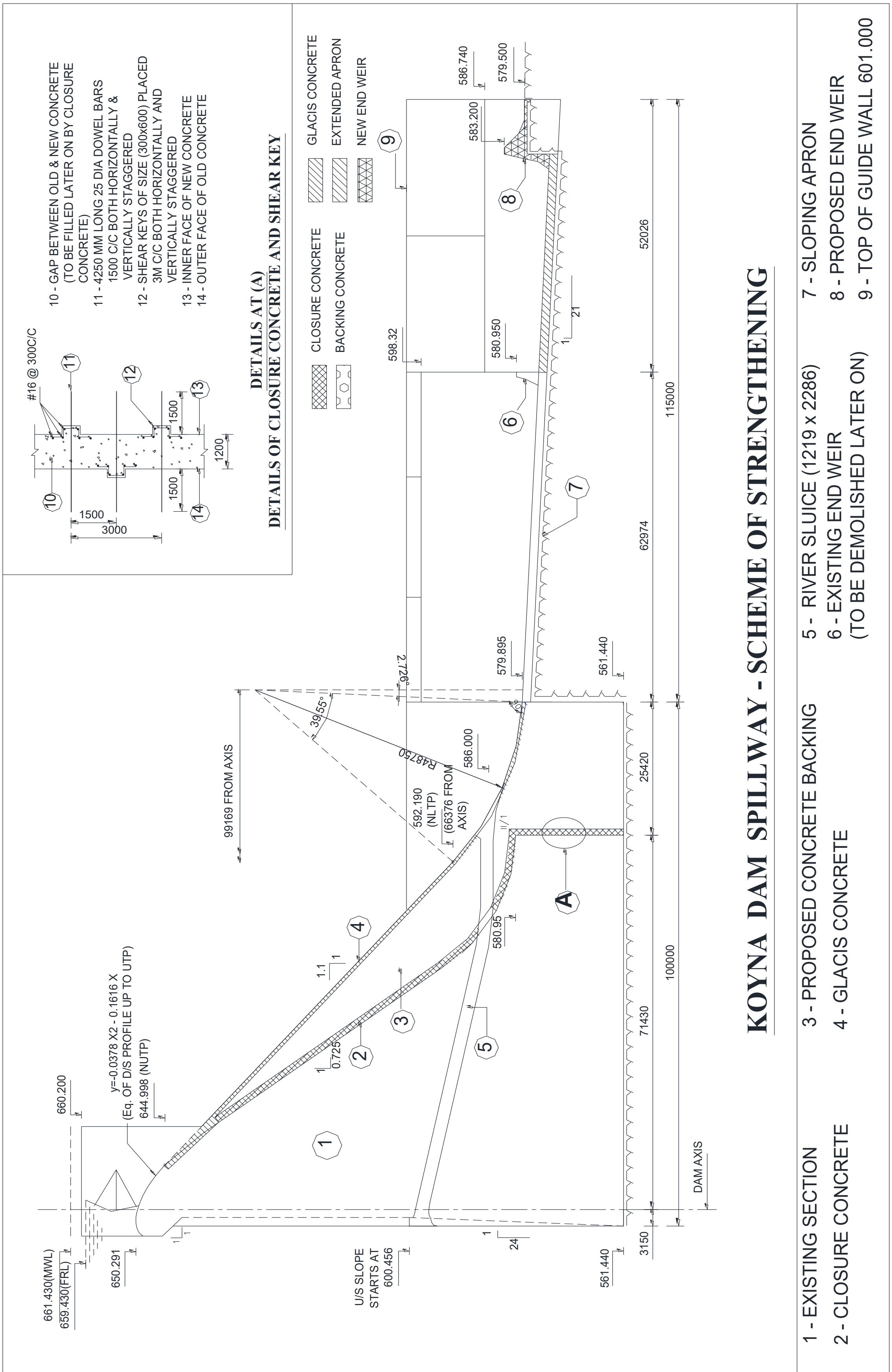

KOYNA DAM SPILLWAY - SCHEME OF STRENGTHENING
1 - EXISTING SECTION
2 - CLOSURE CONCRETE
3 - PROPOSED CONCRETE BACKING
4 - GLACIS CONCRETE
5 - RIVER SLUICE (1219 x 2286)
6 - EXISTING END WEIR
(TO BE DEMOLISHED LATER ON)
7 - SLOPING APRON
8 - PROPOSED END WEIR
9 - TOP OF GUIDE WALL 601.000
DETAILS AT (A)
DETAILS OF CLOSURE CONCRETE AND SHEAR KEY
10 - GAP BETWEEN OLD & NEW CONCRETE (TO BE FILLED LATER ON BY CLOSURE CONCRETE)
11 - 4250 MM LONG 25 DIA DOWEL BARS 1500 C/C BOTH HORIZONTALLY & VERTICALLY STAGGERED
12 - SHEAR KEYS OF SIZE (300x600) PLACED 3M C/C BOTH HORIZONTALLY AND VERTICALLY STAGGERED
13 - INNER FACE OF NEW CONCRETE
14 - OUTER FACE OF OLD CONCRETE
#16 @ 300C/C
CLOSURE CONCRETE
BACKING CONCRETE
GLACIS CONCRETE
EXTENDED APRON
NEW END WEIR
661.430(MWL)
659.430(FRL)
650.291
U/S SLOPE STARTS AT 600.456
561.440
660.200
y=-0.0378 X2 - 0.1616 X
(Eq. OF D/S PROFILE UP TO UTP)
644.998 (NUTP)
99169 FROM AXIS
R48750
39.55°
2.726°
592.190 (NLTP) (66376 FROM AXIS)
586.000
580.95
579.895
598.32
580.950
583.200
586.740
579.500
DAM AXIS
3150
71430
100000
25420
62974
115000
52026
A

Fig. 5

Strengthening of spillway portion of any dam with full backing is essentially a time bound work especially so when the dam is to be kept in operation during the strengthening work. It is for the first time in India that the spillway of such a large dam as Koyna is being strengthened. The Water Resources Department of Maharashtra State had carried out the strengthening works of NOF sections of various dams in the past but this particular work needed meticulous planning and adoption of equally competent construction methodology for its successful completion.

After studying various intricacies of the proposed strengthening work and the available expertise for performing this work, it was planned to complete the said work in two seasons (i.e. Oct. 2004 to May 2006). In the first year i.e. from Oct. 2004 to May 2005, it was planned to complete the strengthening work up to KRL 580.90 m i.e. below riverbed portion. This also included the extension of stilling basin, construction of new end weir, raising the guide walls of EDA and excavating and back filling with concrete the 25 m wide, 20 m deep and 90 m long foundation block. The first year's strengthening work included 76000 Cum of rock excavation with controlled blasting, 85000 Cum soil excavation, 32000 Cu.m. conventional concrete, 3000 sq.m. of pressure grouting, reinforcement, anchoring and 36000 Cu.m. backing concrete of grade M21 with MSA 150 mm and 2000 Cum closure concrete. To complete this work suitable machinery like concrete batching plant of 150 Cu.m./hour capacity was installed at site, for precooling the concrete, 105 MT/day capacity ice plant, stone crusher of capacity 125 Tons/hour etc. and other state-of-the-art equipments and machinery was also deployed at the site.

In second year construction work i.e. from Oct. 2005 to May 2006, strengthening of OF section from KRL 580.90 m to KRL 645.0 m (UTP of OF section) would be completed. In this season 52000 Cu.m. of backing concrete, 9000 Cum of closure concrete and 10000 Cum of other concrete would be placed.

Studying all possible difficulties in construction, the construction planning was done in such a way that the both the season's targeted work would be completed before start of mansoon at the end of every season.

6. CHALLANGING AND TIME BOUND WORK OF EXCAVATION AT THE TOE OF THE DAM

Total 70000 Cu.m. excavation in hard rock was necessary for foundation block and for extension of stilling basin at the toe of dam. Though sound rock was available at riverbed level itself, due to underlain volcanic breccia layers and structural design requirement to increase the dam section right from original foundation level and hydraulic performance requirement of EDA as resulted through model studies, the excavation in intact rock was mandatory. Controlled blasting was certainly warranted, but this was not usual open ground excavation. The work area was very close to the dam proper and also to the Dam Foot Power House. Due to hydropower commitments along with irrigation and drinking water requirements in the region, closure of the powerhouse was impracticable. At the same time blasting was to be completed within stipulated time frame to avoid future deviation in construction programme. This particular problem was referred to Central Water and Power Research Station, Pune for study and recommendations of blasting pattern, blasting material to be used etc. Detailed studies and trial blasts in similar rock type on nearby site were carried out to find out the attenuation characteristics of the foundation rock. The primary objective was to safeguard the dam and dam foot power house and also the other nearby structures from the ill effects of blasting viz. blast vibrations characterized by Peak Particle Velocity (PPV), fly rocks and air over pressures. After detailed studies and analysis the safe PPV for dam was stipulated as 70 mm/s and for powerhouse as 10 mm/s. The entire excavation plot was divided into five zones. Blasting pattern, quantity of blasting material, depth of holes etc. was finalised for each zone considering the limitations on PPV. The accompanying sketches show the plan and cross section of the blasting area and the methodology adopted. The blast vibrations were measured using Instantel (Canada) - make Seismogragh kept both at the dam toe and in the power house. Frequency content of blast vibrations and the air overpressures were also measured. In all 770 blasts were taken. The following table depicts the control and monitoring achieved during excavation by control blasting. The excavation was completed successfully ahead of schedule.

PPV mm/s	Below 20	20-40	40-50	above 50	Total
	741	22	3	nil	770

Air over-pressures recorded were less than 120dB for all the 770 blasts.

Frequency content of blast vibration was more than 60 Hz in 723 cases and less than 60 Hz in only 47 cases.

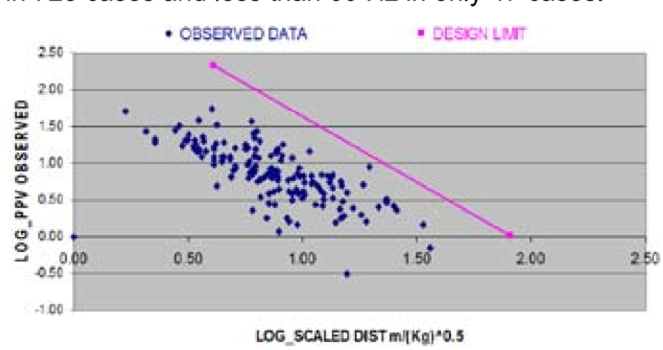


Fig. 6 : Graph Showing PPV recorded and Allowable limit

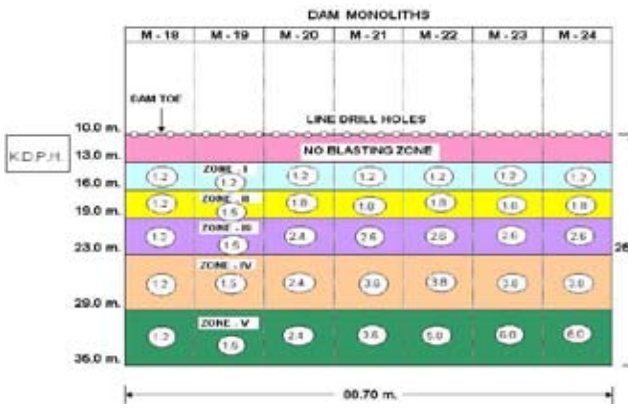


Fig. 7 : Drawing showing the blasting zones and charge per delay

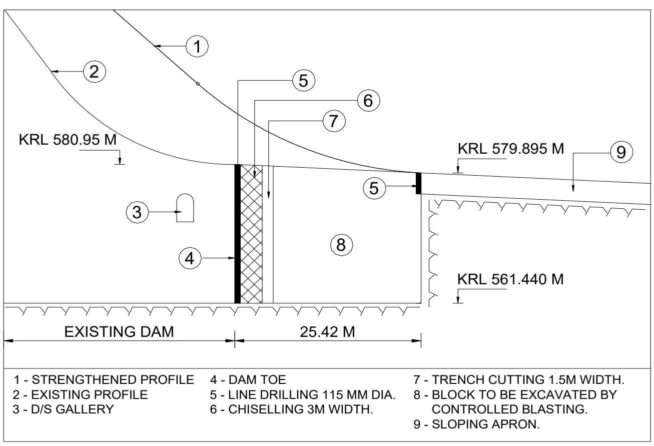


Fig. 8 : Sketch showing the methodology of controlled blasting

7. CONCRETE PLACEMENT

Placing of the backing concrete in position is one of the important activities. Time period available for concreting was very limited due to construction activities like completion of blasting, restriction of concrete lift height and minimum required period between two successive lifts from heat of hydration point of view etc. Before starting the concreting no of alternatives were studied to prepare mix design to satisfy the design requirements like minimum unit weight of 2.65 t/Cu.m. and minimum compressive strength of 2100 t/sq.m with as little cement as possible. The cement content governs the heat of hydration during setting of concrete. Finally, M-21 grade concrete with 177 kg/Cu.m. of cement was designed in the project laboratory, the refinement work was done later on by the IIT, Mumbai.

Mix Design

Plum concrete using 150 mm size plums and with an effective cement content of 177 Kg/m^3 was used for construction of original dam. It was thought that using the state-of-the-art high-speed mixers of sufficient capacity, a more cohesive concrete mix of 150 MSA could be obtained very easily. The concrete placed in the 1.2 m gap between the new strengthening concrete and old dam concrete was of M-20 grade with cement of 200 Kg/m^3 and 80 MSA.

The initial mix designs were done in Koyna laboratory and for further refinement the mix design was done by the IIT Mumbai The proportion of ingredients is shown in Table 1.

The water as above was partially replaced with flake ice 50-70% based on the requirement to achieve the placement temperature.

Next important job was to design the thickness of lift, minimum curing/cooling period required before placing next lift, method of cooling of concrete like precooling and/or post cooling etc. This thermal study of backing concrete was also carried out with help of CW&PRS, Pune. Studies to find out various properties of concrete like tensile and compressive strengths, tensile strain capacity, coefficient of thermal expansion etc. on the various samples of backing concrete were carried out in the laboratory at Pune as well as some in-situ tests were carried out. From this study the lift of backing concrete as 1.50 m, minimum gap between two successive lifts as 3 days were finalised. In view of the constraint on the construction schedule pre-cooling as well as post cooling was resorted to. Precooling was to be achieved by adding chilled water and ice flakes during mixing concrete in batching plant and post cooling with circulating chilled water through embedded G.I. pipes in backing concrete for at least 30 days. The recommended placement temperature for concrete was 18^0 C and that of chilled water used for post cooling was 22^0 C. The

Table 1

Mix	Cement (Kg).	Coarse Aggregate (Kg)					Fine Aggregate (Kg)		Water (ltr)	Admixure (ml)
		150-80	80-40	40-20	20-10	10-4.75	Sand	Grit		
Strengthening Concrete	177	897	243	243	194	121	364	364	106	1200
Closure Concrete	200	--	858	348	232	186	348	348	120	860

cooling water was circulated in the concrete through G.I. pipes embedded in concrete in the form of pipe network with 1.0 m horizontal spacing and 1.5 m vertical spacing. Following photograph shows the post cooling pipes embedded in concrete. A graph of inlet and outlet temperature of the post cooling water and the resulting concrete temperature obtained from the thermocouples embedded in the concrete is enclosed which gives the extent of control on the concrete temperature achieved. A maximum temperature of 44^0 C was envisaged in the thermal analysis against which temperatures of 38 to 41^0 C were recorded actually by the thermocouples.

Fig. 9 : Photograph showing concrete placement and post cooling arrangement

Fig. 10 : Photograph showing Second Year Strengthening work in progress

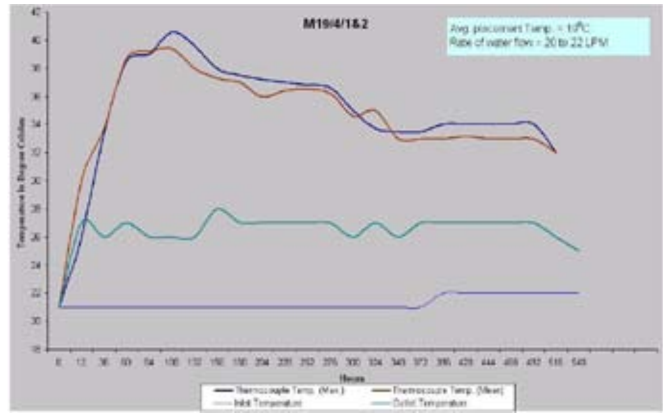


Fig. 11 : Graph showing results of post cooling and concrete temperature

8. PROBLEMS FACED AND REMEDIAL MEASURES

8.1 Excavation of Rock Covered by RCC Layer

Line Drilling 115 mm dia 400 mm c/c upto full depth to provide separator between Dam & excavation plot

Fig. 12 : Breaking of reinforcement to enable concrete excavation and Face opening for concrete excavation is done

The foundation portion of the extended dam was previously the sloping apron of the stilling basin and as such it was covered with reinforced concrete. The thickness of the concrete varied from 70 cm to 270 cm. Initially it was planned to excavate the concrete and the rock in a similar manner. When drill holes were taken in the concrete and blasted, the holes used to get blown off. Some times the underlain rock used to get shattered but the concrete provided a comparatively unaffected surface. This was because of the presence of the reinforcement in the concrete.

To overcome this problem, the concrete top was first broken using hydraulic breaker in the form of grid of 3 m x 3 m, to expose the reinforcement. The reinforcement bars were then cut using gas cutters. Regular drilling blasting as per the pre-determined pattern gave very good results later on.

8.2 Cutting of Shear Keys

Fig. 13 : Cutting of Shear Keys

Shear keys were cut in the old concrete to have a proper bonding between the old concrete and the new strengthening concrete via closure concrete. Therefore, the success of the bonding depended upon the efficacy of the shear keys. However the cutting of shear keys was not only a difficult job but also a time consuming one. Efforts were made to cut these shear keys while the excavation was in progress to save the time during concreting period. However, the cutting went on lagging behind and subsequently it was abandoned at certain locations as the excavation which was equally critical and time bound could not be stopped. Once the concreting started, the hydraulic breaker could not be employed for cutting the shear keys. And also, as the concreting progressed as per the time schedule, it was difficult to work with the usual tools in the limited space of 1.2 m between the two concretes. One innovative method of using drilling and splitting concrete solved this problem. The concrete face where shear keys are to be cut was drilled using 50 mm dia drill for a depth equal to the size of the shear key. Hydraulic splitter was used to wedge out the concrete in this portion. This method proved to be effective and fast and all the balance shear keys were cut in time and to their full section.

8.3 Vibrating 150 MSA Concrete

Initially during the trial mixes and during the casting of the test block for the shrinkage study, it was found that the concrete of 150 MSA was not getting vibrated at all. The efforts employed by the usual 40 or 60 mm dia needle vibrators were found inadequate. Then, high frequency internal vibrators of 140 mm dia with powerful in-built electrical motors of 3000 rpm MK 747 make, model FA 250, capable of vibrating at 7500 cycles per minute were specially ordered which turned out to be adequate for vibrating the 150 MSA concrete and the concreting went on very smoothly.

9. CONCLUSION

The experience of various analyses performed for design of the strengthening section of Koyna dam has underlined the importance of the dynamic analysis in designing.

For such designs use of “factual strength parameters” obtained through large diameter core testing of the existing dam material appears most essential.

When such a time-bound situation arises the only tool in hand is meticulous planning and truthful follow-up of the same.

During the construction for facing adhoc situations change of methodology is the only alternative, which has to be adopted at right time.

Support of appropriate machinery for speeding of construction is of vital importance as seen from the situation arose during the first year’s work.

Acknowledgement

Author are grateful to all the engineers those were involved in design and construction of strengthening of Koyna dam. Specially from Koyna Design Circle, Pune, Koyna Construction Circle, Satara, Maharashtra Engineering Research Institute, Nashik, Central Water and Power Research Station, Pune.

REFERENCES

1. “Report of the Experts committee under the chairmanship of Shri V. R. Deuskar, Retd. Secretary, I.D., Maharashtra, to review designs of all high major dams in the earthquake prone regions of the state & to carry out strengthening measures where necessary & to ensure stability in future seismic events.
2. “Proposed strengthening of overflow section of Koyna dam – An overview”, by V. V. Gaikwad, Chief Engineer, Koyna Project, Pune & R. V. Panse, Superintending Engineer, Koyna Design Circle, Pune.

Applications of Pumping Pressure Measurements while Pumping Concrete through 2.432 km for Tunnel Lining

Chetan Hazaree
Hindustan Construction Company Ltd., Mumbai

Sunil Bauchkar
BASF India Ltd., Navi Mumbai

1. INTRODUCTION

Pumping, known for its speed, reliability, efficiency and safety, is a widely accepted and reliable method of transporting concrete. Periodically newer pumping records are being created in terms of height, length, volume and type of concrete. Albeit, pumping has become a preferred choice for many concreting applications, its applications in hydro-sector have remained limited. This paper presents one such caseof concrete lining of the 3.76 m diameter head-race-tunnel (HRT) at the 100 MW run-of-the-river Sainj Hydroelectric Project situated in Himachal Pradesh, India. The concrete was pumped through 2.4 km in extreme weather conditions. Figure 1 shows the Schematic layout of Sainj HEP and long-distance pumping; the 1.36 km was on the downstream side, while 2.432 km on the upstream.

1.1 Concrete Flow in Pipe

Concrete flow in a pipe typically occurs in three layers or regions as shown in Figure 1, detailed discussion is considered to be beyond the scope of this paper.

(i) Slip-layer or lubrication layer,

(ii) The shearing region or layer, and

(iii) The inner concrete or layer, also referred to as a plug flow layer[1;2]

1.2 Pumpability of Concrete

Pumpability is the ability of confined concrete to flow (to be mobilized) under pressure while maintaining its initial properties[3;4]. It can also be defined as the aptitude of concrete to be placed using a pump. The concept of pumpability therefore relates to the formation of blockages and does not refer to such pumping parameters as flow and pressure[5]. Some research studies dealing with this practical problem of pumping have been reported but have offered limited understanding[6;7;8;5;9;10;11]. Pumpability is not an intrinsic quality of concrete but a concept that involves all the pumping parameters including the composition of the concrete, the configuration of the plant, and the pumping procedure[5;12].

Concrete pumping involves the flow of a complex fluid under pressure in a pipe, predicting its flow requires detailed knowledge of its rheological properties. However, proper characterization needed to predict flow is not easy to achieve. This is because it involves understanding of a variety of factors such as dynamic segregation, the stability of entrained air, the geometry of the pumping circuit, the dynamics of a slip-layer formed between the bulk concrete and the pipe wall, and the relationship between the pressure and the flow rate[13]. Pump pressure measurements while pumping concrete are not used for estimating either satisfactory progression of concrete or in

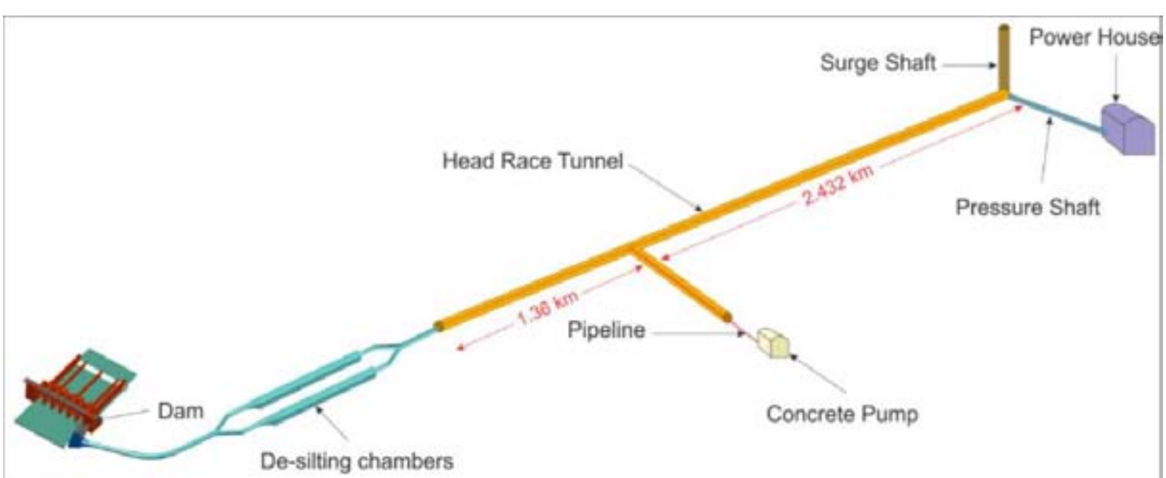


Fig. 1 : Schematic layout of Sainj HEP and long-distance pumping

anticipation of any blockages. Some attempts have been made to appreciate the dissipation of pumping pressure along the flow line(14).

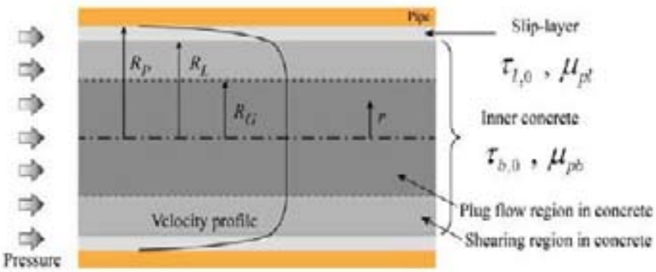


Fig. 1 : Profile of flow of concrete in a pipe(2)

1.3 Typical Challenges

Typical challenges with pumping involve

(i) Instability under pressure leading to segregation or forced bleeding and is often associated with mixtures having deficient particle size distribution or with excessive water/cement ratio(7)

(ii) Hydraulic conditions are present during concrete pumping and the amount of friction is not related to the pressure applied, which in turns means that the energy loss in a straight length of pipe is linear(15). This means that there has to be adequate energy considerations during the selection and designing of concrete pumping system

(iii) Blockages taking place during priming, pumping, restarting and cleaning(5)

(iv) Managing the safety aspects during pumping(16)

Some researchers have discovered some tests for assessing the pumpability potential of concrete mixtures(5;2), however pumping as an interactive activity is difficult to predict by only testing the concrete.

1.4 Premise for Selecting Pumping Pressure

While pumping concrete through long distances as was done for Sainj, due to multitudes of factors involved, it was getting difficult to predict successful pumping of concrete using tests on concrete only. Hence, an ardent need for assessing other, non-concrete parameters was felt. Of multiple such parameters assessed, the parameter of monitoring of pumping pressure was found to be of great practical utility for estimating concrete reach and anticipating any blockages and/or other issues. This paper reports studies conducted around this topic. While the pumping pressure was found to be useful, it by itself cannot predict successful concreting. A combination of concrete test results, practical insights into balancing with weather conditions and monitoring of speeds (production, pumping, placement, etc.) form a critical set of parameters that influenced successful pumping.

2. PLANNING FOR PUMPING

2.1 Pump and Pipeline

Besides routine, planning for pumping involves Plant and equipment selection, layout finalization, manpower and gangs' planning and risk and safety planning. Selection of the diameter of pipeline, length of pipeline, pipeline layout and fixing scheme, pipe material and thickness, concrete mix design and arriving at optimal working pressure are some of the key aspects of planning. The initial equipment and pipeline selection were based on the asking rates for concrete placement and cycle time for tunnel lining. For a 2.5 km pipeline, concrete mix with 20 mm maximum size of aggregate, a DN150 (diameter of 150 mm), a concrete output ranging between 15 – 30 cum/h, an estimated concrete pressure requirement of 200+ bar was arrived at. For this requirement, a concrete pump with configurations summarized in Table 1was selected.

While planning for pumping for this project, no practical or experience based guidelines on pumping for such long distances were available. Hence, after initial planning and mobilization, most of the fieldwork was based on an intuitive combination of experience, theory and trial and error.

Table 1 : Concrete pump parameters

Parameter	Unit	Side	
		Piston	Rod
Max. number of strokes	/ min	14	21
Max. Concrete Pressure	bar	243	156
Max. theor. concrete output	m^3/h	43	66
Pumping Cylinder, Dia x stroke	mm	180 x 2000	
Piston displacement, 2 cylinders	l	50.89	
Capacity of charging hopper	l	600	
Drive power	kW	330	
Driving speed	rpm	2100	

2.2 Weather Conditions

The differences and the variations between the external (concrete production point) and internal (i.e. at gantry) climates caused substantial changes in the behaviour of concretes. This is because the concrete goes through changes as it moves through long distance pipe. Periodic performance trials and measurements at the production and placement points are essential. The transitions in temperature and humidity need to be carefully monitored along with specific responses of concrete viz. workability, retention, setting behaviour, etc. Figure 3 shows an example of temperature and humidity monitoring profiles along the tunnel length. Such profiling was conducted before undertaking all pours.

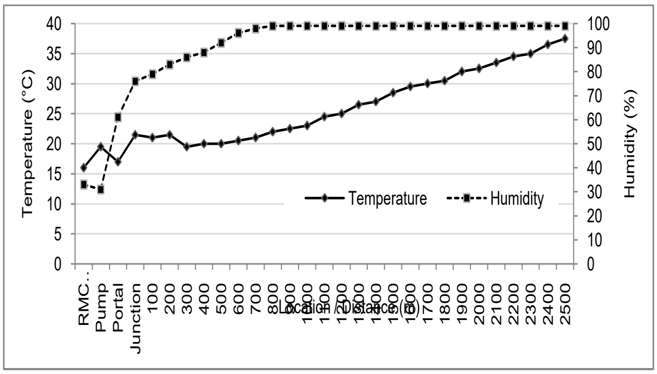


Fig. 3 : Temperature and humidity profiles along the tunnel length (an example)

2.3 Framing Concrete Specifications

Long-distance pumping is actually a method of transporting concrete. Accordingly the contract specifications have to be suitably modified while conforming to the contract stated strength and durability requirements. This includes defining the behaviour of concrete while pouring at the concrete pump hopper and that at the pouring point. After studying all the relevant parameters and considering various aspects involved a set of concrete specifications were derived and are summarized in Table 2.

Table 2 : Summary of the targeted mix design characteristics

Parameter	Minimum	Maximum
Flow (mm), season dependent	550	700
Type of concrete*	Flowable	Flowable
Final Setting time (h) in lab	16	22
Pumping distance (m)	450	2500
Retention time (h)	4	12
Ambient temp. outside the tunnel (°C)	5	33
Ambient temp. inside the tunnel (°C)	25	45
Relative humidity inside the tunnel (%)	95	100
Stripping strength of concrete (MPa)	5	7
Characteristic strength (MPa)	25	-

*Homogeneous, non-segregating, enough paste, stable

2.4 Risks Associated with Pumping

The risks associated with concrete pumping are many and varied and can originate from hazards such as:

(i) plant and equipment (concrete placement booms, pump gauges, concrete pipelines, pipe clamps, anchor brackets, pipe movement, delivery hose, receiving hopper,

(ii) placement of plant and equipment – proximity to traffic, public, utilities, etc., ground stability

(iii) pumping tasks - concrete delivery, pump and boom operation, concrete pouring and line cleaning and on-road travel time

(iv) controlling of noise and fumes[16]

Retention- risked volume-setting times

The concrete workability retention period is a function of required concrete output, diameter of the pipeline, distance through which pumping is undertaken and anticipated contingencies (min. of 2 h) in the entire operations. Moreover, the concrete mixture has to account for the workable pump pressure range. The retention and setting times guide the selection of admixture to be used.

One fact that a significant volume of concrete is always at risk inside the pipeline in case of long-distance concrete pumping needs to be always borne in mind. For a fixed diameter, with the increase in pipe length, the risked volume of concrete that remains within the pipeline between the starting and ending points increases (refer Figure 4). For example, a pipe diameter of 150 mm requires a concrete volume of 0.18 m^3 per m of pipe hence and for 2.432 km, the risked volume of concrete is about 44 cum. The time cycle requirements of 30 h necessitated targeted setting time of 18-20 h.

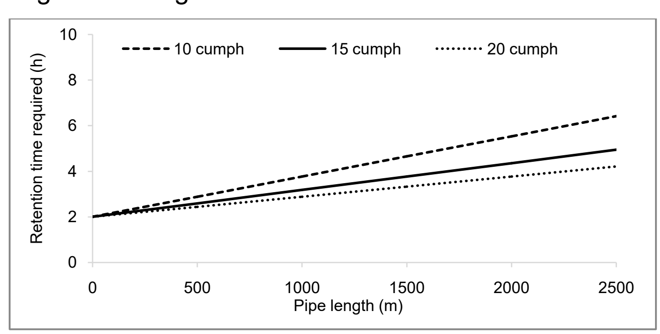


Fig. 4 : Influence of delivery rate for a fixed diameter (150 mm) on retention time of concrete

3. MATERIALS' DEVELOPMENT

Following materials were used in developing flowable concrete:

(i) OPC (specific gravity 3.15) conforming to IS 8112[17]

(ii) Fly ash (specific gravity 2.1) conforming to IS 3812[18]

(iii) Crushed aggregates with 20 mm maximum size meeting IS 383[19]

(iv) A specially developed admixture conforming to IS 9103[20]

A typical mixture for medium range distances (1.2 – 1.5 km) consisted of 340 kg OPC, 135 kg fly ash, 200 kg water, 948 kg coarse aggregate, 766 kg of fine aggregate and admixture varying between 2.4-5.2 kg per m^3. The observed flow at various admixture dosages was 600±50 mm in lab initially, and final setting times 20±1 h, 30 h. The concrete was designed for M25 grade. As the project progressed, this mix design was altered to suit specific distance ranges and accordingly the paste contents were varied. Table 3 summarizes a typical flowable concrete mixture.

Table 3 : Typical flowable concrete mix with properties

Parameter	**Value**
OPC	340 kg/cum
Fly ash	125 kg/cum
Coarse aggregate	950 kg/cum
Fine aggregate	777 kg/cum
Water	200 kg/cum
Admixture	0.50 – 1.1%, by weight of binder
Slump flow – Initial, 15 min	550-650 mm
Slump flow – terminal	450-500 mm
Setting time – Initial	15.0 – 17.5 h
Setting time – Final	19.0 – 21.0 h
24 h strength	6.5 – 3.5 MPa
3-day strength	19.0 – 28.0 MPa
28-day strength	32 – 38 MPa

Many permutations and combinations were tried in the lab followed by full-scale trials involving rotating the concrete multiple times to ensure adequate retention of concrete and other key properties.

4. PUMPING PRESSURE

4.1 Priming

Priming or lubricating of pipes is primarily done with the objectives of keeping the aggregate particles in suspension and impede their counter productive advancing; to keep the concrete-steel interface coated with a film of grout, while sealing minor gaps and iii) to prevent pumped concrete from drying and helping plug formation. In case of long-distance pumping, priming becomes much more critical. The adopted grout mixture contained 645 kg/m^3 of cement and fly ash, w/cm ratio 0.38 and admixture 7.1 kg/m^3 and occasionally retarder was used at 2.5 kg/m^3. The details of priming in upstream directions are quite different than in downstream direction.

4.2 Pressure-volume Relationship

The pumping pressure development during continuous pumping was used as a continuous monitoring tool for assessing smooth conduct of concrete pumping, anticipating blockages or similar issues, possible safety alarms. The pumping pressure was the maximum pressure shown on the dial gauge and not the pulsation. For longer distances, the maximum pressures developed were higher and reduced as the pumping distance reduced. For a pour, as the concrete travels through the pipeline, the pump back-pressure increases, reaches a peak and then remains more or less stable during the complete pour. Minor fluctuations in the pressure take place due to various reasons. Figure 5 shows stable regime pump pressure-volume profiles for two pumping distances.

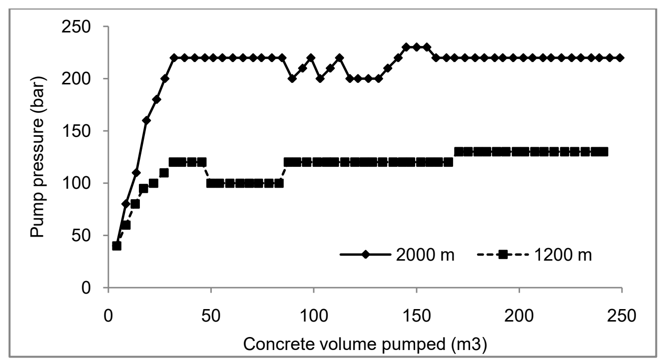


Fig. 5 : Stable regime pump pressure as function of pumped concrete volume for two distances

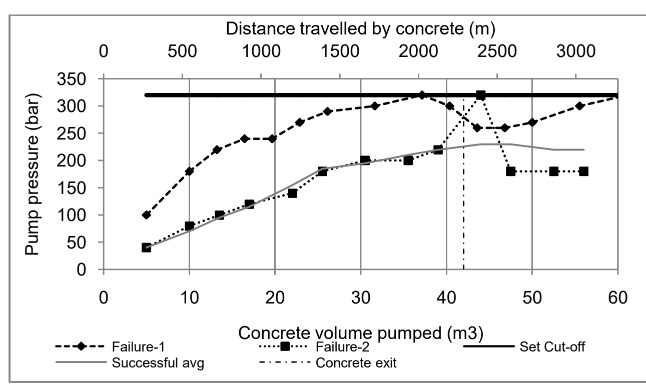


Fig. 6 : Pumping cases – 1

4.3 Cases

Anticipating concrete pipeline blocking during pumping is vital since immense investment, manpower and other efforts are involved in making even one pour successful. Being able to anticipate smooth running and blockage is a vital moral booster to the operating team as well. As the volume of concrete inside the pipeline increases, the pumping pressure also increases gradually until finally the concrete emerges from the other end, after which, the pumping pressure remains more or less constant. Here the pumping pressure is measured on the pressure gauge provided on the pump. If there is a sudden change in the pressure (increase or decrease), then a problem can be anticipated. In general, it was observed that a gradual built-up indicates blockage at a longer distances from pump, while abrupt peaking of pressure indicates blockage nearer to the pump.

(i) Failure-1 (refer Figure 6) – at about 1600 m from the pump, failure-1 took place because of an abrupt bend in two straight pipe segments leading to blockage formation at this point

(ii) Failure-2 (refer Figure 6) – at about 370 m, some blockage occurred leading to squeezing out of water and slurry, thus further intensifying the blockage due to no movement. Once the blocked segment was removed, cleaned and reinstalled, the pump back-pressure resumed its original value and pumping was resumed normally

(iii) Failure-3 (refer Figure 7) – a gradual built-up of pressure took place and was released, while pumping continued. However, within 5 min, another peaking of pressure took place as a result of blockage, which in turn led to bending of 3 m long pipe and disruption of pumping

(iv) Failure-4 (refer Figure 7) – an abrupt rupture of rubber gasket lead to opening up of a pipe-to-pipe joint at about 570 m from the pumping point. This lead to opening of a joint, misalignment followed by choking of concrete and eventually disruption

(v) Failure-5 (refer Figure 7) – concrete pumping was smoothly going on until at a length of 2100 from concrete pump, suddenly a pipe clamp opened up releasing the pressure and blocking the entire pipeline. After opening the clamp, it was observed that some concrete had mixed for a longer length in the priming grout

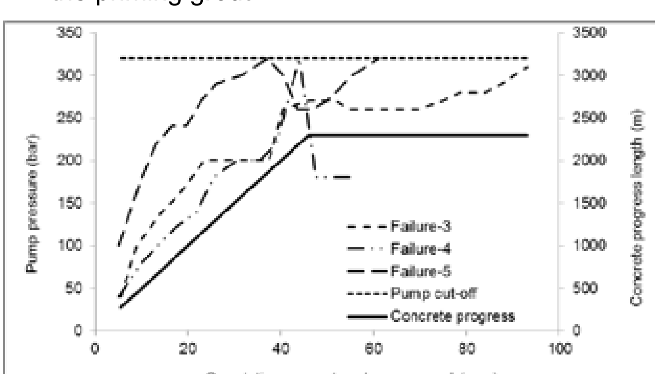


Fig. 7 : Pumping cases – 2

5. SUMMARY

(i) A well-coordinated and meticulous planning is essential for properly establishing the synergies between materials-equipment-weather-manpower. Selection and behaviors of each of these components decide the success of long-distance pumping

(ii) In Herculean efforts with risks on high volumes, it is important to be able to anticipate smooth running and/ or blockages in the pipeline during concreting. Continuous monitoring of pumping pressure offers very practical insights into the running of concrete pumping for long distances

(iii) While pumping pressure provides meaningful anticipatory information, it is crucial to blend it with concrete's properties, weather conditions and the overall pace of concrete pumping

REFERENCES

1. Lubrication layer properties during concrete pumping. Choi, M S, et al. 3, 2013, Cement and concrete research, Vol. 45, pp. 69-78.

2. Choi, M S.Prediction of concrete pumping performance basedon the evaluation of lubrication layer properties, PhD Thesis. s.l. : Department of Civil and Environmental Engineering, Korea Advanced Institute of Science and Technology, 2012.

3. Jolin, M, et al. Pumping Concrete: A Fundamental and Practical Approach. [book auth.] D R Morgan and H W Parker. Shotcrete for Underground Support X. s.l. : ASCE, 2006.

4. Laboratory procedure for comparing pumpability of concrete mixtures. Gray, J. Washington D.C. : s.n., 1962. Sixty-fifth annual meeting of the society, National Crushed stone association, June 24-29. pp. 964-971.

5. Avoidance of blockages in concrete pumping process. Kaplan, Denis, de Larrard, Francois and Sedran, Thierry. s.l. : ACI International, 2005, ACI Materials Journal, Vols. 102-M21, pp. 183-191.

6. Best, J F and Lane, R O. Testing for optimal pumpability of concrete. Concrete International. 1980, Vol. 2, 10.

7. Tests to establish concrete pumpability. Browne, R and Bamforth, P. 5, s.l. : ACI, May 1977, ACI Journal-Proceedings, Vol. 74, pp. 193-203.

8. Chapdelaine, F. Fundamental and practical study on pumping of concrete (in French), PhD thesis. Quebec, Canada : Laval University, 2007.

9. Prediction of concrete pumping: Part I - Development of new tribometer for analysis of lubricating layer.

Kwon, S H, et al. 6, 2013, ACI Materials Journal, Vol. 110, pp. 647-655.

10. On the real time assessment of pumping mixes under actual or quasi actual conditions. Rio, O and Rodriguez, A. Reykjavik, Iceland : RILEM, 2009. 3rd RILEM symposium on rheology of cement suspensions such as fresh concrete 19-21 August. pp. 97-104.
11. Pumpability of fresh concrete. Sakuta, M, et al. Tokyo, Japan : Takenaka technical ressearch laboratory, 1989.
12. Single stage concrete pumping through 2.432 km (1.51 miles): Weather and execution challenges. Hazaree, Chetan and Mahadevan, Viswanath. 2015, Case studies in construction materials, Vol. 3, pp. 59-69.
13. Choi, M, et al.Research needs to advance concrete pumping technology,NIST TN 1866. Gaithersburg : NIST, 2015.
14. Riding, K A, et al.Best Practices for Concrete Pumping. Manhattan : Kansas DOT, 2016.
15. The resistance of concrete pumped through pipelines. Ede, A N. 27, Magazine of concrete research : s.n., 1967, Vol. 9, pp. 129-140.
16. The State of Queensland.Concrete Pumping, Code of Practice. Queensland : The State of Queensland, 2012.
17. Bureau of Indian Standards (BIS). IS 8112 : 1989 43 grade ordinary portland cement specifications. New Delhi, India : Bureau of Indian Standards, 2005.
18. IS 3812 : 2003 Pulverized fuel ash - specifications. New Delhi, India : Bureau of Indian Standards, 2003.
19. IS 383 : 1970 Specifications for coarse and fine aggregates from natural sources for concrete. New Delhi, India : Bureau of Indian Standards, 2002.
20. IS 9103: 1999 Concrete admixtures - specifications. New Delhi, India : Bureau of Indian Standards, 1999.

Rivers, ponds, lakes and streams -
They all have different names, but they all contain water.
Just as religions do - They all contain truths.

Why Not Small Dams Attain Assured Success : Experiences of Gujarat

Vivek P. Kapadia

Narmada, Water Resources, Water Supply and Kalpasar Department, Government of Gujarat

1. INTRODUCTION

India being a country with rich history and traditions, water harvesting and water conservation are the domains with opulent inheritance. Indian history has also been associated with deeply rooted social values focusing on the principle that the water is a common resource sharable amongst the stakeholders without any monetary consideration and without any right of encroaching the riparian rights of the populations inhabited in the downstream of any watercourse. Gujarat is a state of India with extreme water scarcity and hence the Government of Gujarat tried to resurrect the traditional systems of water harvesting and got constructed over 0.15 million small dams across rivulets and rivers in last fifteen years. The experience of small dams is thus very rich and the issues with them have been studied and presented in this paper. Some case studies are discussed in brief as reference only but general diagnosis made therefrom is very important.

2. MOVEMENT FOR SMALL DAMS ON LARGE SCALE IN GUJARAT

2.1 Background for Movement

Gujarat is divided in to four regions - South Gujarat which also includes Central Gujarat, North Gujarat, Saurashtra and Kachchh as shown in Fig. 1. South Gujarat is water reach and is also having clayey soil, North Gujarat is water stressed and having alluvial soil, Saurashtra is facing shortage of water and is having a mix of black cotton soil with gravels whereas Kachchh is having sandy soil and a large area of dessert. Coastal length of Gujarat is the highest in India which is 1664 kilometers.

Except South and Central Gujarat there has been scanty water in the entire state. If compared the per capita water availability, it is much less than India's average and from 2001 onwards there is a significant reduction therein which has paused concern. This is because of population growth, development of industries and climate change. Quality of groundwater has also been an issue in Gujarat. Electrical conductivity, Chloride, Nitrate and Fluoride are the parameters on which the quality of groundwater is monitored regularly. When quantity and quality of groundwater were found declining and deteriorating, some interventions were felt necessary. Gujarat started from a very bad scenario and worked among many challenges. Overall scenario of Gujarat i.e. surface water and groundwater is shown in Table 1.

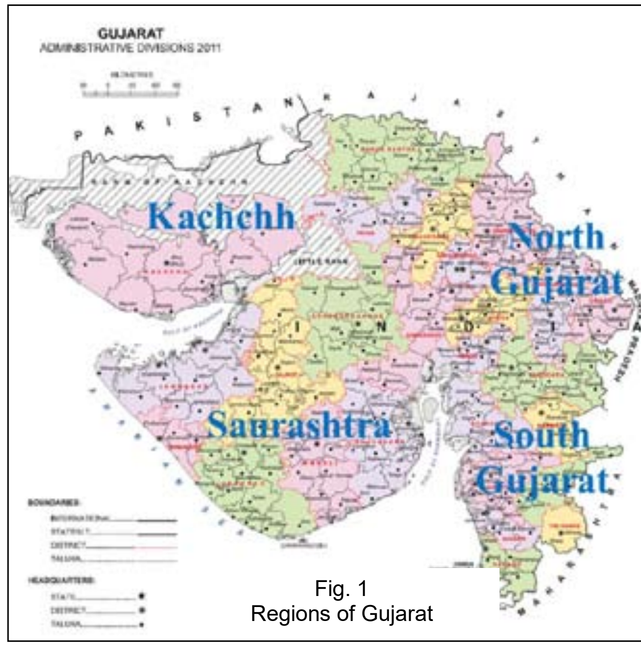


Fig. 1
Regions of Gujarat

Table 1 : Surface water scenario of Gujarat

Region	Area in % of Gujarat	Surface Water Mm^3	Ground Water Mm^3	Total Water Mm^3	Per Capita Availability m^3 Per Annum
South and Central Gujarat	25	31750	3950	35700 (71%)	1695 *(1880)
Saurashtra	33	3600	4300	7900 (16%)	487 *(540)
North Gujarat	20	2100	3300	5400 (11%)	309 *(343)
Kachchh	22	650	450	1100 (2%)	658 *(730)
Total	**100**	**38100**	**12000**	**50100 (100%)**	**893 *(990)**

* Indicates figures based on 2001

Figure 2 shows that in Saurashtra there are many river basins but are small and in Kachchh they are many more and very small. In South and Central Gujarat, a few but big river basins are there. Table 2 shows that rainfall and number of river basins are inversely proportionate and therefore in Saurashtra and Kachchh, rivers are many

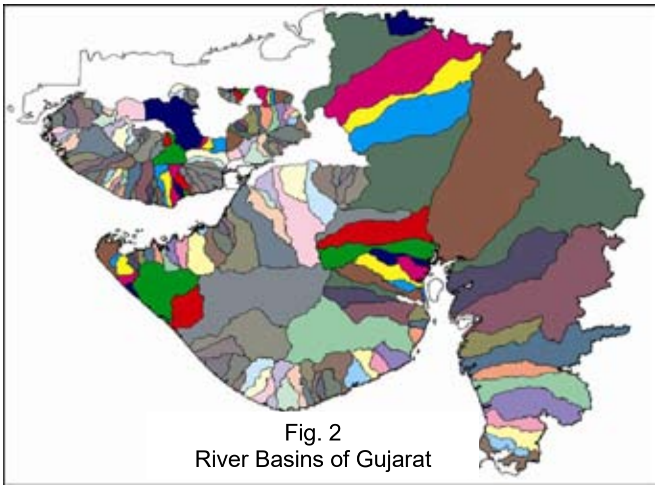


Fig. 2
River Basins of Gujarat

but small and non-perennial. Medium and major dams are feasible on a few rivers because of such a discrepant situation. In total 85 major and medium dams have been constructed as per availability of sites and no more sites are technically feasible for such dams.

Table 2 : Rainfall Distribution of Gujarat

Name of Region	Annual Rainfall in mm	No. of River Basins
North, Central and South Gujarat	800 to 2000	17
Saurashtra	400 to 800	71
Kachchh	Less than 400	97

Technical feasibility became a major challenge for the construction of major and medium dams in Gujarat and hence small dams were the only way-out for water conservation. Small dams in big number in distributed manner were constructed from the year 2000 onwards. Acute water shortage and lack of feasibility of major and medium dams became the main reasons for the said approach. The movement aimed at constructing small dams at many locations in a short period of about 10 years so that protective irrigation could be provided to a large area which was not in the command of medium or major irrigation project. By doing so, minimum one crop i.e. Karif – monsoon crop, could be reliably taken by the farmers in such areas. Government of Gujarat targeted a large area under rainfed agriculture to be covered under protective irrigation.

2.2 Results of Movement

In total more than 1,50,000 small dams were constructed in a short period of about 10 years with the concept of distributed water resource management and immediate benefits remained very encouraging. Summary of a study carried out by the Indian Institute of Management, Ahmedabad underlines some important achievements of the movement as following :

- Average 7 wells in surrounding could be recharged.
- Ground Water Table came up between 3 and 9 meters.
- Soil fertility was improved.
- Quality of drinking water was improved.
- Electricity consumption for drawl of water from ground was reduced.
- Overall prosperity increased in villages.
- The government had to make less expenditure compared to contract approach.
- Instant results were found.
- Change in attitude of people as well as government officers was witnessed.
- Where checkdams (small dams) are constructed, land prices have gone up by 20 %.
- Crop Yield has been improved by 35 %.
- Approximately 10 hectors of land is benefited by each checkdam (small dam).
- 8 to 10 families could get sustenance.
- Control on salinity could be effectively done.

Situation of groundwater could be specifically improved a lot has been recorded by the Government. The basis of study was the number of overexploited and dark blocks varying from 1997 to 2013. Table 3 clearly shows that the demand-supply imbalance got gradually improved in Gujarat. Obviously this was because the surface water storage was improved significantly by constrcuting many small dams which finally resulted in to reduction in demand of groundwater. Over-exploited blocks and critical blcoks decreased with passage of time, saline blocks also got decreased and in contrast the safe blocks got increased. This is the net results of the sensibly planned recharge interventions. The intense efforts from 2001 to 2005 led to this effect in a short period which was really a big change. Controlling worsening groundwater scenario is really a huge challenge and that too, when the area to be addressed is waste. The encouraging results gave a boost to the concept of small dams and they were further constrcuted.

3. RECENTLY SURFACED ISSUES WITH SMALL DAMS

3.1 Structural Vulnerability

Small Dams are not designed as per the standards for major and medium dams due to large number and economic viability. Return period for flood discharge calculations is generally taken as 1 in 25 years which makes it more susceptible to failure because of underestimation of floods while designing.

Table 3 : Enhancement in Groundwater Scenario

Year	Over Exploited Blocks (G.W. Development Above 100%)	Critical Blocks (G.W. Development Between 90% and 100%)	Semi-Critical Blocks (G.W. Development Between 70% and 90%)	Safe Blocks (G.W. Development Below 70%)	Saline Blocks (G.W. T.D.S above 2500 ppm)
1997	35	13	45	122	9
2002	30	13	62	105	14
2007	26	8	51	125	14
2009	27	6	50	127	14
2011	24	5	13	172	10
2013	22	6	9	177	10

Fig. 3 : Bodywall failure – Village - Kundawa, Taluka – Dhanpur, District - Panchmahal

In small dams, scour depth is also not considered with required flood discharge and soil characteristics of foundation and therefore in some cases foundation failure owing to ineffective or short cut-off results in to structural failure. Sometime sufficient energy dissipation is not ensured because of low return period taken in to account for computation of flood. Sometime the bed and banks are firm in semi-mountainous regions but the velocity is very high because of steep gradient which causes high pressure on the bodywall and it fails.

A small dam was constructed at village – Kundawada, Taluka – Dhanpur, Distrcit – Panchmahal in Central Gujarat across a small rivulet 10 meter wide. Height of the dam was 2.0 meter considering the depth of the rivulet and the storage capacity was 0.04 Million Cubic Meter. The cost of the construction was Rs. 0.18 million. Fig. 3 shows the image of this small dam. It was constructed in year 2002. It was across a rivulet in semi-mountainous region and hence the topography was rocky and the bed and banks were firm. In the monsoon of year 2012 the bodywall of the dam collapsed because of transverse flexure and shear due to gushing flow. Because of rocky bed, the bodywall was directly rested up on it and therefore the flexure and shear subjected the bodywall. In semi-mountainous regions, the flow is powerful due to steep gradient which induces high pressure on the bodywall. The vulnerability in such regions is of different type as compared to the dams in alluvial strata.

Fig. 4 : Bodywall failure – Village - Bhatia, Taluka – Talod, District - Sabarkantha

A small dam was constructed near village – Bhatia, Taluka – Talod, District Sabarkantha on River Khari in the year 2004. Its length was 95 meter and height was 2.0 meter. Its cost of construction was Rs. 2.1 million. It was constructed in the form of a concrete box filled with sand which means it was designed as a gravity structure. The design was carried out as a weir on permeable foundation. The river bed was sandy for quite a significant depth. In 2005 there was a devastating flood in this river. As the cut-off was not provided, water flow passed through the foundation and the soil beneath the bodywall got displaced and hence the entire bodywall failed in shear and bending due to its own weight as shown in Fig. 4. Another small dam was constructed near village – Gadhi, Taluka – Prantij, District Sabarkantha on the same river in the year 2004. Its length was 45 meter and height was 2.0 meter. Its cost of construction was Rs. 1.53 million. It was also constructed with same design philosophy. In 2005 it was also destroyed in the same manner as shown in Fig. 5. Its remnants were also dragged away by the flow of water. However, the abutments were intact. In both the cases, foundation failure due to lack of sufficient cut-off was the reason of structural failure. Had the design been carried out for scientifically worked out sour depth, the cut-off would have made the dams much costlier and hence to make them economically viable some technical requirements were ignored which resulted in to failure of several dams like these. Mostly such dams were constructed in a series on a river.

Fig. 5 : Bodywall failure – Village - Gadhi, Taluka – Prantij, District - Sabarkantha

3.2 Vulnerability of River Banks

When a small dam is constructed across a small water course with poor banks and the bodywall of the dam is strong enough, the water either outflanks the banks or erode one of the banks during flood.

A small dam was constructed at village – Saliav, Taluka – Kalol, Distrcit – Mahisagar in Central Gujarat across a small river 75 meter wide. Height of the dam was 2.2 meter considering the depth of the river and the storage capacity was 3 Million Cubic Meter. The cost of the construction was Rs. 5.68 million. It was constructed in 2012-13. The next monsoon resulted a significant flood and the left bank of the rivulet was eroded and the water made its waterway from the left of the dam as shown in Fig. 6. When the height of the bank is low, waterway is not sufficient in the post construction scenario, the banks are not sufficiently firm and the dam is strong, such situations happen.

3.3 Siltation

Channels in alluvial strata are generally flat and carry heavy silt. Small dams across such channels are silted up in a very short period and hence their storage capacity is reduced drastically. In order to maintain the storage capacity of such structures, periodical desilting is required which is not economical.

Fig. 6 : Bank erosion – Village - Saliav, Taluka – Kalol, District - Mahisagar

Fig. 7 : Siltation in reservoir Village – Sureli, Taluka – Kalol, District - Mahisagar

In rivers in alluvial strata, because of the flat gradient at bottom, desilting is required for a significant length and hence is costly. Tendency of river to attain regime also causes siltation when the flow is checked. Siltation depends on the parameters like geography and soil property of catchment area, cross-sectional area of river, longitudinal slope, velocity, silt charge, etc. but has no relationship with the size of the obstruction and hence whether the dam is big or small, quantum of silt settles in the same quantity and hence percentage storage point of view, small dams more susceptible to siltation. Removal of silt leads to very high maintenance cost in small dams constructed on rivulets or rivers passing through alluvial soils.

A small dam was constructed near village – Sureli, Taluka – Kalol, Distrcit – Mahisagar in Central Gujarat on a small river name by Goma having 100 meter width. Height of the dam was 2.0 meter and the storage capacity was 2.50 Million Cubic Meter. It was constructed in 2012-13 with a cost of Rs. 14.4 Million. It was found that its storage capacity was reduced by 80% because of siltation in only 3 years. Fig. 7 is the image of this small dam from its downstream and it suggests that the silt mound on the reservoir side is taller than even the height of the dam at somewhere upstream.

Fig. 8 : Siltation in reservoir Village – Kalol, Taluka – Kalol, District - Mahisagar

A small dam was constructed near village – Kalol, Taluka – Kalol, District – Mahisagar in Central Gujarat on the same river. Height of the dam was 2.0 meter and the storage capacity was 2.75 Million Cubic Meter. It was constructed in 2012-13 with a cost of Rs. 21.8 Million. It was found that its storage capacity was reduced by 82% because of siltation in only 3 years. Fig. 8 is the image of this small dam from its side which clearly suggests that water is there in the stilling basin which the dam received a few days before and the reservoir is almost completely silted up and the silt is also wet. The dam is strong enough to resist the soil pressure with saturated soil on one side with the other side empty. If the dam was not strong to take this much force, it would have failed due to the force of soil pressure that was exerted due to siltation.

4. CONCLUSION

Generally, the small dams are constructed on the sites which are not suitable for medium or major dams. Their benefits are viewed in a specific perspective like recharge, protective irrigation, avoidance of land acquisition, short gestation period, economy, etc. But at the same time some issues are also associated with them which are due to ignoring some crucial technical requirements to make them economically viable. Some issues are because of the scale and independent of the technical parameters. For example, quantum of siltation is irrespective of the size of obstruction but that reduces significantly the storage of a small dam and is a scale related issue. All the issues related to the small dams are required to be paid proper heed at. By doing so, the vulnerability of them would be possible to be ascertained a priori and that is how it would be feasible to either make the small dams viable against the challenges paused by the site situations or to avoid problematic sites so that public funds could be saved. The factor of chance remaining in the design of the small dams is also necessary to be appreciated and controlled for their reliable success level. If some lessons discussed here are learnt with a positive mindset, appropriate site selection would be done to attain the real benefits of the small dams.

Hydraulic Model Studies for Sediment Management by Drawdown Flushing of Run-of-the-River Hydroelectric Project

Neena Isaac, P.S. Kunjeer, S.A. Kamble and (Mrs) V.V. Bhosekar
Central Water and Power Research Station, Pune

ABSTRACT

Reservoirs across the world are losing storage capacity due to sediment deposition. Construction of new dams to compensate for the capacity loss is not feasible since most of the ideal sites have already been developed. An alternative is to develop projects to sustain its life by sediment management. The concept of sustainable development is gaining popularity and many of the recent hydroelectric projects are designed and operated on this concept. The prime design criteria of such projects are to provide efficient sediment management technique. Hydroelectric projects are being developed as run-of-the-river schemes. Drawdown flushing is the most efficient sediment management technique in such projects and is being practiced widely for sediment management. Conditions for effective flushing are mainly natural and hence design of such projects is site specific.

Most of the hydroelectric projects in the Himalayan region are recently being developed as run-of-the-river schemes. The projects are designed by providing large size dual purpose sluice spillways with crest very near to the riverbed for passing flood and sediment during flushing/sluicing. The alignment, size and crest/ invert levels of various components; viz., spillways and intakes are governed by the sediment deposition levels along the reservoir reach. Hence, estimation/ prediction of sedimentation pattern in the reservoir, efficiency of flushing and distribution of sediment deposits after flushing is essential for optimizing the design of various hydraulic structures of the project. It is also required to optimize the reservoir operation schedule. Hydraulic model simulations are the generally accepted tool for such investigations, where the governing parameters are mostly site specific.

Prediction / estimation of the sediment deposition pattern in the reservoir is the prerequisite for predicting the efficiency of hydraulic flushing. Numerical model simulations can be used to predict the sediment deposition pattern in the reservoir. The long-term sedimentation pattern in narrow and elongated reservoirs of run-of-the-river projects can be predicted using one dimensional (1D) numerical model.

In the present study, simulations conducted using hydraulic model for sediment removal by drawdown flushing of the reservoir of the Tangon Limb of the run-of-the-river Etalin hydropower project in Arunachal Pradesh, India is presented. Three alternate sedimentation profiles computed using 1D numerical model simulations were studied on a 1:70 scale geometrically similar model. To optimize the flushing schedule, simulations were carried out with flushing discharges of 200, 500 and 750 m^3/s for 36 h duration. Flushing simulations were carried out with three alternative sediment deposition profiles: viz., Profile I in which the delta front of sediment deposition reaches near the intake, Profile II where the five year deposition profile joined to the spillway crest and Profile III with ten year profile joined to the spillway crest. Studies indicated that 1D numerical models can be used to predict the long term sediment deposition pattern in narrow and elongated reservoirs of run-of-the-river projects. Simulations of drawdown flushing on physical scale model indicated that in all the three cases of sedimentation profiles, the reservoir bed levels stabilize after a flushing duration of about 36 h. However, the quantity of sediment flushed increases with the flushing discharge. It was observed that about 0.83 Mm^3 of the deposited sediment can be removed with the flushing discharge of 750 m^3/s in case of Profile I. Similarly, 0.66 Mm^3 of sediment can be flushed with the flushing discharge of 500 m^3/s. It was observed that for profile II, about 0.54 Mm^3 of the deposited sediment can be removed with the flushing discharge of 750 m^3/s. The quantity of sediment flushed with the flushing discharge of 500 m^3/s was 0.37 Mm^3. For profile III, the quantity of sediment flushed was almost same as for profile II. Flushing with the discharge of 750 m^3/s may be carried out for a duration of about 36 hours for effective sediment removal. If the discharge of 750 m^3/s does not occur in a year, flushing may be carried out with a lower discharge of about 500 m^3/s for a longer duration. Flushing with lower discharge of 200 m^3/s is not effective. Since the flushing channel is developing along the right bank from dam to about 600 m length of reservoir upstream, the intake remains clear of sediment deposition for all the flushing conditions.

1. INTRODUCTION

Reservoirs across the world are losing their storage capacity due to sediment deposition at much faster rates than that estimated during the planning and design stage. The average capacity loss of worldwide reservoirs is estimated to be 1-2% per year. The average capacity loss rate of reservoirs is more than 2% in China (Morris and Fan[1]; Basson[2]; Schleiss et.al.[3]). Studies reported by Basson[2] and Schleiss et.al.[3]) indicated that 80% of the useful storage capacity of hydropower reservoirs will be lost by the year 2035. The situation is of great concern since suitable dam sites for developing new reservoir projects to compensate the lost capacity are very limited and/or non-existent. The solution to the problem is to implement sustainable development of water resources projects by adopting sediment management techniques (Schleiss et al.[3]; Isaac & Eldho[4]). Alternative techniques are being effectively adopted for sediment management of reservoirs and details are reported (Morris and Fan[1]; Schleiss et al.[3]; Lai and Shen[7]; Shen[8]; Yoon[9]; White[10]; Yang[11]; Annandale[12,13]. The sediment management techniques of reservoirs can be grouped into three categories: (i) reducing the sediment reaching the reservoir by catchment area treatment or diverting sediment concentrated flows; (ii) passing sediment-laden flows through the reservoir by sluicing and thereby reducing the settlement of sediment in the reservoir; and (iii) removing the already deposited sediment hydraulically by drawdown flushing or mechanically by dredging[5,8,9,10,11].

A combination of sediment sluicing by operating the reservoir at Minimum Draw Down Level (MDDL) during high flow season and drawdown flushing during annual peak flow are widely being practised for sediment removal from small and medium reservoirs and run-of-the-river projects. Sediment sluicing is to route the incoming sediment load through the reservoir without allowing it to get deposited. During drawdown flushing, the reservoir water level is lowered sufficiently to create riverine flow condition and already deposited sediment can be removed by retrogressive erosion[1,7,8,9,10,11,and14]. However, the effect of flushing in restoring the reservoir capacity depends on many factors: hydrological, topographical, technical and operational conditions. The study by Atkinson[15], of 50 reservoirs for which flushing of sediment are being practised indicated that the favourable conditions for effective flushing operations are mainly natural. Topographical conditions are that reservoirs should be elongated and narrow. In wide reservoirs, a flushing channel will be developed and retrogressive erosion will occur in the flushing channel only. The hydrological conditions require that sufficient water should be available for flushing and capacity to inflow ratio should be small such that reservoir can be refilled. Technical requirement is that sluicing outlets having sufficient capacity to effect drawdown of reservoir should be provided. Operational conditions should permit lowering of reservoir water level sufficiently to achieve riverine flow conditions.

In India, huge potential for hydropower development exists in the Himalayan region, due to the topographical and hydrological conditions of the rivers. Due to the steep slope of the rivers, high head is available in short distances and since rivers are perennial, water for power generation is also available. However, due to the fragile geology and steep slope of the rivers and valley, the rivers carry heavy sediment load during monsoon. The main challenge in development of hydropower projects on these rivers is designing and operating the projects with effective sediment management technique. Many projects are nowadays proposed in this region as run-of-the-river schemes with sediment management through drawdown flushing. Run-of-the-river schemes are designed with low storage and provided with large low level sluice outlets for passage of the flood and sediment during flushing events. Due to the typical site conditions, standard design procedures are not effective in such projects. The design of various components; viz., spillways, energy dissipating arrangements, intakes and water conductor systems are to be evolved through hydraulic model simulations[4,5,6]. Hydraulic design of the above components is highly dependent on the sediment deposition profile in the reservoir. Hence, prediction/ estimation of quantity of sediment deposition in the reservoir and the quantity of sediment removed by flushing and its distribution in the reservoir are essential during the planning and design stage of the projects. Simulations using hydraulic models are generally used during the planning stage of such projects to finalize the design[4,5,6]. In this study, the details of the simulation of hydraulic flushing of the reservoir of a run-of-the-river power project using physical scale model is presented.

2. METHODOLOGY

Most of the hydroelectric projects in the Himalayan region are recently being developed as run-of-the-river schemes. The projects are designed by providing large size dual purpose sluice spillways with crest very near to the riverbed for passing flood and sediment during flushing/sluicing. The alignment, size and crest/invert levels of various components; viz., spillways and intakes are governed by the sediment deposition levels along the reservoir reach. Hence, prediction / estimation of sedimentation pattern in the reservoir, efficiency of flushing and distribution of sediment deposits after flushing is essential for optimizing the design of various hydraulic structures of the project. It is also required to optimize the reservoir operation schedule. Hydraulic model simulations are the generally accepted tool for such investigations, where the governing parameters are mostly site specific.

Prediction / estimation of the sediment deposition pattern in the reservoir is the prerequisite for predicting the efficiency of hydraulic flushing. Numerical model simulations can be used to predict the sediment deposition pattern in the reservoir. The long-term sedimentation pattern in narrow and elongated reservoirs of run-of-the-river projects can be predicted using one dimensional (1D) numerical model[4,5,6,and16]. In the preset case, 1D numerical model is used to estimate/predict the sediment deposition profile in the reservoir.

Most of the available 1D numerical models are not capable of simulating the highly turbulent flow conditions during the flushing. Moreover, 2D or 3D numerical models are required to predict the variation of sediment deposition levels along the depth and width of reservoir, particularly near intakes. Due to the limitations in modelling the highly complex flow phenomena during flushing, application of 2D and 3D numerical models for simulating sediment flushing is not yet been widely practised. Hence, physical scale models are generally used to simulate the reservoir flushing.

3. CASE STUDY OF HYDRAULIC MODEL FOR RESERVOIR FLUSHING

In this study, experiments were conducted on a (scaled) model to simulate the flushing of sediment from the reservoir of the Tangon Limb of the run-of-the-river Etalin hydropower project in Arunachal Pradesh, India (Fig.1)[17].

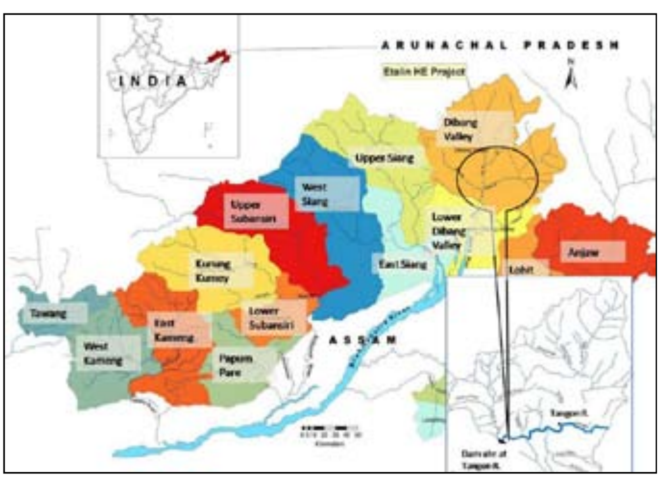


Fig.1 : Location of the study area: Tangon river in the Dibang valley, Arunachal Pradesh, India

3.1 Characteristics of the Project

The Etalin hydroelectric project (3097 MW) is the largest hydroelectric project presently being developed in India in Upper Dibang Valley district of Arunachal Pradesh, India[17]. The project consists of two reservoirs for hydropower generation; one on Dri limb and another on Tangon limb of Dibang river. The Dri and Tangon rivers are tributaries of Dibang river and meet near Etalin village. Downstream of the confluence, the river is called Dibang, which forms a major tributary of the Brahmaputra, contributing about 8.5% of its flows. The project site is about 300 km from Tinsukia, the nearest railhead. The nearest airport is at Dibrugarh, about 350 km from the project site via Dhola/Sadiya Ghat.

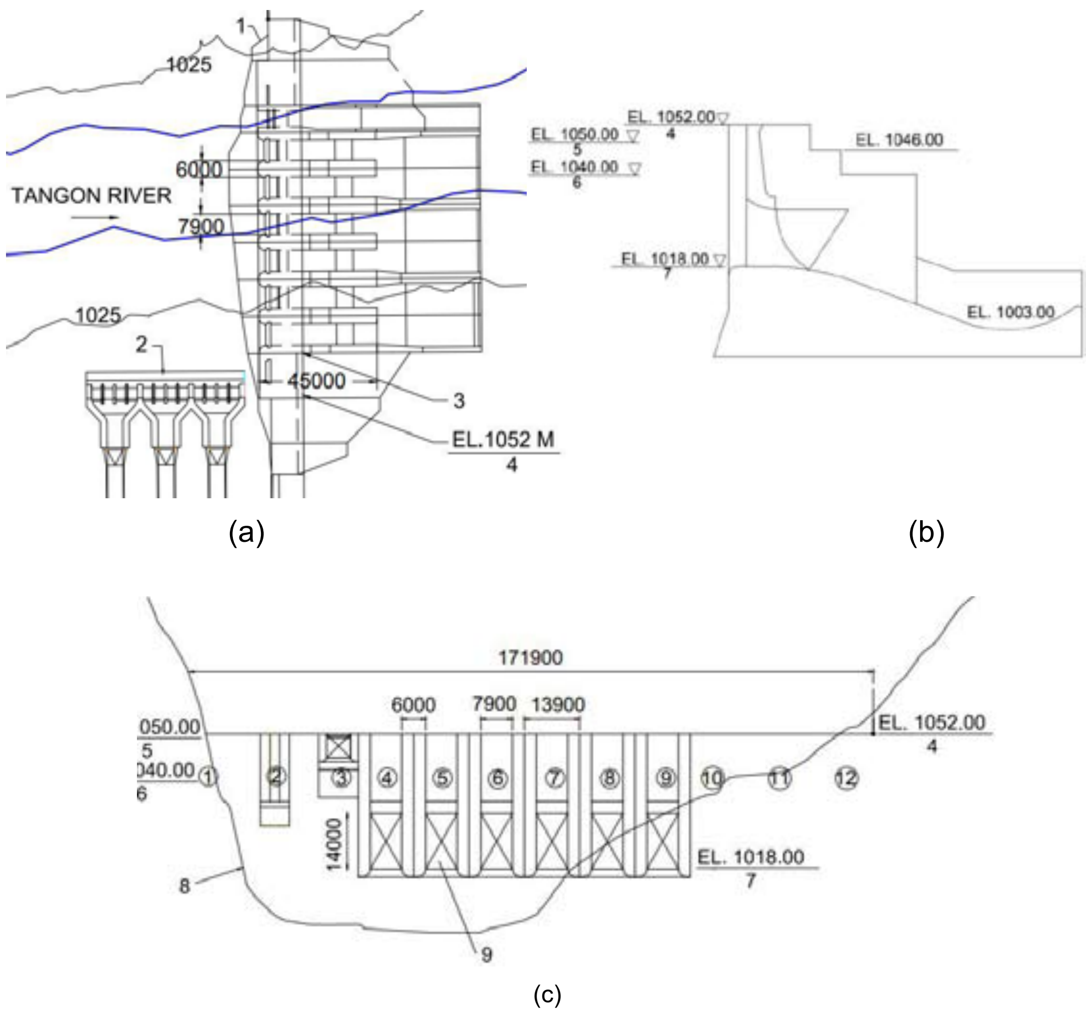
Tangon river originates near Kaya pass in southern flanks of the Himalayas close to the Tibetan border at an altitude higher than El. 5000 m. The river flows in western direction before joining river Dri near village Etalin. The catchment area of the project upto the proposed dam site on Tangon limb is 2573 km^2. The Probable Maximum Flood (PMF) is 10,218 m^3/s and Glacial Lake Outburst Flood (GLOF) is 2,143 m^3/s. The generation capacity of the project at installed capacity is 1228 MW (4 x 307 MW).

The dam site for Tangon limb is located at about 17 km from Etalin village along Etalin-Maliney road, near Avonli village. The proposed dam is an 80 m high concrete gravity structure with top at El. 1052 m and deepest foundation level at El. 972 m; the existing average riverbed level at the dam axis is El. 1002 m. The dam consists of 12 blocks comprising of six main spillway blocks, one auxiliary spillway block with log bay, and six non-overflow monoliths (three each on right bank and left bank respectively). Total length of dam at top is about 171.90 m. The main spillway is of under sluice type with opening size of 7.9 m width (W) and 14 m height (H), and 6.0 m wide piers. The size of log bay spillway is 6.0 m (W) x 6.0 m (H). The crest level of the main spillway is at El. 1018 m while that of the log bay is at El.1046 m. The Full Reservoir Level (FRL) and MDDL of the reservoir are at El. 1050 m and El. 1040 m, respectively. The live storage capacity of the reservoir is 2.94 Mm^3 for diurnal peaking.

The plan, upstream elevation of dam and spillway section are presented in Fig. 2. The power intake is provided at 98° to the dam axis. The invert of intake is at El. 1027.50 m. The design discharge of the power intake is 438 m^3/s.

3.2 1D Numerical Model for Reservoir Sedimentation

1D numerical models have been successfully applied to predict the long term sedimentation pattern in reservoirs. Details and examples of few such models are available in literature; Morris and Fan[1], Annandale[12], Batuca and Jordan[14] and Sloff[18]. Application of 1D numerical models for prediction of long term deposition pattern in reservoirs is reported by Schleiss et al.[3]; Isaac & Eldho[4]; Ahn and Yang[16]; Castillo et al.[19,20] and USBR[21]. The models solve the one dimensional momentum and continuity equations



1	Dam	6	MDDL
2	Intake	7	Crest
3	Dam axis	8	River bed level
4	Dam top	9	Spillway 6 Nos, opening size 7900 (W) x 14000 (H) each
5	FRL/MWL		

All dimensions are in mm and levels are in m.

Fig. 2 : Structural details of Tangon H.E. project; (a) plan, (b) spillway section, (c) upstream elevation of dam

of water flow (St. Venant equations) and continuity of sediment in the river-reservoir system.

In the present study, one-dimensional numerical model was developed using HEC-RAS (USACE[22]) software to estimate probable sedimentation profile in the reservoir of Tangon Limb, Etalin project. The river reach from dam axis to 2700 m upstream and 300 m downstream of the dam was modelled. The topography/bathymetry of the study area was represented in the model by the river plan and cross sections. The sediment rating curve developed from the suspended sediment concentration observed at the gauging site for a period of about 4 years and adjusted for unmeasured load and bed load was used as upstream sediment boundary and the daily discharge hydrograph was used for the upstream discharge boundary. The 1D numerical model was calibrated for hydrodynamic conditions by adjusting the roughness coefficient; viz., the Manning's 'n'. Steady flow computations for observed discharges and corresponding water levels in the prototype were carried out to arrive at the value of 0.048 for the roughness coefficient. Simulations were carried out with reservoir operation level FRL (El. 1050 m) at dam axis to predict the sedimentation levels. Three alternative sedimentation profiles were computed for defining the initial deposition levels in flushing simulations (Fig. 3):

Profile I: The extreme sedimentation profile

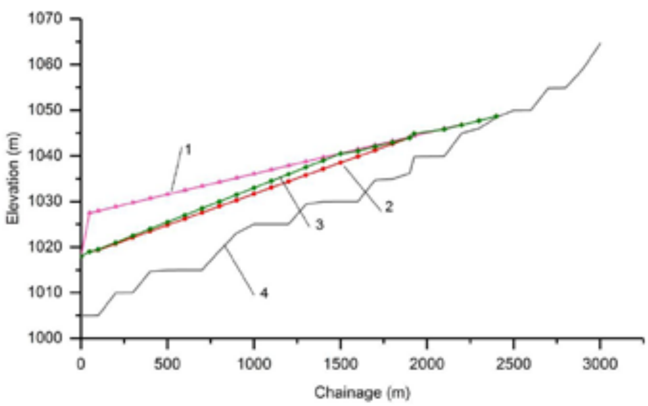
- The profile I, viz., the extreme sedimentation profile is the deposition pattern in which the sediment deposition near intake reaches the invert level of intake. The profile was selected as an extreme case of deposition when the reservoir is operated without flushing.

Profile II: 5 year profile joined to spillway crest

- The profile II is the deposition pattern in which the reservoir is operated without flushing for initial five year period and then joined to the spillway crest.

Profile III: 10 year profile joined to spillway crest

- The profile III is the deposition pattern in which the reservoir is operated without flushing for initial ten year period and then joined to the spillway crest.



1 Profile I

2 Profile II

3 Profile III

4 River bed

Fig. 3 : Sedimentation profile - I, II, III

3.3 Physical Scale Model for Reservoir Flushing

A 1:70 Geometrically Similar (GS) scale model of Tangon river was constructed for flushing simulations. The reservoir reach of 2700 m upstream and 300 m downstream was constructed. The dimensions and hydraulic parameters of model and prototype were related by the Froude's law of similitude. The dam with spillways, gates and intake structure as per original design were reproduced in the model. Arrangements had been provided for measurement of inflow discharge (measured at the upstream end of model using standing wave flume), discharge through intake (measured by V-notch), water levels (pointer gauge) and bed levels (measured along each cross-section using pointer gauge). A trap chamber was provided to collect the sediment flushed out.

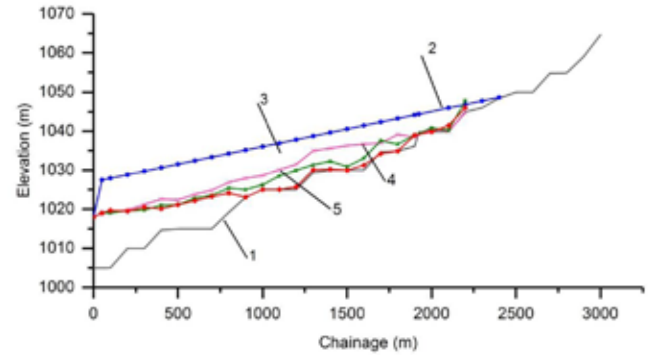
Drawdown flushing was simulated in the model with the three deposition profiles computed using the 1D numerical model as initial level. For each case, the model was filled with silt according to the sedimentation profile. The average sediment size (d50) of the bed material in Tangon reservoir is 10 mm. Various criteria for simulation of sediment transport such as Shield's criteria and Yang's incipient motion criteria were considered. Simulation of bed material in the model was done using locally available sand having d50 of 0.23 mm.

Flushing of reservoir was simulated in the model for different flushing discharges and durations. The discharges and durations corresponding to prototype discharges of 200, 500 and 750 m^3/s and durations of 12, 24, and 36 h were studied in the model. The reservoir water level was kept at FRL at the start of the experiment. All the spillway gates were opened fully to draw down the water level and to achieve free flow condition. Flushing was simulated for the specified duration and at the end of experiments, the volume of sediment collected in the trap chamber and downstream of the spillways was measured. The bed levels at each cross-section were also measured and volumetric computations made to estimate the quantity of sediment flushed from the reservoir. The same procedure of experimentation was followed for each set of experiments. Figure 4 shows the reservoir model with the sediment filled according to profile I.

Fig. 4 : View of model showing sedimentation in reservoir as per profile-I

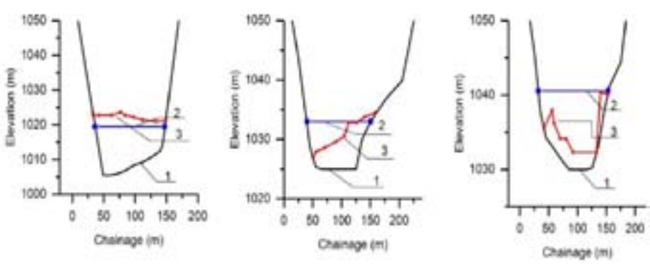
3.3.1 *Flushing Studies with Sedimentation Profile I*

Experiments were carried out with the extreme sedimentation profile i.e. Profile I. The Profile I is the sedimentation profile when the deposition reaches the intake invert level. Reservoir flushing was simulated in the model with discharges corresponding to prototype discharge of 200, 500 and 750 m^3/s and duration of 36 h. The total quantity of sediment flushed through the spillway was measured separately for each flushing discharge. The longitudinal section of the deepest bed levels of Tangon reservoir model after flushing for 36 h duration are given in Fig. 5 and representative cross sections for the flushing discharge of 750 m^3/s are shown in Fig. 6. Figure 7 presents the views of the model after flushing for 36 h with the flushing discharge of 750 m^3/s.



1 River Bed
2 Profile I Filling
3 Profile I, Q = 200 m^3/s, T = 36 h
4 Profile I, Q = 500 m^3/s, T = 36 h
5 Profile I, Q = 750 m^3/s, T = 36 h

Fig. 5 : Longitudinal sedimentation profile after flushing

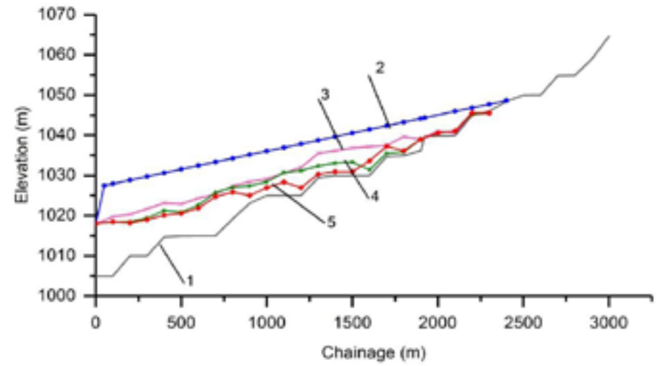


1 River Bed
2 Profile I Filling
3 Profile I, Q = 750 m^3/s, T = 36 h

Fig. 6 : Cross sections @ 100 m U/s, @ 1000 m U/s, @ 1500 m U/s after flushing (Profile-I, Q = 750 m^3/s, T = 36 h)

3.3.2 *Flushing Studies with Sedimentation Profile II*

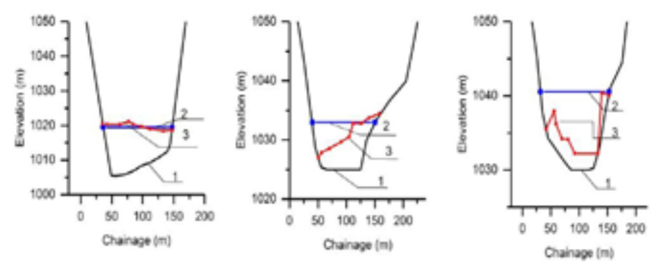
Experiments were further carried out with the Profile II. The Profile II is the deposition pattern in which the reservoir is operated without flushing for initial five year period and then joined to the spillway crest. Reservoir flushing was simulated in the model with discharges corresponding to prototype discharges of 200, 500 and 750 m^3/s and duration of 36 h. The total quantity of sediment flushed through the spillway was measured separately for each flushing discharge. The longitudinal section of the deepest bed levels of Tangon reservoir model after flushing for 36 h duration are given in Fig. 8.



1 River Bed
2 Profile II Filling
3 Profile II, Q = 200 m^3/s, T = 36 h
4 Profile II, Q = 500 m^3/s, T = 36 h
5 Profile II, Q = 750 m^3/s, T = 36 h

Fig. 8 : Longitudinal sedimentation profile after flushing

Representative cross sections for the flushing discharge of 750 m^3/s are shown in Fig. 9. Figure 10 presents the views of the model after flushing for 36 h with the flushing discharge of 750 m^3/s.



1 River Bed
2 Profile II Filling
3 Profile II, Q = 750 m^3/s, T = 36 h

Fig. 9 : Cross sections @ 100 m U/s, @ 1000 m U/s, @ 1500 m U/s after flushing (Profile-II, Q = 750 m^3/s, T = 36 h)

(a) (b) (c)

Fig. 7 : View of the model after flushing (a) Between dam axis and 200 m U/s, (b) Between dam axis and 700 m U/s, (c) Between 1200 m U/s and 1600 m U/s (Profile-I, Q = 750 m^3/s, T = 36 h)

(a) (b) (c)

Fig. 10 : View of the model after flushing (a) Between dam axis and 200 m U/s, (b) Between dam axis and 700 m U/s, (c) Between 1200 m U/s and 1600 m U/s (Profile-II, Q = 750 m³/s, T = 36 h)

3.3.3 *Flushing Studies with Sedimentation Profile III*

Experiments were continued with the Profile III. The Profile III is the deposition pattern in which the reservoir is operated without flushing for initial ten year period and then joined to the spillway crest. Reservoir flushing was simulated in the model with discharges corresponding to prototype discharges of 200, 500 and 750 m³/s and duration of 36 h. The total quantity of sediment flushed through the spillway was measured separately for each flushing discharge. The longitudinal section of the deepest bed levels of Tangon reservoir model after flushing for 36 h duration are given in Fig. 11 and representative cross sections for the flushing discharge of 750 m³/s are shown in Fig. 12. Figure 13 presents the views of the model after flushing for 36 h with the flushing discharge of 750 m³/s.

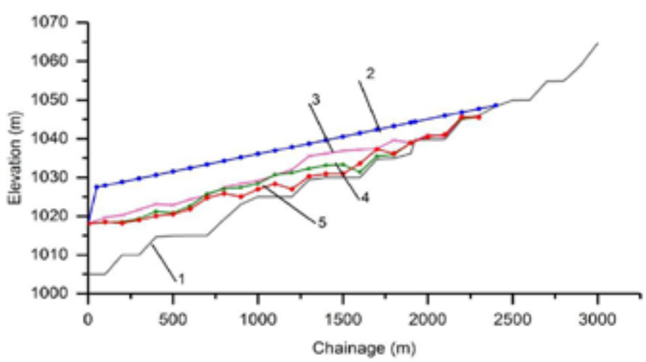
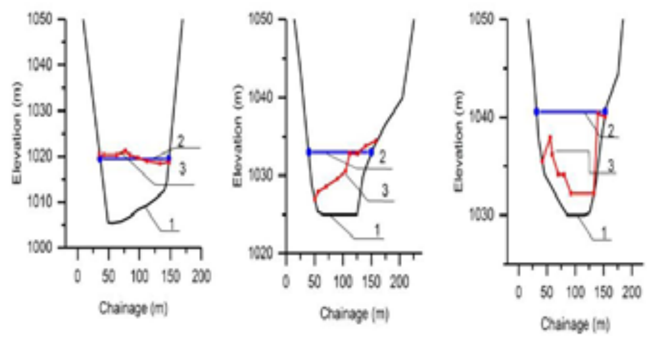


Fig. 11 : Longitudinal sedimentation profile after flushing

1 River Bed
2 Profile III Filling
3 Profile III, Q = 200 m³/s, T = 36 h
4 Profile III, Q = 500 m³/s, T = 36 h
5 Profile III, Q = 750 m³/s, T = 36 h



1 River Bed
2 Profile III Filling
3 Profile III, Q = 750 m³/s, T = 36 h

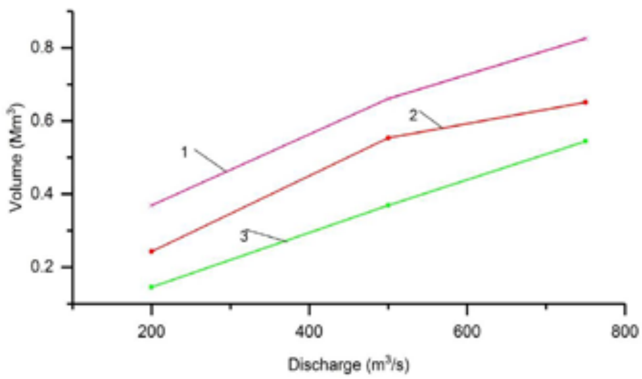
Fig. 12 : Cross sections @ 100 m U/s, @ 1000 m U/s, @ 1500 m U/s after flushing (Profile-III, Q = 750 m3/s, T = 36 h)

3.3.4 *Quantity of Sediment Flushed*

The quantity of sediment flushed in 36 h duration with different flushing discharges for alternative sedimentation profiles is given in Table 1 and Fig.14.

(a) (b) (c)

Fig. 13 : View of the model after flushing (a) Between dam axis and 200 m U/s, (b) Between dam axis and 700 m U/s, (c) Between 1200 m U/s and 1600 m U/s (Profile-III, Q = 750 m3/s, T = 36 h)



1 Profile I

2 Profile II

3 Profile III

Fig. 14 : Quantity of sediment flushed in 36 h with different discharges

Table 1 : Quantity of Sediment Flushed in 36 h with Different Flushing Discharges

Discharge (m^3/s)	Quantity of Sediment Flushed (Mm^3)		
	Profile I	Profile II	Profile III
200	0.37	0.17	0.24
500	0.66	0.37	0.55
750	0.83	0.54	0.65

3.4 Discussion on Results of Experiments

Experiments with the three alternative deposition profiles indicated that flow conditions and channel formation are similar for all the cases studied. It was observed during the experiments that, as soon as the reservoir water level was lowered for draw down flushing, very rapid movement of silt towards downstream occurred over the entire reach of reservoir.

In the experiments with Profile I, during the initial period of flushing with the discharge of 750 m^3/s, the flow was bankfull and all the spillway bays were active till the free flow condition was achieved. During the above phase, scouring occurred in front of all the spillway bays and intake. As the flushing operation continued, a flushing channel was formed along the right bank for a distance of about 600 m upstream from dam axis. The deep channel shifted towards the left bank in the reach between 700 to 1300 m upstream of dam axis. The sediment deposition from the reservoir reach of 1400 m to the end of reservoir was completely removed during flushing. Similar flow conditions and channel formations were observed for all the discharges studied on the model. The quantity of sediment flushed was about 0.37, 0.66, and 0.83 Mm^3 for flushing discharges of 200, 500 and 750 m^3/s. It was observed that the intake remains clear of sediment deposition after flushing operation.

Experiments with Profile II also indicated that the flow is bankfull and all the spillway bays were active during the initial period of flushing with the discharge of 750 m^3/s. As the flushing operation continued, the channel was formed along the right bank for a distance of about 700 m upstream from dam axis. The deep channel was along the left bank in the reach between 900 to 1300 m upstream of the dam axis. Similar flow conditions and channel formations were observed for all the discharges studied on the model. The quantity of sediment flushed was about 0.17, 0.37 and 0.54 Mm^3 for flushing discharges of 200, 500 and 750 m^3/s. The area in front of the intake along right bank remained clear of sediment deposition for all the discharges.

The flow conditions and flushing channel formations in case of Profile III were similar to case of flushing with Profile II. The flushing channel was along the right bank for a distance of about 700 m upstream from dam axis. There was marginal increase in the quantity of sediment flushed as compared to Profile II. The quantity of sediment flushed was about 0.24, 0.55 and 0.65 Mm^3 for flushing discharges of 200, 500 and 750 m^3/s. At the end of flushing, similar to the case of Profile II, the area in front of the intake along right bank remained clear of sediment deposition for all the discharges.

Experiments indicated that the quantity of sediment flushed is more for Profile I since the delta front of sediment deposition has already reached the dam axis. There is marginal increase in the quantity of sediment flushed for Profile III as compared to Profile II. The quantity of sediment flushed increases with the increase in the flushing discharge. Flushing stabilizes after duration of 36 h and there is not much increase in the quantity of sediment removed when flushing is continued for longer duration beyond 36 h.

4. CONCLUSIONS

Hydraulic model simulations were carried out for reservoir flushing of Tangon Limb reservoir of Etalin hydroelectric project in Arunachal Pradesh, India. To optimize the flushing schedule, simulations were carried out with flushing discharges of 200, 500 and 750 m^3/s for 36 h duration. Flushing simulations were carried out with three alternative sediment deposition profiles: viz., Profile I in which the delta front of sediment deposition reaches near the intake, Profile II where the five year deposition profile joined to the spillway crest and Profile III with ten year profile joined to the spillway crest. Following are the important conclusions from the studies:

- Simulations of drawdown flushing on physical scale model indicated that in all the three cases of sedimentation profiles, the quantity of sediment flushed increases with the flushing discharge. However, the reservoir bed levels stabilize after a flushing duration of about 36 h.

- It was observed that about 0.83 Mm^3 of the deposited sediment can be removed with the flushing discharge of 750 m^3/s in case of Profile I. Similarly, 0.66 Mm^3 of sediment can be flushed with the flushing discharge of 500 m^3/s.
- It was observed that for profile II, about 0.54 Mm^3 of the deposited sediment can be removed with the flushing discharge of 750 m^3/s. The quantity of sediment flushed with the flushing discharge of 500 m^3/s was 0.37 Mm^3. For profile III, the quantity of sediment flushed was almost same as for profile II.
- Flushing with the discharge of 750 m^3/s may be carried out for a duration of about 36 h for effective sediment removal. If the discharge of 750 m^3/s does not occur in a year, flushing may be carried out with a lower discharge of about 500 m^3/s for a longer duration. Flushing with lower discharge of 200 m^3/s is not effective.
- Since the flushing channel is developing along the right bank from dam to about 600 m length of reservoir upstream, the intake remains clear of sediment deposition for all the flushing conditions.

Acknowledgements

Authors are thankful to the Director, Central Water and Power Research Station, Pune for giving permission to publish the paper. Support and input given by Officers from WAPCOS India Ltd., Jindal Power Limited (JPL) and SNC-LAVALIN Engineering India Ltd. are thankfully acknowledged. Authors also express thanks to officers and staff members of Sediment Management division and Hydraulic Analysis and Prototype Testing division of CWPRS for their co-operation in constructing the model, conducting the studies and preparation of the manuscript.

REFERENCES

1. MORRIS, G. L., JIAHUA. FAN. Reservoir Sedimentation Hand book. McGraw-Hill Book Co., New York. 1997
2. BASSON, G. “Reservoir sedimentation – An overview of global sedimentation Rates, sediment yield & sediment Deposition prediction”. International Workshop on Erosion, Transport and Deposition of Sediment, Berne, 28 - 30 April 2008.
3. SCHLEISS, ANTON, J., GIOVANNI, DE CESARE, FRANCA, MARIO, J., PFISTER, MICHAEL (ED.). Reservoir Sedimentation. CRC Press, Taylor & Francis Group, London, UK, 2014
4. ISAAC, NEENA, AND ELDHO, T. I. "Sediment management studies of a run-of-the-river hydroelectric project using numerical and physical model simulations". International Journal of River Basin Management, 14:2, 165-175, DOI: 10.1080/15715124.2015. 1105234, 2016.
5. ISAAC, NEENA, AND ELDHO, T. I. "Sediment management studies of run-of-river hydroelectric power project in the Himalayan region using hydraulic model studies". Sādhanā (2017), doi: 10.1007/s12046-017-0666-0, 2017.
6. ISAAC, N., ELDHO, T. I., AND GUPTA, I. D. "Numerical and physical model studies for hydraulic flushing of sediment from Chamera-II reservoir, Himachal Pradesh, India." ISH Journal of Hydraulic Engineering, 20:1, 14-23, DOI: 10.1080/09715010.2013. 821788, 2014.
7. LAI, J. S., AND SHEN, H. W. "Flushing sediment through reservoirs". Journal of Hydraulic Research, 34: 2, 237-255, 1996.
8. SHEN, H. W. "Flushing sediment through reservoirs". Journal of Hydraulic Research, 37: 6,743 — 757, 1999.
9. YOON, Y.N. "The State and the Perspective of the Direct Sediment Removal Methods from Reservoirs." International Journal of Sediment Research, 7(20), 99–115, 1992.
10. WHITE, R. "Flushing of Sediment from Reservoirs." HR Wallingford, UK, 2000.
11. YANG XIAOQING. Manual on Sediment Management and Measurement. WMO-No. 948, Operational Hydrology Report No. 47, 2003.
12. ANNANDALE, G. W. Reservoir sedimentation. Elsevier Science, Amsterdam, Netherlands, 1987.
13. ANNANDALE, G. W. "Going full circle." International Water Power Dam Construction, April, pp. 30–34, 2011.
14. BATUCA, D.G., JORDAN, J. M. Silting and Desilting of Reservoirs. A.A. Balkema, Rotterdam, The Netherlands, 2000.
15. ATKINSON, E. "Feasibility of flushing sediment from reservoir." Report OD137 HR Wallingford, 1996.
16. AHN, J., YANG, C. T. “Simulation of Xiaolangdi Reservoir Sedimentation and Flushing Processes.” 2nd Joint Federal Interagency Conference, Las Vegas, NV, June 27 - July 1, 2010.
17. CWPRS. Hydraulic model studies for flushing of sediments from reservoir of Tangon Limb, Etalin hydro-electric project, Arunachal Pradesh. Technical Report no. 5429, CWPRS, Pune, India, 2016.
18. SLOFF, C.J. Reservoir sedimentation; a literature survey. Communications on hydraulic and geotechnical engineering, No. 1991-02, TU Delft, The Netherlands, 1991.
19. CASTILLO, L. G., CARRILLO, J. M., AND ÁLVAREZ, M. A. "Simulation of the flushing into the dam - reservoir Paute - Cardenillo". Reservoir sedimentation- Schleiss et al. (Eds), Taylor & Francis Group, London, ISBN 978-1-138-02675-9, 2014.
20. CASTILLO, L. G., CARRILLO, J. M., AND ÁLVAREZ, M. A. "Complementary Methods for Determining the Sedimentation and Flushing in a Reservoir". Journal of Hydraulic Engineering, Volume 141, Issue 11, 2015.
21. USBR. Erosion and Sedimentation Manual. U.S. Department of the Interior Bureau of Reclamation November 2006.
22. USACE. HEC-RAS River Analysis System –Hydraulic Reference Manual and User’s Manual. U. S. Army Corps of Engineers, Hydrologic Engineering Center, 690 Second Street, Davis, CA 95616, 2010.

Highlights from ICOLD 2018, Vienna

The ICOLD week in Vienna combined Committee meetings, a technical Symposium, various Workshops, meetings of the Regional Clubs, and the General Assembly, as well as a number of networking opportunities during social events and excursions. This year the triennial Congress also took place, with four issues being covered in depth, during parallel sessions.

ATCOLD President Dr Gerald Zenz welcomed delegates to the two-day Symposium on Hydro Engineering. In his opening remarks, he referred to the very significant role of dams and hydropower in his country, noting that Austria's portfolio of dams represented great diversity, in terms of dam types and purposes. The large-scale implementation of hydropower, he said, had begun around 1949, and had continued since then. He made special reference to the important role of pumped-storage plants.

ICOLD President Prof Anton Schleiss, in his welcome message, noted that at least 1500 international participants were present for the Annual Meeting and Congress, representing a large proportion of ICOLD's 100 member countries.

Dr Harald Kainz, and Dr Gunther Rabensteiner, giving their opening addresses

Secretary-General Michel de Vivo expresses appreciation to EDF for its long-term support for ICOLD, as the Commission celebrates 90 years.

ATCOLD President Gerald Zenz welcomes more than 1500 delegates to ICOLD 2018 in Vienna, during the opening of the Symposium.

He commended the host country, Austria, for its achievements in the field of hydropower and dams, noting that there were 171 large dams in operation, of which four were higher than 150 m. He also observed that hydropower was supplying more than 60 per cent of national electricity.

Referring to the environmental benefits of hydro, and the large amount of unexploited hydro potential which remained worldwide, he noted that theoretically hydropower would be capable of replacing most coalfired generation. He referred to hydro as the "champion of renewable energy", and reported that ICOLD was playing a leading role in an EU initiative, which was a 'Roadmap for Hydropower in Europe'.

ICOLD President, Prof Anton Schleiss, gives his opening address at the Symposium.

Speakers in the opening ceremony of the Symposium cut a ribbon to mark the beginning of ICOLD 2018.

Mr. Gilles Feuillade of EDF, France, mentions that ICOLD's 27th Congress will take place in Marseille in 2021.

An opening speech was then given by Dr Harald Kainz, Rector of the Technical University of Graz (cohost of the meeting in Vienna). He told delegates that TU Graz was one of the three leading centres of excellence in Austria; it was also the oldest university of technology, and had the highest expenditure on research. He underlined the importance of collaboration with industry, and with other scientific institutions, both nationally and internationally.

Dr Günther Rabensteiner, Member of the Board of Verbund, spoke next. He described hydropower as the "sustainable backbone" of the country's power system, supplying two-thirds of electricity (around 10,000 MW). About one-third of Austria's hydro was at storage schemes, including pumped storage, Rabensteiner said, and the remainder at run-of-river plants. In total Austria was operating around 3000 hydro facilities, he added. He predicted very favourable prospects for hydropower, describing it as the "green battery for the future".

ICOLD Secretary-General Mr. Michel de Vivo drew attention to the fact that ICOLD celebrates its 90th Anniversary this year, and he took the opportunity to express his thanks to the French utility EDF, for supporting the Commission since its inception (providing, over the years, the headquarters, financial support, the secondment of the general-secretaries, and the provision of many experts). He introduced as a guest speaker Mr. Gilles Feuillade, Director of Hydro Production and Industry at EDF, who expressed his pleasure on behalf of his company for the collaboration in supporting ICOLD, and he commended the work of the Commission.

The following sections feature some selected highlights from the many parallel technical sessions of the Symposium.

TECHNICAL SESSIONS

Stability of Reservoir Slopes

In this session, chaired by Sophie Messerkinger (Consulting Engineer), Dr Martin Wieland (Pøyry, Switzerland) reported seismotectonic features at the 156 m-high Rudbar Lorestan earth core rockfill dam, located in a narrow canyon in the seismically active Zagros mountain range in Iran. The project had been designed against multiple seismic hazards including ground shaking. "The only dam types suitable for such sites are conservatively designed earth core rockfill dams", Wieland said. The main concerns in the dam design were the discontinuities formed by joints, bedding planes, fissures and faults although no active faults existed in the dam foundation, which could produce large earthquakes, he said. In concluding, Wieland proposed that additional

Dr Martin Wieland (Switzerland) spoke about seismic hazards at the 156 m high Rudbar Lorestan scheme in Iran.

Ms. Dwi Kubontubuh (Indonesia) discussed landslide stability at the Jatigede dam in Indonesia and the schedule for reservoir impounding.

Mr. Feng Jin (China) spoke about innovative technologies at rehabilitation projects in China.

Mr. Des Hartford (Canada) reported on different and improved approaches for assessment of potential dam failure.

studies are required for the release of floods through the ungated spillway and for the operation of the bottom outlets to minimize any erosion damage and future maintenance works.

Mr. Mengxi Wu (Institute of Mechanics, Chinese Academy of Sciences, China) drew attention to the hazards caused by landslides from reservoir slopes during periods of water storage and reservoir operation, which could lead to significant casualties and loss of property. “A large-scale, high-speed landslide may cause high wave surges that may endanger the safety of a water power project”, he said. Therefore, the stability of those slopes is an especially important aspect in the design of a hydropower station.

Mr. Wu presented research findings on both the sliding mode and factors of safety along a circle arc and the combined rigid body slip surfaces at various water levels. The strength reduction method and the improved limit equilibrium method based on finite element stress calculation were reported. Mr. Wu pointed out that a rise in reservoir water level could influence the sliding mode and safety factor.

Mr. Gary Power (Reinforced Earth Company, Australia) presented case study experiences of using mechanically stabilized earth to upgrade exiting dams. The approach was described by Power as representing a unique, composite construction material with high strength and stability, a limited footprint and the ability to distribute loads uniformly, even on poor foundation soils.

Mechanically stabilized earth, Power said, has been used since 1973, soon after development of the technology in the late 1960s. Since this time, the technology had evolved to meet the site conditions imposed by geotechnical and hydraulic structures. One development had been the use of geosynthetic materials as an additional reinforcement option to be used at hydraulic-based structures, he said.

Mr. Dwi Kubontubuh (Ministry of Public Works and Housing, Indonesia) discussed landslide stability at the Jatigede dam in Indonesia. Completed in 2016, the 114 m-high dam is located on the island of Java. The main purposes of the scheme are to provide 90,000 ha with irrigation, generate electricity from the two 55 MW hydro units, and to help control flooding in a 14,000 ha area around the Cimanuk river. Mr. Kubontubuh noted that recent visual inspection and instrumentation readings had shown a diminishing sliding magnitude that was consistent with rainfall intensity. She strongly advised that initial impounding be carried out gradually with a temporary halt at el. 221 m to have a better understanding of the correlation between water level and land sliding.

The proposed sequential countermeasures, namely bored piles, counterweight construction and drainage system, could be implemented in parallel with or after the initial impounding, Mr. Kubontubuh added.

Permission and Safety Assessment

Mr. Edwin Staudacher (Graz University of Technology, Austria) chaired the session on permission and safety assessment at dams. The first speaker, Mr. Muyiwa Alalade (Bauhaus-University, Germany), discussed identification of full wave-based damage in dams to help identify the deterioration in structural integrity. Alalade proposed the application of numerical methods and inverse analysis. He reported that the research had been inspired by the successful application of full wave form inversion in geotechnical exploration and non-destructive testing.

Mr. Alalade observed that the proposed method was capable of identifying most damage. “The resolution of damage was higher”, Mr. Alalade added, “when sensors are placed closer to regions of (expected) critical damage”.

Mr. Feng Jin (Tsinghua University, China) proposed that for many rehabilitation projects, underwater concrete technology could be adopted without reservoir drawdown. “Self-protected underwater concrete (SPUC)

Mr. Alexander Arch (Pøyry, Switzerland) spoke about the risk of landslides and reservoir bursts in Peru.

Mr. Nikolaos Efthymiou (Fichtner, Germany) spoke on the applicaiton of sedimentation management software.

is an innovative concrete technology for underwater construction invented by Tsinghua University and Sinoconfix company in China", Mr. Jin said. Before concrete pouring, a small amount of protective agent is added into the water body. He presented practical engineering applications including the rehabilitation of a gravity dam, an underwater concrete poured foundation and an underwater pier. These successful applications, Jin said, demonstrated the strong applicability of SPUC technology.

Mr. Sergio Oliveira (LNEC, Portugal) stressed the importance of acquiring robust knowledge regarding the dynamic behaviour of dams. "Furthermore", he added, "this must be continuously updated and improved, relying on the combined use of advanced vibration monitoring systems and efficient model identification software."

Reporting a case study from the Cabril scheme in Portugal, Oliveira said that the excellent agreement achieved between experimental and numerical results demonstrated the value of the dam's monitoring system and associated software. "It also helps to demonstrate the importance of vibration monitoring systems (and the associated hardware and software) in the scope of dam safety control, as well as the potential of their combined use with numerical models of dam-reservoir-foundation systems," he added.

Mr. Des Hartford (BC Hydro, Canada) reported on monitoring for the prevention of dam failure within the design envelope. Monitoring and surveillance had provided the first line of defence in dam safety management practices for many years, he said. Historically, monitoring and surveillance had focused on determining if, by observation, measurement and data analysis, the observed structural functioning of a dam, appurtenant structures and foundations conform to the design expectation under the conditions expected normally to occur in the life of the structure. Mr. Hartford proposed that the approach could be enhanced by including, by data records and back analysis, an assessment of the design basis and intent, the expected performance of the dam and its structural vulnerabilities, and its operational modes, expectations and functional vulnerabilities. Further enhancement of the management system, he continued, could be achieved by performing analysis of functional modes of operation giving consideration to associated failure modes.

"Incredible accidents arise from combinations of credible failure modes of essential components", Mr. Hartford said. In this way, he continued, "the credible accident can have exceedingly low or even insignificant probability".

Mr. Benno Zund (Pøyry, Switzerland) reported on an estimation of scour at the spillway plunge pool for the Nam Thuen 1 scheme in Lao PDR. The scheme comprises a 180 m-high RCC gravity dam. "The flood release structures including the plunge pool at the dam toe need to be designed to withstand the high dynamic loads", Mr. Zund said, "and this is one of the major hydraulic design issues".

Comparing a range of pressure coefficients for the proposed plunge pool at Nam Theun 1 with those available in the literature, Mr. Zund reported that the

Mr. Young Jin Hong (South Korea) spoke about the environmental and social measures taken at the Gulpur project in Pakistan.

proposed pre-excavated plunge pool would be sufficiently deep not to be eroded in an extensive area deeper than the proposed invert level of 105 m.

"The calculations also show that the spillway jets for all range of floods will be fully developed before impacting the plunge pool surface which will reduce the erosive power of the plunging jets", he said. "Provision of the dents on flip buckets, which are not embedded in the disintegration length relationships, will further improve the jets disintegration and aeration", Mr. Zund added.

Climate Changes Reservoir Operation

Mr. Martin Fuchs (Pøyry Energy, Switzerland) led this session. His colleague, Mr. Alexander Arch (Pøyry, Switzerland), spoke on the topic of adapted operations at tropical glacier reservoirs as a result of climate change. "In the past, there have been a series of catastrophic events in the Cordillera Blanca generated by mudflows coming from glacier reservoirs. This hazard, together with an increasing mean temperature, and a glacier loss of 46 per cent between 1930 and 2016 in tropical glacier regions, highlight the necessity to understand the risks of operating reservoirs in Peru", he said.

Mr. Arch reported on the potential risk of 18 ice blocks falling into a reservoir with a volume of 29 ¥ 10^6 m^3 and a maximum depth of 90 m. The modelling analysis supported action to reduce the water level in the lagoon by between 2 m and 40 m to prevent overtopping at the dam. However, Mr. Arch noted that prevention of overtopping by reducing the water level would become ineffective when impact momentum reaches 45 GNs.

"To assure future safe operation of these reservoirs, as well as the security of people living downstream, permanent observation of the glacier by satellite and direct photography might help enable fast reaction in case of an event", he concluded.

Mr. Dissanayake Mudiyanselage (Irrigation Department, Sri Lanka) reported a study on soil erosion at the upper basin of the Mahaweli river under climate change. Increased rainfall intensity resulting from climate change has the potential to exacerbate soil erosion in reservoir catchments causing rapid reservoir sedimentation. Mahaweli is the longest river in the country and the basin consists of steep slopes conducive to significant soil erosion, where 39.9 per cent of the area consists of slopes of more than 30 per cent.

A soil water assessment tool, a continuous simulation hydrological model, had been used to analyze soil erosion above Victoria reservoir in the Mahaweli basin. Mr. Mudiyanselage reported that the main inputs to the model had been a digital elevation model, land-use map, soil map, rainfall and temperature. Unavailable climatic data including solar radiation, wind speed and relative

humidity were generated by the model itself, he noted. "Annual sediment yields in the area were estimated for the period from 1960 to 2016, with the highest annual sediment yield in the basin being 1,022,717 ton in 2013. The average annual yield between 1960 and 2016 was 49 ton/km^2," he said.

Time series analysis of annual sediment yields denoted a strong upward trend at all four major reservoirs in the basin. At sub basin level, average sediment yields indicated a considerable spatial variation. Therefore, Mr. Mudiyanselage concluded, "appropriate river basin management is essential for retarding soil erosion".

Francesco Galante (Dragflow, Italy) proposed technological opportunities to reduce reservoir sedimentation. Recent studies had, he said, estimated global gross storage capacity at 6,000 km^3 and annual reservoir sedimentation rates at 31 km^3. "Accordingly, global reservoir storage capacity will be reduced by 50 per cent by the year 2100".

Climate change has increased the frequency of flash floods, therefore sedimentation in artificial reservoirs is one of the most important challenges of both public and private Authorities responsible for reservoir management.

He proposed a number of possible solutions, including bypassing sediment to the downstream area, discharging high flows through the dam during periods of high inflows, drawdown flushing, turbidity current venting, and dredging with specialized equipment.

Mr. Galante presented experiences of dredging at the Ambiesta reservoir in the Italian Alps. The design, supply, installation, testing and management of the dredging system had conformed to very stringent environmental constraints established by the Tagliamento river authority, he said.

Mr. Nikolaos Efthymiou (Fichtner, Germany) presented on the rapid screening of sediment management techniques for the Moragolla scheme in Sri Lanka using RESCON-2. The Moragolla project will have an installed capacity of 30.2 MW and will be the latest scheme in the upper Mahaweli complex, a cascade of five reservoirs providing hydropower generation and supply of irrigation water.

Mr. Efthymiou said the purpose of the study had been to help in the selection of the optimum, technically feasible and environmentally friendly sediment management technique. The analysis had been performed, he said, using RESCON-2 software that had been recently developed for the World Bank Group. “The application of the software allowed a rapid assessment of the state-of-the-art sediment management techniques, including sediment inflow reduction through implementation of a check dam upstream of the reservoir, flushing, dredging, hydrosuction removal, sluicing, by-pass and density current venting,” Mr. Efthymiou said.

The RESCON-2 results had compared with the results of a widely used one-dimensional numerical model and very good overall agreement had been obtained. “Therefore, the model can be recommended for further use in similar applications”, he concluded.

Mr. Young Jin Hong (Mira Power, South Korea) reported on the sustainable development of the 102 MW Gulpur hydro project in Pakistan. The original environmental social impact assessment for the scheme had been put on hold as a result of the Poonch river being designated a National Park. The implications of this included the requirements to minimize the impact of resettlement to

128 households, achieve ‘betterment of the park’ and to meet with the requirements of net environmental gain as specified by ADB and IFC.

To achieve these conditions, the project design had been changed and various studies had been undertaken relating to critical habitats, integrated ecological flows and biodiversity action plans demonstrating the net environmental gain.

Mr. Hong reported that the changed design had helped to reduce resettlement to almost zero from 128 households. In conclusion, he said that the lenders had conditionally approved the revised reports including the ESIA.

Technical Committee Updates

ICOLD's 28 Technical Committees held meetings, and in some cases Workshops, during ICOLD 2018. Three examples to illustrate the activities of the committees are given below (cases where either a member of the H&D team attended the meeting, or where reports were presented by Chairmen during the General Assembly).

Committee on Climate Change

Mr. Denis Aelbrecht (EDF, France) chaired this committee meeting and began the proceedings by reviewing guidelines for the operation of ICOLD technical committees. He then focused on the work of the Climate Change Committee and reminded members that the work of this committee has been structured into three complementary themes:

- *Theme 1* : Climate-induced water shortage and drought management.
- *Theme 2* : Flood risk evolution associated to climate change.
- *Theme 3* : Assessing the role of hydropower in climate change mitigation and the future energy mix.

Mr. Aelbrecht said, "it is not yet decided how the structure of the next bulletin(s) will be. The basis of the work is to start with case studies that could clearly illustrate each theme, and derive generic guidelines based on these actual examples". The work and discussions that followed in Vienna focused on Theme 1, while at the same time, committee members are continuing to finalize the translation of bulletin 169. Members then individually reported the latest events and trends from their own countries in relation to climate change impacts on water resources. Mr. Osborne N. Shela presented the challenges of recent droughts in Malawi. In 2017 and previous years, he said the country's hydraulic infrastructure had been needed to regulate Lake Malawi resources. The need for this and the associated risks, he said, were being exacerbated by climate change. One development that he felt would help was the Kholombizo dam project, currently under consideration in the country.

Mr. Gerald De Jager (South Africa) shared an update on the management of the multi-year drought event in the Cape Town region, South Africa, especially during the period of 2017 to early 2018. The so-called 'Day-Zero' event was widely publicized internationally. Mr. Jager reported a study for balancing supply and usage for every basin in South Africa, using a probabilistic approach. The national study could, Aelbrecht proposed, be used as a potential case study for Theme 1.

Mr. Junichi Tsutsui (Japan) presented a study undertaken by JCOLD about Phase 3 work focused on the use of existing dams as climate change mitigation counter-measures. He reported a case study of the Hiyoshi dam, where new operational procedures had been implemented to limit downstream flood risks. The Hiyoshi dam case could be used as a potential case study for Theme 2, Aelbrecht noted.

Mr. Kristoffer Hallberg (Sweden) shared updates on climate change related activities in Sweden. He reported the ambitious domestic target of having 100 per cent renewable generation by 2040. Hallberg added that Skellefteåkraft, SWECO and the Boston Consulting Group had concluded that the 100 per cent renewable target was possible through several different pathways. Also, he said, "hydropower will be even more important in a new energy mix". A new project had been launched to evaluate the implication of a new energy mix on dam safety conditions. The work should be completed by mid-2019, he said. This new project could be used as a potential case study for Theme 3, Mr. Aelbrecht commented.

Mr. Markus Moeller (Germany) gave a presentation on the potential impacts of climate change on the Neustadt water resources system, where a local water company had recently requested to increase water withdrawal from the reservoir, a project completed in 1905. Mr. Moeller reported a study that had analysed changes at the dam's inflow gauging station, two near by stream gauges, two met stations, three rain gauges and a phenological station. The investigation had revealed significant changes in the flow regime. The observed changes in the flow regime, Moeller said, could not be explained by precipitation data at the site, as this showed neither any significant trends nor any breakpoints. This implies a change in the partitioning of precipitation into runoff, evapotranspiration and groundwater recharge. "Going forward", he said "it is expected that future trends and sudden shifts of meteorological and hydrological parameters will potentially increase the demand and expectations of society on dams and their buffering role in the hydrological cycle". However, some of these functions and purposes were contrary to each other, hence dam management would potentially become more challenging, he noted.

The case for Neustadt could be used as a potential case study for Theme 1, Mr. Aelbrecht said. He then gave a short talk about the close connection between this ICOLD committee and the work of the World Bank. The World Bank with the support of Mott-Mc Donald, he said, had just completed its first version of climate resilience guidelines for the hydropower sector. ICOLD committee members (Aelbrecht and Annandale) had helped to review these guidelines and ensured recommendations from ICOLD Bulletin 169 were incorporated when relevant.

Aelbrecht reported that the guidelines were to be tested against real cases, and a panel of experts had been assigned by EBRD to help review the testing phase and to help simplify World Bank resilience assessment procedures. This work is expected to be complete by mid-2019.

ICOLD Hon. President Dr Jia Jinsheng gives an address at the inaugural ceremony of the new Committee on Resettlement.

Mr. Wang Xijiong, Chairman of the new Committee on Resettlement.

New Committee on Resettlement

A new Technical Committee on Resettlement due to Reservoirs was launched in Vienna. It is chaired by Mr. Wang Xijiong, Vice Director of the Relocation and Resettlement Bureau of the China Three Gorges Corporation. An inaugural ceremony was held on the morning of 2 July, at which Chairman Wang welcomed delegates and set out the mission of the Committee. Short speeches were then given by Prof Anton Schleiss, Dr Gerald Zenz, Dr Jia Jinsheng, Michel de Vivo and Mr. Alison Bartle.

The overall mission of the Committee is to exchange worldwide experience on methods, theories, achievements, research, training and capacity building in the field of land acquisition, compensation and other mitigation measures in the case of resettlement for large and medium-scale water resources projects. One of the first iniatives will be to collect best practice case studies, and to reflect on successful and less successful resettlement schemes. It is proposed to produce guidelines based on this work, and also to increase the focus on this subject in the academic world. The first major planned output from the Committee will be a Bulletin, which will review the current status of research in the field of resettlement, and present a summary of case studies.

Issues relating to various stages of project development will be covered, from initial coordination of resettlement with a reservoir project, and with local society, as well as the planning and design of resettlement, including compensation policies, then implementation and management of resettlement, and finally care for those in the resettlement areas, including the host residents. Public consultation and stakeholder engagement will be major areas of concern.

The Committee already has members representing the USA, Canada, Brazil, Colombia, Argentina, UK, France, Switzerland, Russia, Turkey, Portugal, Iran, Laos, India, Pakistan, Australia, Ghana, and South Africa.

Following the inaugurations, the Committee hosted its first Workshop, in which presentations were given on resettlement schemes in China, Brazil, Colombia and Japan.

Prospective and New Challenges for Dams

Mr. Luc Deroo of France, Chairman of ICOLD's Ad Hoc Committee on Prospective and New Challenges for Dams and Reservoirs during the 21st Century, reported that his Committee was currently preparing three Bulletins, and "scouting two additional fields of interest".

The first Bulletin, he said, would be entitled 'Trends and solutions for dams and reservoirs in the 21st Century', and he felt it would pave the way for ICOLD's mission to encourage 'Better dams for a better world'. The second topic was that of low hazard dams, highlighting their important multipurpose roles. Deroo said that a future Bulletin would look at optimizing costs and enhancing performance of these dams.

The third topic related to the innovative use of reservoirs, and would look at a wide range of new approaches to the use of reservoirs, Examples would include: off-stream reservoirs; empty reservoirs constructed for flood routing; underground reservoirs; lagoons out at sea; lakes for solar-hydro or windhydro hybrid developments; and, reservoirs built to encourage biodiversity.

The new topics under consideration were: dams and reservoirs in arid and semi-arid areas; and, lifecycle costs assessment (including tools to compare costs).

Members of ICOLD Technical Committee on Climate Change. The Chairman, Mr. Denis Aelbrecht, is shown on the far left

First Reports from ICOLD's 86th Annual Meeting

Representatives of 66 of ICOLD's 100 Member Countries were present for the 86th General Assembly in Vienna on 3 July. President Mr. Anton Schleiss welcomed delegates, and Secretary-General Mr. Michel de Vivo drew attention to the ever-increasing membership of ICOLD, reporting that Kazhakstan had recently joined, and representatives of the new committee planned to attend next year. Meanwhile applications were pending from Uganda, Haiti, Laos, Ecuador and Congo Brazzaville.

Prof Anton Schleiss welcomes delegates to the General Assembly, and right, Mr. Michael Rogers, newly elected as ICOLD's next President.

ELECTION OF OFFICERS

Mr. Michael F. Rogers of the USA was elected and warmly welcomed as President for the term 2018-2021. He had been nominated by the Argentian Committee on Dams, and Mr. Alejandro Pujol spoke in the meeting to support the nomination. Delegates heard that Mr. Michael Rogers has been a civil engineer working on large dams throughout his entire career, starting at Harza Engineering Company. Today he holds the position of Vice Present and Global Practice Leader for Dams at Stantec Consulting Services.

The two newly elected Vice Presidents, Michael Abebe of Ethiopia, and right, Dr Ali Noorzad of Iran.

Mr. Michael Rogers, Pujol continued, is a practitioner who is well recognized as an international expert in dam safety and roller-compacted concrete (RCC) dams. He has worked on more than 200 dam projects around the world and published more than 50 technical papers. Most recently, he was the Lead RCC Engineer for the Oroville Emergency Recovery - Spillway work in the USA following the major incident at the gated and emergency spillways in February 2017.

Mr. Alejandro Pujol drew attention to the fact that Mr. Michael Rogers had been active in ICOLD for 12 years, having attended meetings since 2006; he was organizer of the Annual Meeting in Seattle in 2013, and currently serves as Chairman of the Technical Committee on Concrete Dams. During his acceptance speech, President-elect Mr. Rogers spoke of ICOLD as an organization which peacefully brings together nations as one family; he also paid tribute to the work of the current President, Prof. Anton Schleiss.

His ideas for ICOLD during his term of office are summarized in the article on p32, which is partly based

Mr. Alejandro Pujol of Argentina, who spoke in support of the election of Michael Rogers.

Votes are collected for the election of ICOLD officers.

From left: Ms. Johanne Bibeau, Canada, giving an update on next year's Annual Meeting in Ottawa; Dr George Dounias, Greece, announcing the 11th European Club Meeting to take place in Crete next October; and, Mr. Jean- Jacques Fry, France, with Mr. N. Matsumoto, Japan, giving details of their collaboration on the subject of dynamic analysis.

on his inaugural speech at the closing ceremony of the Vienna Congress.

Mr. Michael Abebe of Ethiopia was elected Vice President for the Africa Zone, taking over from Ahmed Chraibi of Morocco, who had completed his term of office. In his inaugural speech, Mr. Michael Abebe spoke of representing a continent which lagged behind the rest of the world in terms of water infrastructure in operation, but he drew attention to the ambitious plans. He referred to the UN Sustainable Development Goals, with their emphasis on water management, and he also paid tribute to the PIDA initiative, led by the African Union, UNECA and the African Development Bank, which had set strong targets for water storage facilities and hydropower.

During his term of office, he aimed to contribute to capacity building, and regional collaboration. Dr Ali Noorzad of Iran was elected Vice President for the 6th Post, in place of Mr. Michael Rogers, whose term as Vice President had been completed. Dr Noorzad had been nominated by the Swiss National Committee. In supporting the nomination, Mr. Laurent Mouvet, President of the Swiss Committee, referred to Dr Noorzad's constant valuable input to ICOLD, since he had first met him when they had worked together on the Surveillance Committee in 1999. He recalled that Dr Noorzad had been teaching at some of the most prestigious universities of Iran. He had then been awarded his PhD in Canada, specializing in geotechnical engineering and seismic design of dams, and was now well known as an expert in this field in his country. Dr Noorzad has been responsible for the design and construction of more than 100 large dams, and has served as both Secretary-General and President of the Iranian Geotechnical Society.

Vice President Mr. Zhou Jianping reports on the activities of the Asia-Pacific Regional Club.

FUTURE MEETINGS

87th Annual Meeting

Mr. Johanne Bibeau, a Director of the Canadian Dam Association, gave an update on arrangements for the 87th Annual Meeting, to take place at the Shaw Centre in Ottawa, Canada, from 9 to 14 June next year. She presented a video showing some of the attractions of Canada's capital city, and summarized some details of the planned programme. She explained that CDA would be offering some training courses as pre-meeting events, with a particular focus on safety. Technical Workshops were also planned, with topics including: working with indigenous communities on hydropower development; climate change and adaptation; tailings dam technologies; reservoir triggered seismicity; and, modern approaches to the regulation of dams.

The Symposium, for which abstracts are invited by 15 September, will be on the theme 'Sustainable and safe dams around the world', and will include sessions on: innovation, including recent advances in techniques for investigation, design, construction, operation and maintenance; sustainable development in planning, design, construction and operation; mitigation and management of hazards affecting dams, tailings dams and appurtenant structures; and, extreme conditions, such as permafrost, ice loading, wet climates and geo-hazards.

Johanne Bibeau also highlighted some of the major dam and hydro schemes in Ontario, Québec and British Columbia, which could be visited on technical excursions and post-meeting study tours.

88th Annual Meeting

Mr. D.K. Sharma, President, of the Indian National Committee, welcomed ICOLD members to the 88th ICOLD Annual Meeting, which is to take place in New Delhi, India, in April 2020. He showed a video reflecting India's state-of-the-art conference facilities, rich cultural heritage, and world famous sites to be visited.

11th European Club Symposium

Dr George Dounias of Greece announced plans for ICOLD's 11th European Club Symposium, which will take place from 2 to 4 October 2019 in Crete. The Greek Committee is inviting abstracts to be submitted by 30 November this year. The main topics will be:

- Dam safety and risk management; social impact and awareness
- Dams and reservoirs in a climate change perspective
- Successes and problems in implementing the European Water Frame - work Directive
- Managing infrastructure
- Advances in dam engineering
- The future of dams in a changing EU environment.

AFRICA 2019, Namibia

Mr. Alison Bartle of Aqua~Media presented details of AFRICA 2019 to the General Assembly, outlining plans for the event which will take place in Windhoek from 2 to 4 April 2019. She mentioned that the conference had the full support of the Namibian authorities, notably Nam -Water and NamPower, and she encouraged ICOLD delegates to submit abstracts for the programme. She reported that study tours are planned to the north of the country (to the Ruacana hydro plant, as well as the famous Etosha game park), and also to the south, to Neckertal RCC dam now nearing completion, as well as the Naute, Oanab and Hardup dams.

WORKING GROUPS, CLUBS AND COMMITTEES

Mr. Jean-Jacques Fry of France gave an update on the activities of the European Working Groups, noting that there were currently groups working on:

- Legislation, including updating the national legislations database
- Internal erosion, including research on mechanisms, criteria and feedback from ICOLD Bulletin 164
- Dams and earthquakes, covering dam safety under extreme earthquakes

Mr. Saturo Ueda of the World Bank, outlining the partnership agreement with ICOLD.

- Levees and flood defences.

Mr. Fry noted that a new Working Group was being formed in 2019, to deal with the subject of overtopping. Three areas of study would be: embankment dams and fluvial levees; concrete dams and appurtenant structures; and, sea protection dykes.

Dr Ahmed Chraibi reported on the well attended meeting of the African Regional Club, where many national representatives had reported on activities in their countries. He also referred to the ARC's initiatives on training and capacity building, and said that a side event was planned for the time of the AFRICA 2019 conference in Namibia, being co-hosted by Aqua~Media International and ICOLD.

Vice President Mr. Zhou Jianping of CHINCOLD reported on the successful meeting of the Asia Pacific Regional Club. He first spoke of the great activity in dam construction in the Asia-Pacific region, where more than 100 large dams were under construction each year, with many more planned. He announced details of the 10th East Asia Dam Conference, which will take place in October this year, in Zhengzhou, China, jointly sponsored by CHINCOLD, JCOLD and KNCOLD.

Hon. President Mr. Adama Nombre of Burkina Faso spoke of the importance of capacity building, as chairman of the Committee on that subject. He emphasized that it was a complex and long-term process, and referred to some of ICOLD's ongoing training iniatives, for example organized by the Chinese and Moroccan Committees; he added that there were initiatives arising through ICOLD's regional clubs, and his Committee was ready to support these.

E-book on Dynamic Analysis

A joint presentation by Mr. Jean-Jacques Fry of France and Mr. N. Matsu moto of Japan, informed delegates of a collaborative project undertaken by the French and

Mr. François Lempérière receives his Lifetime Achievement Award from Secretary-General Mr. Michel de Vivo, during the opening of the Congress.

Prof. Anton Schleiss is presented with a medal and certificate, as he is awarded the title of Honorary President at the closing of the Congress.

Japanese National Committees, which had culminated in the publication of an E-book entitled 'Validation of Dynamic Analysis of Dams and their Equipment'.

PARTNERSHIP WITH THE WORLD BANK

A new partnership between ICOLD and the World Bank was announced during the General Assembly. This will enable representatives to attend each others' meetings, share data, and enhance collaboration.

Some of the opportunities for collaboration which were highlighted in particular, concerned:

- dam safety regulations and riskinformed dam safety;
- sustainable operation and maintenance of dams and hydropower;
- climate change resilience;
- sedimentation management;
- dam safety management on international rivers; and,
- capacity building.

Mr. Saturo Ueda, Lead Water Resources Man agement Specialist at the Bank expressed his appreciation at this development, and later in the week, during the Congress, the agreement was officially signed.

LIFETIME ACHIEVEMENT AWARD FOR FRANÇOIS LEMPÉRIÈRE

During the Opening Plenary Session of the Congress, on 5 July, Mr. M. François Lempérière received a special Lifetime Achievement Award, in recognition of his long and distinguished career in the field of dam engineering and construction.

As well as being responsible for the construction of many large dams on some of the world's greatest waterways, his career has been, and continues to be, characterized by innovation. He had submitted four entries for ICOLD's Innovation Awards.

Many of his proposed concepts are featured papers in Hydropower & Dams, most recently in Issue 3, 2018.

YOUNG ENGINEEERS' FORUM (YEF)

The meeting for young engineers followed the symposium sessions and started with a keynote presentation by Mr. Sebastian Perzlmaier (TIWAG, Austria). He spoke about the importance of the dam profession in supplying sustainable infrastructure for the developing and growing population, as well as the exciting opportunities for young engineers to contribute to this development.

Mr. Priska Hiller (YEF Chairperson) then introduced the candidates standing for election to the Board. She reminded participants that the YEF Board should reflect the diversity of ICOLD. "It is preferable that each ICOLD region is represented on the board", she said. YEF Board Members are elected for three years, and there are usually two to three positions open for election each year. The candidates are responsible for checking that his or her National Committee supports their candidature. There can be a maximum of one candidate per National Committee. This year there were two positions open, with Mr. Giulia Buffi from Italy and Tim Ivanov from Russia successfully elected to fill them.

After breaking into working groups to hold discussions on various topics relating to dam development and the operations of the YEF, the attendees enjoyed a relaxed networking evening at a local bar.

PROF SCHLEISS COMPLETES HIS TERM OF OFFICE

During the closing session of the Congress, as President Mr. Anton Schleiss completed his term of office and was awarded the title of Honorary President, there was time to reflect on activities and achievements during his three-year term. These had included:

- establishing guidelines to improve the formation and working practices of the Technical Committees;
- establishing guidelines on the ICOLD Congress process;
- the introduction of innovation awards;
- better dissemination of information, with the help of the publisher Balkema;
- reaching a new record of 100 member countries;
- signing of a contract with the EU for an iniative called ‘Hydropower in Europe’, to be led by ICOLD;
- continuing with improvements to the ICOLD website; and,
- more emphasis on climate change.

In his speech handing over the Presidency, Prof Schleiss recalled that he had said that at the outset he had wanted to serve ICOLD with a full heart. He said that he felt pleasure as well as relief after three years to be leaving ICOLD “in good shape”, and he paid tribute to the Vice Presidents and the team in ICOLD’s Central Office. He wished Michael Rogers all the very best for his term of office.

Reflections and Priorities of ICOLD President, Michael Rogers

“I am honoured to have been elected as the 25th President of the International Commission on Large Dams/ Commission Internationale des Grands Barrages during the 86th ICOLD Annual Meeting in Vienna. In taking these first steps as the new President, I must look back to recognize the wonderful leadership of outgoing President, Prof Anton Schleiss over the last three years. Along with the hard-working Central Office team, ICOLD has made significant strides to strengthen the Commission and I’m excited for this opportunity to lead our world-class organization for the next three years.

Coming home from Vienna with many good memories, I look back on one of the best organized and successful ICOLD annual conferences that I have had the good fortune to attend. I congratulate Prof. Zenz, President of ATCOLD and Chairman of the ICOLD 2018 Organizing Committee, and his entire committee, for this tremendous success.

As I start this next chapter of my career, I cannot help but look further back over the 35 years that I have spent working on dams around the world. Starting as a college intern at Harza Engineering Company, I am fortunate to have grown through my career as the company too has grown, which is now Stantec Consulting, Inc. Dams have been my devotion and obsession throughout my entire career, and this role as ICOLD President is truly a dream come true for me. I am thankful to Stantec for supporting this dream.

So now I look forward to what I regard as key priorities for the next three years, and to working with ICOLD Secretary-General Mr. Michel de Vivo, the Central Office team and my fellow ICOLD officers, to maintain the strength and harmony of our organization, while also working to improve some areas for the benefit of all of our 100 national committees and thousands of individual members.

I have established five priorities for the Board for the next three years:

- strengthen our national committees;
- improve technical committees;
- focus on capacity building;
- expand the relevance of ICOLD to the young engineers; and,
- re-dedicate ICOLD to worldwide safety of dams and levees.

Since becoming active in ICOLD in 2006, I have always heard and felt that ICOLD is a family: a family of nations and a family of individuals, that care and support each other and the ICOLD organization itself.

Many of our ICOLD family are represented in struggling or failing national committees. Some of these committees no longer participate in meetings or pay their share of the ICOLD financial commitments. This puts a larger strain on those countries that do participate, and it also threatens the services that ICOLD provides through the Central

Office. But, more importantly, this deprives those countries and individuals from the benefits of full engagement with ICOLD. I look forward to searching for opportunities to strengthen these struggling national organizations, and bring them back into the ICOLD family in good standing.

Having been a member of ICOLD Committee D (Concrete Dams) for almost 20 years, I understand that there is sometimes frustration between the technical committees and support from the ICOLD parent organization. The Board has started to address this issue with a recent document prepared by ICOLD Vice Presidents Mr. A.F. Chraibi (Moro cco) and Mr. M. Lino (France) that summarizes the ICOLD technical committee organization in the form of a concise set of guidelines. Hon. President Mr. Schleiss initiated an outside publishing process for our bulletins, which will help reduce the extensive backlog of pending publications. I am committed to moving these initiatives forward until the process from an approved bulletin to a published bulletin is less than the term of ICOLD Board members, three years!

In November 2008, ICOLD celebrated its 80th Anniversary, and unveiled a World Declaration for Dams and Hydropower for African Sustainable Development. This recognized the huge needs of the people suffering in developing countries for clean water and power. The Declaration was a plea to the world to help address these needs, which were characterized by one of the powerful concluding statements: “The needs of the African population are now almost in a state of emergency.” All nations of ICOLD should look for benevolent opportunities to support capacity building in Africa and other developing countries around the world. A good example can be taken from the leadership of our Chinese Committee on Large Dams, which regularly supports the participation of ICOLD countries from Africa in Special Roundtable Sessions as part of its annual meetings.

Looking to the future, ICOLD’s young engineers are our leaders for tomorrow, both at home in their own countries, where they learn the art of dam engineering, and also at ICOLD Meetings, where they will be the chairpersons and vice presidents of tomorrow. These young engineers have been coming to ICOLD in ever increasing numbers as the word has spread about the opportunities on technical committees and for integrating with professionals from around the world. However, more than just encouraging their participation with low registration fees and special awards, we must look for opportunities to engage them and make ICOLD more relevant to them. With a strong connection to ICOLD that relates directly to their lives, these young members will stay engaged in ICOLD and, with time, will bring others to our organization until they too become mentors.

ICOLD was founded 90 years, ago as a benevolent organization to share knowledge for the safe design of some of our world’s most expensive and critical infrastructure: large dams. In this our 90th anniversary year, ICOLD must confirm its strong commitment to the safety of dams of all sizes, as well as related infrastructure such as levees. In this past year, I have had a front row seat at one of the largest dam safety incidents in the world, the Oroville spillway. This has strengthened my resolve that ICOLD as an organization must be true to our founding mission: to share technology and commitment to the safety of all dams and levees.

Finally, I am energized at this dawn of new opportunities for me as ICOLD President. I am humbled by the vast energy and intellect of our organization, especially our individual members. I will strive with all of my abilities to be a good President for all members and all countries, and to represent ICOLD proudly around the world, as we continue to strive for ‘Better dams for a better world’”.

CLIMATE CHANGE

POOR COUNTRIES FACE 'UNFAIR PATTERN' OF TEMPERATURE SWINGS

3 May, 2018

Poor countries are expected to see the greatest swings in temperature as a result of climate change, a new study suggests. Using models for the last report of the United Nations Intergovernmental Panel on Climate Change, European researchers analyzed 37 climate-model simulations to calculate which regions would likely experience the higher highs and lowest lows until the year 2100.

http://www.cbc.ca/news/technology/climate-change-temperature-variability-1.4646781

SATELLITES REVEAL HOW QUICKLY PATAGONIAN GLACIERS ARE MELTING

3 May, 2018

A team of ESA scientists used the polar-ice monitoring satellite, CryoSat to determine that some of Patagonia's glaciers are melting away faster than any of the other glaciers in the world. In a paper published in the journal Remote Sensing of Environment, the researchers describe how they used their eye in the sky to detect complex patterns in the changing height of the ice. They then used this data to create a map depicting the rate at which it is melting.

https://www.inverse.com/article/44458-satellite-reveal-glaciers-melting

CLIMATE CHANGE COULD REVERSE ALL REDUCTIONS IN CHILD MORTALITY: STUDY

8 May, 2018

In the past 25 years, the world has made major progress improving child health and reducing child mortality. But all that hard work could be undone by climate change, a study published in Pediatrics warned. The authors say climate change "threatens to reverse the gains in global child health and the reductions in global child mortality made over the past 25 years." While the impacts of climate change could be felt by all humans, the authors say they'll be disproportionally felt by poor people and children.

https://inhabitat.com/climate-change-could-reverse-all-the-reductions-in-child-mortality-made-in-the-last-25-years/

TREE-MURDERING FUNGI AND INSECTS ARE INCREASINGLY CONTRIBUTING TO CLIMATE CHANGE

8 May, 2018

Forests have long been an important resource, and not only in human development — even today, they account for 80 to 90 percent of land-dwelling biodiversity. But forests are currently threatened by deforestation, rising global temperatures, increasing extreme weather events, and intensifying pathogenic outbreaks — caused by widening pest ranges and invasive species. The global impact of this forest die-off is extreme: Carbon that accumulates year after year in living trees escapes from dead and decomposing trees back into the atmosphere.

https://massivesci.com/articles/forest-insects-fires-carbon-warming/

TOURISM IS RESPONSIBLE FOR NEARLY ONE-TENTH OF THE WORLD'S CARBON EMISSIONS

7 May, 2018

Tourism accounts for around 8 per cent of global greenhouse gas emissions, according to a new study that marks the first attempt to quantify the industry's total carbon footprint. The researchers said flying less and investing in payment schemes to offset damage caused by travel will be essential to avoid "unchecked future growth in tourism-related emissions".

https://www.independent.co.uk/environment/tourism-climate-change-carbon-emissions-global-warming-flying-cars-transport-a8338946.html

CLIMATE CHANGE: AN 'EXISTENTIAL THREAT' TO HUMANITY, UN CHIEF WARNS GLOBAL SUMMIT

15 May, 2018

Both leadership and innovation are essential for climate action, Secretary-General António Guterres said in his keynote address to the global gathering, known as the R20 Austrian World Summit – a long-term initiative to help regions, States and cities implement the Sustainable Development Goals and meet the Paris Agreement targets.

https://news.un.org/en/story/2018/05/1009782

EXPERTS MEET TO SCALE UP EFFORTS TO TACKLE CLIMATE CHANGE DISPLACEMENT

15 May, 2018

Stakeholders from all over the world are gathered in Switzerland last week to contribute to the work of the

United Nations Framework Convention on Climate Change (UNFCCC) Task Force on Displacement and to assist in drafting recommendations to avert, minimize and address displacement in the context of climate change.

https://reliefweb.int/report/world/experts-meet-scale-efforts-tackle-climate-change-displacement

CLIMATE CHANGE SPELLS TROUBLE FOR NEW ZEALAND'S CROPS, SAY SCIENTISTS

7 May, 2018

New Zealand's crops could be hit by more - and more serious - disease as climate change brings harsher, longer droughts. A team of New Zealand scientists say an increase in droughts over the coming decade is likely to increase the severity of a wide range of diseases affecting vital primary industries.

http://www.scmp.com/news/asia/australasia/article/2145013/climate-change-spells-trouble-new-zealands-crops-say

CLIMATE CHANGE: AN 'EXISTENTIAL THREAT' TO HUMANITY, UN CHIEF WARNS GLOBAL SUMMIT

15 May, 2018

Both leadership and innovation are essential for climate action, Secretary-General António Guterres said in his keynote address to the global gathering, known as the R20 Austrian World Summit – a long-term initiative to help regions, States and cities implement the Sustainable Development Goals and meet the Paris Agreement targets.

https://news.un.org/en/story/2018/05/1009782

EXPERTS MEET TO SCALE UP EFFORTS TO TACKLE CLIMATE CHANGE DISPLACEMENT

15 May, 2018

Stakeholders from all over the world are gathered in Switzerland last week to contribute to the work of the United Nations Framework Convention on Climate Change (UNFCCC) Task Force on Displacement and to assist in drafting recommendations to avert, minimize and address displacement in the context of climate change.

https://reliefweb.int/report/world/experts-meet-scale-efforts-tackle-climate-change-displacement

FLOODS

CHOLERA EPIDEMIC FEARS IN SOMALIA, KENYA AS SEVERE FLOODING FORCES FAMILIES TO FLEE

30 Apr, 2018

A potentially lethal epidemic of cholera and other disease is set to sweep parts of Somalia and Kenya after severe flooding left hundreds of thousands homeless in the two countries, aid workers have warned. More than 100,000 people were forced to flee Beledweyne, a town in the Shebelle Valley 206 miles north of the capital Mogadishu, over the weekend, local officials said.

https://www.telegraph.co.uk/news/0/cholera-epidemic-fears-somalia-kenya-severe-flooding-forces/

HAWAII'S KAUAI PURSUES TOURISTS AS ISLAND RECOVERS FROM FLOODS

28 Apr, 2018

Honolulu: Residents and businesses are still cleaning up from flooding that deluged parts of Kauai, but community leaders are urging tourists to keep coming so residents don't suffer an economic calamity on top of record-breaking rains that smothered a normally green landscape in reddish-brown water.

https://www.reviewjournal.com/news/nation-and-world/hawaiis-kauai-pursues-tourists-as-island-recovers-from-floods/

N.B. SHUTS TRANS-CANADA HIGHWAY AS FLOODING HITS RECORD LEVELS

3 May, 2018

New Brunswick shuttered a large section of the Trans-Canada Highway Thursday night, and warned motorists to be on watch for wildlife seeking refuge, as floodwaters rose to record levels along the Saint John River.

https://www.ctvnews.ca/canada/n-b-shuts-trans-canada-highway-as-flooding-hits-record-levels-1.3913086

DROUGHT-BREAKING DOWNPOUR LEAVES HUNDREDS DEAD IN EAST AFRICA

4 May, 2018

Weeks of torrential rain after a long drought have turned from blessing to curse in East Africa, killing hundreds of people and displacing hundreds of thousands of others. In Kenya, which had suffered from three failed rainy seasons, 120 people have died in two months, including eight who were swept off a bridge in a flash flood.

https://www.dawn.com/news/1405587/drought-breaking-downpour-leaves-hundreds-dead-in-east-africa

RECORD FLOODS SHOW WORLD HAS CHANGED AND NEW BRUNSWICK MUST ADAPT, SCIENTISTS SAY

7 May, 2018

New Brunswick's record-breaking floods are a jarring reminder climate change is bringing a watery future that will wash away old patterns of life and force many to higher ground permanently, say environmental scientists and hydrologists. The hydrologist says the public needs to understand that historical levels of water flow are no longer guides to the future.

http://www.news1130.com/2018/05/07/record-floods-show-world-has-changed-and-n-b-must-adapt-scientists-say/

FLASH FLOODS IN AFGHANISTAN KILL AT LEAST 34

15 May, 2018

Flash floods caused by heavy rain in the past week have killed at least 34 people in several Afghan provinces and caused serious damage to property and livestock, officials said. The flooding, hitting provinces mainly in the north and center of the country, had caused serious damage to around 900 houses, killed hundreds of cattle and damaged agricultural land. The floods, which came after an unusually dry winter that has led to drought in many areas, underline Afghanistan's vulnerability to natural disasters.

https://www.yahoo.com/news/flash-floods-afghanistan-kill-least-34-111628596.html

'CATASTROPHIC' FLOODING IN GRAND FORKS, B.C.; DOZENS RESCUED

12 May, 2018

Firefighters rescued more than 30 people by boat, sometimes swimming through muddy and debris-laden water, by the time floodwaters had receded. Large parts of Grand Forks were underwater after three of the region's rivers — the Granby, Kettle and West Kettle — all broke 1948 water level records by about 60 centimetres, said Chris Marsh, emergency operations centre director for the Regional District of Kootenay Boundary.

http://vancouversun.com/news/local-news/bc-flooding-officials-fear-weekend-hot-weather-could-speed-up-melting-snowpack

FLASH FLOODS IN AFGHANISTAN KILL AT LEAST 34

15 May, 2018

Flash floods caused by heavy rain in the past week have killed at least 34 people in several Afghan provinces and caused serious damage to property and livestock, officials said. The flooding, hitting provinces mainly in the north and center of the country, had caused serious damage to around 900 houses, killed hundreds of cattle and damaged agricultural land. The floods, which came after an unusually dry winter that has led to drought in many areas, underline Afghanistan's vulnerability to natural disasters.

https://www.yahoo.com/news/flash-floods-afghanistan-kill-least-34-111628596.html

'CATASTROPHIC' FLOODING IN GRAND FORKS, B.C.; DOZENS RESCUED

12 May, 2018

Firefighters rescued more than 30 people by boat, sometimes swimming through muddy and debris-laden water, by the time floodwaters had receded. Large parts of Grand Forks were underwater after three of the region's rivers — the Granby, Kettle and West Kettle — all broke 1948 water level records by about 60 centimetres, said Chris Marsh, emergency operations centre director for the Regional District of Kootenay Boundary.

http://vancouversun.com/news/local-news/bc-flooding-officials-fear-weekend-hot-weather-could-speed-up-melting-snowpack

FLOODS CAUSED BY CLIMATE CHANGE COULD WRECK KRUGER NATIONAL PARK, SCIENTISTS WARN

17 May, 2018

Climate change could cause a catastrophic loss of species in South Africa's Kruger National Park (KNP). That is the warning from scientists whose work in the park as shown that some of the world's most sensitive and valuable river habitats are being destroyed by increasingly frequent extreme floods driven by cyclones.

https://www.timeslive.co.za/news/sci-tech/2018-05-17-floods-caused-by-climate-change-could-wreck-kruger-national-park-scientists-warn/

WATER RESOURCE

NASA TO LAUNCH TWIN SPACECRAFT TO STUDY EARTH'S CHANGING WATER CYCLES THIS MONTH

3 May, 2018

NASA will launch two twin spacecrafts to observe the continually changing and evolving water cycle on Earth using what it describes as an "innovative technique" to do so from space. The U.S. space agency reports in a news release that it will launch the Gravity Recovery and Climate Experiment Follow-On mission in conjunction with the German Research Centre for Geosciences no earlier than May 19.

http://www.mlive.com/news/us-world/index.ssf/2018/05/nasa_to_launch_twin_spacecraft.html

GRAPHENE: THE WONDER MATERIAL THAT COULD SOLVE THE WORLD'S WATER CRISIS

3 May, 2018

First developed by scientists at the University of

Manchester in 2004, graphene, an ultra-strong material composed of a single layer of carbon atoms, has been tipped for many uses from hair dye to super long-life batteries. But one of the material's most exciting properties may be its ability to filter out even the tiniest impurities in water.

https://www.telegraph.co.uk/news/2018/05/03/graphene-wonder-material-could-solve-worlds-water-crisis/

WATER SCARCITY IS CENTRAL TO THE NARRATIVE OF DEVELOPMENT: FAO

11 May, 2018

Rome: At the Food and Agriculture Organization (FAO) conference on the Near East, the talk is all about water scarcity, sustaining small family farms and creating "resilient" agriculture. But however noble the aims and urgent the need, the fact is that there is barely a corner of the Middle and Near East that is not beset by conflict.

http://www.arabnews.com/node/1300456/world

KENYA RECEIVES SH2.6BN WORLD BANK GRANT FOR WATER PROJECTS

9 May, 2018

Kenya has received $25.8 million (Sh2.6 billion) financing from World Bank's International Development Agency to boost water and irrigation projects. The grant to the Water and Sanitation ministry will address water security challenges in the country, through the Kenya Water Security and Climate Resilience Project (KWSCRP).

https://www.businessdailyafrica.com/economy/Kenya-receives-Sh2-6bn-World-Bank-grant-for-water-projects/3946234-4552878-2ud8ww/index.html

ZAMBIA CHARGES FOR GROUNDWATER USE AMID DROUGHT AND RAISED DEMAND

16 May, 2018

Faced with longer droughts and growing water demand, the Zambian government has introduced fees on groundwater use. Under a new executive order that came into effect in March, owners of domestic boreholes are, for the first time, required to pay a one-off fee of 250 kwacha ($25) to have their well licensed.

https://www.businesslive.co.za/bd/world/africa/2018-05-16-zambia-charges-for-groundwater-use-amid-drought-and-raised-demand/

WATER SHORTAGES TO BE KEY ENVIRONMENTAL CHALLENGE OF THE CENTURY, NASA WARNS

16 May, 2018

Water shortages are likely to be the key environmental challenge of this century, scientists from Nasa have warned, as new data has revealed a drying-out of swaths of the globe between the tropics and the high latitudes, with 19 hotspots where water depletion has been dramatic.

https://www.theguardian.com/environment/2018/may/16/water-shortages-to-be-key-environmental-challenge-of-the-century-nasa-warns

ZAMBIA CHARGES FOR GROUNDWATER USE AMID DROUGHT AND RAISED DEMAND

16 May, 2018

Faced with longer droughts and growing water demand, the Zambian government has introduced fees on groundwater use. Under a new executive order that came into effect in March, owners of domestic boreholes are, for the first time, required to pay a one-off fee of 250 kwacha ($25) to have their well licensed.

https://www.businesslive.co.za/bd/world/africa/2018-05-16-zambia-charges-for-groundwater-use-amid-drought-and-raised-demand/

WATER SHORTAGES TO BE KEY ENVIRONMENTAL CHALLENGE OF THE CENTURY, NASA WARNS

16 May, 2018

Water shortages are likely to be the key environmental challenge of this century, scientists from Nasa have warned, as new data has revealed a drying-out of swaths of the globe between the tropics and the high latitudes, with 19 hotspots where water depletion has been dramatic.

https://www.theguardian.com/environment/2018/may/16/water-shortages-to-be-key-environmental-challenge-of-the-century-nasa-warns

UNF: 'PIONEER' FEMALE ENGINEERING STUDENTS DESIGN WATER SYSTEM FOR GUATEMALAN VILLAGE

21 May, 2018

Collecting water for cooking, cleaning and other household chores used to be an all-day affair for the women of La Esperanza, a rural village of about 1,000 people in the highlands area of Guatemala. But the University of North Florida's first all-female senior civil engineering team designed a new water distribution system for the community, which was built by local workers and began operations this spring.

http://www.jacksonville.com/news/20180521/unf-pioneer-female-engineering-students-design-water-system-for-guatemalan-village

Events

Sr. No	Description	Dates	Country	websie
1	Hydropower Caspian & Central Asia	13 Feb - 14 Feb 2019	Tbilisi, Georgia	Email: events@ vostockcapital.com
2	NWHA Annual Conference	20 Feb - 22 Feb 2019	Portland, OR, USA	www.nwhydro.org/events-committ...
3	Hydropower & Stations	19 Mar - 20 Mar 2019	Tucson, AZ, USA	URL: www.ceati.com/ events/conferenc...
4	Water Power week in Wahsington	April1 to 3	Washington D.C	www.waterpowerweek. com/
5	2019 USSD Conference and Exhibition	8 April -11 April 2019	Chicago, Illinois -- Hilton Chicago Hotel	www.ussdams.wildapricot. org
6	ASDSO West Regional Conference	25 Mar - 27 Mar2019	Westminster, CO, USA	URL: damsafety.org/ training-center/...
7	WORKSHOP: Digitalization in Hydropower	25 Apr - 26 Apr2019	Graz, Austria	URL: www.vgb.org/en/ digitalization_...
8	World Hydropower Congress	14 May - 16 May2019	Paris, France	URL: congress. hydropower.org
9	ICOLD 2019 Annual Meeting	9 Jun - 14 Jun2019	Ottawa, Ontario, Canada	
	URL: www.icold-cigb2019.ca			
10	Dam Europe2019	27June-28June	London	/www.henrystewart conferences.com/ DAME urope2019/
11	HydroVision 2019	23 Jul - 25 Jul 2019	Portland, OR, USA	URL: www.hydroevent. com/future-even...
12	Dam Safety 2019	8 Sep - 12 Sep 2019	Orlando, FL, USA	URL: damsafety.org/ training-center/...
13	ICOLD 2019 - 11th ICOLD European Club Symposium	2 - 4 October 2019	Crete, Greece	http://www.globalevents.gr/
14	Canadian Dam Association Conference and Exhibition	6-10 October 2019	Calagary,Alberta Canda	www.Cda.ca
15	NZSOLD/ANCOLD 2019 Combined Conference	10-11 OCTOBER 2019	Australia	www.ancold.org.au
16	Dam Safety 2020	20 Sep - 24 Sep 2020	Palm Springs, CA, USA	URL: damsafety.org/ training-center/...
17	HydroVision 2020	14 Jul - 16 Jul 2020	Minneapolis, MN, USA	URL: www.hydroevent. com/future-even...
18	24th ICID International Congress + 71st IEC Meeting	22 Sep - 28 Sep2020	Sydney, NSW, Australia	www.icid2020.com.au
19	HydroVision 2022	12 Jul - 14 Jul2020	Denver, CO, USA	www.hydroevent.com/ future-even...

Aims & Scope

INCOLD Journal is a half yearly journal of Indian Committee on Large Dams (INCOLD) which is involved in dissemination of the latest technological development taking place in the field of dam engineering and its related activities all over the world to the Indian dam/hydropower professionals.

The aim of the journal is to encourage exchange of ideas and latest technological developments in the field among the dam engineering Professionals. The journal is for fully-reviewed qualitative articles on planning, design, construction and maintenance of reservoirs, dams and barrages and their foundations. The articles cover scientific aspects of the design, analysis and modelling of dams and associated structures including foundations and also provides information relating to latest know how in the field of construction technology for the related works. In addition to the information on the research work on the relevant subjects, the journal provides information on the related technical events in India and abroad such as conferences/ training programmes/ exhibitions etc. Information related to ICOLD (International Commission on Large Dams) activities such as ICOLD Congresses, its technical symposia, workshops, technical lectures, technical bulletins are also highlighted for the benefit of INCOLD members.

The original unpublished manuscripts that enhance the level of expertise and research in the various disciplines covered in the Journal are encouraged. The articles/technical papers are peer reviewed by editorial Board consisting of renowned experts before publication. The Journal has both print and online versions. There are no publication charges on the author.

V.K. Kanjlia
Secretary General
Indian Committee on Large Dams

Subscription Information 2018/ (2 issues)

Institutional subscription (Print & Online)	:	Rs. 900/US$75
Institutional subscription (Online only)	:	Rs. 600/US$50
Institutional subscription (Print only)	:	Rs. 600/US$50
Membership for 10 Years (Print only)	:	Rs. 5,000
Membership for 10 Years (Print & Online)	:	Rs. 8,000

Recent ICOLD Technical Bulletins

Bulletin 107 (1997) Concrete dams, control and treatment of cracks : Review and case histories.
Bulletin 108 (1997) Cost of temporary and permanent flood control in dams.
Bulletin 109 (1997) Dams less than 30 m high.
Bulletin 110 (1997) Impact of cost of rules, criteria, specifications and contractual issues.
Bulletin 111 (1998) Dam break flood analysis.
Bulletin 112 (1998) Neotectronics and dams.
Bulletin 113 (1999) Seismic observations of dams - Guidelines and case studies.
Bulletin 114 (1999) Embankment dams with bituminous concrete facing review and recommendations.
Bulletin 115 (1999) Dealing with reservoir sedimentation guidelines and case studies.
Bulletin 116 (1999) Dams and fishes - Review and recommendations.
Bulletin 117 (1997) The gravity dam : A dam for the future - Review and recommendations.
Bulletin 118 (2000) Automated dam monitoring systems - Guidelines case histories.
Bulletin 119 (2000) Rehabilitation of dams and appurtenant works - State-of-the-art and case histories.
Bulletin 120 (2000) Design features of dams to resist seismic ground motion.
Bulletin 121 (2000) Tailing dams risks of dangerous occurrences.
Bulletin 122 (2000) Computational procedures for dam engineering.
Bulletin 123 (2002) Seismic design and evaluation of structures appurtenant to dams.
Bulletin 124 (2002) Reservoir land slides : Investigation and management - Guidelines and case histories.
Bulletin 125 (2003) Dams and Floods - Guidelines and case histories.
Bulletin 126 (2003) Roller compacted concrete dams - State of the art and cast histories.
Bulletin 127 (2004) Remote sensing for reservoir water quality management - Examples of initiatives
Bulletin 128 (2004) Management of reservoir water quality - introduction and recommendations.
Bulletin 129 (2004) Dam Foundations - Geology considerations investigation Methods Treatment Monitoring.
Bulletin 130 (2004) Risk Assessment - In Dam Safety Management - A Reconnaissance of Benefits, Methods and Current Applications.
Bulletin 131 (2006) Role of Dams in Flood Mitigation - A Review
Bulletin 132 (2008) Bulletin 132 - Shared Rivers : Principles and Practices
Bulletin 133 (2008) Bulletin 133 - Embankment Dams on Permafrost
Bulletin 134 (2008) Weak Rocks and Shales in Dams
Bulletin 135 - Geomembrane Sealings Systems for Dams
Bulletin 136 The Specification and Quality Control of Concrete Dams
Bulletin 137 Reservoirs and Seismicity
Bulletin 138 General Approach to Dam Surveillance
Bulletin 139 Tailings Dams Safety
Bulletin 140 Sediment Transport and Deposition in Reservoirs
Bulletin 141 Concrete Rockfill Dams - Concepts for Design & Construction
Bulletin 142 Report on Safe Passage of Extreme Floods
Bulletin 143 Historical Review on Ancient Dams
Bulletin 144 (2011) - Cost savings in Dams
Bulletin 145 - The physical properties of hardened conventional concrete in dams
Bulletin 146 - Dams and Resettlement - Lessons learnt and recommendations
Bulletin 147 - Sedimentation and sustainable use of reservoirs and river systems
Bulletin 148 - Selecting seismic parameters for large dams - Guidelines (revision of Bulletin 72)
Bulletin 149 - Role of dams on the development and management of rivers basins.
Bulletin 150 - Cutoffs for dams
Bulletin 151 - Tropical residual soils as dam foundation and fill material
Bulletin 152 - Cost savings in specific dams
Bulletin 153 - Sustainable design and post-closure performance of tailings dams
Bulletin 154 - Dam safety management: Operational phase of the dam life cycle
Bulletin 155 - Guidelines for use of numerical models in dam engineering
Bulletin 156 - Integrated flood risk management
Bulletin 157 - Small dams: Design, Surveillance and Rehabilitation
Bulletin 158 - Dam surveillance guide
Bulletin 159 - Supplement to the Position Paper on Dams and the Environment
Bulletin 160 - ICOLD Dam Decommissioning - Guidelines
Bulletin 161 - Dam and Water Transfers – An Overview
Bulletin 162 - The Interaction of Hydraulic Processes of Reservoirs
Bulletin 163 - Dams for Hydroelectric Energy
Bulletin 164 - Internal Erosion of Existing Dams, Levees and Dikes and their Foundations
Bulletin 165 - Selection of Materials for Concrete in Dams
Bulletin 166 - Inspection of Dams following Earthquake – Guidelines, 2016 (Revision of Bulletin 062A)
Bulletin 167 - Regulation of Dam Safety: An Overview of Current Practice World Wide
Bulletin 168 - Recommendations for Operation, Maintenance and Rehabilitation
Bulletin 169 - Global Climate Change, Dams, Reservoirs and related Water Resources
Bulletin 170 - Flood Evaluation and Dam Safety

List of ICOLD Congress Proceedings

15th Congress - 1985 – Lausanne
16th Congress - 1988 – San Francisco, USA
17th Congress - 1991 – Vienna, Austria
18th Congress - 1994 – Durban, South Africa
19th Congress - 1997 – Italy
20th Congress - 2000 – Beijing, China
21st Congress - 2003 – Montreal, Canada
22nd Congress - 2006 – Barecelona, Spain
23rd Congress - 2009 – Brasalia
24th Congress - 2012 – Kyoto, Japan
25th Congress - 2015 – Stavanger, Norway
26th Congress - 2018 – Vienna, Austria

Purchase of ICOLD Technical Bulletins and Congress Proceedings, please contact INCOLD Secretariat.

V.K. Kanjlia
Secretary General
Indian Committee on Large Dams (INCOLD)
CBIP Building, Malcha Marg, Chanakyapuri,
New Delhi-110021, India
Phone : 91-11-26115984, Fax : 91-11-26116347
E-mail : sunil@cbip.org; cbip@cbip.org; Web : www.cbip.org

INCOLD MEMBERSHIP

Institutional Member - Annual Membership Fee - Rs. 20,000/-
- Life Membership Fee (10 Year) - Rs. 1,80,000/-

Individual Member - Annual Membership Fee - Rs. 1000/-
- Life Membership Fee (10 Year) - Rs. 9,000/-

For new membership, please contact the INCOLD Secretariat